Gulley optimisation

An investigation into the potential for gulley optimisation to reduce maintenance requirement and to reduce surface water flood risk

Simon Doncaster, John Blanksby, Will Shepherd, and Gavin Sailor, Pennine Water Group, University of Sheffield

Summary

This report describes the work carried out to investigate the potential for gulley optimisation to reduce maintenance requirement and to reduce surface water flood risk.

The work includes full scale laboratory testing and field monitoring of gullies and the development of simple analytical approaches appropriate to the quality of data that would normally be available in practice.

The study has shown that in the study catchment there is a potential opportunity to reduce the number of gullies and hence reduce the maintenance requirement by as much as 50%. However, there are potential cost implications resulting from the need to make gulley gratings perform more effectively.

Furthermore the study has demonstrated the potential of using relatively simple approaches to assessment of the interactions between surface and sub surface drainage systems to maximise the utilisation of capacity in both systems to reduce the frequency of surface water flooding.

However, the study has highlighted the uncertainties involved in the collection of data for the assessment of gulley performance on site. This means that there is a need to collect more data to improve the understanding of the uncertainties and the best way to manage them.

The next steps to be taken are as follows:

- To continue with the data collection, including the reinstallation of a rain gauge close to the test site in Thornton Road
- To explore the implementation of the findings of the report by applying them at locations where there is an identified need to solve problems. This is likely to involve the participation of the teams responsible for gulley maintenance and those teams and organisations responsible for the sub surface drainage systems.
- To explore the application of the findings of the report to the design of new developments to help improve the effectiveness of gullies and other inlets and at the same time to reduce the burden of maintenance.

This report should be updated as these steps are undertaken.

Acknowledgements

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Contents

1 Introduction

Gullies are the common, almost ubiquitous form of connection between surface and sub surface drainage systems. However, in many countries they are treated as the Cinderella of the water sector. Their performance is not well understood and they are seldom accounted for in the analysis of the performance of drainage systems. It is just assumed that they will work, whereas they frequently block, either partially or completely.

In the United Kingdom, Phase 1 of the Flood Risk Management Research Consortium[1] set out to improve the representation of flow regimes through integrated 1D – 1D and 2D – 1D surface – sub surface modelling. The results of this research were demonstrated in collaboration with United Kingdom Water Industry Research Ltd[2]. As a result of this work, the need to improve understanding of the performance of gullies and their individual components was identified, and in Phase 2 of the Flood Risk Management Research Consortium[3] a combination of laboratory testing and 3D computational fluid dynamic software was used to develop coefficients for sewer models.

However, although integrated surface – sub surface modelling is now relatively well advanced, it is by no means a common approach for the analysis and assessment of flood risk or for the development of solutions. The reasons for this are as follows:

- Models of sub surface drainage systems are normally confined to combined sewer systems operated by sewerage undertakers. Surface water drainage systems are seldom modelled
- Many models of combined sewer systems are old and in many cases lack sufficient detail to simulate local flooding.
- 2D surface models are only as good as the resolution of the data that drive them. There are potentially many sources of error that need to be resolved, which is not practicable at drainage area scale. Modelling and data needs to focus on local problems.
- It is not always practicable for the managers of drainage systems and urban surfaces to work hand in glove which is a requirement of integrated modelling

The combination of the above leads to a need for other, more cost effective modelling approaches which align the modelling of surface pathways with that of sub surface drainage systems rather than integrating the two. This will enable the prioritisation of locations at greatest risk of flooding whilst reducing the required resources and the overall cost.

This project report describes the work carried out through the SKINT project to demonstrate that this can be done, how to do it and how to identify the further work needed to provide simple effective tools to do the job.

1.1 Project Aim;

The aim of the study is to develop a better understanding of the performance of gullies and the interaction between surface and sub surface drainage to enable the design of more effective gulley inlets that can be more efficiently maintained whilst reducing flood risk.

1.2 Project Objectives

- To determine a standard value for the hydraulic capacity of a gulley subject to proper maintenance by laboratory testing.
- To confirm the value determined in the laboratory by means of field testing.
- To demonstrate the use of this value through a case based desk top study.

[1] http://www.floodrisk.org.uk/images/stories/Phase1/UR4%20signed%20off.pdf
[2] http://www.ukwir.org/web/ukwirlibrary/93202
[3] http://web.sbe.hw.ac.uk/frmrc/downloads/fact_sheets/FRMRC2%20Fact%20Sheet%203-3%20v04%20newformat.pdf

2 Rationale

The landscaping of the hard and soft surfaces of any development is an essential part of the drainage of urban areas. This is because once the capacity of any formal drainage system (e.g. sewers or SuDS) is exceeded, and it is inevitable that it will be exceeded, the water will flow over the land surface. Unless it is managed effectively there is every chance that it will go somewhere where it is not wanted. This means that those involved in the design and maintenance of those surfaces, which comprise buildings, green space, highways, pathways and driveways, are key players in the drainage of urban areas.

Formal urban drainage systems have an important role to play in the effective management of flows. However, that role is limited by the capacity of the system. Therefore they should be designed and constructed to ensure that when capacity is exceeded, the flow will go where it is needed, or to where it causes least damage.

In order for landscapes and formal site drainage systems to work together effectively, they must be properly linked through well designed and maintained inlets that are large enough to accommodate the flow and that will not become blocked.

When drainage systems are designed, it is assumed that the areas that they drain are divided into discrete drainage compartments, whether they drain through inlets into sewers, drains or SuDS features. However, if the assumed compartments are not constructed as intended, or the inlets do not operate effectively, the flow will move from one compartment to another, potentially overloading the second compartment, its inlet and the drainage system to which it is connected. Eventually, given sufficient rainfall, this can result in flooding. Many of our urban areas, which are mostly drained by a combination of gullies and urban drainage systems, are vulnerable to this form of flooding.

In other cases, urban drainage systems can become overloaded through lack of capacity or where the land surface locally dips below the water level in the sewer. Where this happens flow can be ejected from the system at gullies, causing flooding and in the case of combined sewers, pollution.

Because there are so many gullies in our urban areas, maintenance is infrequent and this leads to blockage which exacerbates the situation and means that the assumptions of unfettered connectivity between urban surfaces and drainage systems are frequently incorrect. This can increase uncertainties associated with modelling to the point where modelling predictions start to become meaningless.

2.1 The impact of urban and near urban green space on surface water flooding

Urban drainage systems have traditionally been designed to convey intense rainfall from paved areas, buildings and gardens. Large areas of urban and near urban green space have generally been ignored in the design of urban drainage systems. However, this may be imprudent as the process of urbanisation often separates green space from natural drainage systems and in some cases destroys natural drainage. For the majority of rainfall events this has no adverse effect, but where natural conveyance systems are lost, long periods of moderate rainfall can saturate the ground and result in surface runoff flowing by gravity into developed urban areas. The local inlet capacity of urban drainage systems can therefore be exceeded, not because of rainfall intensity but by an effective increase in impermeable area. This can result in surface water flooding. Studies of depth - duration - frequency curves, backed up by anecdotal evidence, suggest that this type of flooding is several times more frequent than flooding from intense rainfall (Appendix 1). However, during moderate rather than intense rainfall, urban drainage systems tend to have spare capacity and as runoff from green space runs along highways, water will enter local drainage systems through gulley inlets. Eventually all the flow will enter the drainage systems and the downstream extent of the excess surface flow causing flooding is reached. Therefore as well as designing urban landscapes to carry flows in excess of drainage system capacity during periods of intense rainfall, it is also worthwhile to consider the contributions that urban drainage systems can make to the management of runoff from saturated green space during periods of prolonged heavy rainfall.

2.2 Use of surface and sub surface modelling tools

There are many proprietary surface and sub surface modelling tools that are available, some of which are evaluated by Neelz and Pender[4]. The aim here is not to suggest that one modelling tool is better than another, but to provide examples of modelling tools and their output. The tools used in this study are commonly used in the UK, and are InfoWorks CS by Innovyze[5], and ArcGIS by ESRI[6].

The overall context in which these may be used is illustrated by the conceptual model in Figure 2.1, which was first presented by Blanksby et al[7].

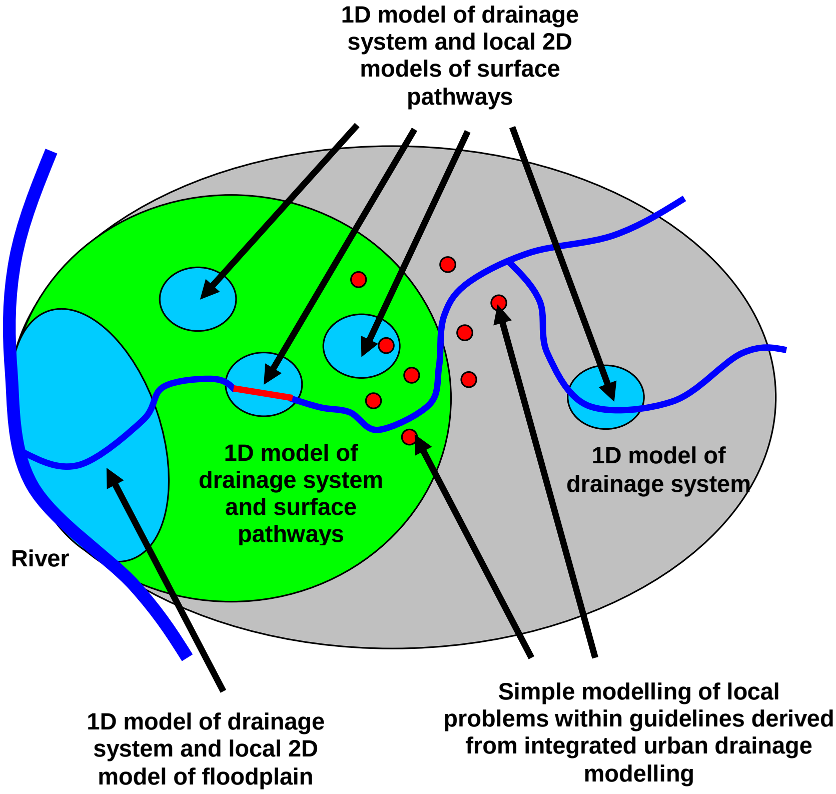

Figure 2.1: Conceptualisation of the context of integrated surface – sub surface modelling

Within this context, the work described by this report is relevant to locations represented by the small red circles in Figure 2.1.

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[4] Neelz S., and Pender G., "*Desktop review of 2D hydraulic modelling packages*", Science report, SC080035, Environment Agency, July 2009, ISBN 978-1-84911-079-2, http://publications.environment-agency.gov.uk/PDF/SCHO0709BQSE-E-E.pdf

[5] http://www.innovyze.com/products/infoworks_cs/

[6] http://www.esri.com/software/arcgis

[7] Blanksby J., Saul A., Ashley R., Djordjević S., Chen A., Leandro J., Savić D., Boonya-aroonnet S., Maksimović C. and Prodanović D., *Integrated urban drainage: setting the context for integrated urban drainage modelling in the United Kingdom"*, Aquaterra, Amsterdam, 2007.

3 Methodology

The methodology combines the results of full scale laboratory testing at the University of Sheffield (UoS), with field monitoring of gullies in the Thornton area of City of Bradford Metropolitan District Council (CBMDC) in order to improve understanding of gulley performance. This is linked to a case study-based desk to study demonstrating approaches to gulley optimisation using ArcGIS with a 1m horizontal resolution Digital Elevation Model (DEM) and 1D modelling of the local sub surface drainage system.

3.1 Laboratory modelling

The aim of the modelling was to provide information on the performance of the different elements of a typical UK gulley pot, under a range of flow conditions. The test gulley had a 150mm pipe connecting it to a drainage system and had a trap which could be bypassed by the removal of the plug from the rodding eye allowing cleaning of the connection. Two different grate designs were tested. Full details of the test rig and the test programme can be found in Appendix 2.

3.2 Selection of monitoring and case study site

Field monitoring was carried out at Thornton Road in Bradford. The 400 metre long stretch of road has a normal camber and wide grass verges which allow for monitoring options and the potential to test alternative technologies with minimal disturbance of the road surface. The site, in common with much of the area, has readily available data including a nearby rain gauge along with regular gulley spacing which helped with the monitoring programme. In addition to this, a combined sewer overflow within the test site lowers the depth of water within the sewer system so that the performance of the gullies is not affected by the flow conditions within the sewers except under extremely rare circumstances (less frequent than an annual probability of 1%).

Gullies along this stretch of road are spaced at 25 metre centres with the exception of one location where the spacing is 50 metres. The contributing area for each gulley was determined using the DEM and checked from a topographical survey. Full details of the selection of the test site can be found in Appendix 3.

3.3 The monitoring programme

The approach to the monitoring was to install depth monitors in two gullies which were directly opposite each other on either side of the road. Once the monitors were calibrated, the intention was then to sequentially block the upstream gullies on one side of the highway. Flow entering the gullies was determined using the measured rainfall intensities and the contributing area, and this was then plotted against the depth of flow measured within the gullies in order to determine the gulley capacity. Full details of the monitoring programme can be found in Appendix 4.

3.4 Desk top study

The desk top study sets out to demonstrate the application of procedures that will:

1. Use pre existing 1D drainage system model outputs or of cost effective 1D drainage system modelling where existing outputs are not available, to identify those sections of the system which are susceptible to flooding and those parts of the system which have spare capacity

2. Enhance and use readily available digital elevation models to identify pathways carrying flows from developed urban surfaces and urban green space to drainage systems, and to identify routes that can accept surface water in order to avoid overloading sewers.

3. Use GIS based records of gullies and data on the capacity of connections to facilitate the identification of the potential for reducing the number of gullies.
4. Identify appropriate gulley grating enhancements for remaining gullies to ensure effective operation
5. Help identify needs for full or partial disconnection through surface flow management and flow control at connections of remaining gullies.

Details of each procedure are given in Appendix 5

4 Results

4.1 Laboratory testing

The laboratory testing indicated that the gulley outlet dictates flow capacity, with the grate dimensions having little effect on internal gulley performance. Removal of the rodding eye cover significantly increases capacity at the cost of beneficial functions of silt and oil trapping. However, if a grate becomes sufficiently blocked it will start to affect performance.

