Hurlston Brook Flood Risk Study

Lancashire County Council

Modelling Report

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Appendix A. Hydraulic Modelling Technical Note

A.1 Introduction

A.1.1 Project Background

Jacobs were appointed by Lancashire City Council (LCC) to carry out a study to identify the flooding concerns in Ormskirk. Special attention is paid to a number of areas that have a history of flooding from surface water. The study also intends to understand the interaction of fluvial flooding with the surface water flooding. The main watercourse, Hurlston Brook, runs through Ormskirk from south to north.

An integrated hydraulic model was constructed with the intention of understanding the flooding mechanisms and the flow paths which aggravate the flooding mainly at the town centre.

The modelled study area covers the catchment from which the surface water can drain towards the centre of Ormskirk. Figure A.1 shows the model extents as well as the drainage system which involves storm and combined system pipes.

A.1.2 Objectives of Hydraulic Modelling

The main objective of the integrated hydraulic model was to first understand the existing flooding mechanism in the Ormskirk area from both surface water and Hurlston Brook as a result of a storm event. This provided a Baseline scenario for the analysis of economic damages in the area.

The model was then used to inform an option appraisal process to identify potential suitable flood risk management options to mitigate flooding. Model results were used to evaluate the economic benefits and benefit cost ratio from the scheme.

In summary, the model has been designed to represent:

- Interaction between the Hurlston Brook (fluvial), surface water (pluvial) and the drainage network;
- Overland water depths, flows and velocities;
- Scheme design components.

This appendix to the Viability report details the process followed for the construction of the integrated hydraulic model, the data used, the hydrological analysis, the model verification process, model sensitivity to various parameters and the limitations associated with the model. A brief summary of the results is also provided; however for details of flooding mechanism and results, this appendix should be read in conjunction with the main Viability report as well as the other appendices of the Viability report.

A.1.3 Methodology

The model simulates rainfall events during which it computes the rainfall runoff volume that would be routed overland by gravity and also the runoff volume that would drain into and be conveyed through the drainage pipe network. The model also simulates the flow routed through Hurlston Brook channel and its dynamic interaction with the surface water and drainage system.

The model was built using TUFLOW software and it is a pluvial-fluvial integrated One-Dimensional (1D) – Two-Dimensional (2D) model. The storm and combined pipes of the drainage system and Hurlston Brook were modelled as ESTRY (1D engine of TUFLOW). This process is illustrated in Figure A.2. The Hurlston Brook open channel includes representation of all the structures across the watercourse. The ground surface is represented in 2D in TUFLOW and the rainfall falling on this 2D surface is converted to volume that generates runoff. TUFLOW version 2013-12-AD-iDP-w64 was used for the model simulations.

The fluvial component of the model was imported from an existing Environment Agency model that was developed for Hurlston Brook as part of the Strategic Flood Risk Management (SFRM) framework by Capita[1]. This model included a representation of Hurlston Brook in Ormskirk and further downstream, but for this study it has been truncated to cover Ormskirk only. The Hurlston Brook representation has not been updated from the SFRM model (except the weir crest representing a sluice in Coronation Park). The cross sections are generally less than 100m apart with quite a large number of footbridges in the urban area.

[1] Hurlston Brook SFRM, Final Report, March 2010. Commission Reference: NW005

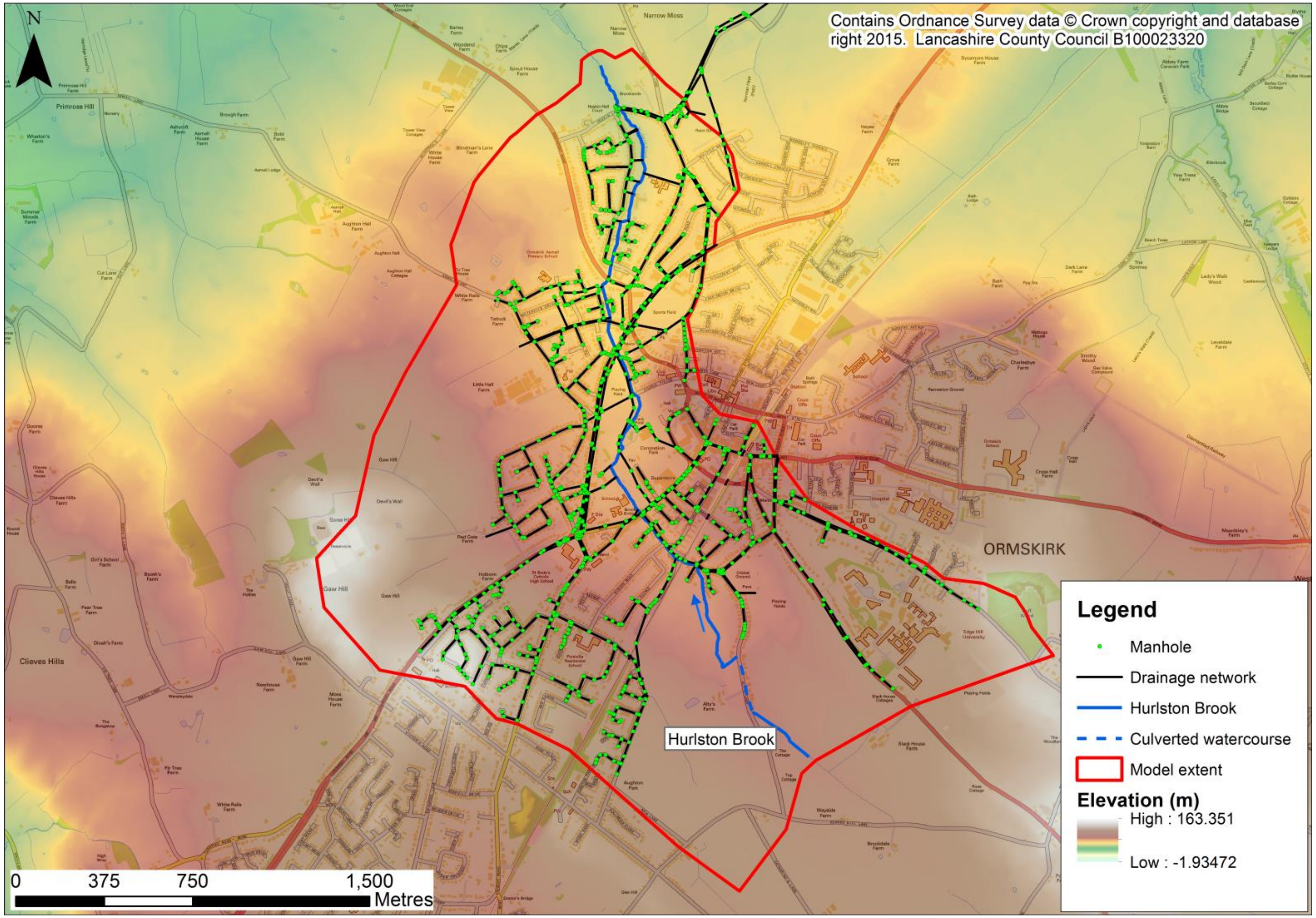

Figure A.1 : Study area showing manholes, the drainage network and the length of Hurlston Brook included in the model

The SFRM model was updated by Jacobs to include the storm and combined drainage system, which was represented in ESTRY 1D. A dynamic link between the drainage system and the above ground surface represented in the 2D model was also implemented. The drainage system information was based on an existing United Utilities InfoWorks CS model of the drainage catchment[2], supplemented by GIS data provided by United Utilities (UU). The InfoWorks CS model was converted to ESTRY because the fluvial component of Hurlston Brook was already available in ESTRY.

To represent surface water processes and their interaction with the drainage system and the fluvial system, design rainfall profiles were applied to the 2D TUFLOW model grid which is a fixed grid of individual square cells of 4m x4m.

No additional survey was carried out for the model update during this study. LiDAR (Light Detection And Ranging) Digital Terrain Model (DTM) data at 2m horizontal resolution was provided by LCC and was used in the model after resizing it to 4m resolution. The resizing was done to reduce the mass errors in the 2D domain.

In the TUFLOW model, the Manning's roughness coefficients are based on the different landuse types provided by the Ordnance Survey (OS) MasterMap data. Natural rainfall infiltration into the ground was initially applied in the TUFLOW model using the Green-Ampt method for the permeable surfaces, which were also identified using the MasterMap data. This approach however was replaced with a simpler approach based on runoff coefficients (applied directly to the rainfall profile) because the surface runoff and routed flows were not reconciling well with the hydrological flow estimates at the downstream end of the modelled area (see section A.4.2 for details).

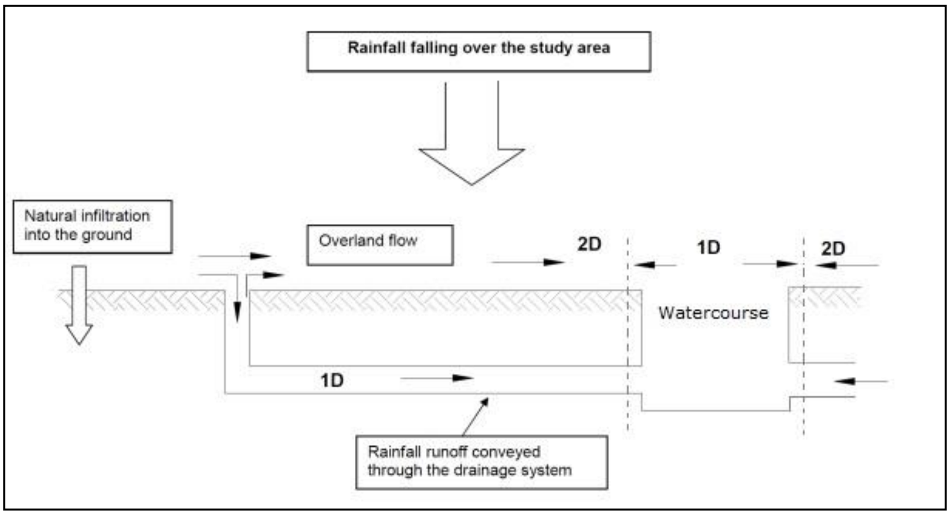

Figure A.2 : Overview of the integrated catchment modelling process

A.1.4 Input Data

The datasets used to construct the hydraulic model are summarised in Table A.1.

