River Bride (Blackpool) Certified Drainage Scheme

Blackpool Hydraulic Modelling Report

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Revision History

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Contract

This report describes work commissioned by John Kelly, on behalf of the Office of Public Works as part of the Lower Lee Flood Relief Scheme. The OPW project manager for the contact was Michael Collins. David Forde of JBA Consulting carried out this work.

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Purpose

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Acknowledgements

JBA would like to acknowledge the Blackpool Flood Committee and Cork City Council for their provision of historical flood information and invaluable local knowledge.

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Executive Summary

Arup and JBA Consulting were commissioned to develop the Lower Lee Flood Relief Scheme, including flood relief works for the village of Blackpool. This commission builds upon the findings of the Lee CFRAMS, and is in response to the frequent and severe fluvial flooding experienced in Blackpool.

This report presents the work carried out by JBA Consulting to develop a hydraulic model for the Blackpool catchment that is robust enough to provide accurate flood extent maps, as well as testing possible alleviation measures.

Flooding in Blackpool is not a recent problem. Anecdotal testimony provided by the Blackpool Flood Committee states that flood events have occurred intermittently since the mid-1970s. Instances of flooding from the Glen Stream have decreased in recent years, with flooding on the Bride increasing. This apparent decrease in flooding from the Glen may have been influenced by the construction of a culvert system on the Glen, upstream at Spring Lane. The largest flood event in recent memory occurred on the 28th of June 2012 when the River Bride inundated the centre of the village. Areas that were acutely affected were Great William O'Brien Street, the Watercourse Road, Orchard Court and Thomas Davis Street. Further upstream, the Commons Inn hotel, Dulux Paint Factory and the North Point Business Park were also inundated by flood waters. The primary cause of the event was an inability of the Bride channel to convey flow effectively; this was compounded by the restrictive hydraulic capacity of the culvert system in the village.

The hydrology for the project was derived from catchment descriptor data. The index median flood (Qmed) was multiplied by an adjustment factor and the study growth curve was derived from gauged sites in the Lee catchment. A summary of predicted flows is shown below:

Location		Predicted Peak Flows (m3/s)						
	Watercourse	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.1% AEP
Blackstone Bridge	Bride	2.55	3.58	4.29	5.24	5.90	6.69	10.17
North Point Business Park	Glenamought	7.42	10.39	12.47	15.22	17.15	19.45	29.54
Orchard Court	Bride	13.04	18.26	21.91	26.74	30.13	34.17	51.91
Glen Stream	Glen	4.58	6.42	7.70	9.39	10.59	12.00	18.24

The Lee CFRAMS hydraulic model for Blackpool was rebuilt to include a more suitable and detailed representation of the culvert system in the village and to account for structural changes in the last 7 years since the Lee CFRAMS was undertaken. In the intervening period, various trash screens have been removed and the conditions within the culvert system have deteriorated. Thus, the hydraulic model had to be changed to reflect these developments.

Two hydraulic models were constructed using the same 'base' or 'framework'. The primary model was the model used to derive the study flood extents and test proposed options. The other model was a variation that incorporated some channel and structure conditions present during the June 2012 event. The purpose of this latter model was to test its ability to replicate observed flow paths and hydraulic mechanisms. The following flow routes and mechanisms were confirmed as a result of this work:

- Flood waters overtopped the left bank of the Glenamought, just upstream of the North Point Business Park and flowed through the park. This is consistent with what was observed in the June 2012 event.
- Out-of bank flooding was modelled just upstream of the Topaz Garage on the Commons Road as a result of the low conveyance capacity of the existing Fitz's Boreen arch bridge; thus, increasing water levels upstream.

- This out-of bank flooding inundated the Commons Road and extended to the Sunbeam Industrial Estate. Again, this is confirmed when cross-referenced against anecdotal evidence for the June 2012 event.
- In Blackpool village, the modelled flooding also proved to be a good match with the observed flood extents. Flood waters overtopped the left bank of the Bride at Orchard Court and flowed overland through Wherland's Lane. It then continued into the village centre, along Thomas Davis Street, before ponding on the Watercourse Road adjacent to the church. The open railings beside the church allow the transfer of flow in both directions between the open channel and the Watercourse Road. As the modelled event continued, flood water continued overland, down the Watercourse Road towards Brewery Corner. It also ponded in Great William O'Brien Street, consistent with anecdotal and photographic evidence.
- There was no gauged data available in the Bride catchment in June 2012, therefore the actual flow rate experienced in Blackpool is unknown. However, the surrogate flow rate, equivalent to the 2% AEP event, was able to reproduce most of the observed flow routes and mechanisms.

The hydraulic model was calibrated using the findings of a specially commissioned flow monitoring contract that began in March 2014 and finished in February 2015. The objective of this contract was to investigate the flow and level patterns at the Madden's Buildings junction; where the Bride and Glen watercourses meet in the culverted system. Useful flow and level data was recorded which informed the construction of the hydraulic model. The monitoring period was exceptionally dry, however, with a maximum flow rate of 7.5m^3/s recorded downstream of Blackpool Church.

The calibrated hydraulic model was then capable of deriving the following information:

- The river channel at North Point Business Park, Dulux Paint Factory and Orchard Court is not of sufficient capacity to contain flood flows. The flooding witnessed in June 2012 reinforces these mechanisms identified in the modelling.
- A flow rate of approximately 18 m^3/s at Fitz's Boreen arch bridge causes overtopping of the right bank upstream of Topaz.
- The hydraulic model confirms the flood risk at the Dulux Paint Factory. The low-lying right-bank is particularly vulnerable, with flood waters able to pond as the factory premises is reasonably flat. The threshold for out-of-bank flow is between the 10% and 5% AEP events.
- The threshold for flooding at Blackpool Shopping Centre is the 1% AEP, with flooding observed at Heron Gate. Channel capacity here is approximately 29.5 m^3/s, before the left bank is overtopped.
- The culvert system at Orchard Court and Blackpool Church is hydraulically inefficient for large flood events i.e. in excess of 5% AEP. Multiple culvert sections and a potential for debris accumulation are a primary cause for concern.
- The threshold of flooding at Orchard Court is somewhere between 20% and 10% AEP events. The capacity of the channel at Orchard Court is approximately 19.7 m^3/s.

A number of flood alleviation options were tested to find the optimum solution to flooding in Blackpool. The hydraulics of the culverted sections of the lower sections of the River Bride control flood levels and are the main contributor to flood waters escaping from the channel and flooding properties. The modelling has identified that should all these flood waters be contained within defences, the culverts would become heavily surcharged and significantly elevate water levels.

The suite of alleviation options tested using the hydraulic model is as follows:

- Option 1 'Do Minimum' - Involved the incorporation of a maintenance and cleaning regime throughout the length of the Bride-Glen-Glenamought system. This was represented in the hydraulic model by reducing channel roughness values ('typical conditions').
- Option 2 'Upstream Storage' - Involved the inclusion of an upstream storage area in the townland of Ballincrokig, in the upstream reach of the Glenamought.
- Option 3 'Direct Defences' - This option investigated the feasibility of erecting direct defences (i.e. walls) at Orchard Court to prevent flooding of Blackpool village. The objective of this option was to contain all flood waters within channel and force it through the Blackpool culvert system.
- Option 4 'Culverting at Orchard Court' - This option examined the complete culverting of the Bride watercourse from just upstream of Orchard Court to meet the existing Orchard Court inlet.
- Option 5 'Replacement of Blackpool Church Culvert' - This examined the effectiveness of replacing the existing Blackpool Church culvert over its entire length (as far as the Madden's Buildings junction) with an increased section size.

Option 4 was chosen as the 'preferred option' for the following reasons:

- It removes the need for defences at Orchard Court, keeping all flow underground. This immediately cuts off a major flood route at its source.
- It removes a possible entry point for debris that has previously caused problems in the culverted system. Therefore, large obstructions should not find their way into the Madden's Buildings junction.
- It makes use of the existing culvert system downstream of Blackpool Church.
- There is some presurrised flow in the proposed culvert during the design event. However, the degree of pressurisation is not large enough to affect water levels upstream relative to other options tested, in the design case.

The preferred option was refined and optimised for robustness under the adopted flow sensitivity scenario. From the hydraulic model, it was demonstrated that an increase in flow could increase modelled water levels considerably. To help lower water levels in the system during these flow exceedance scenarios, a number of structures were amended or replaced to improve the robustness and reliability of the scheme. The goal of this optimisation was to ensure that modelled water levels did not exceed the proposed scheme freeboard of 600mm. This value was chosen in a response to the system's sensitivity to increased flows; ensuring that defence heights were kept within a manageable range. A sensitivity factor of 18% was applied to the 1% AEP event to test the optimised option. This 18% factor was derived from the uncertainty in the index flood (Qmed) and the study growth curve. The optimised preferred option proved to perform quite well when subjected to this flow rate. However, the testing identified the Blackpool Retail Park and Blackpool Shopping centre as quite sensitive to slight increases in flow. The culvert system downstream has the potential to control water levels locally and essentially create a 'reservoir' when enough flow exceeds the capacity of the culverts. Therefore, the fixed freeboard was increased to 1.35m along this local reach to cater for this uncertainty.

The optimised scheme for Blackpool includes the following elements:

- Realignment of the Madden's Buildings junction to facilitate an easier transition from the Phase 5 GBK culvert to the Phase 3 GBK culvert.
- Limiting the inflow to the Brewery Branch culvert in the design 1% AEP event to 9.5 m^3/s; preventing surcharging of the conduit.
- Realignment of the existing Blackpool Church culvert inlet to allow an easier transition downstream.
- Culverting of the existing open channel at Blackpool Church to keep pressurised flows in the system.
- Culverting of the open channel adjacent to Orchard Court from the existing inlet to the outlet of the N20 culvert.
- Replacement of the Short Sunbeam Culvert.
- Removal of the Long Sunbeam Culvert.
- Construction of a sedimentation area on the left bank at Sunbeam to manage sediment loads in the watercourse.
- Replacement of the Fitz's Boreen masonry bridge.
- Construction of a winter channel just downstream of the Commons Inn.
- Replacement of the North Point Business Park Culvert.
- Replacement of the Kilnap Glen House Access Bridge.
- Direct defences to be placed in Blackpool Retail Park, Dulux, Upstream of Topaz, Commons Inn and North Point Business Park.

During the course of the modelling work, it was found that the risk of flooding from the Glen Stream was low; despite some minor flooding occurring at Spring Lane during the June 2012 event. This was the case even during the design event (1% AEP) model runs. Therefore, upon careful consideration of this and examination of photos of the surveyed culvert inlet at that location, it has been concluded that the extremely poor condition of the culvert inlet and trash screen during the event was the probable cause of flooding. The channel and culvert capacity is sufficient for the quantity of flow to be conveyed during the design event. On this basis, further detailed work on the Glen Stream has not been pursued.

It is proposed that a roughing screen be installed upstream of the existing Spring Lane trashscreen. Consequently, it is proposed to remove the existing screen at Spring Lane. The existing wall on both banks of the Spring Lane channel will be formalised and repaired where necessary.

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Abbreviations

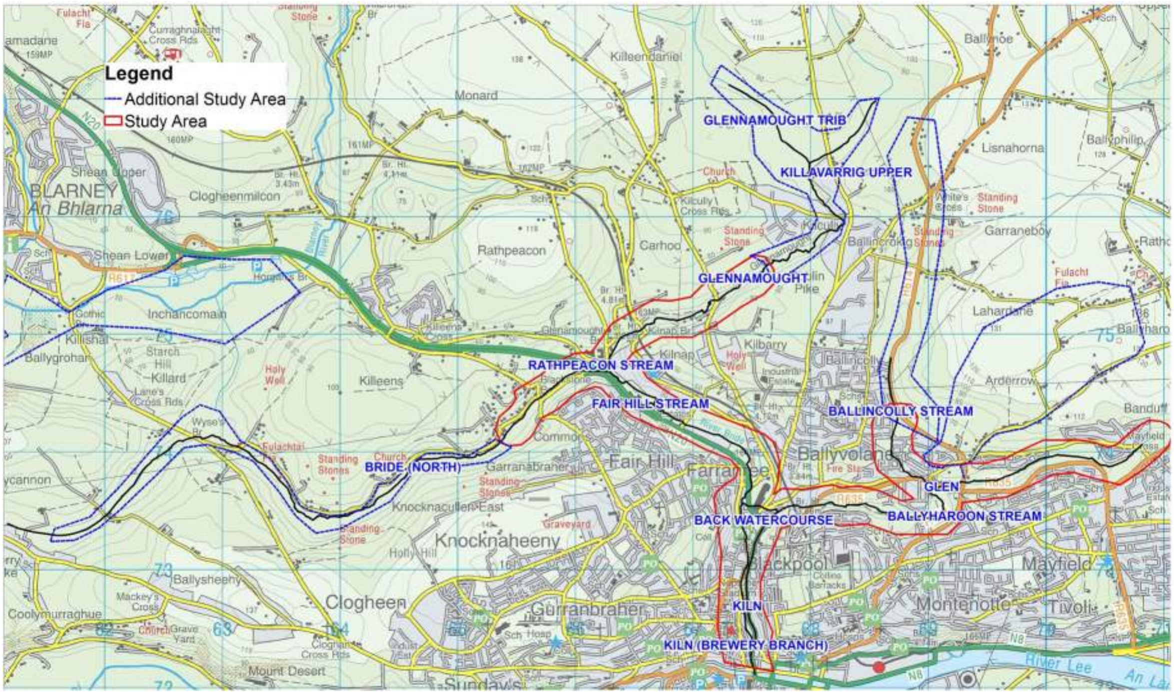
1D	One Dimensional (modelling)
2D	Two Dimensional (modelling)
AEP	Annual Exceedance Probability
CCTV	Closed Circuit Television
CFRAM	Catchment Flood Risk Assessment and Management
CFRAMS	Catchment-Based Flood Risk Assessment and Management Study
DTM	Digital Terrain Model
ESB	Electricity Supply Board
FSR	Flood Studies Report
FSU	Flood Studies Update
HEP	Hydrological Estimation Point
ID	Identifier
ISIS	Hydrology and hydraulic modelling software
LIDAR	Light Detection and Ranging
mOD	Meters above Ordnance Datum
OPW	Office of Public Works
Q100	Flow at the 100-year return period
TUFLOW	Two-dimensional Unsteady FLOW (a hydraulic model)
XS	Cross Section

1 Hydraulic Modelling

1.1 Bride (North) Study Area

This section summarises the hydraulic modelling that has been carried out to develop a suitable baseline model for the design and testing of a flood risk management scheme in Blackpool. The area covered by the Bride (North) Study is shown in Figure 1-1. The key flood risk location is Blackpool village itself, at the downstream extents of the Bride (North). The Commons Road region, just downstream of the Bride (North)/Glenamought confluence is another significant flood risk location.

Figure 1-1 Bride (North) Study Area

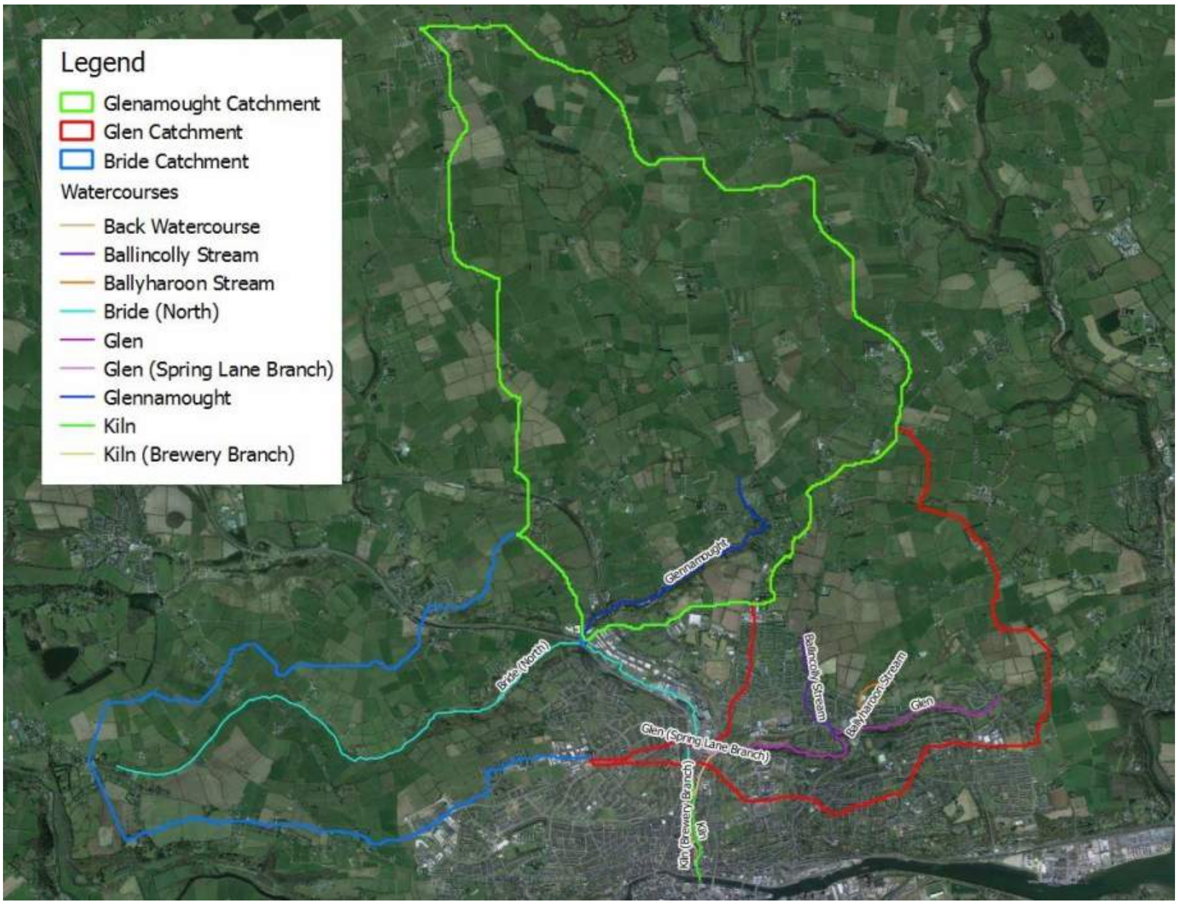


1.2 Catchment description

The Bride (North) study area encompasses three major watercourses: the Bride (North), the Glenamought and the Glen. The total catchment area upstream of Blackpool Village is 41.7 km^2. The Bride (North) rises in the townland of Ballycannon, near Healy's Bridge, before flowing in an easterly direction towards the city. The Glenamought River rises in Whitechurch and flows in a southerly direction before making an abrupt right-turn in the townland of Ballincrokig. The Bride (North) and the Glenamought meet each other in a culverted system at the North Point Business Park on the N20. The Glen River flows in a westerly direction from Mayfield, through the Glen River Park, before entering a culvert under Spring Lane. It then merges with the Bride (North) in a large culvert junction under Madden's Buildings, 100m downstream of Blackpool Church, before discharging to the River Lee at Christy Ring Bridge. The culverted system in Blackpool has been incrementally constructed since the early the 1980s as part of the Glen-Bride-Kiln (GBK) River Improvement Scheme which was commissioned by Cork Corporation in 1981. The topography of the entire catchment varies between 188mOD at Whitechurch and 8mOD in the Blackpool village centre. Figure 1-2 shows the contributing catchments draining to Blackpool village.

The catchment has been subject to a previous flood risk management study, the Lee CFRAMS. The study was a pilot project commissioned by the OPW to help understand the flood risk to Cork and propose initial management measures. This report builds on that work and provides the detail necessary to support scheme design and decision-making.

Figure 1-2 Overview of contributing hydrological catchments



The figures shown in the following pages highlight key study locations that are mentioned frequently throughout this report. Included in the figures are the locations of hydraulic modelling nodes, however they will be dealt with in more detail in Section 1.9.

Figure 1-3 Significant locations along Commons Road and at the Bride/Glenamought confluence

Figure 1-4 Significant locations in Blackpool village and at the Bride/Glen culvert system

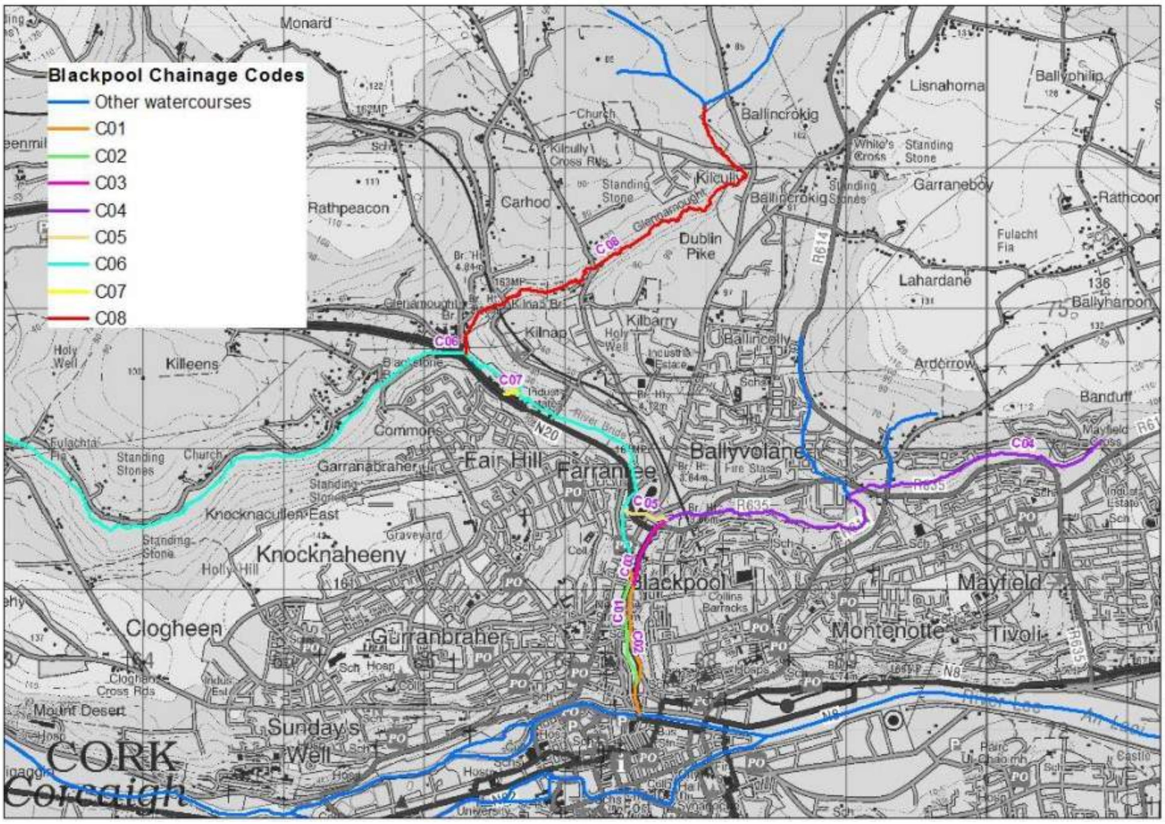
1.3 Watercourse referencing and terminology

The Lee CFRAMS project assigned referencing codes to the three major watercourses in the study area: the Bride (North), the Glenamought and the Glen. These references can be seen in Table 1-1. This is the same referencing system used in the original Lee CFRAMS hydraulic model. This study has retained the same system for the purposes of hydraulic modelling to allow for quick comparison. However, such a system is not best suited for the design of a flood relief scheme. For design and construction, the watercourses have been further sub-divided using the codes provided in Table 1-1. These codes can be combined with chainages to give any location along the study watercourses. '0' chainage starts at the downstream end of the respective watercourse. The location of each watercourse reach is shown in Figure 1-5.

Table 1-1 Comparison of Lee CFRAMS/hydraulic model and study watercourse references.

Watercourse Name	Lee CFRAMS/Hydraulic Model Reference	Lower Lee Flood Relief Scheme Reference
Kiln	7BR2	C01
Kiln (Brewery Branch)	7BREW (not included in original CFRAMS model)	C02
Back Watercourse	7BR2	C03
Glen	7BR2	C04
Glen (Spring Lane Branch)	Spring Lane	C05
Bride (North)	7BRI	C06
Fair Hill Stream	7BRI	C07
Glenamought	7BR1	C08

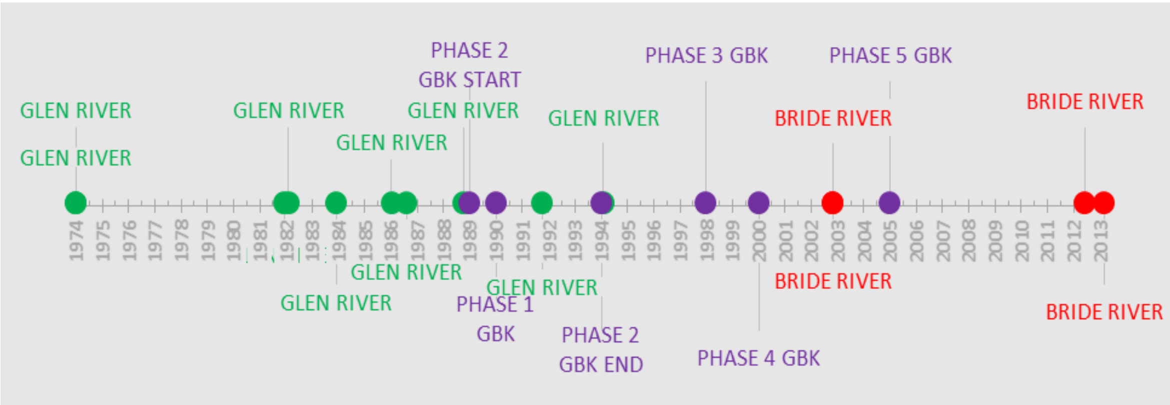
Figure 1-5 Chainage codes for Lower Lee Flood Relief Scheme



1.4 Flood history

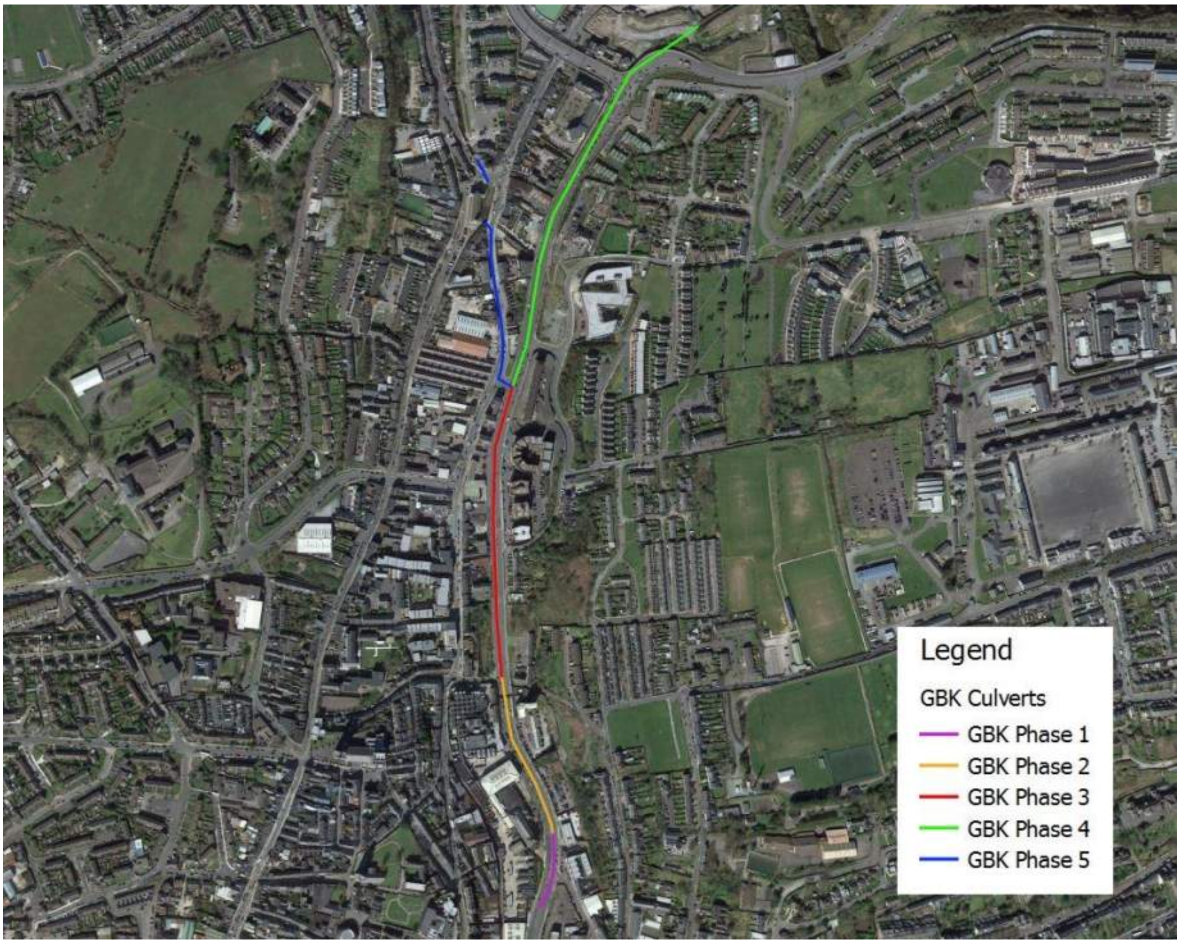
There has been a comprehensive history of flooding in the Blackpool area in recent years. Prior to the installation of the GBK culvert system in the early 2000s the primary source of flood risk came from the Glen River. However, in recent years the risk has perceived to have been transferred over to the Bride River. This could be coincidental, as well as perhaps an increase in development upstream and the addition of more, complex crossing structures. Figure 1-6 summarises the flood history and illustrates the increase in flood events from the Bride in recent years. The purple annotation denotes construction dates for various phases of the GBK scheme. It must be noted however, that examination of the OSI historic 25 inch mapping (which dates from the c1890s) shows that the lands adjacent to the Bride at the present-day Orchard Court and Commons Road are labelled 'liable to floods'. Therefore, it is safe to assume that there was a flood history associated with the Bride prior to contemporary flood events. Further details about a number of significant recent events are provided in the following sections.

Figure 1-6 Timeline and main contributory source of recent flood events in Blackpool



For information purposes, the different phases of the GBK River Improvement Scheme are shown in Figure 1-7.

Figure 1-7 Overview of GBK River Improvement Scheme



The information presented in the following sections has been extracted from a variety of Cork City Council and Office of Public Works post-event reports.

1.4.1 November 2002 event

This flood event occurred on the 21st November 2002[1]. It occurred as a result of flow exceeding the channel capacity of the River Bride at both Orchard Court and Blackpool Church. This led to flood depths of up to 1.0m in parts of Thomas Davis Street, the Watercourse Road and Great William O'Brien Street. A second flood event occurred on the 27th November 2002; the cause of which was identical to the earlier event. The old Blackpool Bridge culvert was also liable to blockage; as confirmed by post-event surveys. Therefore, as a result of these events the following remedial measures were recommended by Cork City Council:

- Installation of a new 4.8m x 2.1m culvert at Orchard Court to replace the older Blackpool Bridge section.
- Installation of a new 4.8m x 1.6m culvert at Blackpool Church to replace the older culvert section that extended downstream to Madden's Buildings junction.
- Installation of a collection of trash screens along Orchard Court. All of the above measures, together, formed the majority of the work required as part of Phase 5 of the GBK scheme.

1.4.2 June 2012 event

This event occurred in the early hours of Thursday, 28th June 2012[2]. The primary cause of flooding was the magnitude of flow in the River Bride, which exceeded the capacity of the culvert

[1] City Manager, Cork City Council (November 2002). Report on flooding at Blackpool.

[2] Engineer, Office of Public Works, South West Region Maintenance (10th July 2012). Report on flooding that occurred

system at Orchard Court and Blackpool Church and the channel capacity upstream at the Commons Inn. This may also have been exacerbated by debris accumulating at restrictive structures and junctions such as Fitz's Boreen arch bridge and Madden's Buildings junction. Thus, despite the adoption of remedial measures on the back of the 2002 event, the Orchard Court culvert system was still a major source of flooding. A peak flood depth of 1.2m was reported in Blackpool village. A summary of the locations affected by the 2012 flood event are presented below:

- North Point Business Park - Flooding occurred in the North Point Business Park as a result of flow exceeding the Glenamought River channel capacity downstream of the Kilnap viaduct. This was exacerbated by the restrictive nature of two hydraulic structures just upstream of the business park. Elevated waters escaped the channel on the left-bank and proceeded to enter the business park adjacent to the diner. The roundabout at the entrance was also subjected to flooding.
- Commons Road - The Commons Inn experienced flooding as a result of flow overwhelming the right-bank of the River Bride immediately upstream and flowing overland into the car-park. A collection of houses on the Commons Road experienced flooding of their properties; however the dwellings themselves were not affected. Flow also escaped over the right-bank, just upstream of the Topaz garage, and flooded the Commons Road. The poor hydraulic capacity of Fitz's Boreen arch-bridge is a major contributor to the flooding in this area. There was some flooding coming from the west due to a storage tank that overflowed on the other side of the Commons Road.
- Dulux Factory & Sunbeam Industrial Estate - The Dulux Factory and Sunbeam Industrial estate also experienced a substantial degree of flooding. Most of the damage caused was concentrated at the downstream extents of the site and particularly on the lower, right-bank. A combination of excessive flows and restrictive structures was the primary cause of flooding. However, the vegetated nature of the channel along this reach may also have been a contributing factor.
- Blackpool Shopping Centre - Blackpool Shopping Centre and Retail Park, itself, did not experience any flooding. However, the wetlands at the upstream extent of the site were inundated at the peak of the event. A trash screen at the inlet of the culvert at the main entrance to the shopping centre was also severely damaged as a result of the sheer force of water flowing through it and strikes from debris.
- Blackpool Village - Flooding in Blackpool village occurred as a result of flow exceeding the capacity of the open channel adjacent to Orchard Court, as well as the culvert system immediately downstream. Flood waters poured over the left-bank at Orchard Court before moving through Wherland's Lane. It then flowed down Thomas Davis Street where it collected in the natural depression adjacent to the pharmacy and the shop. Flow also escaped from the open channel adjacent to Blackpool Church. This resulted in flooding of Great William O'Brien Street, the Watercourse Road and the buildings supplies premises at Madden's Buildings junction. Flooding extended as far downstream as the Fever Hospital Steps and Brewery Corner.
- Dublin Street & Spring Lane - Some flooding occurred along Spring Lane and Dublin Street as a result of the Glen River overflowing its banks just downstream of the railway crossing. The primary source of flooding on Dublin Street was from pluvial runoff. It must be noted that the inlet to the Phase 4 GBK culvert at Spring Lane is poorly designed and is liable to debris accumulation, which is likely to have caused some of the flooding on Spring Lane.

1.4.3 March 2013 event

This flood event occurred on Thursday, 21st March 2013[3]. Flooding resulted from large flows in the River Bride, coupled with a debris load in the channel. Partial blockage of the trash screen at Orchard Court caused sufficient restriction in the capacity of the channel and culvert and elevated water levels, resulting in overtopping of the banks locally. As a result, substantial overflow was experienced in Blackpool village, as well as upstream at Commons Road and Killeens Road. The scale of flooding, however, was not as large as the event experienced in June 2012.

in Blackpool, Cork City, June 28th 2012.

[3] City Manager, Cork City Council (25th March 2013). Report on 21st March flooding in Blackpool.

1.5 Available data

1.5.1 Survey data

Cross-sectional survey was collected in May 2007 by Maltby Land Surveys Ltd. as part of the original Lee CFRAMS. Additional survey data was supplied by Murphy Surveys and delivered in October 2013.

The ID code assigned to each watercourse, as represented in the hydraulic model, is detailed in Table 1-2.

Table 1-2 Watercourse references in hydraulic model

Reference	Description
7BRI	Bride River
7BR1	Glenamought River
7BR2	Glen River
7BREW	Brewery Branch Culvert

The LIDAR data used in this study is based on the datasets collected for the Lee CFRAMS. Specifically, the DTM used is a combination of a 2m resolution grid of levels covering Cork Harbour (which includes the central island and shorelines of both North and South channels of the River Lee), which was commissioned by the Department of Communications, Marine and Natural Resources and a more extensive dataset commissioned by the Office of Public Works. Both LIDAR sets were flown in 2006.

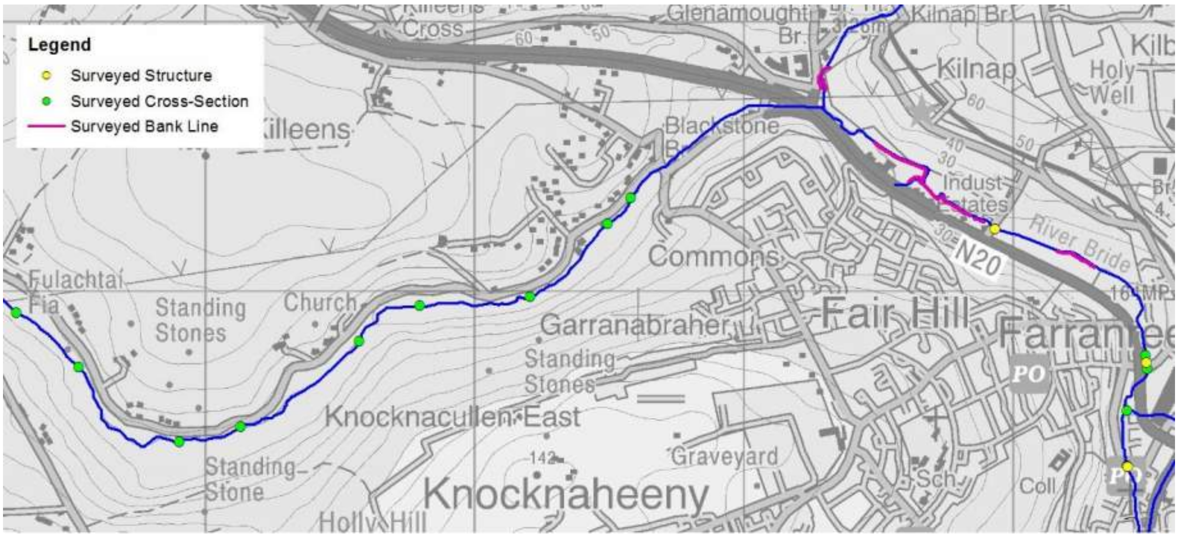
1.5.2 Infill Survey Data

Early in the contract period, there were some areas identified that would benefit from further survey. There were some deficiencies in the survey data collected during the Lee CFRAMS project and the infill survey hoped to address some of this. Murphy Surveys Ltd. were appointed to this contract and the survey work was completed in September 2013. Figure 1-8 shows where infill survey was completed for the Blackpool study area.

The cross-sections that were added to the model are as follows:

- 7BRI_6107
- 7BRI_4424
- 7BRI_3281
- 7BRI_3140
- 7BRI_332

Figure 1-8 Infill Survey locations and information collected



1.6 Existing model

The starting point for the Bride (North) hydraulic model was the final post-1D calibration version of Model 7 (Bride) that was produced for the Lee CFRAMS[4]. This model was a '1D-only' model. Flood extents were derived by applying a maximum water level across a DTM rather than by using a 2D modelling software package. It was deficient in a number of areas and following its review it was considered unfit and not robust enough for use in the design of a scheme.

1.7 Hydraulic modelling approach

The Bride (North) hydraulic model was developed in a number of stages:

- The supplied 1D (ISIS) model was divided into two separate modules: the Blackpool Village/Commons Road model and the "upstream" model (i.e. everything upstream of North Point Business Park and Spring Lane). The Blackpool Village model allowed for focused examination of flooding in the urban areas acutely affected by the June 2012 event.
- The Blackpool Village model was reviewed against construction drawings and supplied survey data.
- Adjustments were made to the 1D model to address errors found in the review.
- A 2D (TUFLOW) model of the Blackpool Village and Commons Road floodplain was created and linked to the 1D component of the model.
- The "upstream" model (i.e. everything upstream of the Commons Road and Spring Lane) was examined for any errors and corrected, but was retained as a 1D model.
- The reviewed "upstream" model was then linked back onto the Blackpool Village model to yield the final 1D-2D model for the catchment.
- The Glen Stream, in its entirety, was retained in 1D only i.e.it was not connected to the TUFLOW model of the floodplain.

1.8 Review of CFRAM 1D model

A general review of the CFRAM model was undertaken with respect to checking the configuration of all structures, coefficients used and investigating ways to improve model stability. It was found the supplied model became unstable when attempting to run it using default ISIS parameters. This is generally an indicator of poor overall model performance and deficiencies. It also failed to predict the scale of flooding seen in June 2012 or the mechanisms by which it occurs.

However, the process of creating the linked 1D-2D (ISIS-TUFLOW) model inevitably led to more detailed checking of local detail in many areas.

[4] Lee Catchment Flood Risk Assessment and Management Study (Lee CFRAMS). 2010. Halcrow Group Ireland Ltd.

For the review of hydraulic structures, each structure (bridge, culvert or weir) within the study area was checked for reasonableness (e.g. modelled headloss and key parameter values), as well as being compared against the original survey and observations made during site visits.

Some issues encountered in the review were:

- There were a large number of interpolates that did not appear to contribute significantly to the accuracy or stability of the model results.
- Orifice flow had not been enabled for surcharging bridges.
- The weir coefficients for in-line weirs were generally consistent at the default value of 1.7, however in some locations, a more conservative value should have been considered. This statement also holds true for some structure-overtopping spills.
- Roughness values applied to culverts and some river reaches were not in-keeping with the current, on-the-ground situation, and led to a misleading reduction in water levels at those locations.
- The Lee CFRAMS 1D (ISIS) model was tested with the corresponding CFRAM 1% AEP design flows for stability and performance, with the following outcome:
    - Several ISIS advanced parameters had been changed to run the supplied model (dflood = 100, psdeep = 3m). It is advisable where possible to avoid changing the default parameters since they have the potential to impact on the model results.
    - Automated Preismann Slots were activated for the model's river sections, allowing it to run at low flows. The default depth is 1m, but the depth used in the model was 3m. Most models do not require Preismann slots to run, so the review and update to the model aimed to remove or improve them, particularly for culvert sections.
    - The model was being run with a lower timestep (1s) than the minimum specified for the adaptive unsteady run (3s). At the higher timestep the model was very unstable at the beginning of the run, indicating a possible problem with the initial conditions.