It is clear from simple testing that the performance of a given gulley design is primarily based on the outlet capacity. Due to its gradient the sewer connection itself has significantly higher hydraulic capacity than the gulley outlet is able to present, even in its un-trapped condition (rodding eye open). This leaves the gulley outlet as the determining design parameter and where well maintained in terms of sediment and grate blockage, there is little to diminish capacity from other parts of the system. Poor maintenance resulting in blockage from debris, leaves, etc. is likely to have significant impact on the flows entering the gulley and potentially result in a significant flow past on the roadside channel. Further testing will reveal the significance of grate blockage on both the internal performance and the impacts on the urban surface caused by this diminished drainage capacity.

Figure 4.1.1 shows the depth of water in the gulley pot for the range of flow rates tested. This “Terminal Flow” test presents all surface flow to the gulley grate as the only exit from the system; flow approaches the grate from all directions and in so doing provides a stress test for increasing surface flood conditions. Clearly demonstrated here is the dominant effect of the water trap restricting maximum flow without surcharge to 20 l/sec as compare to the open Roding eye (un-trapped) state which allows flows beyond the capability of the testing tanks at 45 l/sec and beyond.

Thus the capacity of a normal modern gulley with a 150mm diameter connection can be taken as 20 l/sec and by inference a 100mm connection will be in the order of 9 l/sec

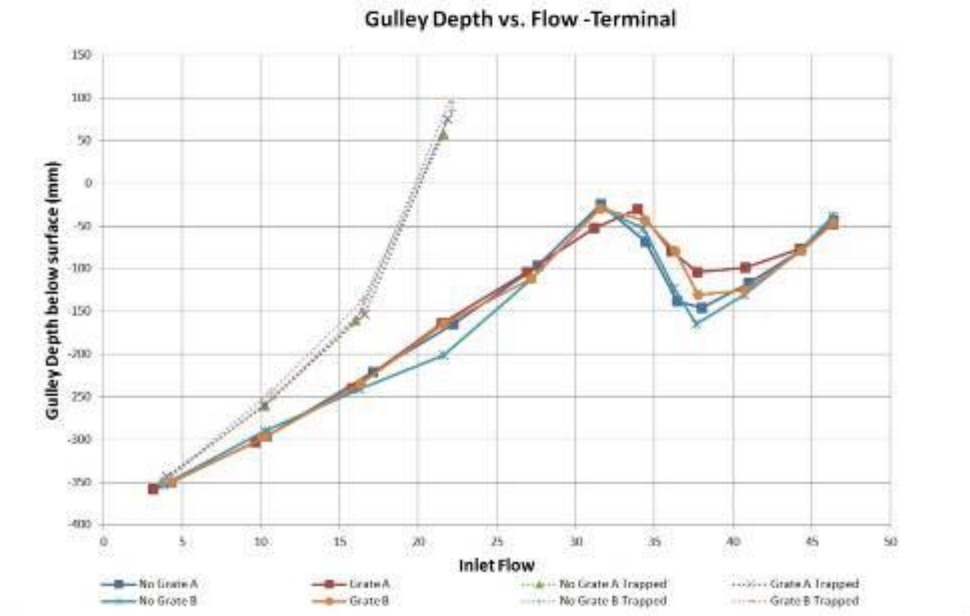


Figure 4.1.1: Depth discharge relationship for the gulley with different grates and with and without the rodding eye plug in place

4.2 Field monitoring

Although further monitoring is required, including re-installation of a rain gauge close to the site, the results of the field testing indicate that the depth discharge relationship for the gulley connection determined in the laboratory is appropriate for use in the rationalisation of gulley numbers.

4.3 Desk top study

4.3.1 1D drainage system model outputs

The performance of the drainage system is illustrated in Figures A4.3.1 and A4.3.2. Performance is presented in four bands coloured blue, green, yellow and red.

Blue indicates those lengths which have a low likelihood of their capacity being exceeded

Green indicates the lengths that are likely to meet the required level of service.

Yellow indicates those lengths which are more likely to underperform

Red indicates that there may be a problem and that a greater degree of understanding is required.

Figure A4.3.1 shows that the sewers upstream and around the combined sewer overflow in Thornton Road are unlikely to cause flooding and that they are likely to have spare capacity to accept flows, thereby helping manage runoff from urban green space if this were needed.

Downstream in Leaventhorpe Lane, Figure A4.3.2 indicates that the situation is different. The yellow and especially the red indicate that there may be a real problem and that there is a need to gain a greater understanding of the situation at these locations.

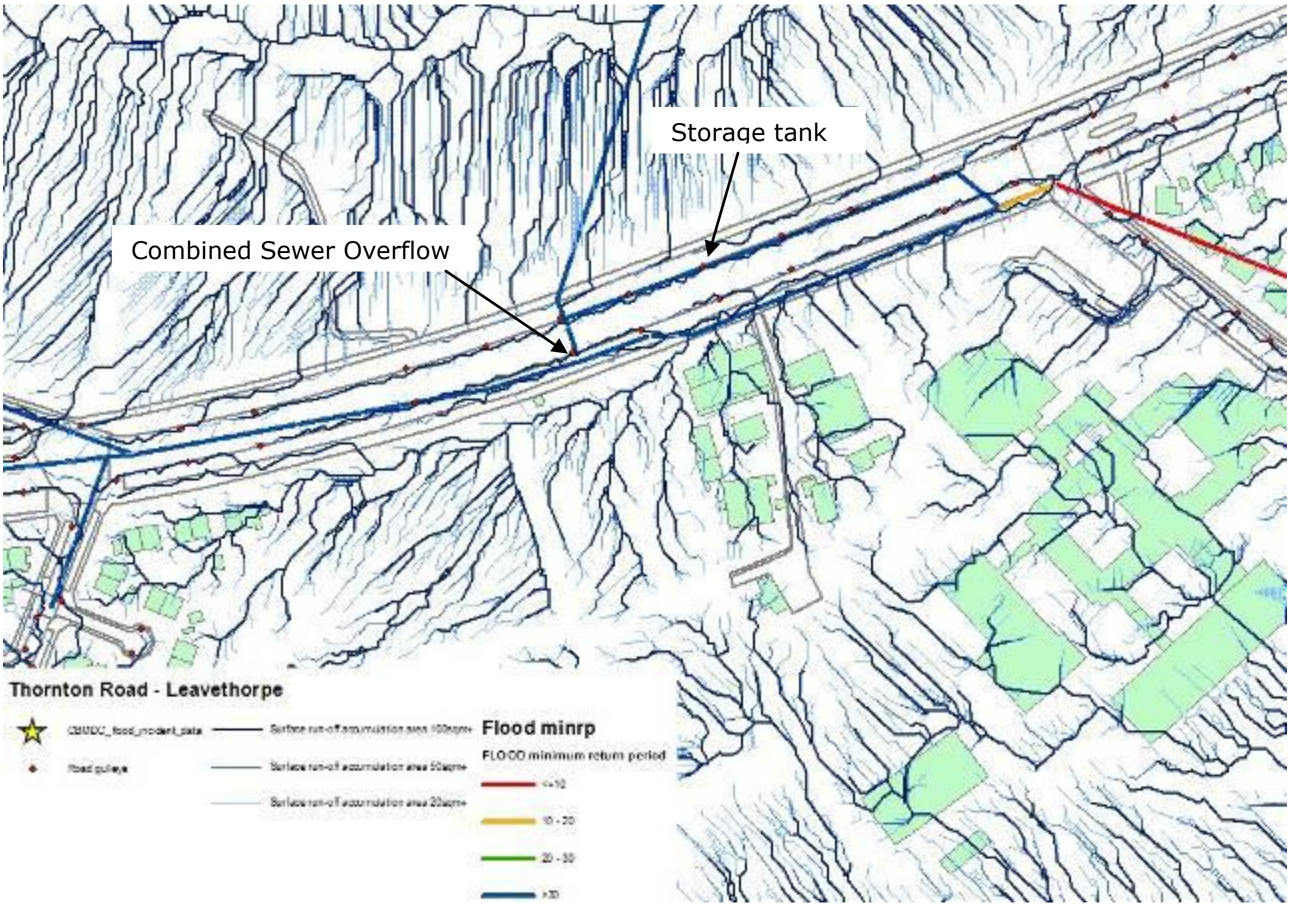


Figure A4.3.1: Sewer capacity and surface flow paths on Thornton Road

Although the sewer in Leaventhorpe Lane is relatively small with a diameter of only 300mm, the flows that it carries are limited, comprising the discharge passing through the combined sewer overflow and some discharge from the school to the southwest of the junction of Thornton Road and Leaventhorpe Lane.

Analysis of the sewer system model outputs show that the overloading is only minor and lies within the expected levels of uncertainty within the model.

Flood incident records show only two incidents within this neighbourhood, one property being affected by a groundwater problem and the second property from an undefined problem. This suggests that there is not a major flood problem and so the situation should be flagged but kept under review rather than taking immediate action.

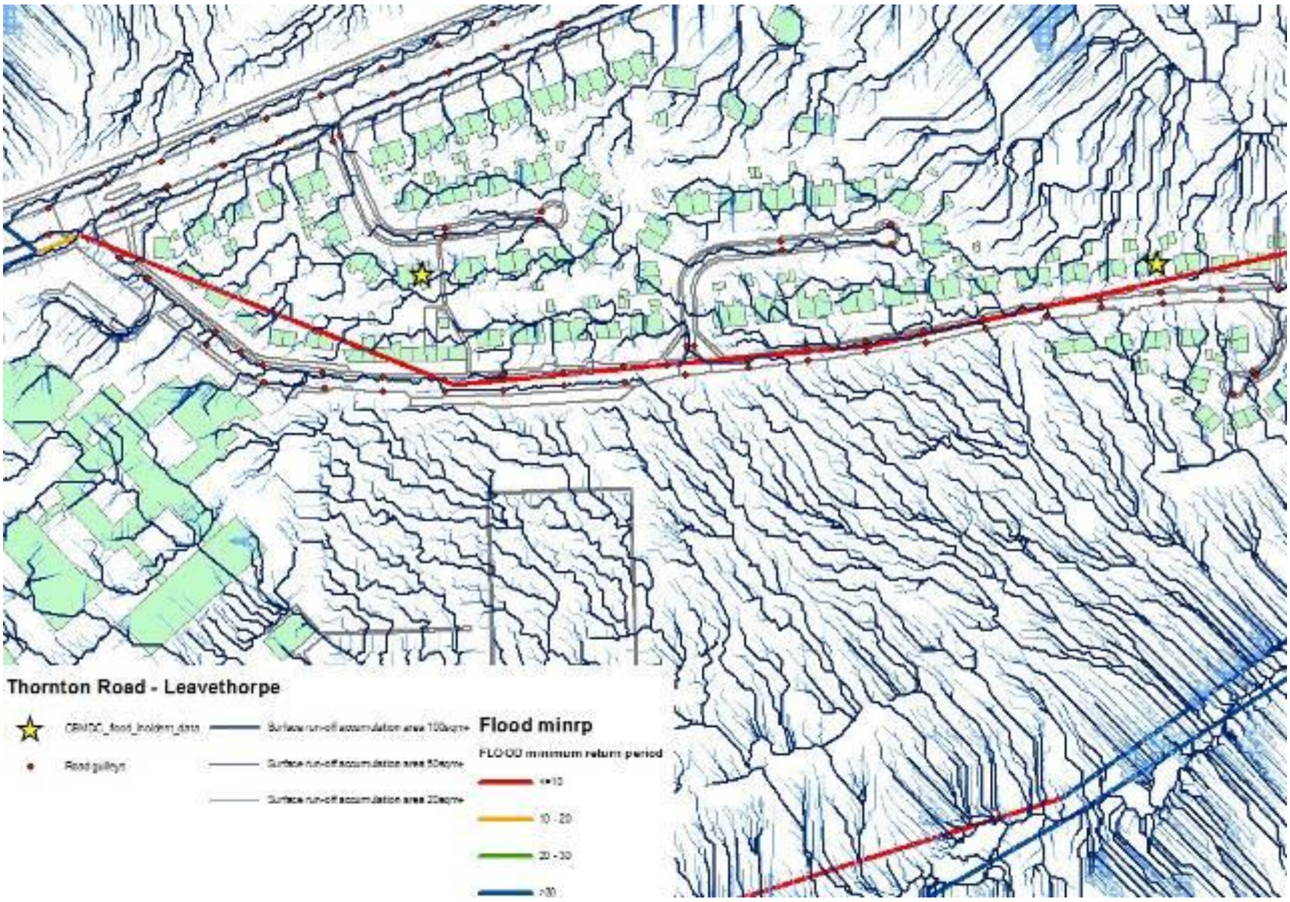


Figure 4.3.2: Sewer capacity and surface flow paths on Leaventhorpe Lane

4.3.2 Flow pathways and flow disconnection

Figures A4.3.1 and A4.3.2 show that over most of the study area the flow pathways fall away from the highways, although there are areas of green space which drain into Thornton Road from the south and also into a small length of Leaventhorpe Lane from the west. It should be noted that these flows discharge onto the highway downstream of the combined sewer overflow and where they to enter the sewer through the gulley system, they could contribute to the overloading of the sewer in Leaventhorpe Lane. Therefore if there is a need to take action in the future, then flow disconnection should be considered.

4.3.3 Gulley optimisation

There are a large number of gullies on Thornton Road and on Leaventhorpe Lane. The results of the laboratory and field study indicate that there is a good case from the perspective of performance to reduce the number of gullies by as much as 50%. However, at the same time the design of gulley grates needs to be improved to ensure effective operation through the reduction of blockage and bypassing.

5 Conclusions and recommendations

The study has shown that in the study catchment there is a potential opportunity to reduce the number of gullies and hence reduce the maintenance requirement by as much as 50%. However, there are potential cost implications resulting from the need to make gulley gratings perform more effectively.

Furthermore the study has demonstrated the potential of using relatively simple approaches to assessment of the interactions between surface and sub surface drainage systems to maximise the utilisation of capacity in both systems to reduce the frequency of surface water flooding.

However, the study has highlighted the uncertainties involved in the collection of data for the assessment of gulley performance on site. This means that there is a need to collect more data to improve the understanding of the uncertainties and the best way to manage them.

The next steps to be taken are as follows:

- To continue with the data collection, including the reinstallation of a rain gauge close to the test site in Thornton Road.
- To explore the implementation of the findings of the report by applying them at locations where there is an identified need to solve problems. This is likely to involve the participation of the teams responsible for gulley maintenance and those teams and organisations responsible for the sub surface drainage systems.
- To explore the application of the findings of the report to the design of new developments to help improve the effectiveness of gullies and other inlets and at the same time to reduce the burden of maintenance.

