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Designing for exceedance in urban drainage – good practice

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Summary

This guidance provides good practice advice to drainage engineers, regulators, planners and the construction industry on the design and management of urban sewerage and drainage systems to reduce the impacts from drainage exceedance.

It includes information on the effective design of both underground systems and overland flood conveyance. It also provides advice on risk assessment procedures and planning to reduce the impacts that extreme events may have on people and property within the surrounding area.

This book constitutes Environment Agency R&D Report SC030219/TRI

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1 Introduction to the guidance

1.1 Aims and objectives of the guidance

This guidance aims to provide best practice advice for the design and management of urban sewerage and drainage systems in order to reduce the problems which arise when flows occur that exceed their capacity. It includes information on the effective design of both underground systems and overland flood conveyance. It also provides advice on risk assessment procedures and planning to reduce the impacts that extreme events may have on people and property within the surrounding area.

The broad objective of the guidance is to improve the engineers, planners and designers' appreciation of the risks associated with urban drainage systems and their understanding of how these risks may be mitigated. It provides guidance so that systems can be designed to safely and sustainably accommodate periods when the design capacity of drainage systems are exceeded during extreme events. The guidance will be relevant to areas drained by piped systems or SUDS.

PPG25 *Development and flood risk* (DTLR, 2001) identifies that flooding can occur on a local scale due to runoff exceeding the capacity of the minor system during extreme events and it can only be addressed on a site-specific basis. *Sewers for adoption 5th edition* (Water UK and WRc, 2001) states that properties should be protected against flooding from extreme events and that flood pathways are identified when the drainage system is exceeded. Yet there is no standard way to meet these challenges. This guidance aims to address this anomaly. It complements CIRIA publication C624 *Development and flood risk* (Lancaster *et al,* 2004) by focusing on those extreme events which are as a result of flooding in the urban environment.

The specific objectives of the guidance are to:

- address the key issue of designing urban drainage systems that can cope with periods of exceedance
- provide guidance on risk assessment procedures to determine the likelihood and impacts of drainage exceedance
- provide guidance on planning and layout to reduce the impacts of exceedance in drainage systems
- offer best practice guidance for the design of urban drainage systems that can sustainably accommodate periods of exceedance.

1.2 Limitations of this guidance

This guidance presents information which will enable a variety of stakeholders to identify risks and subsequently design mitigation measures. The publication focuses on extreme events, and considers the water quantity aspects of volume, depth, velocity and duration. Water quality issues are not considered in this document.

This guidance document is applicable across the UK. However different regional and national planning policies, stakeholder interactions and legislation must be taken into account when applying the guidance to each case. The guide is based on the planning guidance and legislation in place from January 2005. The reader should ensure that designs and processes are consistent with current regulatory and legislative frameworks.

1.3 Structure of the guide

The guidance is divided into four sections:

- **Part A Overview** is a strategic overview of the guidance. It covers the main issues in general, and will be useful to planners, developers, regulators and other stakeholders who wish to understand the principles, and obtain an overview of the processes, but do not require an in depth understanding of detailed design.
- **Part B Detailed design** offers detailed risk assessment and design, and is aimed at practitioners with a day-to-day responsibility for drainage design.
- **Part C Case studies** includes case studies demonstrating the important stages of the design and risk assessment process covered in Part B.
- **Part D Appendices** give important supplemental information and details of further information that the user can refer to.

1.4 Sources of information

This guide has been compiled following a worldwide literature review. There is significant information available for flooding and its consequences, however information regarding designing for exceedance events is less common. The guidance identifies good practice from around the world and applies it to the UK.

A consultation workshop was held to gather information and opinions from representatives of the various interested parties including water companies, planners, local authorities, drainage engineers and regulators.

1.5 Associated publications

The work provides good practice guidance on assessing the risk from flooding in extreme events and how to design mitigation measures which can prevent or limit flooding through conveyance and storage. It can be used in conjunction with a variety of other publications and sources of information, which are listed below:

Book 14 *Design of flood storage reservoirs* (Hall *et al,* 1993). Guidance to assist the practising engineer with the detailed design of flood storage reservoirs for flood control in partly urbanised catchment areas.

C523 *Sustainable urban drainage systems – best practice manual for England, Scotland, Wales and Northern Ireland* (Martin *et al*, 2001). This publication provides guidance on employing sustainable methods for surface water drainage and implementing sustainable development into practice.

C521 *Sustainable urban drainage systems – design manual for Scotland and Northern Ireland* (Martin *et al,* 2000a). Like C522 this manual describes good practice in Scotland and Northern Ireland.

C522 *Sustainable urban drainage systems – design manual for England and Wales* (Martin *et al,* 2000b). This manual describes current good practice in England and Wales, and sets out the technical and planning considerations for designing SUDS.

C609 *Sustainable drainage systems. Hydraulic, structural and water quality advice* (Wilson *et al,* 2004). This technical report summarises current knowledge on the best approaches to design and construction of sustainable drainage systems.

C623 *Standards for the repair of buildings following flooding* (Garvin *et al,* 2005).

C624 *Development and flood risk – guidance for the construction industry* (Lancaster *et al,* 2004). This book offers practical guidance when assessing flood risk as part of the development process.

X108 *Drainage of development sites – a guide* (Kellagher, 2004). This guidance is intended to assist all those involved with foul and surface water drainage of development sites.

Information can also be found on CIRIA's flooding and SUDS websites at <www.ciria.org/flooding> and <www.ciria.org/sud>

1.6 Background to drainage exceedance

It is inevitable that as a result of extreme rainfall the capacities of sewers, covered watercourses and other drainage systems will be exceeded on occasion. Periods of exceedance occur when the rate of surface runoff exceeds the drainage system inlet capacity, when the pipe system becomes overloaded, or when the outfall becomes restricted due to flood levels in the receiving water.

Underground conveyance cannot economically or sustainably be built large enough for the most extreme events and, as a result, there will be occasions when surface water runoff will exceed the design capacity of drains. This is especially problematic where the drain is a combined sewer and sewage flooding can result. When drainage system capacity is exceeded the excess water (exceedance flow) is conveyed above ground, and will travel along streets and paths, between and through buildings and across open space (Figure 1.1). Indiscriminate flooding of property can occur (Figure 1.2) when this flow of water is not controlled.

Flooding can have huge social, economic and environmental impact (ICE, 2001). The Ofwat consultation on sewer flooding (Ofwat, 2002) highlighted that the damage to property is a small element of the human impact of floods. This is evident if there is internal flooding of property, as the impacts are a lot more severe and difficult to cope with (Figure 1.3). The stress associated with losing personal belongings, living in temporary accommodation, in addition to the trauma of the clean up and restoration process can be considerable.

Figure 1.1 *Highway acting as a flood pathway in an extreme rainfall event (courtesy Scottish Water)*

Figure 1.2 *Property flooding from overloaded sewerage system (courtesy Scottish Water)*

Figure 1.3 *Example of property damage due to storm sewage flooding (courtesy Pennine Water Group)*

Current climate change predictions indicate that severe weather events will become more frequent. Rainfall could increase by 40 per cent leading to at least a 40 per cent increase in surface runoff and a 100 per cent increase in flood volumes (UKWIR, 2004). This may affect 130 per cent more properties leading to a 200 per cent increase in flood damage (Evans *et al,* 2004). These values although theoretical have been produced using models verified on past performance to predict future changes and are by no way, the most extreme of all the climate change predictions.

Although designers of drainage in new developments are now required to consider the effects of extreme wet weather in their designs, there is no obligation to properly manage the consequence of such events. *Sewers for adoption 5th edition* (Water UK and WRc, 2001) identifies that overland flood pathways should be considered, but no recommendation of the level of protection is given. BS EN 752-4:1998 (BSI, 1997) identifies areas where a level of service check should be undertaken and to what return period, but there is no guidance for dealing with extreme events.

Experience has shown that much of the recorded flooding in urban areas is attributable to the passage of above ground surface flow. However, this above ground conveyance is essential in allowing runoff from extreme events to drain from developed areas effectively. It is clear that much can be done to mitigate the effects if surface flood flow is managed proactively. Recognising the importance of flood pathways along highways and other routes, and the storage of water in low spots, is the first step to better management. Through good design, a second important step is to direct flood flows along routes where the risk of property flooding and the risk to health and safety is minimal. Options to achieve this are available, and explored within this guidance.

Defra's consultation document *Making space for water* (Defra, 2004) has suggested that highways can be used to facilitate the management of extreme events. If highways and other urban features are to be effectively used to convey exceedance flow, then careful design will be essential. Relatively minor features of the urban landscape, such as kerb heights, traffic calming and property threshold details can significantly affect flood risk.

Engaging stakeholders to collectively manage and maintain flood routes, and designing buildings to be more flood resistant, is another important factor in the equation. The Building Regulations (2000) do not take into account property flooding and flood resistance, however in Approved Document C (ODPM, 2004b) which came into force in December 2004, provides advice on flood risk. It states that "*…when local considerations necessitate building in flood prone areas the buildings can be constructed to mitigate some effects of flooding…*"

A greater understanding of the mechanisms of drainage in extreme events and improved guidance on how above-ground flood pathways can be effectively managed can assist in reducing the risk of urban flooding. This guidance aims to address these issues.

Part A Overview

2 The process of exceedance and definitions

Traditionally, urban drainage systems are designed to meet a particular and specified **level of service**, known as the **target level**. This is normally expressed as a frequency of property flooding. A **level of protection** of one in 100 years (annual probability of 0.01 being equalled or exceeded) might be defined for internal property flooding as a suitable target for a new development. This can be delivered using a conventional below ground piped drainage system, designed to a pipe full capacity using a one in two year return period rainfall (annual probability of 0.5 being equalled or exceeded), and then checking the performance for flood protection using a suitable sewer simulation tool. Alternatively SUDS (sustainable (urban) drainage system) might be specified. Its performance may be checked in a similar way. Following such checks, the design may be amended to ensure that the desired level of protection is achieved across the drainage area.