The following issues were problems specific to the Blackpool Village/Commons Road segment only:

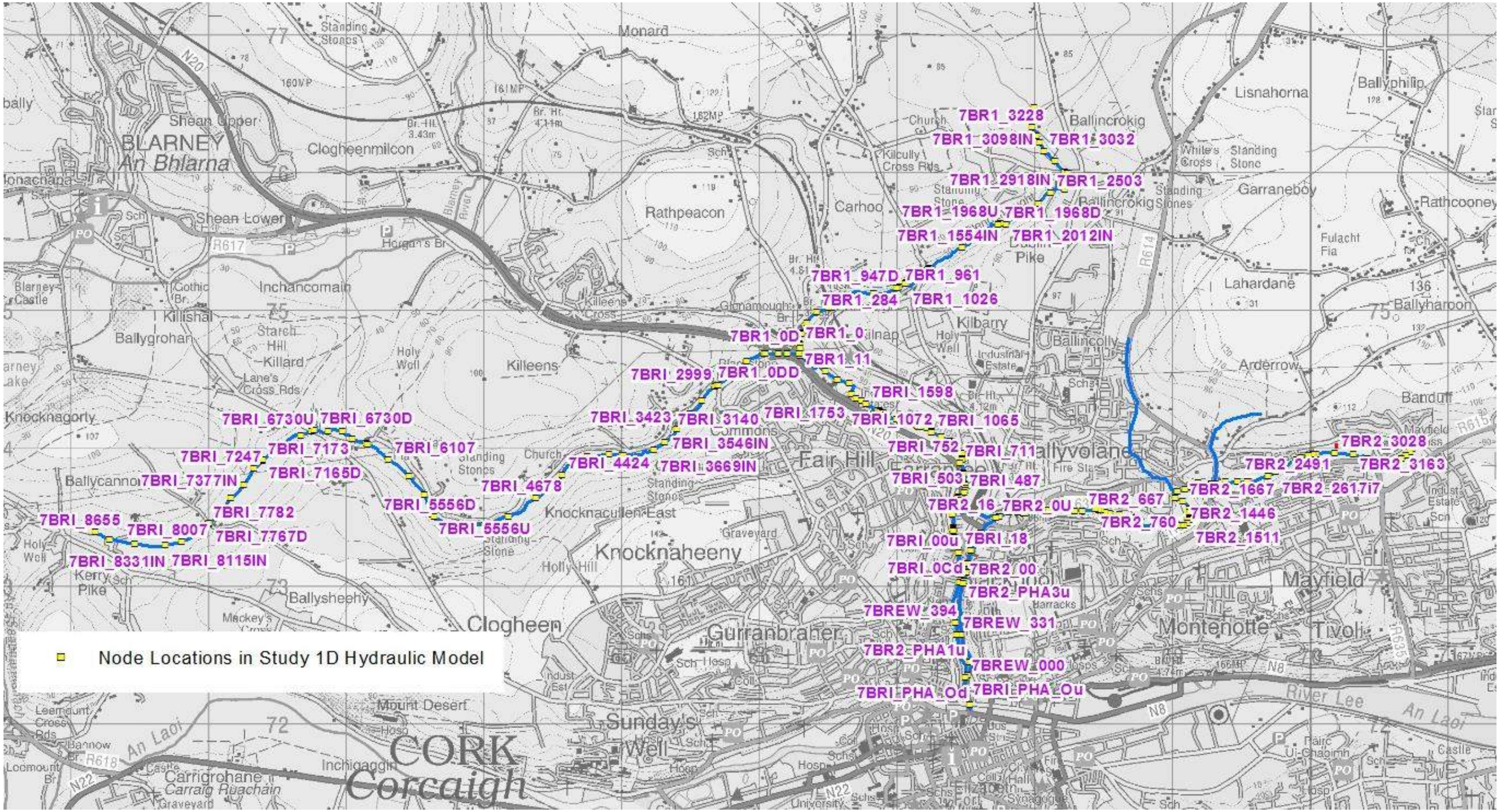
- In the original model the culvert system in Blackpool village had been significantly oversimplified. The 'Brewery Branch' culvert that extends from Madden's Buildings to Carroll's Quay had been omitted entirely from the model. As well as this, the culverts extending from Spring Lane (Phase 4) and Orchard Court (Phase 5)[5] were not accurately represented with respect to sizing or slope when compared to the 'as-built' scheme drawings.
- In the original model the downstream boundary was located at a point approximately 210m upstream of the actual outfall at Christy Ring Bridge.

1.9 Updating the 1D model

Following the 1D model review and prior to calibration, the supplied CFRAM model was updated to address the issues highlighted above as far as possible. The changes made are summarised in the following sections. A map showing the extent of the study 1D model, after the necessary changes had been made to the CFRAM version, is shown in Figure 1-9.

[5] Glen, Bride & Kiln (GBK) River Preliminary Report Review. 2003. E.G. Pettit & Company.

Figure 1-9 Study 1D Hydraulic Model Extent



1.9.1 Culvert system in Blackpool village

The 'Brewery Branch' culvert was added to the model using a combination of survey drawings created in the early 1980s as part of the GBK scheme design and a CCTV survey commissioned by Cork City Council in July 2012. The culverts constructed under Phase 3 (N20), 4 and 5 of the scheme were amended in the model to better reflect the GBK 'as-built' drawings in terms of dimensions and slope. Figure 1-10, Figure 1-11, Figure 1-12 and Figure 1-13 highlight the differences between each model after the changes had been applied.

Figure 1-10 Comparison of model geometry between original Lee CFRAMS and revised models - Commons Inn to Blackpool Shopping Centre

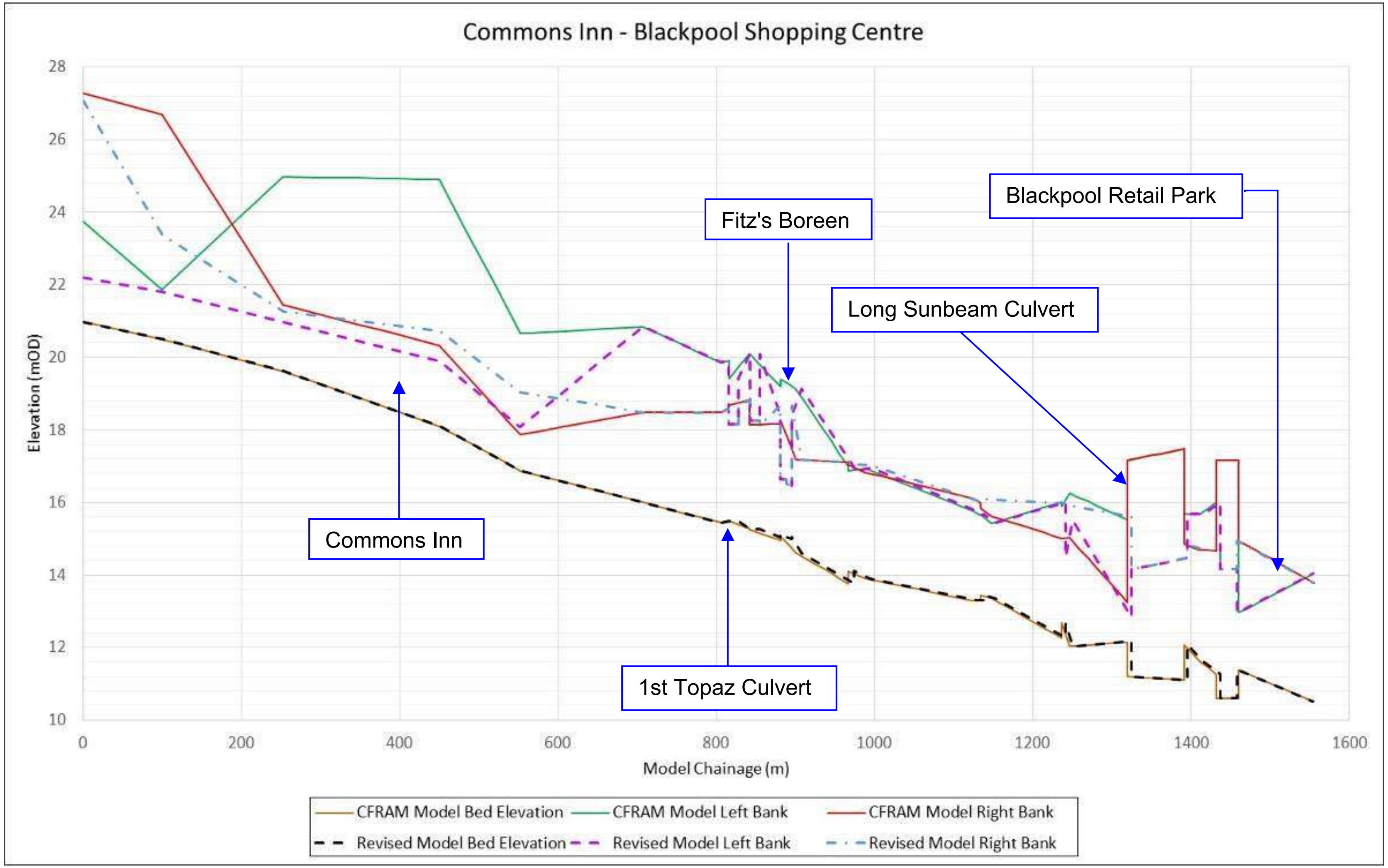


Figure 1-11 Comparison of model geometry between original Lee CFRAMS and revised models - Blackpool Shopping Centre to Madden's Buildings Junction

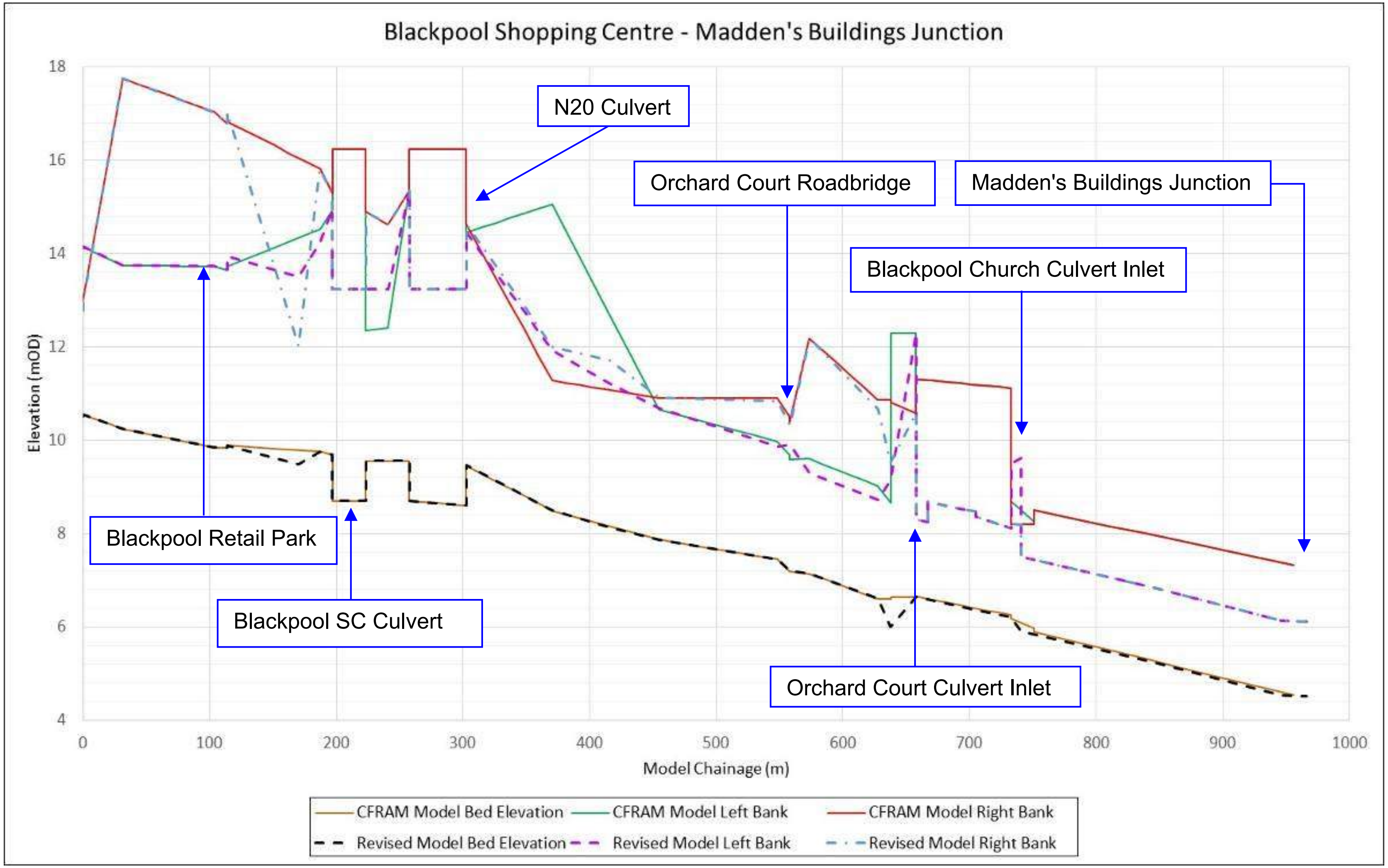


Figure 1-12 Comparison of model geometry between original Lee CFRAMS and revised models - Phase 4 GBK Culvert entrance to Madden's Buildings Junction

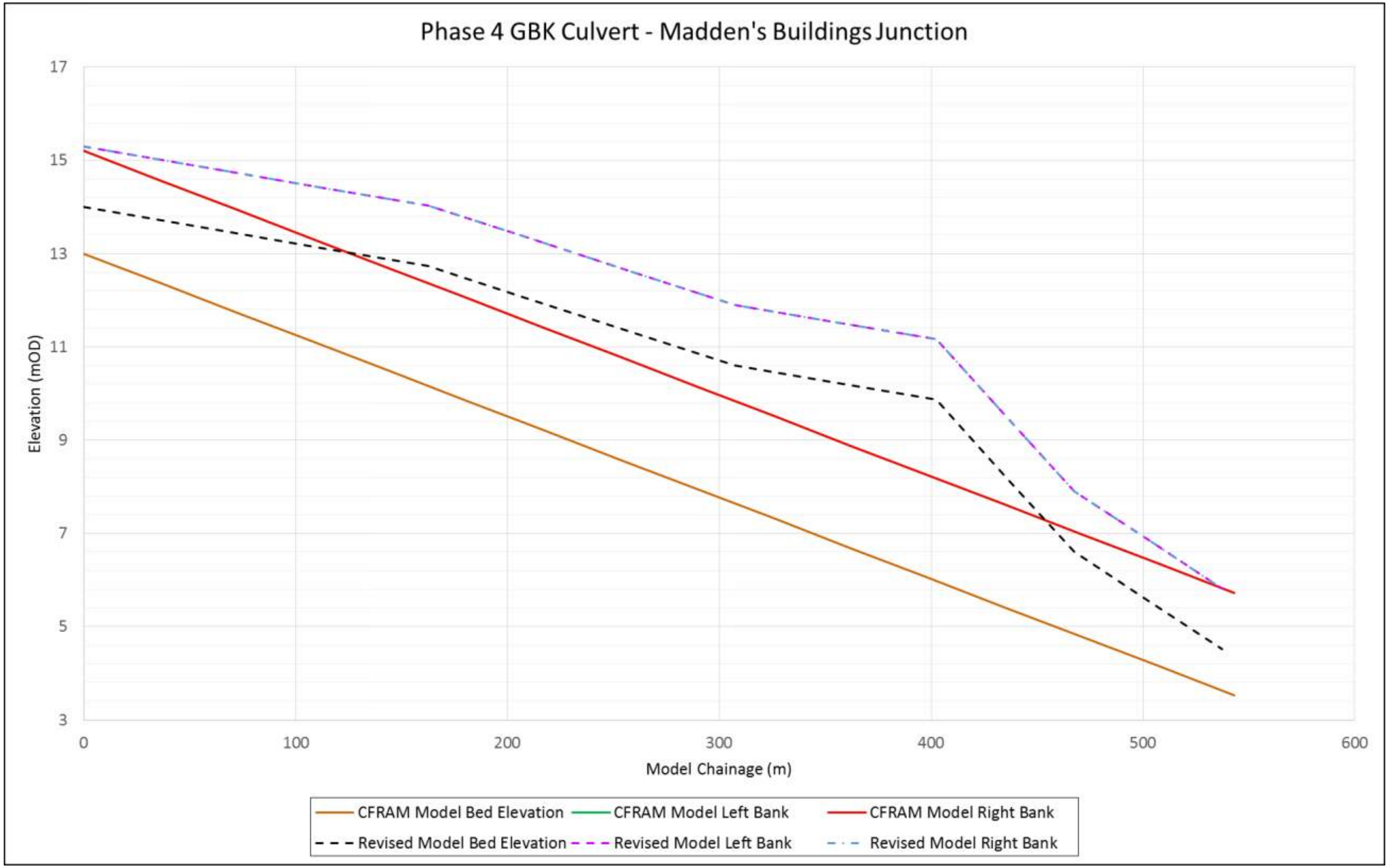
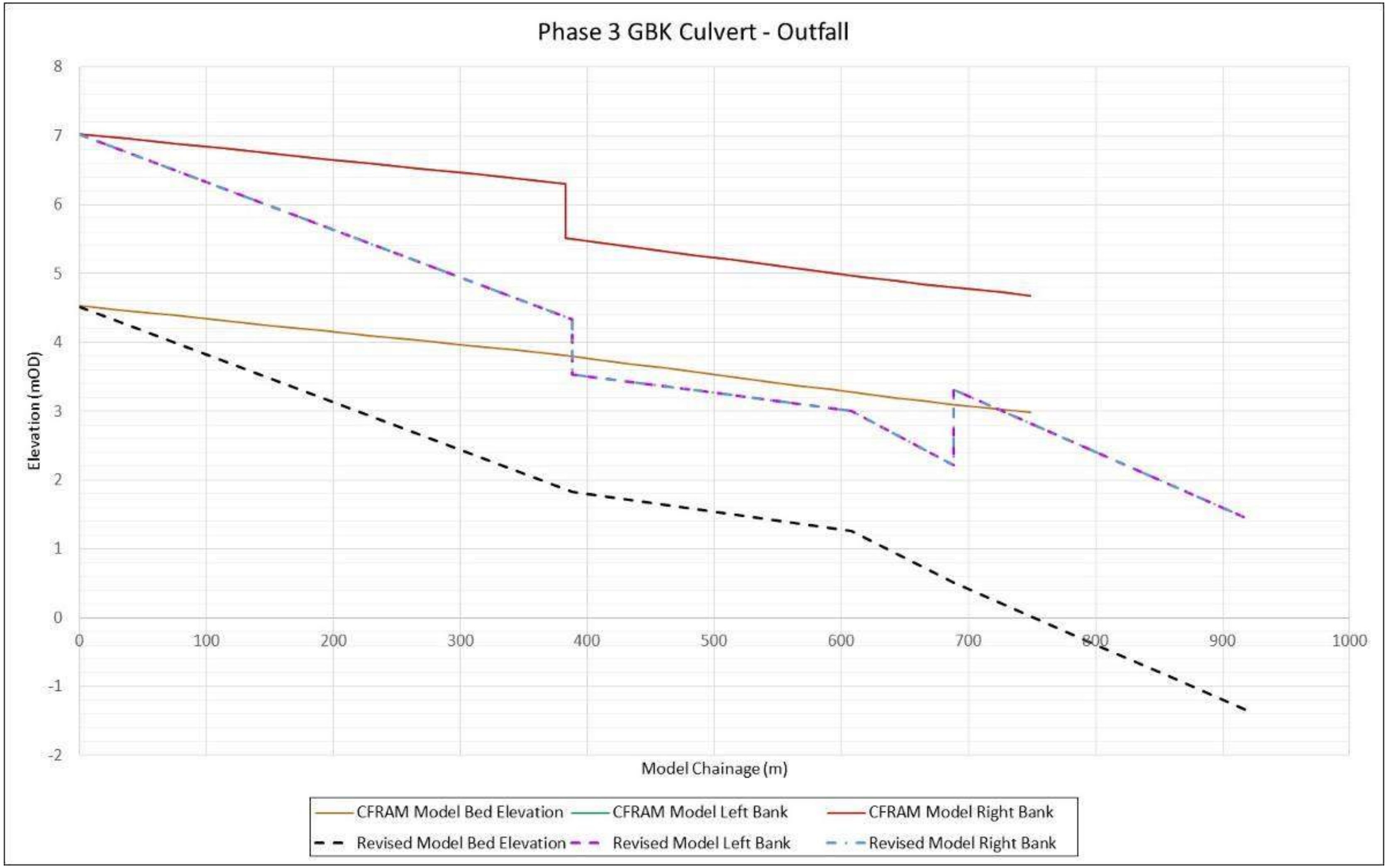


Figure 1-13 Comparison of model geometry between original Lee CFRAMS and revised models - Phase 3 GBK Culvert entrance to model outfall at Christy Ring Bridge



1.9.2 Downstream boundary

The original CFRAM study used a tidal (HT) boundary at the junction of the Brewery Branch and Phase 1 culverts to form the downstream extents of the model. However, for this study, the model was extended to the outfall at Christy Ring Bridge using construction drawings for the GBK scheme. It was also concluded that a tidal boundary was unnecessary as it did not influence water levels in Blackpool village but increased instabilities in the model. Indeed, upon testing of the revised model, it was found that the tidal influence only extended as far upstream as Brewery Corner, adjacent to the Fever Hospital Steps. Therefore a NORMAL DEPTH boundary was used for this study.

1.9.3 Channel obstructions and headlosses

A critical element of recent flooding in Blackpool has been the blockage of trash screens and openings along the Bride watercourse as it travels through the village. The accumulation of debris at the inlet of the Orchard Court culvert and in the Madden's Buildings junction has the potential to exacerbate the effect of high flows. The modelling of such blockages was not part of the Lee CFRAM scope but has been recognised as an important element in validating the scheme model against the June 2012 event. Therefore, various provisions have been made in the revised model to account for potential obstructions. A number of important headlosses due to channel bends and complex culvert inlets were also accounted for as part of the modelling work. The following summarises the work conducted:

- BLOCKAGE units were attached to the upstream faces of both Orchard Court bridges to simulate the effect of obstructed trash screens. A variety of blockage scenarios, including 0% (no blockage), can be modelled using these units.
- In an earlier iteration of the model, the ESB services tray that traversed the roof of the Orchard Court culvert, just inside the inlet, was represented using a pair of ORIFICE units. However, during the course of the model build this tray was removed from the culvert by the ESB. Therefore, in the final iteration of the model build, it has not been accounted for. It did prove critical, however, in attempting to recreate the flooding witnessed in June 2012.
- A GENERAL HEAD LOSS unit was attached to the upstream face of the Blackpool Church culvert inlet to simulate the losses that occur as a result of such a relatively sharp entrance.
- A CULVERT BEND unit was used inside the Blackpool Church culvert to simulate the second 30° turn that the conduit takes before running relatively straight to Madden's Buildings junction.
- The build-up of debris at the entrance to the Brewery Branch culvert was represented using a SPILL unit. This allowed the shape of the diversion block and blockage, as shown on the July 2012 CCTV survey (see Figure 1-14), to be input into the model and an appropriate spill coefficient to be applied. This unit can be altered as is required, depending on the current conditions at the culvert entrance.
- A significant accumulation of sediment was noted on the floor of the Orchard Court culvert which could act as a hydraulic control. Therefore, the roughness value in the culvert itself was increased to reflect this obstruction to flow.

It must be noted however, that structure blockage is not being accounted for in the design models. This is in keeping with CFRAM best-practise. However, in the validation of the model against a known event (June 2012), a blockage factor is used to replicate the head losses at these pinch points.

Figure 1-14 Accumulation of debris at the entrance to the Brewery Branch culvert

1.9.4 Removal of interpolates

In the original CFRAM model, interpolates were added to most reaches; the reason for which was unclear in many cases. In the revised model, any unnecessary interpolates were removed. Interpolates were only retained if they markedly contributed to model stability or accuracy of the results. The number of interpolates in the original Lee CFRAMS model was 638. In the revised model this number has been reduced to 105.

1.9.5 Model roughness

The channel and structure roughness values have been adjusted in some areas of the model to better reflect what is seen in reality. For example, the roughnesses of culverts have been changed from the default Colebrook-White value of 0.00015m (for a new, typical rectangular conduit) to values ranging from 0.03m to 0.3m, depending on the condition of the conduit floor. The higher 0.3m values represents a Manning's value of 0.031. This is analogous to a straight, natural, earthen, river channel; free from shoals, boulders and weeds[6]. Therefore, as the conditions within the Blackpool system is best described as a dynamic, inconsistent riverine environment rather than a typical closed, inert culvert system, the Colebrook White value of 0.3m is acceptable for application in some reaches. Figure 1-15 provides an example of such conditions. Despite cleaning works undertaken in June 2014, it is not unreasonable to assume that a level of sediment and debris has already begun to collect in the system; particularly as a result of the relatively high flows of the recent winter.

[6] Glen Bride & Kiln River - Preliminary Report Review. October 2003. Cork City Council

Figure 1-15 Conditions at the Orchard Court culvert outlet prior to the June 2014 cleaning works

In terms of channel roughness, a comprehensive examination of the photos taken as part of the 2006 river survey was conducted, along with a review of conditions noted during site walkovers, to determine the appropriate values for application in the model.

Further information on model roughnesses can be examined in the Blackpool Hydraulic Check File in Appendix D.

1.9.6 Spring Lane Abstraction

There is a culvert under Blackpool Shopping Centre that connects the Phase 4 GBK culvert with the River Bride. The culvert is reported to accommodate approximately 1-2m^3/s and forms part of the surface water management system of the shopping centre[7]. The head of the culvert is located just inside the Phase 4 culvert at Spring Lane and the outlet is a concrete manhole just upstream of Orchard Court. The flow route (Spring Lane Branch) has been included in the revised model as a point inflow and abstraction to account for the flow gain that occurs during flood events. It has been assumed in the hydraulic model that a flow rate of 1m^3/s can be conveyed through this connection for all model runs; based on the information provided by Cork City Council.

[7] Personal communication with Eamonn Walsh. Senior Engineer. Cork City Council.

Figure 1-16 Spring Lane Branch outfall, upstream of Orchard Court

1.10 Comparison of original CFRAM and revised 1D models

Table 1-3 compares and summarises the differences in the 1D node counts between the CFRAMS and revised models. The revised model ID is "Damages Model June 2015.DAT".

Table 1-3 Comparison of Lee CFRAMS and revised ISIS models

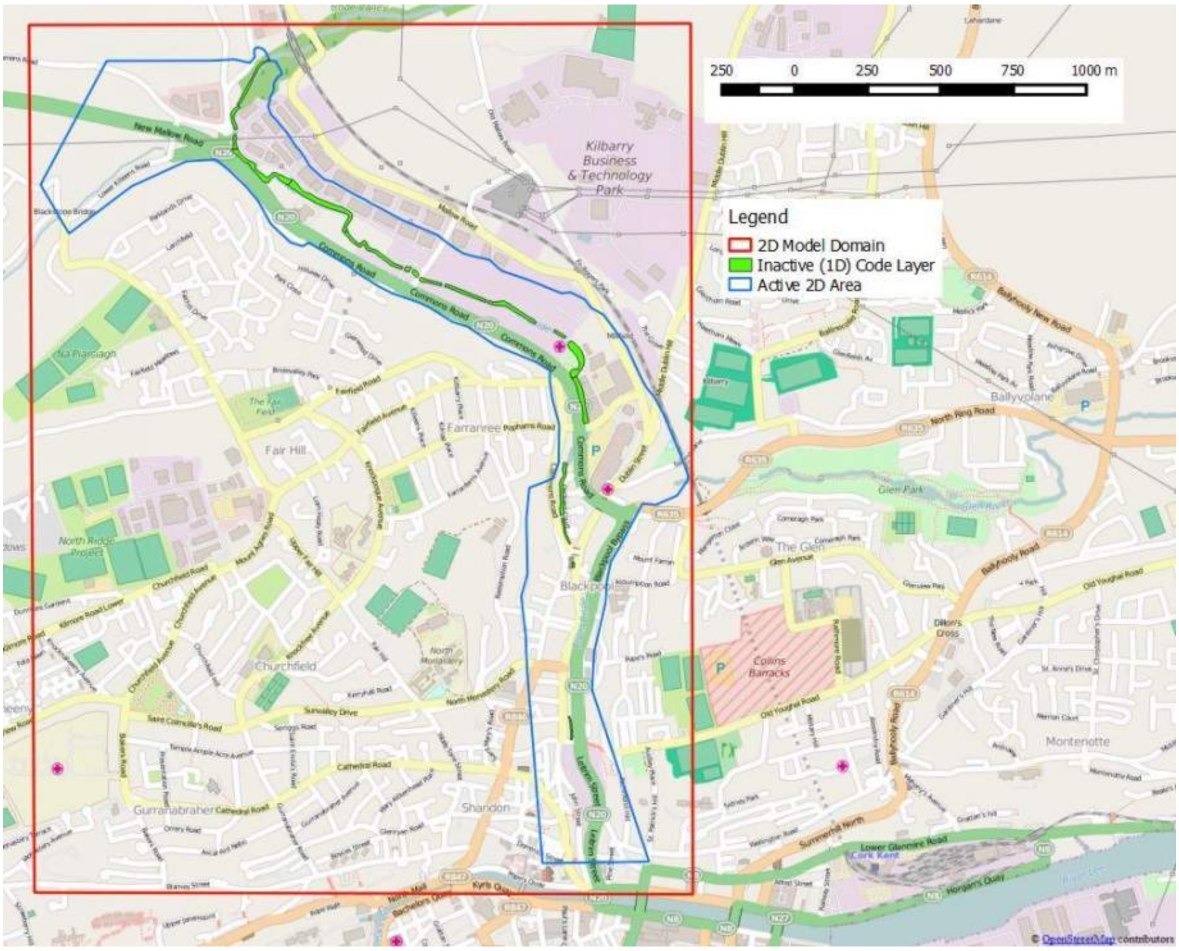
CFRAM model			Lower Lee FRS Revised Model		
Unit	Sub-Unit	Count	Unit	Sub-Unit	Count
ABSTRACTION	n/a	1	ABSTRACTION	n/a	
BRIDGE	ARCH	21	BRIDGE	ARCH	16
BRIDGE	USBPR1978	5	BRIDGE	USBPR1978	5
CONDUIT	CIRCULAR	2	CONDUIT	CIRCULAR	10
CONDUIT	FULL ARCH	0	CONDUIT	FULL ARCH	8
CONDUIT	RECTANUGLAR	2	CONDUIT	RECTANGULAR	10
CONDUIT	SPRUNG ARCH	0	CONDUIT	SPRUNG ARCH	25
CONDUIT	SYMMETRICAL	27	CONDUIT	SYMMETRICAL	45
CULVERT	INLET	8	CULVERT	INLET	19
CULVERT	OUTLET	0	CULVERT	OUTLET	21
INTERPOLATE	n/a	626	INTERPOLATE	n/a	105
JUNCTION	OPEN	88	JUNCTION	OPEN	104
LATERAL	n/a	4	LATERAL	n/a	10
LOSS	n/a	0	LOSS	n/a	1
ORIFICE	OPEN	9	ORIFICE	OPEN	7
REPLICATE	n/a	12	REPLICATE	n/a	
RESERVOIR	n/a	2	RESERVOIR	n/a	2
RIVER	SECTION	170	RIVER	SECTION	170
SLUICE	VERTICAL	3	SLUICE	VERTICAL	3
SPILL	n/a	53	SPILL	n/a	48
WEIR	n/a	0	WEIR	n/a	0
TOTAL NODES		1033	TOTAL NODES		578

Following these changes, it was found that periods of non-convergence were significantly reduced.

1.11 Floodplain modelling - 2D model

In the CFRAM model, the Glen, Bride and Glenamought floodplains were represented within the 1D ISIS domain. This is an appropriate set up where there is limited out of bank flow, and the flow routes are largely parallel to the channel. However, as witnessed in the more recent flood events, complex overland flow paths were established, with water overtopping the banks, flowing down roads and re-entering the channel further downstream. Such complex flows across a floodplain are much more accurately modelled using a 2D representation, such as is provided using the TUFLOW software. The active TUFLOW domain is shown in Figure 1-17.

Figure 1-17 2D model schematic



The Glen, however, has not been modelled in the 2D domain. For much of its reach, the Glen is conveyed through a steep valley; with a large elevation difference between channel and properties. The only known flooding witnessed from the Glen in the June 2012 event was as a result of water overtopping the GBK Phase 4 culvert inlet and sweeping through a gravelyard on Spring Lane and onto Dublin Street. However, it was found that during the course of modelling as part of this study, the same flow route could not be replicated without blocking the Phase 4 GBK culvert inlet. The size of the channel at this location seems to be sufficient to convey the 1% AEP flow, which means that flooding should only occur in this event if the inlet is blocked. Therefore, there is no need to link the Glen to the 2D domain but it is still included in the 1D model to ensure an accurate representation of flow and stage at the Madden's Buildings junction. The minor works proposed on the Glen are detailed in Section 7.7.1.

1.11.1 Key features of the 2D model

- A primary characteristic of flooding in Blackpool village in recent events has been the restriction in culvert capacity and the ability for flow to escape from the channel at Orchard Court and travel overland into the channel adjacent to Blackpool Church and the Watercourse Road. This is possible because of two open railing sections on the left bank of the channel, as shown in Figure 1-18. At the peak of events, water can spill onto the road, increasing water levels in nearby properties. However, it is important to note that these properties would be flooded regardless, even without contribution from flow spilling through the railings due to the overland flow route from Orchard Court. When the flood-waters are receding, the direction of flow is reversed as can be seen in Figure 1-19. This flow route has been accommodated in the revised model by connecting the Blackpool Church channel to the adjacent floodplain using lateral SPILL units, approximately 3m long. The length of the openings has been estimated based on existing survey data and visual assessment.

- The 2D roughness template is based on OSI NTF land use polygons. This provides a high definition dataset within urban areas due to the prevalence of roads and buildings in the NTF data. The 2D roughnesses allocated to the key land use categories were as follows:
    - General natural surfaces - 0.040
    - Buildings - 0.3
    - Roads, tracks and paths - 0.015

Verification of the 1D roughnesses chosen for modelling was done using the June 2012 validation event and localised changes were made where necessary. The 2D roughnesses were unaltered as they are standard values; as defined in the CFRAM modelling approach.

Increased roughness values were used to represent buildings in the 2D domain, rather than increasing LIDAR locally under building footprints (i.e. 'stubby building approach'). The reasoning for this was that many building thresholds in Blackpool are flush to the footpath and roads outside. A detailed threshold survey would be required to accurately adopt the 'stubby building approach'.

Figure 1-18 Example of the open railing sections in the left-bank, masonry wall adjacent to Blackpool Church

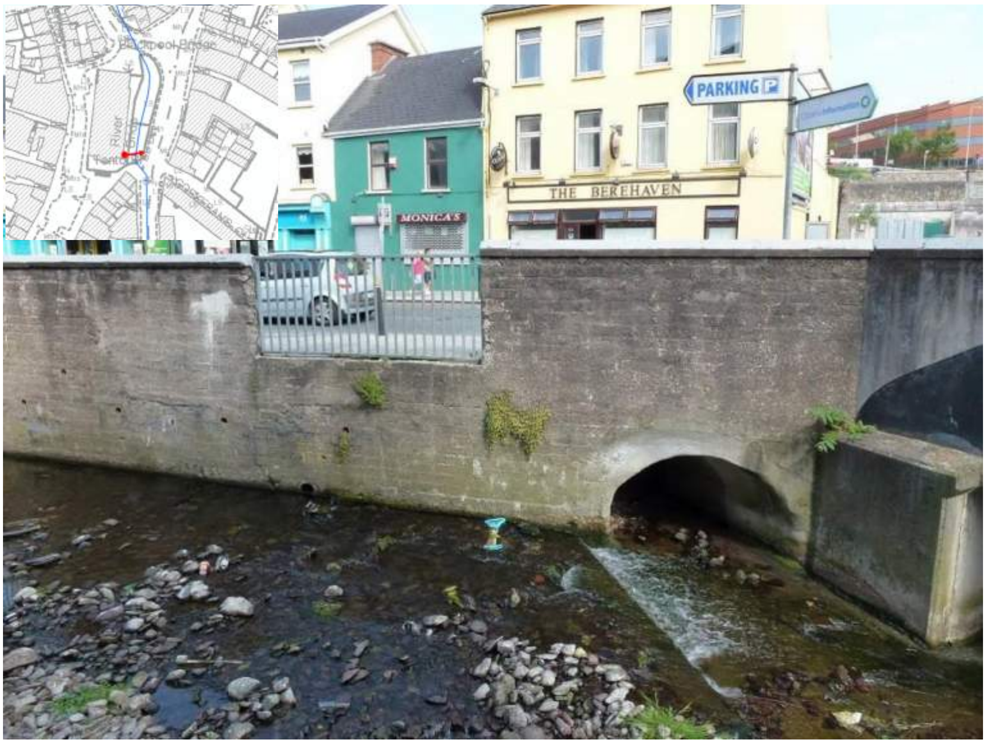


Figure 1-19 Flood-water travelling through the openings during the June 2012 event

2 Application of hydrology

2.1 Calculation of Inflows

At the commencement of this study in the summer of 2013, four hydrometric gauges were installed by the Office of Public Works at hydrologically significant locations in the catchment, as highlighted in Figure 2-1. Whilst, their record length (2 years) was not long enough to input into the extreme event analysis, it was hoped the gauges would help in 'framing' various hydrological estimates at the lower end of the flow scale. These gauges are described in further detail in Section 2.6.

Figure 2-1 Location of hydrometric gauges within entire Bride catchment, installed in July 2013

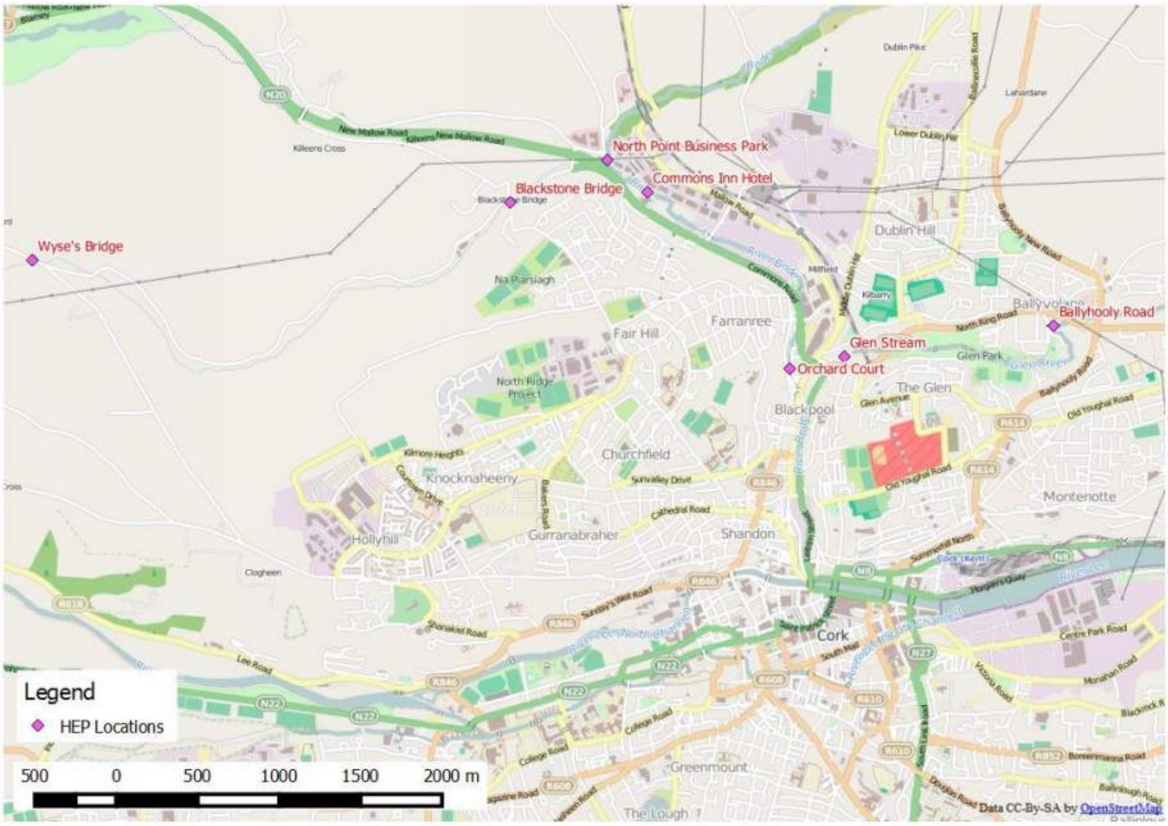
In the absence of gauges with a long period of record, an ungauged analysis was carried out for the Bride catchment. This involved the following:

- The calculation of design flows for the Bride catchment using ungauged catchment techniques using the Flood Studies Update (FSU).
- Further refinement of these estimated flows using a catchment adjustment factor calculated from analysis of all the catchments in the Lower Lee catchment.
- A study growth curve was developed using gauged sites within the larger Lee catchment. This growth curve was then used to determine the flow rates for each respective return period.

2.1.1 Hydrological Estimation Points

The Bride rises close to Healy's Bridge and flows in an easterly direction. After Blackstone Bridge it is joined by the Glenamought Stream before flowing down through Blackpool where it enters a culvert at Orchard Court and is joined with the Glen Stream downstream at Madden Building's. The Bride is culverted from here until it discharges into the river Lee underneath the Christy Ring Bridge. Figure 2-2 shows the Hydrological Estimations Points (HEP). For each of these it is necessary to calculate design flows to apply to the model.

Figure 2-2 Northern Bride HEP Points



2.1.2 Calculation of Qmed

Qmed is defined as the flood that is expected to occur or be exceeded, on average, every other year. In statistical terms, the flood is said to occur or be exceeded on average once every two years and have a 50% probability of annual exceedance.

The Flood Studies Update (FSU) method to estimate Qmed as described in research reports produced from FSU work packages 2.2 and 2.3 has been used. Qmed can be estimated using a regression equation based on seven different physical catchment descriptors, in conjunction with an urban adjustment, developed in FSU work package 2.3.

The multivariate regression equation was developed on the basis of data from 199 gauged catchments, linking QMED to a set of catchment descriptors.

$$QMED_{rural}=1.237x10^{-5}AREA^{0.937}BFIsoils^{-0.922}SAAR^{1.306}FARL^{2.21}DRAIND^{0.341}S1085^{0.185}(1+ARTDRAIN2)^{0.408}$$

Where: AREA is the catchment area (km^2).
BFIsoils is the base flow index derived from soils data
SAAR is long-term mean annual rainfall amount in mm
FARL is the flood attenuation by reservoir and lake
DRAIND is the drainage density
S1085 is the slope of the main channel between 10% and 85% of its length measured from the catchment outlet (m/km).
ARTDRAIN2 is the percentage of the catchment river network included in the Drainage Schemes

The FSU catchment descriptors provided for this catchment are incorrect as they assume that the river is flowing in the opposite direction. Essentially, they assume the river flows from Blackpool shopping centre to the River Shournagh at Healy's Bride. Therefore, the descriptors had to be amended. Catchment descriptors that would be affected by switching the direction of flow, such as S1085, URBEXT and AREA were recalculated for input into the FSU regression equation. Descriptors such as BFI, SAAR and DRAIND would be less sensitive to change as they are averaged over the entire catchment, and thus, were derived from existing nodes. Qmed was calculated using the FSU regression equation. The results of the FSU estimation can be seen in Table 2-1.