This report should be updated as these steps are undertaken.

Appendices

Appendix 1: The impact of urban and near urban green space runoff on urban surface water flooding

Appendix 2: Laboratory testing

Appendix 3: Selection of monitoring and case study site

Appendix 4: The monitoring programme

Appendix 5: Desk top study

Appendix 1: The impact of urban and near urban green space runoff on urban surface water flooding

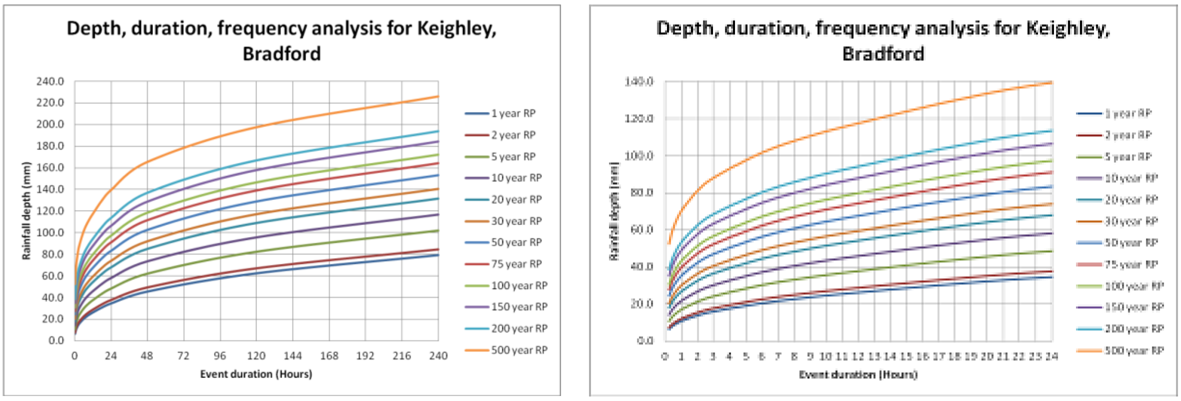
It has been common practice for large areas of urban green space and for near urban green space to be omitted from the analysis by urban drainage engineers[8]. To some extent this is understandable as the core of many of our cities, where urban drainage practice was first established, are largely impermeable and urban drainage systems, both surface water and combined, tend to have relatively short critical durations when compared with natural drainage systems. The latter means that the design rainfall intensities for urban drainage systems are relatively high when compared with those for more natural systems. Combining the design frequencies suggested in Tables 2 and 3 of EN 752[9] for simple and complex drainage systems and based on a typical intensity - duration - frequency relationship analysis such as in Table 1 below, the design intensity for urban drainage systems would be expected to be in excess of 30 mm/hour for a 15 minute duration, one in two year return period simple drainage system, and may be as much as 50mm/hour for a 60 minute one in thirty year return period for a complex drainage system. In many cases the critical duration would be less than these so the design intensities would be greater.

Runoff from permeable surfaces does not start to happen until the ground becomes saturated and this takes time. This means that depending on the ground conditions, it may need to rain for several hours for runoff from urban and near urban green space to occur. Whereas urban drainage systems are designed to take the rapid runoff from impermeable surfaces during convectional rainfall, runoff from green space tends to occur as a result of frontal rainfall associated with cyclonic weather systems. In these cases, the rainfall is less than the design intensity and any flooding that occurs is caused by an increase in effective contributing area from the permeable systems causing overloading of the drainage system, or by the runoff from the permeable surfaces being unable to enter the urban drainage systems because of constrictions at the inlets. Figure A1.1 shows the rainfall depth, duration and frequency for the same location as in Table A1.1. In Figure A1.1a, the durations range between 15 minutes and 10 days, and in Figure 1.1b the durations up to 1 day are shown in more detail.

[8] WaPUG 2009, "*Catchment Data For The Wallingford Procedure Percentage Runoff Model*", WaPUG User Note 21(Rev 2), WaPUG Committee, 2009, http://www.ciwem.org/media/144861/WAPUG_User_Note_21.pdf
[9] EN 752, "Drain and Sewer Systems outside buildings", European Committee for Standardisation, 2008

Table A1.1: Rainfall intensity (mm/hour) for a matrix of durations and return periods at Keighley, Bradford

Duration	Return Period											
Hours	1	2	5	10	20	30	50	75	100	150	200	500
0.25	26.4	29.8	43.5	56.8	71.6	81.5	97.2	111.0	122.4	140.0	154.3	209.4
0.5	16.9	19.0	27.2	35.1	43.7	49.4	58.4	66.2	72.7	82.6	90.5	121.0
1	10.8	12.1	17.0	21.6	26.6	29.9	35.0	39.5	43.1	48.7	53.1	69.9
2	6.9	7.7	10.6	13.3	16.2	18.1	21.0	23.6	25.6	28.7	31.2	40.4
3	5.4	5.9	8.1	10.1	12.2	13.5	15.6	17.4	18.9	21.1	22.8	29.3
6	3.4	3.8	5.0	6.2	7.4	8.2	9.4	10.4	11.2	12.4	13.4	16.9
9	2.6	2.9	3.8	4.7	5.5	6.1	7.0	7.7	8.3	9.1	9.8	12.3
12	2.2	2.4	3.2	3.8	4.5	5.0	5.6	6.2	6.7	7.3	7.9	9.8
18	1.7	1.9	2.4	2.9	3.4	3.8	4.2	4.7	5.0	5.5	5.8	7.2
24	1.4	1.6	2.0	2.4	2.8	3.1	3.5	3.8	4.1	4.4	4.7	5.8
48	1.0	1.0	1.3	1.5	1.8	1.9	2.1	2.3	2.5	2.7	2.9	3.4
120	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.2	1.3	1.4	1.6
240	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9



a) DDF for up to 10 days **b) DDF for up to 1 day**

Figure A1.1: Rainfall depth (mm) for a range of durations and return periods in Keighley, Bradford

Figure A1.1 shows that at the example location, a rainfall depth of 80mm with a duration of two hours has a return period of once in 500 years whereas the same depth of rainfall falling over a 1 day period has a return period of about once in 50 years. Additionally, this depth of rain falling over 10 days has a return period of once in 1 year. Thus there is a much greater probability of having a period of moderate to heavy rainfall over a period of many hours than it is to have a very intense but short duration rainfall.

In Table A1.1 80mm of rainfall falling over two hours has an average intensity of 40mm per hour which is around the design intensity for an urban drainage system. However, the same depth of rainfall falling over four hours gives an average intensity of 20mm per hour which is well within the capacity of an urban drainage system. However, this type of event is often the cause of runoff from urban and near urban green space and causes local flooding.

Tables A1.2 and A1.3 illustrate the depth discharge frequency relationships for two UK flood events, one in Glasgow and one in Hull, the latter occurring on 25th June 2007 which achieved an international reputation. In these tables, the typical design range for urban drainage systems is bounded by the red rectangular border and the return period of the maximum rainfall depth for each duration is highlighted in yellow. The maximum rainfall depth is presented at the top of the table immediately above the maximum rainfall intensity.

In Table A1.2 (Glasgow), it can be seen that the maximum rainfall intensity was at or above the design intensity for the urban drainage system and this was evidenced by fountains of water gushing from the sewer chambers as shown in Figure A1.2.

Table A1.2: Rainfall depths (mm) for a matrix of durations and return periods at Glasgow on 30th July 2002

Max Depth (mm)		16.92	25.91	36.78	52.78	62.76	68.79	82.59	95.00	95.00	95.00	95.00	95.00	95.00
Max Intensity (mm/hr)		67.67	51.81	36.78	26.39	20.92	11.47	9.18	7.92	5.28	3.96	1.98	0.79	0.40
Duration	Minutes	15	30.00	60.00	120.00	180.00	360.00	540	720	1080	1440	2880	7200	14400
	Hours	0.25	0.50	1.00	2.00	3.00	6.00	9	12	18	24	48	120	240
	Days	0.010	0.02	0.04	0.08	0.13	0.25	0.375	0.500	0.750	1.000	2.000	5.000	10.000
Return Perion	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1	6.85	9.19	12.34	16.57	19.69	26.43	31.40	35.48	41.41	46.21	60.18	84.38	108.98
	2	8.58	11.40	15.13	20.00	23.72	31.49	37.17	41.82	48.49	53.85	69.36	95.85	122.43
	5	11.57	15.14	19.81	25.92	30.34	39.70	46.46	51.95	59.72	65.94	83.69	113.45	142.80
	10	14.50	18.77	24.29	31.43	36.55	47.30	55.00	61.22	69.93	76.85	96.47	128.87	160.43
	20	18.16	23.26	29.78	38.11	44.03	56.36	65.12	72.14	81.87	89.56	111.19	146.39	180.24
	30	20.74	26.38	33.55	42.66	49.10	62.44	71.87	79.41	89.78	97.96	120.83	157.72	192.94
	50	24.50	30.90	38.98	49.17	56.33	71.05	81.39	89.63	100.85	109.66	134.17	173.25	210.22
	75	27.96	35.04	43.92	55.04	62.81	78.72	89.83	98.66	110.60	119.93	145.79	186.66	225.03
	100	30.71	38.31	47.79	59.63	67.86	84.66	96.35	105.62	118.08	127.80	154.65	196.80	236.17
	150	35.05	43.44	53.85	66.74	75.67	93.80	106.35	116.26	129.49	139.78	168.05	212.04	252.82
	200	38.49	47.49	58.60	72.30	81.76	100.87	114.07	124.46	138.25	148.95	178.26	223.56	265.33
	500	51.89	63.09	76.71	93.28	104.58	127.17	142.57	154.62	170.29	182.37	215.08	264.59	309.47

Figure A1.2: Sewer flooding in Glasgow, July 2002

In Table A1.3, it can be seen that the maximum rainfall intensity in Hull was significantly less than the design intensity for urban drainage systems, even though the longer duration return periods were far greater than in Glasgow. Therefore, providing that the sewer system was adequately designed and free of blockages, it should have coped with the flows for which it was designed. Therefore it may be deduced that the problems were caused by inadequate management of surface water.

The extremity of the Hull event was widely quoted in the press, but as is often the case the headline figures belied the actual situation. This is described in Table 4.

In Table A1.4, the yellow highlighting shows the peak return period for each duration as the event progressed. The headline figures did not arise until the early evening, whereas flooding started to occur between 9:00 and 10:00 am and most of the flooding had been initiated by 12:00 noon[10]. This means that the flood frequency is much greater than suggested by the headline figures and the return periods at the onset of flooding support the case that it was long duration moderate rainfall rather than short duration high intensity rainfall that was at the root of the flooding. It can be concluded from this that although runoff from urban and near urban green space is by no means the sole cause of urban flooding, it can be a significant contributor and deserves consideration in its own right.

Table A1.3: Rainfall depth (mm) for a matrix of durations and return periods at Hull on 25th June 2007

Max depth (mm)	14.45	24.85	38.25	66.20	92.15	102.61	113.01	118.81
Max intensity (mm/hr)	14.45	12.43	12.75	11.03	10.24	8.55	6.28	4.95
Duration (Hours)								
Event probability (1in in x years)	**1**	**2**	**3**	**6**	**9**	**12**	**18**	**24**
2	12.36	15.67	17.99	22.8	26.19	28.89	32.62	35.56
5	17.41	21.66	24.62	30.64	34.82	38.13	42.6	46.09
10	21.84	26.85	30.3	37.26	42.05	45.82	50.84	54.73
20	27.14	32.99	36.98	44.96	50.4	54.65	60.23	64.53
30	30.76	37.14	41.47	50.08	55.93	60.48	66.4	70.94
50	35.96	43.07	47.86	57.32	63.7	68.66	75.01	79.87
75	40.68	48.41	53.59	63.77	70.6	75.88	82.59	87.7
100	44.4	52.59	58.06	68.77	75.93	81.46	88.41	93.71
150	50.21	59.08	64.99	76.48	84.11	89.99	97.31	102.86
200	54.78	64.17	70.39	82.45	90.44	96.58	104.15	109.89
500	72.28	83.43	90.74	104.75	113.92	120.92	129.29	135.58

[10] Coulthard T., Frostick L., Hardcastle H., Jones K., Rogers D. and Scott M., "*The June 2007 floods in Hull*" Interim Report by the Independent Review Body 24th August 2007.

Table A1.4: Peak return period for rainfall at Hull, for a range of durations

Time	1 hr	2 hr	3 hr	6 hr	9 hr	12 hr	18 hr	24 hr
25/06/2007 09:00	<2	2 - 5	2 - 5	2 - 5	2 - 5	<2	2 - 5	5 - 10
25/06/2007 10:00	2 - 5	5 - 10	10 - 20	10 - 20	5 - 10	5 - 10	10 - 20	20 - 30
25/06/2007 11:00	<2	5 - 10	10 - 20	20 - 30	20 - 30	10 - 20	20 - 30	30 - 50
25/06/2007 12:00	2 -5	5 - 10	20 - 30	20 - 30	30 - 50	30 - 50	30 - 50	75 - 100
25/06/2007 13:00	<2	5 - 10	10 - 20	75 - 100	75 - 100	50 - 75	50 - 75	100 - 150
25/06/2007 14:00	<2	2 - 5	10 - 20	75 - 100	100 - 150	100 - 150	75 - 100	100 - 150
25/06/2007 15:00	<2	2 - 5	5 - 10	75 - 100	150 - 200	150 - 200	100 - 150	150 - 200
25/06/2007 16:00	<2	2 - 5	5 - 10	50 - 75	200 - 500	200 - 500	150 - 200	200 - 500
25/06/2007 17:00	<2	2 - 5	5 - 10	30 - 50	200 - 500	200 - 500	150 - 200	200 - 500
25/06/2007 18:00	<2	<2	<2	10 - 20	100 - 150	200 - 500	150 - 200	200 - 500
25/06/2007 19:00	<2	<2	<2	10 - 20	50 - 75	200 - 500	150 - 200	200 - 500
25/06/2007 20:00	<2	<2	<2	5 - 10	30 - 50	200 - 500	200 - 500	200 - 500

Appendix 2: Laboratory testing

The test rig

For the purpose of this study, a full scale laboratory system (Figure A2.1) has been used to mimic and measure the hydraulic interactions between the above and below ground storm drainage flow via gully inlets. The laboratory system consists of a test channel with inlet tanks of equal dimension (92.44m x 0.61m) at each end. The test channel is a rectangular platform (4.27m x 1.83m) the floor of which can be rebuilt inside the test rig to provide various longitudinal gradients to imitate varied road slopes. The flow rates used for this study are between 0 - 50 *l*/s which are provided by gravity from overhead tanks and controlled via software operated inlet control valves. The flow from the overhead tank may be presented either over the surface of the channel from the inlet tank, or as surcharge / reverse sewer flow from the sewer side connection back through the gulley pot and onto the channel surface. This flexibility provides the opportunity to asses and measure performance in both surface water removal and the impacts of surcharged sewer outflow onto the urban surface.

