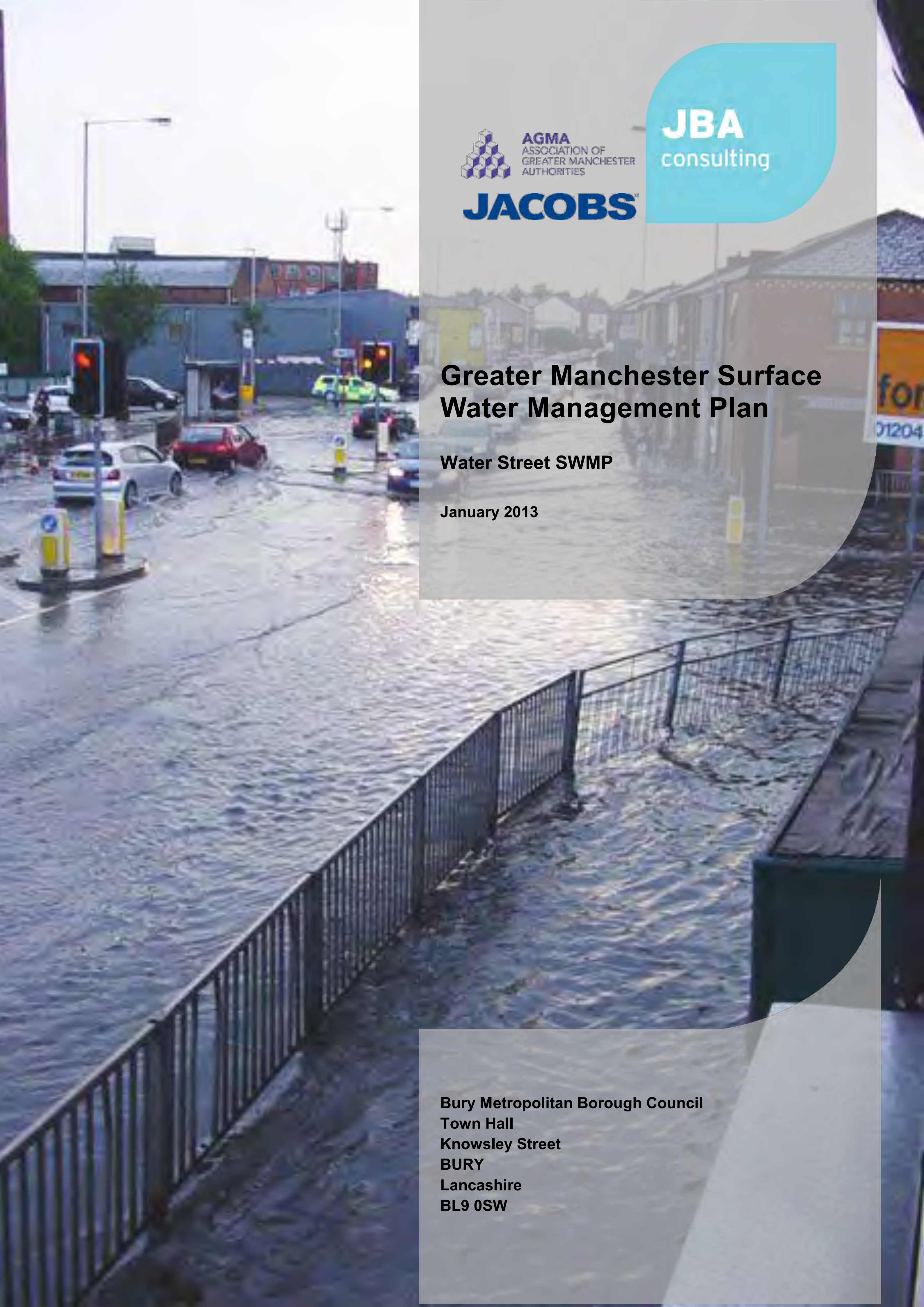

Greater Manchester Surface Water Management Plan

Water Street SWMP

January 2013

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Contract

This report describes work commissioned by the Association of Greater Manchester Authorities (AGMA) through its nominated lead authority Rochdale Metropolitan Borough Council (MBC) by letter dated 13 Feb 2012. Rochdale MBC's representative for the contract was Francis Comyn.

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Purpose

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Executive Summary

The junction of Water Street and Ainsworth Road is within the Bury Metropolitan Borough Council (MBC) boundary in Radcliffe and was identified as a hotspot during Stage 1 of the Greater Manchester SWMP. The site had been part of ongoing investigations by United Utilities (UU), Bury MBC and the Environment Agency (EA) and the SWMP was identified as a means to take this work forward.

Flooding at the site is a result of the limited capacity of both the stormwater culvert and the combined system. The limited capacity of the stormwater culvert is understood to cause flow to back up and then, because the combined sewer is lower than the stormwater culvert, enter the combined sewer via the BRY0129 CSO. This puts further pressure on the combined system eventually resulting in flooding at the Water Street junction.

The system is further complicated by the presence of an unauthorised overflow from the combined system into the Manchester, Bolton and Bury Canal. The EA would like this overflow removed however its removal at the current time would further exacerbate flooding at the Water Street junction.

The SWMP has reviewed in detail the assumptions associated with the flooding mechanism at the site as part of the intermediate assessment. It is concluded that the stormwater culvert is sufficiently sized for stormwater only flows but does not have the capacity to take the additional flows from the combined system. Because the combined sewer is located at a lower level than the stormwater sewer it is inevitable that the stormwater sewer will backup into the combined sewer before flooding occurs, however the main issue is the limited capacity of the culvert is preventing the sum of the combined and stormwater sewer flows discharging.

Prior to the assessment of management options a review of the hydraulic requirements and limitations at the site was completed. These are summarised as follows:

- Due to the shallow gradient of the stormwater culvert the capacity of the system before water backs up into the combined system is approximately 1.0 m^3/s, this is equivalent to the 50% AEP event on the stormwater system only.
- A provisional estimate of the required capacity below the crest level of the CSO to prevent flows discharging into the combined sewer in the 3.33% AEP event is 1.8 m^3/s.
- The maximum capacity of the culvert downstream of the canal is 3.8 m^3/s
- Required storage volumes in the combined sewer system are in the region of 4,000 m^3 if the unauthorised overflow is removed.
- Whilst there may be some scope to increase pass forward flows the capacity of the culvert downstream is insufficient for all additional flows required by an upgraded CSO proposed by UU.

A short list of options has been developed based on the feasibility of options discussed in a wider long list and those that can provide an immediate reduction in flood risk to the hotspot area. These options do not negate the need to consider the longer term management of the catchment.

Three options have been identified but the preferred option for managing the flood risk to the Water Street site will be increased storage capacity within the combined sewer system. Some allocation of funds from FDGiA could be expected to contribute to this work given the impact of stormwater flows to flood risk.

Feedback from UU is required confirm what storage capacity is achievable and available. This will inform further discussions with stakeholder regarding additional management options to be implemented alongside increased storage capacity.

The next steps will largely depend on feedback from stakeholders however it is likely the following key issues will need to be resolved prior to any further investigation:

- Engagement strategy – this will need to be developed and include the Canal and Rivers Trust, Greater Manchester Ecology Unit and the riparian owners along the alignment of the stormwater sewer as a minimum. It is recommended no further work into proposed

options that directly impact these stakeholders should be undertaken until these stakeholders have been actively engaged.

- Maintenance of the stormwater culvert – this is currently thought to be a limited issue but remains the responsibility of the riparian owner. It is recommended at this early stage that legal advice is sought to confirm if the responsibilities of the riparian owner can be enforced in the future.
- Funding responsibility - The approach outlined in this report assumes that UU are responsible for the combined system in its entirely and as such the level of protection achievable is the 3.33% AEP event, an element of which could be funded through FDGiA given the contribution from the stormwater system. However this is a grey area, the combined sewer system is predominantly discharging surface water runoff in the design event and if a design standard in excess of the 3.33% AEP event is desired it is recommended the potential to secure funding additional funding through FDGiA is investigated by the EA.
- Management strategy - there are strategic considerations within the study area and it is recommended an agreement is put in place between Bury MBC and UU recognising the drainage limitations within the catchment and proposing ongoing management policies such as those mentioned within this report. This agreement would need to consider the local strategy and those responsible for its implementation.

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Abbreviations

ASTSWF .... Areas Susceptible to Surface Water Flooding

AGMA .... Association of Greater Manchester Authorities

BRY0129......................... The Combined Sewer Overflow at the junction of Ainsworth Road and Water Street

CSO .... Combined Sewer Overflow

EA .... Environment Agency

FMfSW .... Flood Map for Surface Water

LIDAR .... Light Detection and Ranging Ground Level Dataset

LLFA .... Lead Local Flood Authority

MBC .... Metropolitan Borough Council

MCM .... Multi-Coloured Manual

NRD .... National Receptor Database

POG .... Planning Officers Group incorporating representatives from all the AGMA councils

SSSI .... Site of Special Scientific Importance

SWMP............................ Surface Water Management Plan

UPM .... Urban Pollution Model

UID................................. Unsatisfactory Intermittent Discharge

UU.................................. United Utilities

1 Introduction

1.1 Introduction

The Greater Manchester Surface Water Management Plan (SWMP) is split between two stages. Stage 1 carried out a Strategic Risk Assessment focusing on the identification of potential areas of significant risk, known as 'surface water hotspots', using

- New strategic surface water modelling hazard outputs across Greater Manchester
- The location of local critical and vulnerable receptors
- Significant flood risk thresholds and weighting
- 500m resolution grid squares

Lead Local Flood Authorities (LLFAs) across Greater Manchester then used these ranked hotspots, local knowledge and flood risk evidence collected during Stage 1, to identify a long list of priority areas, which they would like taking forward for further assessment during Stage 2 of the SWMP.

A short list of individual projects for priority areas was agreed through the Planning Officers Group (POG), with the aim of providing the greatest benefits to all LLFAs by targeting their higher risk areas whilst delivering a good practice toolkit, which consider partnership and funding opportunities between each district, the Environment Agency and United Utilities. A separate GM SWMP Stage 1 report documents the strategic risk assessment and surface water hotspots.

Stage 2 of the Greater Manchester SWMP takes each of these individual priority areas forward and seeks to complete as much of the technical process in Defra SWMP wheel diagram as is practical for individual local hotspot.

1.1.1 Stage 2 SWMP Projects

Each local project taken forward within Stage 2 of the GM SWMP will have its own report (this report) documenting the risk assessment made, options identified and action plan prepared. These individual reports, along with the key findings and lessons learnt, will be collated together to provide a good practice toolkit to aid LLFAs across Greater Manchester prepare their local flood risk management strategy and carryout further flood risk management activities.

The scope of each SWMP project will be different relating to the level of flood risk understanding gained through Stage 1, known data and available budget. Work Plans, agreed through POG, documented the anticipated scope of each project and were continuously developed as more information was collected and level of risk understood. The scope of the project then would adjust accordingly within the budget originally set.

1.2 SWMP Report Template

Each local SWMP report has been prepared around one template and framework to allow each LLFA across Greater Manchester to gain as much knowledge as possible regardless of the study location being within their district or not.

The Defra SWMP wheel diagram, illustrated in the Defra SWMP Technical Guidance, provides the template for this report, with each segment (Preparation, Risk Assessment, Options, and Implementation and Review) relating to particular chapters and the heading colours used.

2 Preparation

2.1 Water Street Local Surface Water Management Plan

The junction of Water Street and Ainsworth Road is within the Bury Metropolitan Borough Council (MBC) boundary in Radcliffe and was identified as a hotspot during Stage 1 of the Greater Manchester SWMP. The site had been part of ongoing investigations by UU, Bury MBC and the EA and the SWMP was identified as a means to take this work forward.

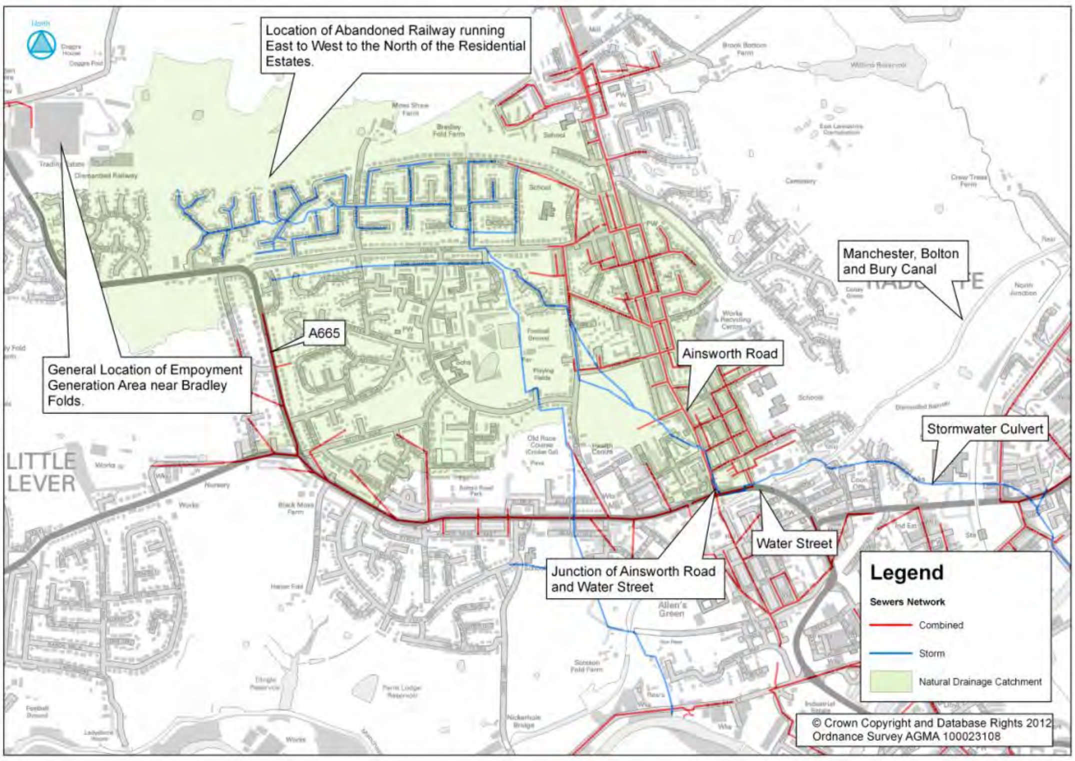
Flood risk to the site is from both surface water and sewer flooding and as such requires a partnership approach in the development of effective solutions. The Stage 1 modelling confirmed the multi-source flood risk to the site with the surface water modelling detailing a flow path following an ordinary watercourse draining from the north along Ainsworth Road and ponding at the junction with Water Street and the sewer modelling showing inundation in the same location. A number of residential and non residential properties were shown to be at risk at this location in the 3.33% AEP flood event.

The site was selected to be taken forward given the partnerships already in place and the historical evidence of flooding to the site.

2.1.1 Study Area

The natural drainage catchment for the area has been estimated from LiDAR and covers an area of approximately 1.7 km^2. The catchment is bounded to the west by the A665 and extends into open ground north of an abandoned railway line, Figure 2-1.

Figure 2-1: Overview of the Natural Drainage Catchment

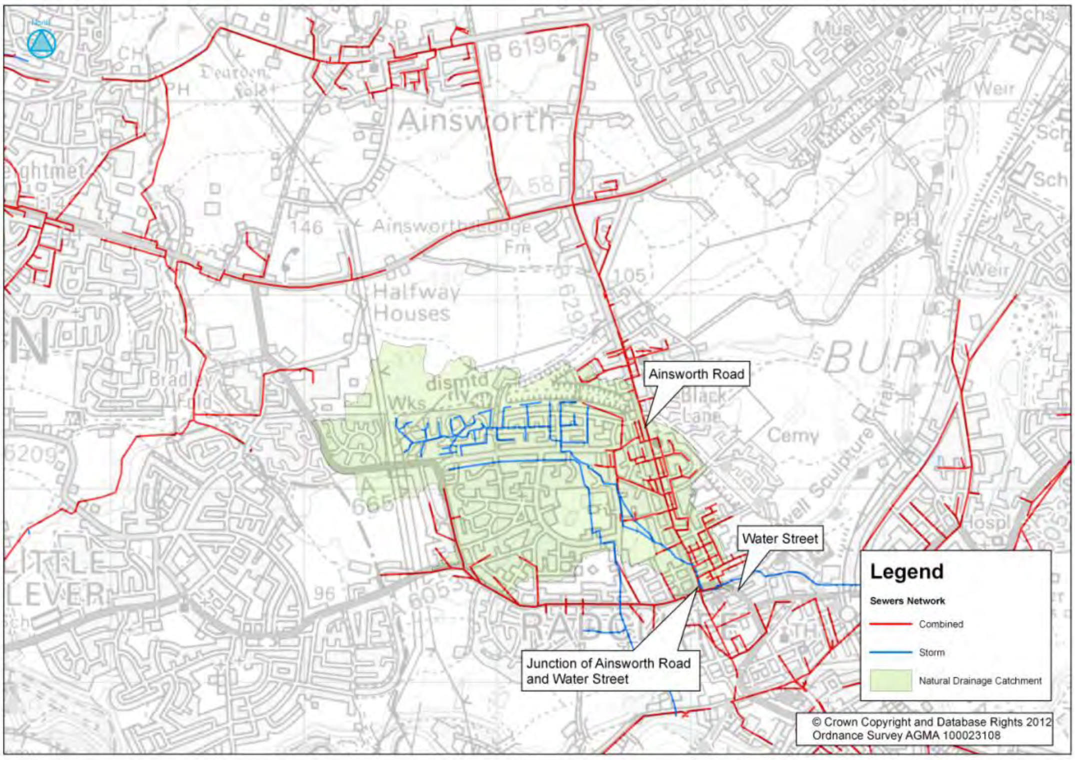


The eastern side of the catchment including Ainsworth Road and a small area to the southwest of the catchment is served by a combined sewer system. The remainder discharges into the stormwater system which roughly follows the natural drainage path to the Water Street/Ainsworth Road junction.

