Property-level stormwater drainage systems: integrated flow simulation and whole life costs

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Abstract

Property-level stormwater drainage systems consist of roof drainage systems and those systems that connect buildings and their surrounding land to main sewer networks. The design of such systems has traditionally been based on simplistic approaches, and the assumption that individual elements are hydraulically “inconsequential” and/or “independent”. At present, the consequences of these shortcomings range from failure to realise performance benefits, through to a general inability to develop improved design methodologies. Given the extra demands associated with climate change, these limitations will take on increasing importance. There is therefore a real need for tools to enable the development of the integrated design philosophies necessary to meet current and future performance requirements. The research reported herein aims to meet this need by developing an integrated property-level drainage simulation model and associated whole life costing model. Application of the developed simulation model illustrates the importance of utilising appropriate simulation techniques that account for flow interactions, in order to assess the hydraulic effectiveness of different adaptation schemes. Similarly, the whole life cost model is shown to be capable of determining through-life costs of adaptation strategies, hence enabling the cost effectiveness of integrated designs to be readily assessed.

Keywords

Property-level; Stormwater drainage; Interaction; Simulation; Costs
1. Introduction

Piped drainage systems form the backbone of urban drainage infrastructure, both in terms of foul and surface water drainage. Considerable attention has traditionally been given to the design of large scale elements of this infrastructure (e.g. main sewers), and their interaction with each other as well as with natural drainage systems (e.g. rivers). This is understandable as this infrastructure is the primary defence against urban flooding, and the performance of such assets is the responsibility of institutional stakeholders, be they government bodies or private utility companies, and is therefore subject to enforceable standards.

However, the same cannot be said about property-level drainage at the upstream end of urban networks, namely roof drainage systems and those “local” systems that connect buildings and their surrounding areas to the main sewer network; examples of local systems range from those serving a single residential property to those draining large retail parks (see Figure 1). The main reason for this is that such infrastructure has traditionally performed well, and the damages associated with operational failures have therefore been relatively modest. In addition, the ownership of property-level drainage infrastructure has historically rested with private property or land owners and, whilst these owners may have notional triggers for remedial action, there is unlikely to be any automatic trigger, and certainly no enforceable performance standards to meet.

Analysis of construction industry data indicates that the costs associated with building and local drainage systems is significant, accounting for up to 6% (~£1.75 billion) of total UK expenditure on new residential, industrial and
commercial developments (BCIS, 2005). It is estimated that flooding from private sewers and private rainwater goods accounts for 17% and 37% respectively of the total number of urban flood events in England and Wales (Evans et al, 2004). As estimated annual damages from all urban flooding in England and Wales is currently some £270 million, those related to property-level systems may therefore be in the region of £146 million per annum. However, such figures pale into insignificance compared to those predicted to occur due to the impacts of climate change and urbanisation; by the 2080s it is estimated that the number of UK properties at high risk of flooding could rise from the current 82,000 to over 300,000, resulting in estimated annual damages up to £23 billion (Evans et al, 2004).

The research reported herein commenced during the AUDACIOUS project (Ashley et al, 2005), and the primary aim was to develop the computer models (flow simulation and financial) necessary to improve the integrated design of property-level drainage systems, and hence address the challenges outlined above. To be truly effective, it was essential that the developed models were both generally applicable and numerically robust.

2. System characteristics and current approaches

2.1 Roof drainage systems
There are basically two types of roof drainage system. Conventional systems operate at atmospheric pressure, and the driving head is thus limited to the gutter flow depth, whereas siphonic systems are designed to run full-bore, resulting in sub-atmospheric system pressures, higher driving heads and higher flow
velocities. Both types of system consist of three interacting components (roof surface, rainwater collection gutters, system pipework).

Notionally, a roof may be described as flat, sloped or green. Flat roofs are defined as those below a certain gradient, e.g. 10% in the UK (BSI, 2000), and are commonly used for domestic properties in climates with low rainfall or for industrial buildings in developed countries. Most residential and many commercial properties in the developed world have sloped roofs, as their ability to drain naturally reduces the risk of leakage. Green roofs involve the planting of roof areas to attenuate and/or store rainfall. The design of roof surfaces falls largely within the remit of the architect, and no account is generally taken of any flow modifications that may occur, which implicitly assumes that rainfall landing on roof surfaces reaches gutters “instantaneously”.