The gully pot outlet is connected to a 150mm diameter pipe (sewer connection) which discharges to laboratory measuring tanks with a similar measuring tank facility for the main channel outlet tank. This design provides for full physical flow measurement to compliment inlet magnetic flow meters in all configurations.

The configuration of inlet flows and outlet control valves allows for a variety of urban flow scenarios at the gulley mouth. An Intermediate (roadside flow past) gully system or Terminal gulley design can be simulated as well as a combination of surface flow and reverse flows from a blocked / surcharged subsurface system.

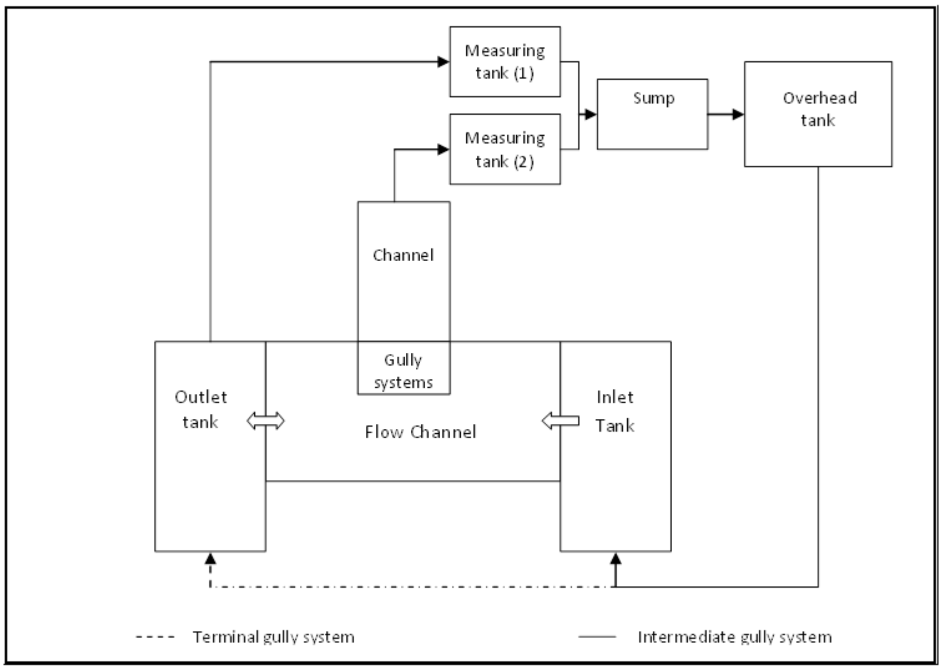


Figure A2.1: Schematic drawing of the laboratory system

Figure A2.2 shows the dimensions of the laboratory rig, and Figure A2.3 provides an overall view of the laboratory system.

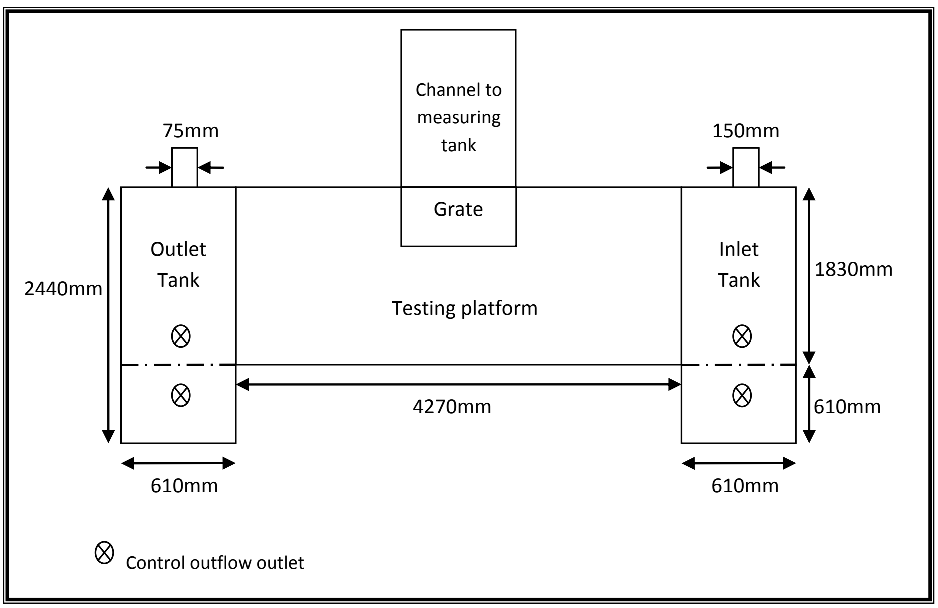


Figure A2.2: Dimensions of the laboratory rig

Figure A2.3: General photograph of the laboratory rig

The test gulley and gulley grates

The testing was performed on a trapped gully (Figure A2.4) with spigot outlet; the more commonly used gully type. Trapped gullies are designed with an outlet that forms a water seal which helps to retain oil and sediments within the gully pot. In service, a rodding eye cover is visible above the water surface which when removed provides access to the sewer connection pipe for cleaning. The gully used is the 375mm diameter, 750mm depth gully described in Table A2.1 and Figure A2.5.

Figure A2.4: Trapped gully with 150mm diameter outlet

Table A2.1: Gully Properties

Internal diameter (mm)	Internal depth (mm)	Outlet (mm)	Inside depth to centre of outlet/ rodding eye (mm)	Outside depth of outlet (mm)	Dimen sion of riser (mm)	Depth of water seal (mm)	Weight (kg)
375	375	150	148	251	85	85	180

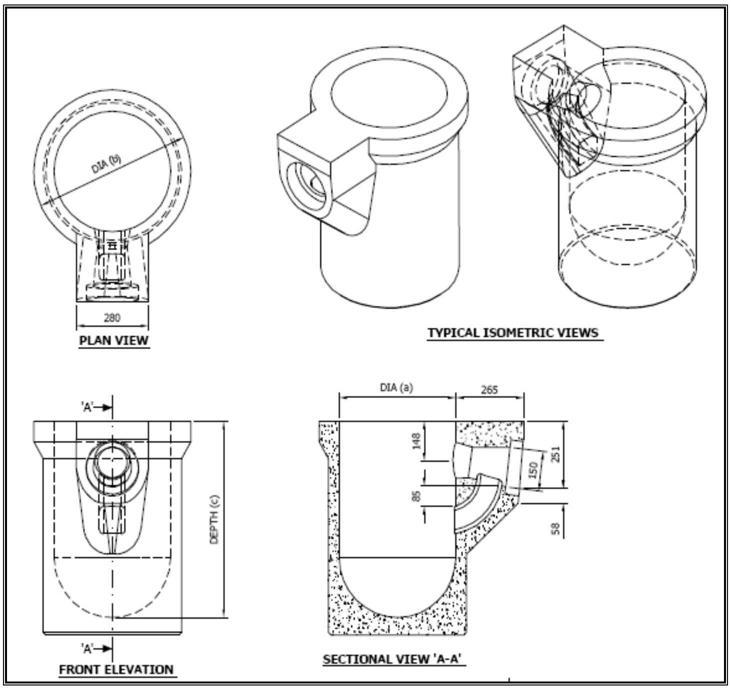


Figure A2.5: Properties of the gully system (Milton Precast)

Gully grates according to HA102 are classified based on their hydraulic capacity and are divided into 5 different types; P, Q, R, S and T, with their hydraulic capacity decreasing respectively. BS EN 124 has also outlined that in order to ensure a reasonable level of hydraulic capacity, the total waterway area of the grate slots should not be less than 30% of the clear opening of the grates. The proposed grates also meet the minimum waterway area of 900 cm^2 commonly used in practice in the UK as highlighted in HA 104/02. The loading class selected is for Group 3 (BS EN 124) – C250 which is suitable for installation in the area of kerbside channels of roads. For the initial series of tests, the grates with clear openings of 400mm x 432mm (HA 102 – R) were used. The grates are described in Figure A2.6 and Table A2.2.

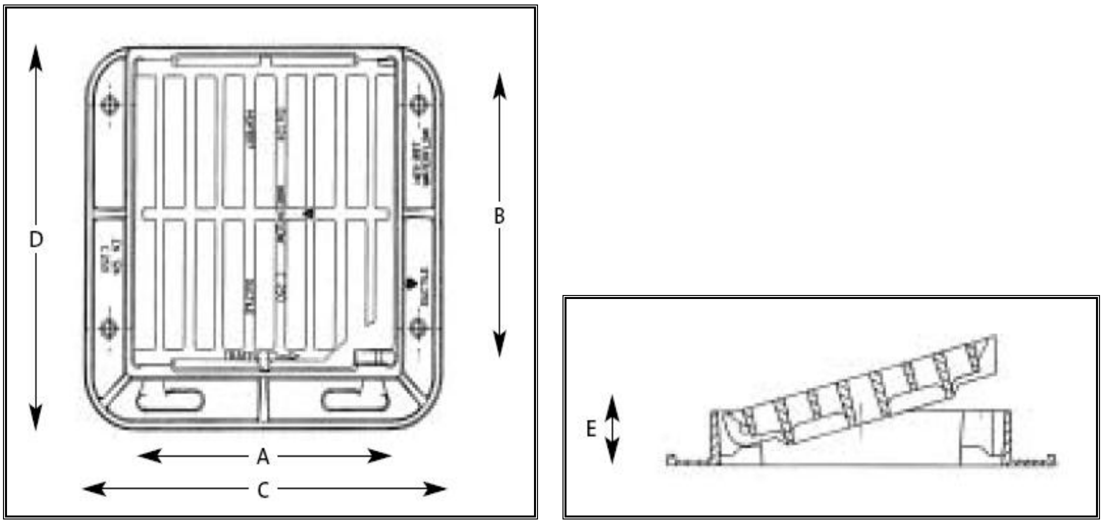


Figure A2.6: Properties of the grates (St. Gobain Pipelines)

Table A2.2: Grate Properties

BS EN 124 loading class	Clear opening A x B (mm)	Over base C x D (mm)	Depth E (mm)	Waterway area (cm^2)	Total mass (kg)	HA 102 reference
C250	325 x 437	475 x 524	75	933	29	S
C250	400 x 432	550 x 530	75	1128	33	R

Photographs of the two grates tested are given in Figures A2.7 and A2.8.

Figure A2.7: Grate with 400mm x 432mm clear opening (HA102 Reference – R)

Figure A2.8: Grate with 325mm x 437mm clear opening (HA102 Reference – S)

Depth measurements

In order to measure the hydraulic depth of the flow, seven pressure transducers were set up on the rig; six on the bed of the Channel and one at the bottom of the gully pot (Figure A2.9). The pressure transducers used in this study is the GEMS 5000 series (0-30mbar) for the bed and GEMS 5000 series (0-150mbar) for the gully pot. Both sensors give an output of between 4-20 mA and uses 9-35V of supply power. These pressure transducers were selected as they have long term stability and high accuracy (±0.2%). Point-gauge measuring equipment was also set up in order to calibrate the pressure transducers. Figure A2.9 shows the position of the pressure transducers on the gully bed. The positions are equally spaced 300mm from the edge of the grates in order to give an average hydraulic depth of the flow going into the gully gratings on all sides.

Figure A2.9: Position of the pressure sensors

Methodology

Preliminary testing for system capacity was carried out under a variety of settings. Configurations for terminal and intermediate type gullies were applied and tested with incremental flow rates until overcapacity of either the gulley or the inlet system were achieved.

For each grate type the following configurations were applied

- Grate in place - Roding eye closed (Trapped)
- Grate in place - Roding eye open
- Grate in removed - Roding eye closed (Trapped)
- Grate in removed - Roding eye open

Each test consisted of a 20 minute run for each setting split into a five minute settling period followed by a 15 minute data capture period. The 15 minute data is then checked for consistency and drift before averages are taken for each data source.

- Bed depth sensors 1-6
- Gulley Depth Sensor
- Inlet Magnetic Flow meter (checked against physical flow measurements x 3)

Results

The testing indicates that the gulley outlet dictates flow capacity with the grate dimensions having little effect on internal gulley performance. Removal of the rodding eye cover significantly increases capacity at the cost of beneficial functions of silt and oil trapping.

It is clear from simple testing that the performance of a given gulley design is primarily based on the outlet capacity. Due to its gradient, the sewer connection itself has significantly higher hydraulic capacity than the gulley outlet is able to present in even its un-trapped condition (rodding eye open). This leaves the gulley outlet as the determining design parameter and where well maintained in terms of sediment and grate blockage there is little to diminish capacity from other parts of the system. Poor maintenance resulting in blockage from debris, leaves etc. is likely to have significant impact on the flows entering the gulley and potentially result in a significant flow past on the roadside channel. Further testing will reveal the significance of grate blockage on both the internal performance and the impacts on the urban surface caused by this diminished drainage capacity.