Existing drainage systems typically do not achieve the same level of service as that required for new systems. This is in part due to the structural deterioration and siltation of the existing network. More often, it is due to the network carrying increased flows from expanding urban areas. Once system performance falls below an acceptable level, known as the **trigger level**, early rehabilitation will be planned. This will then raise system performance to an agreed target level. The performance target of a rehabilitated system will of course be higher than the trigger level, but may be less than the performance level for a new system. Further information on performance levels is given in Table 3.1.

The formal or designed drainage system (piped or SUDS) is referred to in this guidance as the **minor system** (Figure 2.1). For a piped system, the **conveyance capacity** will normally be greater than the pipe full capacity, since additional conveyance can be generated as flow backs up in manholes causing surcharging. The resulting slope of the hydraulic gradient can be greater than the gradient of the pipes themselves, forcing more flow through the system. A similar effect can occur with SUDS.

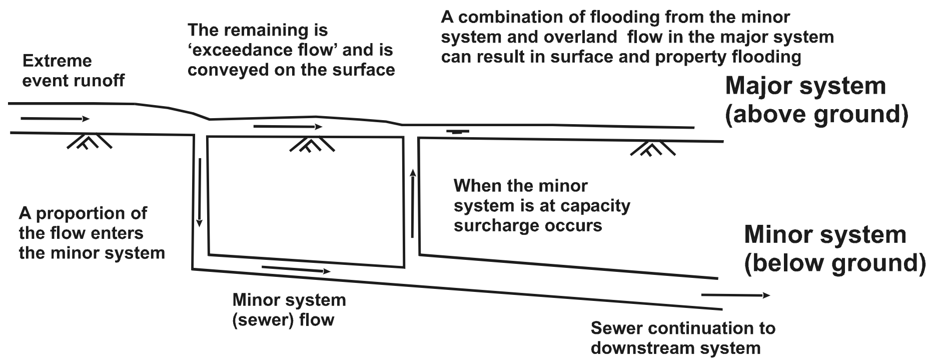


Figure 2.1 *Interaction between the minor and major system during an extreme event*

Once the conveyance capacity of the minor system is exceeded, surface flooding will occur. The excess flow that appears on the surface is known as the exceedance flow. The rainfall events that result in **exceedance flow** are known as **extreme events.** Exceedance flow will be conveyed on the ground by **surface flood pathways**. These may be roads, paths or depressions in the surface (Figure 2.2). Where they have not been specifically designed as flood pathways, they are known as **default pathways**. Otherwise they are know as **designed pathways**. The system of above ground flood pathways, including both open and culverted watercourses, is known as the **major system**.

Even within the target level of service, often there will be some above ground flood flow. Equally, there can be flooding of property before the capacity of the minor system is exceeded. This may occur when the level of property is below the level of the hydraulic gradient in the drainage pipes, especially where there is a direct drainage connection. The connection between the minor and major systems is extremely complex and can only be properly represented by a computer simulation model of both systems. Even then, current capability of modelling above ground flood pathways is limited. A simplified graphical representation of the interaction between the minor and major system is given in Figure 2.3.

Figure 2.2 *Conveyance of exceedance flow in surface flood pathways (courtesy Pennine Water Group)*

The magnitude of surface flooding and the exceedance flow will depend on the return period of the extreme event and the capacity of the minor system. Assuming that the latter is equivalent to the runoff from a 10 year return period storm, Figure 2.4 illustrates typical relative magnitudes for different return periods.

It can be seen from Figure 2.4 (based upon data from a real catchment) that the increase in runoff is by no means proportional to the increase in return period. For example the 100 year runoff is only 1.54 × the 10 year amount. Additionally for the 100 year event, the exceedance flow to be conveyed by the major system is only 1.24 m^3/s compared with the minor system flow (capacity) of 2.34 m^3/s. The minor system capacity is the difference between the exceedance flow of 0 m^3/s and the runoff at approximately the 10 year return period, assuming that all the runoff is drained to the minor system. However, existing sewerage systems rarely convey the full 30 year flow without some surface flooding, so that the surface conveyance can be expected to be greater than this.

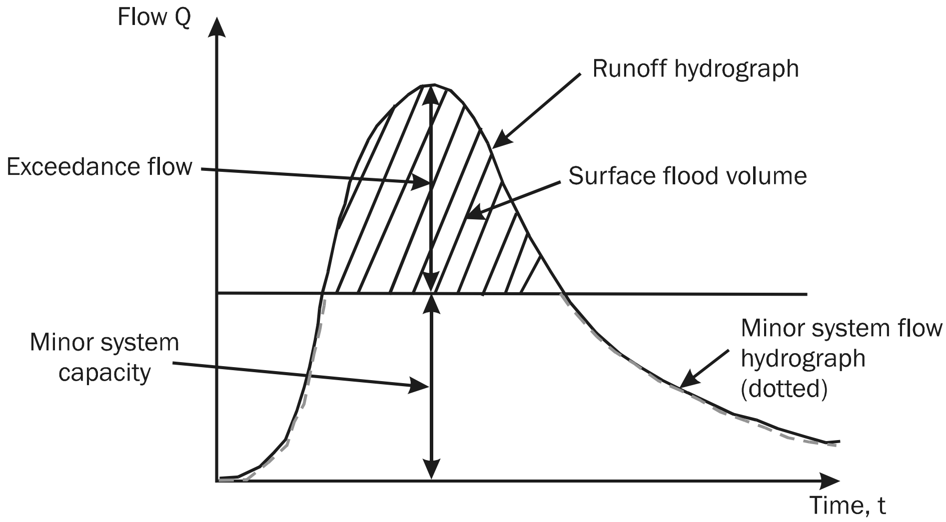


Figure 2.3 *Simplified representation of minor/major system flow*

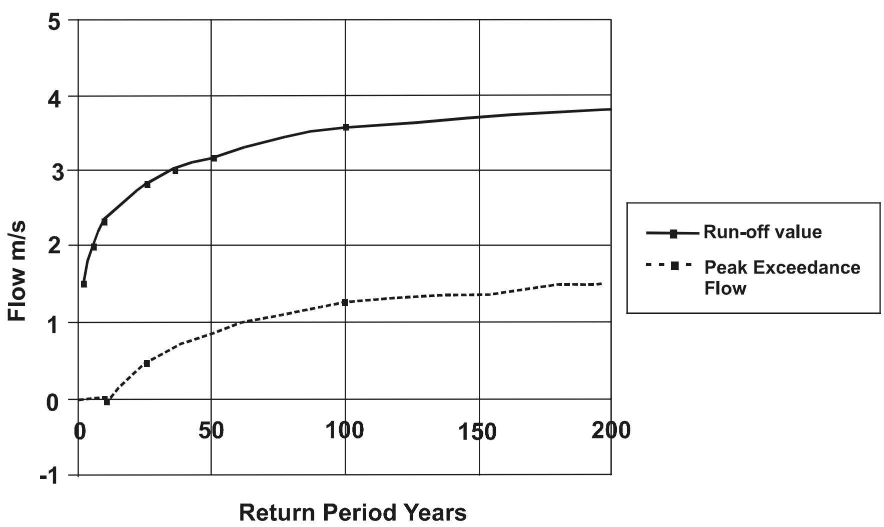


Figure 2.4 *Runoff and exceedance flow for different return period events*

3 Stakeholder roles and drainage performance

3.1 Drainage stakeholders

The ultimate stakeholder of any drainage system is the public. Drainage provides for an essential quality of life and effective drainage is known to be a major contributor to the high levels of public health enjoyed by the developed world. For this reason, effective drainage of wastewater and adequate protection against flooding are prerequisites for both domestic dwellings and industrial and commercial property. Property that frequently floods for example commands a lower value in the market place. Effective drainage is important to property developers, investors and insurers as well as the general public.

In the UK responsibility for drainage is divided between a number of organisations. Sewerage undertakers are responsible for the public sewerage system that serves most urban areas and some rural areas. In England and Wales, the sewerage undertaker is the local water company, in Scotland it is Scottish Water, and in Northern Ireland this function is delivered by the Department for the Environment (NI). Their responsibility extends to the effectual drainage of flow arising from the land within the curtilage of property. Recent English case law (Marcic, 2003) has shown that effectual drainage would not be defined without limit, and that sewerage undertakers may set a reasonable level of service that allows it to fairly distribute its investment in improving sewerage infrastructure to the most needy areas. Sewerage undertakers will define a target level of service, this typically being protection against flooding from storm flows arising from a 30 year return period event. Priority will be given to cases where flooding occurs more frequently, the trigger for early rehabilitation being set typically at a one in 10 year frequency level.

The responsibility for the maintenance of minor watercourses in rural and urban areas falls to riparian owners and the local drainage authority. Local authorities may be unitary or, for example, district or borough councils as part of a two-tier system. For ordinary water courses in England and Wales this lies with the local drainage board or local authority. For statutory main rivers the function lies with the Environment Agency. Similar mechanisms exist in Scotland and Northern Ireland. Highway drainage is normally the responsibility of the local highway authority. Under a non-statutory agreement, highway drains may discharge into public sewers and vice versa.

The responsibility for drainage is fragmented which makes management more complex. Property owners are responsible for drainage *within* the curtilage of their property. They are also responsible for insuring their property against flooding. Historically, insurers have provided insurance for flooding (including flooding caused by the limiting capacities of the minor system in extreme events) for all properties at the same levels of premium. This is now changing because of the increased incidence of flooding in recent years, caused by climate change. Insurers are gradually introducing risk based premiums for flooding. They have advised that protection from a one in 200 year return period event (annual probability of flooding of 0.005 being equalled or exceeded) is the minimum standard at which flood insurance is likely to be available at what most would regard as normal premiums. Where the risk is greater than one in 75 years (annual probability of 0.013

being equalled or exceeded), insurance may not be readily available for new business and could be considered very expensive. A gap has therefore opened between the level of service that sewerage undertakers aim to provide and the minimum standard of protection that insurers consider necessary to provide cover that is affordable.