Current design methods for collection gutters installed in conventional systems are based primarily on empirical relationships and the assumption of free discharge at the outlet (BSI, 2000). Little additional guidance is available for the design of gutters in siphonic systems (BSI, 2007), and the onus is firmly on system designers to ensure adequate capacity. As with roof surfaces, flow modifications within gutters are generally ignored, i.e. it is implicitly assumed that flow entering gutters reaches downpipes “instantaneously”.

In conventional systems, above-ground pipework consists of vertical downpipes connecting the gutter outlets to some form of underground drainage network. Flow conditions are normally free surface and system capacity is usually dependent upon the capacity of the gutters and outlets rather than the vertical downpipes. Empirical approaches again form the basis for current design guidelines (BSI, 2000). In contrast to conventional systems, the flow conditions
within siphonic systems vary enormously, from unsteady free-surface flow through to highly unsteady pulsing flow and steady full bore flow. Current design practice assumes that, for a specified design storm, a siphonic system fills and primes rapidly with 100% water, which allows designs to be based on steady-state hydraulic theory (BSI, 2008). However, such approaches are not applicable when a siphonic system is exposed to a rainfall event below the design criteria or with varying rainfall intensity, and these design methods cannot account for commonly occurring operational problems, such as the blockage of outlets. As with roof surfaces and collection gutters, flow modifications within system pipework are generally ignored, i.e. it is implicitly assumed that flow entering system pipework exits downpipes “instantaneously”.

2.2 Local drainage systems

Local systems are often characterised by a large number of small-medium diameter pipes (typically up to 200mm in diameter), of varying lengths and slopes, and many junctions. As such systems are located in the upper reaches of urban drainage networks, the prevailing flow conditions are unsteady, which contrasts to the type of “quasi-steady” flows located further downstream within main sewer systems.

Although normally designed for free surface conditions, almost all local systems will experience full bore flow events, which can ultimately lead to system surcharging and flooding of surrounding areas. As pipe slopes tend to be relatively steep (e.g. 1 in 100 or 1 in 200), normal conditions are supercritical and the transition to subcritical conditions, which are inextricably linked to those at downstream boundaries (Fox, 1989), can also significantly modify overall performance characteristics.
The relevant UK standards (BSI, 2008) recommend two different approaches to the design of stormwater drainage systems. The design of smaller schemes, where the consequences of failure are relatively minor, is based on the assumption of full bore flows and simple empirical methods, e.g. Manning equation, Colebrook-White equation (Chow, 1959). Where the consequences of system failure are more significant, it is recommended that a flow simulation model is employed to assess the probability and severity of flooding. The relatively large spatial scale of drainage systems, and the need for time varying simulations, means that most simulation methods are based on 1-D principles, and can generally be divided into three categories (steady state, partially dynamic and fully dynamic approaches), all of which have their particular advantages and disadvantages. Whilst fully dynamic approaches are generally the most accurate, their complexity and computational demands, means that such techniques have traditionally been limited to large scale foul and surface water sewer systems, where surcharging events occur relatively frequently and/or the consequences can be severe. Software packages for local drainage systems tend to be based on steady state methodologies, but may include more “advanced” techniques to check for failure modes (e.g. WinDes from Microdrainage).

2.3 Whole life costs

With greater emphasis on sustainability and achieving value for money in both public and private sector investment it is important to consider whole life costs. Any such analysis should incorporate the costs associated with initial installation, ongoing maintenance and any underperformance of a particular asset (e.g. flood damage).
The significant costs and consequences associated with the failure of large scale urban drainage infrastructure has led to the development of generally applicable software tools to assess the whole life costing of such systems (Savic et al., 2005). Whilst there have been a number of case studies investigating the whole life costs associated with individual components of property-level drainage systems, particularly sustainable techniques (HR Wallingford, 2004), there is no generally applicable whole life costing model for general property-level drainage systems. This lack of focus is again probably due to the relatively modest damages historically associated with such systems and the lack of centralised ownership.