The combined sewer drainage area extends significantly beyond the natural drainage area, Figure 2-2. Two branches meet at the junction of Ainsworth Road and Water Street and so have an influence on flood risk at the site. The main branch drains from the north and extends as far as Ainsworth. The second smaller branch drains from the west along the border of the natural drainage catchment and covers western Radcliffe.

A short distance upstream from the Ainsworth Road/Water Street junction on the western branch of the combined sewer there is an unauthorised overflow discharging from the combined system into the Manchester, Bolton and Bury Canal, Figure 2-3.

Figure 2-2: Overview of the Combined Sewer System Drainage Catchment



The combined and stormwater sewer systems are linked at the Ainsworth Road/Water Street junction through a Combined Sewer Overflow (CSO) BRY0129, as shown on Figure 2-3 and Figure 2-4 below.

From the Water Street/Ainsworth Road junction the stormwater culvert turns eastwards, it continues at a shallow gradient until it drops approximately 2m a short distance upstream of a local reservoir in the location of an old mill, now demolished. It continues at a steeper gradient for a short distance before passing beneath the Manchester, Bolton and Bury Canal. From here it runs in a south easterly direction before outfalling into the River Irwell off the end of Rectory Lane. Approximately 200m upstream of the outfall, Crow Tree Farm Brook discharges into the culvert.

Figure 2-3: Overview of the Water Street Site

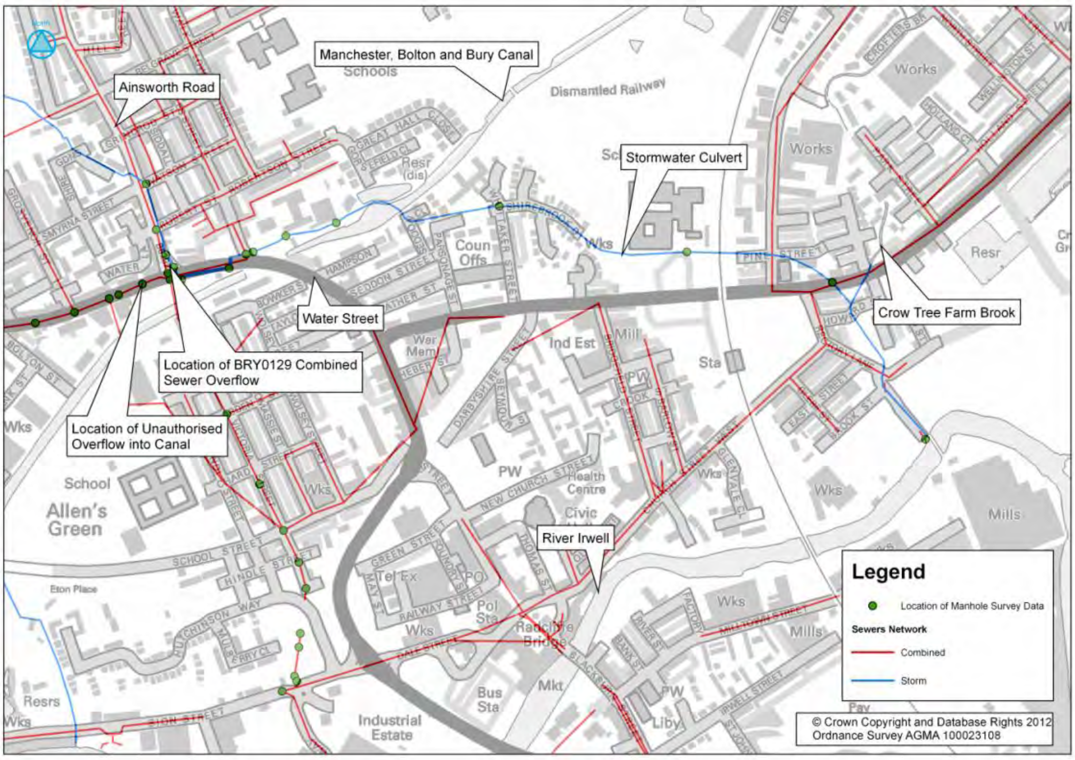
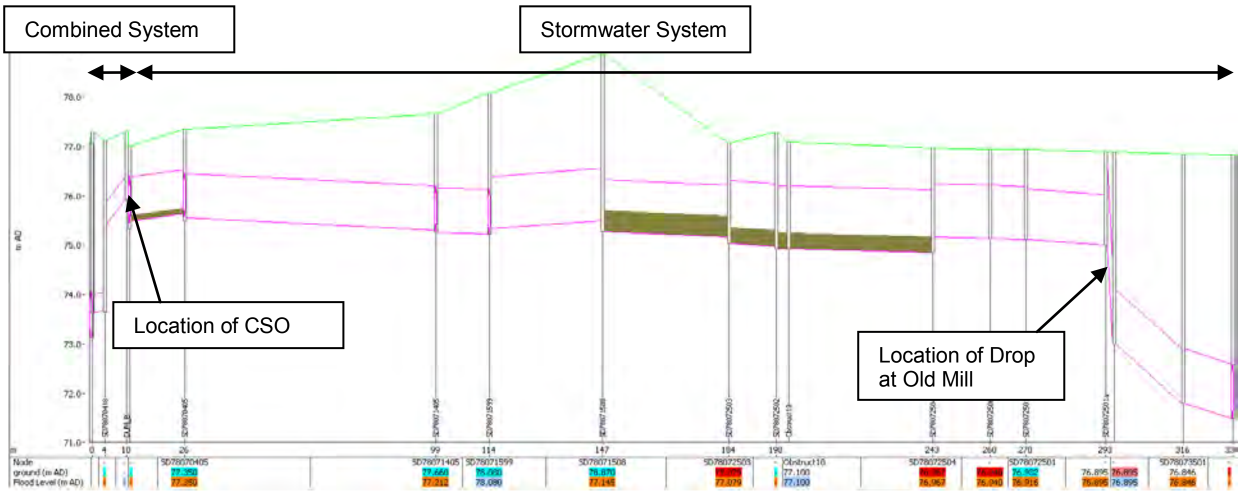


Figure 2-4: Long Section through CSO and Stormwater Culvert



The stormwater culvert is classed as an ordinary watercourse for the majority of its length to the River Irwell. This places the responsibility for its maintenance in the hands of the riparian owners, of which there are many.

The underlying soils in the catchment are loams and clays; the catchment is slowly permeable and seasonally waterlogged. The WRAP soil class for the catchment is 4 and the associated HOST classification for the soils is 24.

The second draft publication of the Core Strategy[1] for Bury is currently out for consultation. The document highlights the Water Street natural drainage catchment as a Critical Drainage Area

[1] Bury Local Plan, Second Draft Publication Core Strategy (Incorporating Development Management Policies), Bury

(CDA), which is an area sensitive to surface water runoff. The key plan for the Radcliffe area indicates the lower reaches coincide with the Inner Radcliffe Regeneration Area.

The upper reaches of the catchment are generally flagged for ecological enhancements although there is an employment generating area to the north west of the catchment towards Bradley Folds, Figure 2-1. This appears to be outside the natural drainage catchment.

2.1.2 Key Flood Risk Issues

Flooding at the site is a result of the limited capacity of both the stormwater culvert and the combined system. Downstream of the canal the capacity of the stormwater culvert increases significantly; the capacity and depth of this reach indicates the current pinch point in the system is upstream of the canal.

The limited capacity of the stormwater culvert is understood to cause flow to back up and then, because the combined sewer is lower than the stormwater culvert, enter the combined sewer via the CSO. This puts further pressure on the combined system eventually resulting in flooding at the Water Street junction.

The system is further complicated by the presence of the unauthorised overflow from the combined system into the Manchester, Bolton and Bury Canal, which is currently relieving pressure on the CSO. The EA would like this overflow removed however its removal at the current time would further exacerbate flooding at the Water Street junction.

BRY0129 has been identified as an Unsatisfactory Intermittent Discharge (UID) by UU and investigations into resolving this issue have been undertaken. The findings found no workable solution to the problem whilst there is no free outfall for the CSO.

2.1.3 Flood History

A number of reports and interviews with local residents have been supplied describing the historical flood risk. It has been estimated by UU that the site floods every 2 years.

There are 13 properties on UUs DG5 register which are reported to have flooded internally and an additional 6 properties reported to have been impacted by external flooding. Internal flooding properties have been assigned probabilities between a 20% and 5% AEP event which provides a reasonable indication of expected flood extents associated with various flood events.

In addition to the above photos collected by a local resident are shown in Figure 2-5 from a flood event on 5 July 2006.

Figure 2-5: Flooding of the Water Street junction on 5 July 2006

Council, October 2012.

Lessons Learnt 1 – choosing an appropriate SWMP project
The study has provided an opportunity to develop on an existing partnership arrangement into the investigation of flooding at Water Street. The development of solutions to the site will require input and buy-in from both UU and the EA and the SWMP therefore provides an appropriate framework to undertake the study.

2.2 SWMP Scope

The scope of SWMP has included the investigation and assessment of management solutions on the stormwater system. The key phases within the SWMP are highlighted within in Figure 2-6.

Figure 2-6: Water Street SWMP Scope

Preparation	Risk Assessment			Options		Implementation and Review
Scoping	Intermediate	Detailed	Map and Communicate Risk	Identification	Assessment	Action Plan Preparation
✓	✓	✓	✓	✓	✓	✓

2.3 SWMP Partnership

The key project partners are those who have currently been attending the tripartite meetings held by UU, namely Bury MBC, the EA and UU.

There are also a number of other local stakeholders who may be interested in the SWMP where they are directly affected by the findings of this study. The most obvious of these will be the Canal and River Trust, responsible for the Manchester, Bolton and Bury Canal, but others are likely to include riparian owners along the alignment of the stormwater culvert.

3 Intermediate Risk Assessment

3.1 Introduction

The first phase of this SWMP was an Intermediate Risk Assessment which focussed on:

- Collating and assessing all currently available data for the site
- Developing an accurate understanding of the flooding mechanism and any uncertainties associated with the site

3.2 Data Collection

Table 3-1 details all data identified and collected as part of the intermediate risk assessment. Regional level data across Greater Manchester was already available through Stage 1 of the SWMP.

Table 3-1: Water Street SWMP Data Collected

Data	Form	Provider
Design levels on the River Irwell	pdf	Environment Agency
LIDAR (1m resolution)	GIS Grids	Geomatics
Summary of Investigations into BRY0129 CSO	Various reports and notes	United Utilities
Manhole survey (RPS Water 2010)	pdf	United Utilities
CCTV survey (2010)	jpg/mpg	United Utilities
Croal catchment sewer model	Infoworks CS	United Utilities
BRY0129 Flow Survey Data (Periods in February, March, April and May 2006)	Infoworks CS	United Utilities

3.2.1 Previous Flood Risk Investigations

UU have been aware of the problems at BRY0129 for some time and the CSO was identified as a UID for resolution within AMP5. Initial investigations to the site determined that the existing models were not reflecting the observed frequency of flooding at the site. As a result UU undertook a period of data collection, (including the flow, manhole and CCTV surveys detailed in Table 3-1), and further work to determine the flooding mechanism to the site.

The findings of the study suggest that the unauthorised overflow into the canal operates approximately 5 times per year and the BRY0129 CSO operates approximately twice a year. Surcharging of the system resulting in flooding at the Water Street junction occurs between a 100% and 50% AEP design flood event.

To calibrate the model to the observed level data, UU increased the silt levels within the culvert downstream of the gauge location. A blockage of up to 50% of the culvert was modelled to replicate observed levels leaving only 350mm of head room available within the culvert immediately upstream of the Manchester, Bury and Bolton Canal.

The study concluded that the constricted capacity of the downstream stormwater culvert was exacerbating flooding to the site.

UU investigated a series of options to reduce flooding from the combined sewers. Due to the interconnectivity of the combined and stormwater sewers these options required improvements to both systems and cost estimates for the work were prohibitively high given the funding available. It was concluded that the flooding is attributable to the capacity of the stormwater culvert and UU were unable to develop a solution in isolation.

The results of this investigation identified the SWMP as a potential route to take the investigations forward by actively engaging project partners and confirming the extent of additional funding available.

Lessons Learnt 2 – Previous Flood Risk Investigations
In this instance the survey data collected by UU and the associated understanding of the interconnectivity between the stormwater and combined systems as a result has proven invaluable. Further clarification of the flood risk mechanism has been possible given the good level of base data available.

3.2.2 Data Gaps

A review of the hydraulic model was completed to determine its suitability for further investigation. A summary of the findings are:

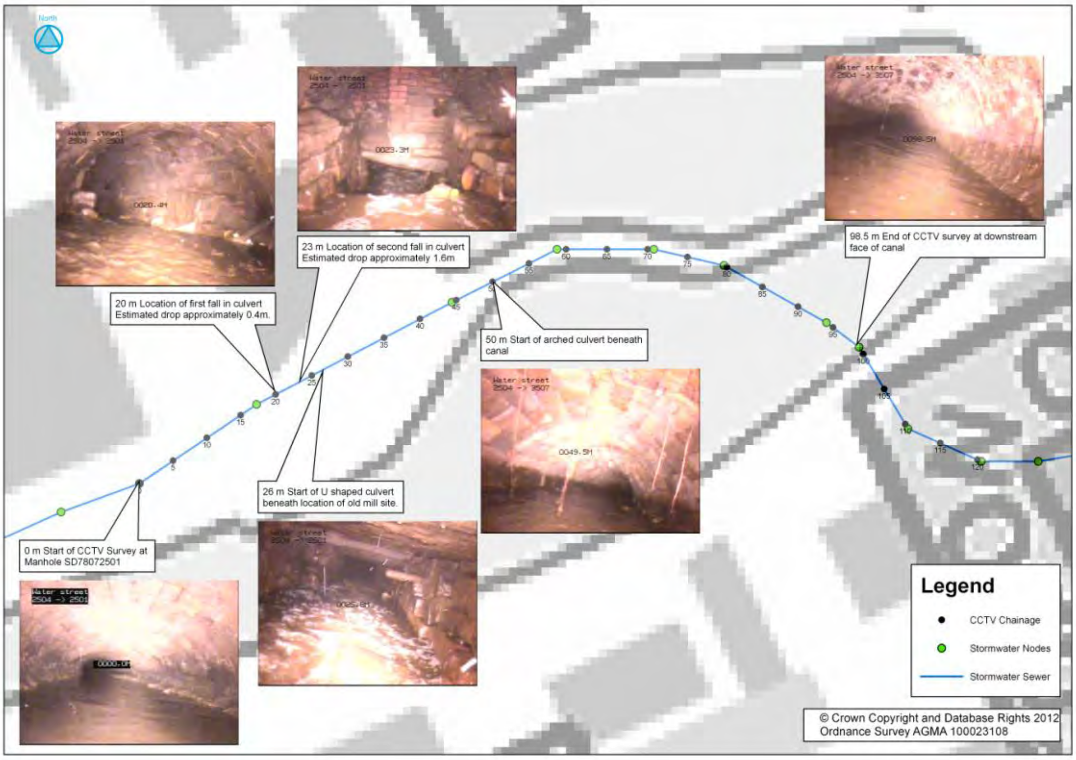
- There are discrepancies between the model and survey data regarding the depth of silt in the culvert and in some cases the dimensions of the culvert. Silt depths have been artificially included based on the flow survey and so may not represent the cause of the flow constriction. CCTV survey of the culvert to confirm the condition (structural and debris) is required.
- There is a significant drop within the model (3m) upstream of the canal. This structure has not been surveyed and dimensions are estimated only. Survey of this structure to confirm dimensions is critical given that this is believed to be the principal cause of flood risk.
- Subcatchments incorporating the area along the alignment of the culvert are not included within the model. Inflows from Crow Tree Farm Brook are not included within the model. To ensure flood risk is understood holistically the model should be updated to include inflows from Crow Tree Farm Brook.
- The culverted watercourse outfall into the Irwell has a free discharge potentially overestimating the capacity of the culvert in this reach. A review of the high water levels on the Irwell is required.

Lessons Learnt 3 – Data Gaps
The findings of the data gap analysis highlighted the importance of properly reviewing the assumptions in previous work. Whilst there has been significant investigation to the site, including the collection of calibration data, the difficulties associated with accessing key areas of the stormwater culvert means that there are some outstanding uncertainties in the system. Observed water levels within the culvert were achieved by applying blockage in un-surveyed reaches. The drop shaft in the culvert, a critical structure, is up to 200m away from the nearest survey location. The impact on the study of these uncertainties will be a reduction in the confidence of developed solutions if the cause of the flood risk cannot be specifically determined, Section 3.5.2.

3.3 Intermediate Assessment Ground Investigations

Following the initial data review it was determined that to provide a suitable level of confidence in recommendations for the study it was necessary to clarify the condition of the culvert upstream of the canal. A CCTV survey was completed on 6 October 2012 a summary of which is detailed in Figure 3-1.