As explained earlier in the previous chapter, effective management of the runoff from extreme events is a growing consideration. Achieving this will require the co-operation of the relevant stakeholders, these being the sewerage undertakers, highway authorities, local authorities, Environment Agency/DOE (NI)/SEPA, property owners and insurers. For new developments the role of the developer and the planning authority will also be important. Further information can be found in PPG25 *Development and flood risk* (DTLR, 2001) and C624 *Development and flood risk – guidance for the construction industry* (Lancaster *et al,* 2004).

3.2 Managing extreme events in existing urban areas

One of the most challenging aspects of managing the effects of extreme events in existing urban areas is the division of responsibility for drainage set out above. The general public do not understand (and do not wish to understand) the technicalities of current legislation. Government has indicated that urban drainage responsibilities may need to be reviewed (Defra, 2004) however a pragmatic way forward needs to be found in the interim.

In cases of actual flooding of property it is all too easy for one body to attempt to pass the blame onto another. When flooding occurs it is usually difficult to be precise about the return period of the event, whether or not flood water has originated from land outside of a curtilage or from the highway, or if local watercourse flooding has contributed (Figure 3.1)

Figure 3.1 *Serious flooding in Glasgow in July 2002. In this case flooding was shown to be due to sewerage, highway and land drainage flooding combined (courtesy Scottish Water)*

The delivery of timely and robust solutions to urban flooding requires the effective co-operation of the various stakeholders. Further guidance on managing stakeholder interaction is given in Part B, Chapter 5.

3.3 The role of the planner and developer in new developments

Where new developments are proposed, the ability to effectively drain the site is very important and should be a concern for both the planner and the developer. From the developer's perspective, effective drainage (in terms of the minor and major system) is essential in order to deliver maximum value from investment as the inability to gain flood insurance on normal terms can significantly affect property values. The planner's role is important, not only to ensure that the proposed development can be effectively drained above and below ground, but also that there are no significant consequential effects downstream.

Planning applications have to be determined in accordance with the provisions of the "development plan". There is a requirement for planners to consult the various bodies responsible for drainage such as the Environment Agency, Sewerage Undertaker and Local Authorities Drainage Engineers including highways. Further information is given in Chapter 5 and in X108 *Drainage of development sites – a guide* (Kellagher, 2004).

Developers need to consider site drainage early in the development process, and certainly no later than the stage of land acquisition, since drainage can affect land value. The layout of a site can have a substantial impact on the ability to cost-effectively manage extreme events in the developed area (see Part B, Chapter 13). Designers should consult responsible bodies at an early stage before submission of the planning application (whether outline or full) and this would greatly assist planners in reaching their decisions. This should be considered in detail by both developers and planners.

3.4 Drainage design and performance standards

It is apparent from the above that different stakeholders are responsible not only for different drainage systems but also for different levels of performance of the same system. There are numerous design guides that cover drainage design and recommend appropriate standards. These standards often overlap and this can cause considerable confusion with stakeholders. To help clarify some of these issues, Table 3.1 highlights the main drainage performance standards and indicates where appropriate the relevant responsible body.

At present there are no guidelines on the return period of event (extreme event) that should be used for designing for exceedance. It is suggested that return periods of one in 30 to one in 100 or one in 200 year events would form a suitable framework for most applications. Where health and safety issues are important it could be argued that the concept of "any conceivable event" inherent in the procedures set out in the Reservoirs Act (1975) might be applicable. For this purpose the 1000 year event may be suitable. Further guidance on design criteria is given in Chapter 11.

3.5 Key stakeholder lessons

The development of sustainable solutions to exceedance flooding will only be fully realised through good stakeholder interaction. At the start of any project, stakeholders should identify who is responsible for the flooding (which maybe a number of parties) and establish who has an interest in its resolution. The problem should be clearly defined and communicated to all parties. Flood liaison and advice groups (FLAGS see Section 5.8) have been shown to be one way of effectively achieving this. The limits within which various stakeholders operate should be clearly defined so that their expectations are managed effectively (Ashley *et al,* 2005).

Table 3.1 *Drainage design and performance standards*

Return Period (1 in n years)	Description of hydraulic performance	General design principles	Area flooding criteria	Property flooding criteria	Code of practice or standard
1	Pipes designed to run just full – sites with average ground slopes greater than one per cent	×			Sewers for adoption 5th edition
1	Surface water highway design	×			Design manual for roads and bridges 4th edition
1-2	Council highway drainage design	×			Council dependant
2	Pipes designed to run just full – sites with average ground slopes less than one per cent	×			Sewers for adoption 5th edition
2	Design of highway sewers	×			Council dependant
5	Where there are consequences of severe flooding (eg near basements) – pipes designed to not surcharge			×	Sewers for adoption 5th edition
5	No flooding of highway sewers		×	×	Highway Agency
5	Design of channel drainage in highway	×			Highway Agency
10	Protection of rural areas from flooding		×		BS EN 752 Part 4
20	Current Ofwat reporting level for internal and external flooding		×	×	Ofwat
20	Protection of residential areas from flooding		×		BS EN 752 Part 4
30	No flooding in any part of the residential area		×		Sewers for adoption 5th edition
	Protection of city centre/commercial areas from flooding		×		BS EN 752 Part 4
	Level of service for no flooding for existing systems		×		Typical water company criteria eg Yorkshire Water, Scottish Water and Southern Water
50	Protection of underground railways and underpasses		×		BS EN 752 Part 4
50	No flooding from the minor drainage system for new sewers		×		ABI lobbying through response to Making Space for Water
75	General level of protection from flooding			×	ABI – Statement of Principles on the Provision of Flooding Insurance
100	Guidelines for new developments where all flows are retained on site (with an allowance for climate change [+20 per cent] which is greater than 1 in 200 year	×			EA guidance
100	No flooding from the minor drainage system			×	Typical insurance company lobbying eg Norwich Union
50-200	Protection from river flooding		Unclear		MAFF 99 (from PPG25)
200	Minimum level of protection for residential properties for flooding giving "normal terms of cover"			×	Statement of intent by ABI
100-333	Protection from coastal flooding		Unclear		MAFF 99 (from PPG25)

4 Effective management of exceedance

4.1 Identifying above ground flood pathways

As explained in Chapter 2, exceedance conditions resulting in above ground flood flow occur either when the capacity of the formal drainage system is exceeded and/or where the rate of runoff exceeds the inlet capacity of the drain. When calculating runoff for extreme events it should be remembered that considerable runoff can occur from undeveloped plots (Figure 4.1), and these should be accounted for. Without good design, flood flow will follow default flood pathways and this can lead to indiscriminate flooding of property. It is possible to avoid this by identifying and designing above ground flood routes.

Figure 4.1 *Exceedance flow generated by runoff from fields to rear of property (courtesy Pennine Water Group)*

In extreme events flood routes form on existing roads, pathways and in dense urban areas through passages between buildings. Such default pathways may be determined by site inspection, and where necessary confirmed by developing digital terrain models. Where data on dimensions and levels of such potential pathways exist, they may be represented in any drainage simulation models (Figure 4.2).

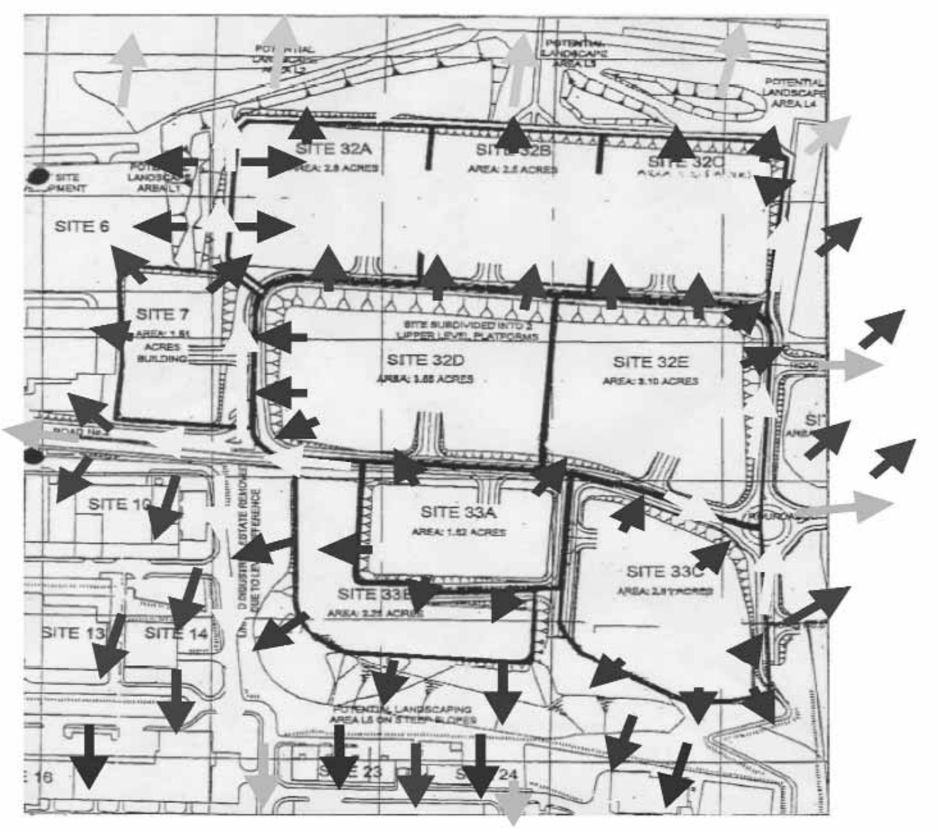


Figure 4.2 *Above ground flood pathways identified during a site development study. Different shaded arrows refer to different types of above ground pathway (courtesy Glasgow City Council)*

4.2 The capacity of surface pathways

The capacity of drainage pathways may be determined by hand calculation. However, by far the better method is to represent them in drainage simulation models. Modern computer models readily allow modelling of such pathways by representing them through open channels. Further information on this is given in Sections 11.3 and 11.4. If property flooding is to be avoided, then the conveyance capacity of flood pathways should be designed so as to convey the whole of the exceedance flow (design pathways). The conveyance capacity can be significantly influenced by relatively minor detail such as kerb heights. Often the effective conveyance of flood flow can be achieved by modifying the detail of a carriageway cross-section, for example by revising the detail of drop kerbs.