3. Drivers for improved design and costing approaches

From the preceding discussion it is apparent that the design of property-level drainage systems is not as advanced as that of other elements of urban drainage infrastructure. Similarly, the design of most local systems is based either on simplistic theory or application of techniques developed for larger scale sewer systems. In addition to these potentially inappropriate design criteria, current approaches (e.g. BSI, 2000; BSI, 2007; BSI 2008) also implicitly treat individual elements of property-level systems as independent elements; whilst the output from one element may well form the input to another element, there is a lack of connection which can lead to under-performance and system failure. Although the costs associated with this deficiency may currently be relatively minor, the impacts associated with climate change are certain to increase these.
The over-simplification of physical processes and the lack of holistic design processes also limit design flexibility, making it difficult to accurately assess the impact of innovative and integrated adaptation measures. This is particularly relevant for rainwater harvesting (RWH) measures, as such systems tend to be extremely sensitive to minor modifications to the performance characteristics of larger system components. In addition, the capital outlay associated with more sustainable approaches to local water management strategies (e.g. green roofs, RWH), often results in relatively long payback periods, and the whole life cost-benefit of an installation can be marginal, particularly if environmental advantages are disregarded. To promote such technologies, more integrated design and whole life costing approaches are required to support further development.

4. Model development

4.1 Property-level drainage model

The property-level drainage model was developed to simulate flow conditions within both roof and local drainage systems, and it consists of a number of different “modules”, all of which are compiled in FORTRAN. The architecture of the property-level drainage model is illustrated in Figure 2 and detailed below. It should be noted that, as the initial development of the roof drainage modules is described in full elsewhere (Wright et al., 2006a), only brief summaries of these modules are given below.
4.1.1 Data input module

Details of the drainage systems to be modelled are entered textually via a spreadsheet interface, and include information describing the physical characteristics and connectivity of: roof surfaces/gutters/pipework, local system pipework, contributing areas within the property boundaries, gullies and storage volumes (e.g. soakaways).

4.1.2 Rainfall module

Rainfall loading for the whole property (roofs and ground surfaces) is specified using an input file containing time varying rainfall intensity. The rainfall module interrogates this data to determine the rainfall intensity at the necessary computational timesteps.

4.1.3 Roof module

The roof flow module can account for multiple roof configurations, including the impact of wind driven rain on non-horizontal surfaces. Flow conditions on sloping roof surfaces supplying traditional gutters are simulated using a simple finite difference solution of the kinematic wave equations (Chow, 1959), which is a computational efficient method of determining conditions over the relatively large roof surface areas. In contrast a volumetric based approach is used to simulate the conditions on flat roofs connected directly to downpipes. To ensure that the impact of adaptation and sustainable approaches can be readily assessed, the roof flow module can also simulate the basic effects of green roof surfaces using the Horton infiltration approach (Chow, 1959), which calculates the quantity of rainfall that infiltrates into a green roof rather than running off into a gutter thus:

\[ f_t = f_c + (f_0 - f_c)e^{-kt} \]  

(1)
Where $f_t$ is the infiltration rate at time $t$, $f_0$ is the initial (maximum) infiltration rate, $f_c$ is the minimum infiltration rate and $k$ is a constant based on soil type.

### 4.1.4 Gutter module

As the accurate representation of gutter conditions is the key element of a successful roof drainage simulation, the gutter flow module incorporates a fully dynamic approach, using the Method of Characteristics numerical solution technique. In common with all fully dynamic approaches, Boundary Conditions (BCs) are required to describe the flow conditions occurring at internal and external system boundaries, including: gutter outlets (fully open, partially open, fully closed), gutter terminations (flow velocity is zero) and gutter edges during overtopping events (conditions described by a weir equation).