Table A.1 : Data used for hydraulic model construction

Data	Description	Source
LiDAR	Filtered Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) data. Used to inform the hydraulic model with ground elevation information.	Lancashire County Council
Proposed Scheme topography	Design layout	Jacobs 2016/17
Ordnance Survey (OS) maps	MasterMap data 1 to 10,000 Scale Raster	Lancashire County Council
Hydrological analysis	Hyetograph for a range of Annual Exceedance Probability (AEP)	Jacobs 2015

[2] Burscough_2015MM_Model_Manual_v4.xlsm

Data	Description	Source
	events. See section A.2 (Hydrological Analysis).	
Drainage network model	InfoWorks CS model and GIS dataset of the drainage network provided by UU	United Utilities

A.2 Hydrological Analysis

A.2.1 Introduction

A hydrological analysis was carried out to provide (1) fluvial inflows from the upstream catchment of Hurlston Brook applied at the upstream extent of the model and (2) rainfall hyetographs for the catchment over Ormskirk. The analysis was carried out for the 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1% and 0.5% Annual Exceedance Probability (AEP) events. Hydrological flow estimates at the downstream end of the model were also calculated in order to compare with the flows routed by the model.

A.2.2 Methodology

The flow locations were firstly estimated along the Hurlston Brook watercourse. These locations are presented in Table A.2 and mapped on Figure A.3 both below.

Table A.2 : Location of flow estimates

Reference	Description	Grid reference	Catchment area (km^2)
HB01	On the Hurlston Brook, upstream extent of model. Inflow modelled as flow-time boundary.	341800, 406950	0.96
HB02b	On the Hurlston Brook, downstream extent of model. Flow used for reconciliation with model's routed flows.	340950, 409800	5.66

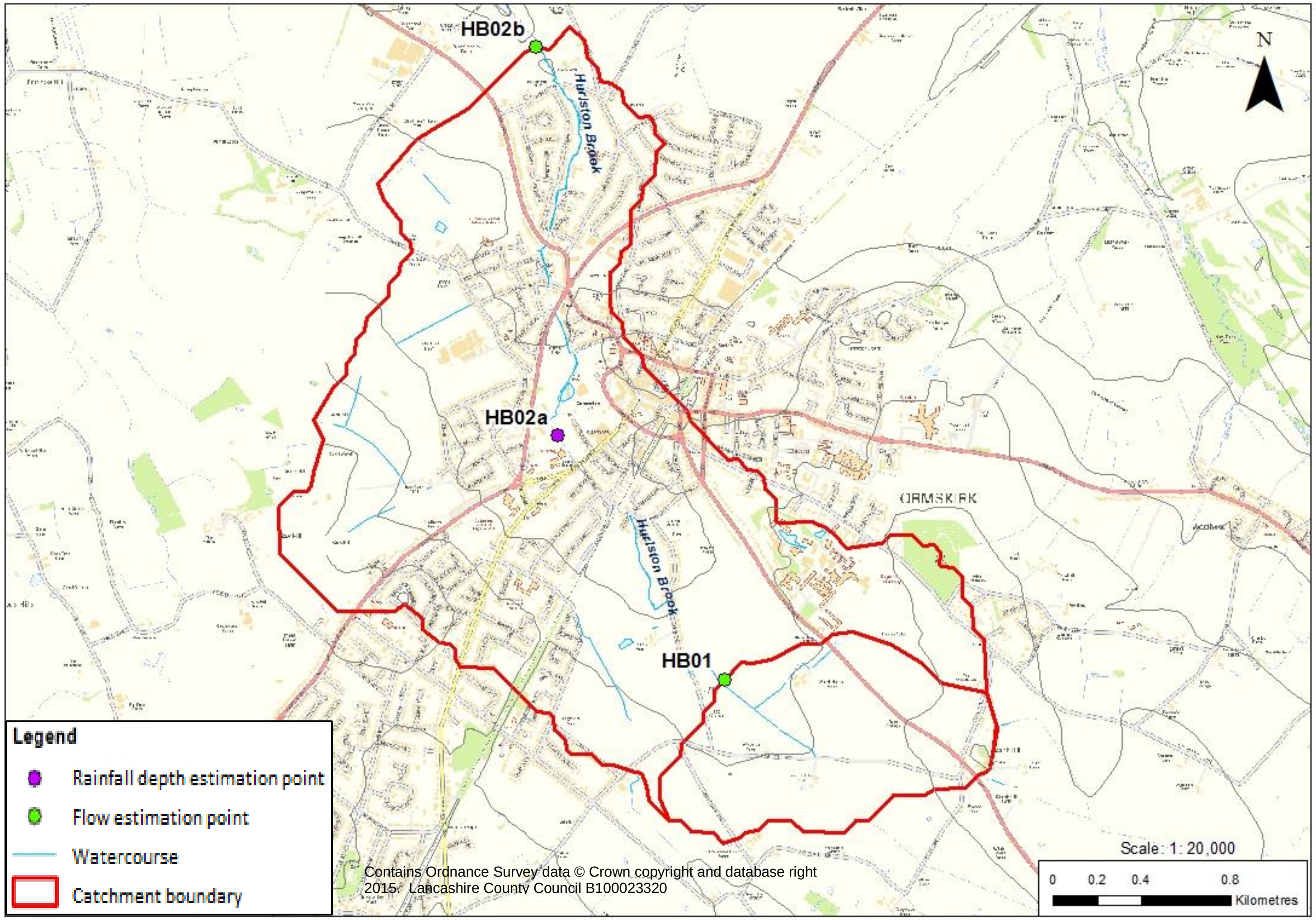

Figure A.3 : Locations of flow estimates and model extent

The following analysis was undertaken:

- Catchment descriptors were extracted using the FEH CD-ROM for selected locations, HB01 and HB02b, and then checked against previous amendments made by Capita Symonds in 2010 as part of the Hurlston Brook Strategic Flood Risk Mapping. From this comparison, it was found that catchment descriptors were representative for the area of study therefore, changes were not made.
- The Flood Estimation Handbook (FEH) catchment descriptors were used to calculate the Median Annual Maximum Flow (QMED). A suitable donor site was not available as sites considered were either large in size or had a large lake/reservoir influence. Although, there are a few rural areas in the upstream section of the catchment, Ormskirk settlement dominates a greater proportion of the catchment. The catchment is classed as “moderately urbanised” to “heavily urbanised” with URBEXT2000 values ranging between 0.0615 – 0.2293. An urban adjustment was therefore made to the QMED.
- A pooling group analysis was undertaken using FEH CD-ROM Version 3.0 (2009) and WINFAP-FEH Version 3.0.003 (2009). At the time of analysis, the Jacobs WINFAP-FEH database used Peak Flow data version 3.3.4 dated August 2014, published on the Centre for Hydrology and Ecology (CEH) website.

  WINFAP-FEH allows for pooled analysis to be completed from a group of hydrologically similar catchments to generate flood growth curves. A growth curve estimate was used to establish peak flows at locations HB01 and HB02b.
- Peak flows have been provided at location HB02b to check the routed flows at the downstream extent of the hydraulic model.
- ReFH1 boundary units were set up in Flood Modeller software for location HB01 using the catchment rainfall with an appropriate areal reduction factor and seasonal correction factor applied. Hydrographs were produced for the required AEP events. Design rainfall events for a range of durations were applied.
- For comparison, ReFH2 analysis for the upstream location HB01 was undertaken and compared to the results using ReFH1.
- The Flood Estimation Handbook (FEH) CD-ROM was used to provide rainfall depths at location HB02a (See Figure A.3). ReFH1 was used within Flood Modeller software to distribute the rainfall event over time. The areal reduction factor and seasonal correction factor were removed from the direct rainfall by setting their values to 1.0. A summer rainfall profile was used to simulate the rapid runoff that is associated with urban areas. Sensitivity testing was undertaken by adjusting duration runs, using the design duration and then a shorter duration and a much longer duration.
- A range of storm durations were created in order to determine which storm duration causes the greatest amount of flooding in the area of interest. This was undertaken for 2.2 hour, 3.3 hour, 4.4 hour and 8.8 hour durations with the critical duration occurring at the 4.4 hour duration using the hydrological results; however a different critical duration was identified once all the hydraulic factors were considered by simulating the events through a hydraulic model. There is a risk that the true critical duration occurs between intervals used with a small error in the determination of the peak flow.

A.2.3 Limitations

The statistical methods used, the ReFH1 and FEH, have a degree of uncertainty in the absence of gauged data within the catchment and the lack of suitable donor station(s) to use for data transfer. As a consequence, both approaches merely depend on catchment descriptors. Stations within the pooling group analysis are widely distributed geographically and may be unrepresentative of the subject site. Most of the stations are in the South West or North East or North Ireland with only two in the North West and more than half of the stations have FARL values less than 1 in comparison to the subject site at HB01, which has a FARL value of 1.

For the larger catchment at HB02b, both methodologies gave similar flows for the 1 in 100 year event, whereas, for a the smaller upper catchment at HB01 the flows using REFH1 method were 39% higher for the same AEP event.

Both catchments (i.e. HB01 and HB02b) are slightly permeable with SPRHOST values 27.97% and 28.85% respectively. The Environment Agency Flood Estimation guidelines (Environment Agency Flood Estimation Guidelines, Technical Guidance 197_08, January 2015) recommends the use of FEH statistical approach for permeable catchments, which is defined as catchments with an SPRHOST value below 20%. The study catchment was only a small amount over this threshold and therefore it was assumed that the pooling group approach would be more reliable.

On this basis, flow estimates using the FEH approach were deemed appropriate and recommended for use for this study. The ReFH hydrographs produced were scaled to the statistical peaks to provide the inflows for the rural upper catchment at HB01. The direct rainfall for Ormskirk was then applied within the hydraulic model.

A.2.4 Inputs for model

The hydrological analysis produced the inflows and rainfall hyetographs for a range of storm durations to enable a critical storm duration analysis to be carried out (see section A.3.2).
The fluvial inflows for 3.3hr storm duration for Hurlston Brook at location HB01 calculated in the hydrological analysis are shown in Figure A.4.