Table 2-1 Qmed Estimation from FSU

Watercourse	HEP Location	Qmed (m^3/s)
Bride	Wyse's Bridge	1.11
Bride	Blackstone Bridge	2.55
Glenamought	North Point Business Park	7.42
Glen	Ballyhooly Road	2.01
Glen	Glen Stream	4.58
Bride	Commons Inn Hotel	10.17
Bride	Orchard Court	13.04

2.1.3 Calculation of catchment adjustment factor

It is possible to improve on the initial estimate of Qmed by refining it using the process of data transfer, in which a representative gauged catchment with suitable quality data is identified and an adjustment factor for Qmed calculated as the ratio of the gauged to the ungauged estimate of Qmed at the gauging station. This factor is then used to adjust the initial estimate of Qmed at an ungauged site or gauging site with poor data records, under the assumption that the factorial error in the Qmed regression model is similar for two catchments. In the terminology of the FSU research reports, the gauging station where the adjustment factor is calculated is referred to as a donor site. In Blackpool there is no gauge either upstream or downstream of the HEP points (of reasonable record length) so an assessment of all the gauges in the Lower Lee catchment was carried out. Full details are supplied in Appendix A of the Lower Lee Hydrology Report.

Table 2-2 shows the results from the different Qmed estimation techniques for the gauged catchments in the Lower Lee catchment. The gauges have been classified according to catchment type.

Table 2-2 Summary of Qmed in Gauged Catchments

	FSU	FSR RR Winter	FSR RR Summer	Lee Cfram	Single Site Average	Adjustment Factor (Single Site/FSU)
Lake Influence						
Lee Dromcarra	81.228	76.96	80.08	80.22	81.51	1.00
Karst Influence						
Ovens	21.701	39.57	40.65	29.5	26.63	1.23
Tower	33.819	56.99	58.5	70.2	70.14	2.07
Excluded Catchments						
Macroom (Sullane)	80.225	76.96	102.17	141.7	148	1.84
Standard Catchment						
Kill (Laney)	30.061	44.36	45.99		50.17	1.67
Dripsey	20.137	35.48	36.49		40.96	2.03
Healy's Bridge	40.848	66.33	68.26	70.5	62.64	1.53
						1.75

(a) Lee Dromcarra which is influenced by Lough Allua

Lee Dromcarra is influenced by Lough Allua and this has the effect of lowering the adjustment factor as the recorded Qmed is lower due to the effect of the lake attenuating the flows.

(b) Catchments that are potentially influenced by Karst geology

When an annual maximum series plot of the recorded data at Ovens is analysed it was found that the karst influence attenuates the peak. At a certain point, the groundwater influence is overcome and its flow values rise rapidly in more extreme events. This is different to the expected normal distribution of an annual maximum series in Ireland.

Tower gauge is also affected by a karst influence. At present, discrepancies exist between Tower gauge and Healy's Bridge gauge, with Tower, a subcatchment of Healy's Bridge registering higher flow for the same event at Tower than Healy's Bridge. Healy's bridge has been calibrated using a recorded flow for the November 2009 event. There are a number of issues with the Tower gauge including its location upstream of the bridge with the effects of the bridge difficult to model and a lack of high flow gaugings. At the location of the Tower gauge there are large floodplains that, once inundated, lead to a small rise in levels but a large rise in flows resulting in a rating that is very sensitive to small changes in level. As a result of the confidence in the Healy's Bridge gauge data it has been included in the analysis and Tower has been excluded.

(c) Excluded Catchments

A large degree of uncertainty remains at Macroom and has been excluded from the analysis. The limited data record, change in gauge location, the exclusion of the River Laney that joins the Sullane just upstream of the Macroom gauge (used in the development of the rating curve for Macroom) and a lack of flow gaugings has led to its exclusion from the analysis.

(d) Standard tributaries

At the remaining catchments (Kill, Dripsey and Healy's Bridge), a Qmed adjustment factor was found to average 1.75 as shown in Table 2-2. The three remaining stations were then weighted according to their record length to give a weighted catchment adjustment factor of 1.71 as shown in Table 2-3. Though gauging stations have been installed in the Blackpool catchment, the hydrological analysis has treated it as an ungauged catchment due to the current record length at the gauges (~ 2years). The adjustment factor of 1.71 has been applied to all Qmed estimations (based on the FSU regression equation) in the Blackpool catchment. Full details of the individual statistical techniques and the calculation of Qmed and a Qmed adjustment factor are included in Appendix A of the Lower Lee Hydrology Report.

Table 2-3 Weighted adjustment factor

Weighted Average				
	Years of Data	Weight	Adjustment Factor	
Kill (Laney)	24	0.34	1.67	0.57
Dripsey	19	0.27	2.03	0.55
Healy's Bridge	27	0.39	1.53	0.59
	70	Catchment Adj. Factor		1.71

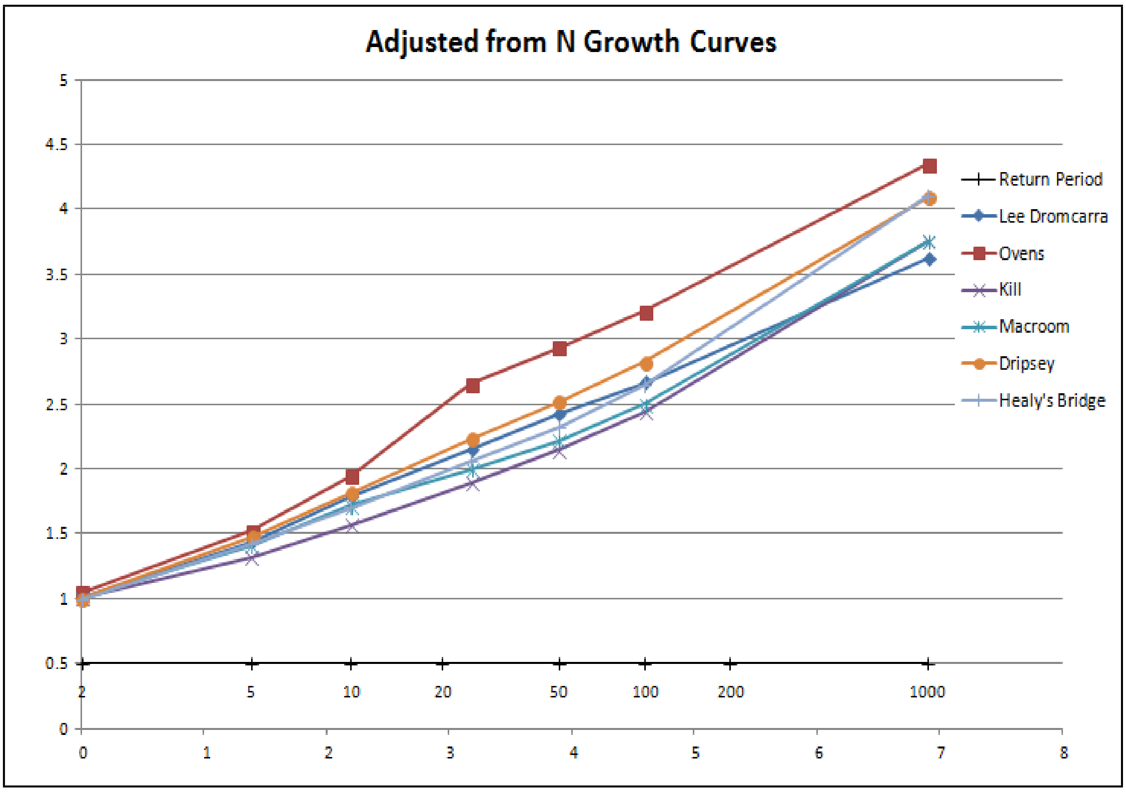
2.1.4 Calculation of Qmed using gauged estimates at Blackpool Shopping Centre

There is limited scope with only two years of data to confirm the Qmed values, however gauged data has been used in order to validate the current hydrological estimates. A POT analysis based on two years of data and using the rating curve developed from the hydraulic model has been completed. This yielded a Qmed of 13.26m^3/s which is equivalent to 1.73 times the ungauged Qmed Estimate from the FSU regression equation. 1.73 is very close to the Catchment Qmed Adjustment factor (1.71) calculated in the study and leads to confidence in our hydrological calculations. However, it should be noted that POT techniques are generally recommended as applicable to record lengths of over 7 years

2.1.5 Calculation of the catchment flood frequency curve

A catchment flood frequency curve was calculated from the gauged catchments in the wider Lee catchment and this was then applied at Blackpool. For each gauged catchment a frequency curve was developed. It consisted of applying single site analysis up to N years of record and apply FSR rainfall runoff curves above the record length. This produced a composite curve for each gauged catchment and is outlined in Figure 2-3. Full details of their development are included in Appendix A of the Lower Lee Hydrology Report.

Figure 2-3 Gauged Site Growth Curves



At all gauged sites, except Ovens the flood frequency plotted similar results as shown in Figure 2-3. The catchment frequency curve was calculated by averaging the individual curves as shown in red in Table 2-4. Macroom has once again been removed from the analysis. This catchment adjustment curve was applied in Blackpool. This was a similar approach to that used on the original Lower Lee study, except the datasets have been updated.

Table 2-4 Catchment Flood Frequency Curve

Return Period (% AEP)	Lee Dromcarra	Healy's Bridge	Kill	Dripsey	Average
50%	1.00	1.00	1.00	1.00	**1.00**
20%	1.44	1.42	1.32	1.47	**1.40**
10%	1.78	1.70	1.56	1.81	**1.68**
4%	2.15	2.06	1.89	2.23	**2.05**
2%	2.42	2.32	2.14	2.51	**2.31**
1%	2.66	2.64	2.43	2.82	**2.62**
0.1%	3.62	4.11	3.75	4.09	**3.98**

2.2 Design flows in Blackpool

Design Flows were calculated for the Blackpool (Northern) Bride by multiplying the Qmed value calculated using the FSU regression equation for each HEP point by the catchment adjustment factor of 1.71 and the catchment flood growth factors for each return period and are shown in Table 2-5.

Table 2-5 Design Flows For Northern Bride

Watercourse	HEP Location	50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.1% AEP
Bride	Wyse's Bridge	1.11	1.56	1.87	2.28	2.57	2.91	4.42
Bride	Blackstone Bridge	2.55	3.58	4.29	5.24	5.90	6.69	10.17
Glenamought	North Point Business Park	7.42	10.39	12.47	15.22	17.15	19.45	29.54
Glen	Ballyhooly Road	2.01	2.82	3.38	4.13	4.65	5.28	8.02
Glen	Glen Stream	4.58	6.42	7.70	9.39	10.59	12.00	18.24
Bride	Commons Inn Hotel	10.17	14.23	17.08	20.84	23.49	26.64	40.47
Bride	Orchard Court	13.04	18.26	21.91	26.74	30.13	34.17	51.91

Whilst there is only a limited record at the OPW gauging station at Blackpool Shopping Centre, the available level data for the winter of 2013/2014 has been converted to flow using the stage discharge relationship from the model. This indicates that regular peak flows of around $10m^3/s$ were measured during the recent storms. This gives some confidence in the calculation of Qmed at Orchard Court, compared to much lower values in previous studies. Similarly when the 50year event is simulated in the hydraulic model a good match is achieved with previous flooding patterns and flood volumes witnessed. This gives a reasonable confidence that the flow estimates are reasonable and appropriate for design.

2.3 Flow Sensitivity Analysis

The degree of uncertainty on the flow analysis has been investigated in order to carry out sensitivity testing on the hydraulic model to inform an appropriate freeboard. This flow sensitivity is made up of two components Qmed uncertainty and growth curve uncertainty and is outlined below.

2.3.1 Qmed Uncertainty

The FSU Work Package WP 2.2 "Frequency Analysis" states the Standard Error (SE) of a gauged site is $(0.36/\sqrt{N})$*Qmed. Based on a gauge record of 40 years, it gives an SE of 1.06 and on the Regression equation (N=1), it gives a SE of 1.36.

For the calculation of Qmed catchment adjustment factor we have used 3 gauged sites, with each of the SE shown below in Table 2-6 and gives an average SE of 0.075 in the calculation of the Qmed adjustment factor. 7.5% was applied for uncertainty in Qmed.

Table 2-6 Qmed uncertainty at gauged sites

Site	Length of Record	SE
Healy's Bridge	27	0.069
Kill	19	0.0825
Dripsey	24	0.073
	Average SE	0.075

2.3.2 Growth Curve Uncertainty

FSU report finds a SE of between 4.6% and 10.6% for ungauged stations using pooling analysis and an SE of between 8% and 15% for single site analysis (based on 85 stations with an average record length of 37 years). In this study we have used 4 donor catchments to calculate the catchment growth curve and in total 92 years of record. Therefore, one would expect the SE to lie in between the SE found for single site analysis and that for pooling group analysis. Taking an average of the two SE bands gives an uncertainty of 9.55%. This was applied as the SE in the growth curve.

2.3.3 Sensitivity Flow

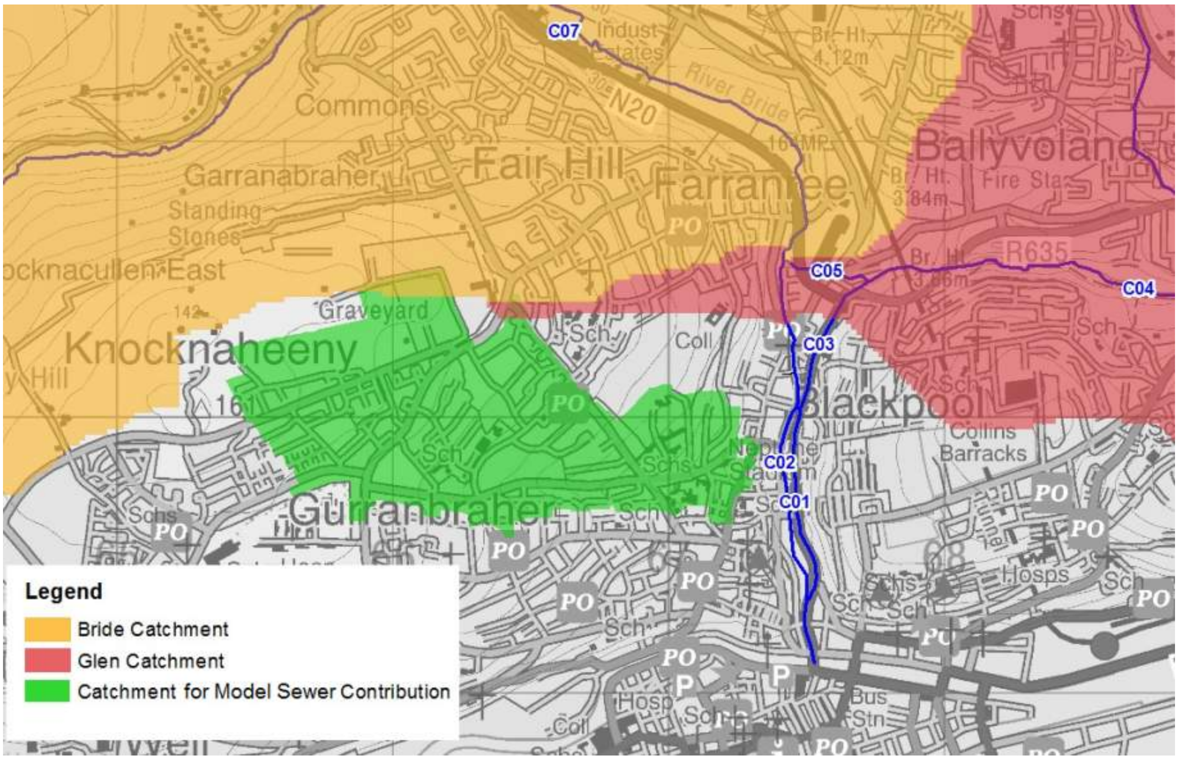
Combining the 7.5% and 9.55% gives a factor of 1.177, so an 18% increase was applied for sensitivity flow testing.

2.4 Surface water drainage and sewer contributions

In addition to the fluvial flows entering the Blackpool culvert system, one must also account for the surface water systems which bring water from neighbouring drainage catchments and discharge directly into the GBK conduits. The area draining directly into the culvert system, the green polygon shown in Figure 2-4, immediately downstream of the Madden's Buildings junction is outside the Bride catchment and is not included in the catchment descriptors used in the hydrological calculations. Therefore, an additional inflow had to be calculated for input into the hydraulic model. As part of the hydrological methodology for this project, the rainfall intensity at each timestep was calculated for a particular return period event and then input into the Modified Rational Equation. This yielded a flow at each timestep that could be applied over the Brewery Branch reach. The flows were applied using the same FSR Rainfall Runoff shape used for the fluvial flows detailed in Section 2.2. The flow was split into 7 equal contributions and applied over the full length of the Brewery Branch culvert. The flows input into the model for each return period can be examined in the Hydraulic Check File in Appendix D. To give an idea of scale, the 1% AEP inflow for the green catchment was calculated to be 5.53m^3/s. It must also be noted that surface water contributions draining to the Glen watercourse are included in the hydrology for the Glen.

The Modified Rational equation is the industry-standard for the calculation of drainage inflows and has a reputation of conservatism. It represents the peak of the overland flow contribution from these sewered areas. However, it does not account for the attenuating effect in the sewer network and surface ponding. As the inflow points into the model are located a considerable distance downstream of the primary risk area, it has been deemed that further sensitivity testing to possibly reduce the magnitude of flow is not merited. However, an arbitrary increase of 20% will be applied to test the hydraulic model's sensitivity to an even larger sewer contribution. This is explained further in Section 7.8.3.

Figure 2-4 Catchment used to provide additional surface water contribution to culvert system

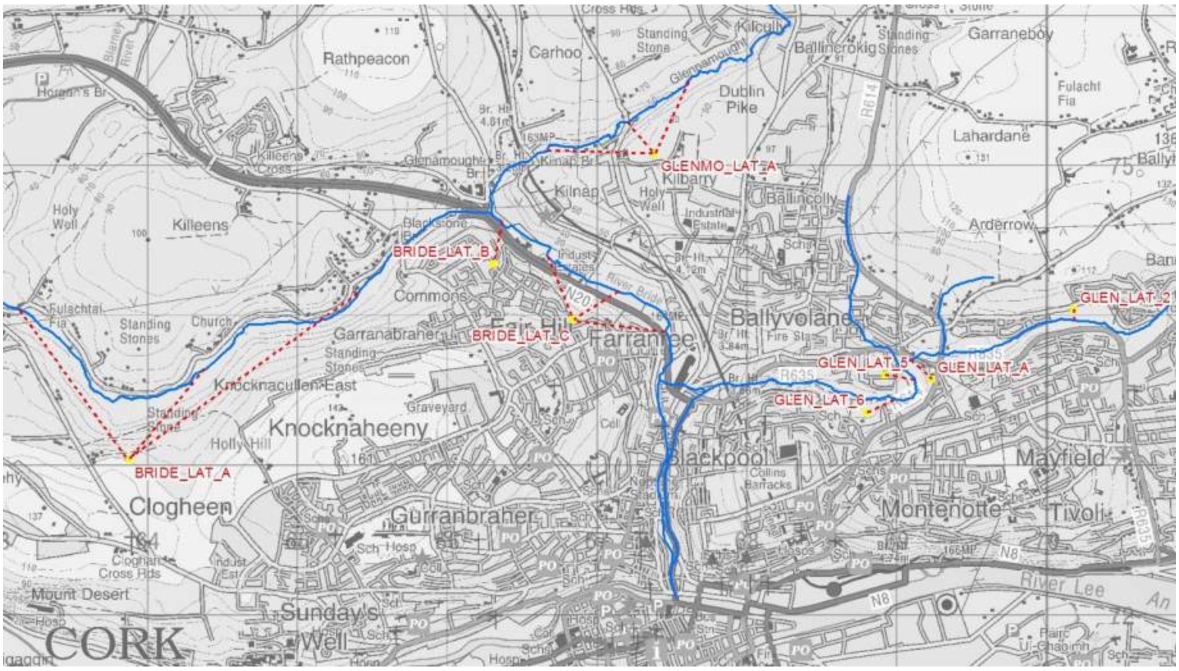


2.5 Application of lateral flow inputs in the model

Laterals were employed to balance flows between different hydrological estimation points. As the majority of the hydraulic model is 1D-only, a lateral with a peak equal to the difference in successive HEPs was applied over each reach. This allowed the correct flow to occur at each HEP without worrying about flow being lost to a 2D domain. A schematic of the lateral inflow locations is shown in Figure 2-5. Laterals were applied to the following reaches:

- Between Wyse's Bridge and Blackstone Bridge
- Between the junction of the Bride and the Glenamought and the Commons Inn
- Between the Commons Inn and Orchard Court
- Between Ballyhooly Road and Spring Lane
- Along the Glenamought River from XS 7BR1_1267L and the North Point Business Park

Figure 2-5 Schematic of lateral units used in study hydraulic model



2.6 Examination of existing gauging stations

As highlighted in a preceding section, in the summer of 2013, four hydrometric gauges were installed by the Office of Public Works at hydrologically significant locations in the catchment, (shown in Figure 2-1). The following sections detail the relevant gauges and their contribution (if any) to the validation of the hydraulic models It should be noted that a Unit Hydrograph approach to catchment inflows is not adopted in this study, and therefore calibration of percentage runoff and time to peak was not required. The analysis of the gauging station records has a limited role in the calculation of design flows for the scheme.

A particularly effective use of the OPW gauge data has been the carrying out of a number of rudimentary flood wave travel time analyses at Blackpool village. This is possible without rainfall data using gauges on the same watercourse. For example, using the stations at Glenamought Bridge (Section 2.6.3) and Blackpool Retail Park (Section 2.6.2), it is possible to calculate the time it takes for flow to travel between each location. This was extremely useful as a sensibility check when combined with the flow monitoring data outlined in Section 2.7.

2.6.1 Glen River Park (Station 19057)

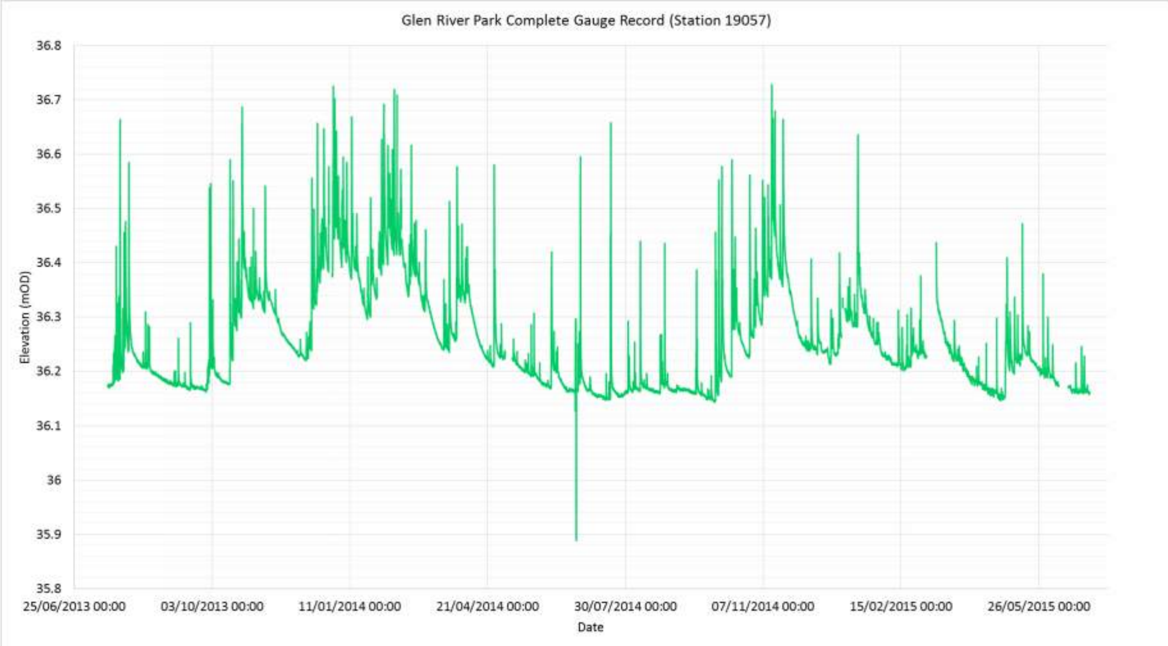
Station 19057 is located on the right bank of the Glen River, in the Glen River Park, approximately 1.3km upstream of the point where the watercourse enters the Phase 4 GBK culvert. The gauge is located at the bottom of a steep reach; with a relatively deep pool upstream of the board, as shown in Figure 2-6.

Figure 2-6 Pool immediately upstream of Station 19057

The full gauge record has some periods of data 'drop-out' as evidenced in Figure 2-7. However, the periods are not long in duration, and there is useable data available. As well as this, JBA have been provided with six spot-gaugings conducted by the OPW since installation of the gauge. However, these gaugings have not been consistently taken at the same location (i.e. at the gauge board), with some spot gaugings being taken at quite a distance away (30m downstream of the gauge). Also, the maximum flow measured by these six gaugings was $0.55m^3/s$. However, the hydraulic model has an initial condition of 3.5 m^3/s due to the complexity of the culvert system downstream. Therefore, the usefulness of this gauge in the study was limited, as no high order events were recorded.

In addition, the cross-sectional data at this location is sparse and it would be difficult to develop a rating curve using the hydraulic model without further survey work. This gauge has been used to help with some rudimentary 'travel time' analysis for both validation events and the study hydrology.

Figure 2-7 Complete gauge record for Glen River Park (Station 19057)



2.6.2 Blackpool Retail Park (Station 19058)

Station 19058 is located on the left bank of the Bride River, adjacent to Blackpool Retail Park. There is a series of small weirs immediately downstream of the gauge location. The complete water level record for the gauge, at the time of writing this report, is shown in Figure 2-8. There are quite a number of downward spikes present in the record, indicating periods of malfunction. 11 spot gaugings were provided for this location by the OPW since the installation of the gauge. As a result, a rudimentary rating curve could be constructed and compared against the model stage-discharge relationship in Figure 2-9.

Upon examination of the graph, one can see that there is an approximate, consistent difference of 1.4m^3/s between both curves; the model rating curve providing the higher estimate. This is not an indicator of poor performance or inaccuracies in the model however. The gauge is located approximately 20m downstream of the nearest model cross-section. This cross-section is a little deeper than the river bed at the gauge location. Therefore, for the same stage, the corresponding flow will be larger at the model cross-section, as shown in Figure 2-9. This adjustment has been made when abstracting data from this gauging station.

Figure 2-8 Complete gauge record for Blackpool Retail Park (Station 19058)

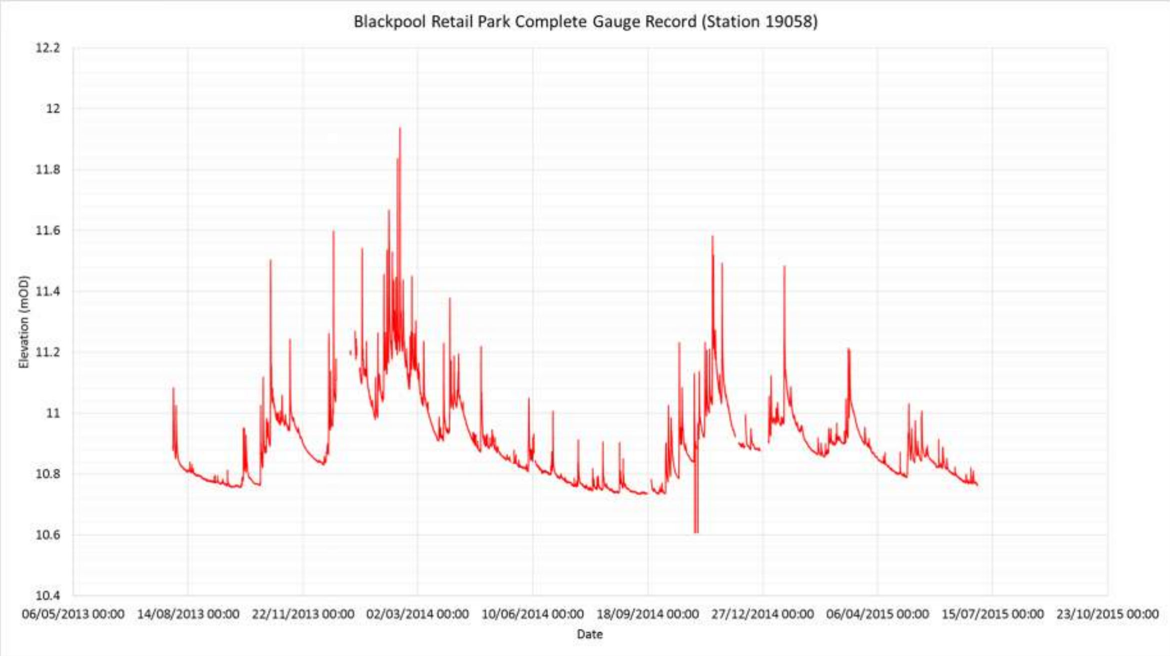
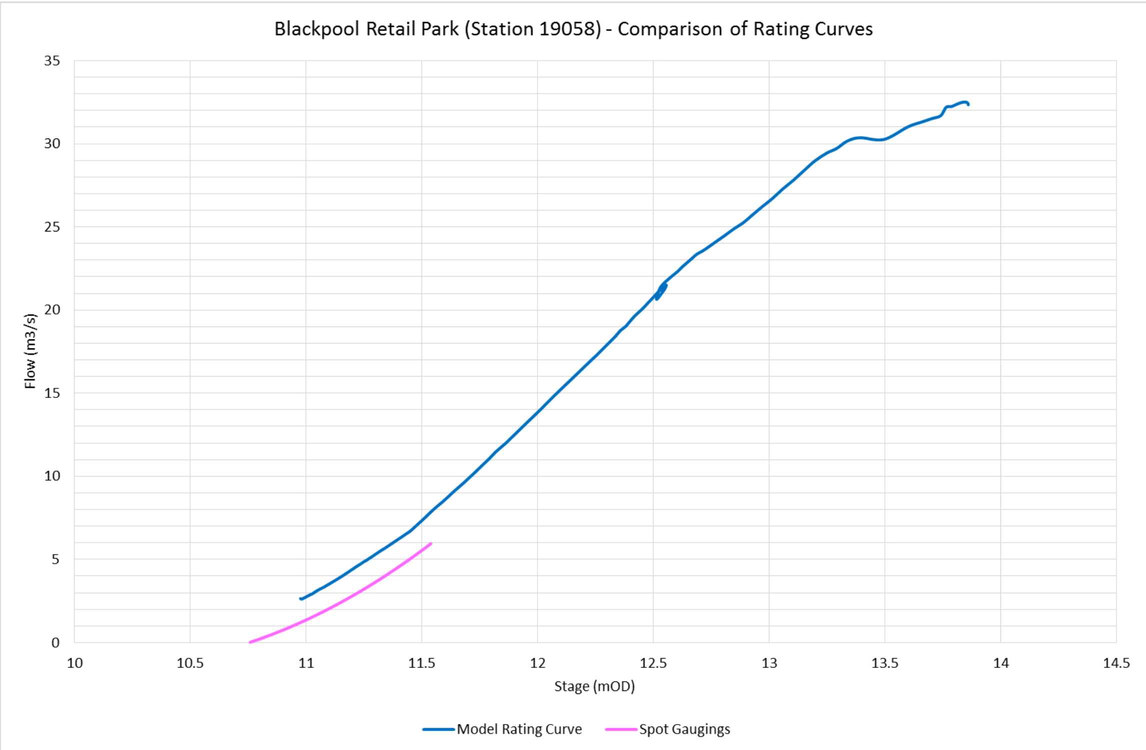


Figure 2-9 Comparison of rating curves - Blackpool Retail Park (Station 19058)



2.6.3 Glenamought Bridge (Station 19059)

Station 19059 is located on the left bank of the Glenamought River, adjacent to a private residence. The gauge board is fixed to the upstream face of Glenamought Bridge. The complete water level record for the gauge, at the time of writing this report, is shown in Figure 2-10. There are some noticeable periods of poor gauge performance. As before, the OPW have taken 5 spot gaugings at this location over the period of the gauge record. The rating curve derived from these gaugings is compared against the curve extracted from the hydraulic model in Figure 2-11.

It is apparent that the spot gauging rating curve is quite a good fit relative to that extracted from the model. The majority of OPW spot gaugings were taken 30m upstream of the gauge, therefore, model XS 7BR1_1297IN was used to provide the rating curve. This is an interpolate, located approximately 30m upstream of a surveyed cross-section at the upstream face of the Glenamought Bridge (7BRI_1267).

Figure 2-10 Complete gauge record for Glenamought Bridge (Station 19059)

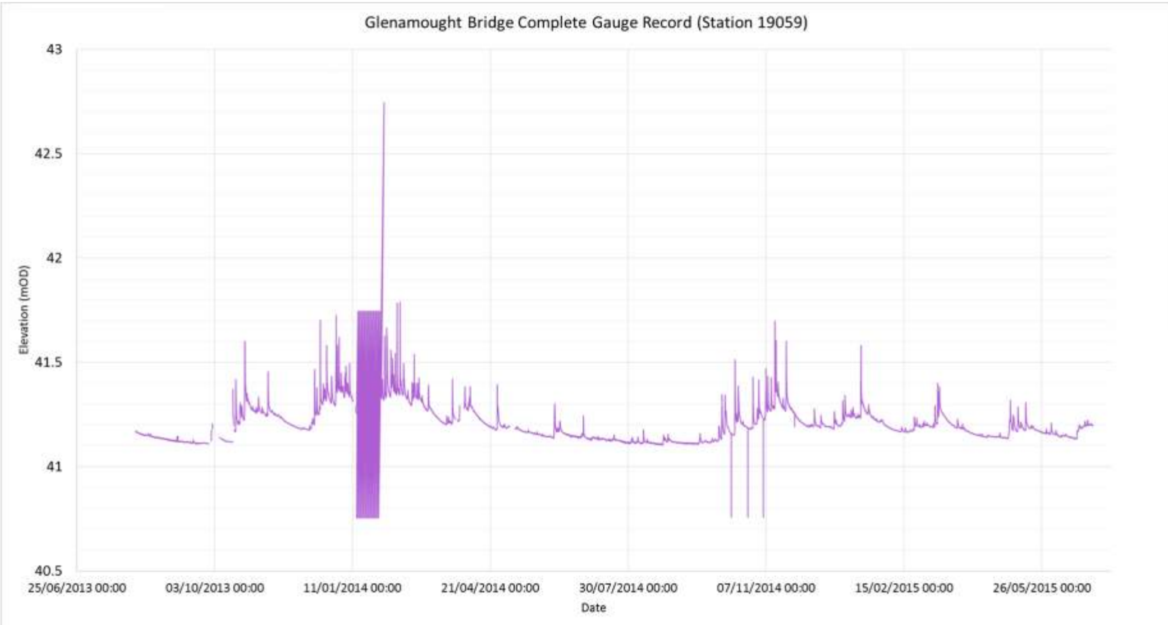
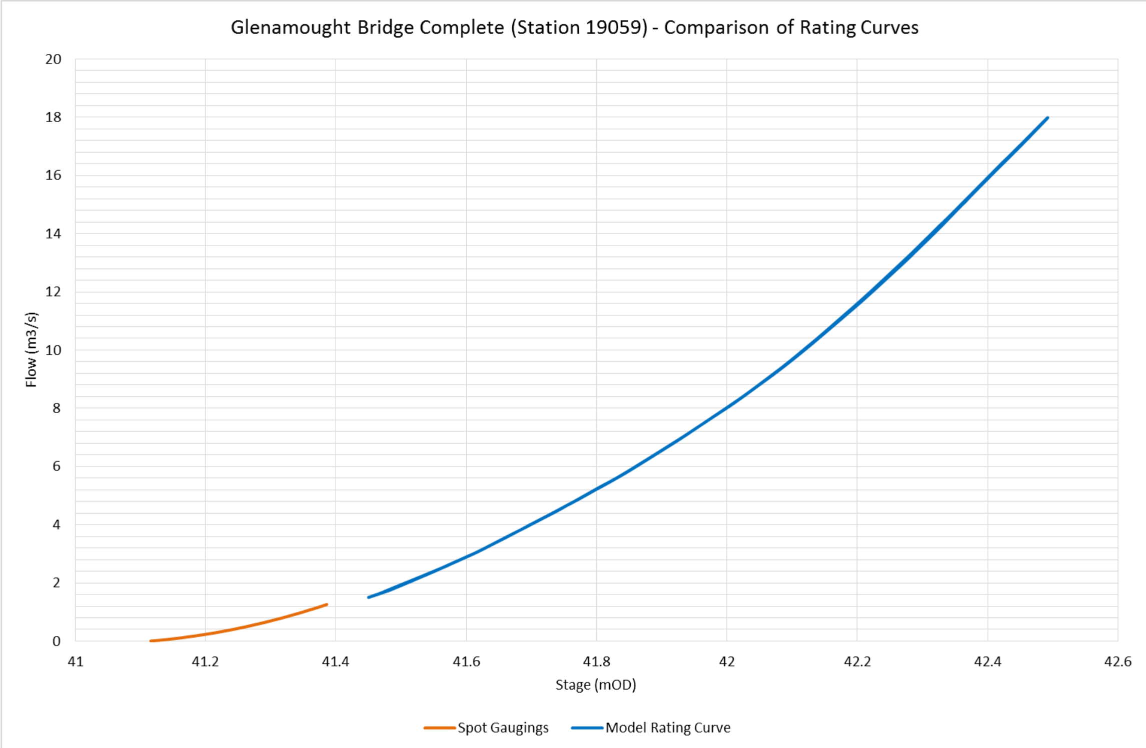


Figure 2-11 Comparison of rating curves - Glenamought Bridge (Station 19059)



2.6.4 Conclusions

It can be concluded from the preceding sections that the events experienced since the gauges' installation have been at the lower end of the hydraulic model rating curves. A more stringent and exacting examination of the hydraulic model could be done if there were more spot gaugings taken at times of high flow. However, in the intervening period since the gauges' installation, it has been relatively dry and the scale of rainfall similar to that of June 2012 has not occurred.

The gauge data contains a sizeable peak that occurred in February 2014 at all gauge locations. Station 19058 (Blackpool Retail Park) peaked on the 14th February 2014, with a level of 11.925mOD. Using the hydraulic model rating curve (after an adjustment has been applied to account for the location of the gauge relative to the nearest model XS), this is approximately equivalent to a flow rate of 9.8m3/s. This flow rate is slightly smaller than the expected median flood at Blackpool Shopping Centre of approximately 11.5m3/s.

The OPW hydrometric gauges have been used to conduct a 'travel time' analysis for the Bride system; this will be examined in greater detail in Section 3.2.

2.7 Blackpool flow monitoring survey

2.7.1 Contract Overview

Early in the project lifecycle, it became clear that there was a significant dearth of flow and level data on the Blackpool culverts; despite the presence of OPW gauges described in the previous section. This lack of data made it particularly difficult to interpret and predict the hydraulic behaviour of the culvert system in the centre of Blackpool village. Also, it was believed by some residents and business-owners in the village that the speed of the Glen in the Madden's Buildings junction had a significant effect on water levels upstream at the church. Therefore, it was decided by the project team that monitoring of the different culvert branches in Blackpool would help gain a greater understanding of the system; particularly around the Madden's Buildings bifurcation. Monitoring would also help to understand how the Glen and Bride branches interact with each other and evaluate headlosses at key points in the system.

Water Technology Ltd. were commissioned by the OPW to carry out this monitoring; and work began in March 2014. The monitoring period extended from the middle of March 2014 to the end of February 2015.

The following sections are a synopsis of the work carried out as part of the flow monitoring survey. For further information, the Blackpool Flow Monitoring Survey Report should be consulted in Appendix E.

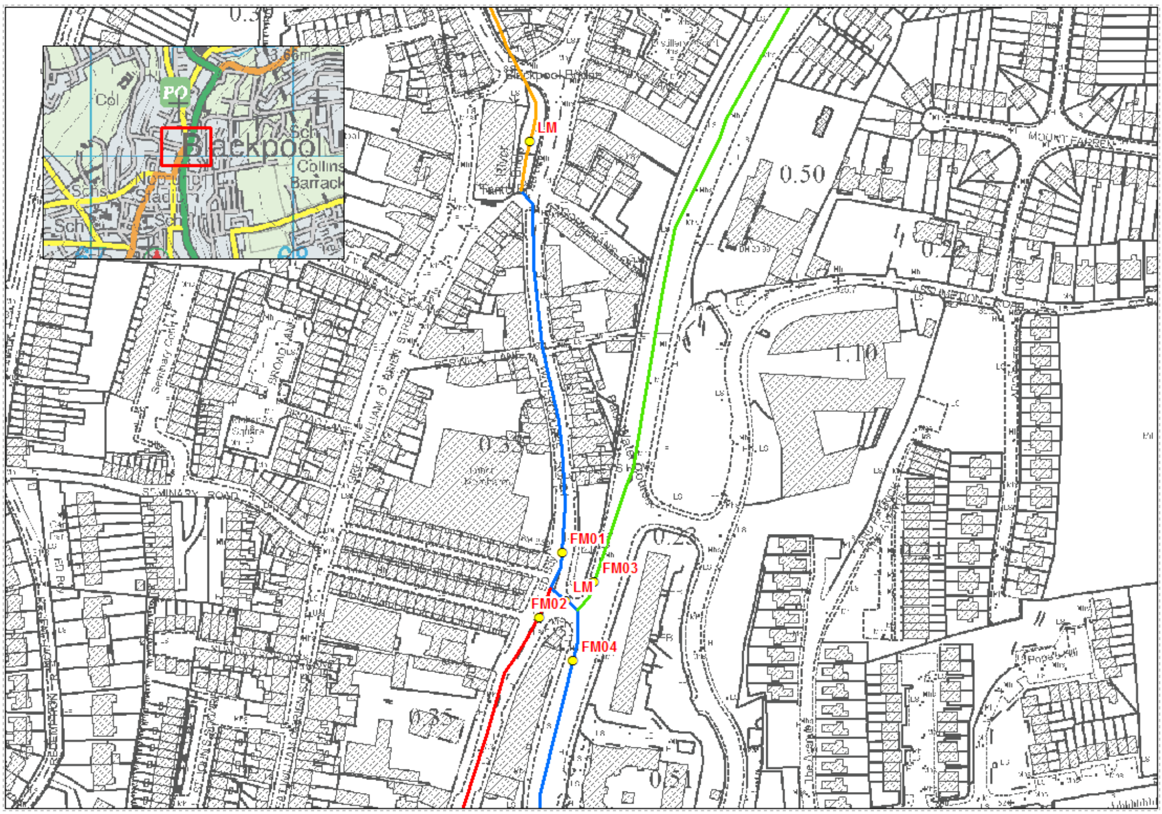
2.7.2 Monitoring Configuration

The brief that Water Technology Ltd. worked to was as follows:

- To determine if there was surcharging or restriction to flow movement during rain events within the GBK culvert system.
- To calculate the flow distribution throughout the bifurcation during rain events.
- To provide rainfall, level and velocity data to inform the calibration of the hydraulic model.

To satisfy these requirements Water Technology Ltd. installed monitoring equipment at the locations highlighted in Figure 2-12.