### 4.1.5 Downpipe module

The downpipe module can simulate the conditions in both conventional and siphonic roof drainage systems, and uses a numerically-robust two step approach to the routing of rainfall from gutter level to the ground. When the gutter outlets are freely discharging, the flow in both conventional and siphonic systems can be assumed to be free surface, and is thus be simply routed to ground level assuming annular flow conditions within downpipes. However, when the conditions at the gutter outlets become full bore, the flow in siphonic systems is assumed to be full bore. A fully dynamic approach is then applied to the simulation of flow conditions, again using the Method of Characteristics solution technique. The necessary internal and external BCs are either derived from empirical data or known hydraulic theory, and include: system entry BC (linked to the gutter outlet BC), blocked entry BC, pipe bend BCs, pipe junction BCs and
system exit BC (including allowance for submerged discharge, discharge to a sealed manhole and discharge to some form of RWH infrastructure).

4.1.6 Local system inflow module

In addition to system layout and characteristics, the model also requires data describing the various inflows to the local system.

- Inflows from gutter downpipes are taken from the downpipe module, in the form of flow/time files.
- Rainwater runoff to gullies is calculated at each timestep using the known rainfall intensities, contributing areas and standard gully flow equations (BSI, 2008).
- Any gutter overspill identified by the gutter module is automatically added onto the volume of water falling onto the contributing areas.
- In the case of combined sewer flows, foul inflows from building drainage systems can be calculated using an appropriate statistical technique (BSI, 2008) or a suitable simulation model (Swaffield and Galowin, 1992, and included in simulations as a direct inflow via a flow/time file. However, as such flows normally only represent a small proportion of the total flows within a system during an extreme rainfall event, it should rarely be necessary to account for them; for example, in the least extreme rainfall event presented in the model application detailed in Section 5, the rainwater flows peak at ~ 161 l/s, whereas the peak flow from a w.c. would be of the order of 1.5l/s.

The inclusion of the Horton (Chow, 1959) infiltration formulation enables the module to simulate the basic effects of pervious ground surfaces.
4.1.7 Local system routing module

To satisfy the key requirements of the overall model (integration, applicability, robustness), a routing based approach was developed to simulate the conditions within local drainage systems. With reference to the simple system layout shown in Figure 3, the flow routing module simulates flow conditions within local drainage systems as follows:

1. The inflow hydrographs \( [Q(t)_1, Q(t)_2] \) for pipes 1 and 2 \((P_1, P_2)\), which terminate at the most upstream junction \((J_1)\), are calculated by summing the known inflows (roof drainage, contributing area, gutter overspill).
2. The full bore (not pressurised) capacities of \( P_1 \) and \( P_2 \) are calculated from the Manning equation (Chow, 1959), which may be written as:
   \[
   Qf_{lb} = A_i S_i^{1/2} R_i^{2/3} n_i
   \]
   (2)
   Where \( A_i \) is pipe full bore cross-sectional flow area, \( S_i \) is pipe slope, \( R_i \) is pipe full bore hydraulic radius and \( n_i \) is pipe roughness coefficient.
3. The inflow hydrographs are analysed to ensure that they are less than or equal to the relevant full bore capacity. Where the hydrographs have values greater than the full bore capacity, the excess flow is subtracted from the hydrograph and, as the pipes are laid at relatively shallow depths, is assumed to constitute surcharge flow (flow expelled from the system, causing surface flooding)
4. From the modified inflow hydrographs \( [Q_1'(t), Q_2'(t)] \), the mean discharges in \( P_1 \) and \( P_2 \) are calculated \((Qm_i)\).
5. From the mean discharges, and assuming steady flow conditions, the corresponding flow depths within \( P_1 \) and \( P_2 \) \((h_i)\) are calculated using an
iterative solution of the Manning equation (as equation 2 but with the area and hydraulic radius terms not restricted to the full bore values).

6. From the calculated flow depths, the mean velocities in P₁ and P₂ are calculated using the Manning equation (Chow, 1959) written as:

\[ Vm_i = \frac{S^{1/2} R^{2/3}}{n_i} \]  

(3)

7. Using the mean velocities, the mean travel time \( tm_i \) of the hydrographs in P₁ and P₂ are calculated thus:

\[ tm_i = \frac{L_i}{Vm_i} \]  

(4)

where \( L_i \) is the length of pipe \( i \).