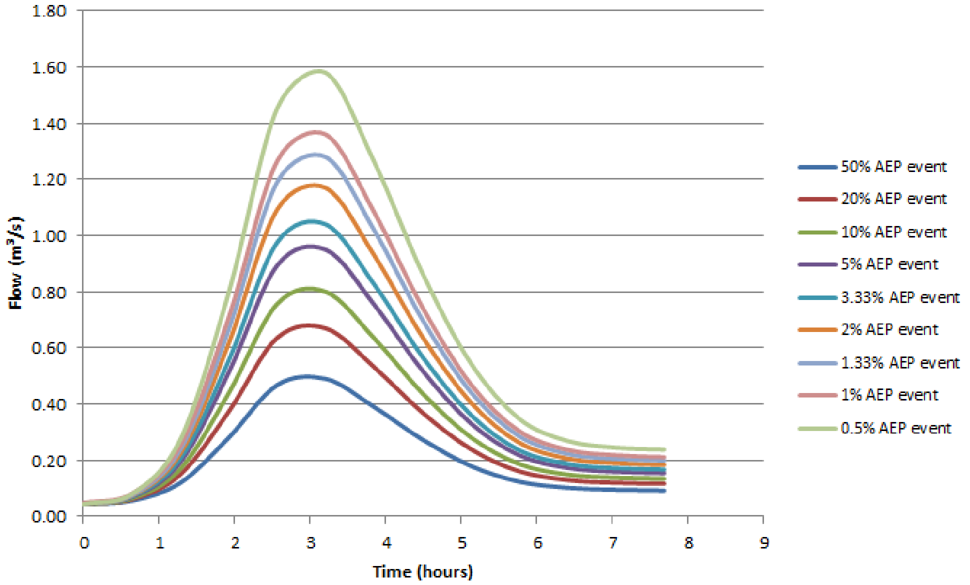

Figure A.4 : Inflow hydrographs for Hurlston Brook inflow at location HB01

The hyetographs for 3.3hr storm duration for the direct rainfall values calculated by the hydrological analysis are shown in Figure A.5.

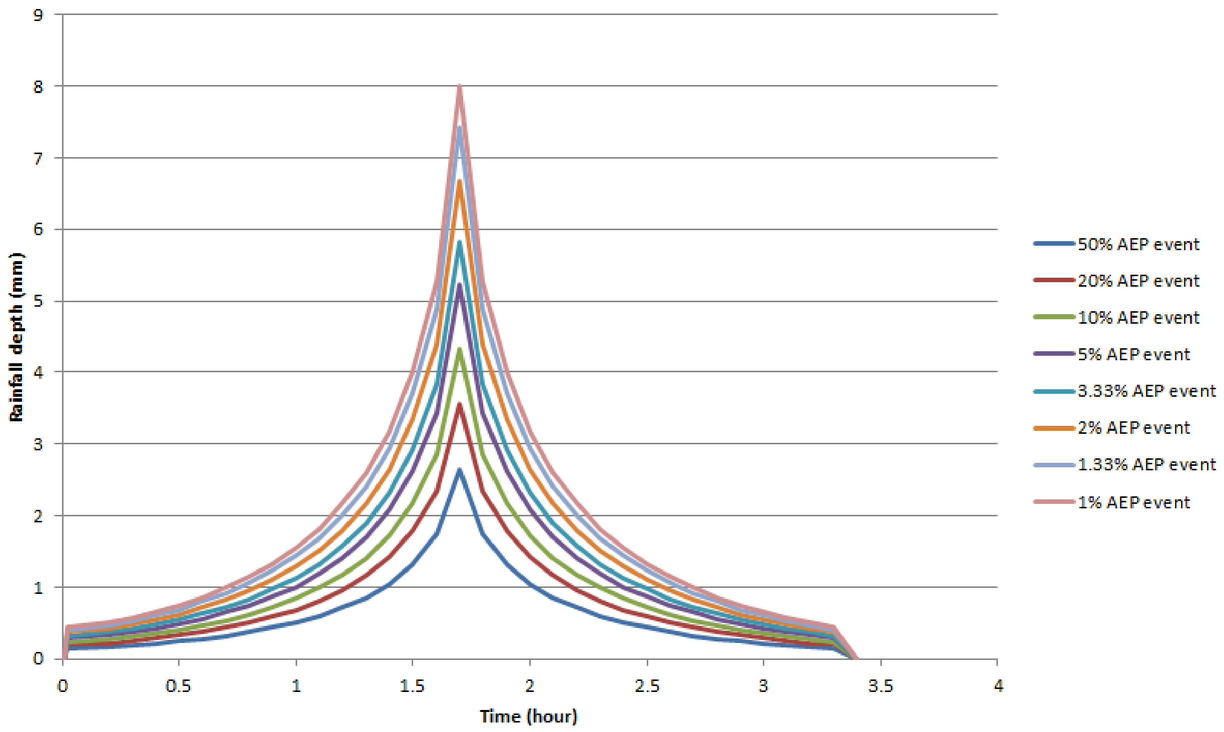

Figure A.5 : Rainfall hyetographs for 3.3hr storm duration

A.3 Baseline Modelling

The following section describes the model schematisation and construction of the Hurlston Brook integrated model.

A.3.1 Model schematisation

A.3.1.1 Fluvial model

As mentioned previously, the ESTRY one dimensional representation of the Hurlston Brook channel throughout Ormskirk was not modified from the incoming SFRM model. Further details on the watercourse schematisation can be found in the Hurlston Brook SFRM report.

- **Boundary Conditions**

The hydrological inflow accounting for the upstream catchment was set as a flow-time boundary at the upstream end of the modelled reach (called HB01 in hydrological analysis).

TUFLOW HX lines were set along the channel bank tops to create hydraulic connectivity between the 1D channel and the 2D floodplain. Elevation points extracted from the LiDAR data were added to inform the bank top lines with accurate ground level elevations.

The downstream boundary of the modelled reach of Hurlston Brook was applied as a HQ boundary using the results from the SFRM model at the truncation point.

A.3.1.2 Drainage network

An existing United Utilities InfoWorks CS network model (Burscough 2015 MM.iwt) was used to create the drainage network component of the hydraulic model. As already mentioned, pipe and manhole information relative to the storm and combined systems were converted into ESTRY and added to the fluvial component of this model. The storm pipes draining into Hurlston Brook were linked dynamically as a 1D-1D connection. The drainage network represented in the model is shown in Figure A.1.

The drainage system was also dynamically linked with the ground surface (2D TUFLOW domain) i.e. water can enter and leave the drainage system depending on the computed flood depth on the surface and the capacity of the drainage system. Road gullies were not modelled explicitly. It was assumed instead that surface water enters the drainage system at each manhole node based on a depth-discharge relationship assuming 4 gullies draining to each manhole. This depth-discharge relationship is provided in Figure A.6 which shows that maximum flow at a manhole is capped at 0.015m^3/s per gully i.e. even if the depth on surface increases the flow entering the drainage system will not increase. This is because flow out of the manhole into the sewer network is limited by a small pipe.

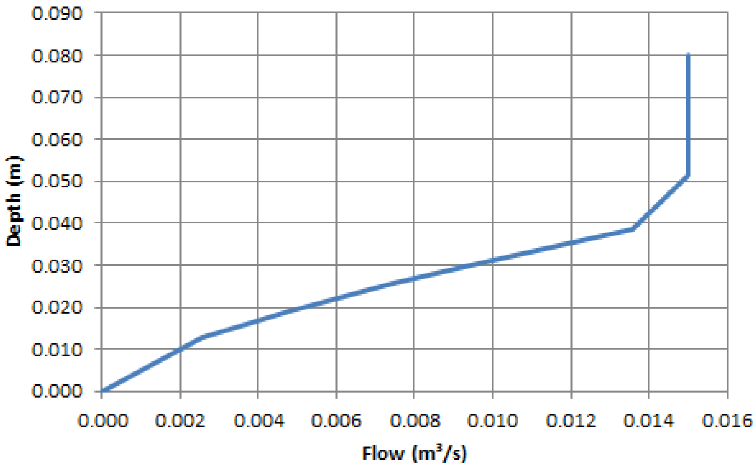

Figure A.6 : Depth-discharge relationship assumed at each Manhole connected to the 2D domain (for each gully)

- **Drainage Network Boundary Conditions**

A number of drainage network pipes ended in Hurlston Brook and so were linked dynamically to the modelled river channel. A free flow boundary condition was applied at the downstream extremities of the other pipes so that the flows that have passed through the area of interest were assumed to be leaving the modelled system.

A.3.1.3 2D component

- **Grid**

The 2D component of the model is based on a grid compound by individual cells of 4m size. This provides adequate accuracy in order to represent urban features such as roads, buildings, gardens, car parks, while not becoming computationally cumbersome considering the size of the study area.

The 2D domain covers an area of 4.96km^2. The ground levels informing the 2D grid are based on the LiDAR DTM (see Table A.1).

Breaklines were used in the 2D domain to define key geographical features that could have been inadequately represented by the 4m DTM grid. In addition, breaklines have been used to improve the model grid definition along the bank tops of the watercourses. A number of breaklines were added in the model to represent minor drains and carve small channels in the ground topography. These breaklines are located north of Little Hall Farm and east of Black Moss Lane. A wall in Coronation Park on the left bank of Hurlston Brook just upstream of Church Fields was already modelled in the SFRM model by Capita. The railway line that runs through Ormskirk, partially in cutting and partially on an embankment was also represented as a breakline.

- **Hydraulic Roughness**

To represent friction in the floodplain, geographical regions of different land use such as roads, buildings, urban areas and green spaces etc. were defined using OS Master Map coverage. The land use regions were processed and input into the 2D model grid so that each 2D grid cell carries a reference number. As shown in Table A.3, Manning's "n" values were assigned to each of the land use reference number.

It should be noted that the use of LiDAR DTM data to populate the 2D model grid means that buildings were not physically represented in the model. Given the fact that any building is an obstruction to the flow and would have a major impact on the overland flow routes, a very high roughness value has been attributed to each building/house to model the effect of the obstruction. The use of a high Manning's "n" value for a building effectively makes it "very difficult" for water to enter / flow through a building, but it does not make it impossible, in contrast to representing a building as a solid "block", through which no water can flow

Table A.3 : Manning's 'n' Coefficients used in the 2D Domain

Land use	MasterMap feature code	Manning's n
Roads, tracks and paths	10172, 10123, 10185, 10183	0.025
General surface manmade, unclassified	10054, 10058	0.025
No element in the area	10119	0.025
Buildings, glasshouses	10021, 10062	1.000
General green areas	10056	0.055
Garden (urban)	10053	0.050
Short grass	10057	0.035
Water bodies	10089, 10210	0.020
Railway	10167	0.050
Trees	10111	0.100
Embankment, cliffs	10093, 10096, 10099	0.050
Channel	100	0.030
General Surface, unclassified	10217	0.035

- **2D Boundaries**

Direct rainfall was applied over the whole 2D domain to generate surface water flows. A free flow boundary condition was applied at the edges of the model at locations where the surface flow was reaching the modelled boundary and it was appropriate to assume that it would leave the modelled system. For this purpose, level-flow (HQ) lines were drawn at the edges of the 2D domain and a slope of 0.01m was assumed (see Figure A.7). TUFLOW then automatically generated the HQ curve to drain the flow from these locations.