Figure 3-1: Overview of CCTV Survey Findings



The survey included the culvert downstream of manhole SD78072501 to the downstream side of the canal. The following key points were identified as part of the survey:

- The drop in the culvert occurs within the car park upstream of the canal, the local landowner confirmed this used to be the site of a mill and the culvert passed beneath the lower floor of the mill (this has been confirmed on historical mapping).
- The culvert shape beneath the mill changes to a U shaped culvert before opening up into an arch culvert beneath the canal.
- There is no evidence of blockage and the culvert itself is in reasonable condition.

3.4 Intermediate Assessment Modelling Updates

Following the receipt of the CCTV survey it was necessary to undertake a series of updates to the model to incorporate the latest data.

In addition to this the review of the hydraulic model detailed in Section 3.2.2 as part of the data gap analysis identified a number of areas for improvement. These were associated with the catchment hydrology, notably including inflows along the alignment of the culvert, culvert dimensions where these did not reflect the survey and the inclusion of an appropriate downstream boundary. All these updates were combined as part of the intermediate assessment.

Model updates have focussed on the stormwater system only. Following the completion of the model updates a series of sensitivity tests have been undertaken on the model as a whole, including the combined system, to identify further modelling updates that may be required as part of the next stages of investigation. The results of these sensitivity tests are included in Appendix B.

To provide clarity in discussions relating to model updates Figure 3-2 and Figure 3-3 provide an overview of the model node labels on the stormwater system.

Figure 3-2: Model Node Labels Upstream of the Canal

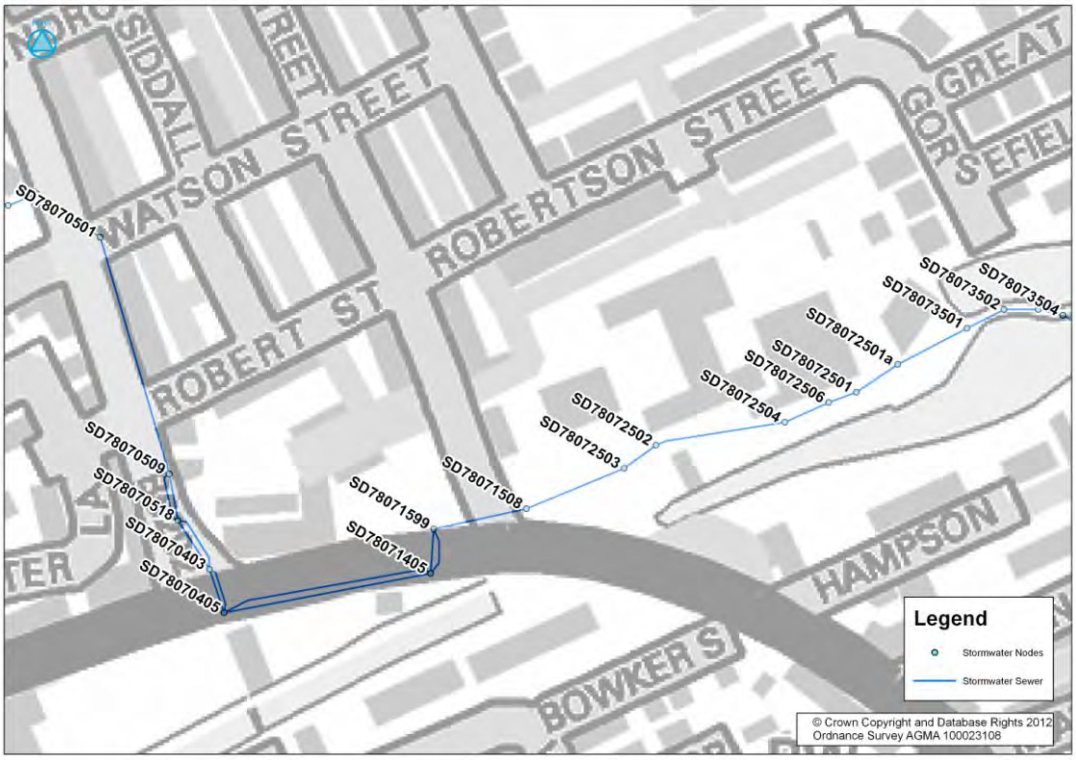
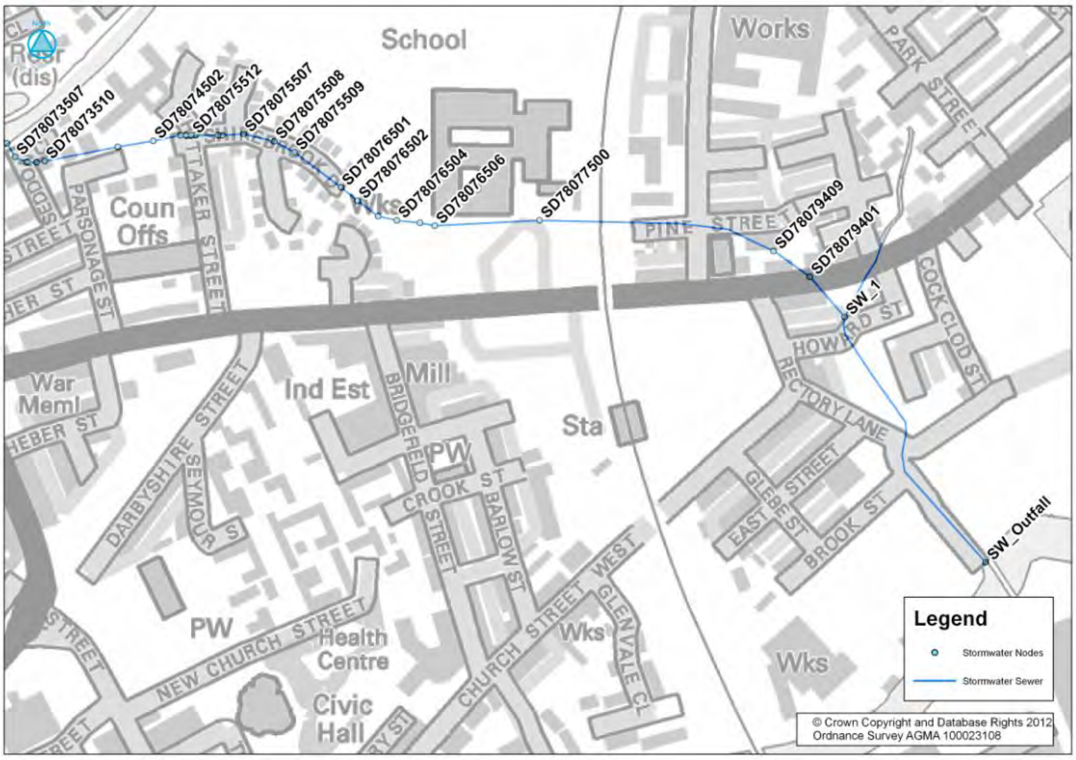


Figure 3-3: Model Node Labels Downstream of the Canal



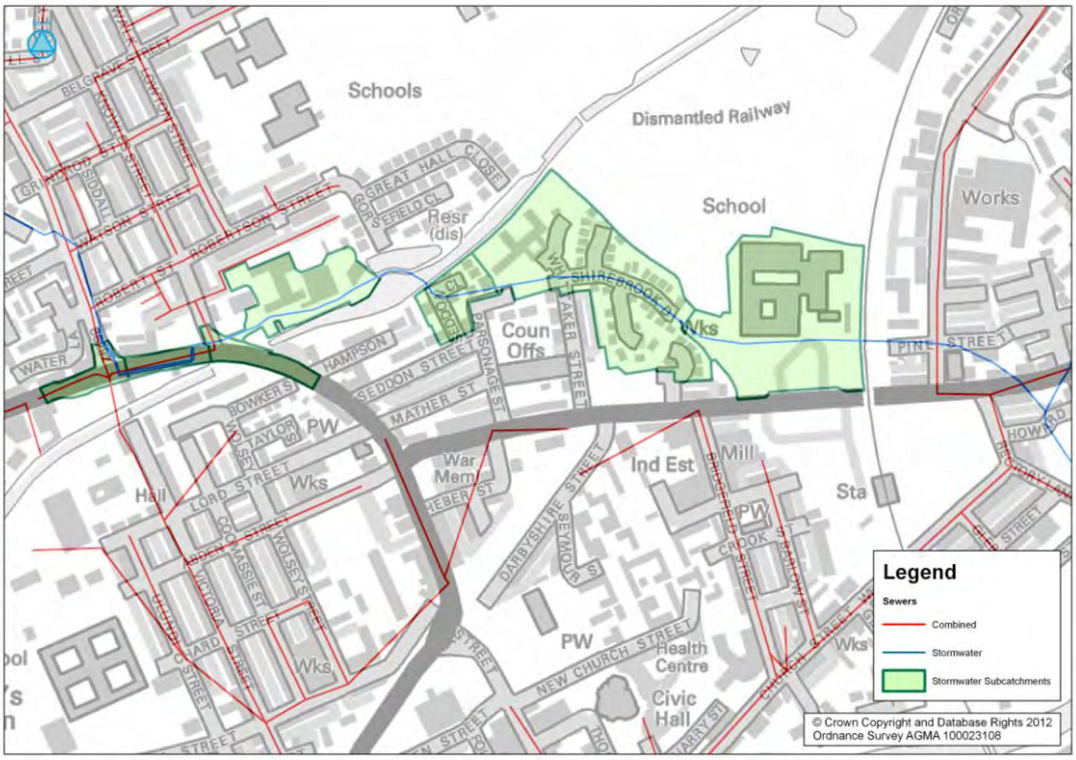
3.4.1 Incorporation of CCTV Survey

To reflect the CCTV data a fall of 2m has been included in the culvert downstream of manhole SD78072501a. Downstream of this fall the culvert has been amended to a U shaped culvert, dimensions 1100mmx700mm; the gradient of this reach has been increased to reflect observations and to tie into the existing levels beneath the culvert.

3.4.2 Stormwater Drainage Catchments

Figure 3-4 details the additional subcatchments delineated along the alignment of the stormwater culvert to calculate inflows to the hydraulic model. Within each of these sub-catchments the contributing areas associated with roads and pavements, roofs and open areas were extracted from mastermap data, impermeable fractions are therefore site specific. The existing runoff models used in the UU model were applied to these surfaces, i.e. fixed runoff for impermeable and New UK for permeable surfaces.

Figure 3-4: Additional Drainage Catchments included in the Model



3.4.3 Stormwater Culvert Dimensions

A review of the culvert dimensions within the model compared to the survey data available has been completed. This confirmed a good agreement with the exception of a short reach downstream of manhole SD78072502. In this location the survey data indicates the culvert is an arch with dimensions 950x2530mm (height x width), the model suggests an obstruction is present in the culvert a short distance downstream of the manhole which then reduces the culvert dimensions to an arch of 950x1920mm.

No data has been provided supporting the presence of the obstruction and it is considered conservative to assume the obstruction would be equivalent to a reduction in the culvert width for 42m, (the distance to the next model node). For the purposes of the assessment the culvert dimensions have been increased to reflect the survey data.

3.4.4 Crow Tree Farm Brook Design Inflows

Crow Tree Farm Brook discharges into the culvert in the vicinity of Howard Street approximately 200m upstream of the outfall into the River Irwell. This location has been identified through discussions with key stakeholders as potentially sensitive to flooding. Options that increase flows downstream should therefore consider how these flows will combine with runoff from Crow Tree Farm Brook.

An FEH analysis has been completed for this watercourse, the details of which are included in Appendix A. The results show reasonable agreement between the statistical and ReFH analysis with peak flows marginally higher using the ReFH. Design flows, Table 3-2, and hydrographs have been extracted for the critical storm durations identified in Section 4.3.1 using ReFH with no scaling factor applied.

Table 3-2: Design Flows for Crow Tree Farm Brook

Flood peak (m^3/s) for the following return periods (AEP)									
	50%	20%	10%	5%	4%	3.33%	2%	1.33%	1%
Winter	2.3	3.0	3.6	4.3	4.5	4.7	5.4	5.9	6.3
Summer	0.9	1.3	1.7	1.9	2.0	2.1	2.4	2.7	3.0

3.4.5 River Irwell Design Levels

The EA has provided design levels of the River Irwell for a range of return periods, Table 3-3.

Table 3-3: Peak Levels of the River Irwell at the Culvert Outfall

Peak Levels (m AOD) on the River Irwell for the following return periods (AEP)					
20%	10%	4%	2%	1.33%	1%
61.85	62.24	62.82	63.34	63.61	63.8

The critical storm for reviewing the capacity of the culvert in this location has been identified as a summer storm with 1hr duration, Section 4.3.1, reflecting the heavily urbanised nature of the catchment. The River Irwell catchment in comparison is approximately 365 km^2 in area upstream of the culvert outfall and is predominantly rural, suggesting the critical season for the Irwell is winter. It can be safely assumed that the likelihood of a joint event on both the River Irwell and the Water Street catchment is limited.

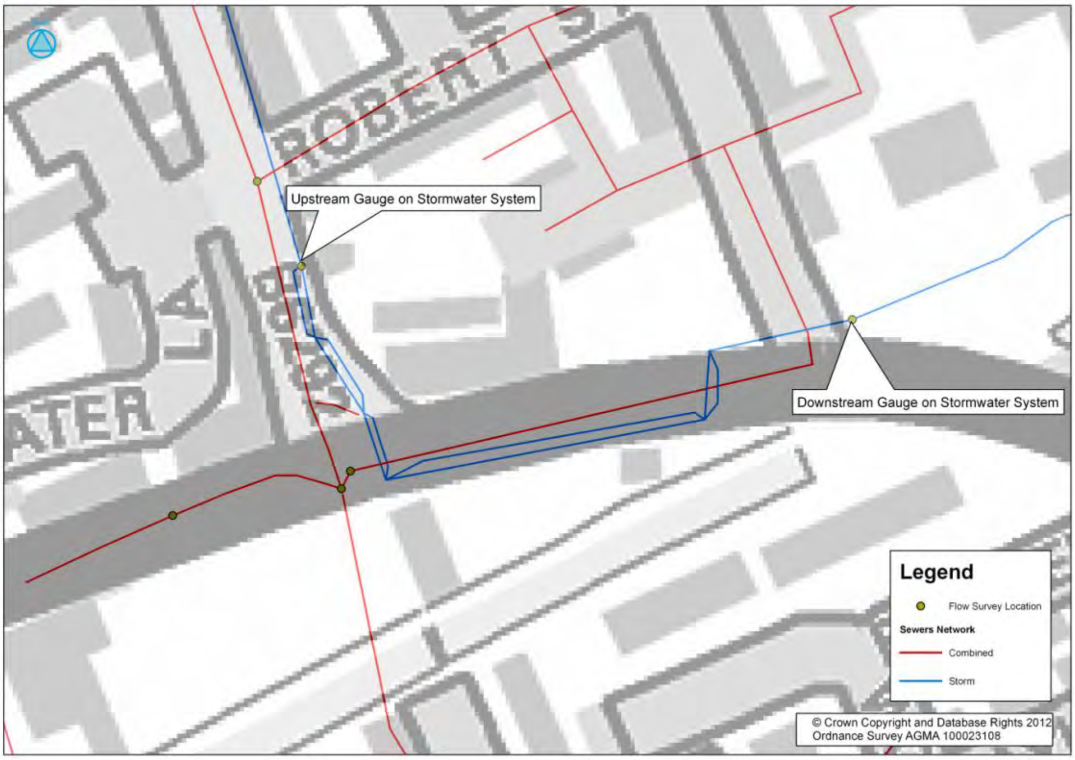
With no further data available a 20% AEP peak level has been applied for the downstream boundary on the River Irwell; this is considered to be conservative. A sensitivity test assessing the impacts of a higher downstream boundary is provided in Section B.5.9.

3.5 Revisit of Model Calibration

As part of UUs investigations into Water Street flow, depth and velocity data were collected at multiple locations within both the combined and stormwater networks over a series of events. Unfortunately throughout all the data collection period flows within the culvert did not get to levels where the BRY0129 CSO was in operation. As a result the calibration of the systems is applied separately with no confirmation of the operation of the system in larger events.

Two gauges were located on the stormwater culvert located upstream and downstream of the BRY0129, Figure 3-5.

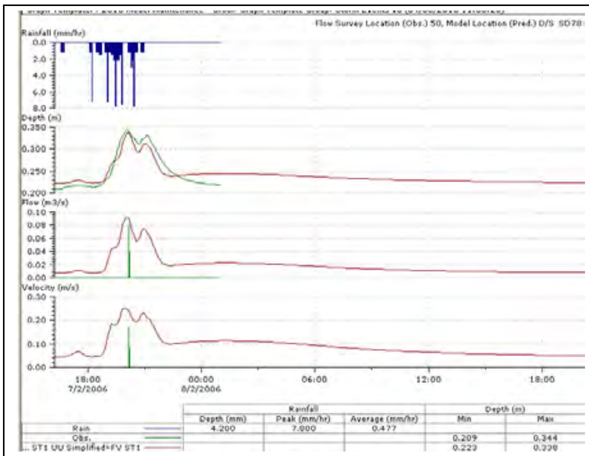
Figure 3-5: Location of Flow Survey Gauges



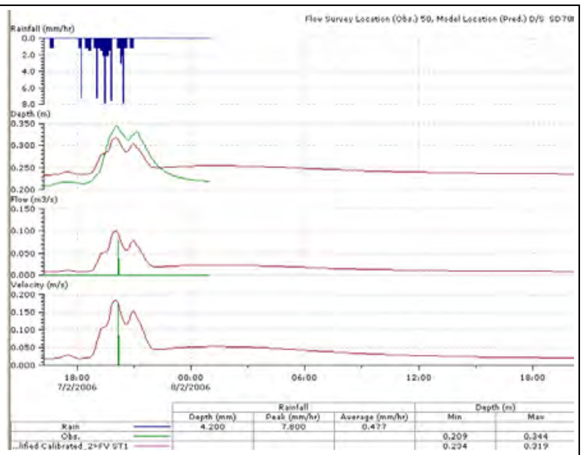
The calibrated model results from UU show good agreement at the upstream gauge with the observed data for both flow and level; the updates undertaken as part of these investigations have no impact at this site. Inflows to the site are therefore reasonable and the review of the calibration has focussed on the agreement between observed and modelled data at the downstream gauge site only.