8. Using the mean travel times, the modified inflow hydrographs from P₁ and P₂ are translated \([Q(t)_1, Q(t)_2]\) to J₁, i.e. add \( tm_i \) to the time base of hydrograph \( i \).

9. The flows conjoining at J₁ are summed, and form the inflow hydrograph to P₄ \([Q(t)_4]\).

10. Following a similar approach, the inflow hydrographs from P₃ and P₄ \([Q(t)_3, Q(t)_4]\) can be routed to the next downstream junction (J₂).

11. Summing the flows conjoining at J₂ forms the inflow hydrograph to pipe 5 \([Q(t)_5]\).

12. A similar approach is followed to route the inflow hydrograph from P₅ \([Q(t)_5]\) to the system exit, and hence yield the system outflow hydrograph \([Q(t)_{exit}]\).

As the approach outlined above utilises mean flow parameters, it is clearly not as accurate as more computationally demanding (and numerically unstable)
simulation techniques. For example, it does not account for wave attenuation or the dynamics surrounding the formation and propagation of full bore flow conditions. However, by translating the pipe inflows to account for travel times, this technique can be considered to be a reasonable approximation to the, often quite complex, flow interactions that occur within local drainage systems.

4.2 Whole life costing model

The whole life costing model was developed to estimate the direct financial costs associated with different maintenance and adaptation strategies for property-level stormwater drainage systems. Unlike whole life costing models for main sewer networks, the model described herein does not automatically interact with the roof or local drainage models for system optimisation. This is because property-level drainage systems are not subject to the same type of prescriptive performance criteria as main sewer networks, and consequently there are no specific limits to act as a trigger for system upgrading. Rather, the assessment of performance and action triggers are both quite subjective, and may often not depend entirely on the prevailing flow conditions; for example, different property owners may well treat localised flooding resulting from system underperformance differently, depending on the activities associated with the affected areas and the financial implications of any adaptation work.

In order to determine the whole life costs of particular systems and adaptation strategies, the developed model incorporates the following key elements:

- A cost and damage database
- A system input module
- A whole life calculation module
4.2.1 Cost and damage database

The accuracy of any whole life costing model depends primarily on the accuracy of the data employed. Data is required for both the costs associated with the maintenance and adaptation of drainage systems (system costs), and the costs associated with the failure of such systems (damage costs).

In general, one of the main limitations in applying whole life costing is the lack of actual through life performance and cost data to inform whole life costing analyses. This is also the case for flood related damages; as such costs are considered commercially sensitive by the relevant organisations, actual data is not generally available. Therefore, use has been made of industry standard cost databases.

1. System costs (updated regularly)
   - BMI Building maintenance price book (BCIS, 2005)
   - Laxtons building price book (Johnston and Partners, 2005)
   - Spons building price book (Spain, 2005)

2. Damage costs (buildings, contents and other)
   - Dundee tables (Black and Evans, 1999)
   - Multi-coloured manual (Penning-Roswell et al., 2006)

Each specific database has its own unique format and ideal application scenario, e.g. the Laxtons database is suited to the costing of medium/large scale remedial works whilst the BMI database is more suited to the costing of maintenance programmes. As the whole life costing model takes the form of a spreadsheet, the chosen data sources were obtained in digital form, hence avoiding onerous data entry.
As is customary with whole life costing analysis, all incurred costs are subdivided into a number of different categories, which are then used to form the analysis framework. System and damage costs are divided into the following categories:

1. System costs
   - Labour costs
   - Materials and equipment costs

2. Damage costs
   - Residential buildings costs
   - Residential contents costs
   - Commercial buildings costs
   - Commercial contents costs
   - Other costs, e.g. alternative accommodation whilst damage is rectified

### 4.2.2 System input module

The system input module enable four types of data to be entered into the model, namely:

- Global data, incorporating macro factors relevant to the analysis being undertaken, including: analysis duration, number of accounting periods, discount rate, materials discount rate, overheads and profit margins, regional cost multipliers, inflation rates, significant additional tax parameters.