Figure 2-12 Location of flow monitoring equipment used in contract



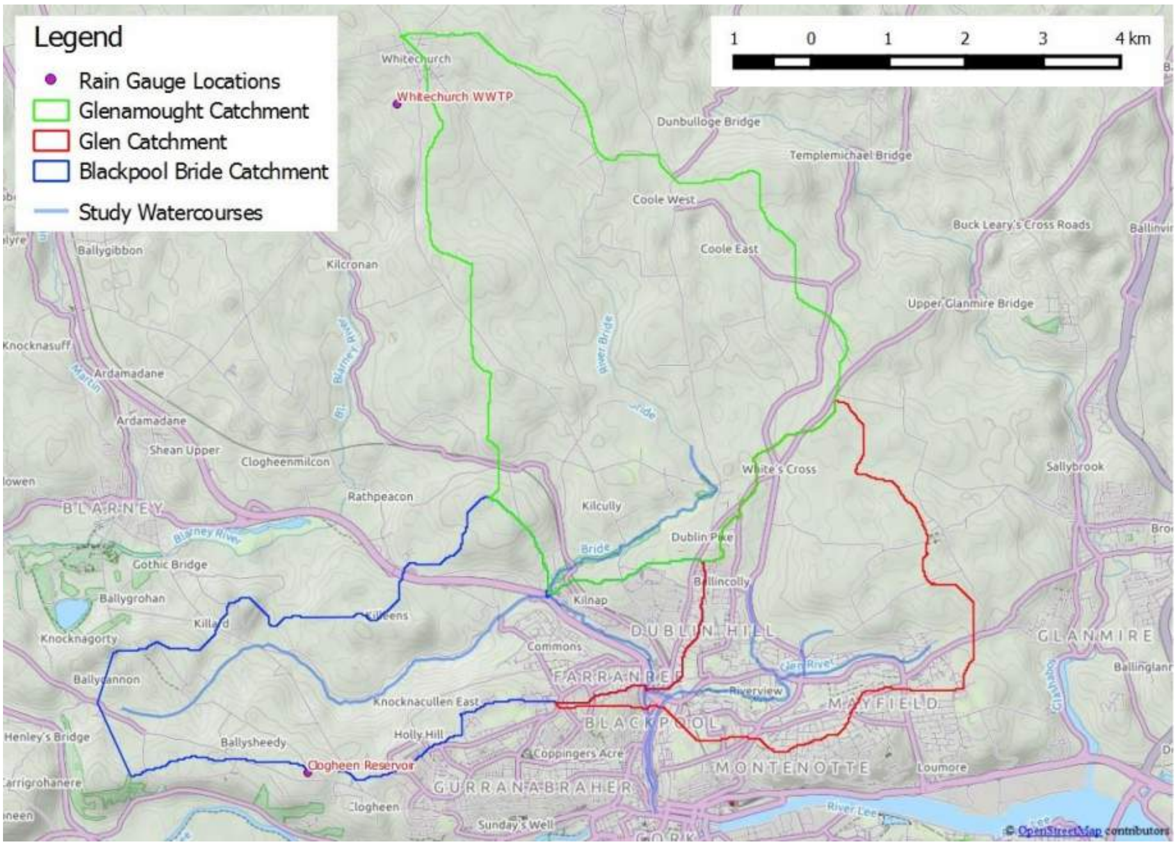
In total, four flow monitors were installed around the Madden's Buildings junction; one for each culvert branch. Each monitor used was an ISCO 2150 Area Velocity Flow Module. It uses Doppler technology to directly measure average velocity in the flow stream. An integral pressure transducer measures liquid depth. Flows are then calculated using the cross-sectional area at that particular location. All monitors used in the study (with the exception of the laser) were fixed to the culvert floor. A brief explanation of the purpose of each flow monitor is provided below:

- Flow Monitor 1 (FM1) - This flow monitor was located just upstream of the bifurcation on the Blackpool Church culvert branch (Bride). This is the larger flow volume draining to the junction. At a later stage in the flow monitoring contract, a supplementary roof-mounted laser monitor was installed here and proved to be quite successful. The reason for this will be outlined in Section 2.7.3.
- Flow Monitor 2 (FM2) - This flow monitor was located on the Brewery Branch, just downstream of the junction. This unit was initially installed 40 metres downstream of the bifurcation in an attempt to find a suitable section of channel for flow measurement. Prior to the official start of the contract period, it was moved back upstream to remove the possibility of errors from storm pipe discharge entering the culvert.
- Flow Monitor 3 (FM3) - This flow monitor measured flow entering the junction from the steep Phase 4 GBK culvert (Glen).
- Flow Monitor 4 (FM4) - This flow monitor measured flow exiting the junction using the primary route through the Phase 3 GBK culvert.

In addition to the monitors outlined above, there were also two level-only monitors installed on the system; one at the outlet of the Orchard Court culvert and the other in the middle of the bifurcation. In December 2014, the latter was moved to the FM2 location. Early in the contract period, the level monitor at Blackpool Church had to be moved further away from the culvert outlet due to vandalism.

To support the flow and level monitoring units, two rain gauges were installed at different points throughout the combined Bride, Glenamought and Glen catchments. A map of the gauge locations is shown in Figure 2-13.

Figure 2-13 Overview of rain gauge locations for flow monitoring contract



Rain gauges were installed at the following locations:

- Whitechurch Waste Water Treatment Plant
- Clogheen Reservoir

Figure 2-13 shows that none of these rain gauges are within the catchment boundaries of their respective watercourses. However, they were the only suitable sites identified where the rain gauge equipment could be safely installed and maintained. To supplement the flow monitoring contract, rain gauge data was taken from the Cork Airport climatic station.

2.7.3 Limitations of monitoring configuration

The following sub-sections present some of the limitations and difficulties encountered throughout the life of the contract.

2.7.3.1 Difficulties in reading depths and velocities

A problem that quickly arose during the monitoring contract was the speed and relatively shallow depth of the waters being gauged during the low flow periods. Low level and/or high velocity clean water is difficult to measure using the Doppler principle. The relatively high velocities were caused by the steep nature of each culvert branch, especially on the Phase 4 GBK culvert. The "mouse" head is placed on the bed of the stream and during normal flows can cause a hydraulic jump over the mouse. It was unfortunate that during the lifetime of the contract it was exceptionally dry; with very few significant rain events (will be discussed in further detail in Section 2.7.4). During flood events these problems were not as prominent, but it made preparing the probes for flood conditions difficult.

2.7.3.2 Ragging and accumulation of debris on probes

It became clear very early on in the monitoring period that ragging on probes was going to be a serious issue. As previously discussed, debris in the Bride watercourse is an ongoing problem due to the urbanised nature of the catchment that it drains. A combined sewer overflow is also located upstream. It was common during the contract period for various plastics, rags and sewer debris and organic materials to snag or catch on the monitoring probes. These materials created errors with level measurement by creating small, localised standing-waves at probe locations. An example of the problem is shown in Figure 2-14.

Figure 2-14 Ragging on the FM1 probe - 20th May 2014

The problem was more pronounced at both the FM1 and FM4 locations. To combat the effect the ragging had on probe readings, Water Technology Ltd. engaged in frequent inspection and cleaning of the equipment. It must be noted that at times of higher flows, particularly in winter 14/15, that ragging became less of an issue. Also, the installation of the laser monitor at FM1 represented a major improvement due to its roof-mounted location not being affected by floating debris.

2.7.3.3 Standing wave observations

Water Technology Ltd. reported seeing waves at all monitoring sites at intermittent times. The waves could occur upstream, downstream or on top of the probes themselves. These waves had a similar effect as the ragging described above. Water Technology Ltd. were unsure as to the cause of these waves at the time. However, upon inspection of photographs and application of hydraulic theory, JBA determined that these waves were a product of turbulence in the bifurcation and water flowing over the probes. The standing waves were particularly noticeable during higher flows; coming under greater influence of the sharp 90 turn.

These standing waves can affect the monitors as there is a risk that a 'false' water level has been recorded at the probe during an event. This can overestimate the flow at that particular point and distort the hydraulic gradient. This will be discussed further in Section 3.

2.7.4 Noteworthy events

During the course of the monitoring work, there were three noteworthy flow events:

- 13th/14th November 2014 event
- 21st November 2014 event
- 14th January 2015 event

The 13th/14th November 2014 event was the largest flow event to occur during the monitoring period. However, the 21st November event was comparable in magnitude. All noteworthy flow events occurred after the diversion block at the head of the Brewery Branch had been removed (August 2014). Therefore, its absence would have a significant effect on water levels in the bifurcation. As highlighted previously, the monitoring period was exceptionally dry and the above events were the only relatively substantial occurrences during the project lifetime. They will be summarised in greater detail in the following sections. The 21st November 2014 event was used for model calibration, so this will be described in Section 3.

Figure 2-15 Timeline of noteworthy events and equipment changes

The following sections outline some key flow events and their relative magnitude but for detailed analysis and discussion, please refer to Section 3.

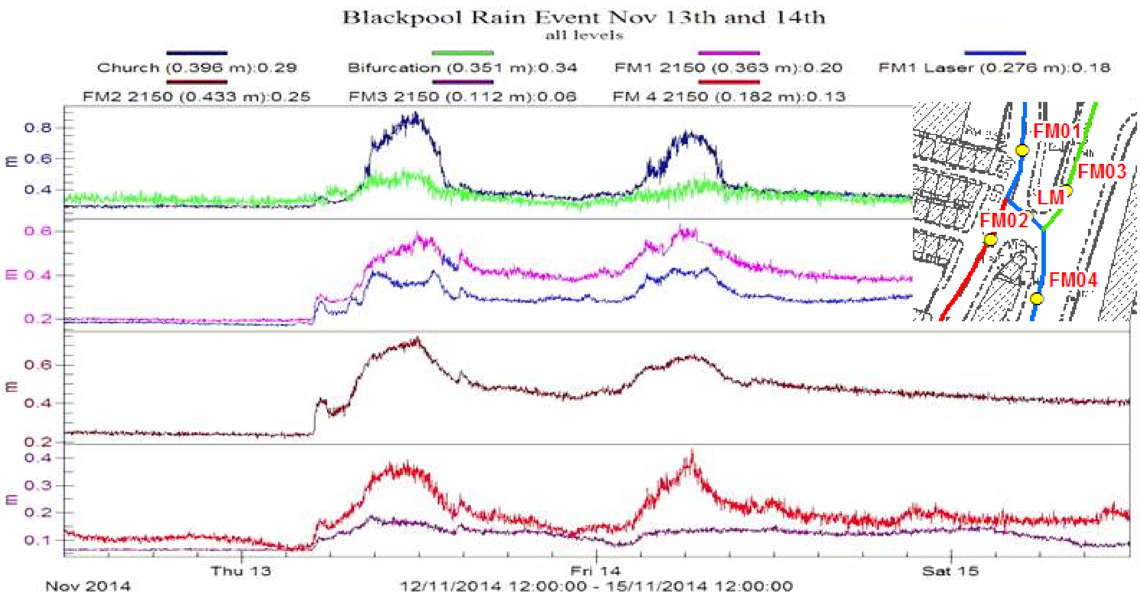
2.7.4.1 13th/14th November 2014

After a dry summer, October and November were wet months; resulting in an increase of water level at Blackpool Church from 6.32mOD (Malin) to 6.49 mOD (Malin) on the 12th November. The most significant event occurred on the 13th November at 4am. Another significant event occurred the following evening, resulting in the high flows shown in Figure 2-16 at Blackpool Church. 50mm of rainfall was logged between 13th and 14th November.

Figure 2-16 Elevated water levels at the Blackpool Church culvert entrance, on the morning of the 14th November

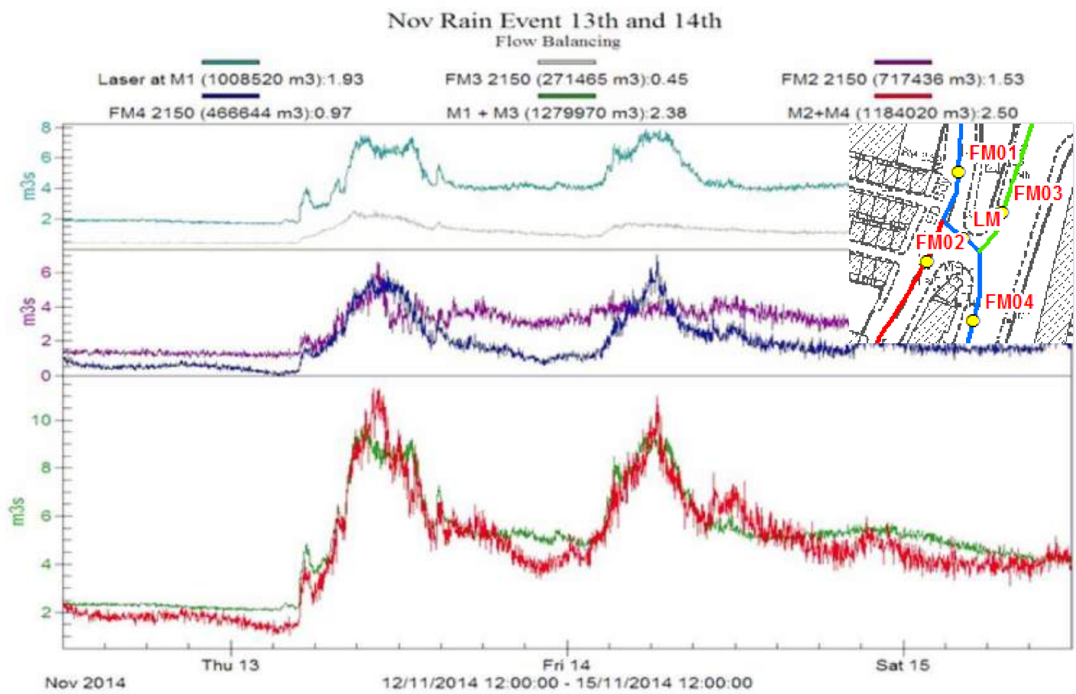
Peak depths of 0.8m (WL 6.99mOD) were recorded at Blackpool Church, which were the highest on record over the project period. In fact, all levels logged during these two rain events in November were the highest recorded at all survey locations. It should be noted that the Blackpool Church level monitor is located directly underneath from where Figure 2-16 was photographed i.e. at the outlet of the Orchard Court culvert. Again, both the FM1 2150 and FM1 Laser were recording at the same location; the difference being that the laser was mounted on the roof and uses contactless technology to measure level.

Figure 2-17 Levels recorded at each flow monitoring location during 13th/14th November events



When these levels were translated to flow data, Water Technology Ltd derived the values shown in Figure 2-18. 'M1+M3' and 'M2+M4' denote the sum of the flows at those particular monitors.

Figure 2-18 Flow values at each monitoring location for the 13th/14th November events



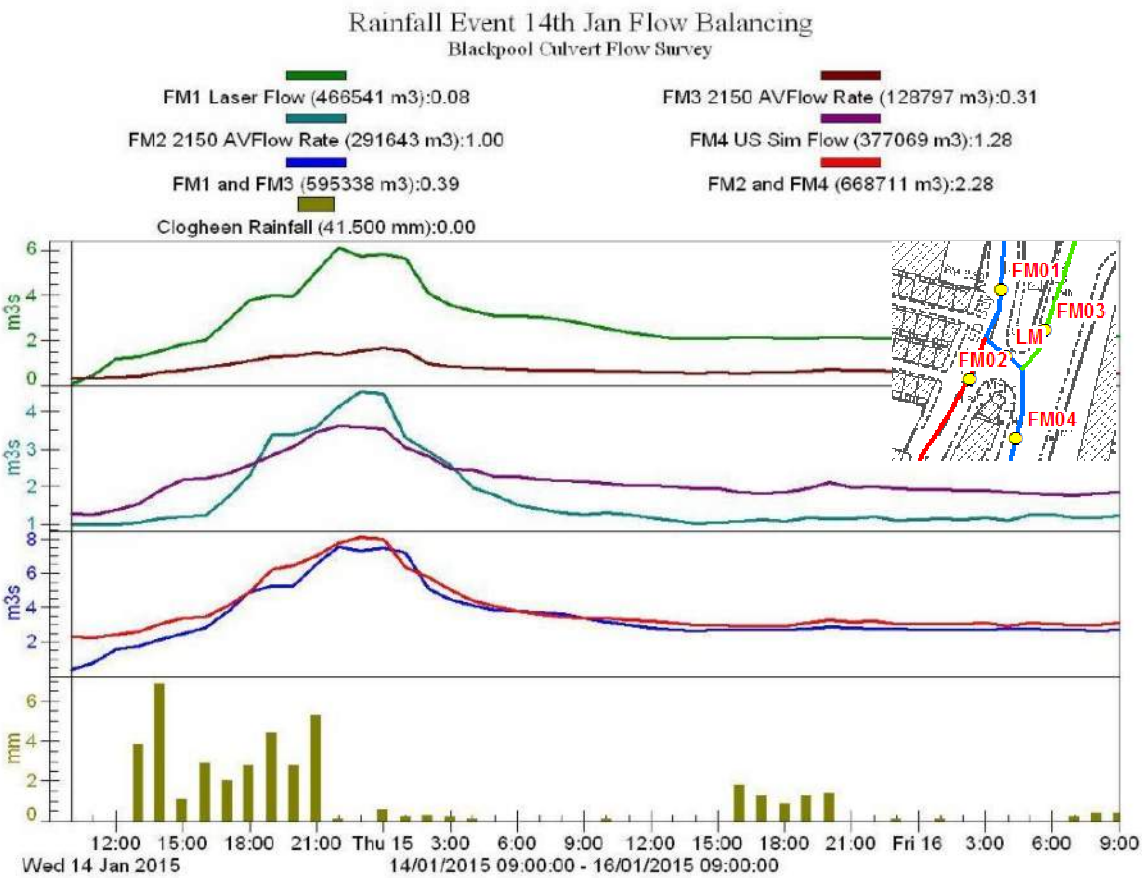
As the Laser was roof-mounted, and thus less susceptible to misreading due to standing waves and ragging, it has been designated as the primary monitor for this study. However, during the 13th/14th November event, a dip has been registered at the peak, as shown in Figure 2-17. This is inconsistent with the smoother peak at FM1. Therefore, as the 21st November 2014 event was similar in magnitude and its Laser level readings seem more consistent with what one would expect, it has been chosen for calibration purposes; and the 13th/14th November event has been used for 'time-of-travel' analysis only (see Section 3.2.1)

It must be noted that due to the irregular nature of the Brewery Branch culvert, it was assumed for ease of calculation to be a regular rectangular channel. It was found that at the peak of the event, there was approximately 70% of the flow coming from the Blackpool Church culvert (FM1), draining down the Brewery Branch (FM2). The remainder was carrying across the bifurcation and into the Phase 3 GBK culvert (FM4).

2.7.4.2 14th January 2015

This was an unusual event as there was a few centimetres of snow covering the catchment at the start of the event. However, the snow quickly melted and contributed to the flow in each watercourse; adding to the flow that had already been supplied by a storm. It was found that Cork Airport recorded 30.3mm of rainfall; the rain gauge at Clogheen recorded 32.7mm. The event occurred over an approximate 18 hour window from midday on the 14th January, to the early hours of the 15th. Figure 2-19 shows the flow rates recorded at each of the monitoring locations during the event.

Figure 2-19 Flow values at each monitoring location for the 14th January event



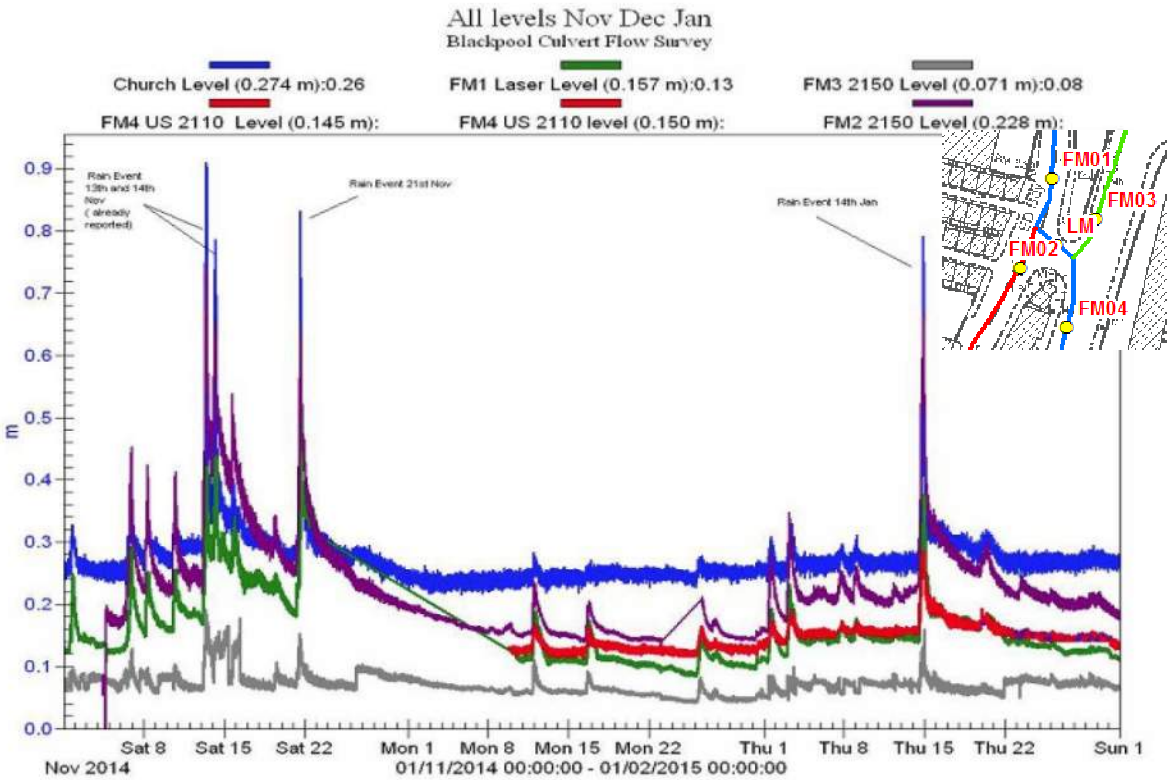
It was found that approximately 49% of flow coming from the Blackpool Church culvert travelled down the Brewery Branch culvert, with the balance flowing across the bifurcation, toward the Phase 3 GBK conduit. The max flow recorded at the downstream end of the Blackpool Church culvert was 6m^3/s. Flows at FM4 (the monitor at the head of the Phase 3 GBK culvert) were derived from an algorithm, established early in the contract from previous level and velocity data.

2.7.5 Findings from flow survey

This section summarises the primary findings from the flow survey with regard to balancing of flows across the Madden's Buildings junction and the change in performance pre- and post-cleaning works. The procedure by which the flow monitoring data was used to conduct time-to-travel analyses and calibration of the hydraulic model is explained in greater detail in Section 3. This will also show a comparison of modelled and measured hydraulic gradients through the culvert system.

As previously stated, the November 2014 and January 2015 events were the most significant recorded during the period of the survey contract. This is evidenced by the plotted water levels in Figure 2-20 for the winter of 2014/2015.

Figure 2-20 All levels recorded in November, December and January



Levels at Blackpool Church never exceeded more than 0.9m in depth. The most significant levels within the culvert were recorded in the Brewery Branch conduit (FM2), with depths as large as 0.7m.

Various analyses were carried out concerning the flow split that occurs at the bifurcation. It was found that approximately 60% of the flow draining down the Blackpool Church culvert continued on through the bifurcation and into the Phase 3 GBK culvert. The balance swept straight into the Brewery Branch conduit. It was found that during higher-flow events the percentage of flow reaching the Phase 3 GBK culvert reduced to between 29% and 45% (of flow entering the junction from the Bride); as it was easier for larger flows to just shoot through into the Brewery Branch. However, at the end of the contract, the percentage of flow capable of entering the Brewery Branch had reduced due to the accumulation debris at its inlet. Therefore, it has been assumed for calibration purposes that approximately 60% of the flow is capable of traversing across the bifurcation and into the Phase 3 GBK culvert from the Blackpool Church branch. This will be addressed further in Section 3.

A summary of the primary contract findings is presented below:

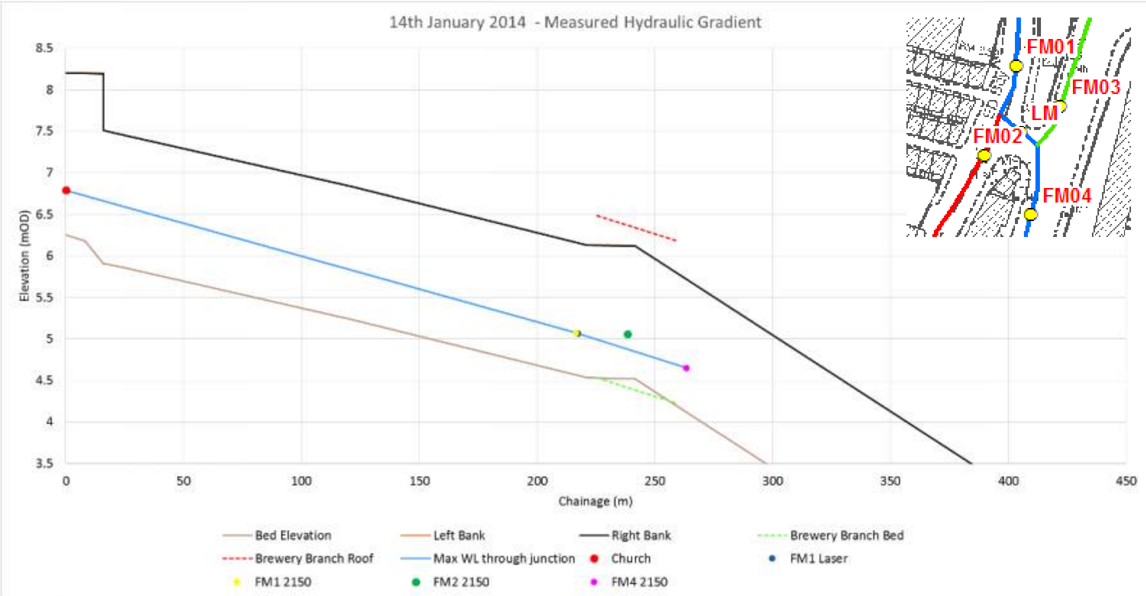
- After the cleaning works and removal of the diversion block between June - August 2014, very little change was observed in the hydraulic performance of the culvert due to the very low flows present at the time. The majority of flow tended to cross the bifurcation from GBK Phase 5 to GBK Phase 3; this did not change upon the removal of the block due to the very shallow depths of water in the junction.
- During some large flow events, it was estimated that approximately 30% to 40% of flow was traversing across the bifurcation. The balance continued straight through the Brewery Branch. However, the removal of the diversion block early in the contract, whilst still leaving the reinforcement bars in place resulted in a build-up of debris at the Brewery Branch inlet. This served to divert more flow toward the Phase 3 GBK culvert.
- During low flows, the majority of flow traverses the bifurcation, into the Phase 3 GBK culvert.
- The flow monitoring has shown that there does not seem to be an impediment to the Bride's flow down the Phase 3 GBK conduit as a result of the Glen's passage into the same culvert. Much of the headlosses associated with the junction occur upstream of this point; closer to the Brewery Branch inlet. Some members of the Blackpool Flood

Group and local residents were concerned that the speed and magnitude of flow down the Phase 4 GBK culvert was a primary cause of elevated water levels upstream in the village. However, inspection of the level plots in previous sections show that water depths remain reasonably consistent at the downstream end of the junction i.e. FM4 and bifurcation level monitors. If the Glen was causing an issue, one would expect to see an increased water depth at the bifurcation monitor, in comparison to FM4, as a result of the backing-up of flow. In reality, one sees increased water depths around the FM1 and FM2 locations (relative to the other monitors), suggesting that the junction layout at the upstream end is causing localised headlosses. This adds weight to the argument that the Glen's effect on water levels in the village is minor, relative to the system inefficiencies upstream.

- At Blackpool Church, prior to the undertaking of cleaning works, flow in the Orchard Court culvert was spread quite evenly across the channel with isolated instances of turbulence where there was a localised build-up of silt and debris. A considerable volume of stone, silt and rubbish was subsequently removed from this section, as highlighted in Figure 7-2. After cleaning was completed, flow was seen to travel in the centre of the open channel as shown in Figure 2-21. Further upstream, flow was seen to prefer the road-side flank of the Orchard Court culvert. It was also found that the works undertaken had lowered water levels, but increased velocities.
- The accumulation of debris and ragging was a major issue throughout the life of the project. It was unfortunate that the Laser flow loggers were not available from the outset. For future monitoring work on this system, only non-contact technology should be considered.

Figure 2-21 Blackpool Church open channel section after June-August 2014 cleaning works

Figure 2-22 Measured water level profile for the 14th January 2014 event



3 Model calibration

3.1 Calibration Data

As highlighted in Section 1.4, there are a number of flood and relatively high-flow events against which the hydraulic model performance could be compared. The majority of data available relates to the recent June 2012 event and the flow monitoring contract.

It was decided to test the model performance using two distinct events due to the number of changes that have occurred on the Blackpool system in recent years:

- The results of the flow monitoring contract were used to calibrate the 'current' or 'existing scenario'. This is the hydraulic model upon which all options modelling and scheme testing would be based. This work will be discussed in greater detail in Section 3.3.
- Another distinct model was constructed, using the same framework as the 'existing scenario', to try and replicate the 28th June 2012 flooding. The differences between each model are summarised in Section 3.4.1. This was used for sensibility-checking purposes i.e. to test the model's ability to replicate the flow routes and hydraulic processes seen using event-specific parameters such as blockage and structural conditions in much larger events. Further conclusions could not be drawn from this model as the flow rate experienced at Blackpool Village for that particular event is unknown.

It must be noted that very limited calibration/validation of the Glen Stream reach of the hydraulic model could be carried out as the recorded levels at gauging stations upstream were low (relative to the initial conditions in the model) and there is not the same quantity of post-event data available for the June 2012 event in comparison to the Bride. Calibration of the Glen was limited to its downstream extents at the Madden's Buildings junction.

3.2 Use of monitoring data to confirm catchment response

As shown in Section 2.7, there are a number of noteworthy events that can be used to understand the behaviour of the Bride catchment, as well as evaluating the accuracy of the hydraulic model. The following sections will outline the catchment response to sustained rainfall and the process by which recorded water levels were used to calibrate the culvert system in Blackpool.

3.2.1 'Time of travel' analysis

'Time to peak' analysis refers to the quantification of the time it takes for a large rainfall event upstream in a catchment to translate to increased water levels downstream at the point of interest i.e. Blackpool village. It can be calculated using the following formula[8]:

$$T_p(0) = 0.604\ Lag^{1.144}$$

Where, Lag is the time between the centroid of rainfall and centroid of hydrograph peak.

However, the primary use of the monitoring data has been the confirmation of the time taken for the flood wave to travel along the watercourse. Using the OPW level recorders and the flow monitoring contract data, a detailed picture can be constructed for each noteworthy event as shown in the following figures. The OPW data has been recorded every 15 minutes; the flow monitoring data has been provided for every 2 minutes.

[8] Flood Studies Report. NERC. 1975

Figure 3-1 Recorded water depths for OPW and Water Technology Ltd. monitors

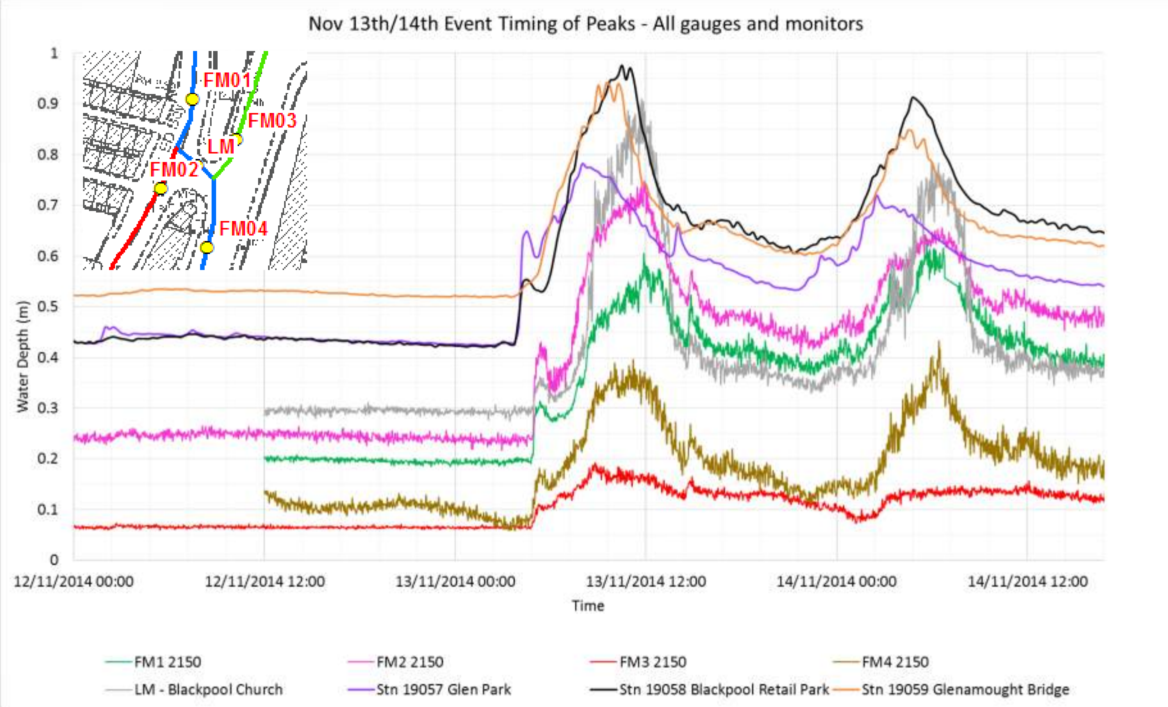


Figure 3-2 Recorded hourly rainfall totals for 13th/14th November 2014 event

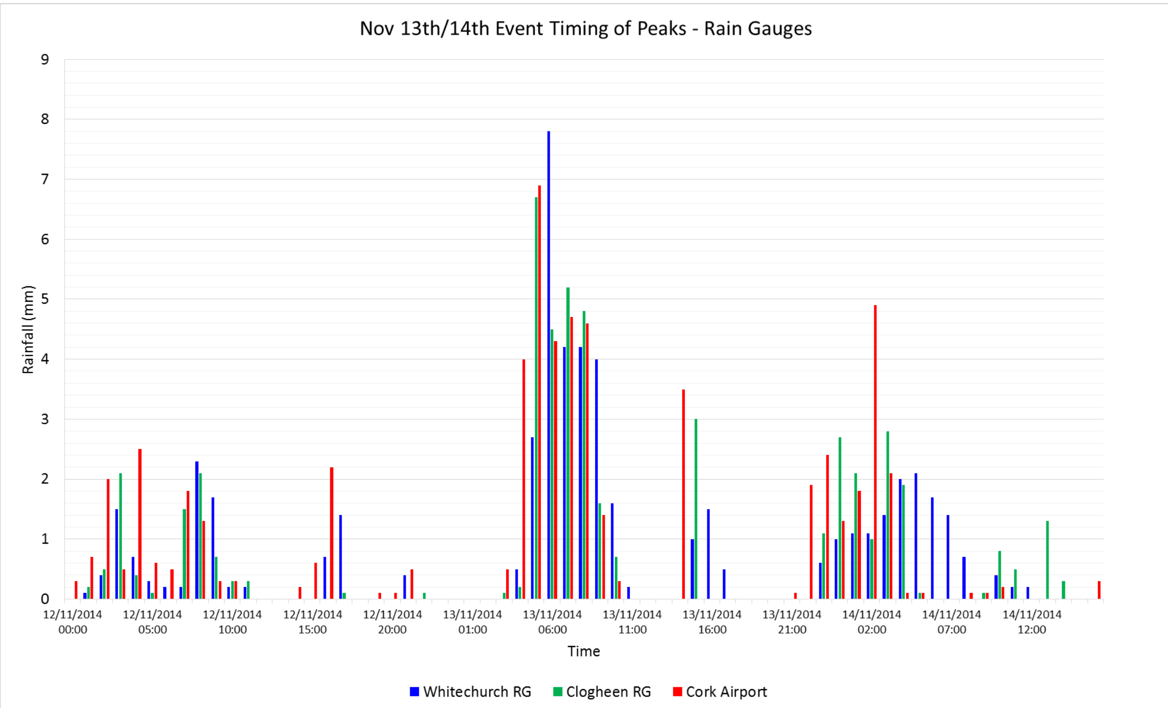


Figure 3-3 Recorded water depths for OPW and Water Technology Ltd. monitors

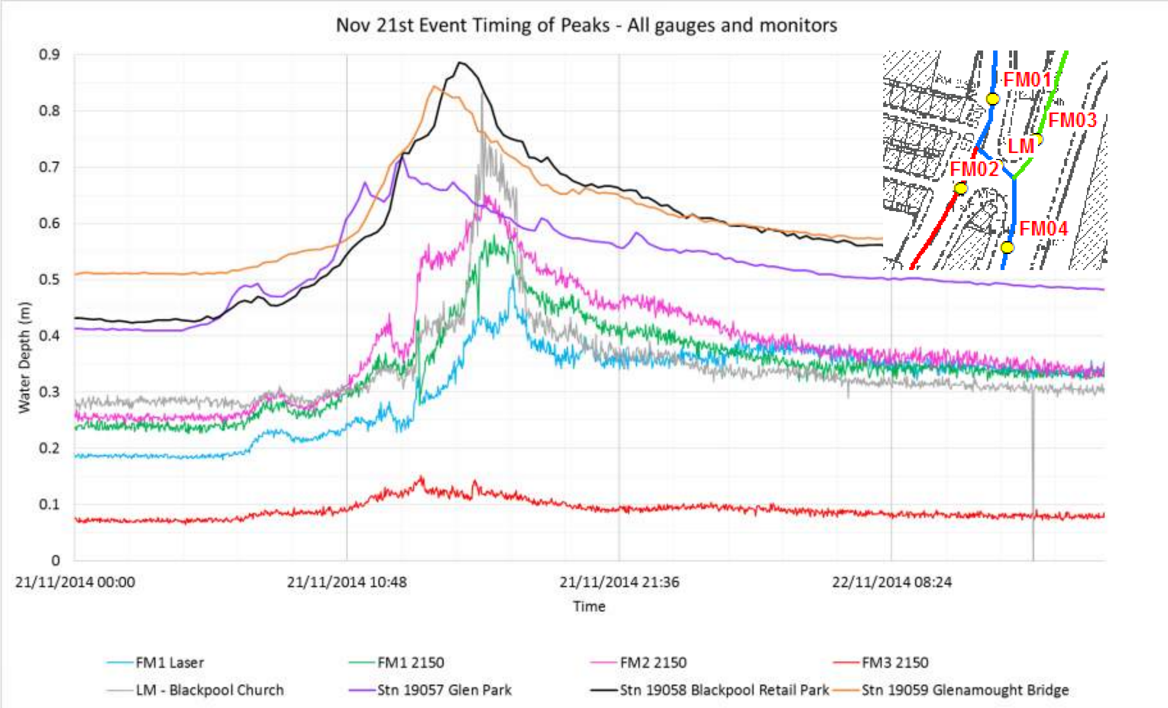


Figure 3-4 Recorded hourly rainfall totals for 21st November 2014 event

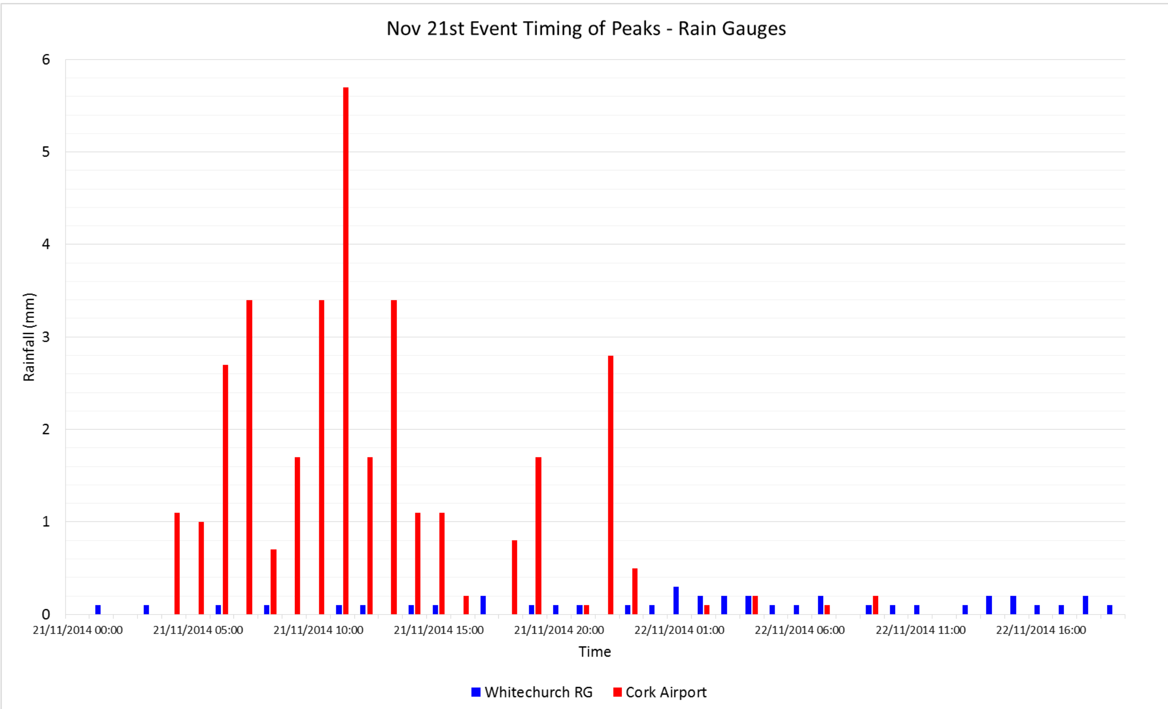


Figure 3-5 Recorded water depths for OPW and Water Technology Ltd. monitors

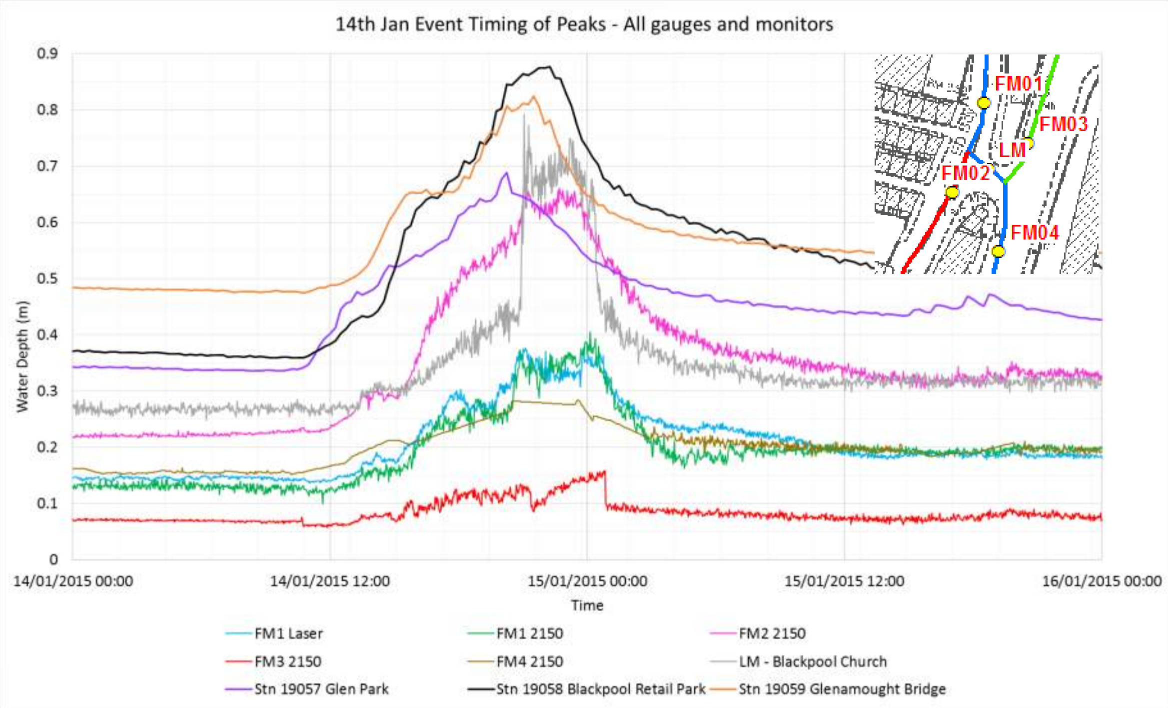
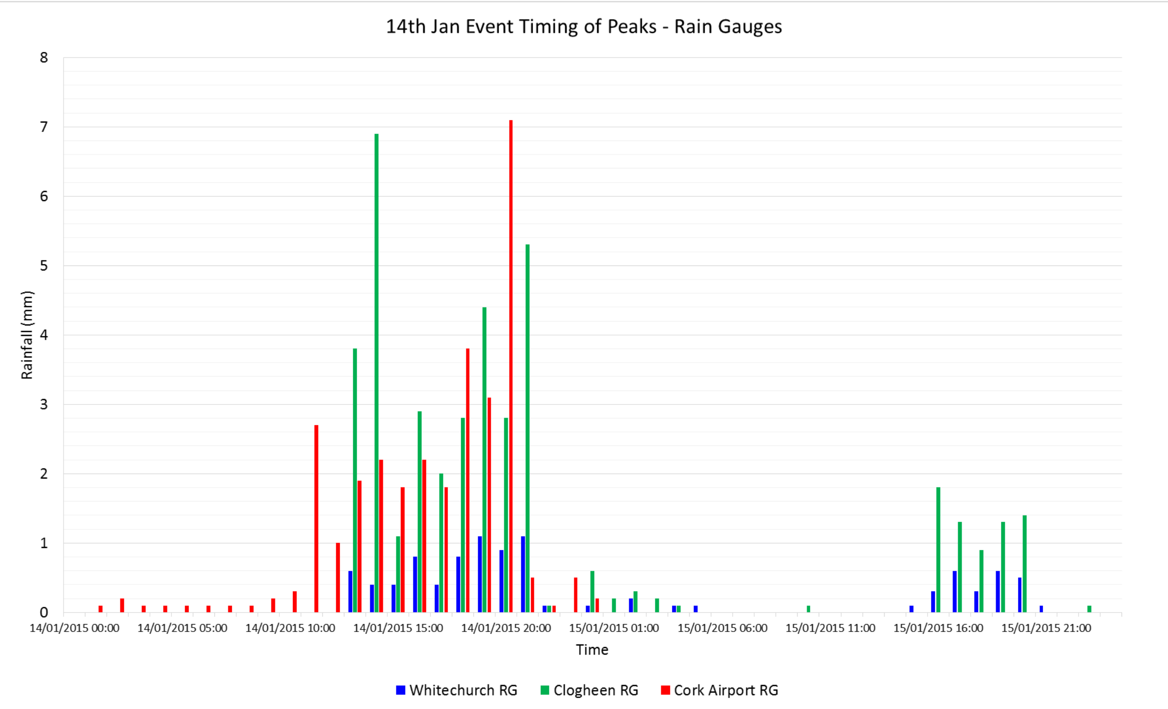


Figure 3-6 Recorded hourly rainfall totals for 14th January 2015 event



It is evident that there is a noticeable lag between peaks upstream and downstream on the same watercourse. This is what would be expected of a 'normal' dataset i.e. a gauge upstream peaks before downstream when there is no substantial intermediate inflow between the two points. The 13th/14th November event has been examined in more detail below to give a sense of the various travel times throughout the catchment.