Surface pathways should be linked together in the same way as conventional drainage networks, so as to provide a system of conduits that effectively conveys the exceedance flows off the developed site. Flows should be prevented from accumulating at low spots except where temporary surface storage is incorporated into the design strategy (Figure 4.3).

When designing surface flood pathways the designer should remember that unlike conventional drainage they will only convey significant flow very rarely. In practice they will be used on a day to day basis for other purposes. For example, a grass lined channel may be used to convey exceedance flow across an area of open space. The channel will normally be used for recreational purposes and the designer needs to consider the implications of this. For example:

- what safeguards will be in place for its continued availability as a flood channel? Its use for this might be compromised if a fence were to be constructed across its path.
- when the channel is in use for flood conveyance, the public may suddenly be exposed to unexpected flow depths and velocities. What criteria will be used to limit depth and velocity in order to protect public safety?
- after the event, what measures will be in place to clear out any sediment, litter or polluting material? How will the public be warned about the potential hazards of a flood pathway in their community?

Further details on the design and management of surface channels may be found in Chapter 11.

Figure 4.3 *Unplanned ponding of surface flood flow at a low spot leading to property flooding (courtesy Pennine Water Group)*

4.3 Providing surface storage

When providing effective surface flood pathways for extreme events in existing urban areas one of the challenges faced is that the space may not be available to achieve the required conveyance capacity. Where full surface conveyance capacity cannot economically be provided, reduced capacity may be accommodated if flows can be attenuated on site. This can be achieved by the planned provision of surface storage.

As with surface flood channels, surface storage can be accommodated using areas that are used for other purposes for most of the time. When considering potential areas the following question should be answered:

- what depth of storage would be necessary to achieve the required flooding volume?
- how long will it take for the area to drain after the event?
- how will the temporary storage of flood volume affect the primary use of the area?

- will any damage or important loss of use occur?
- will standing water create an unacceptable risk to public health or safety?

Storage on an existing car park to a moderate depth less than kerb height might be acceptable. Structural damage is likely to be insignificant, and if only surface runoff is being stored, then health and safety risks are likely to be acceptable. There will be some loss of use, but provided the area drains relatively quickly after the event, this should not be a problem in most situations.

The required storage volume can in many cases be large. This is especially true where storage is being used to mitigate the impacts of exceedance flow conveyance to downstream systems (see Section 4.4 and Chapter 14). The designer should consider the potential of **sacrificial areas** in such cases. These are areas of low value land to which exceedance flood volumes can be discharged and retained for longer periods of time. Such areas may not have identified outfalls with the water stored infiltrating slowly into the ground and/or evaporating over a period of time after the event.

4.4 The effect of building layout

The spatial distribution of buildings on a site can greatly influence the potential for creating flood pathways and considerably affect property flood risk. Little can be done to affect the building layout in existing urban areas except where significant redevelopment is anticipated. However, much can be done to manage cost effectively exceedance flows in new developments by careful layout of buildings (see Chapter 13).

In any new development it is important that the drainage of the site, including extreme events, is considered at the earliest possible date. Ideally its effects should be part of initial negotiations for land acquisition as it may significantly affect land value. Flood flow paths should be considered in the light of the natural drainage pathways on the site, and space left between buildings to accommodate them. Where roads and pathways can be arranged to act in a secondary capacity as flood pathways then the management of exceedance flows will be much easier. Further details are provided in Chapter 13.

In particular designers and developers should be wary of locating high value property such as housing at low spots, as floodwater will always tend to accumulate there. If unavoidable, special care should be taken to ensure that such property is protected from accumulated flood volumes by raising threshold levels, and/or providing additional drainage.

In many cases it may be more cost effective to amend site layout and the above ground flood channels (major system) rather than alter the below ground (minor) drainage system.

4.5 Impact on downstream systems

The rapid transfer of exceedance flow over the surface can have a significant and damaging impact on downstream receptor systems. The situation is exacerbated when such systems themselves are subjected locally to the effects of an extreme event at the same time, and this can impose significant additional liabilities on stakeholders.

Advice on assessing the impact on downstream systems and developing mitigation measures is given in Chapter 14. However a few vital points are worth noting.

It is important to understand the dynamic interaction between the upstream system conveying the flow, and the downstream receptor system. As well as considering the peak rate of runoff and the flood volume, the timing of the peak relative to that in the receptor system is essential. For example, where a small upstream area discharges into a large river system, the actual impact may be small, not because the rate of exceedance flow is small, but because the maximum value occurs ahead of the peak in the receiving river. It may pass downstream without detriment and in such cases it may be detrimental to provide storage attenuation if this leads to the peak flows occurring at around the same time.

The downstream system can also prevent the exceedance flow from freely discharging, increasing the risk of upstream flooding. For example, when discharging to coastal areas, tide levels may affect the performance of surface flood pathways. An extreme event coinciding with a high tide may not drain as effectively as one occurring at the time of a low tide. In such cases a joint probability analysis may be necessary.

Outfalls from surface flood pathways may require agreements/consents from the owners of receiving watercourse, riparian owners and/or environmental regulators. Early planning of such consents or agreements will greatly assist in land (re)development.

Exceedance flows may convey large quantities of sediments, pollutants washed of surface areas, and other pollutants discharged from wastewater collection systems. These may also have a significant impact on receptor systems, however their consideration is beyond the scope of this guidance.

4.6 Post-event clean-up

Any exceedance event may leave debris or even pollution in storage areas and overland flow paths. In such designated areas procedures for a timely clean-up operation by the responsible stakeholder should be agreed. This may require the removal of debris and pollution, and the spraying down/disinfection of areas where combined sewage flooding has occurred.

Part B Detailed design

5 Managing stakeholder interaction

5.1 The planning process

It will be apparent from the previous section that exceedance flows generated from extreme events will be conveyed on the surface of the flood pathways. Surface storage for these may also be provided. The ability to manage such flows effectively, so as to minimise a sudden increase in flow, will depend on topography, building layout and the configuration of other infrastructure, especially highways. Consideration of site drainage early in the planning process is essential.

Sewers for adoption 5th edition (Water UK and WRc, 2001) contains guidance on the means of effectively draining a site, this includes a requirement to consider drainage of flows and overland flow routes from extreme events. Planning applications should explicitly refer to the drainage of exceedance flows. As stated above, and elaborated in later chapters, the provision of effective surface flow pathways and storage areas may have a significant impact on highway design, building layout, and the provision of other infrastructure. It is essential that consultation with the stakeholders responsible for this infrastructure takes place early in the planning process, and certainly before a formal application for planning permission is submitted.

For new developments, consideration should be given to the existing natural drainage of the site, as explained in Chapter 6. Wherever possible, surface flow pathways and storage areas should take account of the natural topography and land form, and should be included in the framework of the development. Further information on this given in Chapters 12 and 13.

The general principles of the town and country planning systems in the four home countries (England, Scotland, Wales and Northern Ireland) are broadly the same. They all have both strategic and local development plans guiding the location and form of development, and powers by which local planning authorities can control detail in granting planning permission for individual developments. However there always have been differences and divergence has tended to increase since devolution in the late 1990s.

The system described below is that operating in England since the Planning and Compulsory Purchase Act came into effect at the end of September 2004. Details of the arrangements in Scotland, Wales and Northern Ireland are available on the respective websites: <www.scotland.gov.uk>, <www.wales.gov.uk>, and <www.drdni.gov.uk>. A planning bill, introducing significant change, is expected in the current session of the Scottish Parliament, 2005.

The planning process involves applying for planning permission to the relevant local planning authority and can be in outline or detail. An outline application is recommended where it is necessary to establish matters of principle in connection with a large or complex development before proceeding to detailed design.

Under the "plan led" system, introduced by the 1991 Planning and Compensation Act, and now embodied in the 2004 Planning and Compulsory Purchase Act, local planning authorities are required to determine planning applications in accordance with the provisions of the development plan, unless material considerations dictate otherwise. This gives the "development plan", and the policies and proposals it contains, a particular significance.

The statutory development plan is now composed of two main elements:

- the Regional Spatial Strategy (RSS), prepared by the regional planning body for each English region
- the Local Development Framework (LDF) – a folder containing a range of planning documents, prepared and adopted by each district and unitary authority.

The new plans, at both regional and local levels, are "spatial" plans. This means that they can be much more integrative and inclusive than the old style plans that were confined to a narrower land use planning remit. As a result, management issues such as flood management for example are now a legitimate concern of development plans.

The more detailed, lower order plans that make up the LDF ought to be in general conformity with the regional spatial strategy. Therefore it is important for stakeholders to make representations during the plan preparation process so that the RSS can contain the right strategic policies to guide more detailed, local policies and proposals. For example it might be appropriate to specify in the RSS that all new development should adopt the principles of sustainable drainage.

Applicants for planning permission need to be aware of the details contained in the development plan, not only about development in specific locations, but also in any generic policies – about requirements for flood management or sustainable drainage, for instance – that cover the whole plan area. Equally, "statutory consultees", such as the Environment Agency, have the facility to ensure that appropriate policies, covering their area of interest, are included in the plan. Stakeholders who are not included in the list of consultees in Annex E of PPS 12 – *Local development frameworks* (ODPM, 2004a) should still have the opportunity to make a contribution to the plan making process. There is now a statutory requirement for local planning authorities to give greater weight to stakeholder consultation and community involvement.

5.2 Stakeholder responsibilities

In the UK, currently there is no single body responsible for urban drainage and flood control. However Defra's consultation document, *Making space for water* (Defra, 2004), which is applicable to England only, suggests a number of alternative options for the management of surface water.