Figure 3-6 details model calibration plots for three events from the UU model and the updated model. The gauge data is shown in green and the modelled data in red.

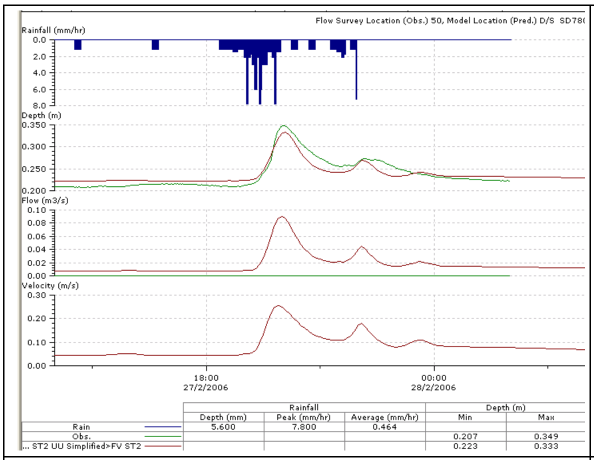
Figure 3-6: Comparison of Calibration Plots for a Series of Events



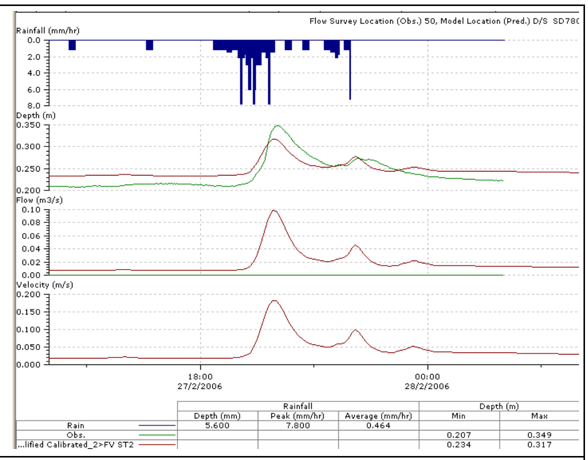
UU Model Flood Event ST1



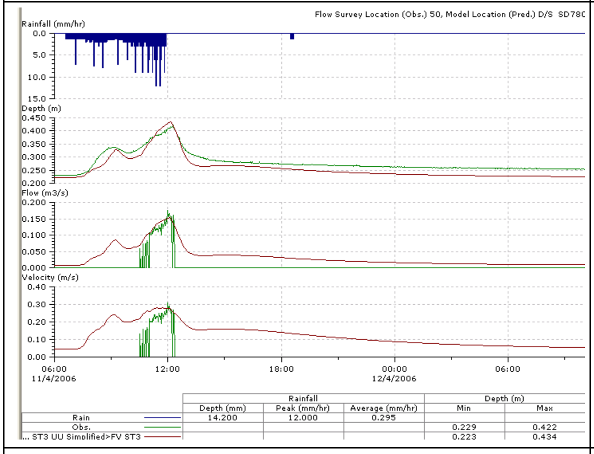
SWMP Model Flood Event ST1



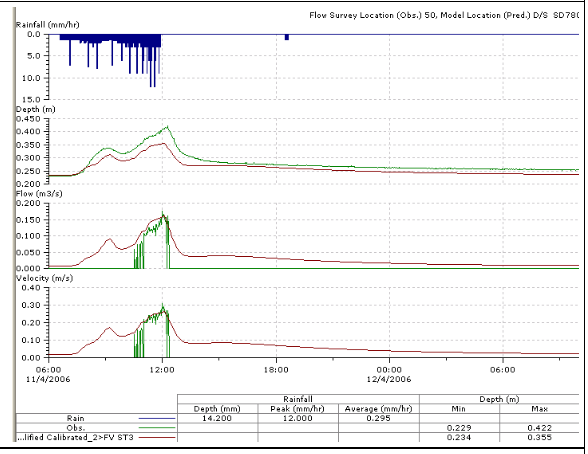
UU Model Flood Event ST2



SWMP Model Flood Event ST2



UU Model Flood Event ST3



SWMP Model Flood Event ST3

The calibration plots show that there is a ponded water level of approximately 0.2m at the gauge site. In the UU model this was assumed to be a result of debris build up in the downstream channel. The CCTV survey has confirmed this is currently not the case and instead levels have been achieved by applying a local silt depth in the pipe immediately downstream. An alternative explanation could be that the gauge was located at the upstream of pipe SD78071599 rather than the downstream as recorded; the invert level of this pipe increases approximately 0.2m from the upstream to the downstream and could explain the observed standing water depth.

The calibrated model is not as sensitive to flows; the rise in predicted water levels associated with the observed flows is not as great as in the UU model. The effect is a poorer agreement at the peak of the events than in the original UU model.

To further investigation the confidence in the model, flooding volumes from the system have been mapped and compared to the reported frequency of flooding at the site, Figure 3-7. Flooding volumes have been extracted from each of the hydraulic models described above and mapped using the 2D modelling software package JFlow+. This allows flooding depths and velocities across the site to be determined.

A sensitivity test has been completed to review the effect of excluding a hydrodynamic link with the 1D model as possible in Infoworks ICM but not in JFlow+, Appendix B. A good agreement was observed between outputs from both software packages.

Figure 3-7: Predicted Existing Risk in a 2hr Summer Storm from the UU and SWMP Models

	
Existing Risk UU Model	Existing Risk SWMP Model

The DG5 register suggests there are 4 properties at risk in the 20%AEP event (2 in 10yrs), 3 in the 10% AEP event and 7 in the 20% 5% AEP event. The UU model flood extents suggest properties are likely to be affected in the 50% AEP flood event. In the SWMP model flooding of the area occurs in the 50% AEP event but only starts to impact properties in the 20% AEP event which is more in line with the DG5 register. It is also noted that flooding of the car park to the south of Water Street is predicted in the 20% AEP event by the UU model; this is not shown in the photo of the July 2006 event, Figure 2-5 although the return period of this event is not known.

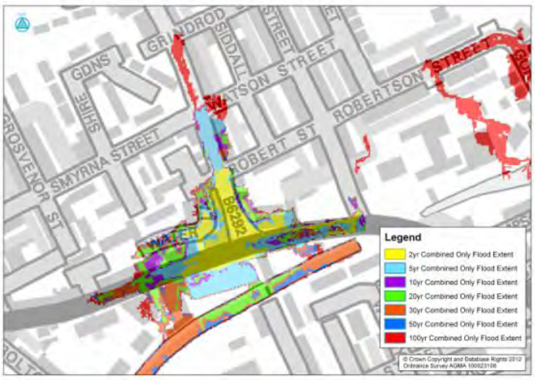
Efforts were made to model the July 2006 event but the local rainfall gauges do not appear to have picked up the event well. A single record of 2.4mm falling in 15 minutes on that day are all that is shown in the records which result in no flooding when modelled.

3.5.1 Review of Flooding Mechanism

The initial understanding of the cause of flood risk to the site assumed that deterioration of the stormwater culvert was resulting in surface water backing up into the combined system and causing flooding. The CCTV survey has indicated that the stormwater culvert is in fact in reasonable condition with no blockages observed.

To understand how the capacity of the stormwater culvert is impacting the combined system the CSO has been removed from the model and the predicted flood extent from both systems independently reviewed, Figure 3-8. A model run has also been completed of the existing system with no stormwater inflows, effectively allowing a free discharge from the combined system into the stormwater culvert.

Figure 3-8: Predicted Existing Risk in a 2hr Summer Storm from the Stormwater and Combined Systems

	
Combined System with no CSO Discharge	Combined System with free CSO Discharge

Stormwater System with no CSO discharge

The modelling results indicate that the majority of flood risk is from the combined system. Even with a free CSO discharge, flooding from the combined system would be expected to impact properties in the 20% AEP event. It is also apparent that stormwater system has sufficient capacity up to the 3.33% AEP event to carry the stormwater only flows if these were not backing up into the combined system.

It is concluded that the stormwater culvert is sufficiently sized for stormwater only flows but does not have the capacity to take the additional flows from the combined system. Because the combined sewer is located at a lower level than the stormwater sewer it is inevitable that the stormwater sewer will backup into the combined sewer before flooding occurs, however the main issue is the limited capacity of the culvert is preventing the sum of the combined and stormwater sewer flows discharging.

3.5.2 Modelling Uncertainty

To assess the uncertainty in the current model a suite of sensitivity tests have been carried and are detailed in Appendix B.

Two main areas of uncertainty have been identified as a result of these tests, the antecedent conditions within the model and the rainfall parameters within the model.

The New UK runoff model requires API30 value for the initial conditions; this is set to 17 within the current model. For the sensitivity test the value has been reduced 7, as appropriate for a summer storm on a catchment with SAAR values in the region of 1100, as extracted from the WaPUG paper by Jamie Margetts[2].

The rainfall parameters within the model have been extracted from a catchment with area 33km^2, significantly larger than the local drainage catchment. A smaller catchment, 1.52km^2 in area, that roughly coincides with the drainage area of interest has been extracted from the FEH.

In both of the above cases the predicted flood risk at the Ainsworth Road and Water Street junction is predicted to decrease but the most significant implications are associated with the initial conditions. It is recommended a more detailed assessment is completed to determine the design API30 as part of further studies.

In addition to the findings of the sensitivity tests there are number of model limitations identified as part of the intermediate assessment.

The largest uncertainty remains the dimensions of the culvert and the level of siltation within the system. The CCTV survey has clarified uncertainties in the reach downstream of SD78072501 to the downstream side of the canal. There remains however an observed standing water depth of 0.2m in the culvert at the location of the gauge site which is attributed to an unknown cause in the unsurveyed reach of the culvert.

2 If The NAPI Fits....., WaPUG Spring Conference 2002, Jamie Margetts.

The flow and level survey whilst extremely useful did not coincide with an event where the CSO became operational. This would have been useful to verify to operation of the CSO and hence interconnectivity of the combined and stormwater systems. It is also noted the flow and level survey now dates back to 2006; there is significant scope in the intervening years for the conditions within the culvert to have changed.

4 Detailed Risk Assessment

4.1 Overview

The detailed risk assessment has focussed on confirming the impacts of flooding to the site in the existing situation.

The key issue for the Water Street site is the multi-source nature of the flooding and determining an appropriate approach to allocate flood risk between these sources. Feedback from the EA suggests funding for flood risk management through the FDGiA process can be justified only on surface water flooding; it is not appropriate to include flood risk from the combined sewer and funding for these works will have to be identified elsewhere. The full extent of flood risk however may not be reflected by considering these sources independently.

4.2 Modelling Approach

In determining how to represent each system separately it is important to consider how each of the systems benefits/suffers in the existing setup.

Because of the layout of the system the stormwater system will back up into the combined system, surcharging both systems, before flooding occurs. Extracting flood volumes from each manhole in this instance could underestimate flood risk from the stormwater system and overestimate flood risk from the combined system. In this instance it seems an appropriate representation of the stormwater flood risk would be to separate the system by removing the CSO and hence the link into the combined system.

The combined system, assuming no flow constriction on the stormwater culvert, will discharge flows through the unauthorised overflow into the canal and the BRY0129 CSO into the stormwater system before it becomes surcharged and flooding occurs. In this instance it seems appropriate from a flood risk perspective to allow the unauthorised overflow to continue operating in its existing manner but to remove the CSO from the system altogether. Allowing the CSO to operate as a free discharge would underestimate the flood risk from the combined system and in the existing scenario would increase flood risk as stormwater flows enter the combined system.

Based on the above, flood risk from three scenarios will be assessed:

- Flooding from the stormwater system only with the CSO removed
- Flooding from the combined system with the unauthorised overflow included and the CSO removed.
- Flooding from both systems interacting as described in the existing model

4.3 Current Risk

4.3.1 Review of Critical Storm Durations

Prior to the assessment a review of the critical storm duration has been completed. The review focussed on the changes in flooding volume from manholes in the vicinity of the Ainsworth Road and Water Street junction and the peak flow rate to the site observed in the both the stormwater and combined systems upstream of the site.

A 3.33% AEP design storm has been used for this analysis to reflect the likely design standard of proposed solutions. The critical storm duration for peak flow rates in the 3.33% AEP in both the combined and stormwater systems is the summer storm with duration of 1hr.

Table 4-1 summarises the flooding volumes for a range of storm durations return associated with the 3.33% AEP return period event and Table 4-2 summaries the flooding volumes associated with a short duration summer and winter storms for a range of return periods.

Table 4-1: Flooding Volumes in the 3.33% AEP Event for a Range of Storm Durations

Flooding volume (m³) for the following durations (minutes)						
Scenario	**30**	**60**	**90**	**120**	**180**	**360**
Summer Storm	1571	2127	2260	**2304**	2186	1664
Winter Storm	1762	2361	2513	**2519**	2215	1120

Table 4-2: Flooding Volumes for a Short Duration Storms for a Range of Return Periods

Flooding Volume (m³) for the following return periods (AEP)							
Scenario	**50%**	**20%**	**10%**	**5%**	**3.33%**	**2%**	**1%**
Summer 1hr Storm	50	403	871	1575	2127	2973	4388
Summer 2hr Storm	36	419	927	1691	2304	3352	5120
Winter 2hr Storm	0	314	891	1789	2519	3669	5727

The largest flood volume in the 3.33% AEP event was identified as a winter storm with duration of 2hrs. The largest predicted flood volume associated with a summer storm in the 3.33% AEP event was also associated with a duration of 2hrs.

The threshold event for the winter storm is the 20% AEP compared to the 50% AEP event in the summer storm of equivalent duration. In addition to this flood volumes in the winter event do not exceed the summer event volumes until the 5% AEP event. This reflects the trade off between higher peak flows generated from summer storms exceeding the sewer capacity but causing limited flooding in frequent events against longer hydrographs generated from winter storms causing grater flooding in less frequent events.

For the purpose of this study the summer 1hr storm duration event has been used when assessing the capacity of the culvert (Section 5.1). The summer storm profile is more appropriate for a 2hr duration storm and as such this represents a more consistent description of a design storm event. The summer 2hr storm has been used when assessing flooding impacts and damages (Section 4.3.3), and peak storage volumes (Section 5.1.3).

4.3.2 Current Flood Risk Extents

Flood extents from the stormwater and the combined system separately are detailed in Figure 3-9 in Section 3.5.1. Both in combination are detailed in Figure 3-8 in Section 3.5.

The numbers of properties affected for each scenario are detailed in Table 4-3.

Table 4-3: Number of Residential and Commercial Properties at Risk from Flood Sources

Number of Properties at Risk for the following return periods (AEP)							
Scenario	**50%**	**20%**	**10%**	**5%**	**3.33%**	**2%**	**1%**
Stormwater Only							
Residential	0	0	0	0	0	0	5
Commercial	0	0	0	0	0	2	9
Combined Only							
Residential	3	8	8	13	13	13	16
Commercial	6	14	20	23	26	27	29
Existing Risk from Stormwater & Combined							
Residential	1	8	17	24	36	39	41
Commercial	1	12	17	22	25	28	32

4.3.3 Future Flood Risk Extents

To provide an indication of the increase in flood risk associated with climate change, the rainfall intensity has been increased by 20%. This is equivalent to the average change expected up to 2100[3].

Flood extents associated with the 2hr summer storm are shown in Figure 4-1 and a comparison of flooding volumes is shown in Table 4-4.

Figure 4-1: Sensitivity to Climate Change Flood Extents

Table 4-4: Sensitivity to Climate Change Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m^3), Summer 2hr Duration Event	Sensitivity to Climate Change Flood Volumes (m^3), Summer 2hr Duration Event	Percentage Variation from Calibrated Model (%)
50%	36	271	653%
20%	419	864	106%
10%	927	1629	76%
5%	1691	2763	63%
3.33%	2304	3638	58%
2%	3352	5019	50%
1%	5120	7336	43%

The effect of climate change on flood risk to the site is significant. The system is already at capacity in this location and additional flows into the system have an immediate impact at the site. The assessment highlights the potential scale of the problem going into the future with

[3] Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities, Environment Agency

increases in flood volumes in the 3.33% AEP event in excess of 50% of the existing flooding volumes.

4.3.4 Damage Assessment

A provisional damage assessment has been completed using damage curves from the "multicoloured manual"[4] (MCM). The National Receptor Database (NRD) has been used as the base dataset for the analysis.

The assessment has focussed on existing flood risk to determine the extent of the problem. Flooding depths have been extracted from the results of JFlow modelling routing the flooding volumes from the scenarios described.