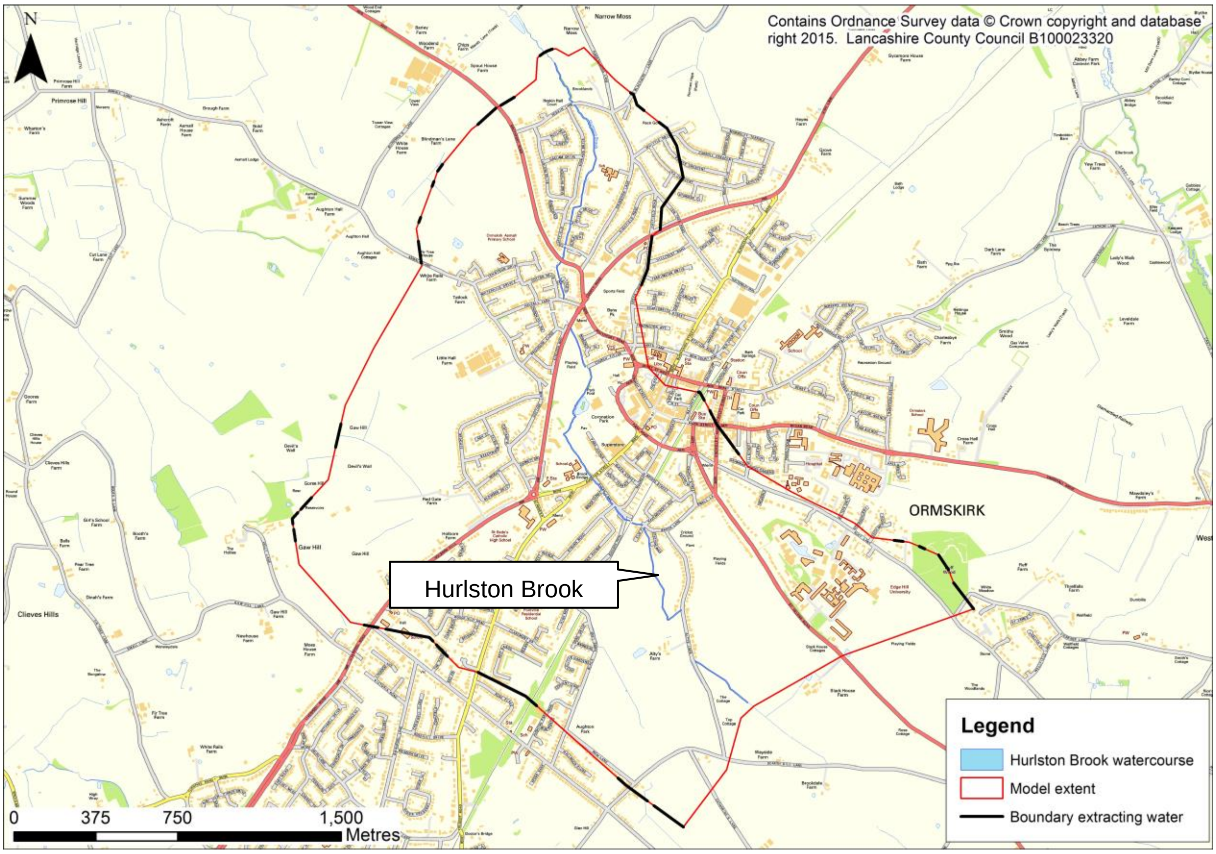

Figure A.7: HQ boundary conditions for model's 2D domain

- **Runoff Coefficients (ground infiltration)**

The rain that falls on the ground surface does not completely get converted to surface runoff as some of it infiltrates into the ground. This rainfall infiltration depends mainly on landuse, the type of soil and the slope of the catchment. The type of soil over the Ormskirk study area is mainly permeable loamy soils over a clayey sub-soil. However, the catchment bedrock is predominantly Sandstone and the superficial geology is dominated by sands with strips of Till on the eastern upstream corner of the catchment and through the middle section of Ormskirk. The sub-soil is slowly permeable due to the higher clay content and can lead to seasonal waterlogging. Types of land use across the study area are further described in the main report in section 4.8 (see Figure 5.2).

Initially a more sophisticated ground infiltration approach based on the Green-Ampt method[3] was applied in the TUFLOW model and various soil types and their combinations were tested. The Green-Ampt method utilises the soil characteristics to calculate the infiltration of the water into the ground. The infiltration rate decreases as the soil becomes saturated. When the soil is dry, the infiltration rate is higher, and as the soil becomes saturated infiltration rate reduces rapidly. Once the soil becomes fully saturated, the infiltration rate becomes constant.

The model results using Green-Ampt method showed that the response of the catchment was not as expected when compared to the observed flood events and the hydrological flow estimates at the downstream end of the model; with unexpectedly large variations from lower order events and higher order events i.e. large growth factors were observed. These results were discussed with the Environment Agency hydrology team and it was decided that a runoff coefficient approach should be adopted.

Instead, a runoff coefficient approach was used. This calculates residual rainfall that would be converted to surface runoff after infiltration. The rainfall coefficient applied is based on the type of surface depicted by the OS MasterMap data and the values used in model are given in Table A.4. Although less sophisticated, the runoff coefficient approach provided a better match with the observed flooding (see section A.4.2).

It must be noted that while there are no standard values for these coefficients, the values used fall in the general range used in the industry for the various landuse types.

Table A.4 : Runoff coefficients used in the integrated model

Land use	MasterMap feature code	Runoff coefficient
Roadside	10183	0.85
Roads or Tracks (Manmade and Natural)	10172	0.85
Paths (Manmade)	10123	0.75
Buildings/glasshouses	10021	0.90
General Surface - Step	10054	0.80
Structure	10185	0.90
Buildings/glasshouses	10062	0.95
General Surface (unclassified)	10217	0.85
General green areas	10056	0.35
Railway	10167	0.35
Trees	10111	0.20
Garden (urban)	10053	0.50
Water bodies	10089	1.00

[3] Section 6.10.1in TUFLOW Manual.2016-03.pdf. This manual uses Rawls, W, J, Brakesiek & Miller, N, 1983, 'Green-Ampt infiltration parameters from soils data', Journal of Hydraulic Engineering, vol 109, 62-71.

A.3.2 Critical Storm Duration Analysis

An analysis was carried out to identify the duration of storm that is critical for flooding of the urban area of Ormskirk. Several summer type storms of durations ranging from 2.2 hours to 8.8 hours were simulated. This analysis was performed using 3.33% and 1% AEP events. The simulation results showed that a storm duration of 3.3 hours was critical for the Ormskirk area as it produced maximum flood depths and extents. The same critical storm duration was used for all events simulated with the hydraulic model.

A.3.3 Baseline Scenarios

Two baseline scenarios were considered for this study:

- A 'Do Minimum' scenario, equivalent to the existing situation, was represented by the baseline hydraulic model.
- A 'Do Nothing' scenario, assuming no maintenance is carried out on both watercourse and the drainage system was represented. The Do Nothing scenario would result, with time, in structural damages/blockages to the assets as well as accumulation of debris and vegetation growth in the watercourses.

To represent this scenario, the following changes were carried out to the 'Do Minimum' model:

- All gullies and manhole connections to the drainage network were removed to mimic blockages to the intakes of the drainage network.
- Key structures on Hurlston Brook prone to blockage were blocked by 90%, a structure at a time, and then the model results aggregated for mapping purposes. These structures were County Road culvert, Aughton Street Bridge, Altys Lane culvert and Southport Road Bridge.
- Channel roughness was increased by 50% for the whole Hurlston Brook channel to account for vegetation growth/debris accumulation.

The results of the baseline scenarios were used for the economic analysis of the flood damages.

A.3.4 Modelled Events

Model simulations were run for a number of AEP events. The majority of these AEP events, namely 20%, 10%, 5%, 2%, 1.33% and 1%, feed into the economic assessment of the options which is discussed in detail within the Scheme Viability report. The use of these AEP events is in accordance with industry wide best practice.

Data gathered during the other AEP events, namely 50%, 3.33% and 0.5% AEP does not contribute to the economic analysis directly, but has been used to help define and develop potential flood risk management options.

Table A.5 shows a summary of the AEP events run for this study for various scenarios.

Table A.5 : Modelled Events

Scenario	AEP Event								
	50%	20%	10%	5%	3.33%	2%	1.33%	1%	0.5%
Do Minimum	✓	✓	✓	✓	✓	✓	✓	✓	✓
Do Nothing	✓	✓	✓	✓	✓	✓	✓	✓	✓
Scheme (glass wall version*)					✓		✓		

Scenario	AEP Event								
	50%	20%	10%	5%	3.33%	2%	1.33%	1%	0.5%
Scheme (with bund crest level set*)		✓	✓	✓		✓	✓	✓	

*See section A.6 for details

A.4 Model proving

The next sections discuss the stages related to the verification process and how the information provided and produced by the model aided to check model performance. Additionally, details regarding additional runs carried out to test the sensitivity of the model in key variables are also discussed.

A.4.1 Performance

TUFLOW hydraulic modelling software provides run performance guidance along with acceptable error ranges that should be achieved during each model run. The concept of an acceptable error range has been adopted by the developers of the software, as numerical errors occur due to the quality of the data used, limitations of the software and underlying equation solving processes.

Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure the optimum model convergence at any computed time step. In particular, the Cumulative Mass Balance Error reports associated with both 1D (i.e. drainage and fluvial system) and 2D domains have been considered. For all the simulations undertaken, the latter parameters were found to be acceptable, staying within recommended tolerance ranges.

In addition, model outputs have been thoroughly reviewed to:

- Track any sign of instability or inconsistency between simulated events of increasing magnitude.
- Ensure energy loss coefficients throughout the drainage system and velocity/ depth distribution across the modelled area were sensible in both 1D and 2D domains.

A.4.2 Model Verification

Calibration of the model was not possible as there are no gauging stations on Hurlston Brook within the study area nor immediately downstream of Ormskirk; however, verification was undertaken with the best available information to have more confidence in the modelled results. This was discussed in detail with the Environment Agency hydrologists. The approach used for verification was:

<u>Hydrological flow estimates</u>

- Comparison of model routed flows against the hydrological flow estimates (ReFH1) shown in Table A.6 at the downstream end of the model (see Figure A.3) was carried out. The aim was to evaluate the reconciliation of routed flows with the hydrological flow estimates at the downstream end of the model.