The delivery of effective drainage involves many organisations and is covered by statute, formal and informal agreements. The process is complex, even more so where above ground flows are concerned. To complicate matters further, legislation and agreements in Wales, Scotland and Northern Ireland are different from England. The main stakeholders relevant to drainage are summarised in Table 5.1 and discussed further in this chapter.

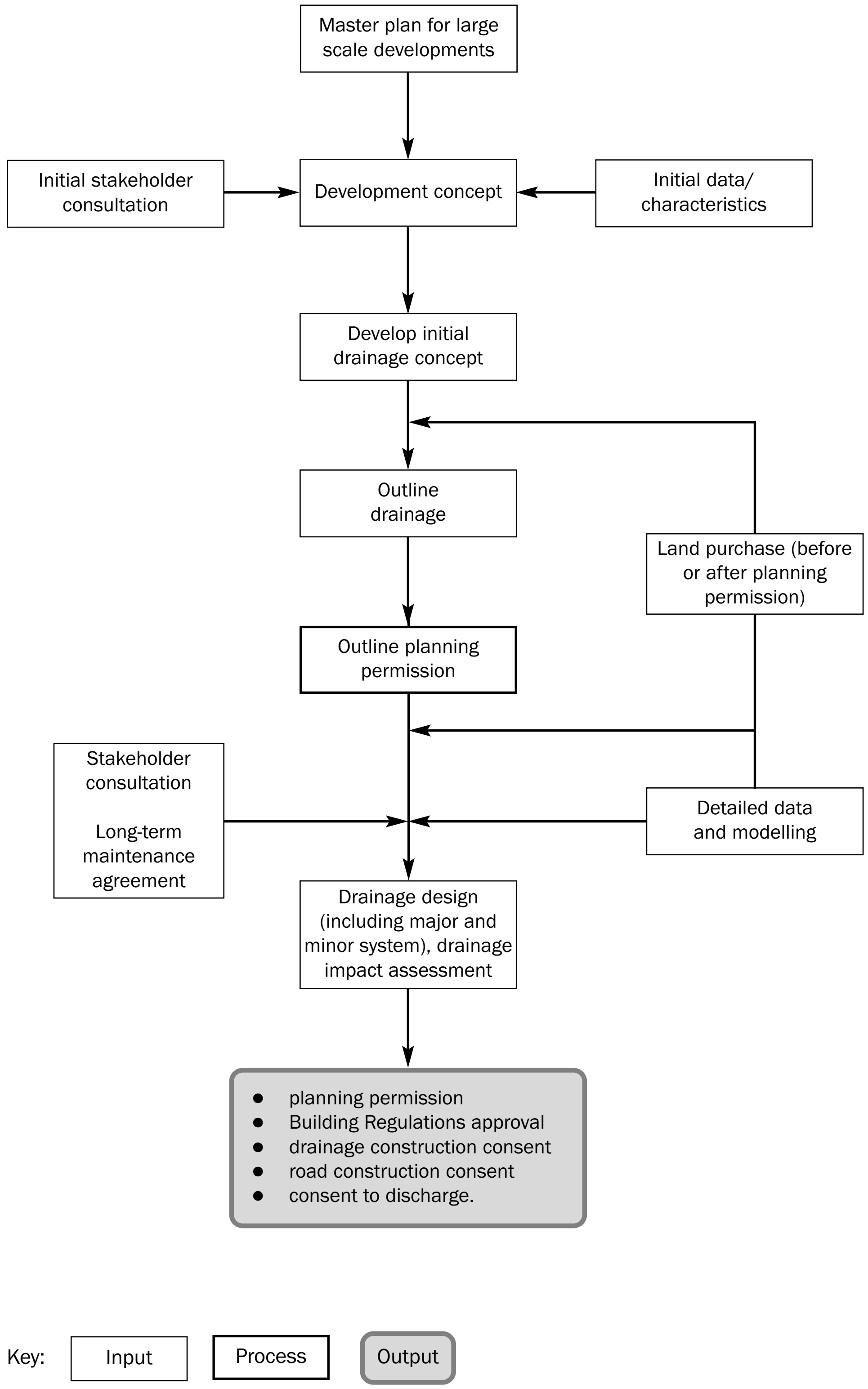


Figure 5.1 *Recommended good practice for managing drainage in the development process (adapted from RP697)*

Table 5.1 *Stakeholders responsible for drainage in England and Wales (after National SUDS Working Group, 2004)*

Name	Function	Authority
Local authority drainage departments	Drainage, flood alleviation and regulation of watercourses, apart from designated main rivers.	Particular responsibilities in drainage districts. Set out in the Land Drainage Act 1991.
Highway authorities	Responsibility to keep the roads (except trunk roads) free from flooding and to make provision for runoff from highways in a proper manner.	Relevant legislation includes the Highways Act 1980 and the Land Drainage Acts 1991 and 1994.
Internal drainage boards	Supervisory duty over flood defence and drainage for low-lying land in England and Wales. Regulation of watercourses apart from designated main rivers within specified areas.	Set out in the Land Drainage Acts 1991 and 1994, covering maintenance, improvement and operation of drainage systems, conservation and revenue-raising.
Sewerage undertakers	Responsibility for maintaining a public sewerage system, which includes sewers carrying surface water away from impermeable areas.	Set out in the Water Industry Act 1991 and 1999, which obliges sewerage undertakers to provide and maintain a drainage and sewerage system, and to authorise and charge for the discharge of trade effluent to sewers. Highly regulated by Ofwat.
Environment Agency	The Agency aims to protect and enhance the environment and to make a positive contribution towards sustainable development in England and Wales. Its water management functions include: ➢ water resources regulation and planning ➢ water quality regulation and planning ➢ flood defence and drainage, maintenance and operations in statutory main rivers.	Powers and duties set out under the Environment Act 1995 and related legislation. Regulation and executive action on water resources, land, water and air quality, flood and coastal defence, flood warning, waste management, navigation, conservation, fisheries and recreation.

5.2.1 Local authorities

Local authorities have a large number of responsibilities including:

- planning
- building control
- local roads
- public landscaping
- highway drainage
- land drainage
- welfare.

There are two systems of local authorities in England, either unitary or two-tier. Their responsibilities are either split or joint. Under a unitary authority the drainage and highway departments will be under one body with the planners. However in a two-tier system, highways are under the control of the county council whereas the detailed planning issues and drainage are the responsibility of the district or borough council.

The local authority building control department (or an accredited private organisation) is responsible for ensuring through their inspectors that the Building Regulations have been adhered to. An important role for the inspectors is to be satisfied with the level of drainage provided and that it does not affect the integrity of the property (this could include above ground pathways).

The local authority planning departments are responsible for approving new development that includes drainage and therefore can influence the adoption of SUDS and the introduction of above ground pathways. CIRIA publication C625 *Model agreements for sustainable water management systems* (Shaffer *et al* ,2004) sets in place a process to enable SUDS systems to be adopted. The funding for ongoing maintenance can be provided through commuted sums (paid by the developer) or through a bond (Shaffer *et al,* 2004).

Highways authorities

The responsibility for the drainage of highways falls to the local highway authority. This will usually be the unitary authority or the county council. Highway drainage may be connected to sewerage and vice versa under a generic non-statutory agreement between sewerage undertakers and highway authorities. Highway authorities are not responsible for the trunk road network. This is the responsibility of the Highways Agency in England, the Transport Directorate in Wales, the Roads Service in Northern Ireland and the Scottish Executive in Scotland.

Land drainage authorities

Land drainage is the responsibility of the local land drainage authority who have permissive powers and are normally the local authority, local land drainage board or county council. For most areas this responsibility lies with the Environment Agency (England and Wales), SEPA (Scotland) and the DOE (Northern Ireland). However in Scotland the local authorities are responsible for the 'primary flood management' so understanding the rate and volume of surface water runoff from any new development is important.

5.2.2 Sewerage undertakers

In England and Wales, the sewerage undertaker is responsible for ensuring the effective drainage of developed areas (Water Act, 1989 and 1991). However, this responsibility is limited to runoff from areas that are within the curtilage of individual properties. The adoption of sewers is generally constrained to adopting piped systems with proper outfalls and can be legally defined as a 'sewer' (National SUDS Working Party, 2004). It does not include the drainage of any highway or undeveloped area, or general land drainage. Views differ as to whether or not a sewerage undertaker has a duty to drain large undeveloped areas within a property curtilage.

The drainage responsibility is normally delivered through the provision of a sewerage system, and a sewerage undertaker is obliged to adopt such systems provided that certain conditions are met. In England, Wales and Northern Ireland, there is a statutory definition of a sewer that specifies its conveyance function but does not restrict it to being a pipe. However the definition does not extend to include surface storage provision. Legislation in Scotland allows for the inclusion of surface storage in order to facilitate sewerage undertakers development. Sewerage undertakers may require a licence to discharge to receiving water (British Waterways Board *v* Severn Trent Water Plc, Court of Appeal, 2001) and may also require a consent for the outfall structure.

The drainage responsibility of sewerage undertakers does not extend to any conceivable event, but is restricted to exclude extreme events. As set out in the Marcic Appeal ruling (Court of Appeal, 2002), this is in recognition that sewerage undertakers have limited powers to raise charges and limited obligations, although there is a requirement for them to set level of service standards as a means of prioritising that investment. Sewerage may be separate (separate pipes for surface water and foul sewage) or combined. A landowner has the right to connect to a public sewer, though where a separate system is provided, this has to be the appropriate sewer. The sewerage undertaker may specify the point of connection.

Within the curtilage, drainage is the responsibility of the land owner. However existing developments may fall under Section 179 of the Water Industry Act where sewers built before 1 October 1937 are the responsibility of the undertaker. This does not prevent a sewerage undertaker making provision for flood protection within the curtilage, say by providing reasonable flood barriers, but there is no statutory obligation for them to do this, and it would have to be with the agreement of the land owner. Landowners also have the responsibility for insuring against flood risk, although the ability to secure such insurance may be influenced by factors outside their control.