No property threshold level above the LIDAR level has been assumed based on observations at the site where properties open directly onto the pavement. A summary of the Present Value (PV) damages, the value when discounted to the present time at the rates below, is shown in Table 4-5, Damages have been discounted over 100 yrs using discount rates of 3.5%, 3.0% and 2.5% at years 0, 30 and 75 respectively as set by the Treasury.

Capping values for both residential and commercial properties have been estimated at £100,000 based on a high level review of local sold property prices and approximate rateable values as detailed on the Valuation Office Agency website.

Table 4-5: Damages associated with Flood Sources

Scenario	Average Annual Damages (AAD) (£k)	Present Value (PV) Damages (£k)	Capped Present Value (PV) Damages (£k)
Stormwater Only	2	45	45
Combined Only	105	3,117	1,934
Existing Risk from Stormwater & Combined	83	2,480	1,962

Flood risk at the site is dominated by the surcharging of the combined system and a standard of protection equivalent to the 3.33% AEP event has therefore been assumed. To estimate the residual risks associated with a design flood in excess of this before options have been reviewed in detail, and so provide an indication of the scale of the benefits associated with any scheme, an approximation using available flood extents has been completed, Table 4-6. For this assessment it has been assumed that the residual risks post scheme in the 2% and 1% AEP events will be equivalent to the 5% and 2% AEP events in the existing scenario respectively. This has been completed for the Existing Risk scenario only.

Table 4-6: Estimated Benefits Associated with a 3.33% AEP Design Standard

Benefits	Existing Risk	Residual Risk
AAD (£k)	83	18
PV Damages (£k)	2,480	525
Capped PV Damages (£k)	1,962	525
PV Flood Benefits (£k)	-	1,437

This is a high level assessment only and the actual benefits of any scheme will need to be investigated further in detail before the full benefits can be assessed. However it does provide an indication of the scale of benefits for comparison against expected costs and so allows a sensibility check to be completed against any schemes that are thought to be prohibitively expensive.

[4] The Benefits of Flood and Coastal Risk Management: A Handbook of Assessment Techniques, (Middlesex University Press 2010).

4.3.5 Damage Sensitivity

4.3.5.1 Threshold Level

A sensitivity assessment has been completed assuming a threshold level of 150mm is present at all properties, Table 4-7.

Table 4-7: Damages associated with Flood Sources Sensitivity to Thresholds

Scenario	Average Annual Damages (AAD) (£k)	Present Value (PV) Damages (£k)	Capped Present Value (PV) Damages (£k)
Stormwater Only	0	2	2
Combined Only	27	811	792
Existing Risk from Stormwater & Combined	19	577	577

Flood extents are generally shallow and the predicted damages are therefore sensitive to reducing flooding depths by 150mm. A threshold level survey would improve confidence in the damage assessment.

4.3.5.2 Storm Duration

A sensitivity assessment has been completed assuming a summer storm with duration 1hr, Table 4-8.

Table 4-8: Damages associated with Flood Sources Sensitivity to Storm Duration

Scenario	Average Annual Damages (AAD) (£k)	Present Value (PV) Damages (£k)	Capped Present Value (PV) Damages (£k)
Stormwater Only	1	31	31
Combined Only	93	2,781	1,836
Existing Risk from Stormwater & Combined	91	2,719	2,183

The results show increased damages associated with the 1hr storm suggesting this event should be used in future damage assessments.

5 Identifying Measures

5.1 Hydraulic Requirements

To provide the background information required to assess potential management measures to the site a short summery of the hydraulic requirements and existing capacity of the stormwater culvert is provided below. As detailed previously the critical storm for peak flows is a summer storm with a duration of 1hr.

5.1.1 Culvert Upstream of the Manchester, Bury and Bolton Canal

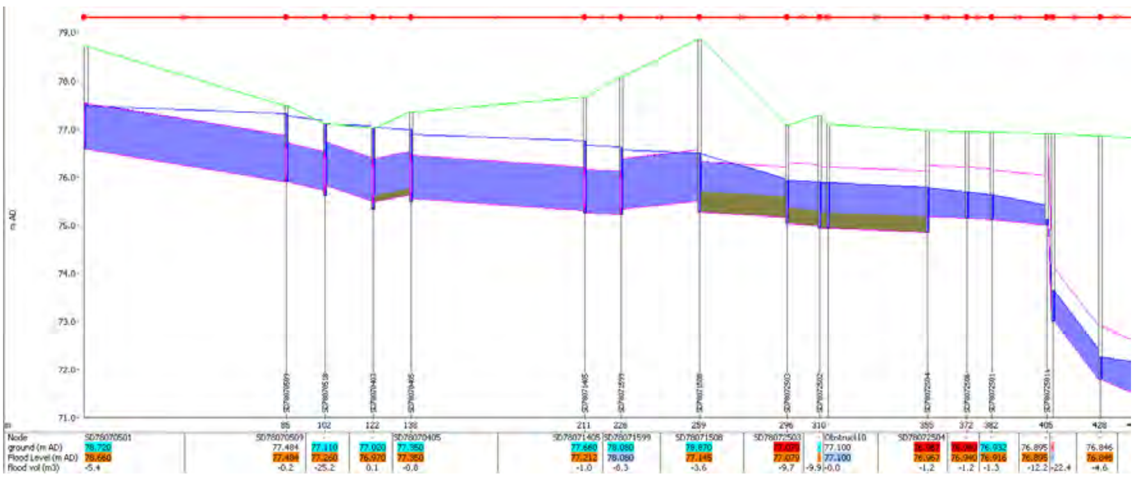
Peak flows into the system have been extracted from the stormwater system upstream of the CSO and from the existing CSO with no stormwater flows in the downstream culvert for the 1hr summer storm duration, equivalent to the critical event for all return periods to 1 decimal place. Hydrographs from both these scenarios have then been summed with additional inflows from stormwater catchments downstream of the CSO to estimate the maximum required capacity of the stormwater culvert for the existing system, Table 5-1.

Table 5-1: Peak Discharge Requirements within the Stormwater System

Flood peak (m^3/s) for the following return periods (AEP) for a 1 hr summer storm							
	50%	20%	10%	5%	3.33%	2%	1%
Stormwater (Pipe SD77079607.1)	0.8	1.0	1.1	1.2	1.4	1.5	1.7
Existing CSO (Weir (DUM_B.2)	0.3	0.4	0.4	0.4	0.4	0.4	0.4
Stormwater & Existing CSO exc Stormwater Catchments DS CSO	1.0	1.3	1.5	1.6	1.7	1.9	2.1
Stormwater & Existing CSO inc Stormwater Catchments DS CSO	1.1	1.4	1.5	1.7	1.8	2.0	2.3

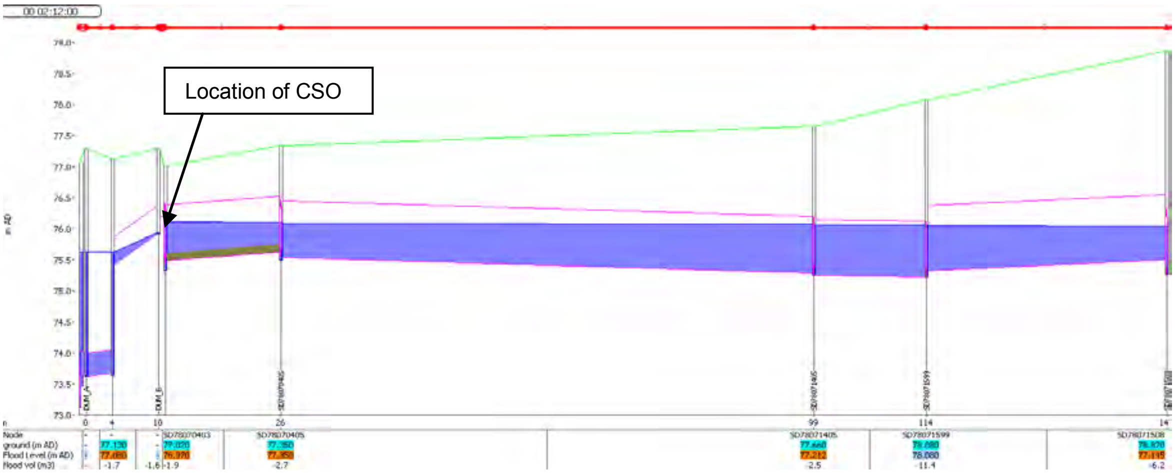
A review of the existing hydraulic capacity of the culvert has been completed using the calibrated model. The 1% AEP flood event has been run assuming all flows exceeding ground level are lost from the system; this provides an indication of the capacity of the culvert with water levels in manholes to ground level only, Figure 5-1.

Figure 5-1: Peak Water Levels in the Stormwater Culvert



The capacity of the existing system at the onset of flooding is 1.3 m^3/s immediately downstream of the CSO suggesting anything above the 20% AEP will result in flooding. In reality due to the presence of the CSO flows discharge back into the combined system delaying observed flooding from the stormwater system until the 5% AEP event. The capacity of the culvert at the threshold of the CSO, recorded in the model as at 76.2 mAOD has also been assessed. The capacity of the system with water levels below the threshold of the CSO is closer to 1.0 m^3/s, Figure 5-2.

Figure 5-2: Peak Water Levels in the Stormwater Culvert at the CSO Threshold



From Figure 5-2 it can be seen that the flooding of the combined system from the stormwater system is a result of the shallow gradient to manhole SD78071508, which has an invert level of 75.27 mAOD.

5.1.2 Culvert Downstream of the Manchester, Bury and Bolton Canal

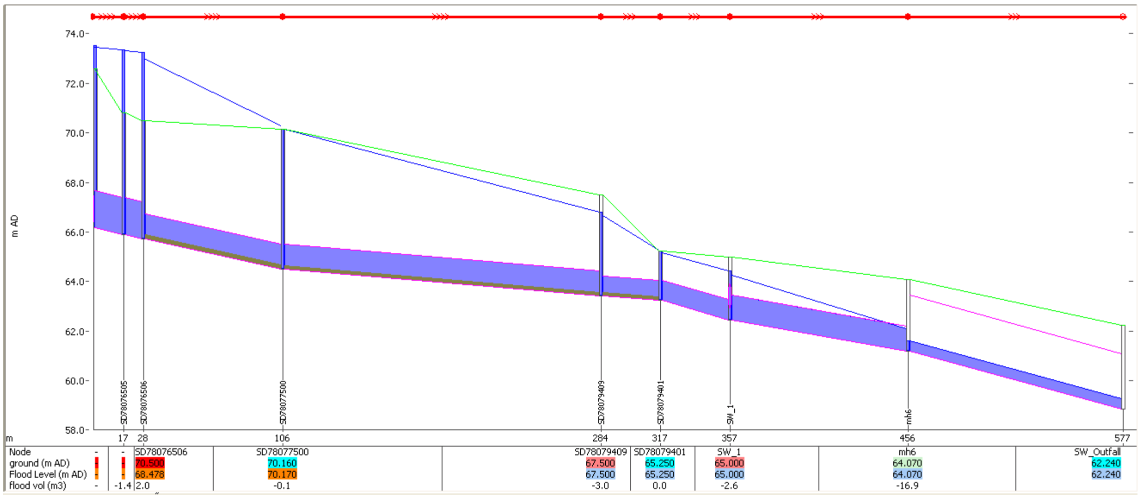
A similar assessment has been completed for the culvert at the downstream end. Peak flows have been extracted assuming all inflows from the stormwater system and CSO are passed downstream, Table 5-2.

Table 5-2: Peak Discharge Requirements within the Stormwater System Downstream of the Canal

Flood peak (m^3/s) for the following return periods (AEP) for a 1 hr storm duration							
	50%	20%	10%	5%	3.33%	2%	1%
Peak flows at DS limit of culvert (SD78079401)	1.2	1.5	1.8	2.1	2.4	2.8	3.4

In this instance due to the significant increase in ground levels moving up the culvert from the outfall there is a significant hydraulic head which increases the capacity of the culvert above what would be expected based on the dimensions alone. Peak flows have been extracted from the model assuming the hydraulic grade line reaches local ground levels at manhole SD78077500, i.e. at the onset of flooding, Figure 5-3. The nodes shown upstream of this manhole in the model are sealed and so will not flood. It is assumed that these nodes have been included for modelling purposes only and in fact there is no manhole shaft in this location, if a manhole shaft and cover are present the flow capacity will reduce. No downstream boundary or inflows from Crow Tree Farm Brook are included in this assessment.

Figure 5-3: Hydraulic Grade Line in Downstream Reaches of the Culvert



The pinch points in the system can be seen to be the culverts downstream of SD78077500 and SD78079401. Flows within the system at this point are approximately 3.8 m^3/s.

5.1.3 Existing Flood Volumes in the Combined System

A review of the existing storage requirements in the combined sewer system has been completed for a 2hr summer storm. This event is the critical duration for the 10% to the 2% AEP events only. Four scenarios have been assessed, Table 5-3:

- Scenario 1 - Flooding from the combined system in the existing scenario in which there is some degree of backflow from the stormwater system
- Scenario 2 - Flooding from the combined system in the existing scenario with no stormwater inflows, i.e. a free CSO discharge into the stormwater culvert
- Scenario 3 – Scenario 1 with the unconsented overflow removed.
- Scenario 4 – Scenario 2 with the unconsented overflow removed

Table 5-3: System Storage Volume Requirements in the Combined Sewer

Flood volumes (m^3) from the combined system for the following return periods (AEP) for a 2 hr storm duration				
	10%	5%	3.33%	2%
Scenario 1	927	1691	2304	3352
Scenario 2	724	1270	1682	2353
Scenario 3	1940	3088	3910	5150
Scenario 4	1583	2448	3077	4080

The volumes detailed in Table 5-3 represent a simplistic understanding of storage requirements due to the fact that multiple combined sewer branches converge immediately downstream of the BRY0129 CSO but it is indicative of the level of storage required in the system.

These volumes are significantly lower than the original estimates provided by UU on the understanding that the stormwater sewer was blocked, which were of the order of 16,000 m^3.

5.1.4 Improvements to the Combined Sewer Overflow

As part of the investigations already undertaken by UU options to increase the capacity of the existing CSO and hence the loading on the stormwater culvert downstream have been reviewed.

Any option that increases downstream flows will need to be discussed and agreed with all project stakeholders, however within the scope of the SWMP it has been agreed to review the feasibility of increasing the discharge from the CSO and the potential impacts on downstream sites.

UU have provided inflows to the stormwater culvert from the CSO for the 3.33% AEP event and 1% AEP event for a range of storm durations for the assessment, Table 5-4.

Table 5-4: Peak Discharge Requirements within the Stormwater System

Flood peak (m^3/s) for the following return periods (AEP) for a 1 hr summer storm							
	50%	20%	10%	5%	3.33%	2%	1%
Existing CSO	0.3	0.4	0.4	0.4	0.4	0.4	0.4
Proposed CSO	-	-	-	-	3.3	-	4.4
Stormwater & Existing CSO inc Stormwater Catchments DS CSO	1.1	1.4	1.5	1.7	1.8	2.0	2.3
Stormwater & Improved CSO inc Stormwater Catchments DS CSO	-	-	-	-	4.7	-	6.3

5.1.5 Hydraulic Requirements Summary

Due to the shallow gradient of the stormwater culvert the capacity of the system before water backs up into the combined system is approximately 1.0 m^3/s, this is equivalent to the 50% AEP event.

A provisional estimate of the required capacity below a level of 76.2 mAOD to prevent flows discharging into the combined sewer in the 3.33% AEP event is 1.8 m^3/s.

The maximum capacity of the culvert downstream of the canal is 3.8 m^3/s but this could be reduced by inflows from Crow Tree Farm Brook.

Required storage volumes in the combined sewer system are in the region of 4,000 m^3 if the unauthorised overflow is removed.

Whilst there may be some scope to increase pass forward flows the capacity of the culvert downstream is insufficient for all additional flows required by the upgraded CSO proposed by UU.

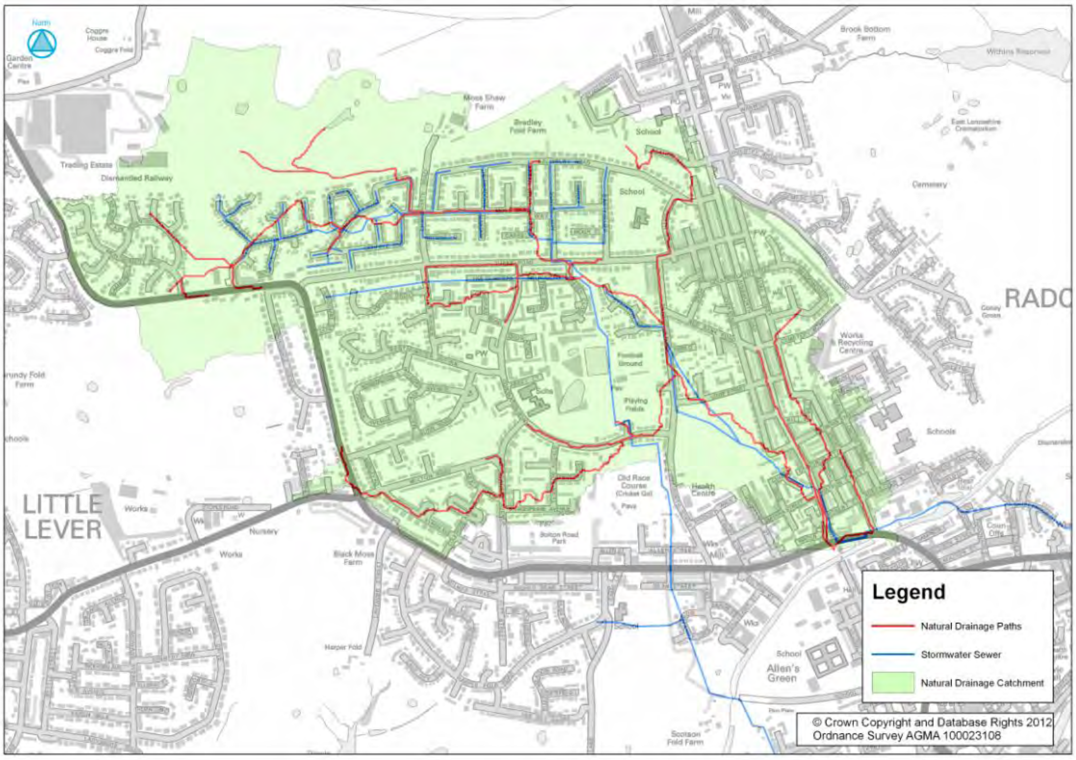
5.2 Long List of Options

5.2.1 Catchment Context

In determining appropriate management options for the site it is necessary to consider the wider catchment and the potential implications that any management options will have.