- In the 13th/14th November event, there was an initial spike in rainfall data between 3:00am and 4:00am on the morning of the 13th.
- A corresponding spike in recorded OPW data occurred approximately 45 minutes later at both the Blackpool Retail Park and Glen River Park gauges.
- The Glen River Park gauge peaked first, at approximately 7:45am.
- The peak at Flow Monitor 03 (i.e. the monitor on the downstream end of the Phase 4 GBK culvert; thus is directly downstream of the Glen River Park gauge) occurred at 8:38am.
- The peak at the Glenamought Bridge gauge occurred at 9:30am. The gauge downstream, at Blackpool Retail Park, occurred at 10:30am.
- Peak water levels are recorded at the level monitor at Blackpool Church and Flow Monitor 01 (i.e. the monitor at the end of the Blackpool Church culvert) at 11:30am and 11:52am respectively. The peak water level in the Brewery Branch occurs 10 minutes later.

3.2.2 Comparison with hydraulic model

Therefore, using the information set out above, an approximation of the travel-times for the system during this event can be established, as shown in Table 3-1. When conducting this type of analysis it is standard practise to measure lag times relative to the centroid of the significant rainfall event.

Table 3-1 Summary of peak times at various gauge locations for the 13th/14th November event

Lag Time (relative to centroid of rainfall event)	Location of peak
+2h10m	Glen River Park
+2h35m	Flow Monitor 03
+3h40m	Glenamought Bridge
+4h25m	Blackpool Retail Park
+5h25m	Level Monitor at Blackpool Church
+5h45m	Flow Monitor 01

The data presented above seems to show that the watercourse system is relatively quick-responding to rainfall in the upstream reaches. For information purposes, if one uses the formula outlined in Section 3.2.1, the Tp for Orchard Court can be estimated at approximately 4.5hrs.

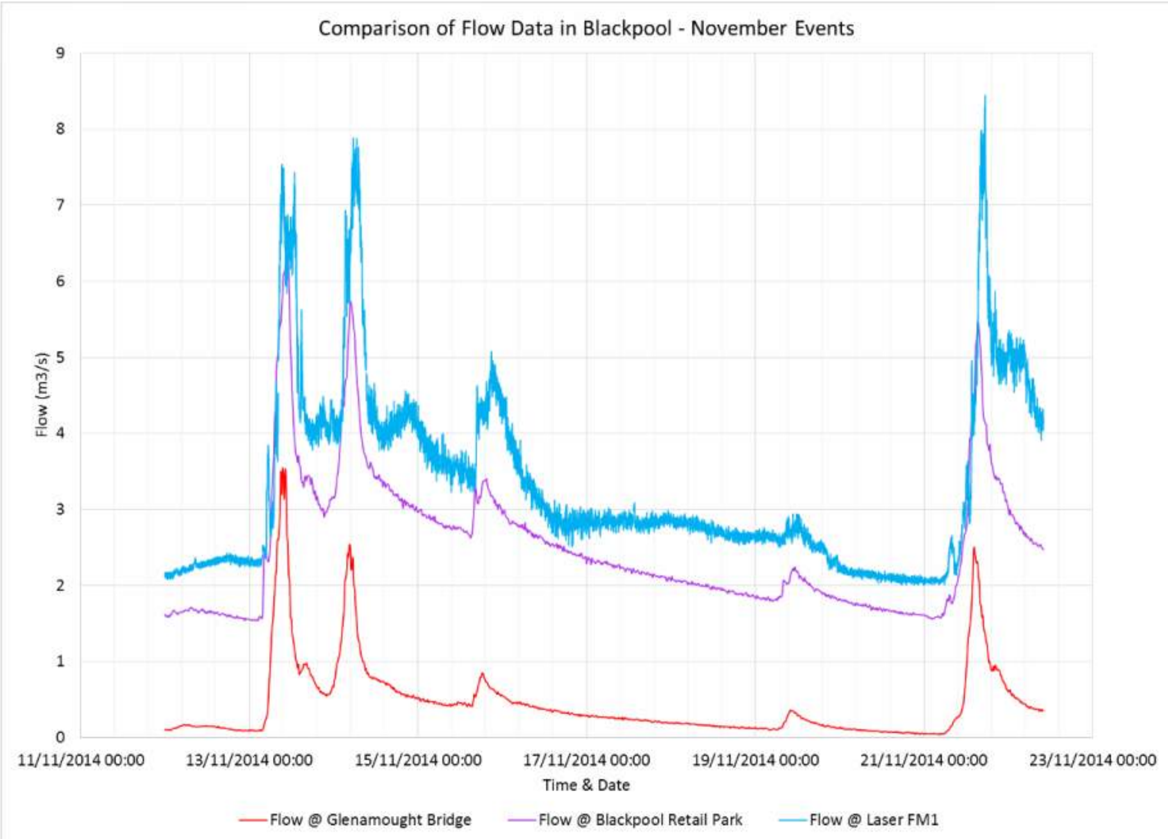
When looking at the 21st November event, the approximate timings established in Table 3-1 are broadly repeated. However, some discrepancies do occur, which is to be expected. The timings are not identical due to a combination of factors:

- The spatial distribution of rainfall was most likely different to that experienced in the 13th/14th November event. It can be seen from Figure 3-4 that the peak hourly totals for the Cork Airport gauge are not as large as those seen a week previously. The Clogheen rain gauge was unavailable at this time and it seems that the Whitechurch rain gauge was malfunctioning. However, the depths experienced downstream at the OPW and flow monitoring gauges are not dissimilar to those of the previous week. Therefore, it is probably safe to assume that the spatial distribution of rainfall was quite different to that of the previous week.
- The antecedent conditions within each sub-catchment can affect the time-to-peak downstream as areas with a larger antecedent moisture can reduce the rainfall's time of travel to the watercourse.

In the hydraulic model, the travel time between Glenamought Bridge and Madden's Buildings is approximately 1hr. For the same locations, the OPW and Flow Monitoring data suggests approximately 2hrs, as shown in Table 3-1. However, for the same event, the recorded travel time to Blackpool Retail Park is only 45mins. A difference of 1hr15mins for a flood peak to travel between Blackpool Retail Park and Madden's Buildings seems excessive when looking at the distances between each point of interest. It is possible that either gauge (the gauge at Glenamought Bridge or FM1) may have registered a false peak. Without a greater number of events, that are greater in magnitude, a definitive conclusion can not be drawn from this.

Figure 3-7 shows the difference in flows at various locations along the Bride system during the November 2014 flow events. The flows at Blackpool Retail Park and Glenamought Bridge have been derived using the hydraulic model rating curves. The figure illustrates quite well the flow gain between the Glenamought Bridge and the Blackpool Retail Park. This is due to the upper reaches of the Bride River merging with the Glenamought at the Commons Road. The figure also shows an increase in flow between Blackpool Retail Park and the bifurcation at Madden's Buildings.

Figure 3-7 Comparison of flow at various locations on the Bride system for November 2014



Without a larger number of events and more stationary and intense rainfall it is not recommended that the FSR rainfall runoff method of devising hydrograph shape is altered. The recorded events appear to suggest a much quicker response than has been observed in similar catchments. The hydrograph shapes have been kept as the default; as per the FSR calculation process. Without more data, it is difficult to justify the deviation from the standard FSR process in favour of the shorter Tp, as tentatively estimated earlier in this section.

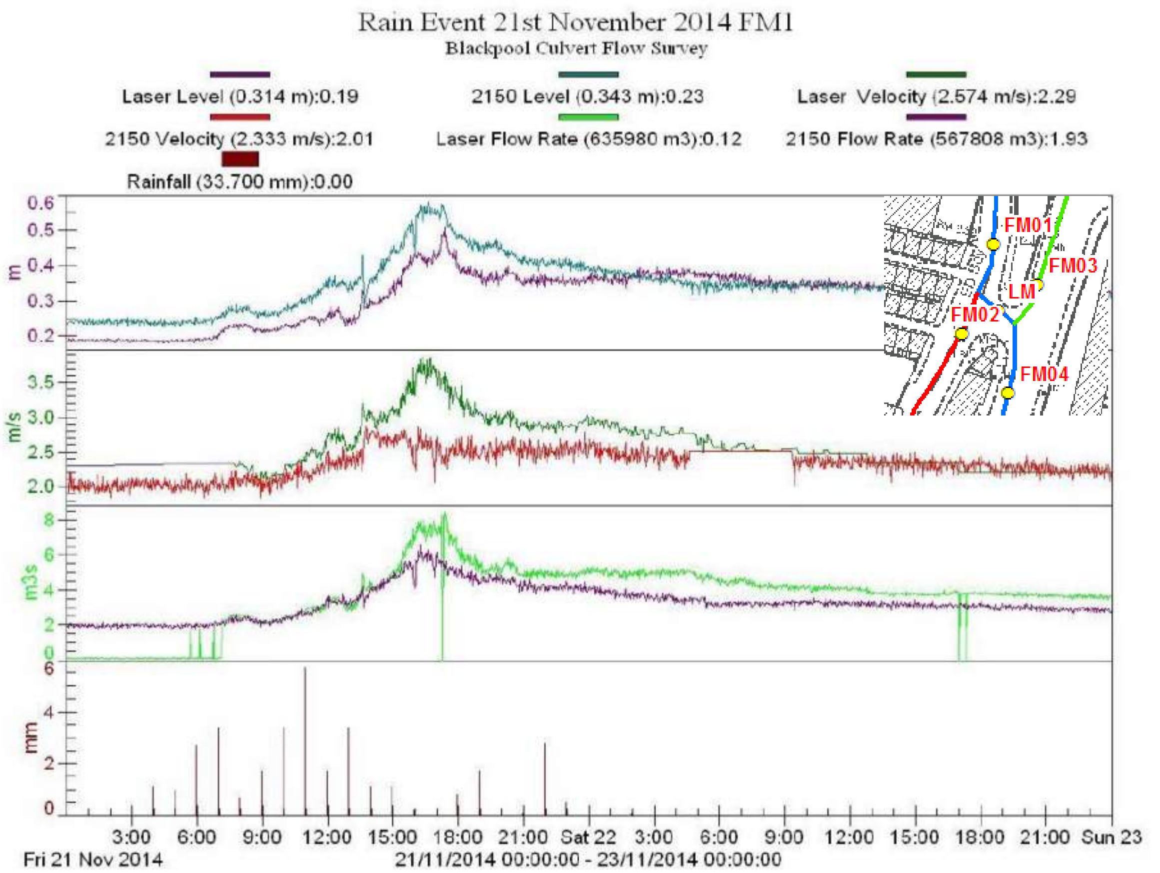
Peak flow is the key parameter, with hydrograph shape a secondary concern. The analysis of the monitoring data shows the impact of rainfall variation across the catchment on response time and flow magnitude. Process wise the travel times in the model seem to be within an acceptable limit, and this was the main objective of the monitoring in this limited period.

3.3 Model calibration

The 13th/14th November 2014 event was the largest flow event to occur during the monitoring period. However, it has already been explained that the primary monitor (i.e. the laser at FM1) registered a dip at the peak of the event, as can be seen from Figure 2-18. The regular monitor at FM1 has a degree of uncertainty in its velocity reading as a result of ragging and debris issues (as detailed previously).

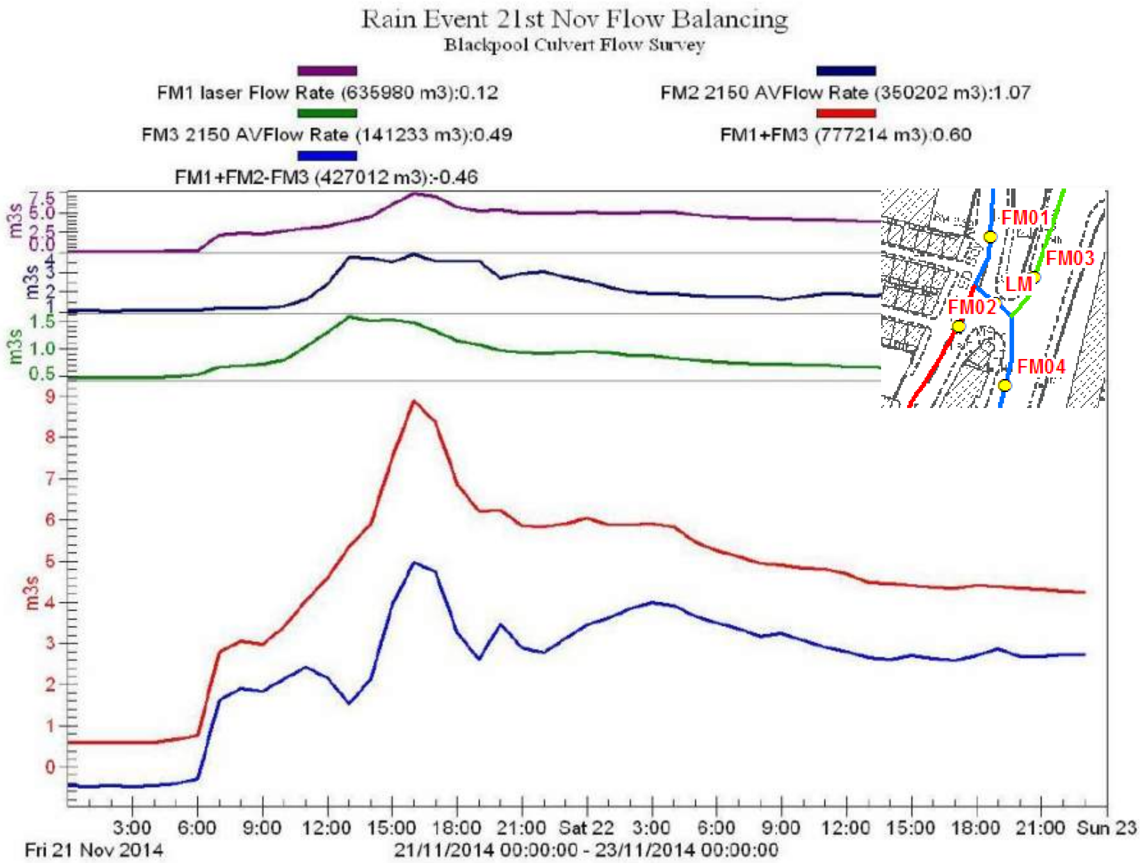
Therefore, rather than calibrate using an imperfect dataset, the 21st November 2014 event was used instead to calibrate the model to ensure that a reliable reading at the laser was included in the calibration process. Figure 3-8 shows the data recorded during the event at flow monitoring location FM1. This event was nearly identical in magnitude as the 13th/14th November 2014 event, and justifies its use for calibration purposes.

Figure 3-8 Summary of data recorded at flow monitoring location FM1 during November 21st event



There is a sudden dip in recorded flow rate at the laser monitor close to, but not at the peak of the event. A possible cause of this could have been a standing wave; as detailed in Section 2.7.3.3.

Figure 3-9 Summary of flow rates recorded at different flow monitoring locations during the 21st November event



3.3.1 Comparison of results

Some of the changes that were made to the model as a result of the flow monitoring include:

- The bend coefficients in the bifurcation were changed to 0.8 and 0.3 respectively to help reconcile levels at the FM1 and FM2 monitoring locations.
- The inlet coefficient at the Blackpool Church culvert was changed to 0.55, which is slightly higher than the default ISIS value of 0.515. This was done to reflect the additional headloss that would be experienced as a result of the abrupt turn to the left at the inlet.

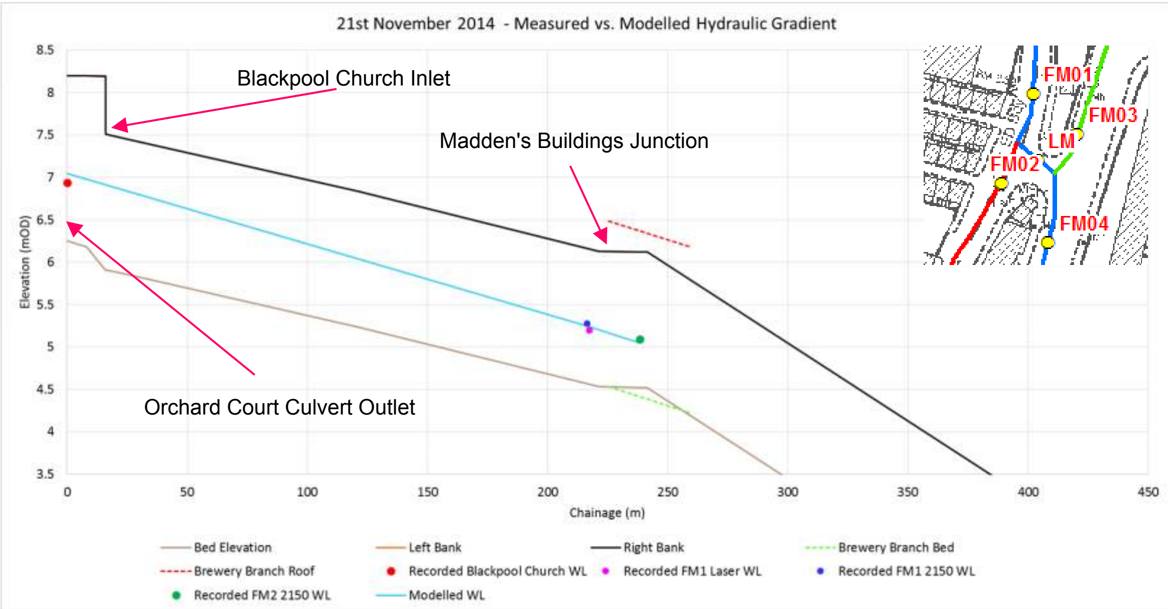
After a number of iterations, the model reached a point at which there was a close resemblance between the observed and modelled water levels, as shown in Table 3-2.

Table 3-2 Comparison of recorded and modelled water levels at a number of locations for 21st November event

Monitoring Location	Recorded WL (21st Nov) mOD Malin	Modelled WL mOD Malin
Blackpool Church Level Monitor	6.94	7.05
FM1	5.28	5.25
FM2	5.09	5.05
FM3	5.07	5.02
FM4	Malfunction (monitor washed off fixing)	4.99
Bifurcation	Malfunction	5.05
Laser	5.2	5.24

There were no glaring discrepancies between the observed and modelled values; all modelled water levels were within an acceptable error-band. The model slightly overpredicts the hydraulic gradient in the upper sections of the Blackpool Church culvert. This may be a result of the local hydraulics at the Blackpool church entrance and the turbulent conditions in this section of open watercourse. This can be examined in Figure 3-10. Unfortunately, as mentioned previously, larger flow events did not occur during the lifetime of the contract and as such, it is not possible to assess the performance of the hydraulic model at flow rates in excess of those recorded. All culverts remain in an open channel flow state, and pressurised flow does not occur.

Figure 3-10 Comparison of measured and modelled water levels for the 21st November 2014 event (hydraulic gradient through the system)



3.4 Verification of conditions against 28th June 2012 event

The 28th June 2012 event has been chosen as a 'common-sense' check by which the hydraulic model can be evaluated:

- The event is the largest flood to have occurred in recent memory.
- It is the largest event to have occurred after installation of the GBK system.
- There is a considerable wealth of information available to inform the calibration process.

The details of the 28th June 2012 flood event can be found in Section 1.4.2.

3.4.1 Validation assumptions

In order to try and replicate the condition of the Bride, Glenamought and Glen Rivers, various assumptions had to be made with regard to roughness, trash screens, blockage etc. These assumptions, for the most part, were based on visual records of the event and witness testimony, along with CCTV survey of the culverts which had been recorded before and after the event. The assumptions made for the calibration of the model are summarised below. Some of these assumptions have been summarised already in Section 1.9.3. Further assumptions are detailed below. In all cases, a range of values was tested in the model and the final coefficients used in the validated 2012 model are reported here. Some photographic evidence to reinforce these assumptions (taken from OPW post-event reports and local residents' records) are shown in Table 3-3.

- A peak blockage ratio of 30% was applied to the upstream faces of both the Orchard Court road-bridge and footbridge respectively. This was to mimic the accumulation of debris as flow met the structures and trash screens.
- The Orchard Court culvert system was given a Colebrook-White friction value of 600mm on its invert (equivalent to a Manning's value of 0.035). This accounts for the debris on the floor of each conduit section.

- The ESB services tray was included in the Orchard Court culvert as a pair of orifice units. The services tray was particularly restrictive as it allowed vegetation to collect above the tray, potentially reducing the capacity of that conduit section by a maximum of 50%. For the June 2012 event, the blockage in the orifice above the ESB tray was estimated to be 50%. This was supported by the subsequent CCTV survey in July 2012 which showed significant vegetation collection in the section above the tray. Therefore, it is not unreasonable to assume the value of 50% blockage based on the CCTV survey findings. This services tray has subsequently been removed.
- At the entrance to the Brewery Branch, the weir used to represent the diversion block was given a loss coefficient of 1.1. This is an extremely conservative figure that served as an attempt to represent the collection of various debris, as shown in Figure 1-14.
- Further upstream at Fitz's Boreen, a peak blockage of 80% was applied to the double-arch bridge just upstream of Dulux Paint Factory. This reflects the ease with which this structure can become blocked during flood events. Post-event photos provided by the Office of Public Works support the adoption of such a high value.
- An 80% blockage ratio was also applied to the upstream face of the North Point Business Park culvert. This was done to reflect the possible degree of obstruction witnessed post-event.
- Unfortunately, there is no flow record in the Bride, Glen or Glenamought catchments that could be used to give inputs to the models for the calibration exercise. All of the gauges in the catchment were installed at the commencement of this study. Therefore, the 2% AEP event (Q50) was used to provide a first estimate of the event inflow to the model. This choice was based on model runs with a range of return periods, as well as various hydrological work undertaken as part of the Glashaboy[9] FRS that estimate that the return period for the June 2012 event was approximately between the 2% AEP and 1% AEP magnitude.

[9] Glashaboy River FRAM Scheme. Final Hydrology Report. January 2015. ARUP-JBA Consulting

Table 3-3 Collection of images taken in Blackpool during and immediately after the June 2012 event

		
Looking d/s at North Point Business Park Culvert	Looking d/s at Fitz's Boreen	Looking d/s at Sunbeam Short Culvert
		
Looking d/s at Orchard Court Culvert inlet	Looking u/s at Orchard Court Footbridge	Looking d/s at Blackpool Church

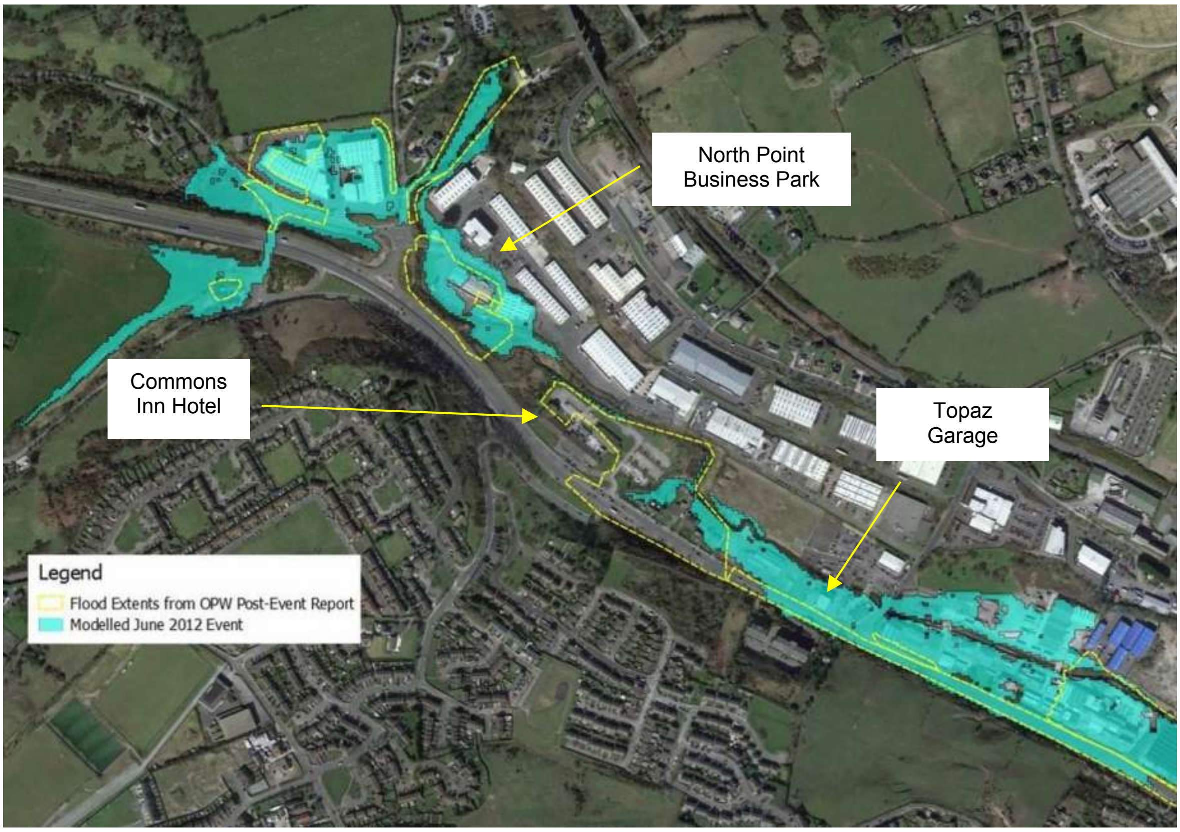
3.4.2 Comparison of results for June 2012 event

Upon comparison of the observed and modelled flood extents for the June 2012 event, it seems that there is generally a good agreement between both datasets.

At the North Point Business Park the extents can be seen in Figure 3-11. The hydraulic model shows that the right-bank upstream of Commons Inn overtops. However, it does not account for the overland flow route that resulted. This is evident from the disparity between the observed and modelled outlines at the Commons Inn. There are a number of possible explanations for this:

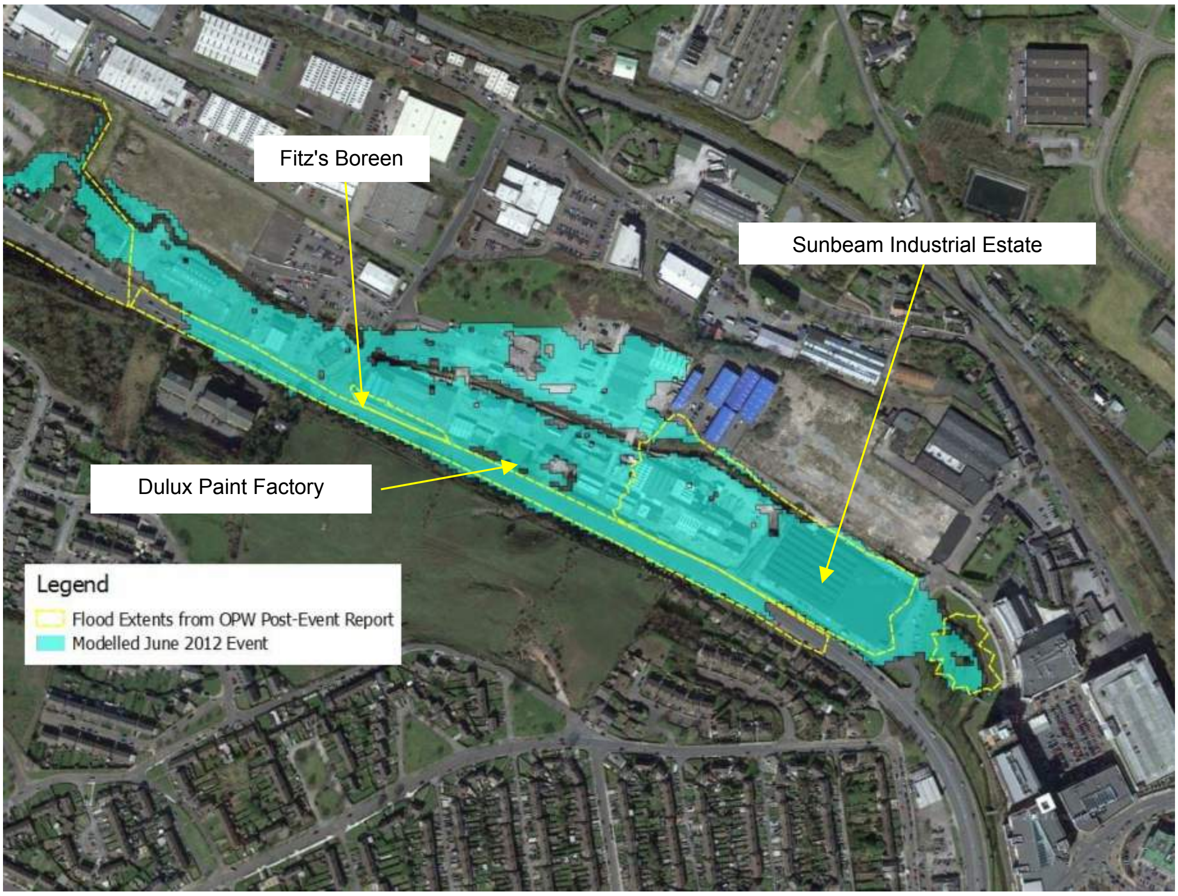
- The flow used in the hydraulic model is slightly less than the actual event hydrology. A larger flow would have generated the overland flow route, resulting in flooding of the Commons Inn and car-park.
- The bank elevations used to define the channel-floodplain boundary in the hydraulic model may not have included a specific low-point through which flow could make its way toward the Commons Inn.
- The LIDAR data used to define the model floodplain might not have picked up a specific low point through which flow could drive itself toward the hotel.
- A localised heavy debris load within the channel could have elevated levels to the necessary overtopping elevation.
- As well as the above, it is known that flooding in this part of Blackpool was exacerbated by a storage tank that overflowed on the other side of the Commons Road. This is likely to have also added to the flood extent at this point.

Figure 3-11 Comparison of observed and modelled June 2012 flood extents at the Commons Road



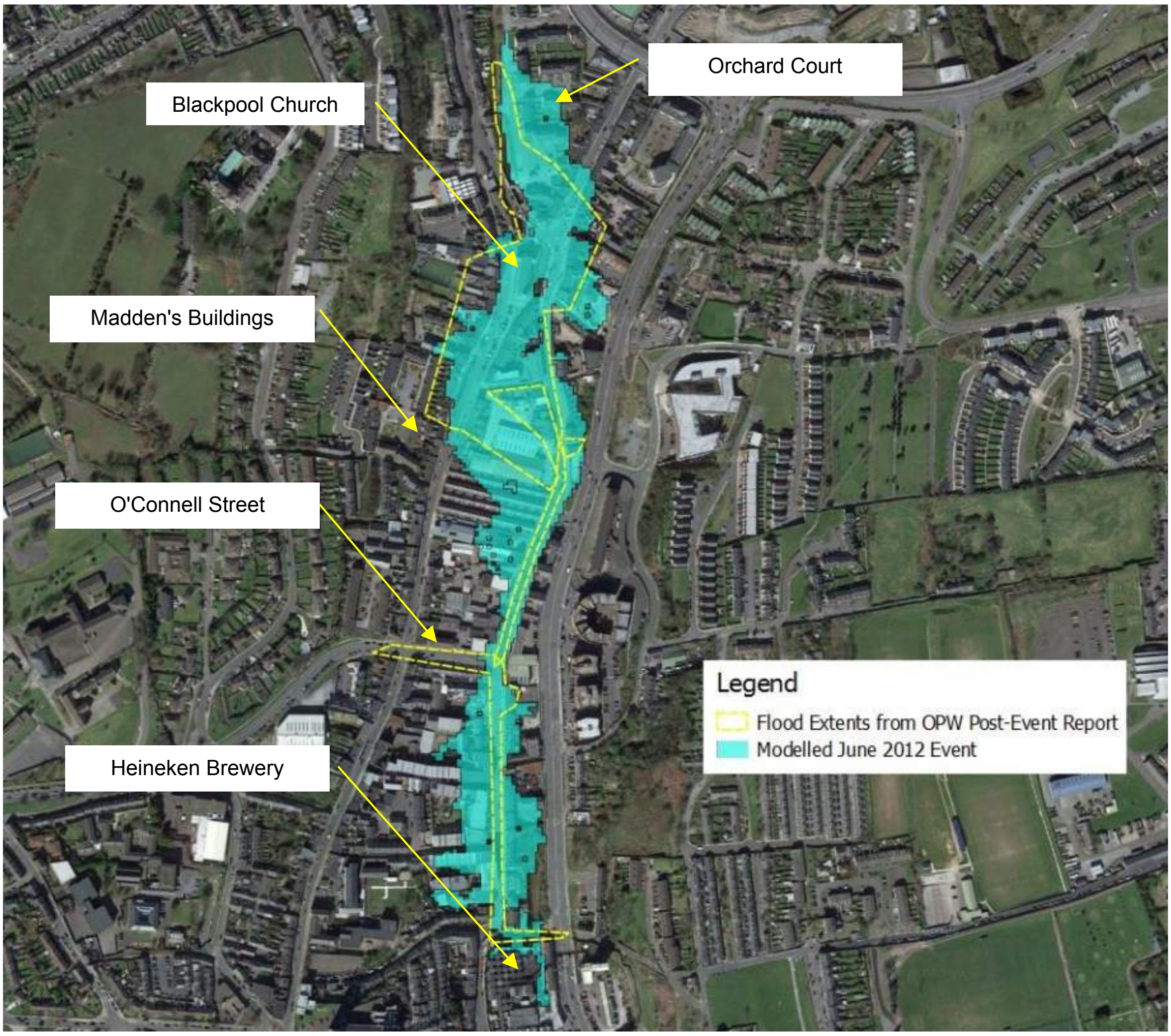
The modelled flood extents downstream, at the Dulux Paint Factory, are consistent with the observed event, as can be seen in Figure 3-12. The eastern portion of the factory and the Sunbeam Industrial Estate are particularly affected. Flood water is seen to overtop the right-bank and overwhelm the low-lying industrial units in the adjacent area. The flow route down the Commons Road is also clearly shown.

Figure 3-12 Comparison of observed and modelled June 2012 flood extents at the Dulux Paint Factory



The modelled flooding in the centre of Blackpool village also proves to be a good match with the observed flood extents, as can be seen in Figure 3-13. It is more difficult to replicate flooding in a dense urban area as it involves a complex floodplain with numerous complications such as surface water drainage systems and building thresholds. However, the model seems to be performing satisfactorily using the initial assumptions in its replication of extents. The observed flooding at O'Connell Street, near Maddens Buildings was caused by the surcharging of surface water systems and subsequently flowing towards the Watercourse Road. Thus, this has not been replicated using the hydraulic model, as shown in Figure 3-13.

Figure 3-13 Comparison of observed and modelled June 2012 flood extents in Blackpool village



The 'observed' flood extents are a combination of data collected both during and after the event which provides an indication as to the extent of the event experienced. The record does not provide coverage across the whole of the area impacted, but does include significant locations and extents, through which engineering judgement can be applied to extrapolate likely extents elsewhere.

To summarise, the flood thresholds (i.e. the flow rates that would overtop banks) at each major location in the system are as follows, using the June 2012 hydraulic model:

- 21.3 m^3/s at the Commons Inn;
- 12.2 m^3/s just upstream of the Topaz Garage
- 16.5 m^3/s at Orchard Court

To provide a greater understanding of the flow routes within the system, a series of time-lapse photos have been provided in Table 3-4 and Table 3-5, for Blackpool Village and Commons Road.

Table 3-4 Timelapse of June 2012 event as modelled at Blackpool Village

		
T = 8.5hrs	T = 9hrs	T = 9.5hrs
		
T = 10hrs	T = 10.5hrs	T = 11.5hrs

Table 3-5 Timelapse of June 2012 event as modelled at Commons Road

T = 5hrs

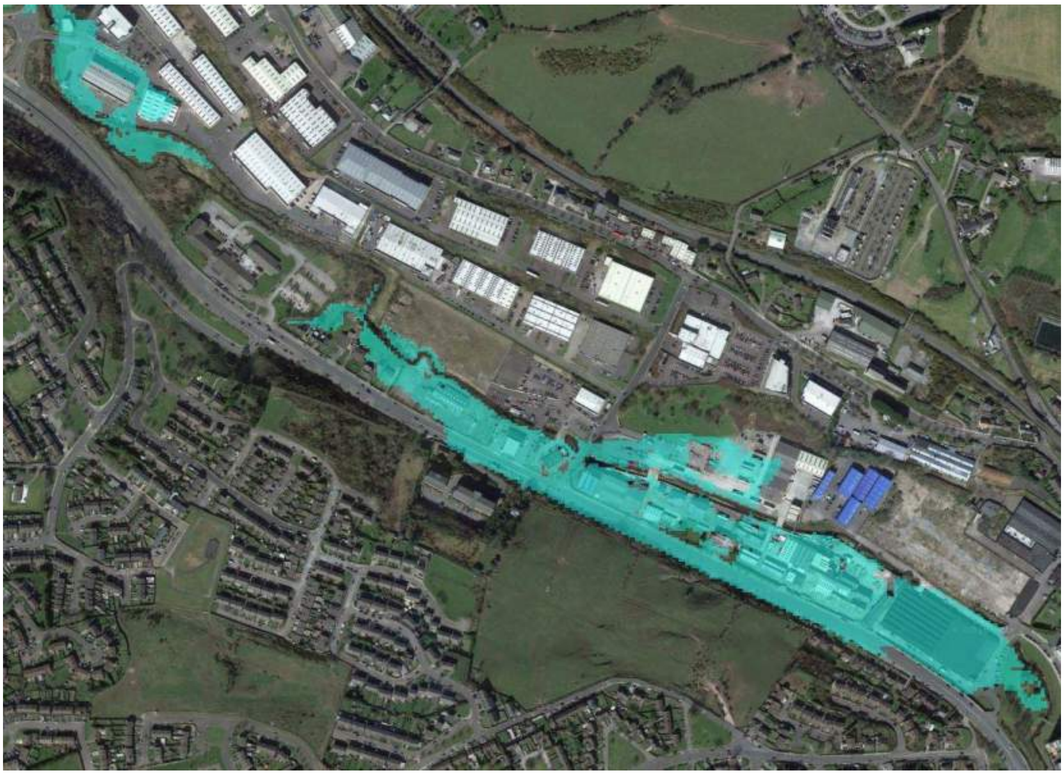
T=6.5hrs


T=7.5hrs

T=8hrs


T=8.5hrs

T=9hrs

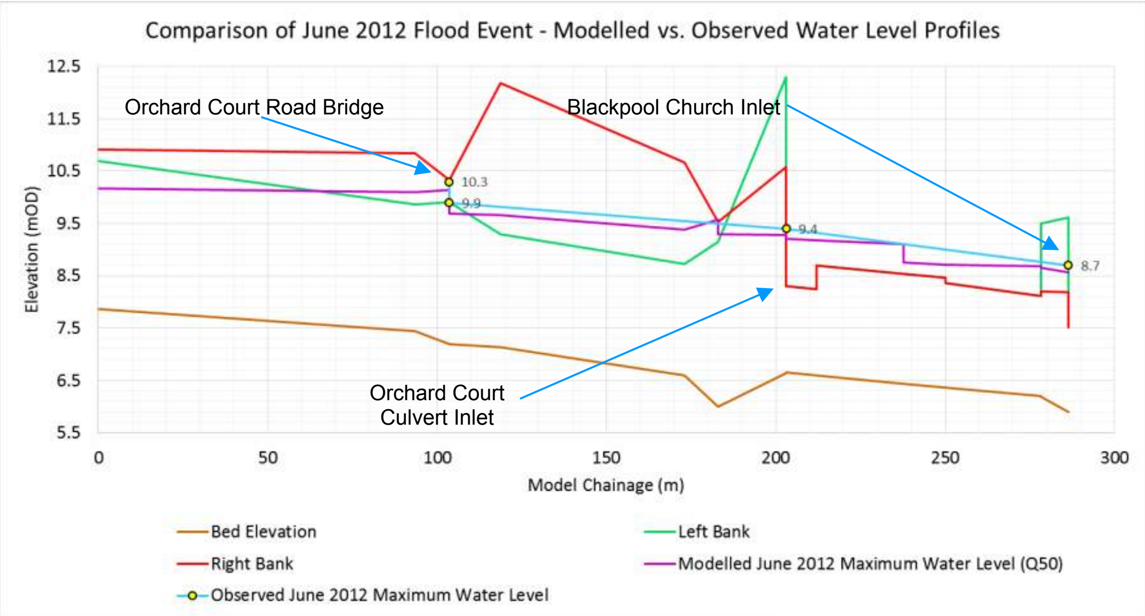


3.4.3 Comparison with OPW water level profile

In addition to the previously discussed extents, a water level profile was recorded by the OPW, which gives a record of levels in and near the channel through the study area. This is useful as it provides an indication of headlosses across structures, and can point to the impact of accumulations of debris. This water level profile has been compared against the modelled profile for Orchard Court and Blackpool Church in Figure 3-14. It can be clearly seen that out-of-bank flooding occurs. There is a relatively good agreement between the observed (blue line) and the modelled (purple line) profiles. The observed water level profile has been approximated using point/spot levels taken by OPW engineers at various critical structures. There is an approximate discrepancy of 200mm between each profile. This consistent difference could be explained by the following:

- The June 2012 event may well have been slightly greater that the adopted 2% AEP event estimate. Perhaps, this event was somewhere in between a Q50 and Q100 flow. This is the most likely explanation for the discrepancy as it is a consistent difference between levels. In a separate study by JBA, it has been found that the June 2012 event in the Glashaboy catchment was approximately a 1 in 90 year event[10].
- Another explanation for the difference is that the contribution from surface water runoff has been underestimated further upstream, particularly from the area around the Dulux Paint Factory. As the immediate environs are so urbanised in nature, there could be a collection of bespoke surface water management connections to the watercourse system that could not possibly be accounted for in the catchment hydrology.
- Screen performance during the event will have had a significant influence on modelled water levels and that would have brought levels in closer agreement in Orchard Court.