5.2.3 Environmental regulators

In England and Wales the Environment Agency (EA) has a wide range of responsibilities, of which the key ones are described in Table 5.1. It can exercise powers to deal with flooding, however, it has no liability relating to it. It is directly responsible for performance and maintenance of main rivers and critical ordinary water courses (where they have been designated as main). The EA may operate and provide flood warning systems (Section 166 of the Water Resources Act 1991).

The EA is a statutory consultee for specified activities as set out in the Town and Country Planning (general development procedure) Order 1995. However, it is not a statutory consultee for all drainage related applications requiring a consent to discharge to a watercourse. It will offer guidance to the planning authority on the rate and volume of surface runoff from new developments. In Scotland this is undertaken by the local authority and in Northern Ireland by the Rivers Agency. Other responsibilities in these areas lie with the Scottish Environmental Protection Agency and the Northern Ireland Environment and Heritage Services.

The EA has the powers to serve conditional prohibition notices related to the quality of the water but not to dictate the standards of how a drainage system should be constructed.

5.3 Stakeholder consultation process

5.3.1 Initial stakeholder consultation phase

The process of reaching early stakeholder agreement is achieved by undertaking the following points:

- identifying criteria for drainage design
- understanding individual stakeholder responsibilities and requirements
- understanding the impact on local communities
- early and regular consultation
- the building of effective personal relationships and trust
- including drainage costs when the site is purchased.

The drainage stakeholders described in Section 5.2 and local authority planners should be consulted during the initial stakeholder consultation phase. During this phase stakeholders should be made aware of and contribute to further developing the surface water network and drainage criteria. This should include proposals to manage exceedance through above ground conveyance and storage if required.

Information collected at this stage of the planning process should enable the planning authority to identify if further details of site drainage are required. This may be through a drainage impact assessment using tools described in this publication for the exceedance element where necessary prior to the submission of a planning application. In addition it may be prudent to consult other stakeholders with interests around the development site.

A drainage impact assessment may be used to demonstrate how surface water will be drained and identify the principles for controlling exceedance flows from extreme events. This will enable the planning authority to set conditions to manage flooding during extreme events.

5.3.2 Stakeholder consultation phase

The main stakeholder consultation phase will require outline drainage design to be submitted which should include the above ground conveyance and/or storage locations if deemed necessary. A risk assessment of the area should be undertaken to determine levels of service and risk of flooding using this guidance. A flood risk assessment may be required depending upon the planning policy in England (PPG25), Wales (TAN15), Scotland (SPP7) and Northern Ireland (PPS15). A drainage impact assessment should be included in any flood risk assessment.

It is critical during this phase that the developer and their drainage designer should liaise with the regulatory authorities to agree the appropriate criteria. In particular, the standards and protection to flooding should be confirmed to ensure the risks are adequately designed and managed. A general summary for drainage design is provided in CIRIA's drainage and SUDS guidance. In areas where changes are being made to existing drainage infrastructure to facilitate the improved management of drainage exceedance, it will be necessary to consult and liaise with a wider range of stakeholders, including existing residents and commercial businesses.

5.4 Good practice in stakeholder interaction

Good practice in stakeholder interaction requires a well planned and open process. Completing a drainage impact assessment (North East Scotland Flooding Advisory Group, 2002) helps to understand and manage the impacts of a proposed development. In Scotland, local authorities are beginning to request this assessment and this method. In conjunction with the guidance it will ensure that above ground pathways are considered, designed and actively managed (National SUDS working group, 2004). The EA in England and Wales have provided guidance for flood risk assessments <www.pipernetworking.com/floodrisk/index.html> depending upon location within flood zones which includes the assessment of surface water impacts and exceedance.

There is little current experience in stakeholder collaboration in achieving designed surface flood conveyance and storage systems for managing extreme events. However, examples of good practice can be drawn from stakeholder collaboration to reach solutions of other flooding problems where responsibilities are shared. Three examples of good practice follow in Sections 5.4.1, 5.4.2 and 5.4.3.

5.4.1 Glasgow east urban flooding

The east end of Glasgow suffered one of the worst urban flooding events in history (Figure 5.2) on 30 July 2002. Over 500 properties were affected as a result of rainfall that represented the one in 100 year event (annual probability = 0.01) in some parts of the drainage area.

Initially the different authorities responsible for drainage and flood control appeared reluctant to admit responsibility for the flooding and a blame culture threatened to develop. Analysis of flood levels and other data allowed the sources of observed flood water to be identified (Figure 5.3).

The advantage of this analysis was that it encouraged different stakeholders to take responsibility for their respective contributions to the flooding. Consequently, this fostered an atmosphere of collaboration in working towards solutions. In this case Glasgow City Council is working in partnership with Scottish Water and SEPA to deliver a holistic solution to the urban flooding. Solutions may include enhancements to the conventional piped sewerage system, removal of restrictions to culverted watercourses, land set aside for local storage attenuation and highways used in flood conveyance for extreme events.

Figure 5.2 *Flooding as a result of overland flow and sewer flooding on the 30 July in Glasgow (courtesy Scottish Water)*

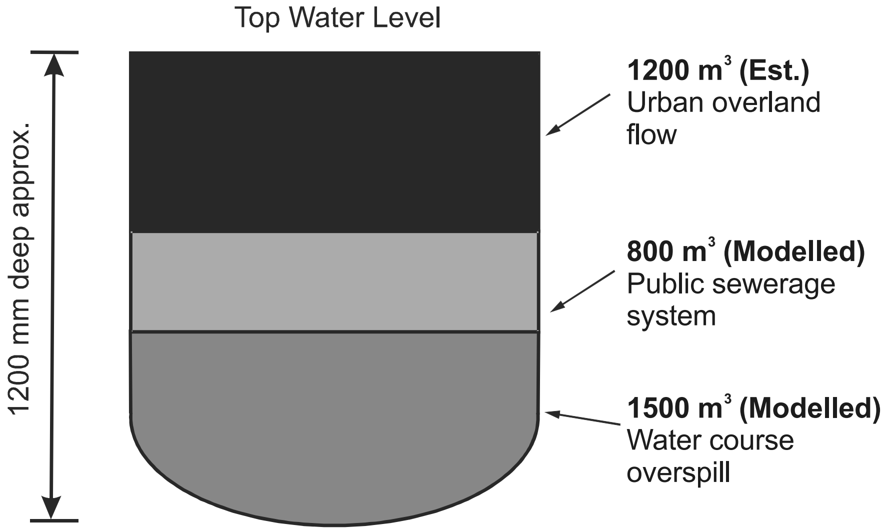


Figure 5.3 *Discrete catchment flood volumes attributable to different sources for 30 July Glasgow floods*

5.4.2 Yorkshire property flooding solutions

The primary driver for this work was to meet Yorkshire Water's AMP3 targets for removing property from the DG5 *Properties at risk [of flooding] register*. In a significant number of cases, however, the cause of observed internal property flooding was unclear. Further investigation and interviews with local residents indicated that the source of flooding might not necessarily be Yorkshire Water's assets. Figure 5.4 shows infiltration of water through the wall of a cellar causing flooding. Although observed flooding incidents had been reported to Yorkshire Water, the source of water was eventually traced to an adjacent water course. In a second example, property flooding was caused by overland flow being diverted off the highway into property set below the level of the carriageway (Figure 5.5). It was only possible to identify the source of flood water in this case by site observation during heavy rain. In both cases a joint approach to solution development was agreed with different stakeholders.

Figure 5.4 *Cellar flooding from infiltration from a local watercourse*

Figure 5.5 *Property flooding by overland flow from the highway*

5.4.3 Flooding of residential area in Birmingham

In this case the flooding of an area of urban housing was traced to a number of sources (O'Leary, 2004). In all, the following agencies were identified as having some responsibility towards that flooding:

- Severn Trent Water.
- Environment Agency.
- Sandwell Council (Riparian Owners).
- Highways Agency.
- British Waterways Board.

Figure 5.6 shows the sources of and properties affected by flooding, while Figure 5.7 indicates the overland flow routes. An agreed scheme was developed where different sub-schemes, each associated with a particular stakeholder, were identified. Subsequent delivery of the sub-schemes is the responsibility of the respective stakeholder. Although early benefits will be achieved from individual stakeholder sub-schemes, the full benefit will only be realised when all are delivered. The process is self regulating and as the various sub-schemes progress, the pressure on the remaining stakeholders to fulfil their obligations increases.