A review of the catchment drainage area has been completed using LIDAR data. This has generated the natural flow routes within the catchment, Figure 5-4, and highlights the fact that the stormwater system in general reflects the natural drainage path of the catchment. It is also apparent that the key reach of interest downstream of Water Street and in particular the route beneath the canal represents a pinch point in the catchment. Site visits confirm the Ainsworth Road and Water Street junction sits in a natural bowl with the lowest lying topography and hence most likely drainage route away from the site situated to the south towards the canal.

Figure 5-4: Overview of Natural Drainage Path



It is also worth considering the potential implications of management options that increase flows downstream. The River Irwell is the receiving watercourse for runoff from the catchment, itself a highly sensitive watercourse to increases in flows downstream in Manchester. Existing flood management schemes within Manchester will be unable to contain multiple incremental increases in runoff resulting from management schemes at small sites such as this. Options that relieve flood risk by increasing pass forward flows should consider storage compensation further downstream on the Irwell to mitigate the impacts in central Manchester. This will need to be agreed in conjunction with the EA and relevant local authorities.

5.2.2 Option Appraisal

5.2.2.1 Catchment Management

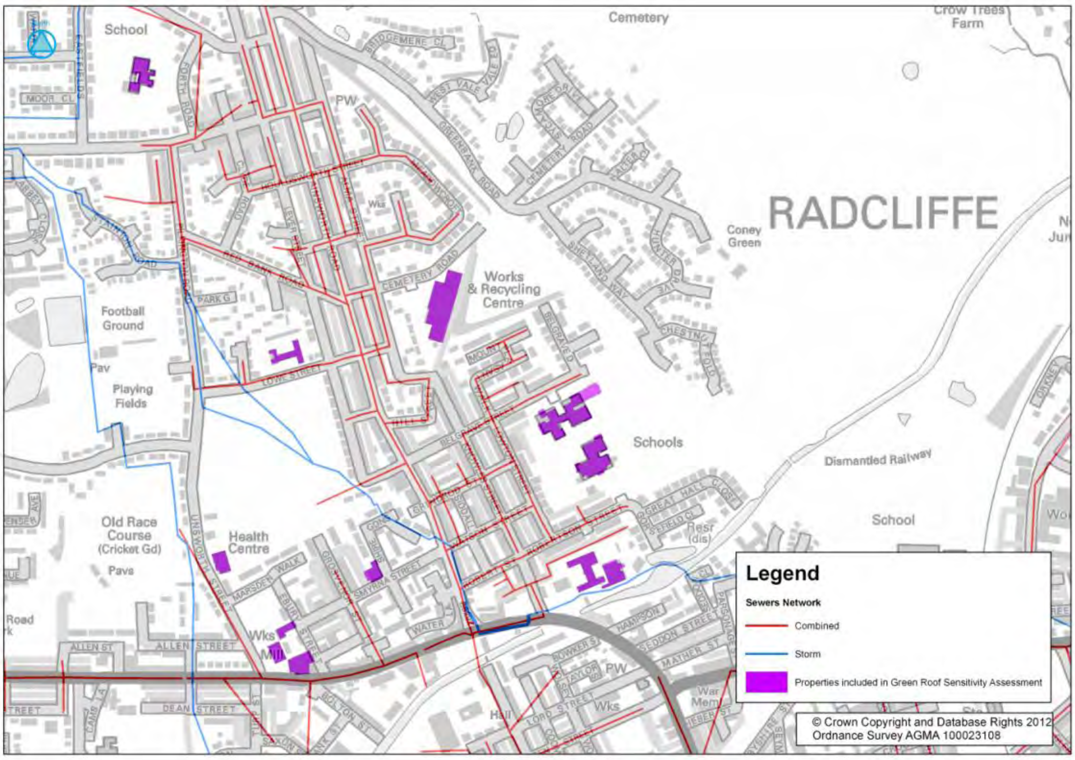
The current understanding of the site can be summarised as a hydraulically limiting culvert asset located at a pinch point along the alignment of the natural drainage path. Long term with climate change, the flows reaching the culvert and associated flood risk will increase significantly, Section 4.3.3, not accounting for ongoing development pressures highlighted in discussions with UU and Bury MBC.

Given the sensitivity of the downstream Irwell within Manchester to increases in flows the preferred long term measures for the site will manage runoff within the catchment responsibly and sustainably. This will require all new development in the catchment to maintain existing levels of runoff as a minimum; given the sensitivity of the site and the 'locked in' increases expected from climate change it is advisable that developments are required to go beyond this and reduce runoff from existing rates.

The core strategy identifies the catchment as a critical drainage area and as such development within the catchment is limited. Regeneration of central Radcliffe provides the only opportunity to incorporate SUDS into the catchment. To assess the potential benefits associated with improvements to managing runoff within the catchment through SUDS a number of sensitivity runs have been undertaken, these provide an indication of the reduction in flooding at the Water Street junction that can be achieved.

The catchment is generally residential however there are a number of large industrial units and schools that could be retrofitted with green roofs to minimise runoff from these sites. Figure 5-5 details large properties within the catchment where this may be possible.

Figure 5-5: Green Roof Sensitivity Assessment



For the purposes of the assessment it is assumed the runoff response from these large buildings is removed from the system. The areas associated with impermeable runoff from roofs within each respective drainage catchment have been adjusted within the model to reflect this and the impacts on flood volumes assessed, Table 5-5.

Table 5-5: Sensitivity to Green Roofs Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m^3), Summer 2hr Duration Event	Sensitivity to Green Roofs Flood Volumes (m^3), Summer 2hr Duration Event	Percentage Variation from Calibrated Model (%)
50%	36	6	-83.3%
20%	419	340	-18.9%
10%	927	819	-11.7%
5%	1691	1539	-9.0%
3.33%	2304	2139	-7.2%
2%	3352	3106	-7.3%
1%	5120	4888	-4.5%

There are a number of sites where stormwater systems are in place but that eventually drain into the combined system and eventually to the Water Street junction, Figure 5-6. Management of stormwater runoff from these sites locally will reduce the pressure on the combined system and reduce flooding at the site of interest. Again for the purposes of the assessment it has been assumed that the runoff response from these catchments will be removed from the system. Table 5-6 details the effect on flood volumes at the Water Street site of this change.

Figure 5-6: Small Catchment SUDS Sensitivity Assessment

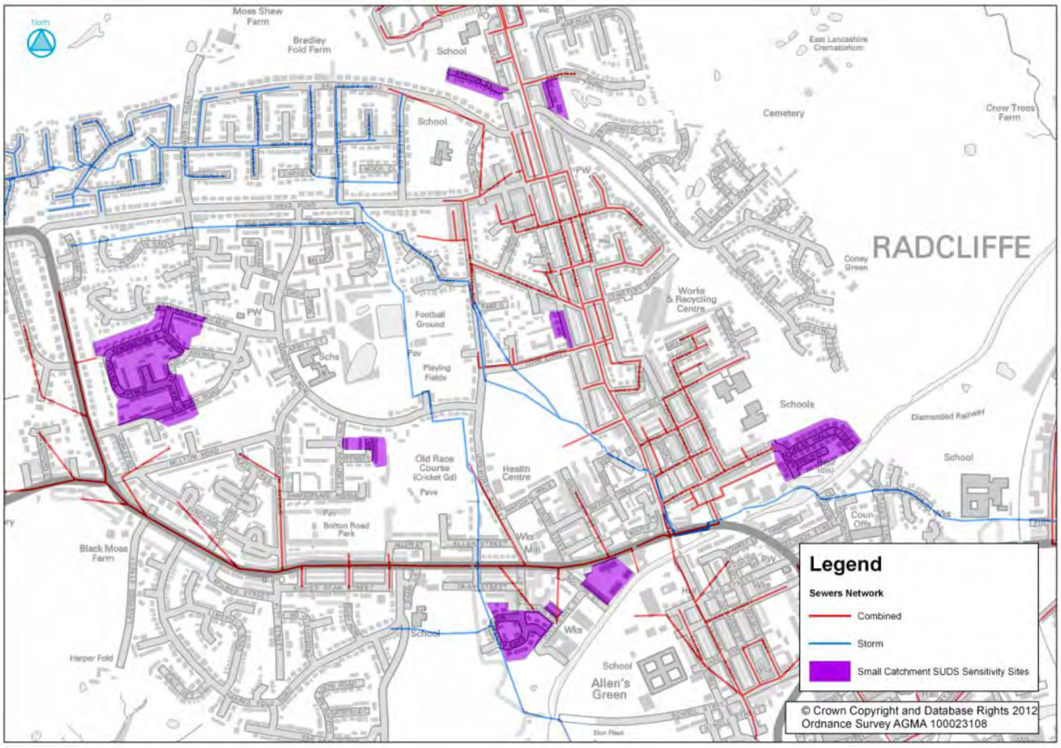


Table 5-6: Sensitivity to Small Catchment SUDS Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m3), Summer 2hr Duration Event	Sensitivity to Small Catchment SUDS Flood Volumes (m3), Summer 2hr Duration Event	Percentage Variation from Calibrated Model (%)
50%	36	1	-97.2%
20%	419	317	-24.3%
10%	927	774	-16.5%
5%	1691	1474	-12.8%
3.33%	2304	2061	-10.5%
2%	3352	3023	-9.8%
1%	5120	4794	-6.4%

The review of SUDS opportunities suggests that benefits can be achieved particularly in the smaller flood events. Given the frequency of flooding at the site, SUDS solutions could therefore reduce the risk at the Water Street junction.

A general catchment policy of identifying opportunities to incorporate additional attenuation into the system is recommended.

There are ongoing negative environmental and social impacts associated with the operation of the BRY0129 CSO. In the long term it would be desirable to integrate SUDS into the upstream catchment as discussed above to reduce the pressure on the combined sewer and providing positive benefits for the local area.

These measures need to be instilled as part of the ongoing management for the catchment but will not be sufficient to provide mitigation to the Water Street site in the larger flood events.

5.2.2.2 Upstream Attenuation

A preliminary review of storage volumes required to attenuate sufficient flows to manage flooding from the stormwater system only has been completed, Table 5-7.

Total inflow hydrographs from the stormwater system and the CSO assuming a free discharge into the stormwater culvert have been developed. All flood volumes above 1.3 m^3/s, the estimated capacity of the existing culvert, are assumed to be stored. This approach will underestimate actual storage volumes but provides a good initial estimate.

Table 5-7: Provisional Storage Requirements to Attenuate Flows in the Existing Culvert

Required Storage Volumes (m^3) in excess of 1.3 m^3/s for the following return periods (AEP)							
Event	**50%**	**20%**	**10%**	**5%**	**3.33%**	**2%**	**1%**
Summer 2hr	-	-	-	-	553	1435	2709

Table 5-5 suggests that an attenuation scheme in the region of 1,500 m^3 would be sufficient to attenuate the 2% AEP event for example. To provide a free discharge for the CSO, i.e. to maintain stormwater flows below 1 m^3/s would require an increase in this volume.

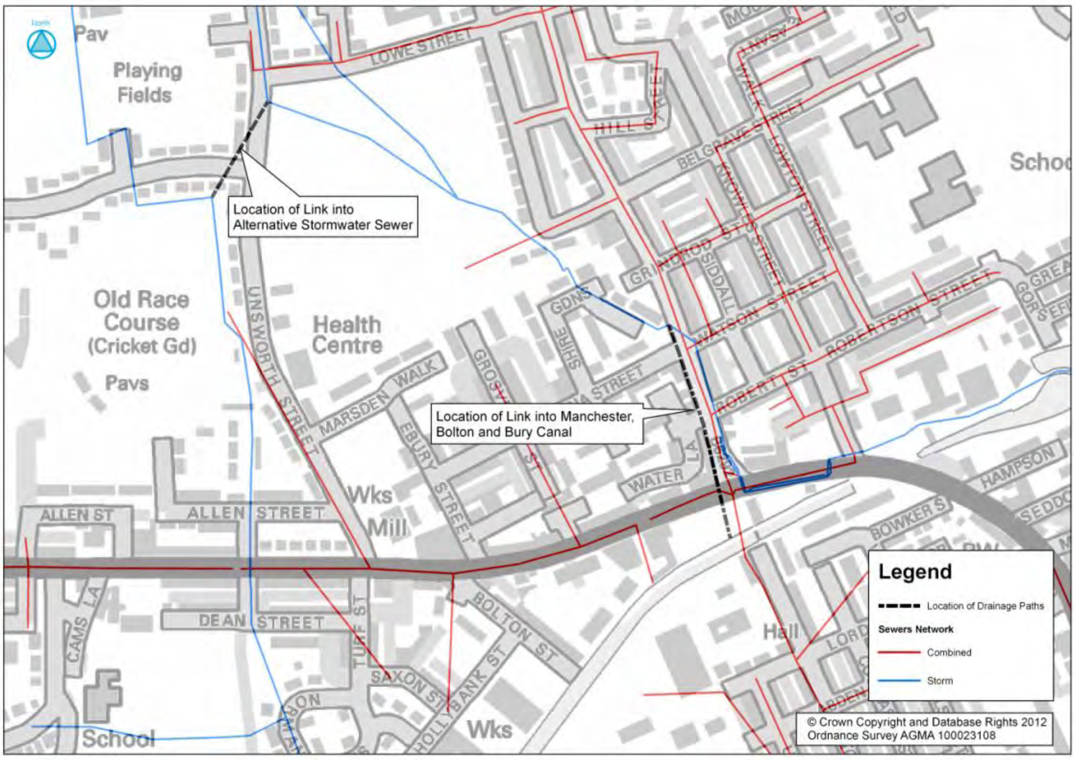
The only available open space is to the south of Lowe Street. The ground drops steeply towards the south and it may be feasible to construct a raised bund around the southern end of the site to provide the required storage. A review of the LIDAR data suggests the bund would be of the order of 1.7m in height at its maximum. In reality the bund would be constructed above the properties in Grosvenor Street and Shire Gardens and as such this is not considered a feasible option given the health and safety implications of the scheme.

5.2.2.3 Alternative Drainage Paths

To reduce existing and expected future capacity pressures on the stormwater culvert flows could be diverted via another route to the River Irwell. Two options suggest themselves for diverting flows, Figure 5-7:

- Provide an overflow into the existing stormwater system immediately to the west which discharges to the Irwell off the end of Hutchinsons Way.
- Provide an overflow into the Canal from Ainsworth Road, which will then discharge to the Irwell via an overflow location.

Figure 5-7: Alternative Drainage Paths to the River Irwell



A link into the existing stormwater system could be installed off the end of Lowe Street but UU have reported that this system is already at capacity with flooding issues located downstream. Further discussion will be required with UU regarding the capacity of this system if this option is to be taken forward.

An option to provide an overflow into the canal from Ainsworth Road could be investigated further. This would need to be located upstream of the CSO to prevent contaminated flows discharging into the canal from this source and there would need to be confidence that water quality in the canal will not be affected from cross connections upstream. The overflow could potentially link into manhole SD77079611 and run along Ainsworth Road to achieve a sufficient gradient in the outfall pipe. The canal itself would provide significant attenuation reducing the impact of flows on the River Irwell and there would be scope to separate the sewer system in the future and so relieve pressure on the existing combined network.

The EA have expressed concerns regarding the water quality of discharges into the canal and some measure to collect silt and debris washing of the roads would be required. The status of the canal is classified as moderate in the North West River Basin Management Plan[5] with the objective to achieve good status by 2027. The canal is also designated as a site of biological importance. Even with this in place it is likely that utilising the canal as part of the flood management regime will not be desirable however this option provides an opportunity to design a scheme that can manage increasing runoff in the future associated with climate change. Further discussion with the relevant authorities would be required but it is recommended this option is considered further.

5.2.2.4 Storage in the Combined Sewer System

Currently the stormwater sewer is backing up into the combined sewer system. However investigations suggest that the stormwater culvert is able to pass up to the 3.33% AEP design event from the stormwater system only with flood risk to properties predicted to occur closer to the 2% AEP event.

[5] River Basin Management Plan, North West River Basin District, Environment Agency, December 2009.