- Existing damage data, describing the existing situation with respect to flood damage, including: the number, type and estimated value (building and contents) of the building(s) being considered, and the predicted depth, frequency and duration of flooding
• System data, describing the work undertaken on the existing drainage system(s) to alleviate flood damage, including: repair of system elements, removal of existing system elements, installation of new system elements, maintenance of system

• Future damage data, describing the flood damage situation after any remedial work has been undertaken, including the same data types as “existing damage data”.

4.2.3 Whole life calculation module
In order to identify the optimum (most economical) adaptation strategy, this module calculates the whole life costs associated with different capital and maintenance schemes. By definition, the costs incurred under such schemes will vary, both in time and scale, and so it is necessary to relate all costs to a common datum. This is achieved using the concept of net present value (NPV), whereby the costs incurred at different times during the analysis period are related to present day value, via the following equation:

\[
NPV = \sum_{i=1}^{n} \frac{C_i}{(1 + r)^i}
\]

(5)

Where \( n \) = number of analysis periods, \( C_i \) = cost incurred during period \( i \) (adaptation, maintenance, damage), \( r \) = discount rate, \( i \) = period.

Data calculation sheets interact with the lookup tables (derived from the data sources) to calculate the costs associated with the measures specified in the data input sheets (global data, system data, damage data).
5. Model application

The case study shown below illustrates the use of the developed models (roof and local drainage) to simulate the conditions within property-level drainage systems under current rainfall loading, future rainfall loading (different climate change scenarios) and the impact of various adaptation strategies (deeper gutters, green roofs, offline storage).

5.1 Systems modelled

The system shown in Figure 4 drains a series of interconnected buildings (Total areas: roof 8110 m$^2$, car park 6135 m$^2$, grass 1110 m$^2$), and had previously been observed to surcharge during periods of heavy rainfall. It is thought that this surcharging may well have exacerbated the flooding problems associated with a nearby school, located a short distance downhill from the case study site.

The data required for modelling purposes (roof, building and collection network) were determined from available drawings, mastermap data, on site observations and estimates. As no definitive gutter or gully data was available, the necessary data were estimated by reference to the requirements of the relevant British Standards (BSI, 2008).

The rainfall conditions selected to represent current conditions were taken from the Flood Estimation Handbook (CEH Wallingford, 1999), and comprised of 3 different event return periods (10, 30 and 100 year RP) and two different event durations (15 and 30 minutes), resulting in a total of six different rainfall events. The short event durations were selected to represent the worst case flow loading for the small area under consideration. To represent future climate change
scenarios, the original FEH event data was uplifted (multiplied) by factors of 1.2 and 1.4 (Evans et al., 2004).

For the purposes of the case study, it was assumed that the prevailing wind direction was from the south east, i.e. perpendicular to the axis of the roof surfaces and gutters. This was considered to be the worst case scenario for the roof drainage system. In reality this would also represent the worst case scenario for the local drainage system, although the developed local drainage model does not account for wind driven rain onto surrounding areas.

5.2 Simulation Results

5.2.1 Roof drainage

Whilst a number of the gutters were not predicted to overtop to any significant degree even under the highest flow loading (100 year RP, 1.4 uplift), Figure 5 shows that gutters 4, 5 and 7 were predicted to overtop under the lowest flow loading conditions (10 year RP, no uplift). As expected, Figure 5 also shows that the same 3 gutters (4, 5, 7) are predicted to overtop significantly under the most extreme flow loading. Examination of the system schematic shown in Figure 4 highlights the reasons for this pattern of performance, namely:

- Gutters 4 and 7 drain roof surfaces on the windward side of the building, resulting in higher exposure to the wind driven rainfall.

- Although gutter 6 drains the largest area of roof surface, and almost half of this is on the windward side of the building, it does not overtop as it is a valley gutter and has hence been designed with a much higher implicit factor of safety than the overtopping parapet gutters (4, 5 and 7).
• Although gutter 5 drains a leeward roof surface, it does overtop as it drains a relatively large area and has only two downpipes along its length.