  Due to the mixed rural and urban nature of the catchment and also because of the particularly steep topography to the west of the catchment the model results show two distinct flow peaks (i.e. one pluvial driven and the other one fluvial driven) for the routed flow through Hurlston Brook.

  Therefore it was not possible to achieve a perfect match between the modelled flows and the hydrological estimates. However, an effort was made to adjust the runoff coefficients in the model such that the simulated results for lower order events, which have been observed by the local residents, provided a good match. The details of this reconciliation process were provided in a technical note submitted to the Environment Agency. Table A.6 shows the outcomes of the reconciliation exercise.

Table A.6 : ReFH1 peak flow estimates at the downstream end of model (HB02b)

AEP	ReFH1 estimate	Routed peak flow (m^3/s) Runoff coefficients approach
20% AEP	2.8	3.0
10% AEP	3.5	4.3

AEP	ReFH1 estimate	Routed peak flow (m^3/s) Runoff coefficients approach
3.33% AEP	4.4	6.7
1% AEP	6.2	8.9

Flood history

- A comparison of the modelled results was carried out using the data compiled from local residents, with the help of questionnaires circulated by Jacobs. This information focused on flow routes observed, estimated flood depth, possible source seen and how frequent the flooding occurred.

  It is not possible to relate the observed surface water flooding events to a design AEP event analytically; however based on the information gathered, and with the help of questionnaires (prior to December 2015 event), best judgement of the flood frequency was used, and for Halsall Lane and Redwood Drive a probability between 20%AEP and 10%AEP event was deemed reasonable. The model results showed a good match with the observed surface water flooding on Halsall Lane and Redwood Drive for 20% and 10% AEP events using the runoff coefficient approach.

Published EA flood maps (uFMfSW)

- Comparison of model flood extents (using a 150mm cut-off depth) against the published flood extent from the EA website was also carried out. This comparison showed a varied picture with some of the areas matching well and other showing differing results.

December 2015 flood event

The December 2015 event was also simulated as part of the verification process. The Environment Agency provided data for 6 rainfall gauging stations around Ormskirk (see Figure A.8) since there were no gauge station at the study area. The Thiessen polygons method was used in order to obtain a representative rainfall hyetograph to be applied to Hurlston Brook model. Two rainfall events were covered within the applied hyetograph, between 25/12/2015 07:00:00 to 26/12/2015 16:00:00. This synthesised rainfall hyetograph was applied as direct rainfall onto the 2D domain as well as for creating flow for the inflow (HB01) at the upstream end of the model.

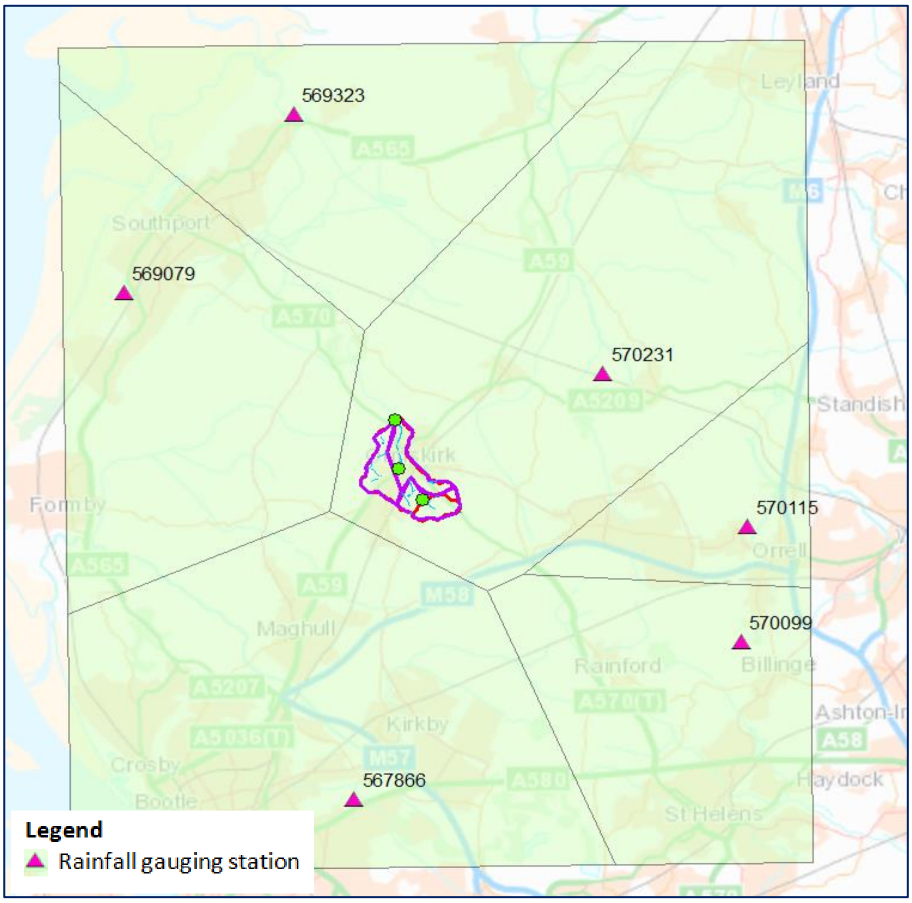

Figure A.8: Rain gauges around Ormskirk for which data was provided for December 2015 event.

The information gathered about the December 2015 flood event did not include information regarding flood depths. However, the results reveal the following:

- The eastern side of Halsall Lane presents a good match between the observed and modelled flooding, similarly at the eastern side of Cotton Drive, Sanfield Close and properties west of Southport Road (near Cotton Drive).

- Flow routes coming from the west and reaching Redgate are also represented in the modelling. This flow route continues east along Garnett Grove and enters Hurlston Brook. The ponding near Redgate Farm is also matched in the observed and modelled results.

- Observed flooding at Altys Lane and the properties on the east side is also represented by the simulation results.

- Properties flooded along Brook Lane on Statham Way are not flooded according to the model results, although the Brook Lane itself is flooded. This flooding could possibly be an impact of local changes (e.g. bridges constructed by residents). It could not be confirmed whether this is a temporary feature or not.

- The model results also show flooding on the east of Dyers Lane immediately south of Aughton Street, which was observed in the December 2015 event.

As a result of the verification exercise using the various sources and after discussion with the Environment Agency hydrologists, it was decided that the model predicted flooding mechanism and flood depths were a reasonable representation of the conditions in the area, and the model was deemed suitable to be used for further work of scheme identification and economic analysis. The Environment Agency also confirmed that the runoff coefficients approach, using the values provided in Table A.4 was a practical way forward.

A.4.3 Sensitivity analysis

The following section outlines the sensitivity analysis applied to the model for the 5% AEP event. The primary focus of this analysis was to test the model sensitivity to key variables to ensure that a robust and conservative approach was adopted. These variables relate to both the model build/schematisation and input data. Information relating to each of these is provided below. Monitoring points (see Figure A.10) were used to track the changes between various scenarios in flood depth and water levels at key locations (see Section 5.3: 'Stage 3 – Testing' of the Viability report for more detail).

A.4.3.1 Number of gullies

As discussed in section A.3.1.2, the 1D drainage system is linked to the surface model at manhole locations assuming that typically 4 gullies would drain to each manhole. A sensitivity test was performed on the number of gullies associated with each manhole (TUFLOW pit channel), using 2 and 6 gullies.

The model results comparison with Do Minimum showed that the change in maximum water depth was less than 40mm in magnitude when the number of gullies was increased or decreased by 2, as shown in Table A.7. This means that the model is not very sensitive to the assumption made for the number of gullies.

A.4.3.2 Runoff coefficient

To assess the sensitivity of the model to the assumed natural ground infiltration, an increase and decrease of 20% was applied to the runoff coefficients for all the landuse types.

The change in maximum water depths compared to Do Minimum is provided in Table A.7 and the sensitivity test showed low sensitivity, at the monitoring points, to the changes in the runoff coefficient as the magnitude of change was less than 80mm. The only outlier in this analysis is the monitoring point 14 (bridge on Heskin Lane) which is close to the downstream end of the model and is thought to be the cumulative impact of the overall increase in flows from the whole catchment, resulting in an increase of water depths over the bridge.

A.4.3.3 Manning roughness

To assess the sensitivity of the model to the assumed ground roughness, the model was run with an increase and decrease of 20% in the Manning's 'n' coefficient applied to all land use types.

The comparison of the maximum water depths at monitoring points with the Do Minimum results indicates that the model is not sensitive to changes in Manning's 'n' coefficient. Table A.7 shows that the magnitude of change is less than 40mm at the monitoring points.

Table A.7 : Difference (m) in maximum water depths between Do Minimum and the modelled sensitivity tests for 5% AEP event

Sensitivity test	1	2	3	4	5	6	7	8	9	10	11	12	13	14
2 Gullies	0.00	-0.01	0.00	-0.01	0.00	0.01	0.00	0.00	0.01	0.00	-0.01	0.01	0.04	0.00
6 Gullies	0.00	-0.01	0.00	-0.02	0.00	0.01	0.00	0.00	0.00	0.00	-0.01	0.01	0.03	-0.01
Runoff coeff.+20%	0.00	0.00	0.00	0.03	0.06	0.05	0.03	0.01	0.03	0.05	0.02	0.04	0.07	0.43
Runoff coeff.-20%	0.00	-0.02	0.00	-0.08	-0.08	-0.06	-0.03	0.00	-0.03	-0.06	-0.05	-0.03	0.00	-0.04
Roughness +20%	0.00	-0.01	0.00	-0.01	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.01	0.04	-0.01
Roughness -20%	0.00	-0.01	0.00	-0.01	0.00	0.01	0.00	0.00	0.00	0.00	-0.01	0.01	0.04	0.00

A.5 Baseline model results

A.5.1 Model outputs

The model outputs from the baseline scenarios were used to help understand the existing flooding mechanism and the extent of flooding reaching the areas with a flood risk history. For more details regarding the flow paths, refer to Section 4 of the main Viability Report.

The integrated model was able to provide the following information which was used in different aspects of this study:

- Maximum water depth grids were produced to compare the different scenarios as well as for the proposed scheme components. Water depths were used for carrying out the economic analysis.

- Maximum water level grids from glass wall version (see section A.6) were used to set the bund crest levels for proposed scheme bunds.

- Animations of the velocity results as vectors (arrows) were created and used to identify the flow routes that helped selection of the locations for scheme bunds.