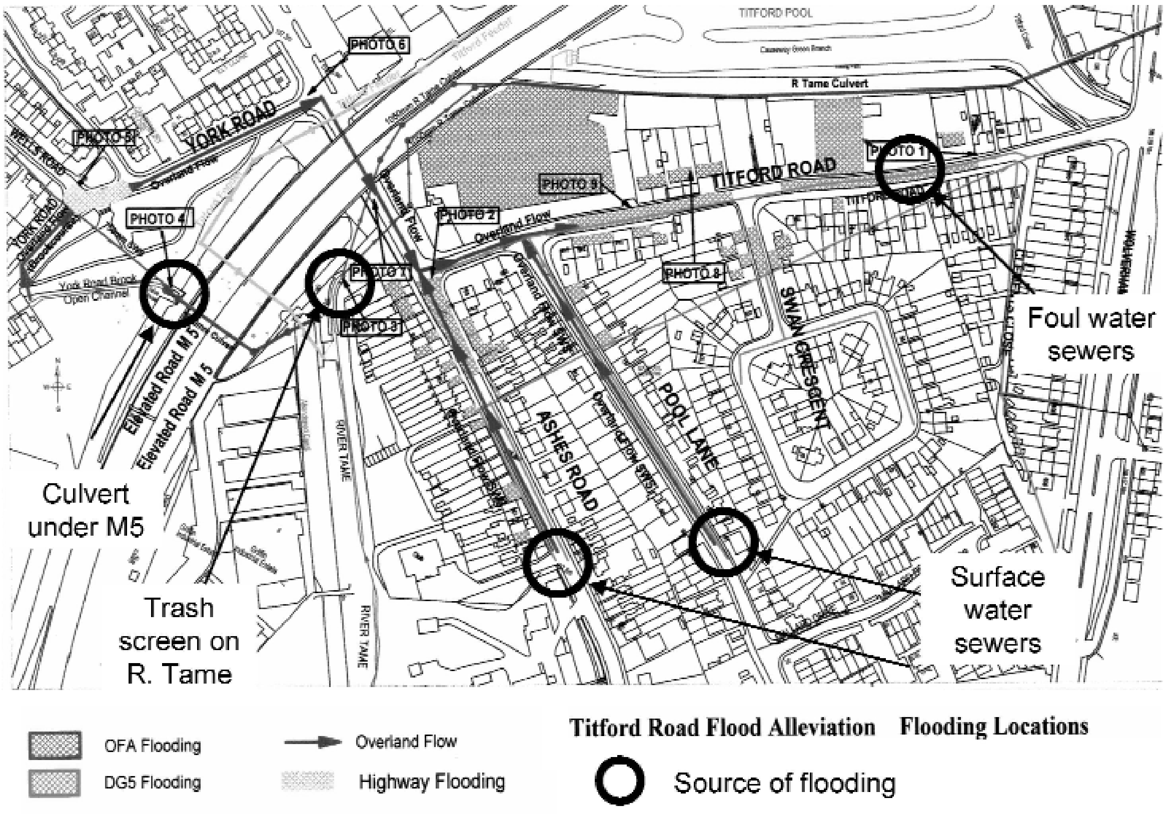


Figure 5.6 *Property location of flooding in a Birmingham suburb attributed to a number of sources (courtesy Seven Trent Water)*

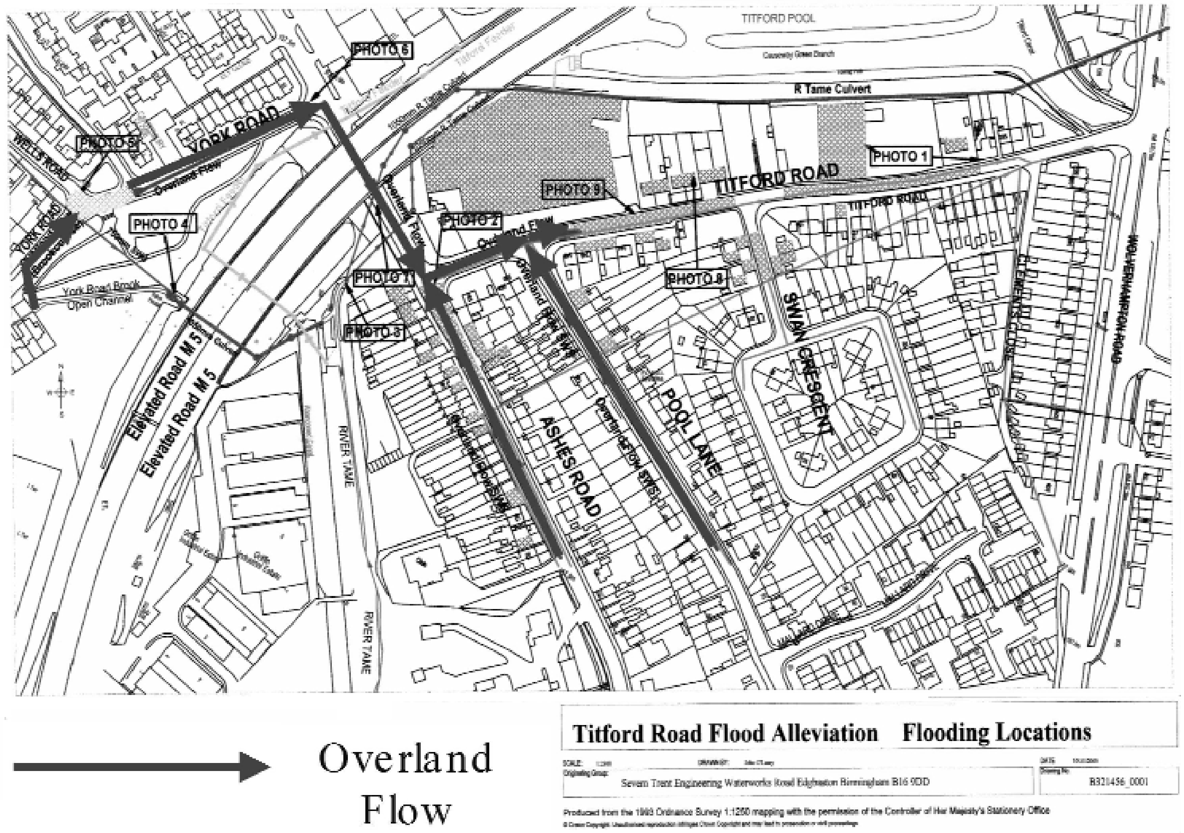


Figure 5.7 *Plan showing overland flow paths for the flooding shown in Figure 5.6 (courtesy Seven Trent Water)*

5.5 Ownership and legal rights

Traditionally as new sewerage is completed, a sewerage undertaker adopts it. Ownership then transfers to the undertaker who is subsequently responsibly for maintenance and periodic capital renewal, in order to assure the required level of service. The situation is more complex with above ground conveyance and storage systems for extreme events. There is no statutory requirement for a sewerage undertaker to adopt since they are designed only to drain extreme events and may be construed as being beyond the definition of "effectual" drainage. Moreover, in many cases the drainage function will not be the primary function of the facility (see Chapter 11). A good example is where a highway is being used for the conveyance of exceedance flows.

It is possible that the sewerage undertaker or other responsible drainage authority will not be the owner of the facilities delivering effective flood control for extreme events. The drainage authority's responsibilities will have to be secured through a license arrangement with the owner, entered into by voluntary agreement. For new developments it would be possible to secure licence agreements by making it a condition in the planning permission. However in retrofit scenarios securing such agreements will be more difficult. No model agreements exist for this purpose, but the model agreements for SUDS may form a useful starting point (Shaffer *et al*, 2004). The license may include certain duties that the owner might be responsible for, such as routine maintenance, and a responsible stakeholder may need to pay a commuted sum, and/or indemnify the owner from liability in him exercising these duties.

5.6 Education – the public as stakeholders

During the planning process, the general public will have very limited involvement. However it is the general public who have to deal with the consequence of exceedance and drainage failure as a result of flooding. Educating them to understand a new strategic approach to managing exceedance is very important. The key areas that should be addressed through an education programme are:

- the minor system cannot convey all flows
- flooding is 'acceptable' if controlled and managed
- it is not sustainable and especially cost effective to design sewers to convey flows with a return period greater than one in 30 years (current standard in *Sewers for adoption 5th edition*)
- explain the concept of return periods and probability with sewerage system design, and the impact of climate change
- excess flows that do occur can be controlled and managed
- flows can be conveyed using a variety of above ground conveyance channels including roads
- during periods of heavy rainfall it is advisable to not travel by foot or car along or in urban flood pathways
- identifying areas for temporary storage will help prevent other areas being flooded.

If these points are addressed, the general public may have a greater understanding of the challenges that exist and how they can be managed.

5.7 Flood warning

The EA have a flood warning process that includes 'monitoring weather, river and coastal conditions, forecasting river and sea levels, disseminating flood warnings, and influencing those at risk to take effective action to prepare for and respond to flood warnings' (Murphy, 2003). It is possible for the EA to perform this function as it is one body with national coverage and the drivers for fluvial flooding are slower than those for pluvial.

It is unlikely that even if a national body existed representing the sewerage system, that flooding in areas related to exceedance could be forecast in time to then issue a flood warning. However, this guide promotes the use of above ground conveyance routes and storage areas to control above ground flows. These need to be adequately signed to warn users that during extreme events, access to the areas should be avoided or even restricted. This is particularly important for storage areas where higher flow depths may be experienced than in conveyance channels. A gradual transition of the build up of flows, rather than a sudden increase, is also important, and will act as a warning to the general public.

5.8 Stakeholder collaboration

The achievement of good exceedance design will be achieved through good stakeholder interaction and dialogue. This should include stakeholders with drainage interests, planners, developers, local interest groups and homeowners. However the exact make up will depend on the location as well as whether it is an existing or new development. The group of stakeholders should be established at the start of the project with the objectives of each stakeholder and the boundaries of operation and their responsibilities clearly set out.

Stakeholder interaction could be enhanced through the setting up of flood liaison and advice groups (FLAGS) that are encouraged in PAN 69 (Scottish Executive Development Department, 2004). In Scotland their purpose is to share knowledge and act in the interest of private and public stakeholders. FLAGS generally have an overseeing role, can cover a wide catchment area and may meet several times per year. FLAGS could be used in a co-ordination role over a wide area and help in the development of good relationships between stakeholders.

6 Runoff from natural catchments

6.1 Introduction

Urban drainage has evolved over the centuries, but in recent times it has become clear that the runoff characteristics of urban areas, both in terms of flow and water quality impacts, can have an undesirable effect on the receiving environment. When natural surfaces are paved, the volume and rate of runoff increases. More sediments and other pollutants are mobilised and transported into the downstream system.

If the impact of urban drainage is to be minimised in the future, then it will need to mimic natural drainage processes much more closely than at present. To achieve this, engineers and practitioners will first need to understand how natural areas drain. From this understanding, criteria for urban drainage can be developed to provide the most appropriate system for a particular site or development:

- understanding rural or greenfield runoff is also important
- water levels in any natural watercourse that runs through a site can be estimated
- runoff that might enter a site from a rural hinterland area can be determined and designed for
- development of a site requires stormwater management controls that relate to greenfield runoff rates.

There are a variety of tools available for the prediction of both rainfall runoff volume and peak flow rate from rural catchments. It should be recognised that the accuracy of these predictions is limited particularly if their use is extrapolated (very steep to very flat catchments, or small to large catchments). This means that the approach to their use should be linked to the purpose for which the estimate is required.