Providing additional storage in the combined sewer system in the region of 2,500 m^3 would be sufficient to manage both the combined system flows and additional runoff discharging into the combined sewer from the stormwater system in the existing situation. This would need to be increased to approximately 4,000m^3 if the unauthorised overflow is removed. This volume is a significant reduction on the 16,000 m^3 originally estimated by UU and it is recommended the option for storage be revisited.

These volumes would need to be confirmed with UU and the potential location of this storage would need to be investigated further outside of the scope of this study.

5.2.2.5 Increasing the Pass Forward Flows

The findings of the CCTV survey have confirmed that the existing culvert is in good condition, albeit with some debris apparent in the invert. The hydraulic constriction as a result of the culvert is therefore not considered to be a riparian owner issue but simply the size of the structure itself.

To increase pass forward flows will require the construction of an additional culvert adjacent to the existing culvert. Flooding is occurring in the vicinity of the CSO and as noted previously is caused by a combination of the size of the culvert itself in this location and the gradient of the culvert downstream. The current capacity of the twin culverts adjacent to the CSO is estimated to be in the region of 1.5 m^3/s, to allow a free discharge into the stormwater system through the CSO would require a capacity in the region of 2 m^3/s.

Providing a free discharge with the CSO in its existing state will only reduce the storage requirement within the combined system by approximately 600 m^3. This option would therefore be combined with improvements to the CSO as proposed by UU to further increase pass forward flows and reduce the existing pressure on the combined system.

The required flow capacity in the culvert associated with an increase in the CSO capacity will require improvements in the vicinity of the CSO and at the junction with Crow Tree Farm Brook.

This option would not be considered the preferred way forward in most cases unless all other options are considered unfeasible. Increasing flows downstream will have negative impacts on downstream sites as discussed in Section 5.2.1 and will also increase the release of contaminants into the watercourse, particularly if the CSO is modified.

5.3 Short List of Options

The short list of options has been developed based on the feasibility of options discussed in Section 5.2.2 and those that can provide an immediate reduction in flood risk to the hotspot area to the 3.33% AEP event. These options do not negate the need to consider the longer term management of the catchment.

At this stage all options will need to be discussed with project partners to determine preferred approaches.

5.3.1 Option 1 – Additional Storage in the Combined System

This option would require UU to provide sufficient storage within the combined sewer network to manage the 3.33% AEP, estimated to be 3910 m^3 if the unauthorised overflow is to be removed. This option would need to be agreed in conjunction with UU and as such no further investigation on proposed locations has been completed.

The storage requirement is increased by approximately 800 m^3 due to the fact that the CSO is unable to discharge freely into the stormwater sewer. It is therefore recommended that partial funding for this additional volume requirement through FDGiA is investigated.

The scale of this option will need to be increased significantly to accommodate any increases in flows in the future.

5.3.2 Option 2 – Increase Pass Forward Flow with Improvements to the CSO Operation and Unspecified Improvements in the Combined System

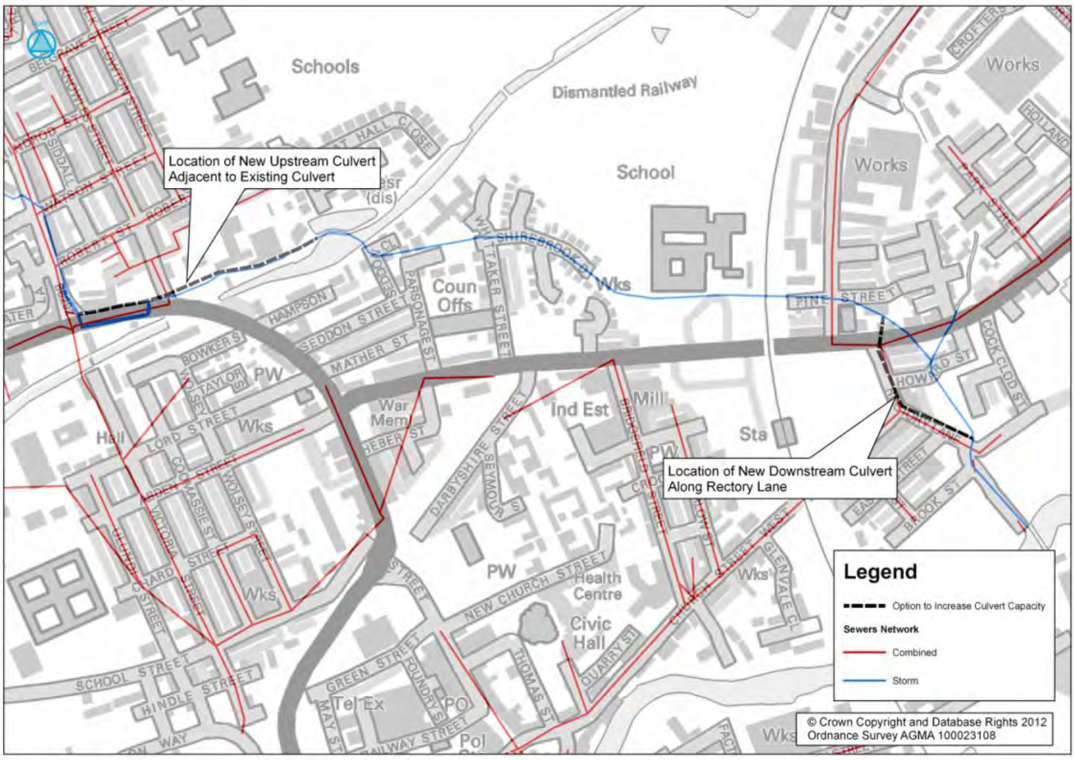
UU have provided details of additional inflows to the stormwater sewer as part of an option to manage flooding to the Water Street from the combined sewer. This option has investigated the feasibility to increase the capacity of the culvert to accommodate these additional flows.

The findings of the assessment suggest the capacity of the culvert will be dictated by the reach downstream of SD78077500 as detailed in Section 5.1.2. A review of increasing the capacity of the culvert between SD78079409 and mh6 suggests that reducing the hydraulic grade in this critical reach will provide a capacity in the downstream culvert in the region of 5.0 m^3/s. In the design storm assessed this capacity translates upstream to a peak flow allowance from the CSO of 2.1 m^3/s. This is not sufficient to manage all inflows required by UU but should provide some benefit from which further options within the combined system can be investigated.

To deliver a capacity to discharge 2.1 m^3/s from the CSO will require two improvements to the stormwater system, Figure 5-8:

- A bypass culvert will be constructed between manholes SD78070403 and SD78073501. This will run adjacent to the existing culvert and tie in a short distance upstream of the canal and downstream of the cascade to maximise the available gradient. The pipe will be a 900*1200mm rectangular pipe or equivalent capacity to minimise backflow into the combined sewer. The pipe itself will be 280m in length.
- A bypass culvert will be constructed between manholes SD78079409 and mh6 of 1200mm diameter or equivalent capacity running along Rectory Lane. The pipe would be 190m in length.

Figure 5-8: Locations of Capacity Improvements within the Stormwater Culvert



This option does not have the capacity to accommodate the increases in flows in the future.

5.3.3 Option 3 – Diversion of Stormwater Runoff into Canal, Improvements to the CSO Operation and Unspecified Improvements in the Combined System

This option will divert the majority of stormwater runoff into the canal thereby providing additional capacity within the stormwater culvert up to a capacity in the region of 1.3 m^3/s. The option would require updates to the CSO by UU and further improvements in the combined system.

This option has not been investigated in detail as it requires input from UU regarding the scale of diversion required. It is unlikely that this scheme will be acceptable to local stakeholders given the water quality impacts on the canal and the increase in concentration of contaminants in the sewer overflow to the stormwater culvert, however this could be offset by the removal of the unauthorised overflow.

If further work is required the effects of the increased flows on the canal would need to be discussed with the Canal and Rivers Trust and Greater Manchester Ecology Unit.

6 Assess Options

6.1 Options Assessment

A range of potential options have been developed from the options discussed in Section 5.2.2 that are likely to provide differing standards of flood protection and improvements to the local environment.

The work undertaken within this SWMP has significantly changed the understanding of the flood risk to the site. Feedback from UU is now required to confirm to what extent runoff can be managed within the combined system. Once this is clear greater confidence can be placed on a preferred option. It is therefore be critical to discuss with UU what can be achieved in light of these findings.

A short summary of the options and associated pros and cons are detailed in Table 6-1.

Table 6-1: Benefits and Limitations of each Flood Risk Options

Option	Description	Pros	Cons
1	Additional storage in the combined system	+ Manages combined flows within the combined system. + No increase in pass forward flows. + Potential for contributions from FDGiA for stormwater impacts on combined system.	- Feasibility of the option to be confirmed with UU, locations of available storage may not be available in which case partial storage only may be an option. - Ongoing operation costs potentially associated with pumping large storage tanks. - To accommodate future increases in runoff with climate change the tanks will need to be oversized.
2	Improve culvert capacity for stormwater and improved CSO flows up to 2.1 m^3/s	+ Reduced pressure on combined sewer system so provision of remaining storage requirement in combined system more likely than Option 1. + Potential for contributions from FDGiA for stormwater impacts on combined system.	- Increase in pass forward will require compensatory storage downstream. - Increase in CSO discharges likely to be unpopular with key stakeholders and riparian owners. - Pass forward flows are restricted so future increases in climate change will need to be accommodated within the combined system.
3	Diversion of stormwater into the canal and improved CSO flows up to 1.3 m^3/s	+ Reduced pressure on combined sewer system so provision of remaining storage requirement in combined system more likely than Option 1.	- Increase in concentration of CSO discharges into stormwater culvert likely to be unpopular with key stakeholders and riparian

Option	Description	Pros	Cons
		+ Increase in pass forward flows mitigated by attenuation in the canal. + Scale of works is smaller than Option 2. + Potential for contributions from FDGiA for stormwater impacts on combined system. + Overflow pipe can be oversized to allow increased runoff associated with climate change.	owners. - Stormwater overflow into canal could potentially transfer pollutants into the canal. - Potential for flood risk from the canal to be investigated. - Pass forward flows from combined system are restricted so future increases in climate change will need to be accommodated within the combined system.

To provide an understanding of the comparative benefits of each scheme in relation to each other a ranking has been developed for a range of issues, Table 6-2. A ranking of 1 to 3 is applied with a value of 1 suggesting there are greater benefits associated with the respective scheme compared to the remaining two.

Table 6-2: Ranked Comparison of Options

	Ranking		
Scheme Parameter	**Option 1**	**Option 2**	**Option 3**
Availability of Storage Capacity within the Combined System	3	1	2
Impacts on Flood Risk to Downstream Sites	1	3	2
Acceptability to Local Stakeholders	1	2	2
Likelihood of capacity to remove unauthorised overflow to canal	3	1	2
Expected Cost of Scheme*	2	3	1
Adaptability to Climate Change	3	2	1

*No detailed assessment of scheme costs is viable until further discussions are completed with UU.

The most acceptable solution for local stakeholders to manage the flood risk to the Water Street site will be increased storage capacity within the combined sewer system. Some allocation of funds from FDGiA could be expected to contribute to this work given the impact of stormwater flows to flood risk.

Further discussions are required with UU to confirm that the required storage capacity is achievable and available. If this is not the case then further discussions with stakeholder regarding partial storage options will need to be undertaken.

7 Action Plan

7.1 Introduction

The action plan outlines the next stages required to take the investigations on from the findings of the surface water management plan. The long term management of the catchment needs to be addressed to deliver sustainable management solutions to Water Street, however in the short term hard engineering solutions are available to reduce risk.

Each of the key stakeholders will need to comment on the findings of this report and provide feedback on viable management solutions for the site. It is unlikely that a solution can be developed without agreement between all parties.

UU remain the primary stakeholder; the SWMP has indicated that the storage requirements to manage flooding within the combined system are less than originally understood and these estimates will need to be reviewed and the feasibility of providing this storage confirmed. The EA and Bury MBC will need to consider the options available to manage flooding to the site in light of the comments supplied by UU on potential storage capacity within the combined system.

Because of the input required from UU it has not been possible to develop final options within the scope of the SWMP. However it is considered there is sufficient information within this SWMP to make an informed decision, in conjunction with all project partners, on the preferred way forward for management of the surface water flood risk to the Water Street site, and for Bury MBC to complete the Medium Term Plan to apply for funding for further works.

7.2 Next Steps

The tripartite meetings currently held between Bury MBC, UU and the EA provide a useful forum for the interests of all stakeholders to be presented. The next steps will largely depend on feedback from stakeholders however it is likely the following key issues will need to be resolved prior to any further investigation:

- Engagement strategy – this will need to be developed and include the Canal and Rivers Trust, Greater Manchester Ecology Unit and the riparian owners along the alignment of the stormwater sewer as a minimum. It is recommended no further work into proposed options that directly impact these stakeholders should be undertaken until these stakeholders have been actively engaged.

- Maintenance of the stormwater culvert – this is currently thought to be a limited issue but remains the responsibility of the riparian owner. It is recommended at this early stage that legal advice is sought to confirm if the responsibilities of the riparian owner can be enforced in the future.

- Funding responsibility - The approach outlined in this report assumes that UU are responsible for the combined system in its entirely and as such the level of protection achievable is the 3.33% AEP event, an element of which could be funded through FDGiA given the contribution from the stormwater system. However this is a grey area, the combined sewer system is predominantly discharging surface water runoff in the design event and if a design standard in excess of the 3.33% AEP event is desired it is recommended the potential to secure funding additional funding through FDGiA is investigated by the EA.

- Management strategy - there are strategic considerations within the study area and it is recommended an agreement is put in place between Bury MBC and UU recognising the drainage limitations within the catchment and proposing ongoing management policies such as those mentioned within this report. This agreement would need to consider the local strategy and those responsible for its implementation.

Appendices

A Appendix – Crow Tree Farm FEH Calculation Record

B Appendix – Hydraulic Model Sensitivity Tests

B.1 Overview

As part of the AGMA SWMP a series of sensitivity tests have been completed to investigate apparent inconsistencies within the UU model. The study itself has used the UU model as provided in general with the exception of updates in the vicinity of the Ainsworth Road and Water Street junction.

These sensitivity runs are provided to highlight areas for clarification by UU and potentially model updates as part of further investigations.

The updated calibrated model completed for the AGMA SWMP has been used as the base model for the sensitivity tests. To compare the models the flooding volumes lost from manholes in the vicinity of the Water Street and Ainsworth Road junction have been extracted and summed; this is a proxy for comparing mapped flood extents. For the purposes of the sensitivity assessment a summer storm with a 1hr duration has been applied.

B.2 Subcatchment Sensitivity Tests

B.2.1 Sensitivity to Total Area

The natural drainage catchment is calculated in the report to be 1.7km^2. Sub-catchments extracted from the model intersecting this natural drainage path sum 9.0km^2. The largest of these sub-catchments are tabulated below, Table B-1.

Table B-1: Summary of the Largest Sub-catchments within the Model

Subcatchment	System	Node ID	Total Area (ha)	Contributing Area (ha)	X	Y	Land Use
EXTRA SW 1	storm	SD77083100	270	270	376904.5	408613.8	98
Canal	storm	SD78073501	200	200	378172.8	407781.4	10
Dummy perm1	storm	SD77074851	150	150	377251.4	407815.4	97
Foul Dummy permeable 1	foul	SD76089302	130	130	376904.5	408613.8	10
SD78075512	storm	SD78075512	14.15	14.15	378209	407969.6	12
SW Extra	storm	SD76085201	10.663	10.663	376282.3	408227.7	12

The top four catchments stand out with a combined area of 7.5km^2. Removal of these leaves a drainage catchment of 1.5km^2 which is closer to the expected drainage area for the catchment. Notes on these subcatchments and discussions with UU suggest that these are dummy catchments added to represent slow infiltration from permeable surfaces such as large open areas to the north of the catchment or from the canal. In all cases the routing value is increased significantly to attempt to represent the slow response of the catchment. It is assumed these were developed using the flow survey data, in which over the longer events the effects of infiltration are apparent.

To review the effect of these catchments on the model results a sensitivity run has been completed with these catchments removed, Table B-2.

Table B-2: Sensitivity to Total Area Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m3)	Sensitivity to Area Flood Volumes (m3)	Percentage Variation from Calibrated Model (%)
50%	50	50	0.00%

Return Period (AEP)	Calibrated Model Flood Volumes (m3)	Sensitivity to Area Flood Volumes (m3)	Percentage Variation from Calibrated Model (%)
20%	403	403	0.00%
10%	871	873	0.23%
5%	1575	1555	-1.27%
3.33%	2127	2112	-0.71%
2%	2973	2948	-0.84%
1%	4388	4343	-1.03%

The removal of the large infiltration catchments has minimal impact on the predicted flood risk in the vicinity of the Ainsworth Road and Water Street junction.

B.2.2 Sensitivity to the Runoff Model

The Wallingford routing model has been uniformly applied to sub-catchments within the model. This is generally appropriate when the majority of sub-catchments are less than 1ha in area.

There are 125 subcatchments out of 540 with an area in excess of 1ha. Whilst this represents the minority of catchments it remains a high percentage.

To determine the effect of applying the Wallingford routing model across all sub-catchments a sensitivity assessment has been completed where the large catchment routing model has been applied to those catchments with an area greater than 1ha only, Table B-3.