- The model also provided information about the state of the drainage system i.e. flow entering and leaving the drainage system, surcharging manholes, pipes running at full capacity etc. This helped understand the constraints in the system and also identify any possible mitigation options.

- 2D time-series outputs were generated using TUFLOW Plot Output (PO) lines and points for flow and water level (see Figure A.9). These time-series results helped understand the behaviour of the flooding over time at different locations. This information, along with the velocity animations, provided a better understanding of the flood mechanism and the magnitude of the contributing smaller flow routes to a major flow route.

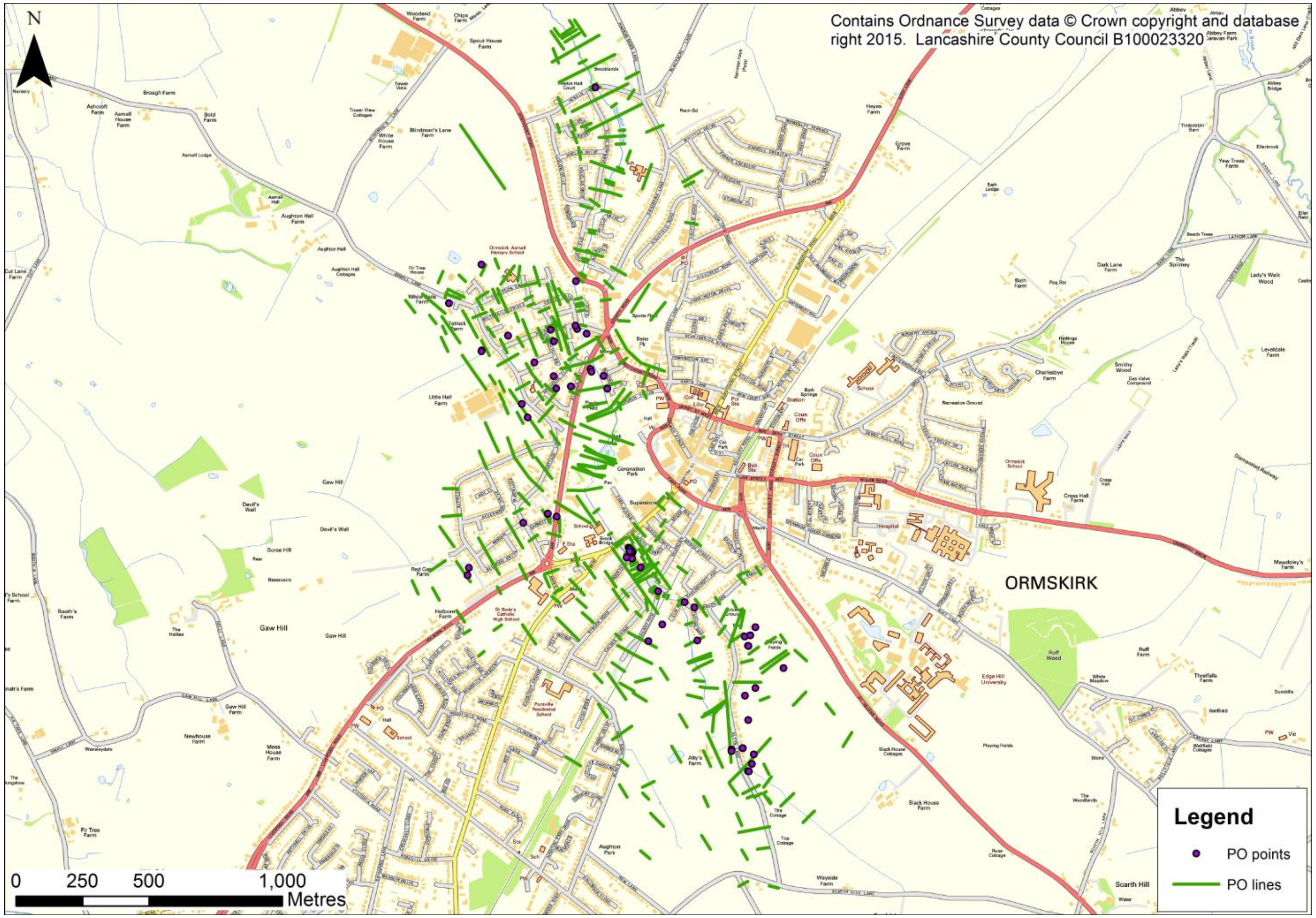

Figure A.9 : PO points and lines at the study area

Labels were not shown because of the limited space to display them

- Monitoring points were used to track the changes between various scenarios in flood depth and water levels at key locations (see Section5.3: ‘Stage 3 – Testing’ of the Viability report for more detail). The monitoring points are shown in Figure A.10. The areas chosen to be monitored were associated with historical flooding (points 1 to 14) as the aim was to reduce the flooding issues at these locations. During the analysis, additional points were added (points 101 to 113) to track the changes occurring at other locations which were important in terms of flow routes, however these additional points are not discussed in this Appendix. Using the data extracted at these monitoring it was possible to compare the effects of the scheme against the Do Minimum.

A.5.2 Model Results

The model results show that the nature of the topography of the catchment is such that the some areas get affected by the flooding from both pluvial and fluvial sources. The ground slopes steeply from the rural area in the west towards Hurlston Brook and there is a drop in ground elevation of 34-35m in 1300m (1 in 37 approximately) from Gaw Hill to Hurlston Brook near Aughton Street. This results in a quick response from the western part of the catchment. The modelled reach of Hurlston Brook watercourse drops by approximately 24-25m in approximately 3500m (1 in 145 approximately). As a result of this topography, the model simulations show two distinct flow peaks for the routed flow through Hurlston Brook for the same storm duration.

The hydraulic model results in terms of flooding mechanism and flood risk at the areas susceptible of flooding are fully described in Section 4 of the main report.

The maximum flood depths at the monitoring points predicted by the model for the Do Minimum scenario are provided in Table A.8. The Do Nothing scenario was simulated four times for each of the blocked culvert independently. The results (Do Nothing minus Do Minimum) are presented in Table A.9 : Difference (m) in maximum water depths Do Nothing (County Road culvert) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.001	-0.005	0.002	0.129	0.202	0.052	0.040	0.005	0.026	0.082	0.023	0.061	0.082	0.019
10%	-0.001	-0.007	0.001	0.057	0.047	0.066	0.029	0.005	0.022	0.066	0.026	0.051	0.060	0.030
5%	0.000	-0.007	0.001	0.032	0.036	0.042	0.019	0.005	0.019	0.055	0.017	0.043	0.061	0.161
2%	0.004	-0.007	0.001	0.018	0.030	0.059	0.013	0.006	0.012	0.042	0.016	0.032	0.064	0.062
1.33%	0.004	-0.006	0.002	0.008	0.042	0.062	0.009	0.007	0.008	0.040	0.017	0.030	0.081	0.022
1%	0.005	-0.006	0.002	0.033	0.023	0.047	0.008	0.008	0.012	0.038	0.017	0.030	0.083	0.022

Table A.10 : Difference (m) in maximum water depths Do Nothing (Aughton Street Bridge) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.001	-0.005	0.002	0.129	0.220	0.419	0.040	0.005	0.026	0.001	-0.008	0.008	0.082	0.019
10%	-0.001	-0.007	0.001	0.057	0.054	0.416	0.029	0.005	0.022	0.000	-0.010	0.010	0.059	0.030
5%	0.000	-0.007	0.001	0.032	0.040	0.404	0.019	0.005	0.019	-0.004	-0.013	0.006	0.054	0.034
2%	0.004	-0.007	0.001	0.018	0.034	0.420	0.013	0.006	0.012	-0.005	-0.014	0.005	0.048	0.009
1.33%	0.004	-0.006	0.002	0.005	0.047	0.413	0.009	0.007	0.008	-0.004	-0.014	0.005	0.056	0.005
1%	0.005	-0.006	0.002	0.042	0.027	0.391	0.008	0.008	0.012	-0.002	-0.013	0.007	0.063	0.006

Table A.11 : Difference (m) in maximum water depths Do Nothing (Altys Lane culvert) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	0.001	0.017	0.318	-0.187	0.221	0.067	0.040	0.005	0.026	0.022	-0.004	0.022	0.082	0.019
10%	0.002	0.015	0.337	-0.279	0.059	0.070	0.029	0.005	0.022	0.018	0.002	0.023	0.059	0.030
5%	0.003	0.012	0.365	-0.332	0.038	0.043	0.019	0.005	0.019	0.015	-0.004	0.018	0.055	0.149
2%	0.013	0.015	0.418	-0.398	0.031	0.062	0.013	0.006	0.012	0.012	-0.004	0.015	0.054	0.060

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
1.33%	0.015	0.014	0.435	-0.428	0.043	0.063	0.009	0.007	0.008	0.011	-0.003	0.014	0.089	0.022
1%	0.016	0.011	0.485	-0.441	0.022	0.048	0.008	0.008	0.012	0.010	-0.003	0.014	0.084	0.021

Table A.12 : Difference (m) in maximum water depths Do Nothing (Southport Road Bridge) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.001	-0.005	0.318	-0.187	0.202	0.052	0.040	0.005	0.026	0.018	-0.005	0.019	0.309	0.019
10%	-0.001	-0.007	0.337	-0.279	0.047	0.066	0.029	0.005	0.022	0.015	0.001	0.022	0.303	0.030
5%	0.000	-0.007	0.365	-0.332	0.036	0.042	0.019	0.005	0.019	0.014	-0.005	0.017	0.308	0.164
2%	0.004	-0.007	0.418	-0.398	0.030	0.059	0.013	0.006	0.012	0.009	-0.006	0.013	0.312	0.049
1.33%	0.004	-0.006	0.436	-0.428	0.041	0.061	0.009	0.007	0.008	0.009	-0.005	0.013	0.310	0.025
1%	0.005	-0.006	0.483	-0.441	0.021	0.046	0.008	0.008	0.012	0.009	-0.003	0.013	0.272	0.024

to Table A.12.