It is also interesting to note that the predicted order (in terms of magnitude) of gutter overtopping is dependent on the precise intensity of the rainfall event (described by the event return period and event uplift) and its interaction with specific gutters. Hence, although gutter 5 may be predicted to overtop more than gutter 7 for a 30 RP event with no uplift, the reverse is true if the event uplift is 1.4. This “order alteration”, which can be seen throughout the data presented herein, illustrates the complexity of the prevailing flow conditions and the need for accurate simulation models.

Figure 6 illustrates the effect, on predicted overtopping volumes, of a 50% increase in the gutter depth dimension. As expected, this adaptation results in a significant decrease in predicted overtopping rates. In fact the use of deeper gutters under the most extreme flow loading conditions results in lower predicted overtopping volumes than that shown in Figure 5 for the original gutters under the lowest flow loading conditions, i.e. the use of 50% deeper gutters will totally offset the effect of climate change represented by the uplift of 1.4. It should be noted that the predicted overtopping curve for the deeper gutter 5 (100 year RP) is very similar to that for gutter 7, and is hence obscured in Figure 5.

Figure 7 shows the effect of an alternative approach to the minimisation of gutter overtopping; that is, replacing the existing impervious roof surfaces with a pervious covering, such as a “green roof”. As shown, this results in an even more dramatic effect than installing deeper gutters, to the extent that significant
overtopping is only predicted to occur under the most extreme flow loading (100 year RP, 1.4 uplift).

5.2.2 Local drainage system

Figure 8 shows the surcharging volumes (excess water volume discharged to surface) predicted to occur under a number of different scenarios. As shown significant volumes of water are predicted to surcharge, particularly under the extreme climate change scenario (uplift = 1.4). Figure 8 also highlights the very large surcharge volume predicted to occur at pipe 69, where two major branches of the system combine (see Figure 4). It is interesting to note that increasing the uplift factor has a disproportionately low effect on the predicted surcharging at pipe 69. This is because the upstream pipes have a finite capacity, and no more water can be conveyed downstream once this maximum capacity is reached. The slight increases evident at pipe 69 are a result of the increase in the time during which surcharging occurs.

One solution to the minimisation of surcharging volumes is illustrated in Figure 9, which shows the predicted surcharging volumes if each of the gully inlets were connected to a small offline, storage facility (volume of 3m$^3$, operational only when the connected pipe was surcharging). As shown, the inclusion of these storage volumes has no effect on the predicted surcharging volumes during the 10 year RP event. This is because, at these relatively low rainfall intensities, the gully inflows are not sufficient to induce full bore flow in the connected pipework, and hence the storage facilities do not become operational; all of the surcharging volumes occur further downstream, where system branches and flows combine. Figure 9 also illustrates how the effect of the storage increases as the event return period, and hence rainfall intensity, increases, i.e. as the pipes connected to the gullies become surcharged and spill over into the connected storage
facilities. It is interesting to note that, as all of the storage facilities are located at “source” (gully inlets), their presence has no effect on the surcharging volumes predicted further downstream at pipe 69.

5.2.3 Whole life costs

Figure 10 shows example model output for one of the local drainage system adaptations presented above, where 3m$^3$ offline storage facilities are installed at each gully inlet to minimise system surcharging under extreme loading. For the purposes of analysis, this adaptation was assumed to reduce the frequency of flooding (to depth of 1m) at the downstream school site from once every 2 years to once every 5 years. The system adaptation costs used for this analysis included those associated with the installation of the storage facilities (based on data from Laxtons price book (Johnstons and Partners, 2005), and those associated with the annual maintenance programme required for the storage facilities (based on data from the BMI price book (BCIS, 2005). The flood damage costs used for this analysis were based on data taken from the multi-coloured manual (Penning-Roswell et al., 2006).