Figure 3-14 Comparison of observed and modelled June 2012 water level profiles at Orchard Court and Blackpool Church



Note: The observed water levels of 10.3mOD and 9.9mOD at the Orchard Court Roadbridge in Figure 3-14 refer to the upstream and downstream faces of the bridge respectively.

[10] Glashaboy River Flood Relief Scheme Hydrology Report. August 2014. JBA Consulting.

4 Model limitations

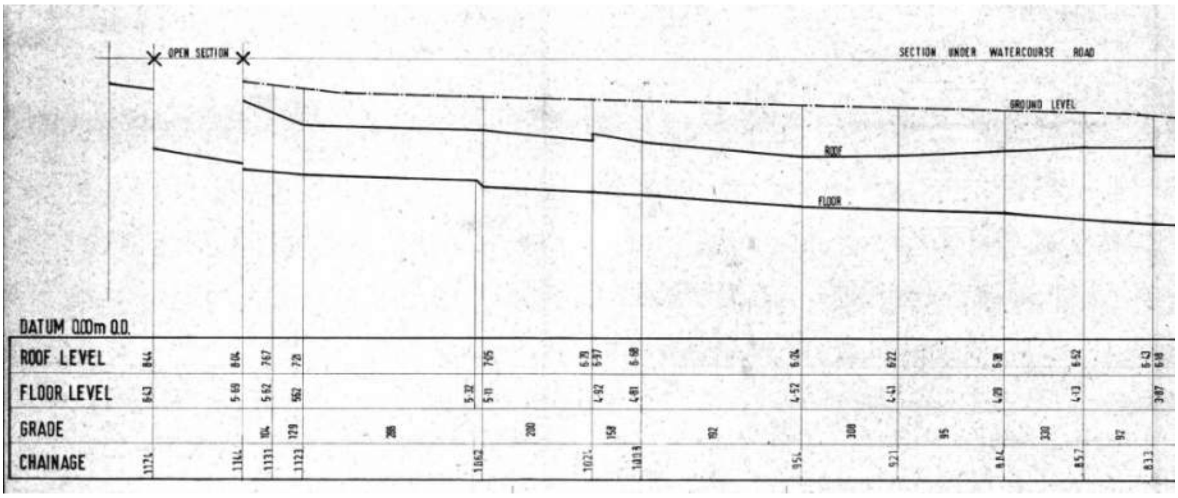
4.1 Surface water management system inflows

The contribution by the surface water management system into the Brewery Branch is an approximation. To give an accurate estimation, each inflow into the Brewery Branch could have been modelled using an appropriate sewer modelling system, however this was outside the scope of work for this project. The method adopted in this study is of sufficient detail to provide a broad picture of the quantity of surface water draining to the culvert and would not need to be of a more precise nature.

4.2 Culvert schematics

The culvert system in Blackpool village has been modelled using the 'as-built' drawings produced by Pettits for the GBK scheme. However, the drawings that have been supplied are fairly crude in nature. They do provide invert elevations at various chainages but do not provide coordinates or inflow locations. An example is shown in Figure 4-1. As well as this, subsequent changes to the conditions in the culvert such as sediment deposits have had to be assessed. Therefore, every effort has been made to assimilate the findings from the July 2012 CCTV survey into the model where at all possible. As well as this, a localised topographic survey of the Madden's Buildings junction has recently been completed as part of the flow monitoring contract, as mentioned in Section 2.7. The deliverables of this survey reinforced the assumption made of the invert level of the Madden's Buildings junction and the culvert floors meeting at that location.

Figure 4-1 Example of schematic drawing for Brewery Branch culvert



For the newer culvert branches such as the Phase 4 and 5 GBK reaches, the conduits were modelled using the as-built drawings. However, for the older branches a combination of both the 2012 CCTV survey and construction drawings were used.

4.3 Cross-section survey data

The cross-section survey data used for this study is the same dataset used for the Lee CFRAMS and dates to May 2007. There may have been changes to the channel geometry since this survey date that may not have been accounted for in the revised model such as channel alteration. Any known changes (primarily in the main risk areas) have been included through re-survey of the channel and structures.

4.4 Calibration data

A major limitation of this study has been the lack of historical flow data available within the catchment as a whole. There were no gauges on the Bride, Glen or Glenamought Rivers until the Office of Public Works installed the four new stations in July 2013. As the gauges become more established and build a longer record, they will become more valuable to future studies on the watercourse. Although there is a good record of the flood extents for the event of 2012, without being able to correlate this flood event to a specific flow, the level of calibration possible in the model is somewhat limited.

The flow monitoring contract deliverables, however, has helped hone in on the hydraulic nuances in the culvert system in Blackpool village.

5 Model results

5.1 Flood risk mapping

The suite of flood risk maps are provided in the Figures Section at the end of this report. The figures give flood risk extents for the 10%, 2%, 1% and 0.1% AEP events, in conjunction with the long section profiles extracted from the hydraulic model.

5.2 Key flood risk mechanisms

Further to the information presented in the flood risk maps, a brief description of the key flood risk sites and flooding mechanisms is provided below.

5.2.1 Flooding at Commons Inn and North Point Business Park

The hydraulic model suggests that flooding at the Commons Inn is caused by an upstream overland flow route. At the 0.5% AEP event, some flow can escape over the right-bank approximately 25m upstream of the hotel and travel downstream into the Commons Inn car-park. The large event threshold (0.5% AEP) suggests that a local low spot on the bank and/or increased complications associated with the series of channel turns downstream have not been included in the model. Flooding at the North Point Business Park is caused by flow overtopping the left-bank at return periods in excess of the 10% AEP event, particularly around the old bridge immediately upstream (7BR1_91) and flowing in the main entrance of the park. At the higher return periods, the entire business park would be inundated as the ground is reasonably flat.

5.2.2 Flooding at Fitz's Boreen

Elevated water levels caused by the restrictive capacity of the Fitz's Boreen arch bridge results in the flooding of a collection of properties, as well as the N20. The arch bridge will also overtop, depending on the magnitude of flow and degree of blockage on the upstream face. The hydraulic model shows that flooding on the N20 results from flow overtopping the right-bank upstream of the Topaz garage before travelling overland. It has been decided that the baseline hydraulic model will assume that there are no obstructions at culverts or bridges. Based on this assumption, a flow rate of approximately 18 m^3/s at Fitz's Boreen arch bridge causes overtopping of the right bank upstream of Topaz.

5.2.3 Flooding at Dulux Paint Factory

The hydraulic model confirms the flood risk at the Dulux Paint Factory. The low-lying right-bank is particularly vulnerable, with flood waters able to pond as the factory premises is reasonably flat. The Sunbeam Industrial Estate and West Link Business Park are also at risk of flooding. The threshold for out-of-bank flow is between the 10% and 5% AEP events. Approximately 12 separate buildings are affected by flooding in the 1% AEP event at this point in the system.

5.2.4 Flooding at Blackpool Shopping Centre

For the most part, Blackpool Shopping Centre avoids any incidence of inundation during the design event. Flooding is primarily confined to the wetland area at the upstream extent of the site. However, in the 1% AEP event, flooding is observed at Heron Gate, a constituent of the Blackpool Retail Park, adjacent to the N20. The hydraulic model predicts that the depth of flooding would be less than 120mm in the ground-floor units. Channel capacity here is approximately 29.5 m^3/s. Flooding at this location is caused by insufficient channel capacity rather than undersized structures downstream.

5.2.5 Flooding at Orchard Court

The threshold of flooding at Orchard Court is somewhere between 20% and 10% AEP events. The flow at the Orchard Court culvert inlet when this overtopping occurs is approximately 20m^3/s. The low left-bank level along the length of the open channel is the primary cause of flooding. However, the road bridge and footbridge, whose respective soffits are quite low relative to the river-banks, also exacerbate water levels by causing headlosses. Future alleviation works should consider the removal of the footbridge, as well as the possible raising of the road bridge's soffit. The artificially lowered entrance soffit at the Orchard Court culvert should also be addressed in future works. This will be discussed further in Section 7.3.6. As flood waters overtop the left-bank, flow collects in Orchard Court, moves through Wherland's Lane and down Thomas Davis Street.

5.2.6 Flooding in Blackpool Village

Flooding in the Blackpool Church area, in the centre of the village, can be attributed to some water escaping through the railings at the open channel section during high-order return periods and combining with a larger overland flow route from Orchard Court. Flow then travels down the Watercourse Road and Great William O'Brien Street. These separate flow routes then merge in the T&A Buildings Supplies premises, where the flood waters continue as far downstream as the Heineken Brewery. Flow re-enters the system at a section of open channel at Heineken Brewery and through the surface drainage system.

The open channel section at Blackpool Church will take some of the flow coming overland from Orchard Court at the start of the event. However, as soon as the channel capacity of 25 m^3/s is exceeded, water will actually spill out from the channel, exacerbating the flooding from Orchard Court and Thomas Davis Street.

5.3 Discussion of model findings

From the various model runs undertaken for this study, the following conclusions can be drawn about the Blackpool system of watercourses and culverts, in its current configuration:

- The river channel at North Point Business Park, Dulux Paint Factory and Orchard Court is not of sufficient capacity to contain flood flows. The flooding witnessed in June 2012 reinforces these mechanisms identified in the modelling.
- The culvert system at Orchard Court and Blackpool Church is hydraulically inefficient for large flood events i.e. in excess of 5% AEP. Multiple culvert sections, a potential for debris accumulation and a number of storm water connections compounds the problem.
- The capacity of the channel at Orchard Court is approximately 19.7 m^3/s.
- The threshold of flooding at Orchard Court is approximately the 10% AEP event.
- A particular problem is the size of the Blackpool Church Culvert (4.8m x 1.6m). Any proposed alleviation option that aims to force the total design 1% AEP flow down this conduit will generate significant elevated water levels as a result; requiring careful detailing and complimentary measures to manage local surface water runoff.
- The current hydraulic model used to derive the study flood extents has a flow split of approximately 60:40 at the bifurcation during the design event i.e. 40% of the flow in the Blackpool Church culvert continues straight into the Brewery Branch conduit. The remainder flows across the junction into the head of the Phase 3 GBK leg. Thus, any proposed flood alleviation option will have to carefully consider how the Brewery Branch is utilised i.e. if can handle the flows required of it.
- The model results are very different to those derived using the simpler Lee CFRAMS model, as shown in Table 5-1. It can be seen that the Lee CFRAMS represents a gross under-estimation of flooding in Blackpool village relative to this study. This can be attributed to the lack of structure detail included in the CFRAM model and a lack of understanding of culvert hydraulics, highlighted in Section 1.8.

Table 5-1 Comparison of model results with Lee CFRAMS at key locations in the watercourse system

Cross-Section Label	Description of Location	Lee CFRAMS Max 1% AEP WL (mOD)	Lower Lee FRS Max 1% AEP WL (mOD)
7BR1_91	At Kilnap Glen House access bridge	25.48	25.50
7BR1_0	At inlet to the North Point Business Park culvert	25.36	25.13
7BR2_0U	At GBK Phase 4 inlet on Spring Lane	15.24	14.79
7BRI_2053	Just upstream of Commons Inn Hotel	21.00	21.20
7BRI_1598	Approximately 81m upstream of Topaz Garage	18.07	18.82
7BRI_1425	Upstream face of Fitz's Boreen arch bridge	17.21	18.40
7BRI_1175	Upstream face of access bridge in Dulux	14.99	15.63
7BRI_711	Heron Gate - Blackpool Retail Park	12.69	13.15
7BRI_197	Approximately 8m upstream of Orchard Court road-bridge	9.03	10.12
7BRI_93	Upstream face of Orchard Court culvert inlet	7.80	9.32
7BRI_00	Upstream face of Blackpool Church culvert	7.63	8.78

The above conclusions require a significant flood alleviation solution. If one were to rely on the installation of walls in all 'at risk' locations, the existing culvert system would need to be heavily pressurised and wall heights would be unfeasibly high. All connections to the system would have to have non-return valves and some sections of culvert would have to be replaced. As well as this, a number of existing structures such as the Fitz's Boreen Bridge, the culvert under the North Point Business Park and the Orchard Court roadbridge would all need replacing as their restrictive capacity would cause water levels to back-up and the associated flood walls would be unfeasibly high. These issues will be explored in further detail in Section 7.

6 Sensitivity testing of existing system

6.1 Overview of sensitivity testing

To test the robustness of the hydraulic model 'existing scenario' predictions, sensitivity analyses were carried out on a number of key model parameters. These sensitivity tests are effective in identifying areas for further research and establishing freeboards for proposed defences.

The sensitivity of the fluvial model to the following parameters was analysed:

- Peak flow
- Afflux at selected structures
- Increasing channel and floodplain roughness
- Decreasing channel and floodplain roughness
- Building representation
- Model cell size

6.1.1 Peak flow

As flow is probably the most critical of all the sensitivity tests it is important to consider the quality of data available in the derivation of the design flows. As the flows were derived using a bespoke methodology and gauged records, it would not necessarily be correct to apply the Standard Factorial Error for the FSU. The approach used to derive the uncertainty in flow and, subsequently, the sensitivity of the model to flow has been described in Section 2.3.3. In summary, a flow sensitivity percentage of 18% was calculated by assessing the uncertainty in both the calculation of Qmed and the study growth curve.

The results of this sensitivity test are presented in Appendix A.3.1.

The sensitivity testing of peak flow shows that new areas of flood risk are introduced due to the increases in flow being conveyed. Blackpool Retail Park and Shopping Centre are amongst areas previously unaffected in the 1% AEP, as are some properties adjacent to the Madden's Buildings junction.

The sensitivity testing also shows a general 150-200mm increase in flood depths throughout the river system. This figure increases to approximately 500mm in the Blackpool Shopping Centre car-park, where water can pond against the N20 embankment and boundary walls. The Heron Gate building in the Blackpool Retail Park also experiences similar depths; 520mm is approximately the largest depth in the building footprint.

6.1.2 Afflux at notable structures

General modelling units and parameters can often not fully represent the head loss which can occur at atypical or complex structures. Key structures identified for this sensitivity test are those that have a controlling influence on local water levels and the resulting influence may be expected to cause flooding to local receptors. The following sections will briefly outline the tests undertaken.

Blackpool Church Culvert

As previously discussed, the Blackpool Church Culvert is a restrictive structure. In the baseline model, the headloss coefficient for inlet controlled flow has been set at 0.55 (K value). This was increased from the ISIS default value of 0.515 (rectangular conduit, 20mm chamfers) using the results of the flow monitoring contract (i.e. the calibration process changed the value to 0.55). To investigate the effect of an underestimation of the structure's influence on upstream water levels, the headloss coefficient was increased to a considerable value of 0.7. This is slightly larger than a value recommended for an 'abrupt contraction' in modelling literature.

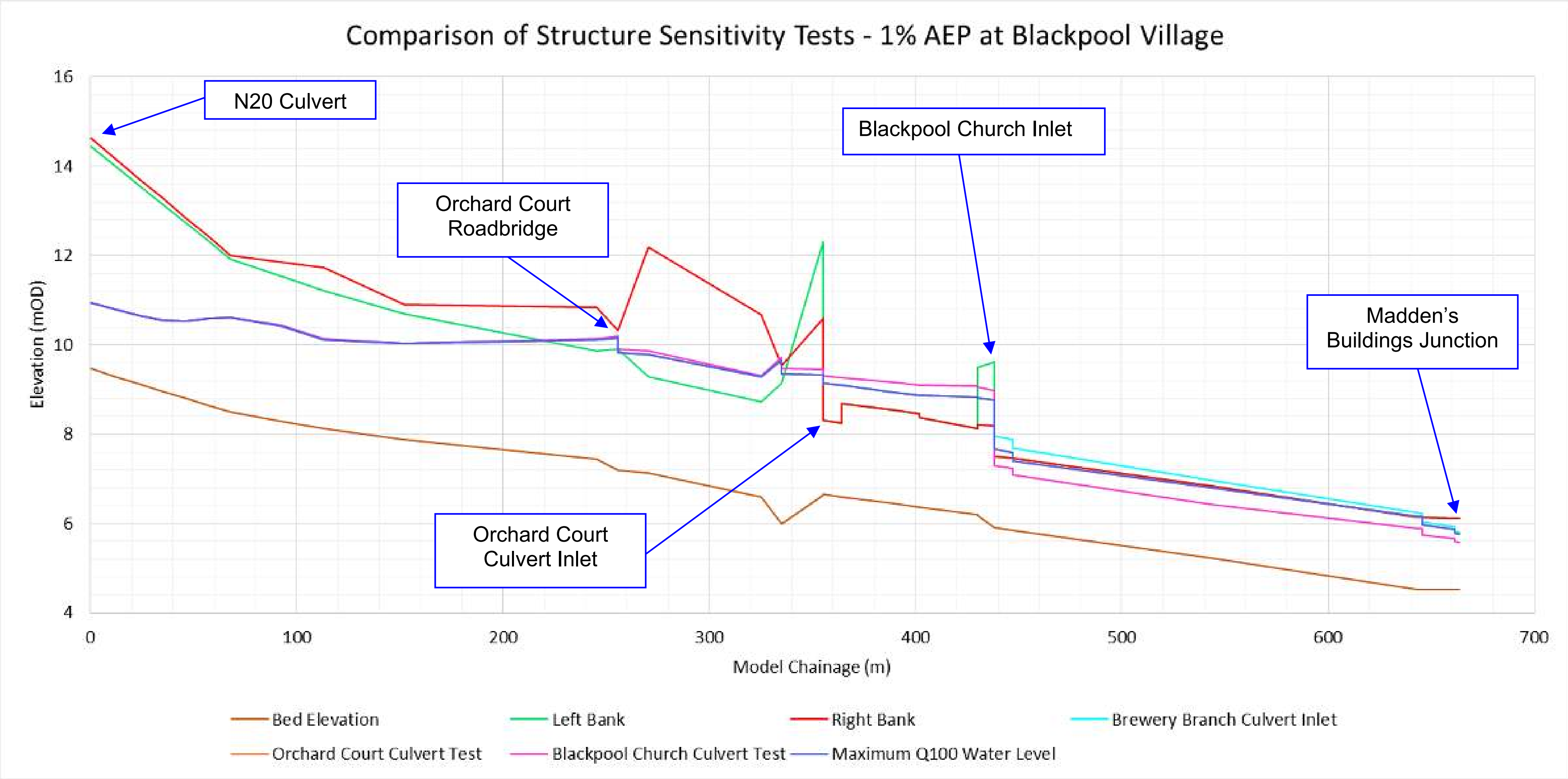
Orchard Court Culvert

The Orchard Court culvert inlet is not a complicated transition for a watercourse; the primary issue with this inlet is its lack of capacity. The opening is perpendicular to the flow of the river; with a slight directional change just inside the inlet. To test the influence of this inlet on upstream water levels, the default K value of 0.515 has been increased slightly to 0.55, which is in excess of the typical culvert contraction value of 0.44 quoted in the CFRAM guidance.

Brewery Branch Culvert

The Brewery Branch culvert, as previously mentioned, has the tendency to accumulate debris at the head of the conduit. At present, starter bars for reinforcement (remnants of the old diversion block) are still fixed to the floor of the inlet. In the baseline model, this impediment is included by means of a 'jagged spill'. However, the spill coefficient is set at 1.7 i.e. as efficient as possible. To investigate any possible underestimation of its influence, the spill coefficient has been changed to 1.4 i.e. it is more difficult for flow to enter the culvert.

Figure 6-1 Water level profile for each structure sensitivity test for 1% AEP event at Blackpool Village



The sensitivity testing shows that the hydraulic model is most sensitive to changes at the Blackpool Church culvert inlet, as shown in Figure 6-1. The increased model coefficient serves to influence water levels as far upstream as Orchard Court road-bridge. As a result, water levels inside the Blackpool Church culvert, and further downstream, reduce. The other structure tests have negligible impact on modelled water levels and extents.

6.1.3 Sensitivity to roughness

The large flood extents in the existing-risk design events mean there is benefit to testing the sensitivity of the model results to both an increase and reduction in floodplain roughness (NTF) values. The sensitivity to both lower and upper bound roughness values for the 1% AEP event, as shown in Table 6-1, has been tested.

Table 6-1 Typical roughness bounds for channel and river banks

Channel Description	Bank Description	Existing Risk	Upper Bound	Lower Bound
Clean, straight, full stage, no rifts or deep pools	-	0.03	0.04	0.02
As above but more stones and weeds	-	0.035	0.05	0.025
Clean, winding, some pools and riffles	Scrub/Long grass	0.04	0.055	0.03
As above but some weeds and stones	-	0.045	0.06	0.035
As above but more stones	-	0.05	0.065	0.04
As above with more pools	-	0.055	0.07	0.045
Sluggish reaches. Weedy deep pools	Trees - flood level not reaching branches	0.08	0.12	0.065
Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	Trees - flood level reaching branches	0.1	0.15	0.08

The results of the sensitivity test are presented in Appendix A.3.3. The results show that the model is particularly sensitive to increasing channel roughness adjacent to the Blackpool Shopping Centre. It introduces the Heron Gate complex to the extent; however predicted depths are less than 50mm. Similar increases are experienced further upstream along the Commons Road.

6.1.4 Building representation

The representation of buildings in the floodplain can have a significant effect on modelled flood extents. For the purposes of this study, buildings have been represented as polygons in the floodplain whose thresholds have been set to the mean-LIDAR level under the building. The polygons have also been assigned an increased roughness value to make it more difficult for water to flow through the buildings rather than around them.

To test building representation, each building level was increased by 300mm i.e. a threshold of 300mm was applied. This made flow through buildings more difficult and generally reduced flood depths downstream.

The results of the sensitivity test are presented in Appendix A.3.4. The test shows that the model is particularly sensitive at Madden's Buildings junction. An increase in building thresholds makes it more difficult for flow to travel downstream. However, an examination of the thresholds of the buildings immediately upstream shows the majority of them are flush to the footpath level. Therefore, adopting this building representation approach may not be appropriate.

6.1.5 Model cell size

The model cell size is the resolution of the digital terrain model (DTM) used to propagate water in the 2D (TUFLOW) floodplain. The smaller the cell size, the more detailed the hydraulic model will be. A smaller cell size can help to replicate intricate flow paths that may not necessarily be picked-up in models with coarser resolutions. For the baseline model, the cell size is 4m. This size was chosen to help with model stability and run times. However, the model is capable of running at a 2m resolution for some of the present-day scenarios, including the 1% AEP event. Therefore, this cell size was used to test the model sensitivity to cell size, as shown in Appendix A.3.5, using the 1% AEP flow.

The results show that a number of properties on Brocklesby Street are introduced to the flood extent in Blackpool village. Depths vary greatly within these properties, between 200mm and 20mm. Flooding at this location is a combination of flood waters from Orchard Court and the open channel section at Blackpool Church.

The reduced cell size also introduces a new flow route at Heron Gate, in Blackpool Retail Park. However, depths within the building footprint are quite shallow (less than 50mm).

The 2m cell size has some effect further upstream on the Commons Road, but primarily serves to reduce flood extents rather than introduce new areas of flood risk. There is a slight increase in extent on the left bank in the Dulux factory but it does not affect any properties.

6.2 Sensitivity testing results and uncertainty

The sensitivity testing presented in the previous sections shows that there are a number of locations in the model that are sensitive to changing of the parameters. In particular, the Blackpool Shopping Centre and Retail Park could be acutely affected by flooding if conditions within the channel were to worsen or an unprecedented flow were to occur. The results of the sensitivity tests were used to inform the uncertainty bounds as detailed in Figure 6-2, Figure 6-3, Figure 6-4 and Figure 6-5 for the 1% AEP event. The uncertainty bounds are a 'merged' extent of all the sensitivity results.

No changes were made to the hydraulic model based on the results of the sensitivity testing carried out.

Figure 6-2 Overview of sensitivity results and uncertainty bounds - Madden's Buildings

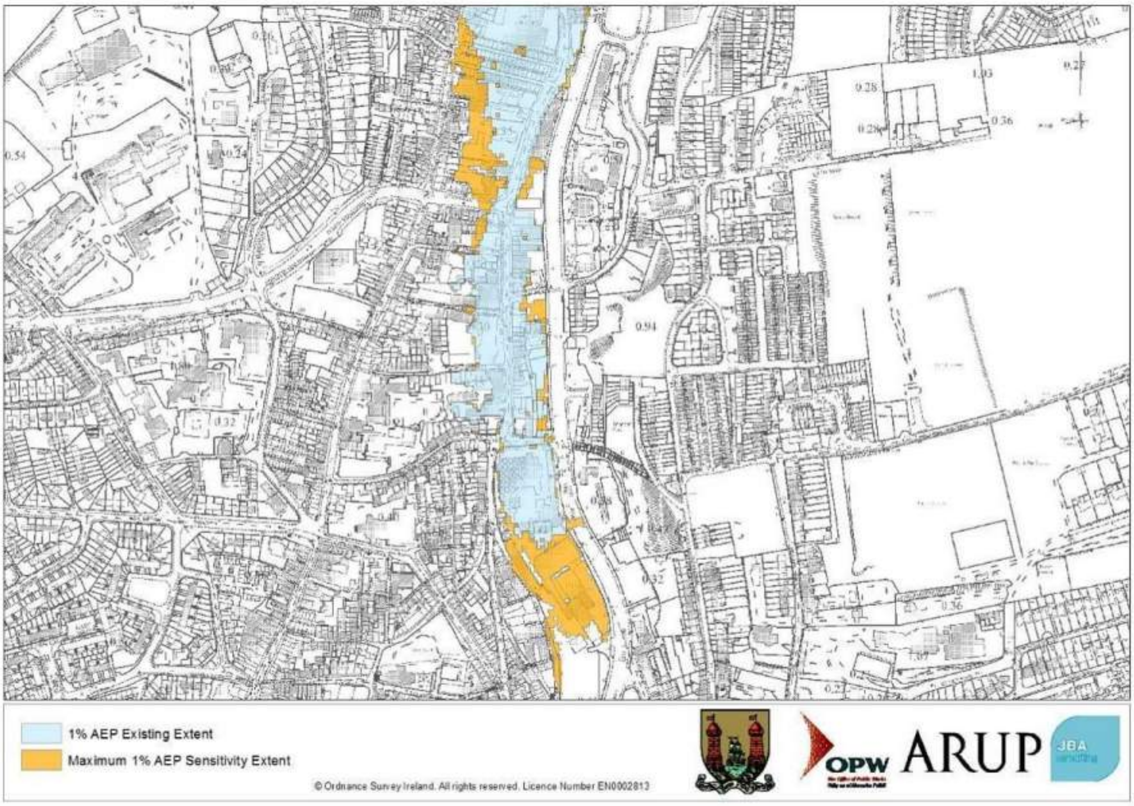


Figure 6-3 Overview of sensitivity results and uncertainty bounds - Blackpool Village

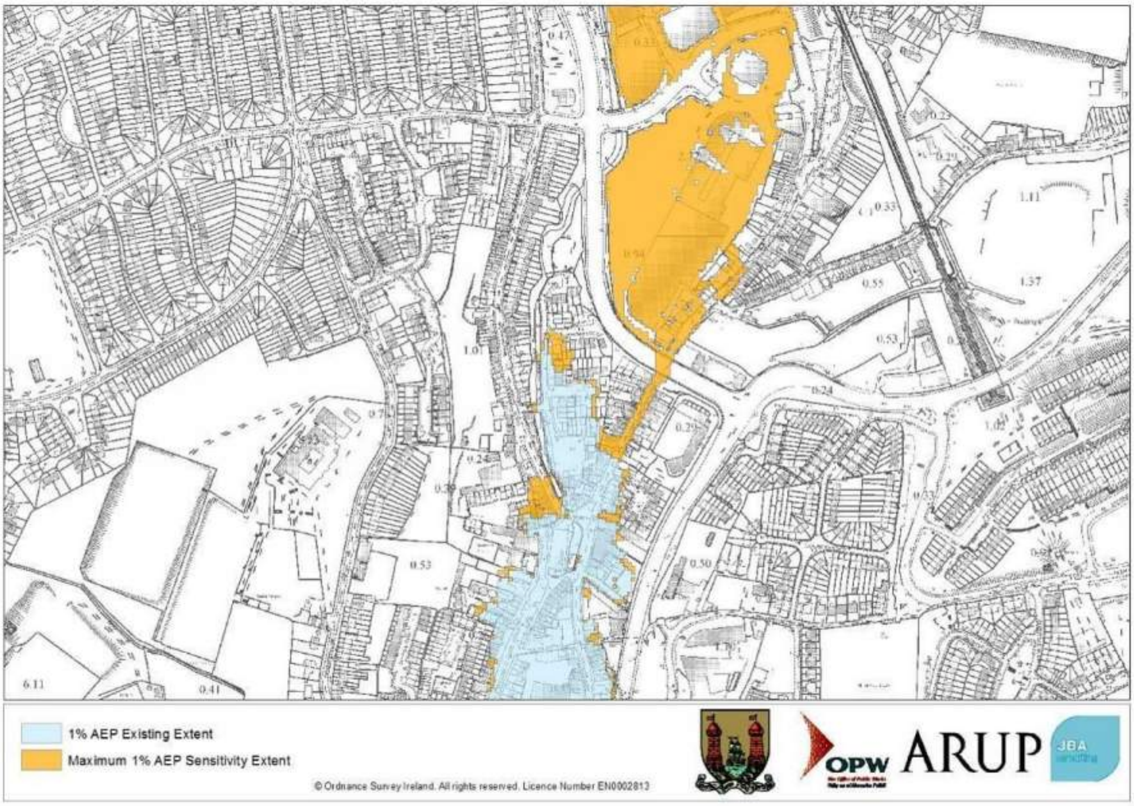


Figure 6-4 Overview of sensitivity results and uncertainty bounds - Commons Road

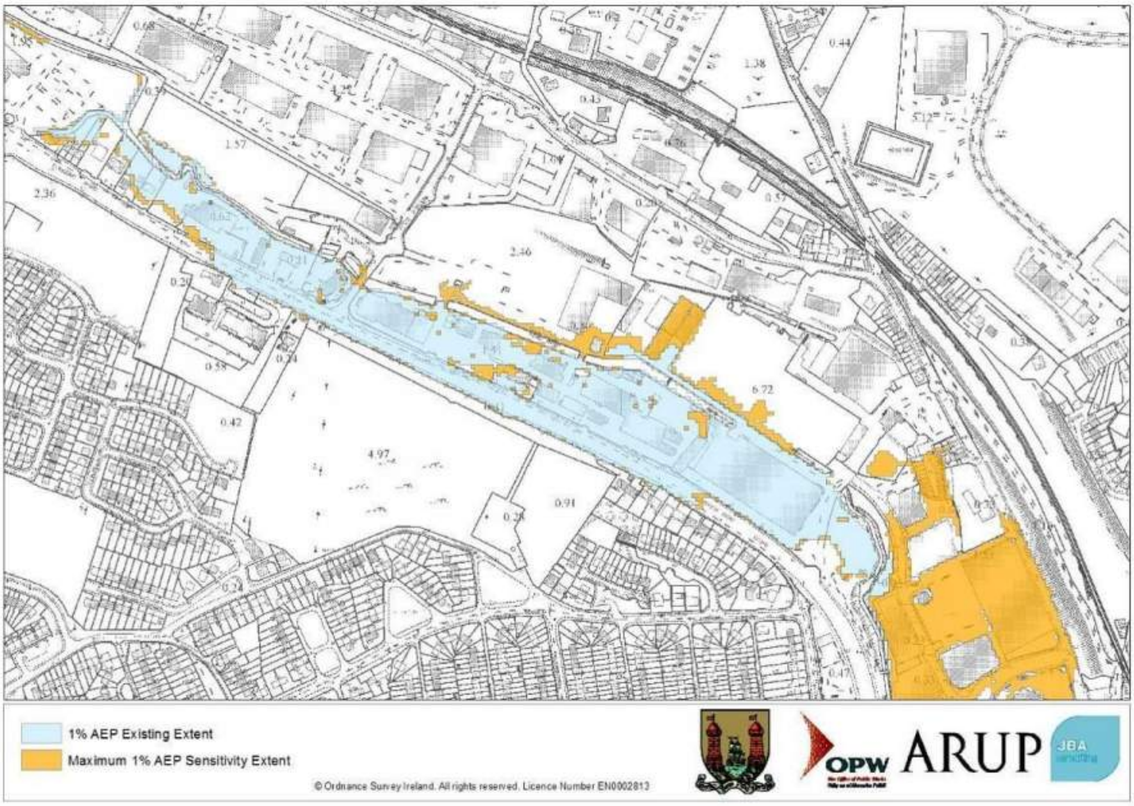
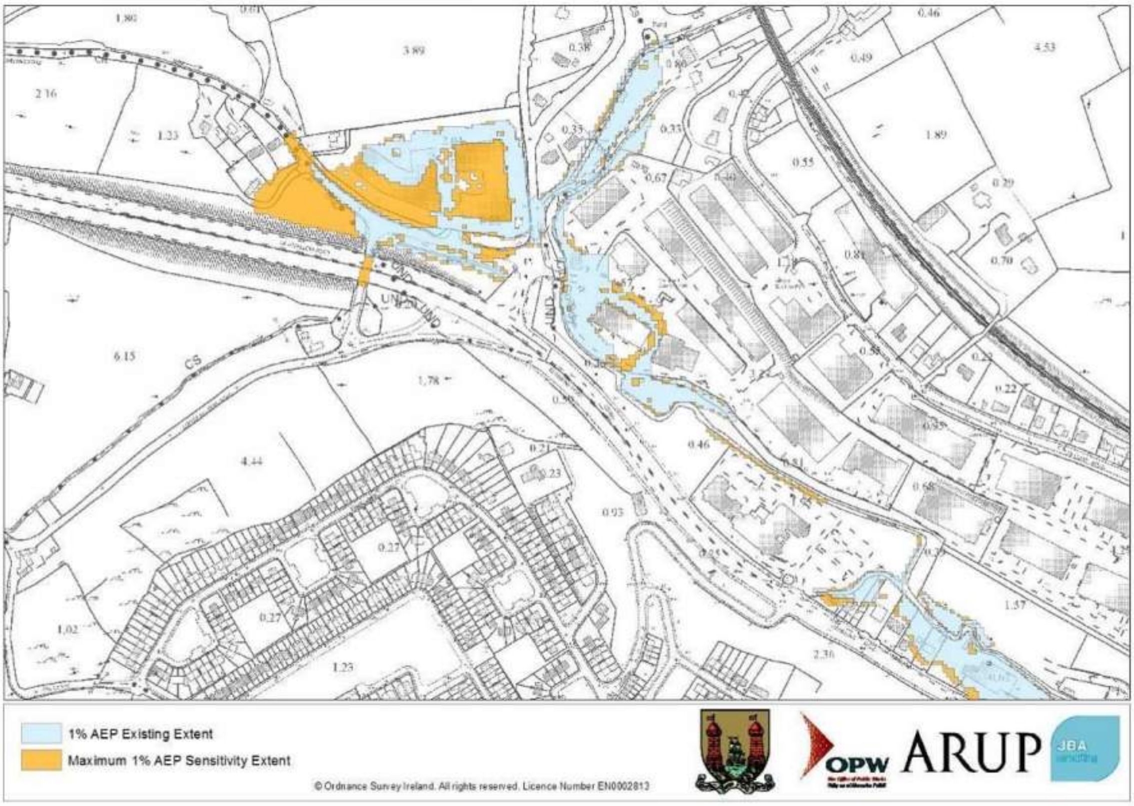


Figure 6-5 Overview of sensitivity results and uncertainty bounds - North Point Business Park



7 Flood alleviation options

7.1 Overview

This section sets out a summary of potential flood alleviation options (Section 7.5), before providing details of each. As highlighted in Section 5.3, the current Blackpool watercourse system requires a significant alleviation solution. The primary causes of flooding in Blackpool village and the Commons Road are summarised below:

- The hydraulics of the culverted sections of the lower reach of the River Bride control flood levels and is hydraulically inefficient for large flood events i.e. in excess of 5% AEP. Multiple culvert sections, a potential for debris accumulation and relatively small channel width compound the problem.
- The river channel at North Point Business Park, Dulux Paint Factory and Orchard Court is not of sufficient capacity to contain flood flows. In some instances, as is the case at the North Point Business Park, it is a restrictive structure immediately downstream that is the root cause of the problem.

The following measures were considered:

1. Reduce flow at source, with particular focus on the application of SuDS techniques from all new development in the Bride Catchment
2. Storage of runoff in the upper tributaries and/or in the Bride in order to reduce flows that have to be managed within the Blackpool system
3. Removal, wherever possible, of restrictions in the watercourse
4. Reduction of sediment load and settlement zones in culverts
5. Upsizing of the Blackpool Church Culvert and improvement of its inlet, as well as the connection to the Brewery Branch
6. Upsizing of the Orchard Court Culvert inlet to improve conveyance
7. Increase capacity by pressurisation of the existing system and by construction of high walls or upstream culverts to increase the head.
8. Climate change adaptation is significantly restricted with further culverting, although storage could be provided upstream as an adaptive measure

A full suite of measures has been tested to inform the emerging preferred option. The measures used in different combinations, that informed the development of options, are summarised in Section 7.2, 7.3 and 7.4.

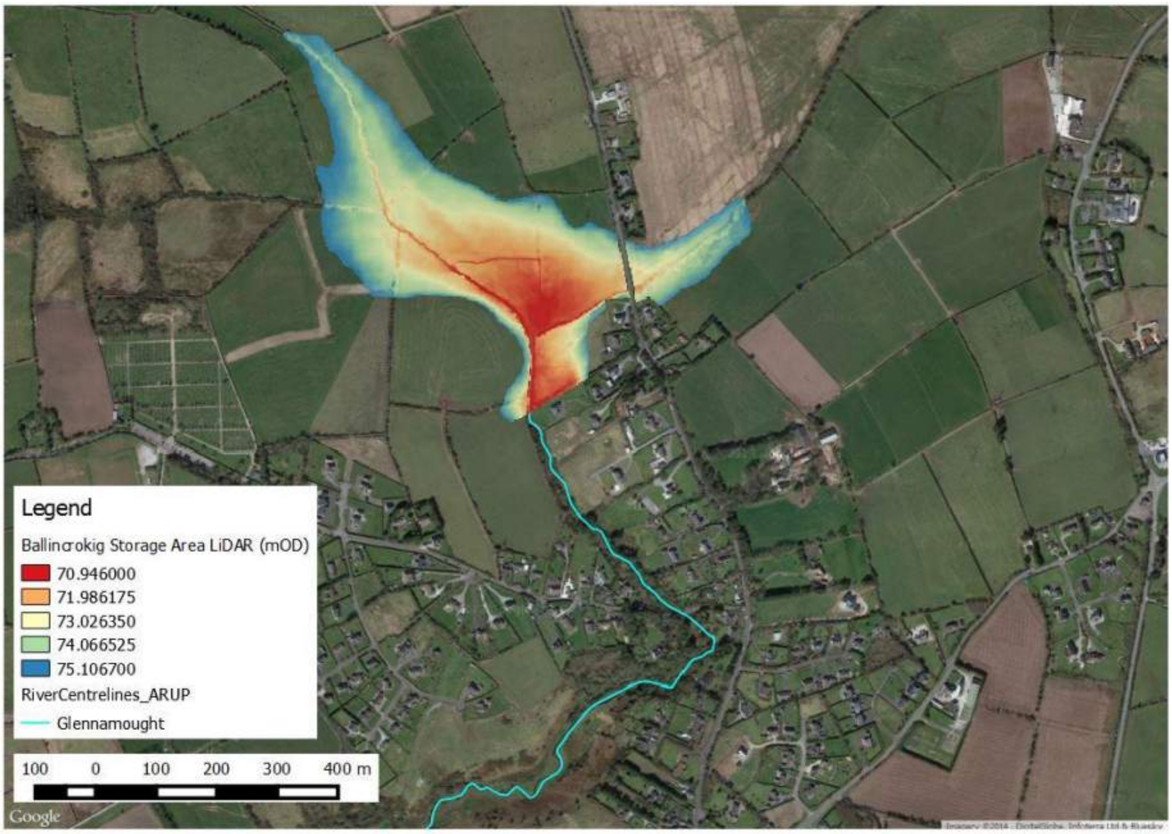
7.2 Flow reduction measures

Flow reduction measures would be a preferred approach as it eases the burden placed on the culvert system downstream in Blackpool village and limits invasive construction work in a heavily-urbanised location.