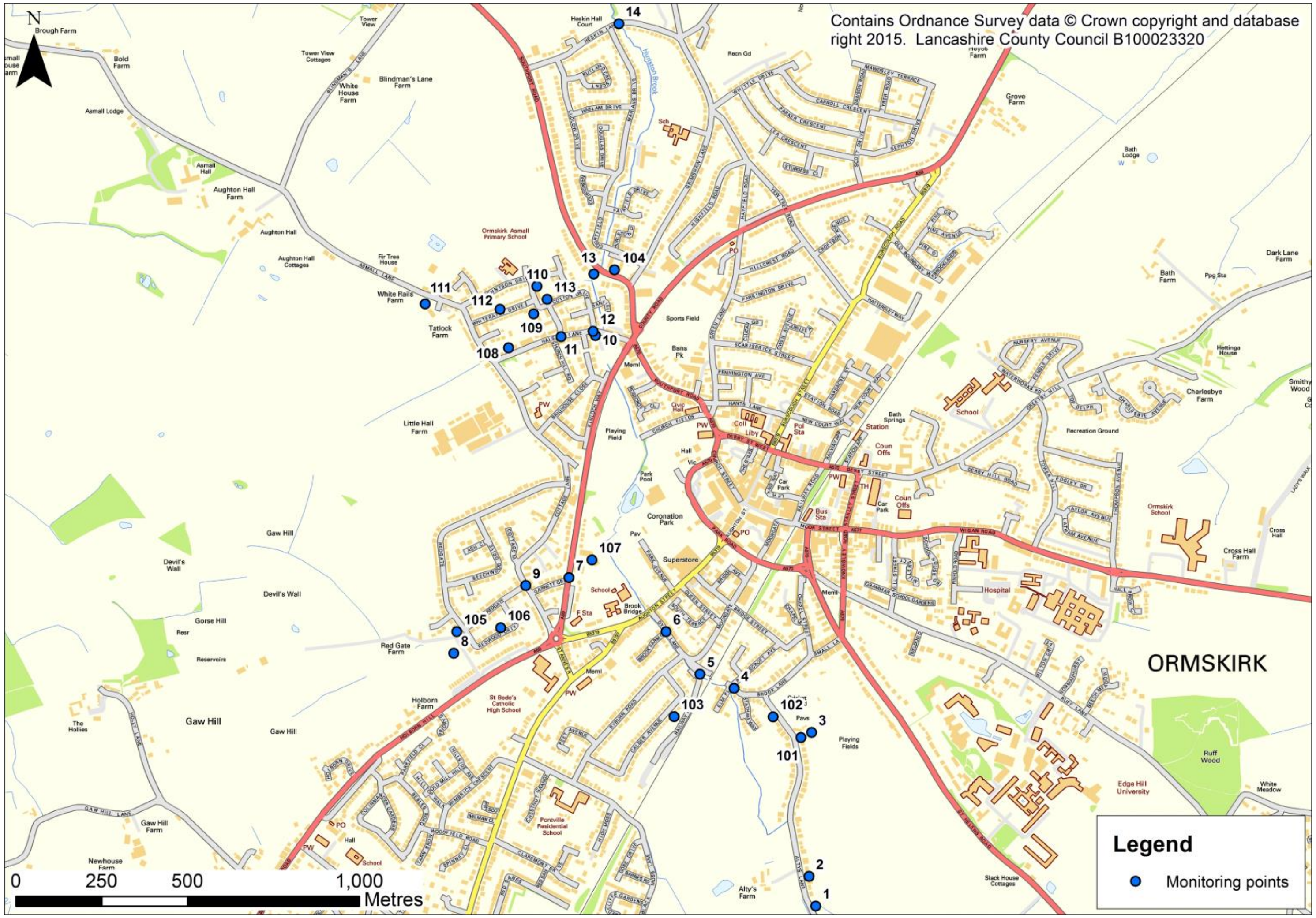

Figure A.10 : Monitoring points used to compare flood depth at key location

Table A.8 : Do Minimum - Maximum water depths in metres at the study area for the monitoring points

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	0.013	0.215	0.014	0.203	0.023	0.100	0.045	0.013	0.070	0.261	0.071	0.125	0.744	0.106
10%	0.015	0.231	0.014	0.294	0.245	0.143	0.073	0.016	0.099	0.312	0.093	0.160	0.784	0.119
5%	0.018	0.245	0.015	0.348	0.334	0.211	0.100	0.018	0.128	0.362	0.128	0.194	0.813	0.142
2%	0.023	0.265	0.015	0.414	0.424	0.276	0.134	0.022	0.167	0.429	0.173	0.241	0.855	0.722
1.33%	0.029	0.275	0.015	0.445	0.463	0.321	0.153	0.024	0.183	0.460	0.197	0.261	0.880	0.821
1%	0.033	0.285	0.016	0.459	0.501	0.360	0.162	0.026	0.190	0.483	0.214	0.275	0.934	0.856

Table A.9 : Difference (m) in maximum water depths Do Nothing (County Road culvert) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.001	-0.005	0.002	0.129	0.202	0.052	0.040	0.005	0.026	0.082	0.023	0.061	0.082	0.019
10%	-0.001	-0.007	0.001	0.057	0.047	0.066	0.029	0.005	0.022	0.066	0.026	0.051	0.060	0.030
5%	0.000	-0.007	0.001	0.032	0.036	0.042	0.019	0.005	0.019	0.055	0.017	0.043	0.061	0.161
2%	0.004	-0.007	0.001	0.018	0.030	0.059	0.013	0.006	0.012	0.042	0.016	0.032	0.064	0.062
1.33%	0.004	-0.006	0.002	0.008	0.042	0.062	0.009	0.007	0.008	0.040	0.017	0.030	0.081	0.022
1%	0.005	-0.006	0.002	0.033	0.023	0.047	0.008	0.008	0.012	0.038	0.017	0.030	0.083	0.022

Table A.10 : Difference (m) in maximum water depths Do Nothing (Aughton Street Bridge) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.001	-0.005	0.002	0.129	0.220	0.419	0.040	0.005	0.026	0.001	-0.008	0.008	0.082	0.019
10%	-0.001	-0.007	0.001	0.057	0.054	0.416	0.029	0.005	0.022	0.000	-0.010	0.010	0.059	0.030
5%	0.000	-0.007	0.001	0.032	0.040	0.404	0.019	0.005	0.019	-0.004	-0.013	0.006	0.054	0.034
2%	0.004	-0.007	0.001	0.018	0.034	0.420	0.013	0.006	0.012	-0.005	-0.014	0.005	0.048	0.009
1.33%	0.004	-0.006	0.002	0.005	0.047	0.413	0.009	0.007	0.008	-0.004	-0.014	0.005	0.056	0.005
1%	0.005	-0.006	0.002	0.042	0.027	0.391	0.008	0.008	0.012	-0.002	-0.013	0.007	0.063	0.006

Table A.11 : Difference (m) in maximum water depths Do Nothing (Altys Lane culvert) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	0.001	0.017	0.318	-0.187	0.221	0.067	0.040	0.005	0.026	0.022	-0.004	0.022	0.082	0.019
10%	0.002	0.015	0.337	-0.279	0.059	0.070	0.029	0.005	0.022	0.018	0.002	0.023	0.059	0.030
5%	0.003	0.012	0.365	-0.332	0.038	0.043	0.019	0.005	0.019	0.015	-0.004	0.018	0.055	0.149
2%	0.013	0.015	0.418	-0.398	0.031	0.062	0.013	0.006	0.012	0.012	-0.004	0.015	0.054	0.060
1.33%	0.015	0.014	0.435	-0.428	0.043	0.063	0.009	0.007	0.008	0.011	-0.003	0.014	0.089	0.022
1%	0.016	0.011	0.485	-0.441	0.022	0.048	0.008	0.008	0.012	0.010	-0.003	0.014	0.084	0.021

Table A.12 : Difference (m) in maximum water depths Do Nothing (Southport Road Bridge) vs Do Minimum scenario

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.001	-0.005	0.318	-0.187	0.202	0.052	0.040	0.005	0.026	0.018	-0.005	0.019	0.309	0.019
10%	-0.001	-0.007	0.337	-0.279	0.047	0.066	0.029	0.005	0.022	0.015	0.001	0.022	0.303	0.030
5%	0.000	-0.007	0.365	-0.332	0.036	0.042	0.019	0.005	0.019	0.014	-0.005	0.017	0.308	0.164
2%	0.004	-0.007	0.418	-0.398	0.030	0.059	0.013	0.006	0.012	0.009	-0.006	0.013	0.312	0.049
1.33%	0.004	-0.006	0.436	-0.428	0.041	0.061	0.009	0.007	0.008	0.009	-0.005	0.013	0.310	0.025
1%	0.005	-0.006	0.483	-0.441	0.021	0.046	0.008	0.008	0.012	0.009	-0.003	0.013	0.272	0.024

A.6 Scheme Modelling

A.6.1 Scheme Model Build

Model outputs from the Do Minimum scenario were used to inform the development of flood mitigation measures designed to alleviate flood risk in key risk areas. These measures were tested using the hydraulic model.

A summary of the scheme development process is given as follows:

- Areas where flooding was aimed to be reduced were identified using flow routes and monitoring points.

- Potential scheme options were first tested individually which comprised of drainage pipe size increase and storage bunds to be simulated as ‘glass walls’. A glass wall bund is a conceptual barrier to flow that allows flood storage behind the bund and does not allow any water to overtop. Tuflow ‘z-shape’ lines were used to model the bunds.

- The increased drainage pipe size options were not progressed because these options would put more treatment load on the Burscough Waste Water Treatment Works. Also, the drainage network assets are owned by United Utilities and not by LCC.

- The impacts of the scheme components on flood depths were analysed using maximum depth grids and maximum depths reported at the monitoring points.

- Combinations of the individual bund options tested previously were prepared and glass wall versions were simulated for the 3.33% and 1.33% AEP events. These bunds did not include any outlets to discharge the flows from the storage areas (except at Altys Lane and Coronation Park that allow discharge through the existing Hurlston Brook culverts).

  The proposed scheme component near Redgate Farm included a 250mm high conceptual road hump on Holborn Hill to divert the flow towards the storage area. The details of this arrangement should be refined at a later design stage.

- For the final proposed scheme, the crest elevations for the bunds were set using maximum water levels from the glass wall versions of the model (without any freeboard).

  - The bund crest levels were first set using the 3.33% AEP event maximum water levels at the storage areas and 20%, 10%, 5%, 2%, 1.33% and 1% AEP events were simulated.

  - The second set of simulations was run with bund crest levels using the 1.33% AEP event maximum water levels at the storage areas and the same six AEP events were simulated.

- The overall length of each bund comprised of two distinct sections depending on the function they carried out. The height of the bund in each section, i.e. the crest level, depended on this function:

  - **Diversion section:** The areas where the bund is designed to divert the flow were set to 250mm added to ground elevation. Because the water is traveling adjacent to these areas and not accumulating behind them, this height was found to be enough to keep the water behind the bund.