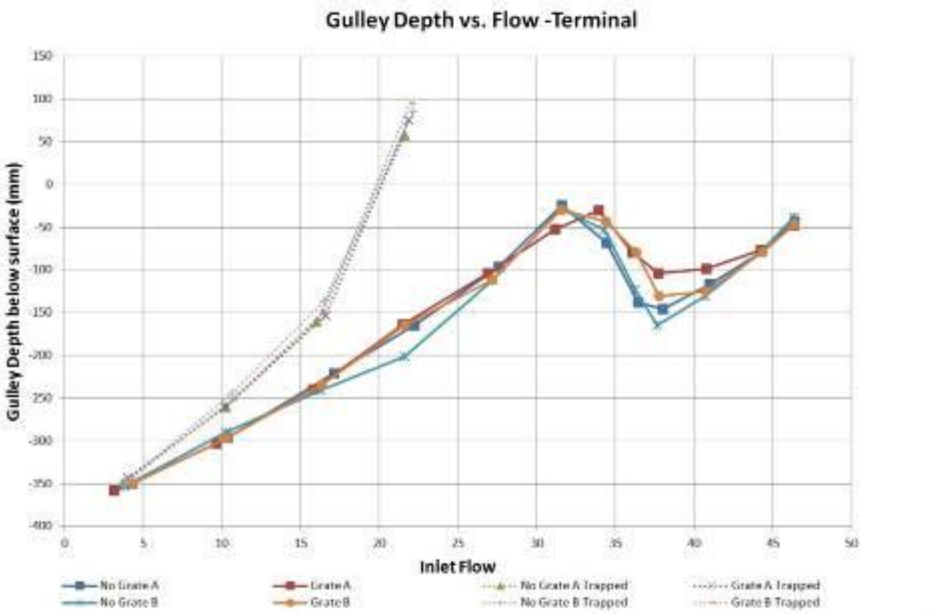


Figure A2.10: Depth discharge relationship for the gulley with different grates and with and without the rodding eye plug in place

Figure A2.10 shows the depth of water in the gulley pot for the range of flow rates tested. This “Terminal Flow” test presents all surface flow to the gulley grate as the only exit from the system; flow approaches the grate from all directions and in so doing provides a stress test for increasing surface flood conditions. Clearly demonstrated here is the dominant effect of the water trap restricting maximum flow without surcharge to 20 l/sec as compare to the open Rodding eye (un-trapped) state which allows flows beyond the capability of the testing tanks at 45/sec and beyond.

Appendix 3: Selection of monitoring and case study site

Introduction

A section of road approximately 400 m long situated in the western suburbs of Bradford was selected for the field monitoring and a case study to demonstrate the application of the methodologies developed. The site was shortlisted due to the availability of data, wide grass verges which allow for monitoring options and the potential to test alternative technologies with minimal disturbance of the road surface. In addition the single carriageway highway is wide with a balanced camber, with the 30 miles per hour speed limit enforced by a speed camera. Figure A3.1 shows the selected stretch of road which runs from the junction at the left (west) of the image to the junction close to the right (east) of the image. The dashed lines represent the edge of the road surface, with the verges and pavements lying beyond these. The gully locations are shown as filled green circles.

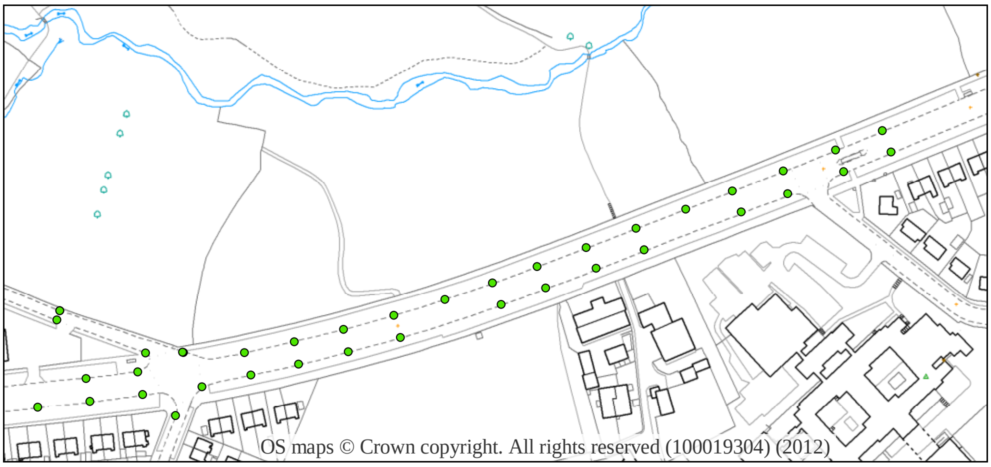


Figure A3.1: Monitoring and case study site, surveyed gully locations shown as green circles

Various types of data were readily available with other data specially collected for the case study site. The data used included a database of road gully locations, LiDAR elevation data, a traditional topographical survey and detailed surveys of the gully grates and pots. The gullies are predominantly uniformly spaced at 25 m intervals on each side of the road, apart from two locations where the spacing is 50 m. A model of the combined sewer system was also available.

LiDAR checks

The LiDAR data available for the study is of 1 m horizontal resolution and ± 0.15 m vertical accuracy. The differences between the LiDAR and surveyed points have been calculated by subtracting the surveyed levels from the LiDAR cell elevations at each of the surveyed points. Figure A3.2 shows these differences as a histogram. It can be seen that the LiDAR DTM is within ± 0.05 m at 48% of the surveyed points, within ± 0.1 m at 84% and within ± 0.15 m at 94% of surveyed points. There is clearly a tendency for the LiDAR to over-estimate the elevations for this area. However, closer analysis of the data (not shown) reveals that the largest variations are at the points on the verge farthest from the road where there are greater variations in the surface elevation. These discrepancies therefore reflect the aggregate nature of the LiDAR elevations over the 1 m^2 area as opposed to the point measurements of the traditional survey. Other discrepancies are located adjacent to the vertical step of the kerb. Points within the road show the smallest differences between the two measurements.

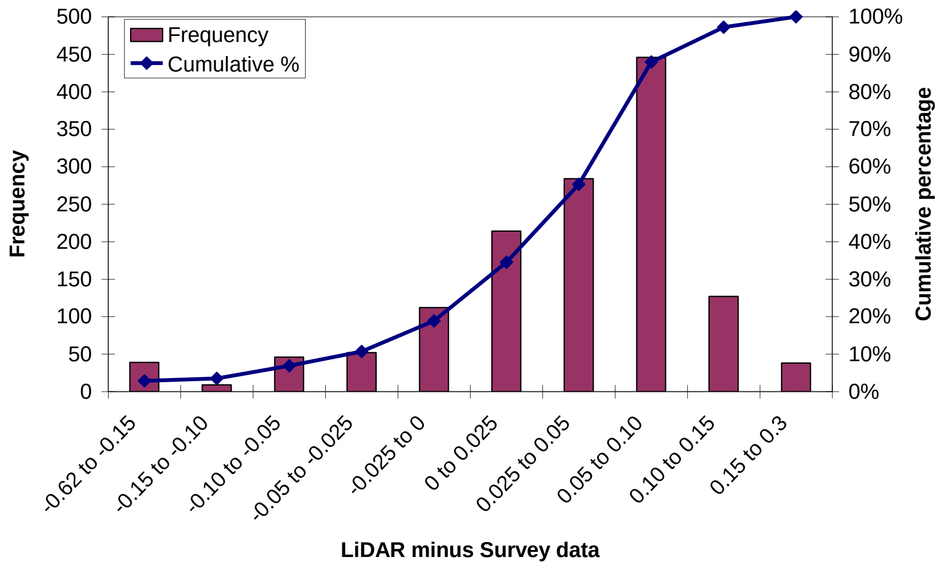


Figure A3.2: Comparison of LiDAR and Survey levels (m)

Assessment of flow paths and watersheds

Having verified acceptable accuracy in the LiDAR data, the Hydrology toolbox within ArcMap was used to determine flow directions and paths. Figure A3.3 shows the results from the Flow Accumulation tool for a part of the case study catchment (a smaller area is used to improve clarity). It can be seen that there are clear flow paths running along the edge of the road, occasionally passing over low / dropped kerbs onto the verge and similarly passing from the verge onto the road.

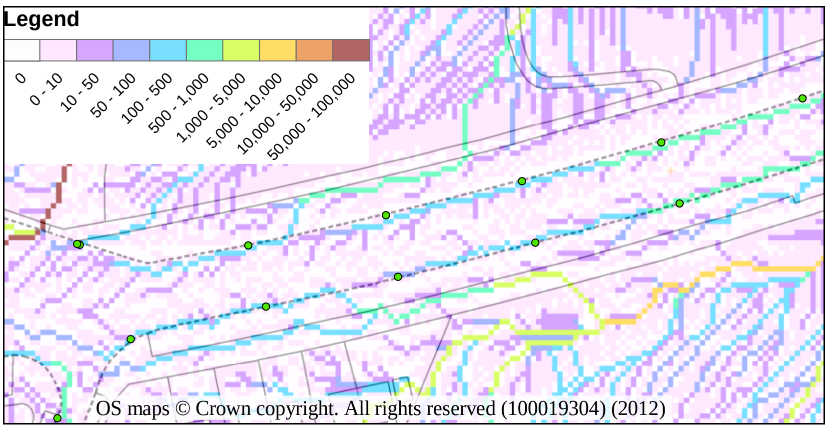


Figure A3.3: Flow accumulations (m^2) for a section of the monitoring and case study site

The watersheds based on the gully locations are shown in Figure A3.4. It can be seen that using this methodology some gullies appear to drain only a small area, whereas others appear to drain very long sections of road. While it may be the case that some gullies are located away from the flow paths, it is considered that in reality the gullies mainly lie on the flow paths but the presence of kerbs means that the LiDAR levels are inaccurate immediately adjacent to the kerbs; hence the flow paths are also inaccurate. Using the ArcMap Snap Pour Point tool to move the gully locations to an adjacent raster cell with a more significant flow path results in the watersheds shown in Figure A3.5. These suggest a more reasonable representation whereby each gully drains the area up to the next gully. It is these watersheds which are used for the remainder of this study.

Using the LiDAR data, the longitudinal slope of the road has been calculated to be between 1 in 23 and 1 in 33, with a mean of 1 in 27. The transverse slope is between 1 in 24 and 1 in 57 with a mean of 1 in 35. The road surface is of average to good quality; hence a Manning roughness of 0.017 is used. Applying a rainfall event of a 1 year return period and 5 minute duration to the areas draining to each gully shows that flow widths (for the 25 m spaced gullies) of between 0.36 m and 0.87 m with a mean of 0.59 m, are expected. For a 5 year return period event of the same duration the minimum, maximum and mean flow widths are 0.43 m, 1.03 m and 0.69 m respectively. The transverse gradient of the road surface is particularly influential in the variability of these values.

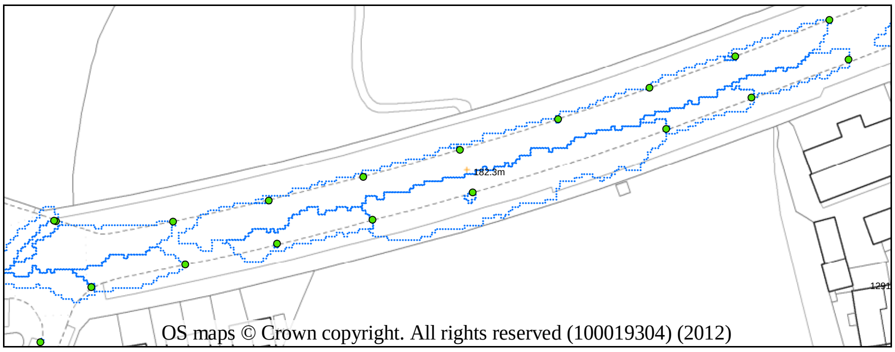


Figure A3.4: Gully watersheds on a part of the monitoring and case study site (blue dotted lines)

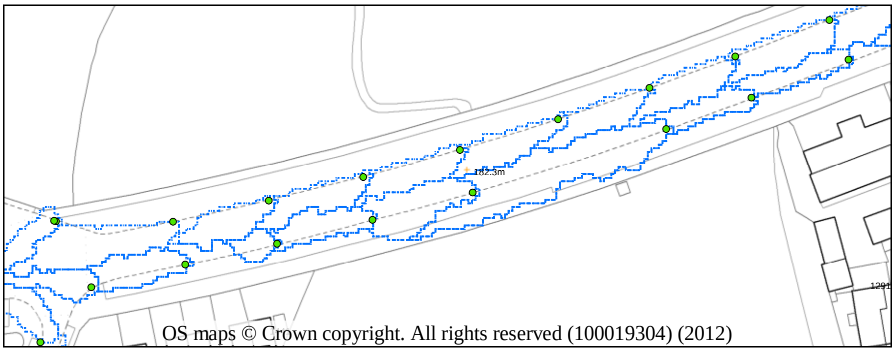


Figure A3.5: Snapped gully watersheds on a part of the case study site (blue dotted lines)

Gully grates installed

The survey of gully grates revealed that there are four types, as shown in Figure A3.6. Type 1, the most common (~ 400x370 mm and of which there are 23), is shown in Figure 6a. This is probably the original grate for the length of road (or at least the original since the last time the road was significantly resurfaced). Type 2, Figure 6b (~380 x 440 mm) occurs in two locations adjacent to a combined sewer storage tank which was installed in the early 2000's. Type 3 (~400x310 mm) occurs in two separate locations and Type 4 (~380x310 mm) occurs in three adjacent locations.

'Spacing of Road Gullies'[11] splits road gullies into 5 classes, P to T, based on their hydraulic characteristics. Using the photographs and measurements of the gratings, the classification of the surveyed gratings has been calculated; the gratings are all class S and are thus in the second lowest capacity group.

a) Type 1

b) Type 2

d) Type 3

e) Type 4

Figure A3.6: Types of gully grate on the case study road

Effect of combined sewer system

It was desirable that the monitoring programme should not be affected by surcharge within the combined sewer system. Hydraulic simulations of the local sewer system were carried out using the results of InfoWorks CS modelling undertaken as part of the Defra River Aire Integrated Urban Drainage pilot project[12]. Figure A3.7 shows the modelled sewer layout in the study area whilst Figure A3.8 shows the peak hydraulic grade line for a 30 minute duration event with an event probability of 1% (1 in 100 years).

[11] British Standards Institution, "BS EN 124: Gully tops and manhole tops for vehicular and pedestrian areas — Design requirements, type testing, marking, quality control", (1994).