7.2.1 Upstream storage

The upstream storage measure involves the installation of a reservoir at Ballincrokig. It would utilise the existing site topography, with a new 3.5m high wall/embankment and hydrobrake unit to control flow draining downstream to Blackpool village. A hydrobrake allows a varying quantity of flow to pass through it, depending upon the head of water behind it (i.e. in this case, within the reservoir). A hydrobrake unit has a maximum discharge value based upon a certain upstream stage. The flow in a hydrobrake cannot exceed its maximum output flow. A generic hydrobrake stage-discharge relationship was used for the purposes of the options modelling. However, if upstream storage emerges as the preferred scheme option, this unit could be refined for optimal performance. The reservoir would cover an area of approximately 146028m^2. The potential storage area and existing site topography are shown in Figure 7-1.

Figure 7-1 Overview of proposed upstream storage area at Ballincrokig



7.3 Flow conveyance measures

Flow conveyance measures are a means to reduce water levels as much as possible without necessarily reducing the design flows.

7.3.1 Channel cleaning and maintenance

It is recommended that channel cleaning and maintenance is included as part of any alleviation or relief works in Blackpool. As will be shown in the following sections, the River Bride is prone to debris-accumulation from the North Point Business Park down through Blackpool village. The problem is at its worst along Orchard Court and within the conduits immediately downstream. In the course of a flood event, floating vegetation gets washed down the system from upstream areas and accumulates at hydraulically sensitive locations. The problem is exacerbated by illegal dumping in the channel, introducing items such as mattresses, bicycles, fridges and clothing. Unfortunately, an open watercourse in such an urban environment is particularly vulnerable to this problem.

A proactive channel maintenance program would be effective in combating the issues highlighted above. In the immediate future, it would be recommended that the channel is cleaned at known trouble-spots. Thereafter, regular spot-checks could be made to maintain the watercourse. It may also be of benefit to educate local residents as to the importance of a clean channel and its benefits in flood prevention. Residents should also become more vigilant as to the detection and reporting of illegal dumping.

7.3.2 Management of sediment

A problem that was regularly encountered during the flow monitoring contract was the accumulation of stone, silt and sand at hydraulically sensitive locations and monitoring points. It quickly became apparent during the lifetime of the project that sediment management and sediment control on the Bride would be an ongoing issue. If left unchecked, sediment has the ability to collect and accumulate at culvert inlets, bridge piers, channel bends etc. This increases water levels locally and can cause a significant headloss in lower-order flows. Figure 7-2 shows some of the sediment removed from the Orchard Court culvert during cleaning works undertaken in June 2014 by Cork City Council.

It is recommended that any future alleviation works would incorporate a sediment management plan in its delivery. This could be done by constructing a sediment trap further upstream where river velocities experience a sudden slowing, allowing sediment and other material to settle out of suspension. A means by which this could be achieved could be widening the channel section at the downstream end of the Dulux Paint Factory. The open ground opposite the Sunbeam Industrial Estate could be used as an access route for excavators to regularly maintain and empty the sedimentation area.

Figure 7-2 The open channel section at Blackpool Church during Cork City Council cleaning works - June 2014

7.3.3 Replacement of North Point Business Park Culvert

It was found throughout the course of modelling that the North Point Business Park culvert is a notable impediment to flow on the Glenamought River, just upstream of the confluence with the Bride. To improve channel conveyance, it is proposed to replace the existing three circular conduits under the North Point Business Park entrance with a new rectangular culvert. In the hydraulic model, this new culvert has been represented using a 7m x 1.5m opening; a similar size to the existing channel dimensions. However, the ability to accumulate debris should be greatly diminished. The structure to be replaced is shown in Figure 7-3.

Figure 7-3 Existing condition of North Point Business Park Culvert

7.3.4 Replacement of Fitz's Boreen Arch Bridge

Another obstruction to flow on the Blackpool system is the Fitz's Boreen Arch Bridge, on the Commons Road. It has two arched openings that are quite small relative to the channel width. It is also a location where debris travelling from upstream areas tends to accumulate. It is proposed to replace this structure with a new a rectangular culvert, approximately 6m x 1.8m. As before, these dimensions are in-keeping with the existing channel size. The existing structure is shown in Figure 7-4.

Figure 7-4 Fitz's Boreen Arch Bridge after June 2012 flood event

7.3.5 Removal of existing footbridge at Orchard Court

The footbridge at Orchard Court has a troublesome trash screen attached to its upstream face that requires constant cleaning by excavator in times of high flows. There is already a means by which pedestrians can enter the estate via Thomas Davis Street; therefore the bridge's removal is not foreseen to be an inconvenience. The footbridge in the 'Do Minimum' scenario generates a 300mm headloss.

7.3.6 Modification of the existing Orchard Court Inlet

The Orchard Court culvert, as surveyed in July 2012, is 4.8m by 2.1m in size. This is consistent with the construction drawings for the GBK scheme. Upon further investigation, the project team found that the culvert inlet soffit was approximately 0.45m lower than the soffit of the conduit inside it (reduces culvert height to 1.6m). This was due to a rolled steel joist and hollow-core concrete slab on the underside of the inlet, as shown in Figure 7-5. This reduction in inlet height resulted in a loss of hydraulic capacity; in summary the full potential of the Orchard Court culvert is not being realised due to the restrictive nature of the inlet.

Therefore, this measure proposes to adjust the inlet so that the design height of 2.1m is restored. This would involve removing the existing inlet and extending upstream the existing culvert run with a new section of similiar dimensions. This would require removal of the driveway and garden at the adjacent property.

Figure 7-5 Illustration of loss in cross-sectional area at the Orchard Court culvert inlet

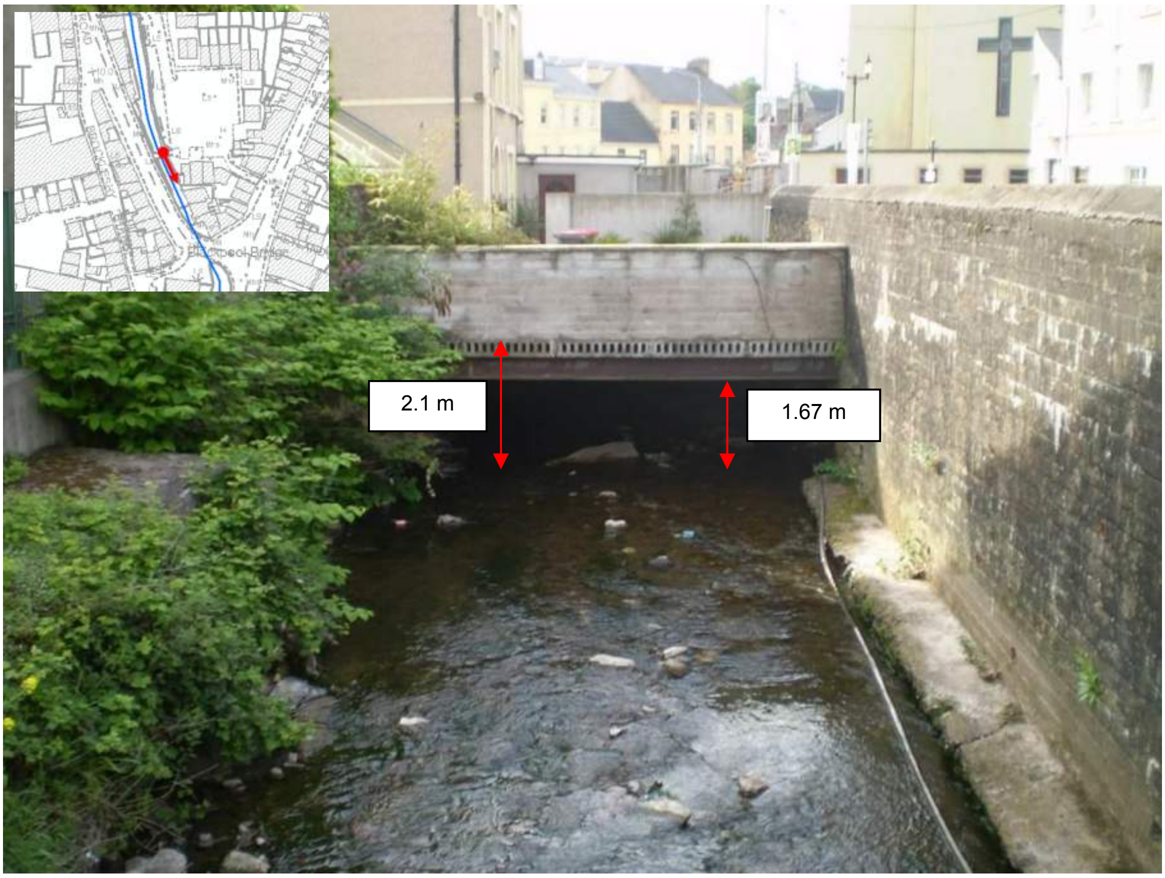
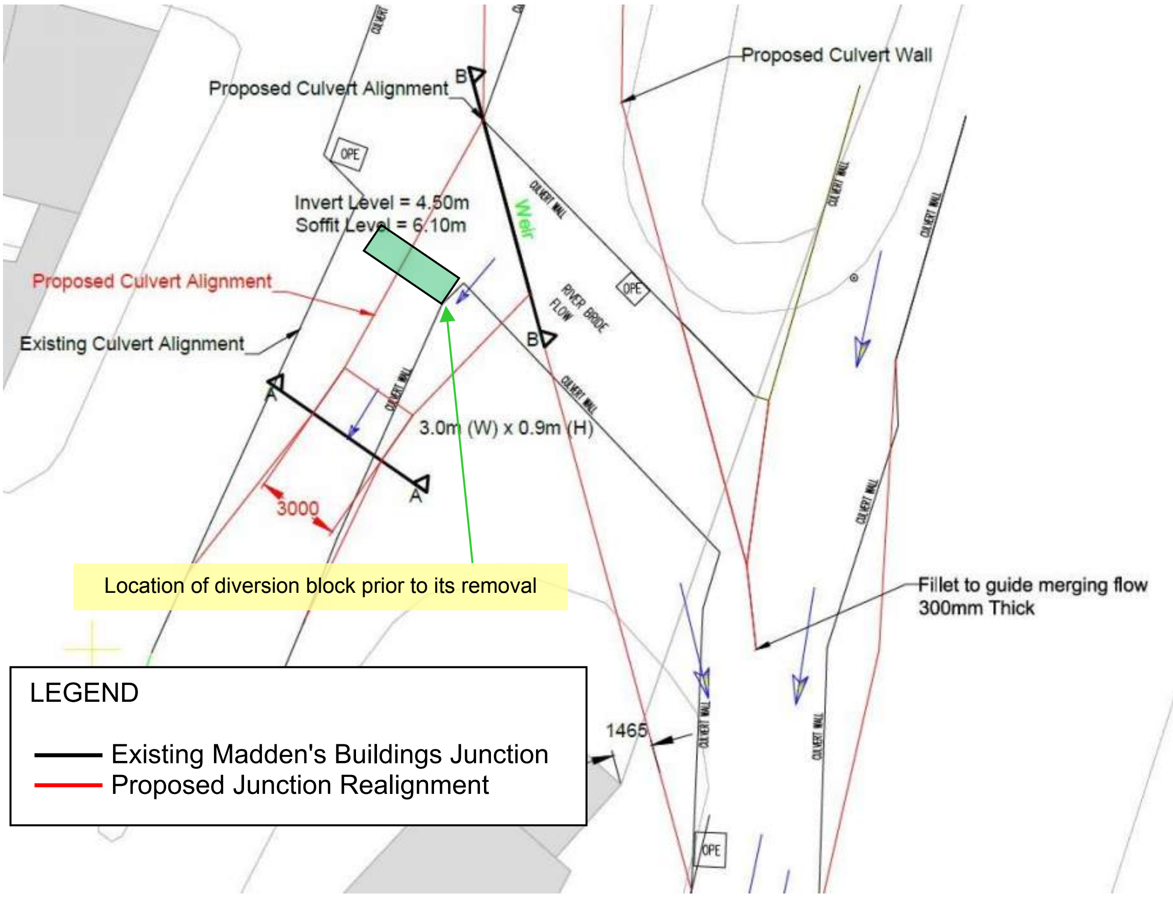


Figure 7-6 Illustration of dimension change just inside the Orchard Court culvert inlet, looking back upstream

7.3.7 Realignment of the Madden's Buildings junction

A possible cause of elevated water levels at Blackpool Church is the complicated nature of the Madden's Buildings junction. It is located approximately 200m downstream of Blackpool Church and forces the culverted River Bride to make two consecutive near-90° turns. As well as this, up until recently, there was a diversion block at the entrance to the Brewery Branch culvert, causing a further localised headloss. This has since been removed. It is quite likely that this collection of hydraulic inefficiencies are a contributing factor to flooding in Blackpool and that their removal could help ease the pressure on existing infrastructure upstream. Initial testing of the hydraulic model shows that the series of turns increase water levels in the junction by approximately 100-200mm. Therefore, this measure proposes to adjust the existing junction geometry so that the Bride's transition into the Phase 3 GBK culvert is straighter, as shown in Figure 7-7.

Figure 7-7 Overview of proposed realignment of Madden's Buildings Junction



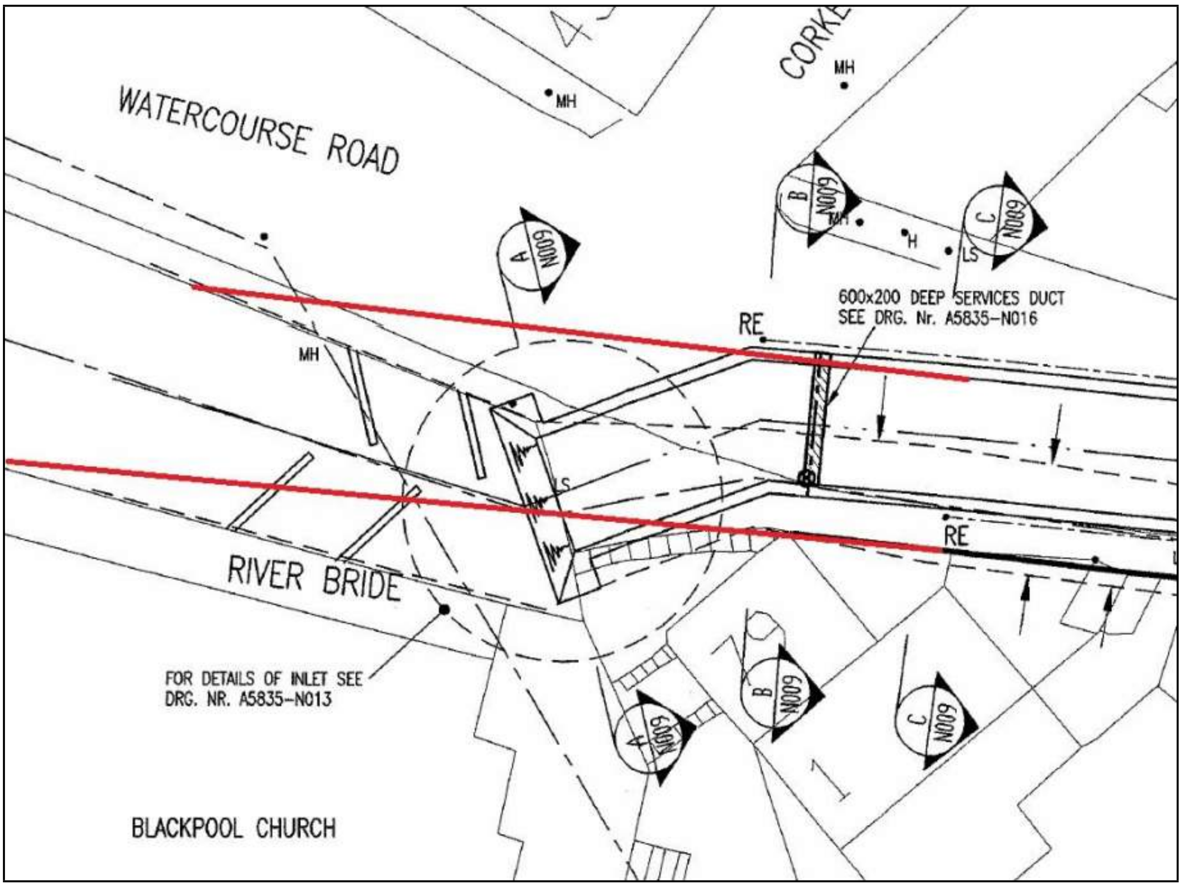
If one allows the full 1% AEP design flow of approximately 34 m^3/s drain to the junction, the flow split is approximately 60:40 between the two branches; the majority is carried down the Phase 3 GBK. This means that at the peak of the 1% AEP, the Brewery Branch would be expected to convey 12.5-15 m^3/s, depending on the upstream measures that were implemented. Unfortunately, this means that the Brewery Branch would be running close to full in some locations along its run; there is also localised surcharging approximately 225m downstream of the inlet. A decision would have to be made if this measure was pursued as part of the scheme if it would be acceptable to convey such a large flow down an old, brick conduit. Despite this, this branch will be required to reduce the hydraulic load on the Phase 3 GBK culvert and junction. Otherwise, forcing more flow down the Phase 3 GBK culvert will cause further pressurisation and subsequently, water levels to rise further upstream.

7.3.8 Realignment of the Blackpool Church culvert entrance

Another source of headloss in the system is the current alignment of the Blackpool Church culvert inlet. At present, the Bride must turn approximately 30º to the left, before turning back 30º to the right. There is also a concrete block at the base of the culvert on the right-hand side that causes localised headlosses. The complicated entrance is probably a result of the proximity of the watercourse's run to the adjacent properties.

It is proposed that the inlet be amended to provide for an easier transition into the Blackpool Church culvert. This would involve beginning the channel turn sooner than the current scenario, as shown in Figure 7-8.

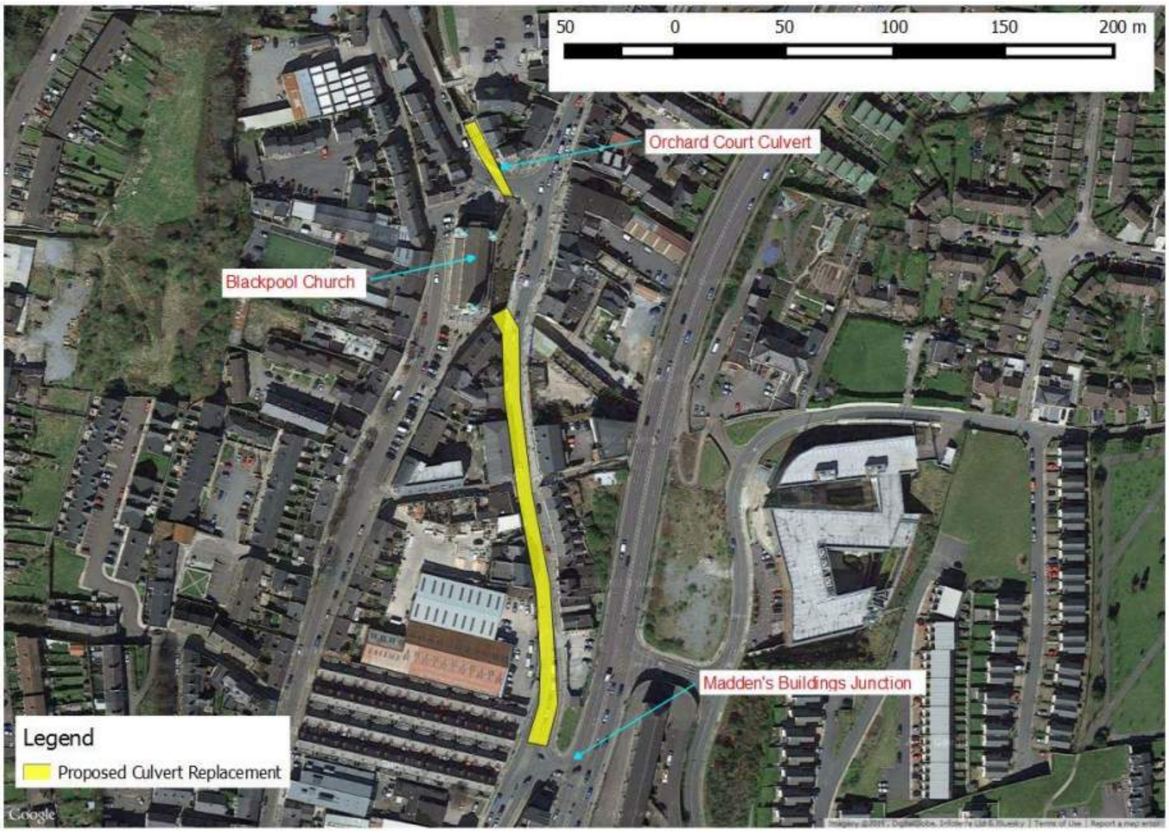
Figure 7-8 Overview of proposed realignment of Blackpool Church culvert inlet



7.3.9 Replacement of the existing Blackpool Church Culvert

Another approach to improving the hydraulic efficiency along the culverted system in Blackpool is to increase the size of the conduit from Blackpool Church to the Madden's Buildings junction; a complete culvert replacement. From the hydraulic modelling this culvert is the main restriction to flow, with approximately 26.4 m^3/s draining to the inlet in the 1% AEP event. This measure should lead to reductions in water levels upstream at Orchard Court; the magnitude of reduction would be dependent on the size of the new culvert section. For the purposes of this exercise, the entire length of the culvert system, from Orchard Court inlet to the Madden's Buildings junction has been altered. The representative cross-section has been taken as 5.3m x 2.1m and the Blackpool Church culvert run was given an average grade of 0.4%. The proposed culvert replacement is shown in Figure 7-9.

Figure 7-9 Overview of proposed culvert replacement at Orchard Court and Blackpool Church



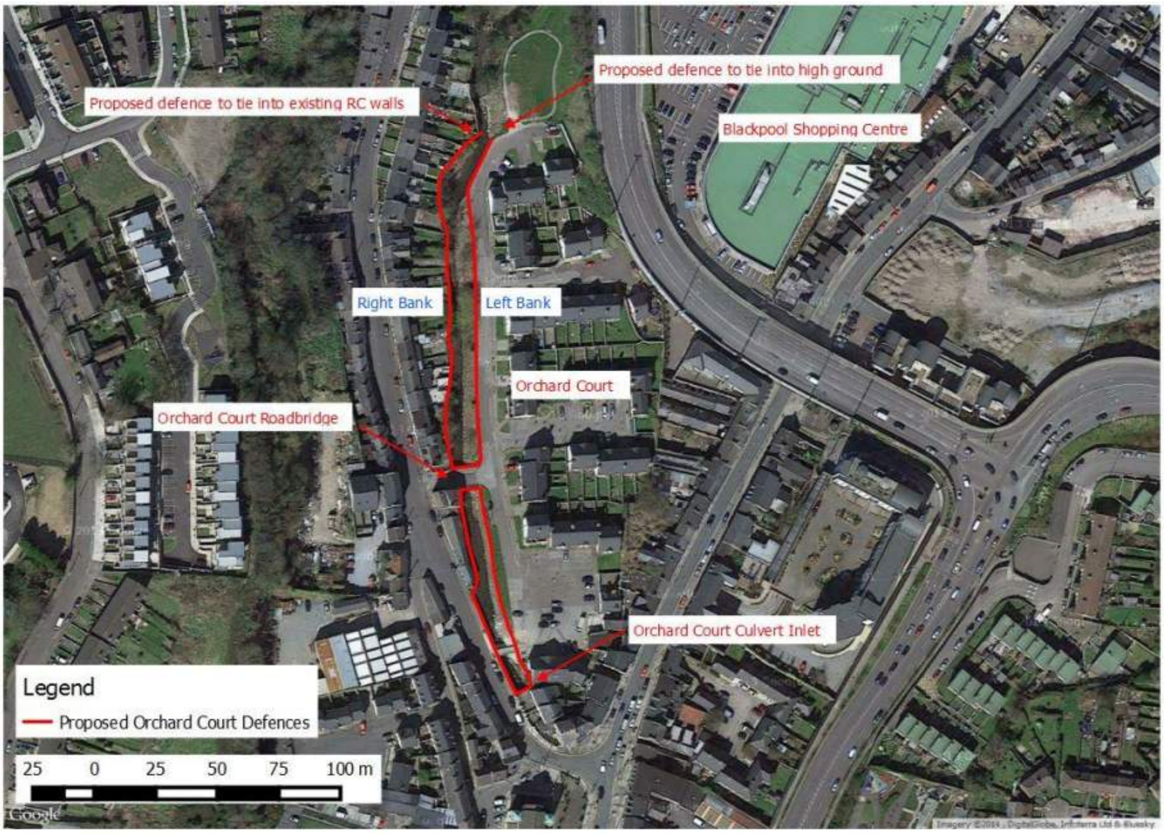
7.4 Flow containment measures

Flow containment measures are measures that usually elevate water levels in excess of the current design case but keep flood waters within the outline of the channel. Examples of flood containment measures include walls, embankments and culverts.

7.4.1 Direct Defences along Orchard Court

The primary cause of flooding in Orchard Court is flow spilling over the left-bank of the Bride. The simplest means by which protection could be provided would be the construction of direct defences (walls) along the left bank of the Bride. This would elevate water levels in the channel, and thus would necessitate similar construction on the right bank, at the back of various properties. The head wall downstream, at the existing Orchard Court culvert inlet would also need to be replaced. The height of these walls would depend on two factors: the maximum flow rate draining to Orchard Court during the design event; and the reduction in headlosses that could be achieved by means of structure/conveyance improvements immediately downstream. In summary, this measure will be modelled by allowing the hydraulic model to 'glass-wall' i.e. all flow will be conveyed in the 1D channel with no connection to its floodplain. Figure 7-10 shows the proposed alignment and location of this measure.

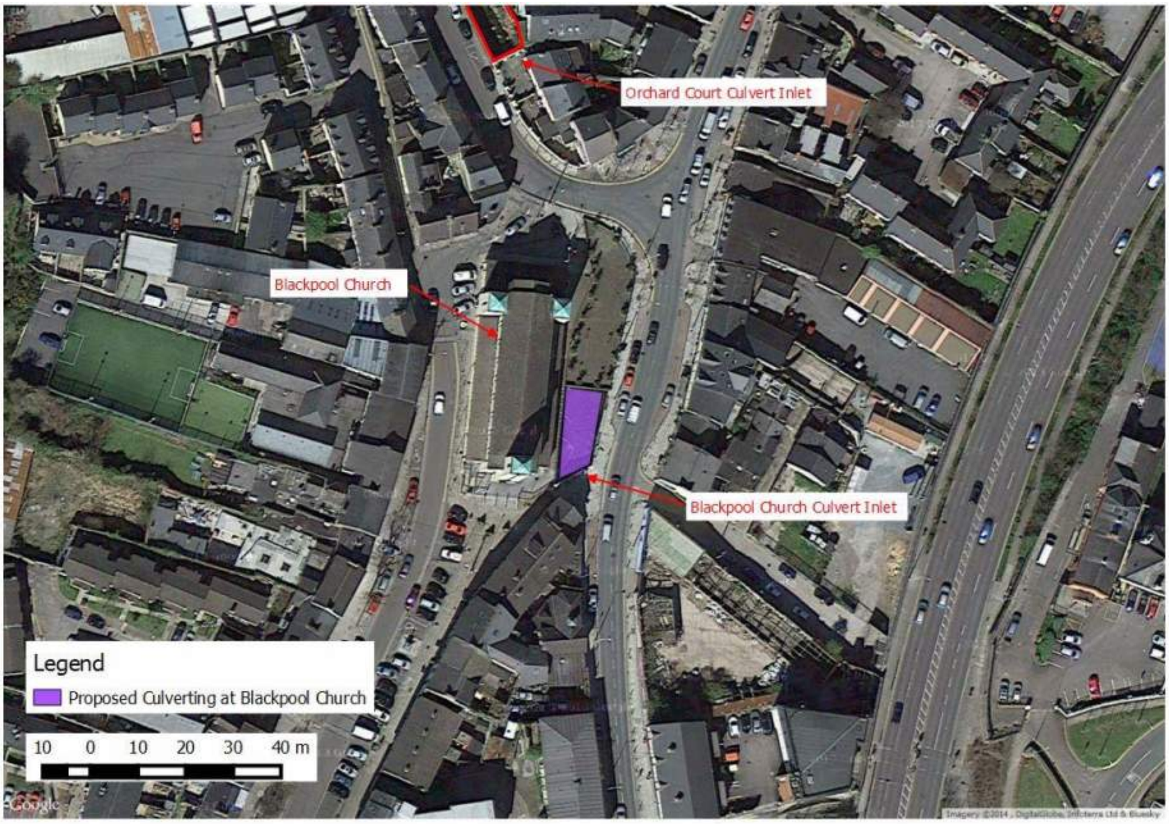
Figure 7-10 Overview of proposed direct defence measures at Orchard Court



7.4.2 Culverting of open channel at Blackpool Church

A significant feature of flood events in Blackpool in recent years has been the interchange of flow between the Watercourse Road and the open channel section adjacent to the Church. In the June 2012 event, flood water was seen to flow from the open channel, down the church steps and inundate Great William O'Brien Street. If flow is prevented from entering Orchard Court by means of conveyance improvements and/or defences, more flow will be forced downstream to the open channel section. Culverting of this section will ensure no flow escapes at this location and is also advantageous in terms of public safety, and the provision of a new civic space. A result of this measure would be to pressurise the existing culvert system further. All connections, gaps and seals would have to be made watertight to prevent water egress due to the increased pressure. The proposed culverting is shown in Figure 7-11.

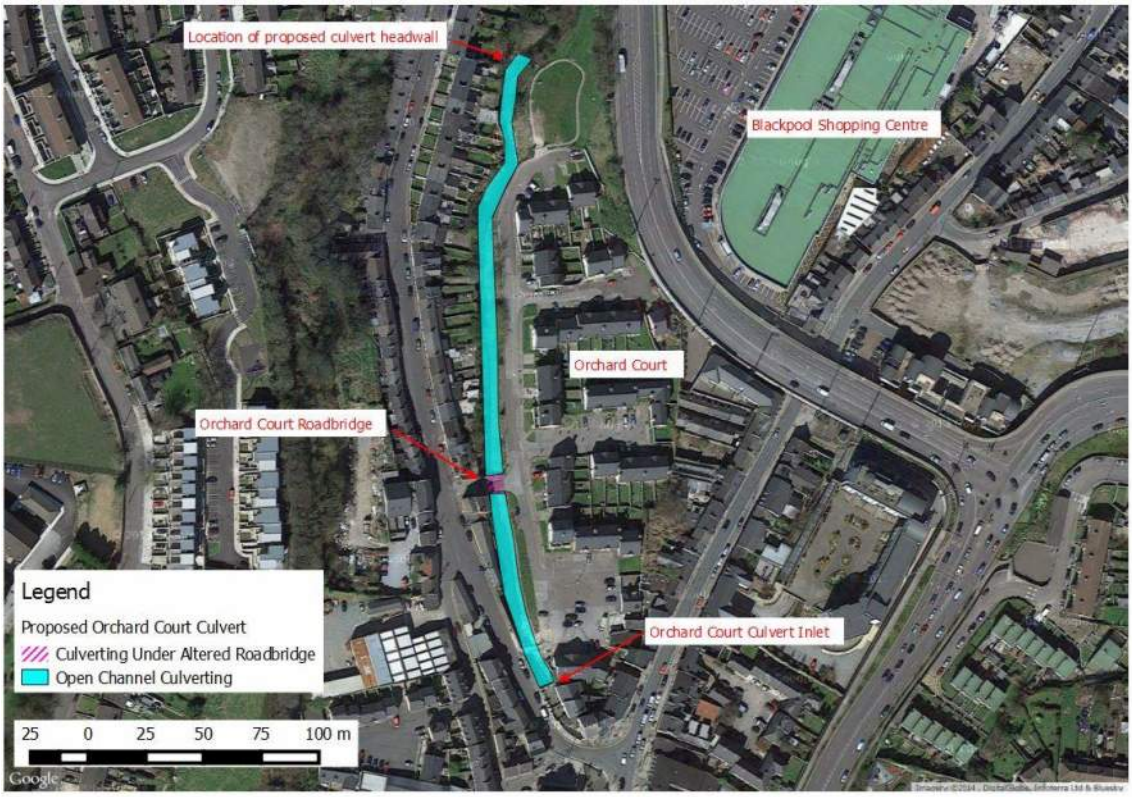
Figure 7-11 Proposed culverting of open channel at Blackpool Church



7.4.3 Culverting along Orchard Court

Another alleviation measure that could be pursued would be the culverting of the open channel along Orchard Court. The culvert would have to be sized correctly to convey the design 1% AEP flow and ensure minimal pressurisation. This measure has been tested in the hydraulic model using a 5m x 2.3m rectangular conduit, extending approximately 287m upstream from the existing Orchard Court culvert inlet. The proposed culvert length and alignment is shown in Figure 7-12.

Figure 7-12 Overview of proposed culverting along Orchard Court



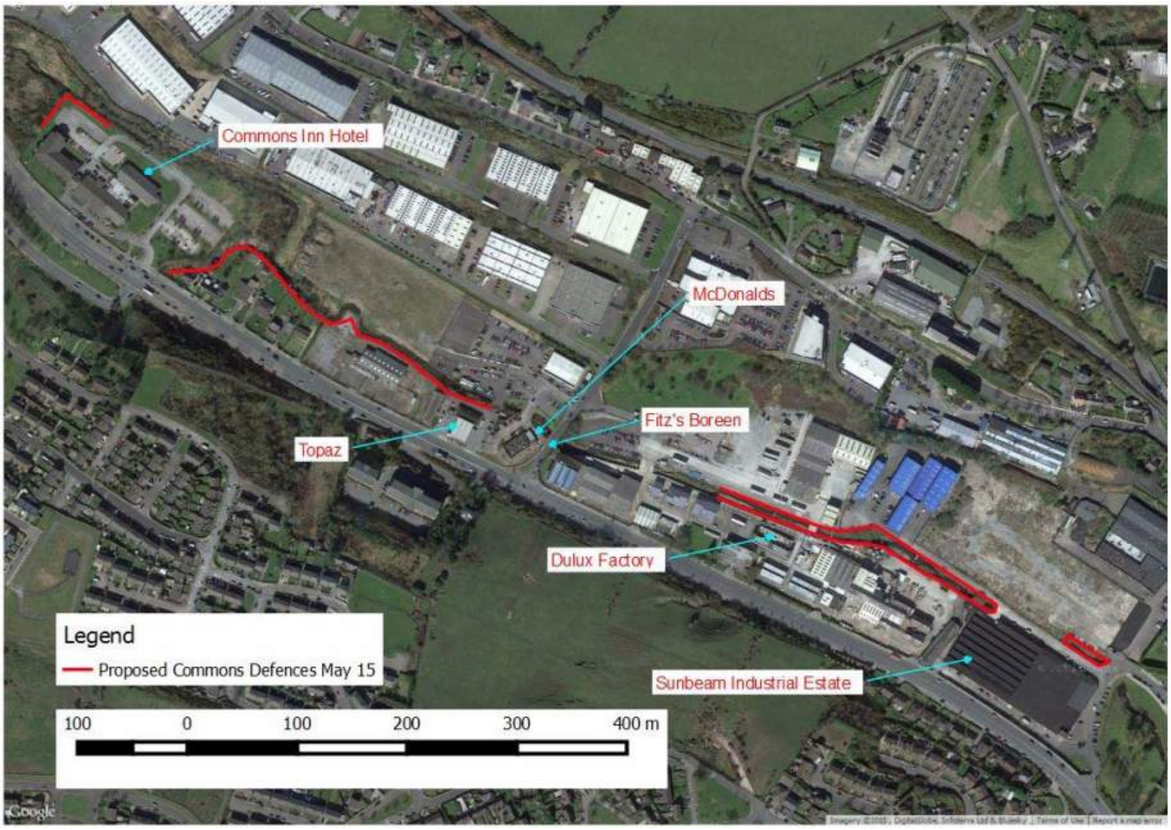
7.4.4 Direct Defences along the Commons Road

The Commons Road was an area acutely affected by the flooding of June 2012. The following flow routes were observed during this event:

- An overland flow route just upstream of the Commons Inn, on the right bank. Flood water covered parts of the car-park and flowed into the rear entrance of the hotel.
- An overland flow route on the N20 (Commons Road), with flood waters overtopping the right bank just upstream of the Topaz Garage.
- An overland flow route in the Dulux Paint Factory Premises, approximately halfway down the site, with flood waters exceeding the right bank.

It is proposed that low spots on the banks of the Bride, particularly the right-bank, are removed in favour of new direct defences. An overview of the proposed defence alignment is shown in Figure 7-13, but it should be noted that defence heights are dependent on what measures are used downstream.

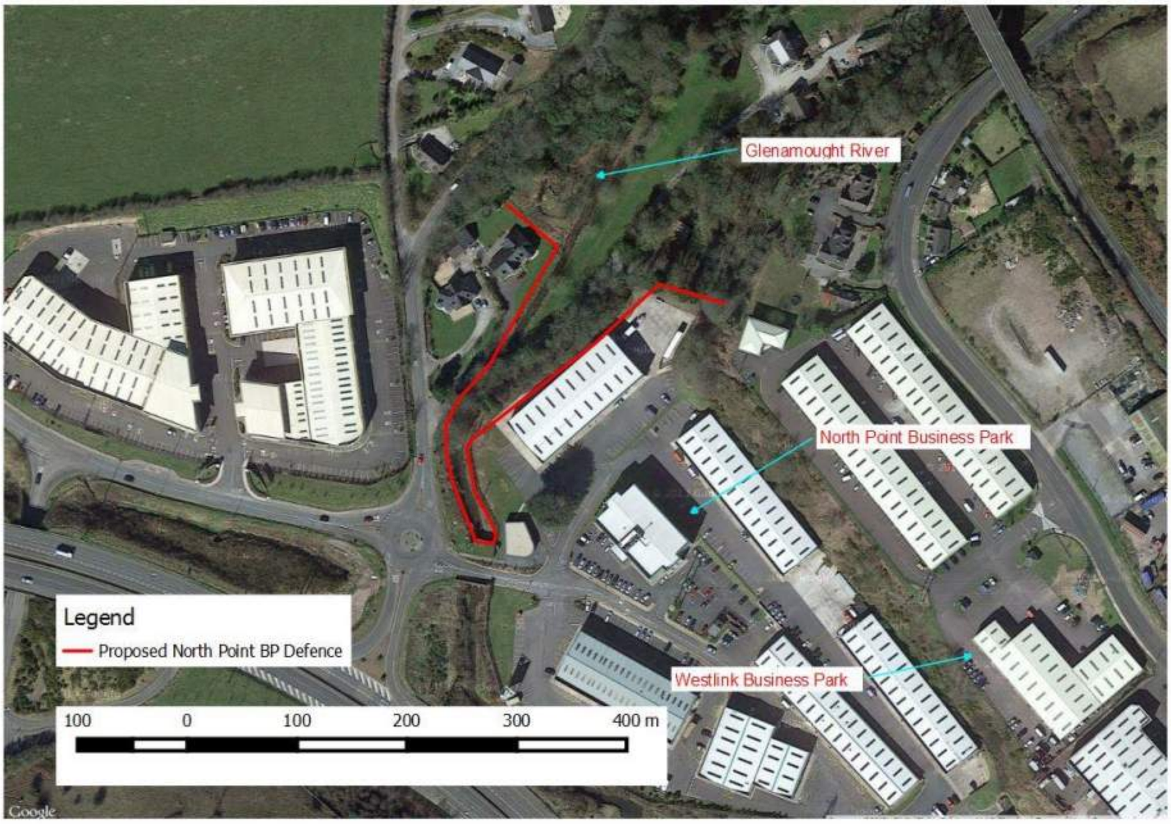
Figure 7-13 Overview of the defence requirements along the Commons Road



7.4.5 Direct Defences along North Point Business Park

Another area where flood water exceeded bank top levels was at the North Point Business Park entrance. As previously explained, the existing culvert unit is quite restrictive and prone to debris accumulation. However, there are also some lows spots on either bank that are at risk of being overwhelmed even if the existing culvert is replaced. It is proposed to provide direct defences in this location, as shown in Figure 7-14. Depending upon the approach adopted for the Kilnap Glen House access bridge, demountables may be required across this bridge if adopting the direct-defences measure. This structure is discussed in further detail in Section 7.7.4.

Figure 7-14 Overview of the defence requirements along the North Point Business Park



7.5 Summary of flood alleviation options tested in hydraulic model

7.5.1 Overview of methodology

To construct the emerging preferred option, a collection of measures were tested using the matrix outlined in Figure 7-15. The following presents the various options that have been initially assessed. For this assessment, the target is a 1% AEP standard of protection. The options tested are as follows:

- Option 1 - 'Do Minimum'
- Option 2 - 'Upstream Storage'
- Option 3 - 'Direct Defences at Orchard Court'
- Option 4 - 'Culverting along Orchard Court'
- Option 5 - 'Culvert replacement in Blackpool Village'

Each option was formed using one or a number of the measures identified in the previous section.

Figure 7-15 Overview of measures used in each option

Measure	Option 1	2	3	4	5
Channel Cleaning and Maintenance	●	●	●	●	●
Upstream Storage		●			
Replacement of North Point Business Park Culvert		●	●	●	●
Replacement of Fitz's Boreen Arch Bridge		●	●	●	●
Removal of existing Orchard Court Footbridge		●	●	●	●
Direct Defences along Orchard Court		●	●		●
Culverting of open channel at Blackpool Church		●	●	●	●
Culverting along Orchard Court				●	
Modification of the existing Orchard Court Inlet			●	●	●
Direct Defences along the Commons Road		●	●	●	●
Direct Defences along North Point Business Park			●	●	●
Realignment of the Madden's Buildings Junction			●	●	●
Realignment of the Blackpool Church Culvert Entrance					●
Replacement of the existing Blackpool Church Culvert					●

7.5.2 Measures common to all options

It is immediately evident that many of the measures have been included in the majority of the options as they are critical works that must be carried out to yield any effective scheme. For example, the existing Fitz's Boreen arch bridge is undersized and tends to head-up substantial flood flows. This forces flow to flank and overtop the structure; and would lead to defence heights in excess of 2m at this location (from previous defended model runs). Therefore, this measure is a necessity to optimise hydraulic performance of each option. The performance of each option is discussed in greater detail in the following sections.

The measures that are explicitly common to all options tested are as follows:

- Channel cleaning and maintenance - Channel and structure condition has been flagged at numerous points in this report as a contributor to flooding and is a particular problem on the Bride watercourse. Therefore, any alleviation option would have to have a cleaning/maintenance cycle incorporated into its proposal.