  - **Storage section:** The other section of the bund is where the water actually gets stored. For the storage sections of the bund, the crest elevations were set using the glass wall version results for the 3.33% AEP and 1.33% AEP events respectively. No freeboard was included to these maximum water levels. Outlets through the bunds were included in the scenario. The simulations were then run for 20%, 10%, 5%, 2%, 1.33% and 1% AEP events.

Locations of the proposed bunds and the crest levels set using the 3.33% AEP event and 1.33% AEP event can be found in Figure A.11.

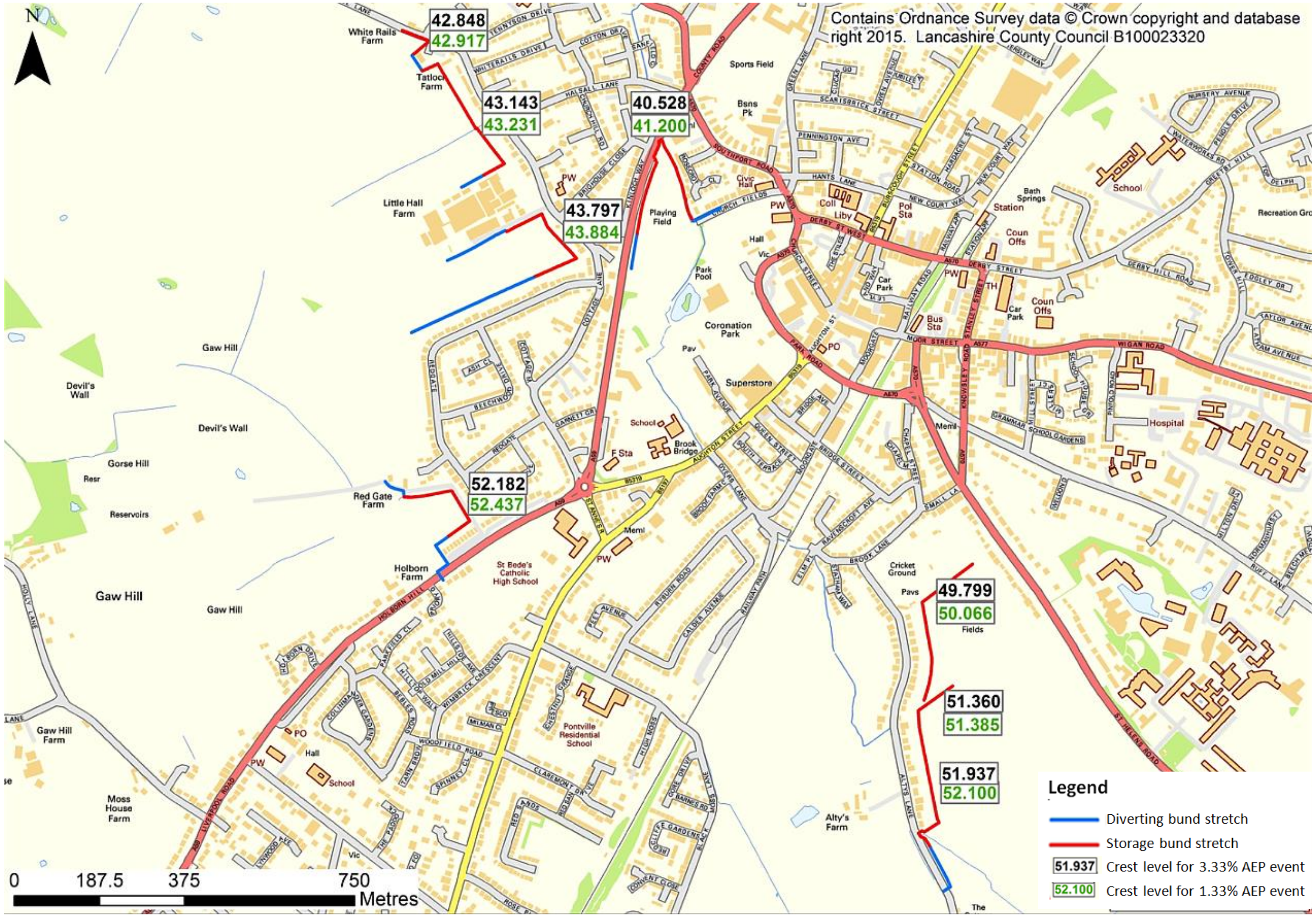

Figure A.11 : Design scheme crest levels and bund locations

A.6.2 Scheme Model Results

As a consequence of the flow being stored, the surface runoff reaching the key flood risk areas reduces significantly compared to the Do Minimum scenario. Maximum flood depth maps are provided in Appendix B of the Viability Report.

Table A.13 and Table A.14 provide the difference in maximum water depth compared to the Do Minimum scenario at the monitoring points (Figure A10) for the 3.33% AEP event and 1.33% AEP event crest level scenarios. Negative values highlight areas where the flood depths have reduced and vice versa.

Table A.13 : Difference (m) in maximum water depths between Do Minimum and the modelled 3.33% AEP event crest level

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.013	-0.146	-0.001	-0.115	0.002	0.006	-0.015	0.578	-0.032	-0.116	-0.023	-0.069	0.030	-0.015
10%	-0.015	-0.157	-0.001	-0.066	-0.219	-0.016	-0.030	0.707	-0.050	-0.144	-0.039	-0.091	0.020	-0.017
5%	-0.018	-0.165	-0.001	-0.059	-0.151	-0.065	-0.028	0.833	-0.064	-0.170	-0.066	-0.107	0.019	-0.027
2%	-0.011	-0.149	-0.002	-0.078	-0.109	-0.054	-0.036	1.014	-0.078	-0.107	-0.084	-0.064	0.005	-0.578
1.33%	-0.012	-0.115	0.002	-0.085	-0.109	-0.071	-0.044	1.099	-0.078	-0.092	-0.082	-0.057	-0.010	-0.631
1%	-0.013	-0.104	0.005	-0.081	-0.124	-0.096	-0.047	1.162	-0.072	-0.087	-0.083	-0.052	-0.057	-0.445

Table A.14 : Difference (m) in maximum water depths between Do Minimum and the modelled 1.33% AEP event crest level

AEP Event	1	2	3	4	5	6	7	8	9	10	11	12	13	14
20%	-0.013	-0.146	-0.001	-0.116	0.002	0.006	-0.015	0.584	-0.033	-0.116	-0.023	-0.069	0.029	-0.015
10%	-0.015	-0.157	-0.001	-0.066	-0.219	-0.016	-0.029	0.722	-0.051	-0.144	-0.040	-0.091	0.020	-0.017
5%	-0.018	-0.165	-0.001	-0.059	-0.152	-0.065	-0.029	0.860	-0.066	-0.170	-0.067	-0.107	0.019	-0.027
2%	-0.023	-0.175	-0.002	-0.078	-0.110	-0.054	-0.037	1.057	-0.085	-0.218	-0.101	-0.137	0.005	-0.577
1.33%	-0.029	-0.179	-0.002	-0.085	-0.110	-0.073	-0.045	1.149	-0.088	-0.242	-0.119	-0.146	-0.010	-0.660
1%	-0.033	-0.185	-0.002	-0.081	-0.124	-0.096	-0.049	1.219	-0.087	-0.220	-0.132	-0.137	-0.057	-0.683

A.7 Assumptions and Limitations

The accuracy and validity of the model results are heavily dependent on the accuracy of the hydrological and topographic data included in the model. Whilst the most appropriate available information has been used to construct the model, there are assumptions and limitations associated with the model. These are listed below:

1) There is a degree of uncertainty in the inflows used in the absence of gauged data within the catchment. The lack of suitable donor station(s) to use for data transfer means that the hydrological methods used are reliant on the catchment descriptors only.

2) The accuracy of the 2D model is 4m cell size informed with the latest LiDAR data and roughness information from OS MasterMap. Although appropriate for the level of detail required for this study, a finer grid resolution could provide slightly different overland flow paths in an urban environment.

3) The hydraulic model does not allow for a detailed representation of the surface water entering the drainage network via road gullies and connecting pipes discharging into the main sewers. Instead, it is assumed that flows enter into the drainage system at the manholes. An assumption of 4 gullies per manhole was made for the Do Minimum scenario. However the sensitivity analysis on 5% AEP event demonstrated that the model results are not sensitive to this assumption.

4) The drainage network represented in the model assumes a clean system with no blockages or sediment built as the status of the whole pipe network used in the hydraulic model is not certain.

5) The Do Nothing scenario assumes 90% blockage at 4 key culverts in addition to other changes. These assumptions are deemed reasonable for the nature of this study.

6) Ground infiltration was accounted for by using a runoff coefficient approach which assumes that the surface runoff is dependent on the landuse type only and not on the antecedent soil moisture conditions. Thus the model assumes that the permeable surfaces can infiltrate some of the rainfall volume. In a worst case scenario the soil could be fully saturated prior to the rainfall event i.e. almost 100% of the rainfall volume would be converted to surface runoff. This scenario has not been simulated.

7) No threshold levels have been applied explicitly for the properties in the hydraulic model. The impact of the buildings on the surface water flow paths has been accounted for by using high roughness over the building polygons.

8) The model has not been calibrated due to an absence of any gauged data. The model was verified through comparison with observed flooding, hydrological flow estimate reconciliation and model performance checks and by generally ensuring model results are sensible. The Do Minimum flood extents were compared to published surface maps for two locations, Halsall Lane and Redwood Drive.

9) The same storm duration has been used for the scheme model as used for the Do Minimum scenario.

10) The downstream boundaries of the model assume free flow in 1D and 2D and therefore they extract the water from the model to avoid ponding. They are located far enough from the area of interest to have any effect.

A.8 Conclusion

A direct rainfall One-Dimensional (1D) – Two-Dimensional (2D) model has been built to represent the existing and scheme scenarios for the area of Ormskirk. The model results have been used to prepare the scheme options and these are discussed in detail in the main Optioneering Report along with discussion about the flooding mechanism.

The proposed scheme components were modelled as bunds and tested by setting the crest levels for two different levels i.e. at 3.33% AEP event’s level and 1.33% AEP event’s level respectively. This means that there is no overtopping of the storage areas for any AEP event more frequent than the set level’s AEP event.

The proposed scheme results in a significant reduction in the flood depths at targeted areas which were historically affected by flooding but there is residual flooding present at some locations. The residual flooding is a result of local flow routes that are not intercepted by the scheme.

A number of assumptions are associated with the model building process; these have been discussed in the Assumptions and Limitation section.