[12] http://archive.defra.gov.uk/environment/flooding/documents/manage/surfacewater/airereport.pdf

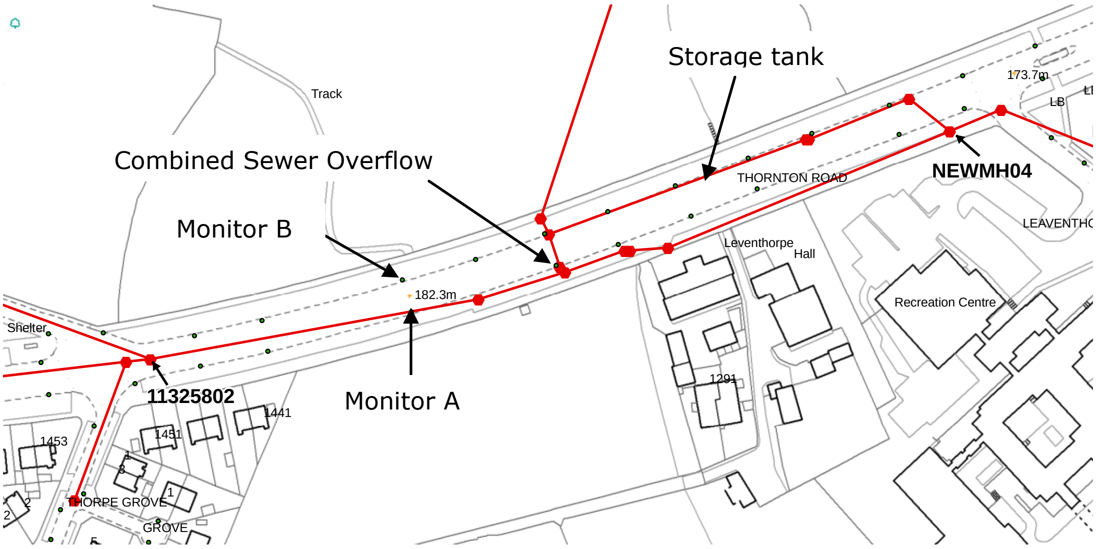


Figure A3.7: The combined sewer system in Thornton Road

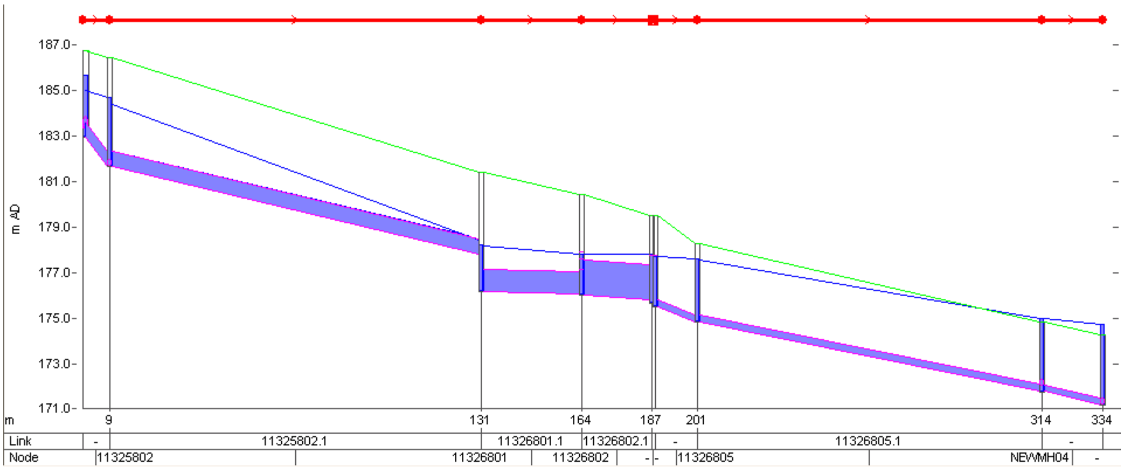


Figure A3.8: Hydraulic grade line for a 30 minute event with an annual probability of 1%.

The plan and long sections in Figures A3.7 and A3.8 can be cross referenced at nodes 11325802 and NEWMH04. The presence of the combined sewer overflow draws down the maximum elevation of the hydraulic grade line shown in blue on the section. This indicates that the gullies just upstream of the CSO are not affected by surcharging in the sewer, but only affected by the capacity of the grate and the connection itself.

The ideal choice of gullies for monitoring would have been those immediately adjacent to the combined sewer overflow. However, it was not possible to construct the chambers to house the depth monitor loggers at this location due to the presence of the overflow structure itself. Therefore the second choice location, the first pair of gullies some 50 metres upstream of the combined sewer overflow as illustrated in Figure A3.7, was used. At this location, the hydraulic grade line nonetheless remained at a considerable depth below the surface.

Appendix 4: The monitoring programme

Monitor Installation

Following site selection, monitors and loggers were installed during December, 2011, with a monitor placed in the selected gullies either side of Thornton Road. The monitor on the south side of Thornton Road is referred to as Monitor A and that on the north side, Monitor B (Figure A3.7). The monitor sensors were located in the gullies and the loggers and power packs were placed in purpose built chambers located in the grass verges. These chambers were secured with lockable manhole covers. Connecting cables were fed through pipes to the road gullies and the sensor heads were secured using custom made brackets (Figure A4.1).

Monitor and secure, kerbside chamber

Monitor A; South roadside **Monitor B; North roadside**

Figure A4.1: Photographs of the monitor installations

Monitor details

Details of the monitors used within the study are as follows:

Manufacturer:	Bühler Montec
Model:	xytec 7050 - Hazardous area open channel flow logger
Data measurement capabilities:	level and velocity
Capacity:	32,000 events
Power supply:	8v battery (approximately 6 weeks battery life)

Monitor settings and data download

Data recoded:	level only
Initial data logging rate:	1 minute
Revised data logging rate:	2 minutes
Loggers calibration:	in accordance with operators manual
Data download:	every 3 to 4 weeks to laptop during site visits (4 weeks maximum)
Battery replacement:	during site visits, maximum every 4 weeks

Dimensions of case study gullies and approximate positions of monitor sensor heads

The dimensions of the gulley pots and the location of the monitors within the gulley pots is shown in Figures A4.2 and A4.3. The depth measurement were calibrated to the invert levels of the connections

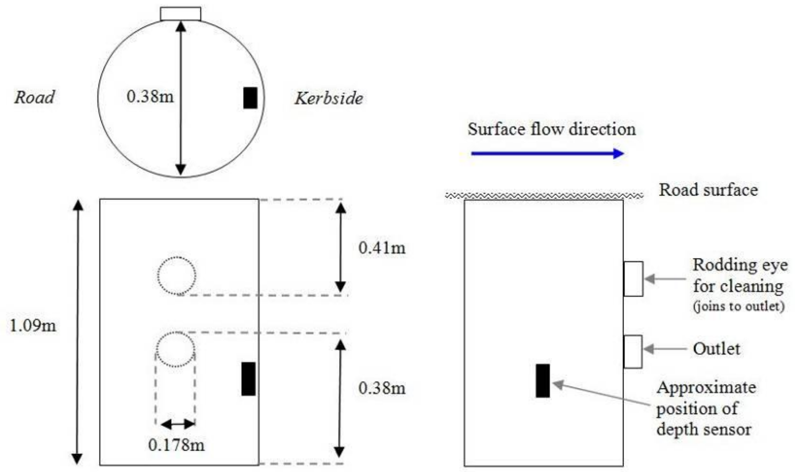


Figure A4.2: Details of Gulley and location of sensor, Monitor A

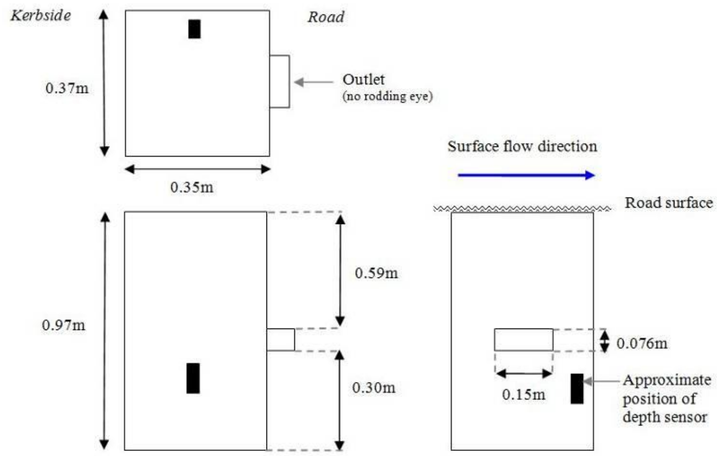


Figure A4.3: Details of Gulley and location of sensor, Monitor B

Logging rate

Initially logging rates were set at one minute intervals. However, it was found that this gave little leeway when site visits to download data were delayed; monitor memory capacity was reached and monitors shut down. This resulted in incomplete data records over time. Comparisons of one and two minute logging rates were undertaken. This identified that an increase in logging rate to two minutes had little significant effect on recoded level values, (Figure A4.4). Logging rates at the case study gullies was therefore revised to two minute intervals in February, 2012.

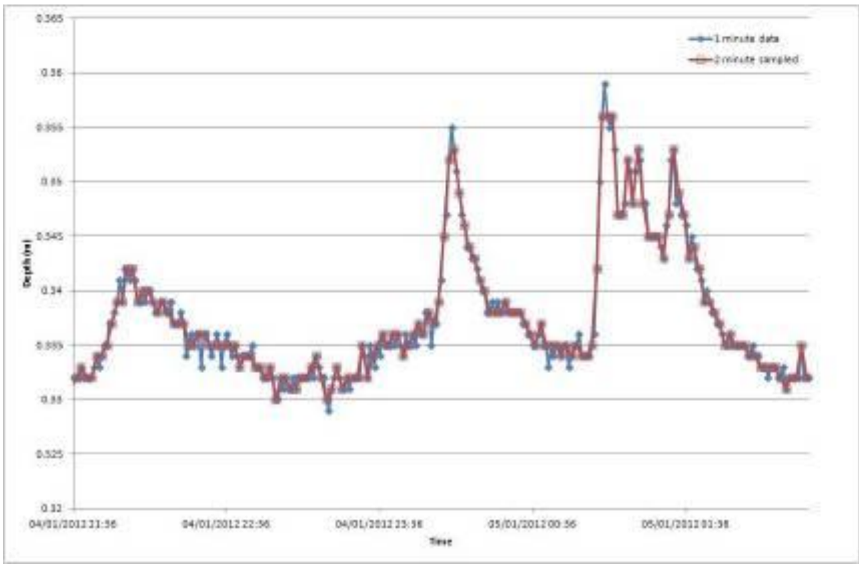


Figure A4.4: Comparison of logged depth for 1 minute and 2 minute recording intervals

Rain gauge data

The rainfall data used in the study was provided by CBMDC. The preferred gauge was located on the roof of a swimming pool located 1.1km from the case study site. However, it was not possible to gain access to this gauge due to closure of the pool and restrictions to site access. Alternative rainfall data was obtained from a gauge located at CBMDC's Jacobs Well offices in Bradford. Although not as close to the case study site, being 4.7km to the east, the Jacobs Well rain gauge provided continuous rainfall data during the study and is indicative of the rainfall falling at the site.

Additional factors - blocked road gullies

During monitor installation and through subsequent site visits, it was noted that several road gullies upstream of the monitored gullies were blocked or partially blocked. Depending on rainfall and volumes of surface flow, it was observed that surface flow was either accommodated by partially blocked gullies, and thus never reached the monitored gullies, or and particularly during heavy and prolonged rainfall, flow by-passed blocked gullies and was captured by the monitored gullies. This was particularly true of the north roadside monitored gully, Monitor B.

The gully pots on the south road side were cleaned at the end of June 2012 following earthworks in an adjacent field. Rapid changes in water levels for Monitor A (south roadside) were apparent from the data as water was sucked from the gully, which then refilled to the outlet level. No information on this or any other gully cleaning regime has yet been obtained from CBMDC.

Additional factors - monitor failure

During the course of the study, on several occasions the data monitors reverted to standby or shutdown mode. On some occasions, this occurred within minutes of the monitors being set to run during a site visit to download data. This meant no more level data was recorded until the next site visit rectified the problem.

Some 'shutdown' instances were related to logger memory capacity being reached (hence revising logging rates to two minutes to reduce data volume). It was also thought that other 'shutdown' instances were due to operator error. However, investigation revealed that poor and loose connections within the battery packs were causing the monitors to shutdown as they were replaced in the monitor chambers following site visits to download data and replace batteries. Due to this, battery packs have been assessed and poor connections repaired.

The only effect on the analysis of the monitoring programme has been the loss of depth data for Monitor A during one event.

Analysis

Rainfall

The analysis focused on three events, during which intensities of greater than 40 mm per hour were experienced. These events occurred on:

- 6th July 2012, where the peak recorded intensity was 48 mm/hour and the 15 minute rainfall depth was 3.8mm, equivalent to a 1 in 1 year rainfall event
- 7th July 2012, where the peak recorded intensity was 102 mm/hour and the 15 minute rainfall depth was 9.0mm, equivalent to a 1 in 3 year rainfall event
- 15th August 2012, where the peak recorded intensity was 66 mm/hour and the 15 minute rainfall depth was 5.4mm, equivalent to a 1 in 1.2 year rainfall event

The rainfall depths associated with fifteen minute duration rainfall, with return periods of 20 and 30 years, are 17.7mm and 20.2mm respectively. Figure A4.5 shows the measured rainfall at Jacobs Well on 7th July.