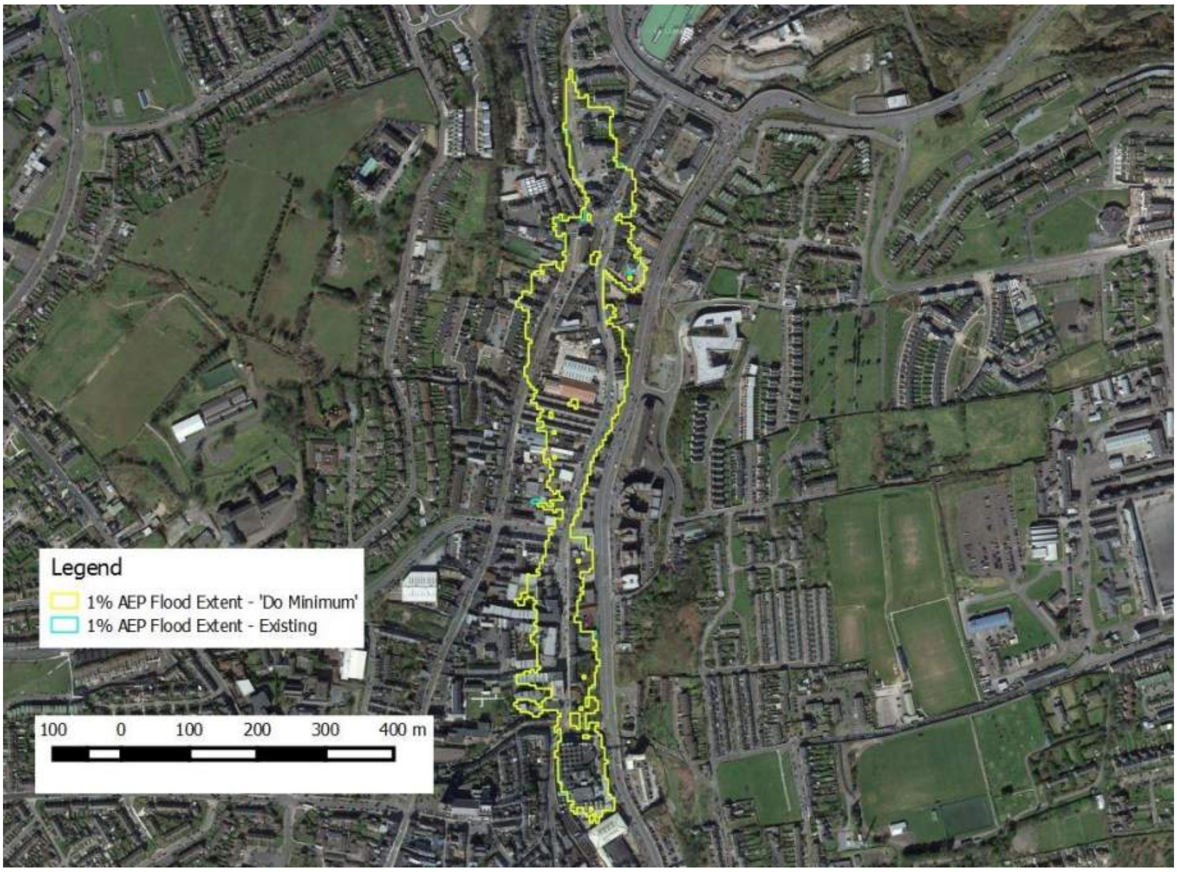
Measures that have not been formally tested but have been implicitly included in all proposed options (apart from Option 1) are as follows:

- Management of sediment - This measure has not been explicitly modelled as a more detailed hydrogeomorphological investigation would have to be carried out to determine the expected sediment and design velocities. This would dictate the size and feasibility of the proposed sedimentation area. However, it is recommended to examine this measure in more detail as the scheme progresses.
- Upgrade of the access bridge at Kilnap Glen House - This measure involves increasing the capacity of the existing vehicular access bridge to reduce water levels locally upstream of the North Point Business Park. This will, in turn, reduce the heights of possible defences along the bank. This measure is explained in further detail in Section 7.7.4.

7.5.3 Option 1 - 'Do Minimum'

This option involved the incorporation of a maintenance and cleaning regime throughout the length of the Bride-Glen-Glenamought system. This was represented in the hydraulic model by reducing channel roughness values ('typical conditions'). It was assumed that none of the sensitive debris-accumulation locations were blocked. It was shown that such work would have very little effect on the overall flood extent, as shown in Figure 7-16.

Figure 7-16 Comparison of 'Do-Minimum' Option with 1% AEP modelled flood extent in Blackpool Village

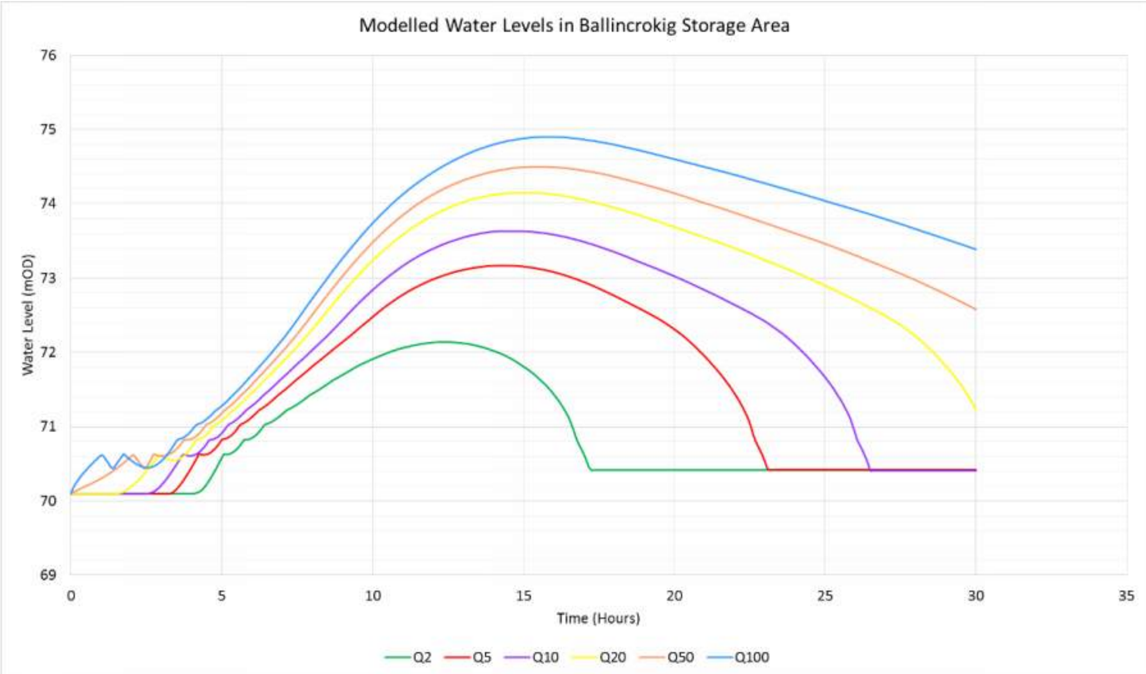


As expected, the 'Do-Minimum' option only attained modest reductions in maximum water levels within the channel. This work confirmed that the problem of flooding in Blackpool is a combination of both insufficient channel and culvert capacity on the system. This option, whilst not being effective on its own, is required as part of other proposed options.

7.5.4 Option 2 - 'Upstream Storage'

The storage area was tested using the design inflow hydrograph (1% AEP) event and the hydrobrake unit was represented as an abstraction unit with various output controls based on its operation curve. The storage area was shown to affect a reduction in peak flow from 16.43 m^3/s to 5.23 m^3/s draining to downstream of Ballincrokig. This resulted in a maximum modelled water level of 74.9 mOD within the reservoir. A graph of the maximum modelled water levels for a range of return periods for upstream storage is shown in Figure 7-17.

Figure 7-17 Graph of maximum water levels for a range return periods for upstream storage modelling



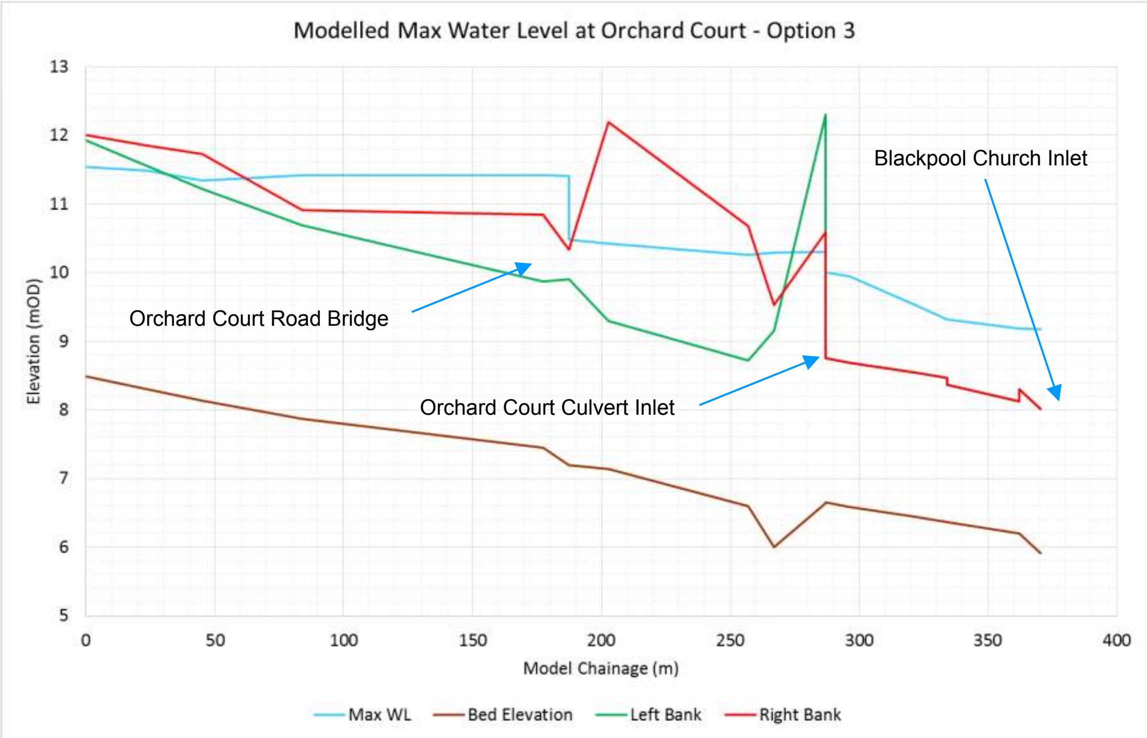
This option decreases defence requirements further downstream; both in terms of extent and height. For example, without freeboard, the defence requirement at Orchard Court would only be approximately 400mm. The issue with this option, however, is its effectiveness when subjected to sustained high inflows or multi-peaked events. The volume requirements for such events would be greater than that of the single-peak FSR hydrograph used in this test. Also, there would be a significant portion of the catchment located downstream, meaning that none of the Bride or the Glenamought below the reservoir would be intercepted. This would introduce great uncertainty to the operation of the reservoir, particularly when there is a lack of spatial distribution of rainfall data. In summary, further investigative work would have to be carried out to determine if the outflow rate and available volume would be enough to cater for a multi-peaked event, so far upstream of the study area. If successful, this option is hydraulically viable.

7.5.5 Option 3 - 'Direct Defences at Orchard Court'

This option investigated the feasibility of erecting direct defences (i.e. walls) at Orchard Court to prevent flooding of Blackpool village. This option tries to contain all flood waters within channel and force it through the Blackpool culvert system. Unfortunately, this generates elevated water levels and would make required defence heights extremely high. For example, Figure 7-18 depicts the maximum water level at Orchard Court for the 1% AEP event. It shows that even with some adjustments made downstream (culvert inlet modification and realignment of Madden's Buildings junction), defence heights of at least 1.6m are required to contain the flow. This does not take account of freeboard and/or climate change requirements.

This option also greatly pressurises the existing culvert system and all seals/connections would have to be appropriately upgraded to cater for this. This option is not particularly attractive due to the height required for protection.

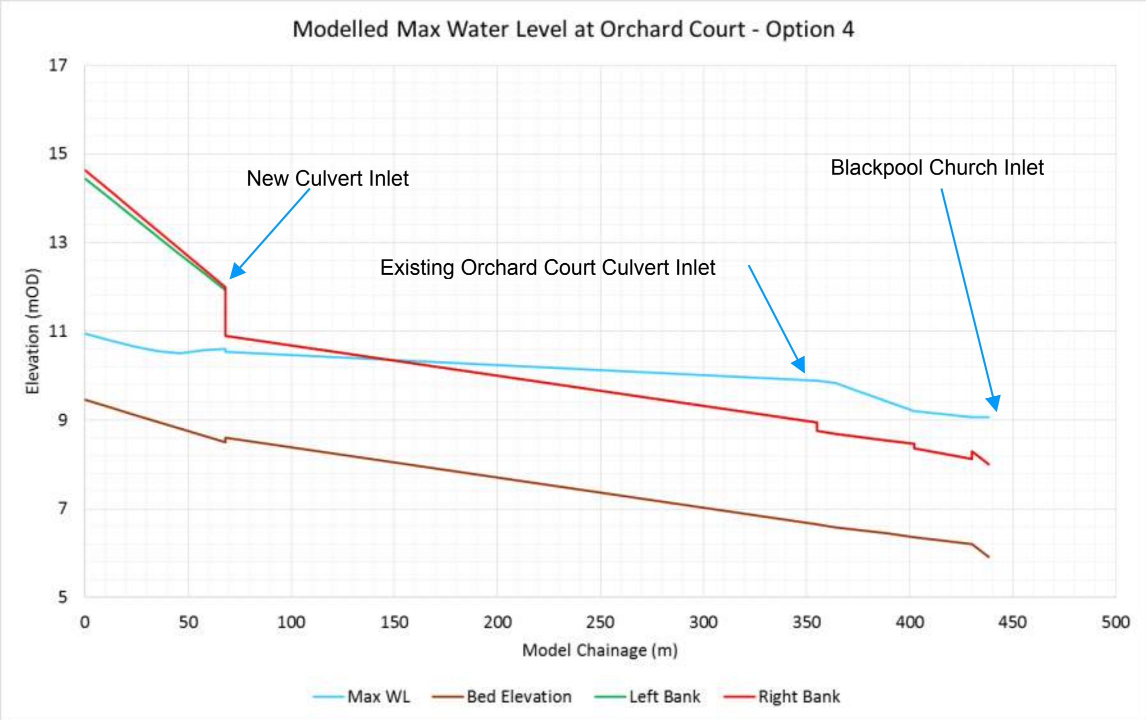
Figure 7-18 Maximum modelled water level at Orchard Court for Option 3



7.5.6 Option 4 - 'Culverting along Orchard Court'

This option examined the complete culverting of the Bride watercourse from just upstream of Orchard Court to meet the existing Orchard Court inlet. This option assumes the modification of the existing inlet. It also assumes that the proposed culvert would be sized as a 5m x 2.3m rectangular conduit. The option is advantageous as it prevents flooding in Orchard Court and also removes the risk of material being dumped in the channel. The maximum modelled water level for this option is shown in Figure 7-19.

Figure 7-19 Maximum modelled water level at Orchard Court for Option 4

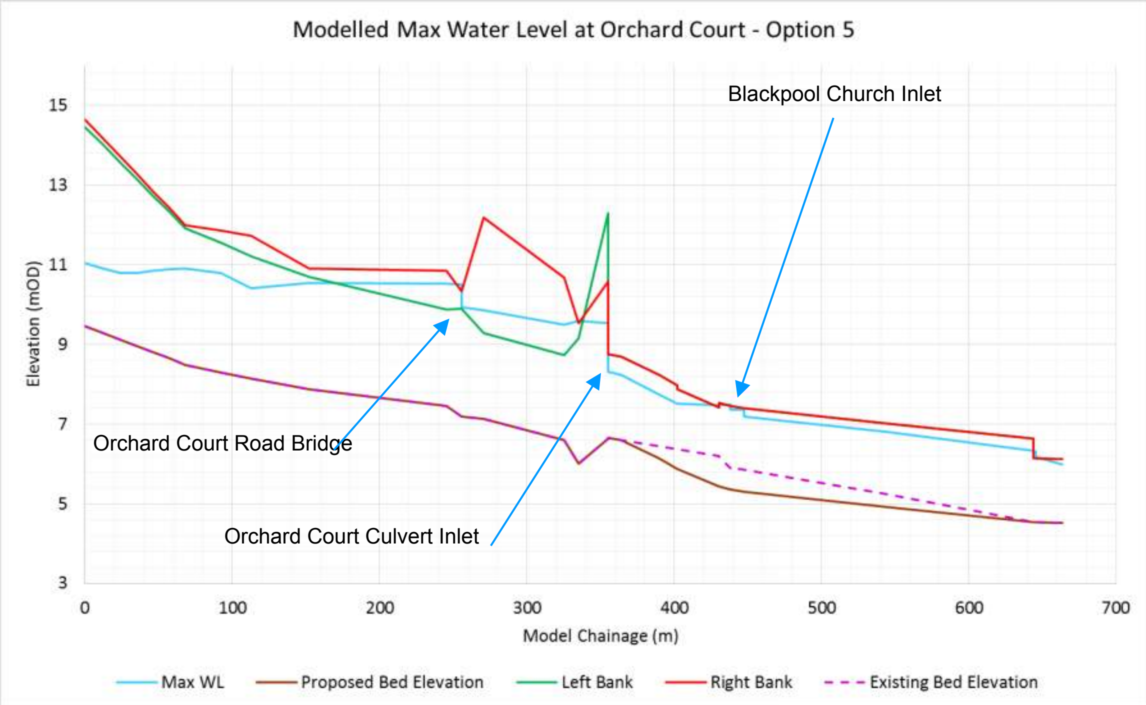


A disadvantage of this option is, as seen previously, a degree of pressurisation occurs in the culvert system downstream; caused by a lack of hydraulic capacity. A rough test showed that if the existing Blackpool Church culvert was increased in size (the test assumed 200mm increase in height), the maximum modelled water level drops below the conduit soffit as far as Madden's Buildings junction. This culvert replacement was the primary measure used in Option 5.

7.5.7 Option 5 - 'Culvert replacement in Blackpool Village'

Option 5 examined the effectiveness of replacing the existing Blackpool Church culvert over its entire length (as far as the Madden's Buildings junction) with an increased section size. It also replaced the existing Orchard Court culvert; both in size and grade. Both culverts were changed to a 5.3m x 2.1m conduit size; the larger culvert section just upstream of the Church remained unchanged in dimensions. The maximum modelled water level, as well as the change in bed shape, are shown in Figure 7-20.

Figure 7-20 Maximum modelled water level in Orchard Court and Blackpool Church Culverts for Option 5



It can be seen from the result presented above that Option 5 is successful in causing a reduction in water levels at Orchard Court, relative to Option 3. However, the existing Orchard Court inlet and the inadequate channel immediately upstream still necessitate the construction of direct defences to provide protection from flooding.

7.6 Preferred option

The maximum water levels derived from each option have been collated together in Figure 7-21 and Figure 7-22. It is immediately clear that the most effective option in terms of level reduction in Blackpool village is Option 2 ('Upstream Storage'). It results in minimal defence requirements and surcharging of the existing system. It also avoids considerable construction risks and disruption associated with works in Blackpool centre. Another advantage of Option 2 is that it reduces defence requirements along the Commons Road. For example, there is an approximate reduction of 800mm in maximum water levels just upstream of Topaz Garage. This difference increases to 1m in the Dulux Factory.

However, the uncertainties associated with Option 2 with regard to its distance upstream of the study location, the lack of rainfall distribution data, its performance when subjected to multi-peak events, appropriate initial water levels in the reservoir area etc. mean that there is not enough certainty in selecting this as the preferred option.

Incorporating the criteria set out in Section 7.1 and the practicality of completion, Option 4 (Culverting at Orchard Court) has been chosen as the 'Preferred Option'. The reasons for the decision are as follows:

- It removes the need for defences at Orchard Court, keeping all flow underground. This immediately cuts off a major flood route at its source.
- It removes a possible entry point for debris that has previously caused problems in the culverted system. Therefore, large obstructions such as fridges and mattresses should not find their way into the Madden's Buildings junction.
- It makes use of the existing culvert system downstream at Blackpool Church.
- There is some presurrised flow in the proposed culvert during the design event. However, the degree of pressurisation is not large enough to affect water levels upstream relative to other options tested (aside from Option 2).

Figure 7-21 Comparison of maximum water levels for all intervention options; Commons Road to Sunbeam

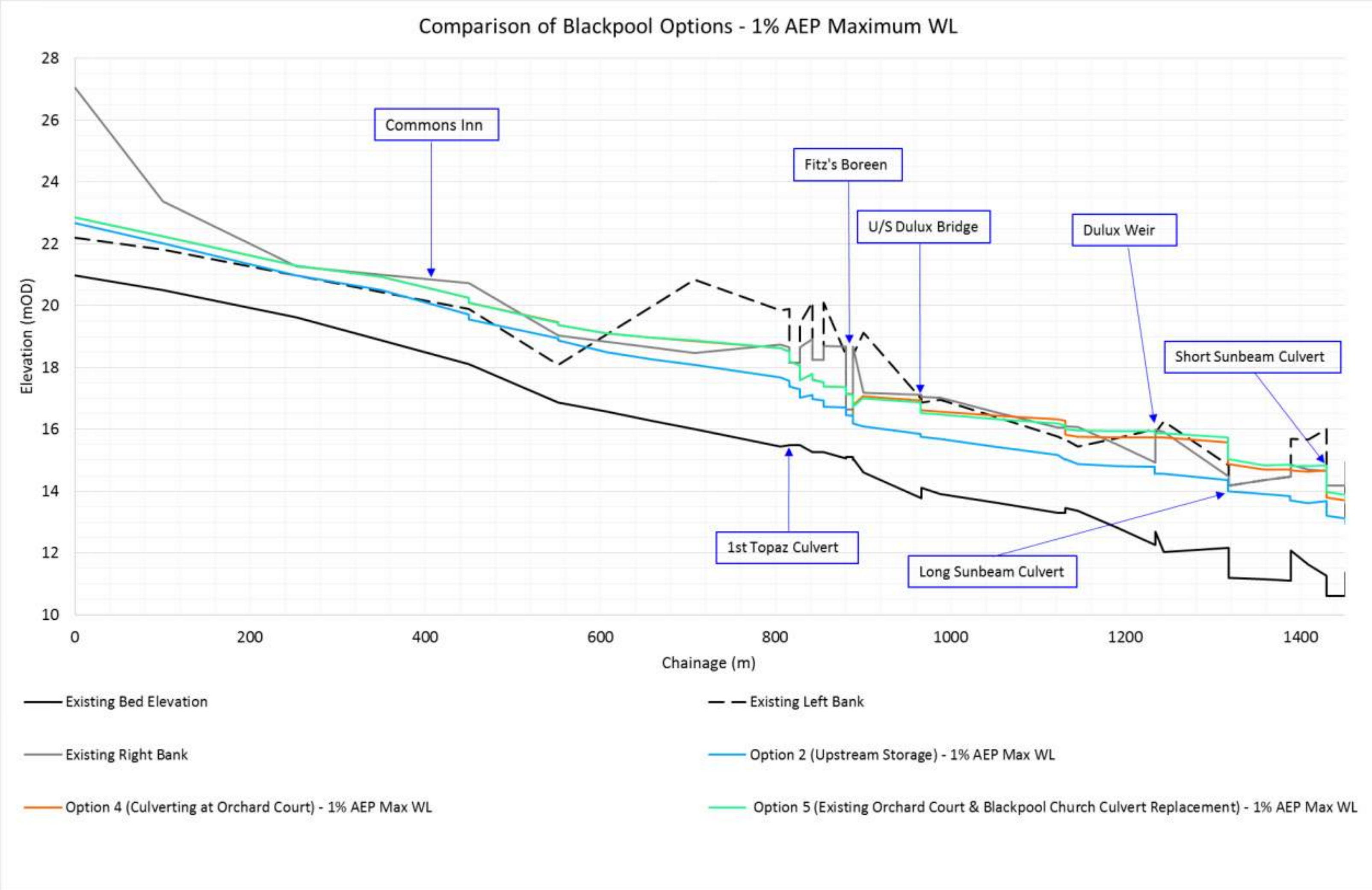
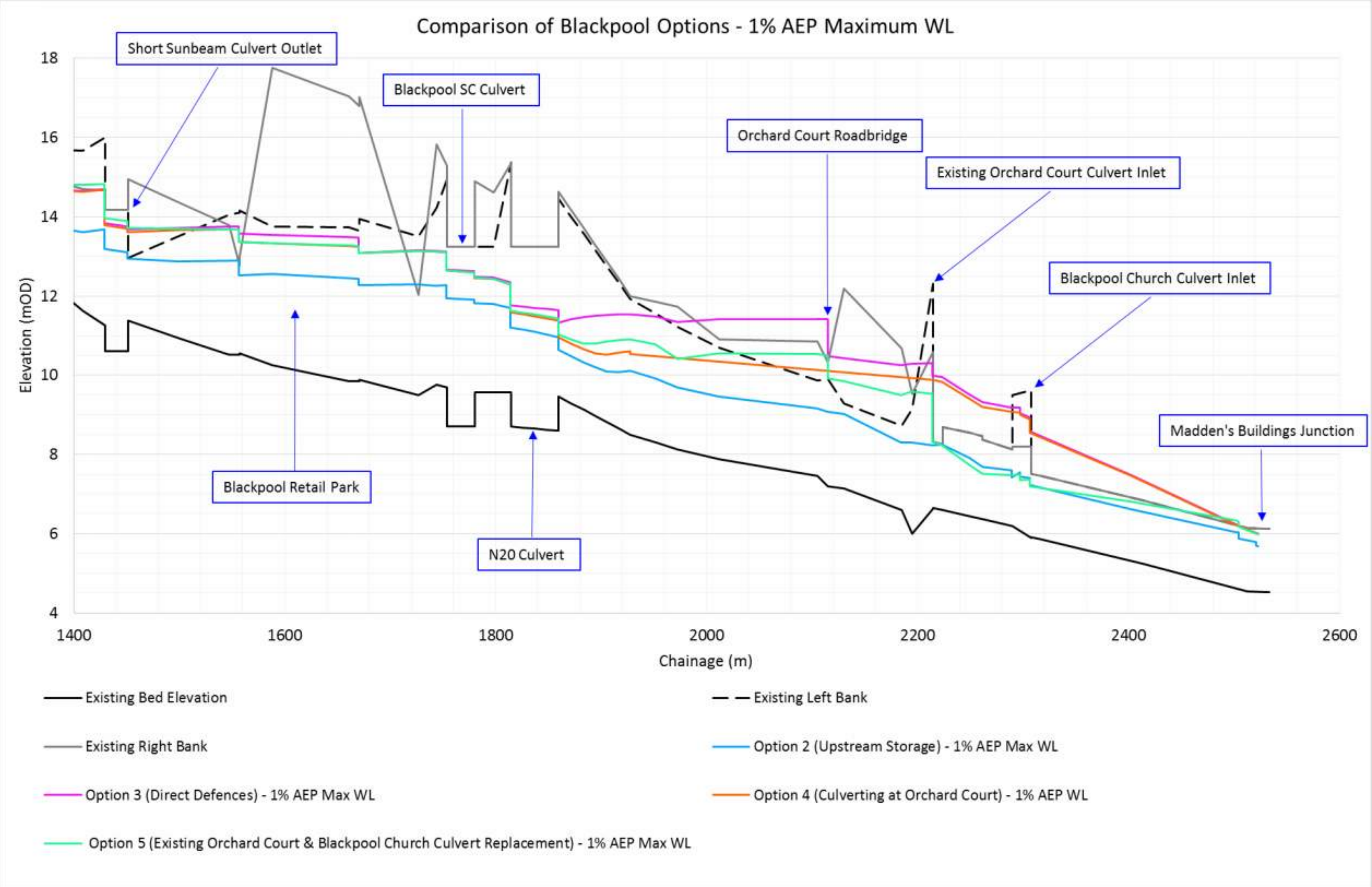


Figure 7-22 Comparison of maximum water levels for all intervention options; Sunbeam to Madden's Buildings junction



In addition to the findings presented in previous sections, the following points should be made based on the full body of modelling work:

- Option 2 aside; all other options tested exhibited minimal level difference at Blackpool Shopping Centre.
- The defences at North Point Business Park could be scaled down depending on the size of the conduit installed at the park entrance.
- The proposed Madden's Buildings Junction replacement would need careful design with regard to the quantity of flow allowed down the Brewery Branch. Option 2 aside, allowing the full 1% AEP design flow to drain to the junction will put pressure on the existing Phase 3 GBK culvert. Therefore, the Brewery Branch culvert will need to be utilised in some fashion and this need should be balanced with its structural integrity. Complete remediation of existing joint seals and sewer connections will be required in the culvert system if the option chosen pressurises the units.
- The proposed maintenance regime must be strictly adhered to as any accumulation of debris at sensitive locations could make certain measures ineffective.

7.7 Other recommended works

This section summarises locations where small-scale works are required but are not the primary drivers of flooding in the system.

7.7.1 Defences along Glen Stream at Gravel & Stone Yard and/or inlet replacement

Flood water was seen to overtop the right bank of the Glen Stream in the June 2012 event at the Gravel Yard, Spring Lane. As previously mentioned, the hydraulic model has confirmed that significant blockage of the Phase 4 GBK culvert and a relatively low right-bank can generate such a flow route. A detailed bank-top survey would have to be completed to more accurately model the issue. Possible alleviation measures would include the formalising and repairing of direct defences on the right bank and a new roughing screen. If this site were redeveloped, such measures could be stipulated as part of granting of planning permission. Figure 7-23 shows the possible defence alignment and Figure 7-24 shows the existing culvert inlet.

Figure 7-23 Overview of proposed defence alignment at gravel yard

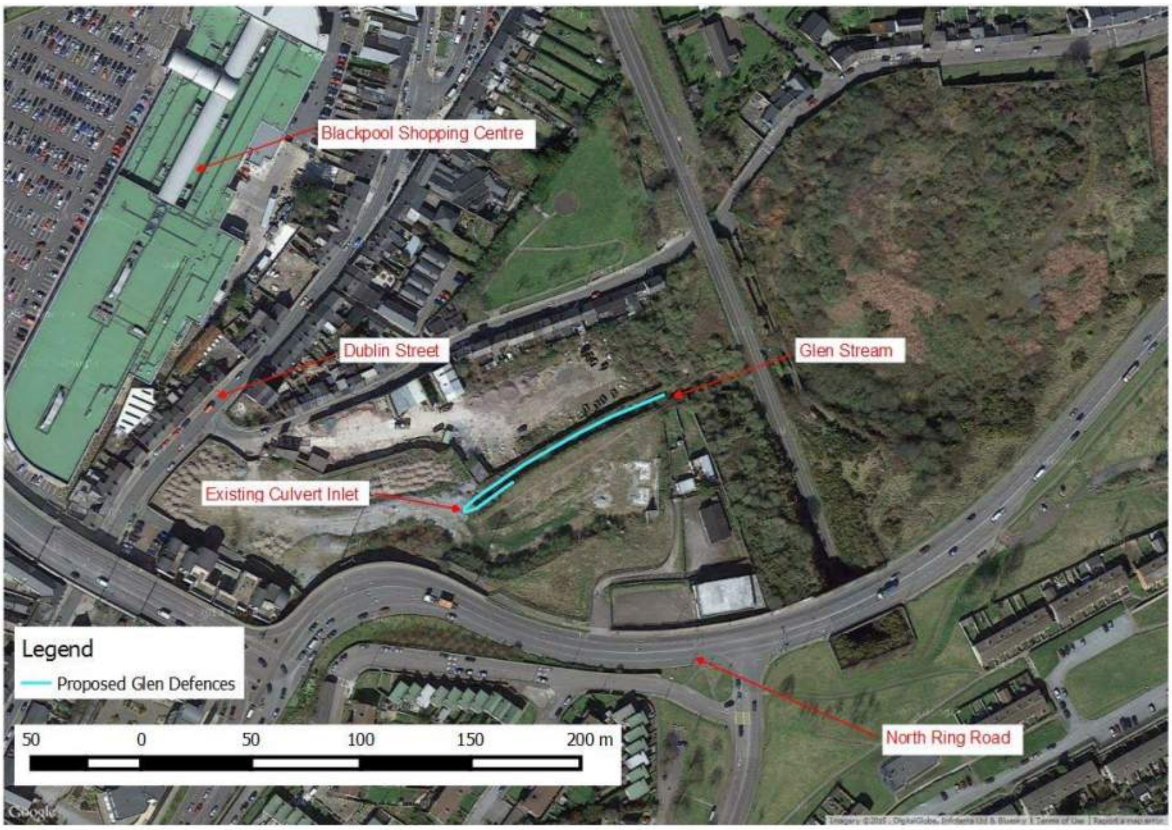


Figure 7-24 Existing condition of Phase 4 GBK Inlet at gravel yard

7.7.2 Straightening of the Bride channel at the Commons Inn Hotel

The river channel immediately downstream of the Commons Inn Hotel is a series of two, sharp 90° bends that force the river to turn suddenly in times of high flows. Hydraulic modelling has shown that the primary factor that governs flood risk on this reach of the Bride is the insufficient capacity of Fitz's Boreen arch bridge. However, these sharp turns do not help flood levels at the Commons Inn. Their partial straightening (as shown in Figure 7-25) would help ease headlosses at this location. The drawback to this would be that velocities and flow rates draining to Fitz's Boreen would increase slightly.

Figure 7-25 Recommended winter channel downstream of Commons Inn Hotel

7.7.3 Remediation of the Brewery Branch culvert

As mentioned previously, it is likely that the Brewery Branch culvert would need to be utilised as part of any proposed option. The conduit is of old, masonry construction and potentially would need to be upgraded to take an increase in flood flows.

7.7.4 Replacement of access bridge at Kilnap Glen House

This is an old masonry bridge providing vehicular access from the Mallow Road to Kilnap Glen House. The structure, as shown in Figure 7-26, is limited in its conveyance capacity and is an impediment to flow. This structure would, subsequently, have an effect on any proposed defence heights on either bank, even with the replacement of the North Point Business Park culvert downstream. The options testing assumed that this structure would be adjusted to increase its conveyance capacity.

Figure 7-26 Access bridge for Kilnap Glen House, on the Glenamought River

7.8 Design sensitivity

7.8.1 Sensitivity Overview

The purpose of this section is to determine an appropriate freeboard for the preferred solution, accounting for an inherent level of uncertainty in its design.

Uncertainties that must be considered in this assessment are:

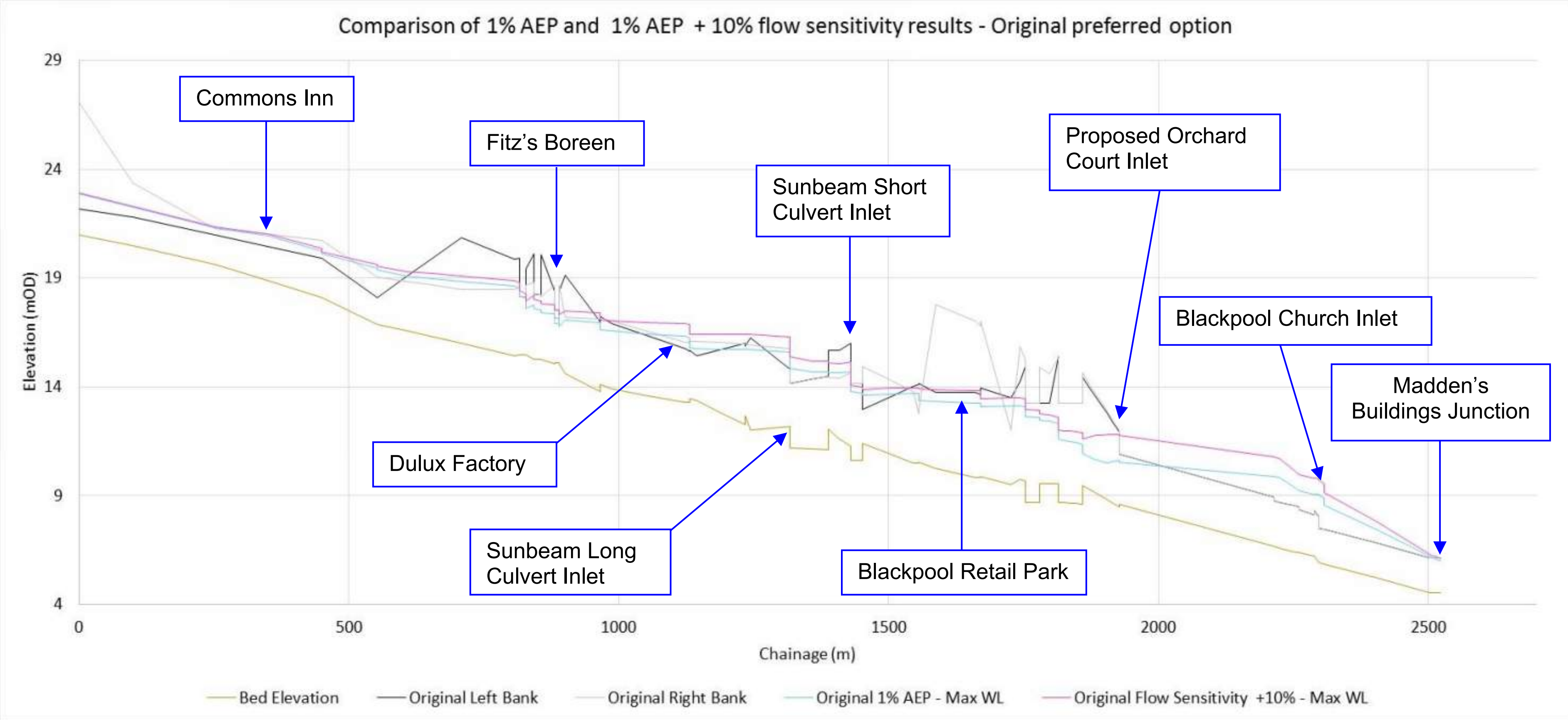
- Channel roughness is variable throughout the year and depends on the maintenance regime.
- Sediment movement and deposition is event dependant, but there are a number of deposition zones that need to be managed.
- Flow, as this is effectively an ungauged catchment, despite the Lower Lee and Glashaboy hydrological investigations; and the recent gauge installation whose record lengths are short.
- The complex hydraulics at Madden's Buildings bifurcation, the N20 culvert and in the section of culvert in Blackpool village. These are not easily replicated in the ISIS model, and any changes to reflect the works at these locations is only comparative. 3D modelling, or a physical model would be required to further reduce the uncertainty in these hydraulics.

Initial testing of the preferred option identified that flow was the major contributor to uncertainty on the system and would be the key driver in identifying a suitable freeboard. Based on this initial assessment (i.e. an arbitrary increase of 10% was used to help with initial examinations of sensitivity), the following pinch points were identified:

- Blackpool Church Culvert
- Orchard Court culvert section
- Long Sunbeam Culvert

As these controls were removed or re-engineered, further controls became notable, particularly when higher bound flows are tested in the hydraulic model. The increase in water level at key pinch points is significant, and cannot managed by purely increasing the freeboard of the defences. The impact on water level is shown in Figure 7-27.

Figure 7-27 Comparison of flow sensitivity results for the Original Preferred Option



The additional areas where the hydraulics are particularly sensitive to the uncertainties in flow in the modelling are:

- Madden's Buildings bifurcation
- N20 culvert, which has a supercritical flow zone downstream
- Short Sunbeam Culvert
- The two culverts upstream of Fitz's Boreen (arched, corrugated metal-lined)

7.8.2 Optimisation of the scheme

The outcome of this hydraulic sensitivity shows that there is a limitation to the quantity of flows that could be conveyed through the scheme without unfeasibly high defences. Therefore, a different approach to freeboard is required. For this scheme, it has been decided to generally apply a fixed freeboard of 600mm above the 1% AEP WL; this is a traditional approach. The removal/amendment of some of the key hydraulic controls justifies this approach as it removes some of the uncertainty from the scheme. The changes made to the preferred option to facilitate the application of a 600mm freeboard, along with their justification, are:

- Removal of the Pedestrian Bridges at Blackpool Retail Park
    - In flow sensitivity scenarios these structures cause headlosses that impact on the very sensitive reach with regard to defence crests. This will be discussed further in 7.8.3.
- Realignment of Blackpool Church Culvert inlet
    - To lower pressurised water levels in the Blackpool culvert system.
- Removal of Long Sunbeam Culvert
    - The low soffit on this structure causes significant heading-up of water levels in larger flow scenarios; the effect of which extended as far upstream as the upper reaches of Dulux.
- Short Sunbeam Culvert modified to rectangular conduit
    - After removal of the Long Sunbeam Culvert, this structure is the important control on water levels upstream in Dulux.
    - Section was converted from arched to rectangular conduit to preserve existing overhead access route into Blackpool Retail Park and increase conveyance capacity.
- Fitz's Boreen proposed replacement increased in size to 7.4m x 2.4m conduit
    - This further reduces water levels upstream of Topaz.
    - The 1st Topaz Culvert (7BRI_1490) is a restriction in flow sensitivity scenarios; however its removal is not a viable option. Therefore, the Fitz's Boreen section was increased to ensure maximum reduction in upstream water levels.
- Proposed North Point Business Park Culvert increased in size to 9m x 2.5m
    - Further reduce water levels upstream of this location.

Other amendments/works were incorporated into the optimised option on the basis of constructability and service issues. These were as follows:

- Limiting flow in the Brewery Branch to 9.5 m^3/s in the 1% AEP event
    - During the course of the optimisation work, a decision was made by the engineering team to limit the flow in the Brewery Branch to approximately 9.5 m^3/s using an orifice structure.
- Access bridge at Kilnap Glen House replaced with larger structure
    - The existing structure is a control on local water levels, as identified in Section 7.7.4. Therefore, it is proposed to be replaced with a 10.5m x 2.3m bridge section.
- Replacement of the existing Blackpool Bridge culvert (9m x 2m) with a 5.5m x 2.1m culvert

- Space in Blackpool village is required to construct a pumping station for the drainage system that must be discharged into the pressurised conduit during large flow events.

- Extending the proposed Orchard Court Culvert upstream to the N20 Culvert outlet
    - There was a significant headloss in the original preferred option at the N20 culvert outlet with transition to turbulent flow. This would have the potential to cause localised scour of the bed and wall foundations on the right bank. A hydraulic jump could have formed requiring much higher defence walls than would be locally acceptable. Therefore, it is proposed to extend the culvert upstream to meet the N20 outlet.

7.8.3 Optimised scheme testing

To test the performance of the optimised scheme with a fixed freeboard of 600mm; a flow sensitivity run was conducted using the Qmed Uncertainty principle outlined in the Lower Lee Hydrology Report. The approach used to derive the uncertainty in flow and, subsequently, the sensitivity of the model to flow has been described in Section 2.3.3. In summary, a flow sensitivity percentage of 18% was calculated by assessing the uncertainty in both the calculation of Qmed and the study growth curve.

The 18% increase in flows in the hydraulic model was applied in the following fashion:

- The 1% AEP flow estimates at each model inflow unit were increased by 18%.
- The laterals were increased by 18%.

The results of this test can be seen in Figure 7-28 and Figure 7-29.