Table B-3: Sensitivity to Runoff Model Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m3)	Sensitivity to Runoff Model Flood Volumes (m3)	Percentage Variation from Calibrated Model (%)
50%	50	50	0.00%
20%	403	400	-0.74%
10%	871	874	0.34%
5%	1575	1583	0.51%
3.33%	2127	2151	1.13%
2%	2973	2998	0.84%
1%	4388	4460	1.64%

The application of the large catchment routing model to sub-catchments with an area in excess of 1ha has minimal impact on the predicted flood risk in the vicinity of the Ainsworth Road and Water Street junction.

B.3 Node Sensitivity Tests

B.3.3 Sensitivity to Total Floodable Area

The total floodable area of nodes intersected the natural drainage catchment is 0.79km^2. A review of the natural drainage catchment covered by the UU model suggests a floodable area of closer to 1.0km^2 is appropriate; this has been generated by creating a catchment covering the network area.

A sensitivity test has been completed to assess the effects of an increase in the floodable area on model results. This has been achieved by applying a flat scaling factor across all sub-catchments equivalent to 1.0/0.79 or 1.27, Table B-4.

Table B-4: Sensitivity to Floodable Area Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m3)	Sensitivity to Floodable Area Flood Volumes (m3)	Percentage Variation from Calibrated Model (%)
50%	50	50	0.00%
20%	403	405	0.50%
10%	871	877	0.69%
5%	1575	1558	-1.08%
3.33%	2127	2120	-0.33%
2%	2973	2961	-0.40%
1%	4388	4368	-0.46%

The increase in the floodable area in the model to reflect the catchment area covered by the model has minimal impact on the predicted flood risk in the vicinity of the Ainsworth Road and Water Street junction.

B.3.4 Sensitivity to Ground Levels

Ground levels within the model have been extracted and compared to the LIDAR data in the same location. The findings of this assessment show isolated variations up to 7m and a mean variation of +0.05m. Out of the 787 nodes compared 498 vary by 0.15m or less, i.e. 37% vary by greater than 0.15m.

A sensitivity test has been completed replacing all ground and flooding level data with LIDAR data extracted from the location of the manhole, Table B-5.

Table B-5: Sensitivity to Ground Levels Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m3)	Sensitivity to Ground Levels Flood Volumes (m3)	Percentage Variation from Calibrated Model (%)
50%	50	90	80.00%
20%	403	471	16.87%
10%	871	980	12.51%
5%	1575	1717	9.02%
3.33%	2127	2290	7.66%
2%	2973	3169	6.59%
1%	4388	4684	6.75%

The results show a significant increase in flood volumes as a result of the sensitivity test.

A review of the manholes where flood volumes are predicted in each case and the observed volumes is summarised in Table B-6 and a comparison of model levels and LIDAR levels are shown in Table B-7.

Table B-6: Sensitivity to Ground Levels Flood Volume Comparison at Specific Manholes

	Flood Volumes (m^3) associated with each manhole for a range of return periods (AEP)													
	50%		20%		10%		5%		3.33%		2%		1%	
Manhole	Original	Sensitivity	Original	Sensitivity	Original	Sensitivity	Original	Sensitivity	Original	Sensitivity	Original	Sensitivity	Original	Sensitivity
101_320	0	0	0	0	0	0	0	0	0	0	0	0	50	25
101_330	0	0	80	41	201	142	381	299	521	411	727	584	1008	848
101_338	50	90	296	429	515	728	765	1035	903	1208	1067	1420	1278	1746
101_345	0	0	0	1	0	52	0	136	0	185	0	266	0	423
144_000	0	0	0	0	0	0	0	1	0	15	0	43	0	84
145_000	0	0	4	0	12	3	23	13	31	20	43	32	65	52
148_160	0	0	0	0	0	0	37	30	87	79	175	171	318	329
148_170	0	0	0	0	0	44	0	114	0	157	3	217	29	342
BRY0129	0	0	2	0	76	9	202	49	284	81	403	119	594	147
SD77079413	0	0	0	0	8	0	48	0	75	0	111	0	157	0
SD78070403	0	0	0	0	0	0	7	0	43	0	98	0	198	0
SD78070408	0	0	0	0	3	3	15	14	25	21	40	30	71	22
SD78070516	0	0	0	0	0	0	0	0	0	0	0	0	2	0
SD78070518	0	0	0	0	0	0	1	24	34	108	123	263	316	581
SD78070606	0	0	0	0	0	0	0	0	0	0	2	0	15	0
SD78070610	0	0	0	0	0	0	0	1	0	5	0	11	0	20
SD78071503	0	0	0	0	0	0	0	0	0	0	1	6	13	22
SD78071505	0	0	0	0	0	0	0	0	0	0	0	2	0	17
SD78071507	0	0	22	0	56	0	96	0	125	0	166	4	226	7
SD78072603	0	0	0	0	0	0	0	0	0	0	14	0	46	0
SD78072606	0	0	0	0	0	0	0	0	0	0	0	2	0	19

Table B-7: Sensitivity to Ground Levels Comparison of Levels at Specific Manholes

Manhole	Ground Level	Flood Level	LIDAR Data	Variation	Data Source
101_320	81.06	81.06	81.06	0.00	Asset Data
101_330	78.61	78.61	78.61	0.00	Asset Data
101_338	77.46	77.48	77.32	0.14	Survey Data
101_345	77.13	77.13	76.96	0.17	Survey Data
144_000	82.71	82.71	80.17	2.54	Asset Data
145_000	81.39	81.39	82.65	-1.26	Asset Data
148_160	77.90	77.90	77.77	0.13	Survey Data
148_170	77.34	77.34	77.13	0.21	Survey Data
BRY0129	77.08	77.17	77.03	0.05	None Provided
SD77079413	77.23	77.23	77.14	0.09	Asset Data
SD78070403	77.02	76.97	76.98	0.04	Survey Data
SD78070408	77.20	77.36	77.08	0.12	Survey Data
SD78070516	78.50	78.50	79.13	-0.63	None Provided
SD78070518	77.11	77.26	77.02	0.09	Survey Data
SD78070606	79.37	79.37	79.33	0.04	Asset Data
SD78070610	81.00	81.00	79.69	1.31	None Provided
SD78071503	79.83	79.83	79.74	0.10	Asset Data
SD78071505	78.54	78.54	78.72	-0.18	Asset Data
SD78071507	77.16	77.16	78.19	-1.03	Survey Data
SD78072603	79.77	79.77	80.80	-1.03	Asset Data
SD78072606	80.18	80.18	80.06	0.12	Asset Data

The results show that in general there is good consistency between the modelled ground levels and the LIDAR levels at those manholes where flooding is predicted. The significant variations in flooding volumes observed through the sensitivity test are therefore attributed to the sensitivity of the manhole to flood risk, as is the case at manhole 101_338, or the wider effects of changes in the model, as is the case at manholes 101_320 and 101_330.

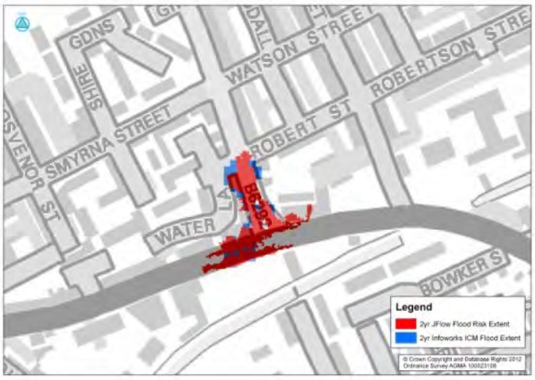
LIDAR data in general is a poor substitute for survey data but the manholes where there are significant variations are cause for concern. The sensitivity is particularly apparent in smaller flood events where flood risk is dominated by surcharging at fewer manholes, in this case manhole 101_338 where the original model uses survey data which should be preferred over the LIDAR data. In larger flood events the results are less sensitive as the greater number of surcharging manholes drowns out the variations at specific manholes.

B.3.5 Sensitivity to a Hydrodynamic Link between 1D and 2D models

The approach to mapping flood risk within the study area has focussed on modelling the effects at the location of interest only. In addition to this the JFlow model routes overland flow only and does not allow flood waters to return to the stormwater system.

To assess both the potential flood risk to the site from the wider area and the potential effects of a hydrodynamic link between the underground and overground systems the model has been imported into ICM and coupled with the LIDAR data. The results of the JFlow and Infoworks ICM model results are shown in Figure B-1 for a range of return periods.

Figure B-1: Comparison of Flood Extents from JFlow and Infoworks ICM

	
50% AEP Event	20% AEP Event
	
10% AEP Event	5% AEP Event

	
3.33% AEP Event	

The results show close agreement, particularly in the smaller flood events where the system may continue to retain capacity and so the effect of the hydrodynamic coupling of the models would be expected to be greatest. In the larger events where the system will be surcharged and so flood risk between the two models would be expected to agree closely there is greater variation. This is attributed to the variable manning's grid applied in the JFlow resulting in preferential flow routes along roads.

B.4 Conduit Sensitivity Tests

B.4.6 Sensitivity to Hydraulic Roughness

The Colebrook-White hydraulic roughness within the stormwater system has generally been set to 3 mm.

A sensitivity test has been completed setting pipe roughness in the stormwater network outside the calibrated reach between the BRY0129 CSO and the canal to 0.6mm, Table B-8.

Table B-8: Sensitivity to Hydraulic Roughness Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m3)	Sensitivity to Hydraulic Roughness Flood Volumes (m3)	Percentage Variation from Calibrated Model (%)
50%	50	51	2.00%
20%	403	415	2.98%
10%	871	870	-0.11%
5%	1575	1585	0.63%
3.33%	2127	2153	1.22%
2%	2973	3036	2.12%
1%	4388	4442	1.23%

The reduction in the hydraulic roughness in the stormwater system has minimal impact on the predicted flood risk in the vicinity of the Ainsworth Road and Water Street junction.

B.5 Boundary Conditions Sensitivity Tests

B.5.7 Antecedent Conditions

The API 30 values in the model are currently set to 17. The SAAR value for the catchment extracted from FEH catchment descriptors is approximately 1100. Using the correlations extracted from the WaPUG paper by Jamie Margetts[2] for a WRAP soil class of 4 gives a summer API of 7 and a winter API of 23.

A sensitivity test has been completed reducing the API 30 values to 7 for the summer storm, Table B-9.

Table B-9: Sensitivity to Antecedent Conditions Flood Volume Comparison

Return Period (AEP)	Calibrated Model Flood Volumes (m3)	Sensitivity to Antecedent Conditions Flood Volumes (m3)	Percentage Variation from Calibrated Model (%)
50%	50	29	-42.00%
20%	403	338	-16.13%
10%	871	776	-10.91%
5%	1575	1426	-9.46%
3.33%	2127	1906	-10.39%
2%	2973	2721	-8.48%
1%	4388	4097	-6.63%

The results are particularly sensitive in smaller flood events as would be expected. It is recommended further investigation be undertaken in later studies to clarify the effects of the antecedent conditions on the design storm.

B.5.8 Sensitivity to Design Rainfall Parameters

The design rainfall events in the model have been developed using an FEH generator using an area of 33km^2. It is assumed this is appropriate for the wider catchment model but this may not be appropriate for the smaller catchment draining to the Water Street junction site.

A smaller catchment has been extracted from the FEH that overlies the drainage catchment of interest. The FEH parameters from the existing model and for the smaller catchment are tabulated in Table B-10.

Table B-10: Depth Duration Frequency Parameters for Sensitivity to Design Rainfall Assessment

Parameters	UU Model	Sensitivity Review
Catchment Area	33.03	1.52
C	-0.025	-0.025
D1	0.376	0.363
D2	0.375	0.367
D3	0.371	0.362
E	0.302	0.301
F	2.5	2.49

The effect of changing the catchment descriptors is to increase the aerial reduction factor from 0.86 to 0.95 in the 1hr design storm.

A sensitivity test has been completed using rainfall events derived from this catchment specific site, Table B-11. For this sensitivity test the API30 value for the initial conditions has also been set to 7.

Table B-11: Sensitivity to Rainfall Parameters Flood Volume Comparison

Return Period (AEP)	Sensitivity to Antecedent Conditions Volumes (m3)	Sensitivity to Rainfall Parameters Volumes (m3)	Percentage Variation from Antecedent Conditions Sensitivity Model (%)
50%	50	19	-62.00%

Return Period (AEP)	Sensitivity to Antecedent Conditions Volumes (m3)	Sensitivity to Rainfall Parameters Volumes (m3)	Percentage Variation from Antecedent Conditions Sensitivity Model (%)
20%	403	316	-21.59%
10%	871	735	-15.61%
5%	1575	1368	-13.14%
3.33%	2127	1844	-13.31%
2%	2973	2614	-12.08%
1%	4388	3943	-10.14%

Again the results are particularly sensitive in the smaller flood events. The overall effect of this change is to reduce the predicted flood extents at the site at each return period. It is noted the reported flooding frequency at the site is every two years and these results remain consistent with that. It is recommended the rainfall profile be updated as part of any further assessments.

B.5.9 Sensitivity to Downstream Boundary

The stormwater culvert outfalls into the River Irwell. A 20% AEP event has been considered an appropriate downstream boundary for the purposes of this study. To confirm the implications of a higher downstream boundary a series of sensitivity runs have been completed. Table B-12 and Table B-13 detail peak water levels at manholes upstream of the outfall excluding and including flows from Crow Tree Farm Brook respectively.

Table B-12: Sensitivity to Downstream Boundary Excluding Crow Tree Farm Brook

Return Period (AEP)	SW_Outfall	mh6	SW_1	SD78079401	SD78079409	SD78077500
Chainage from Outfall	0	120.4	219.8	259.2	292.4	470.2
20% AEP Event on the Irwell						
50%	61.85	61.87	62.81	63.47	63.95	65.00
20%	61.85	61.88	62.88	63.51	64.04	65.08
10%	61.85	61.89	62.94	63.54	64.11	65.14
5%	61.85	61.90	63.03	63.60	64.32	65.30
3.33%	61.85	61.91	63.10	63.64	64.47	65.63
2%	61.85	61.93	63.19	63.71	64.71	66.29
1%	61.85	61.95	63.51	64.02	65.00	67.25
10% AEP Event on the Irwell						
50%	62.24	62.25	62.83	63.47	63.95	65.00
20%	62.24	62.25	62.90	63.51	64.04	65.08
10%	62.24	62.26	62.96	63.54	64.11	65.14
5%	62.24	62.26	63.07	63.60	64.32	65.30
3.33%	62.24	62.27	63.14	63.64	64.47	65.63
2%	62.24	62.28	63.30	63.74	64.71	66.29
1%	62.24	62.29	63.73	64.26	65.18	67.32
2% AEP Event on the Irwell						
50%	63.34	63.34	63.52	63.62	63.95	65.00
20%	63.34	63.35	63.63	63.74	64.04	65.08
10%	63.34	63.35	63.75	63.89	64.14	65.14
5%	63.34	63.35	63.96	64.18	64.59	65.45
3.33%	63.34	63.36	64.12	64.42	64.94	66.06
2%	63.34	63.36	64.34	64.72	65.35	66.75
1%	63.34	63.37	64.71	65.19	66.05	67.99

Table B-13: Sensitivity to Downstream Boundary Including Crow Tree Farm Brook

Return Period (AEP)	SW_Outfall	mh6	SW_1	SD78079401	SD78079409	SD78077500
Chainage from Outfall	0	120.4	219.8	259.2	292.4	470.2
20% AEP Event on the Irwell						
50%	61.85	61.95	63.49	63.49	63.95	65.00
20%	61.85	61.95	63.50	63.51	64.04	65.08
10%	61.85	61.95	63.52	63.55	64.11	65.14
5%	61.85	61.96	63.55	63.61	64.32	65.30
3.33%	61.85	61.96	63.57	63.65	64.47	65.63
2%	61.85	61.96	63.60	63.75	64.71	66.29
1%	61.85	61.97	63.95	64.34	65.24	67.31
10% AEP Event on the Irwell						
50%	62.24	62.29	63.73	63.73	63.95	65.00
20%	62.24	62.29	63.75	63.75	64.04	65.08
10%	62.24	62.29	63.77	63.77	64.11	65.14
5%	62.24	62.29	63.80	63.80	64.32	65.30
3.33%	62.24	62.29	63.82	63.82	64.47	65.63
2%	62.24	62.29	63.85	63.90	64.71	66.29
1%	62.24	62.30	64.15	64.54	65.42	67.40
2% AEP Event on the Irwell						
50%	63.34	63.34	63.52	63.62	63.95	65.00
20%	63.34	63.35	63.63	63.74	64.04	65.08
10%	63.34	63.35	63.75	63.89	64.14	65.14
5%	63.34	63.35	63.96	64.18	64.59	65.45
3.33%	63.34	63.36	64.12	64.42	64.94	66.06
2%	63.34	63.36	64.34	64.72	65.35	66.75
1%	63.34	63.37	64.71	65.19	66.05	67.99

The results show the effects of the downstream boundary with no flows from Crow Tree Farm Brook extend up to manhole SW_1 for all return periods when the event on the Irwell is increased from the 20% AEP to the 10% AEP event. This increase is limited to less than 0.05m for events up to the 3.33% AEP event in the culvert. Flood risk is most sensitive at manhole SD78079401 and no change is observed in levels at this manhole up to the 3.33% AEP event in the culvert.

When flows from Crow Tree Farm Brook are included the effect on increasing the levels on the Irwell extends as far as manhole SD78079401. In this instance some sensitivity of management solutions will be required if it is determined a joint probability event of a 3.33% AEP event in the culvert and a 10% AEP event on the Irwell is likely.

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