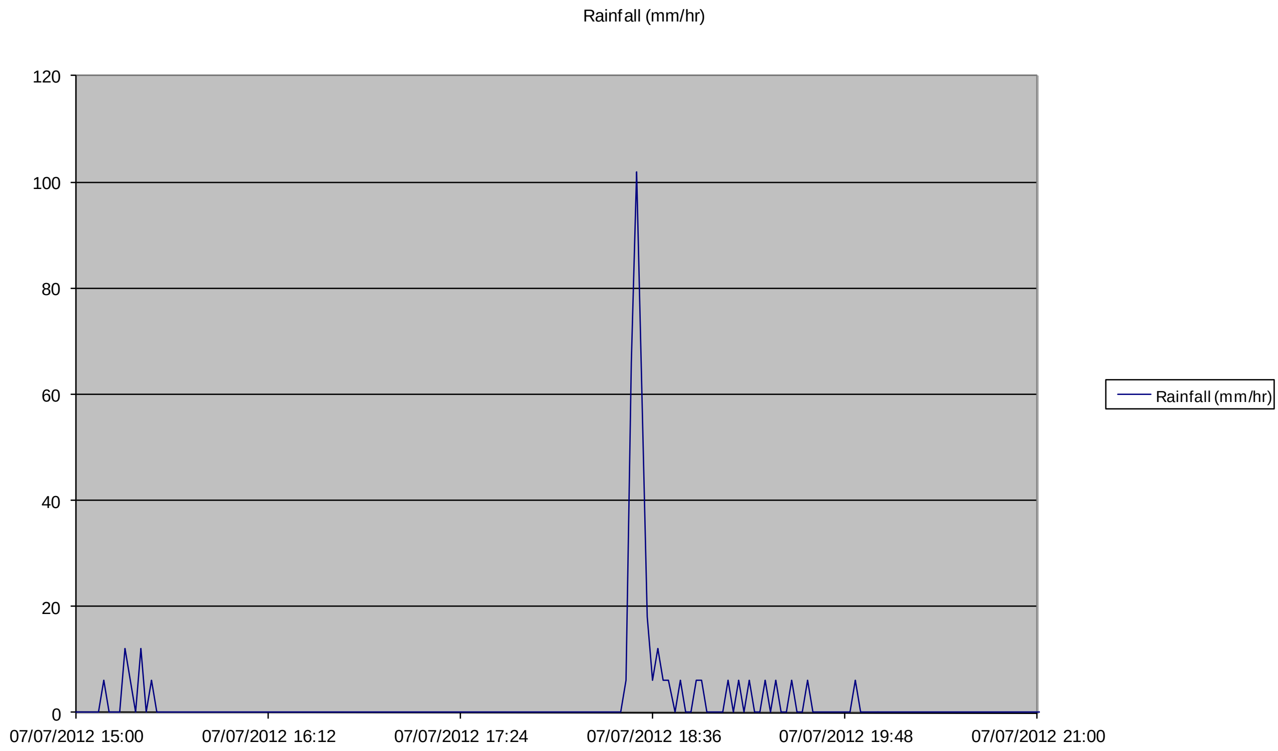


Figure A4.5: Rainfall at Jacobs Well on 7th July 2012

Figures A4.6 and A4.7 show the time varying depth recorded in Gulley A and Figures A4.8 and A4.9 show the same for Gulley B. They also show the flow rates calculated by the rational method as follows;

The time of concentration was assumed to be two minutes; two minute rainfall data from Jacobs Well was used

No losses through imperfect channels were assumed

The contributing areas for each gulley were determined using ArcMap. Two discharge curves were produced

- one for just the impermeable area (A4.6 and A4.8), and
- the other assuming contributions from saturated permeable surfaces (A4.7 and A4.9).

Two conditions were considered:

- The first assumed that all gullies were clear, and
- the second that three upstream gullies were blocked.

In both graphs the horizontal red lines show the invert of the top of the connections. In the case of Gulley A this is at a depth of 0.292 and in the case of Gulley B, it is at a depth of 0.212.

The maximum depth in Gulley A was recorded as 0.341, and in Gulley B as 0.281. All these depths are in metres.

This means that the depth of flow at the top of the connection in Gulley A is 49mm and in Gulley B is 69mm. Both these connections are 150mm in diameter.

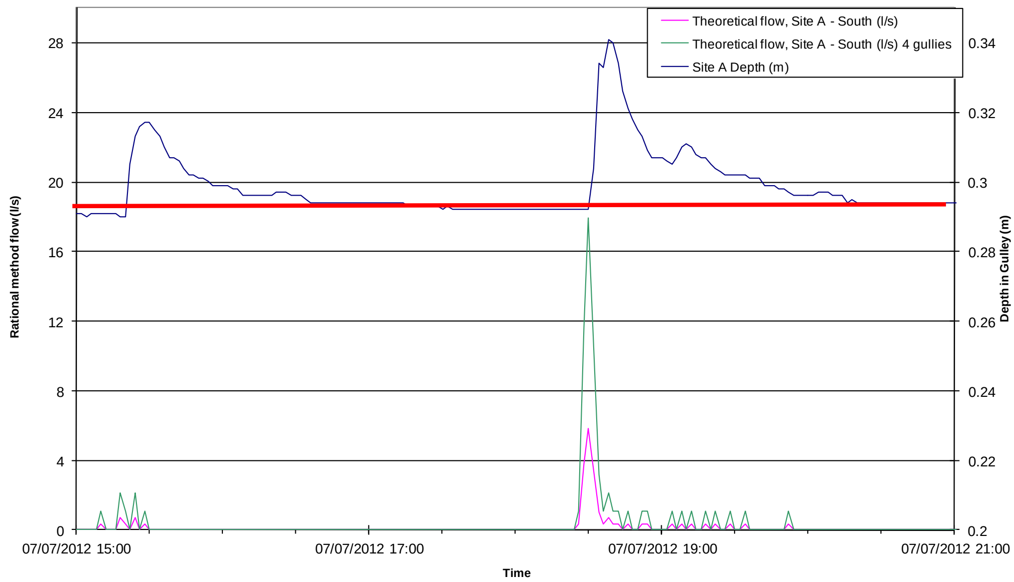


Figure A4.6: Depth and Discharge curves for Gulley A on 7th July 2012 (Calculated discharge based on impermeable area)

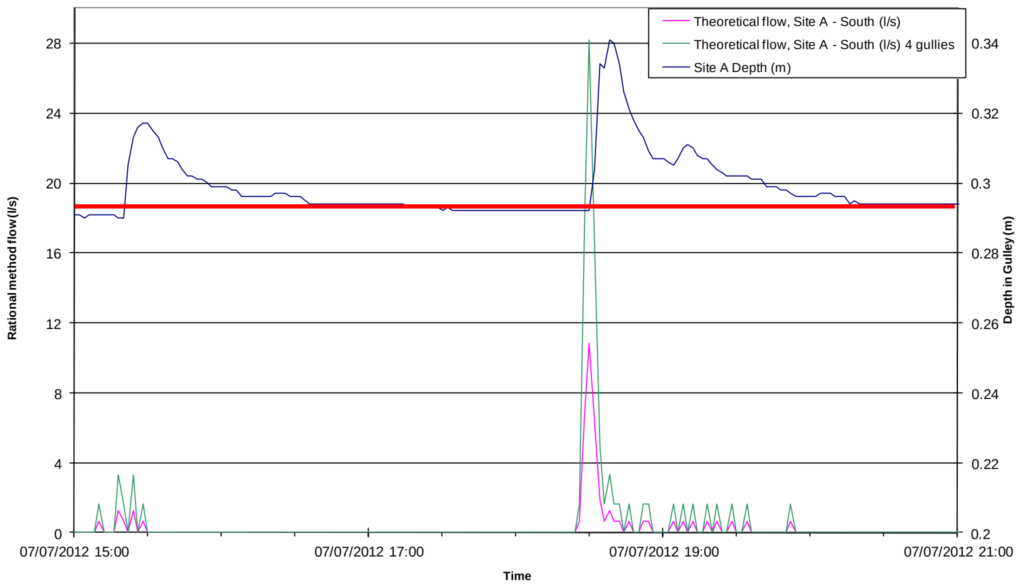


Figure A4.7: Depth and Discharge curves for Gulley A on 7th July 2012 (Calculated discharge based on impermeable and saturated permeable areas)

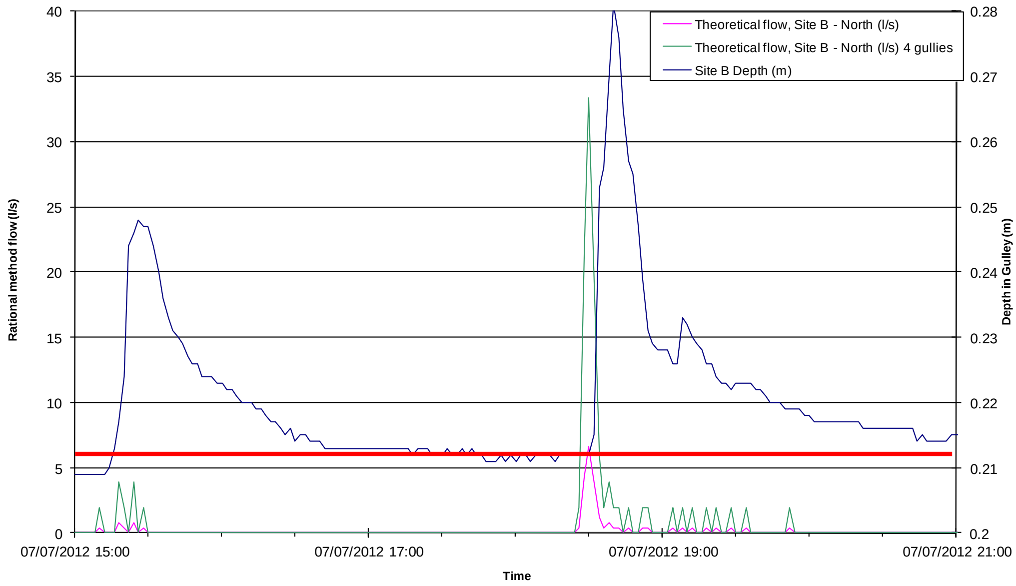


Figure A4.8: Depth and Discharge curves for Gulley B on 7th July 2012 (Calculated discharge based on impermeable area)

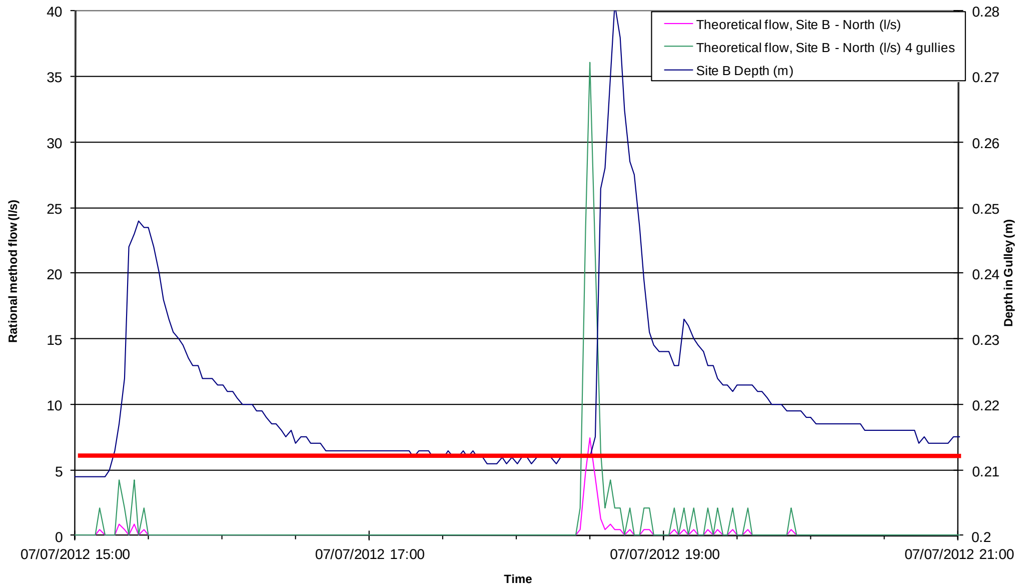


Figure A4.9: Depth and Discharge curves for Gulley B on 7th July 2012 (Calculated discharge based on impermeable and saturated permeable areas)

It can be seen from the graphs that the rainfall at Jacobs Well started to fall earlier than it did at the study site, the delay being around 8 – 10 minutes. The shapes of the calculated flow rates and the measured depths indicate that the rainfall in the centre of Bradford and at the study site was similar, comprising a short pulse of about 15 to 20 minutes, but because of the remoteness of the rain gauge it is not possible to draw conclusions with any real certainty.

Additionally it is possible that some of the upstream gullies could have been blocked or partially blocked and it is possible that the two test gullies were being bypassed to some degree.

Nevertheless, the maximum depth in the connection in Gulley A is only 33% of pipe full depth and that in Gulley B only 46% full.

It may also be inferred from the shape of the depth curves (there is no period of constant depth during the event) that the grates of the two gullies are not throttling the flows into the gullies to constant rates

The gulley pots upstream of Gulley A were cleaned at the end of June so there is a reasonable degree of certainty that the gullies were clear at the time of the event, which means that the peak discharge arriving at the gulley was in the range 6 - 10 l/sec depending on the contribution from permeable surfaces. Furthermore, observations made during the event on the 26th April 2012, suggest that the grating was effective in capturing the flow in the channel without bypassing. Even with some bypassing, it can be deduced that the gulley has similar characteristics to those demonstrated in the laboratory tests.

There is less certainty about the condition of the gullies upstream of Gulley B. However, the depth of flow in the outlet of this gulley is greater than that of Gulley A, which suggests that more flow may be entering the gulley as a result of blockages upstream.

The use of the rational method with no consideration of flood routing to calculate the flow rates at the gullies is likely to overestimate flows and so further consideration was given to this through the application of a fixed PR runoff model (100% runoff) with Wallingford routing to the combined flows from the impermeable and saturated permeable areas for Gullies A and B. The results of this are shown in Figures A4.10 and A4.11

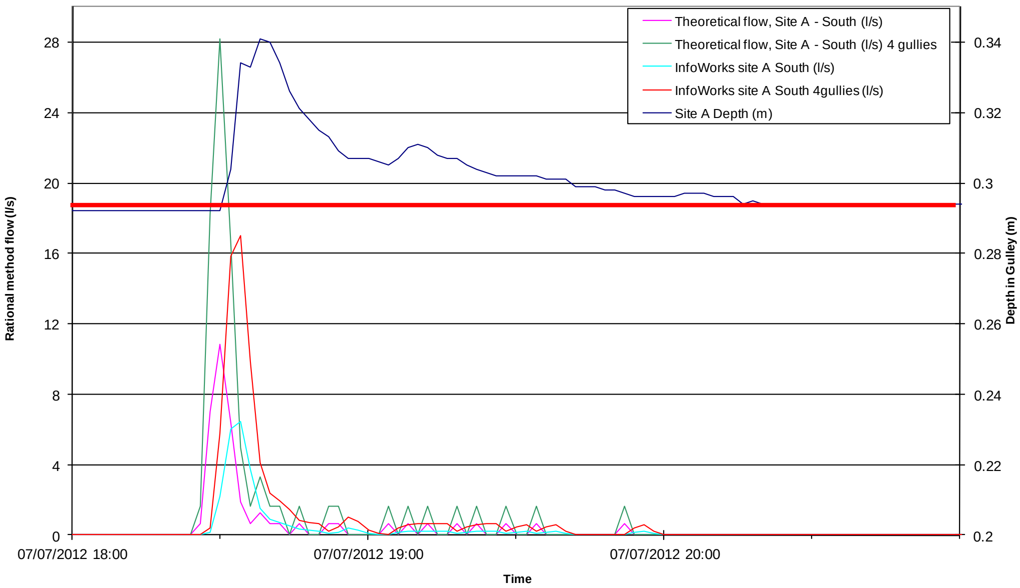


Figure A4.10 application of fixed PR runoff model and Wallingford routing to calculation of flow rates for Gulley A