The data shown in Figure 10 indicates that an initial capital investment of £38k and an enhanced maintenance scheme costing approximately £2.2k per annum, results in an overall net benefit of £70k over the 30 year projected system life (pre adaptation costs – post adaptation costs). Whilst this may not initially appear to be a significant net benefit, it should be recalled that this analysis only accounts for direct financial costs/benefits, and hence does not take account of the less tangible impacts associated with flooding. As the site in question is a school, it can be envisaged that the indirect impacts of flooding, including closure of the school and the consequent disruption to parents normal daytime activities
(including work), would be considerably more significant than the direct economic damages.

6. Conclusions and future work

The work reported herein has highlighted that the design of property-level stormwater drainage systems has traditionally relied on a relatively large number of simplifying assumptions, with little or no consideration given to flow modifications and interactions through/between system elements. The primary aim of this research project was to develop the models required to facilitate the improved design of integrated property-level stormwater drainage systems.

An integrated flow simulation model has been presented, and has been shown to be capable of simulating the conditions from the time/location rainfall lands, through the various system elements (roof surfaces, gutters, downpipes, ground surfaces, local drainage network) to the system outlet. Crucially, this model utilises simulation techniques appropriate for the individual elements of the system, rather than using over-simplistic assumptions or “downscaled” main sewer modelling techniques. In addition, the integrated nature of the model enables flow interactions between the various system elements to be simulated, which is an essential requirement in order to be able to realise the multiple benefits of innovative stormwater management techniques, and hence help address the future challenges posed by climate change. The use of appropriate techniques also helps to transform the perception of property-level systems from
unsophisticated elements of the overall network to flexible, key components within the overall urban drainage cycle.

A whole life cost model has also been presented, and has been shown to be capable of determining the through-life cost of competing drainage system adaptation strategies. Importantly, the model uses a variety of different cost data sources, and hence is generally applicable to property-level stormwater drainage systems of any size.

The models reported herein have been developed over the past 5 years, and continue to be refined. In particular work is underway to incorporate the results of a number of ongoing EPSRC research projects, both to improve the simulation of system surcharging (EP/F029047/1) and to make use of the latest UKCP09 climate projections (EP/F038143/1), rather than the simplistic uplifts employed herein. Work is also underway to better account for the less tangible, social impacts of flooding, so that such impacts may also be incorporated into the whole life cost model.

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References


Figure 1: Typical local drainage system layout
Figure 1: Typical local drainage system layout
Figure 2 Property-level drainage model architecture

- **Data input module (section 4.1.1)**
  - System data entered

- **Rainfall module (section 4.1.2)**
  - Rainfall loading determined

- **Roof module (section 4.1.3)**
  - Roof runoff determined

- **Gutter module (section 4.1.4)**
  - Gutter flow conditions simulated

- **Downpipe module (section 4.1.5)**
  - Gutter flows routed/simulated to ground level

- **Local system inflow module (section 4.1.6)**
  - Systems inflows allocated/determined

- **Local system routing module (section 4.1.7)**
  - Inflows routed through system and surcharging volumes determined
Figure 3 Simple system layout
Figure 4 Schematic of curtilage drainage system

Legend
- roof surface (with slope direction)
- car park (impermeable)
- grass (permeable)
- gutter
- gutter outlet and downpipe
- gutter number
- gully
- manhole
- 225mm dia. vc pipe
- 100mm dia. vc pipe

gutters 1, 4, 5, 7, 8 - 150mm wide square (110mm dia. downpipes)
gutters 2, 3 - 500mm wide, 130mm deep (150mm dia. downpipes)
gutter 6 - 500mm wide, 160mm deep (160mm dia. downpipes)
Figure 5 Variation in overtopping volume with gutter location, event return period and event uplift (15 minute event duration)
Figure 6 Variation in overtopping volume with gutter location, event return period and gutter depth dimension (15 minute event duration, 1.4 rainfall uplift)
Figure 7 Variation in overtopping volume with gutter location, event return period and roof type (15 minute event duration, 1.4 rainfall uplift)
Figure 8 Variation in surcharge volume with event return period and event uplift
(15 minute event duration)
Figure 9 Variation in surcharge volume with event return period and storage
(15 minute event duration, uplift = 1.4)
Figure 10: Predicted financial implications of improvements to a local drainage system and corresponding maintenance programme.