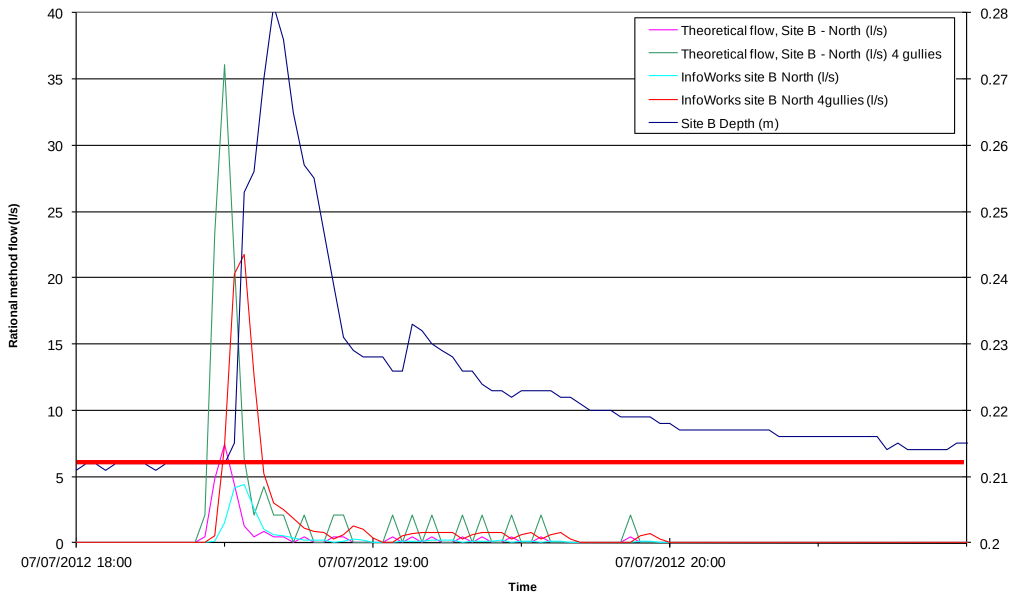


Figure A4.11 application of fixed PR runoff model and Wallingford routing to calculation of flow rates for Gulley B

Predicatbly, the application of the routing model decreases the flow rate by around 40 – 50%, delays the peak and smooths and lengthens the tail of the hydrograph.

The 15 minute rainfall depth for the event on the event on the 7th July 2012 was approximately half the depth of rainfall that the sewer is designed to manage, so the discharge that a single gulley would be currently be expected to manage basaed on the flow rates determined using the Wallingford routing calculations is around 10 – 12 l/sec. Therefore it should be possible to reduce the number of gullies in the area providing that the grates are designed to have sufficient capacity.

Appendix 5: The desk top study

Use of GIS based records of gullies and data on the capacity of connections, to facilitate the identification of the potential for reducing the number of gullies

This is a simple approach that draws on GIS and a simple spreadsheet application to demonstrate how gulley spacing can be modified for normal operation and how runoff from urban or near urban green space enters drainage systems as it flows from gulley to gulley through a developed urban area.

Step 1: Rationalising gullies for normal operation

The laboratory testing of gullies showed that providing gulley grates have sufficient capacity, the controlling factor is the size of the gulley connection and that for a trapped gulley this is around 20 l/s for a 150mm diameter connection and by inference about 9 l/s for a 100mm diameter connection. Thus the gulley spacing can be determined using contributing area determined from the GIS and the rational method with an appropriate value for the rainfall intensity which corresponds to the design or actual capacity of the drainage system. It may be desirable to add a factor of safety to the calculation.

This approach may be applied to the design of new drainage systems and to also enable the identification of existing gullies that are surplus to requirements and therefore could be closed.

Once this has been done, it is then possible to check the response of the gullies and the drainage system to runoff from urban and near urban green space using the GIS to quantify the area of the green space causing the runoff and to identify the pathways through the urban surface. A spreadsheet is used to quantify the rate of runoff, the spare capacity within the urban drainage system and the flood water entering the drainage system through a series of inlets.

Step 2: Runoff from green space

The runoff from green space is determined using the rational equation and is based on an appropriate rainfall intensity based on local experience and using a coefficient representative of the local ground conditions.

Step 2: Assess drainage system capacity

The capacity of the drainage system(s) beneath the flow pathway may be determined from the results of previous hydraulic modelling or if this is not available using design tables for recorded pipe sizes. Failing that, the rational method with rainfall intensities for an appropriate return period and duration event, and the contributing impermeable area, can be used.

Step 3: Assess drainage system capacity available to accept surface water

The rainfall intensity used to determine the runoff from the green space may then be applied to determine the runoff from the impermeable area. This is subtracted from the drainage system capacity to determine its capacity to accept surface water.

Step 4: Assess available inlet capacity

The available capacity of each inlet to accept additional flow from the green space can be determined using the rational method, the contributing area from Step 1 and the difference between the rainfall intensity in step 1 and the rainfall intensity in Step 2.

Step 5: Determine decay of runoff from green space and addition to drainage system flows

Using a progressive approach the available capacity of each inlet is subtracted from the surface water flow rate and added to the drainage system flow rate until there is no more surface water, the capacity of the drainage system is reached or until the surface water flow path leaves the developed urban surface or enters a storage area or water course.

Step 6: Determine depth and width of the flow path

The depth and width of the flow path between each inlet can be determined using the digital elevation model or terrestrial survey data and an appropriate equation such as Manning.

An example of this approach is presented in Figures A5.1 and A5.2 below. Figure A5.1 is a plan showing the runoff from green space entering a highway at points 1 – 4 and gulley inlets at points A – T. Figure A5.2 shows the surface water runoff in the highway in blue and the depth of flow along the highway, how these decay and how they are recharged as runoff from different contributing areas enters the highway.

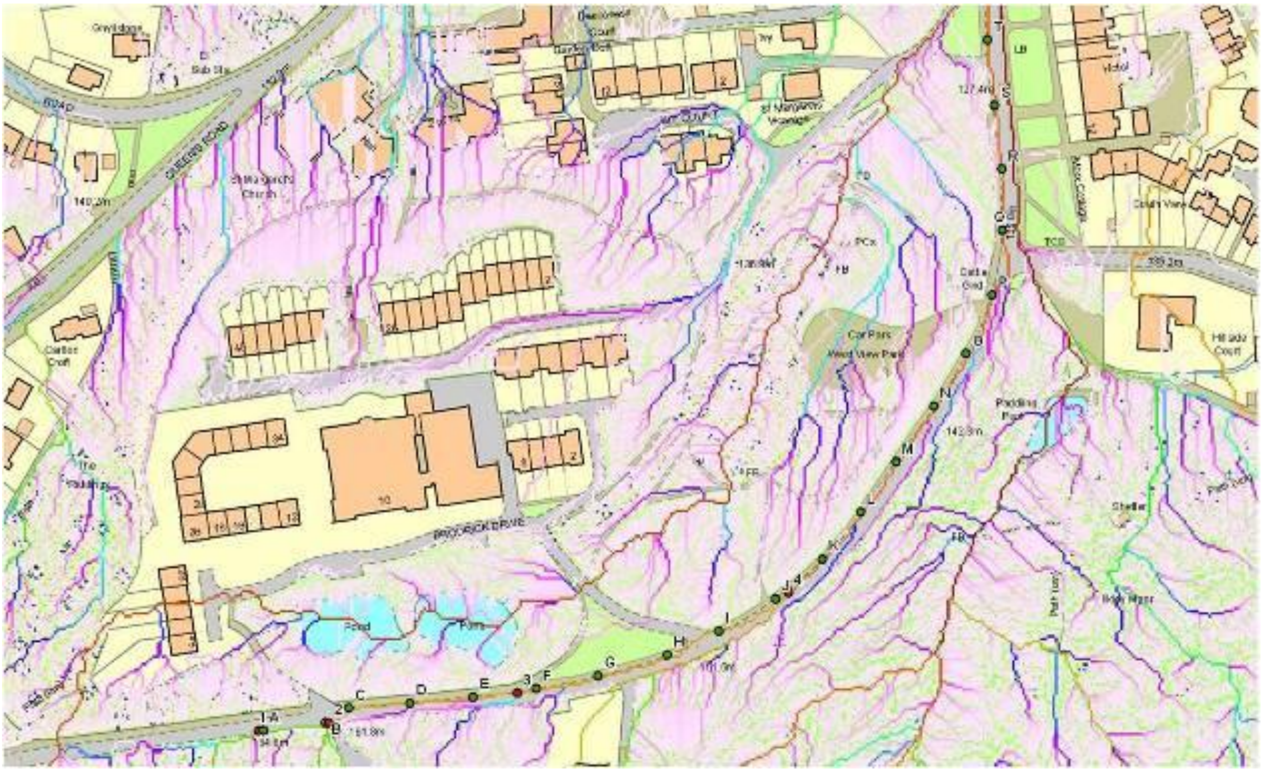


Figure A5.1: Plan showing flow paths and inlets.

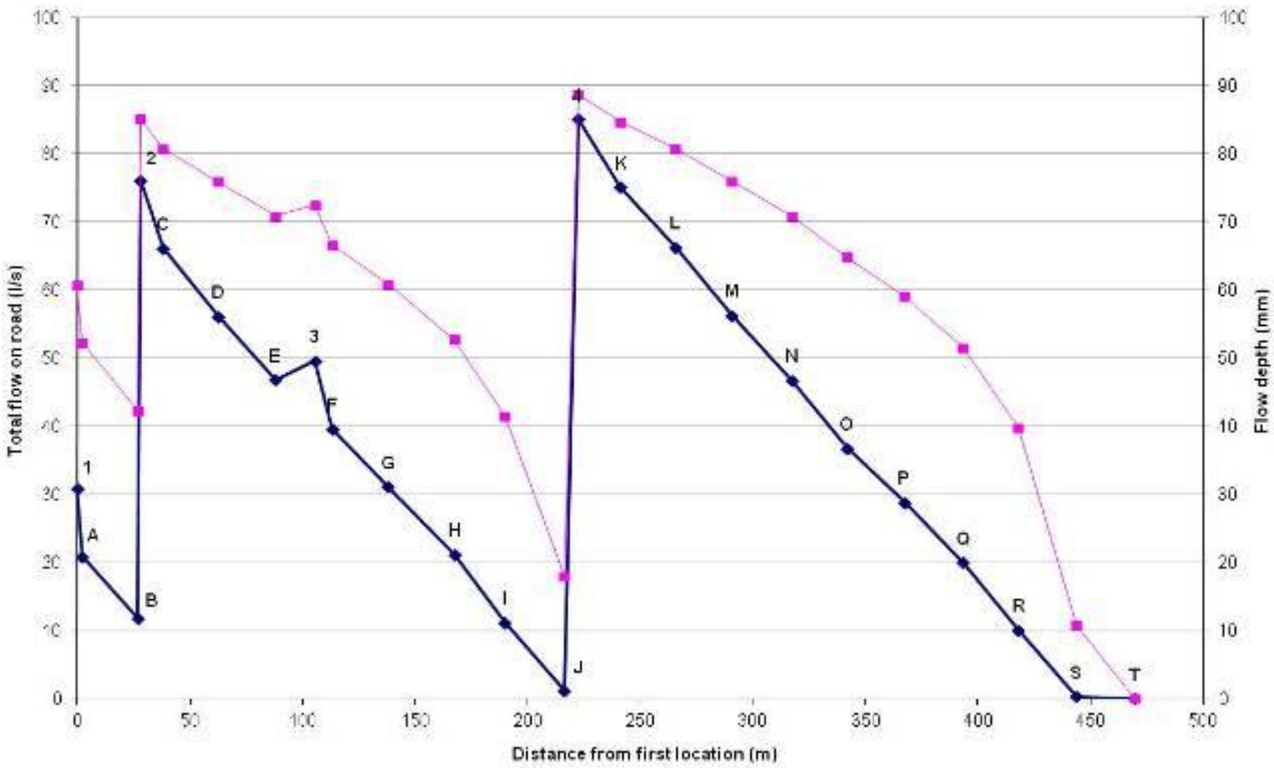


Figure A5.2: Depth and flow rate along highway

Identification of appropriate gulley grating enhancements to ensure effective operation

This is a relatively simple matter. If the requirement is for the gulley capacity to be controlled with any degree of accuracy, the grating must be sufficiently large so as not to limit the flow entering into the gulley nor allow flow to by-pass the gulley.

Figure A5.3 shows gullies in the study catchment operating in wet weather. A5.3a shows a blocked gully, A5.3b shows flow bypassing a gulley but entering it when the grate is raised in A5.3c. A5.3d shows a gulley operating properly.

In Figure A5.4, four gulley grates illustrate the diversity of grates that are used within Europe. A5.4b shows a grate that was designed to block. A5.4b shows a larger grate but which is still designed to trap most solids that arrive at it. A5.4c is a larger grate installed to alleviate a flooding problem, A5.4d is a very large grate at an extremely vulnerable location in Greece. A5.4e is one of a series of carriageway wide grates located a junctions on a steep catchment in Spain and A5.12f is a kerb side gulley at a flatter location in the same township.

Although the last grate in Figure A5.4 requires a length of channel to lead flow into the gulley pot, it is long enough and has a large enough area to achieve this without blocking. It is also narrow enough to fit within the channel section of the highway, thus avoiding problems with cambers or cross falls and does so without presenting a danger to cyclists.

a) Blocked gulley

b) Flow bypassing gulley

c) Flow entering gulley when grate raised

d) Gulley operating properly

Figure A5.3: Gullies operating in wet weather

a) A gulley grate designed to block (Crichton)

b) A larger grate but with small openings with a propensity to block

c) A larger grate in the UK

d) A car sized grate in Greece)

e) A full width carriageway grate on Spain

f) A large channel grate in Spain

Figure A5.4: A selection of effective and ineffective gulley grates

Identification of needs for full or partial disconnection through surface flow management and flow control at connections

Full or partial disconnection of surface flows upstream of locations where drainage capacity is limited are potential options for flood risk management, providing that safe alternative routes are available. The choice between full and partial disconnection depends on local circumstances. Where operation of the alternative flow route has to be limited to a small number of occurrences, then partial disconnection will be required. In this case a non blocking control device, such as a vortex control or a level activated flow limiter, will be required.