This chapter provides:

- an overview of natural drainage processes
- an introduction to rainfall and the resulting runoff characteristics of rural areas
- and a summary of the tools that have been developed to enable the characteristics of rural runoff to be estimated.

6.2 Natural drainage processes

Various characteristics influence the runoff response from natural catchments. This includes the physical characteristics of the catchment such as soil type, size, shape and topography. Other factors include rainfall, groundwater table and the antecedent conditions (the time prior to the storm). The hydrological processes are described in Chapter 7.

6.3 Rainfall

Although this guidance is aimed at addressing the problems associated with extreme rainfall, drainage systems need to be designed to operate for virtually all rainfall conditions. An understanding of rainfall as a whole and not just extreme rainfall, is needed.

To understand a catchment's response to rainfall, it is important to understand the nature of the rainfall that occurs in UK. In the south of England the number of days in which some rainfall takes place is around 130 to 150 days a year while in the north west of England or Scotland it can be between 200 and 250 days. 50 per cent of the time the rainfall depth in a day is less than 3 mm. However the number of days with more than 10 mm is around 30 in the south east of England while in the north west it is only 20 to 25 days. Around once a year, daily rainfall depths in the region of 40 to 50 mm can be expected anywhere in the UK.

Figure 6.1 illustrates the distribution of rainfall depths at three locations across the UK (the South East, Midlands and Scotland). Figure 6.1 also shows that the proportion of days with large rainfall depths is greater in the south compared with the north.

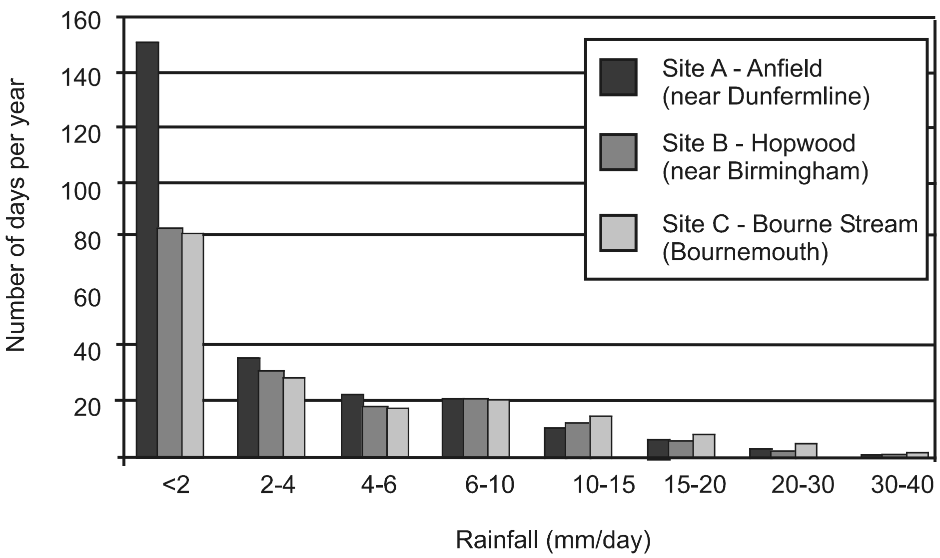


Figure 6.1 *Distribution of rainfall at three locations across the UK*

Figure 6.2 shows the typical rainfall depths variation across the country for extreme events, showing that considerably more rainfall occurs in the north west over 12 hours than in the south east.

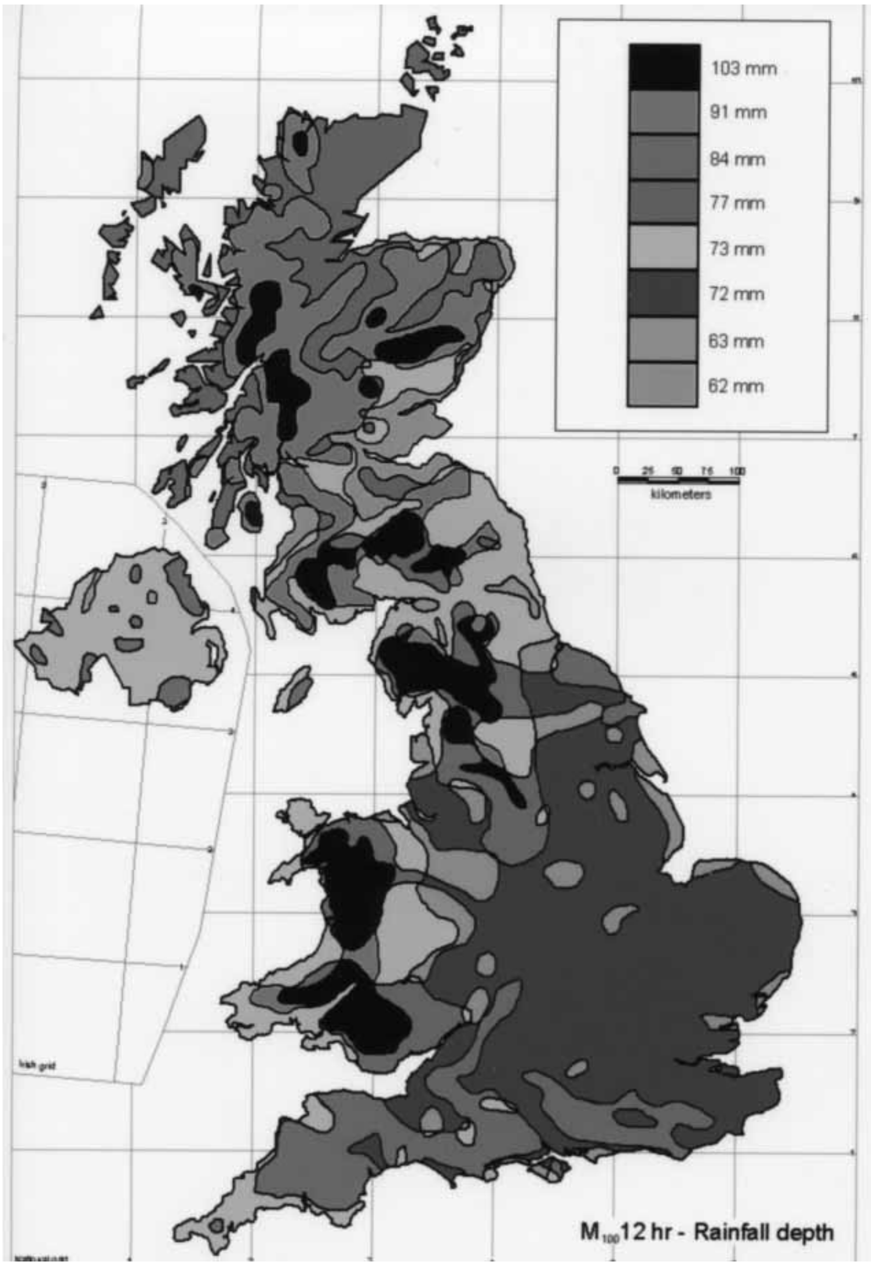


Figure 6.2 *100 year, 12 hour rainfall depths (courtesy HR Wallingford)*

6.3.1 Spatial rainfall

In simplistic terms there are two main categories of rainfall:

- frontal rainfall which falls as a swathe of rain across a wide area and can last for several hours as it progresses across the country
- thunderstorms that usually occur in humid summer periods. This occurs when a body of moist air rises and falls as intense rainfall. They are usually limited to a few kilometres in extent and the downpour may only last for 10 to 20 minutes.

In reality, rainfall often has elements of both types of processes taking place during an event. Figure 6.3 is an image based on radar and shows the high intensities and limited extent of a thunderstorm over Bracknell at a single point in time.

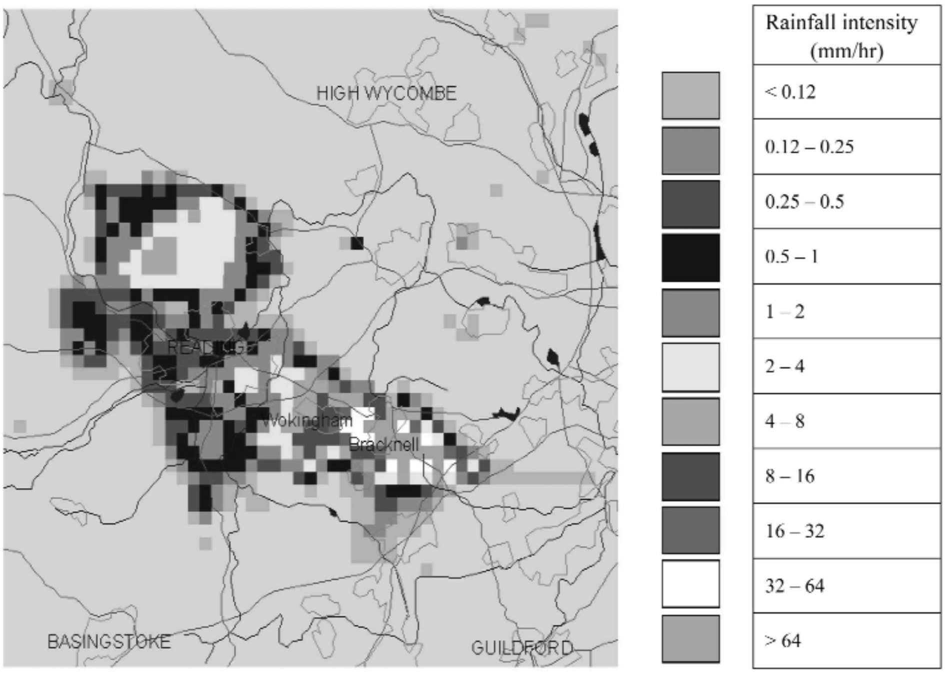


Figure 6.3 *Rainfall radar image Bracknell area 7 May 2000 (1 km resolution) (courtesy Met Office)*

High intensity storms pose a number of problems for drainage engineers. The first is the accurate measurement of these events. Traditional raingauges have difficulty in accurately measuring rainfall that is in excess of 100 mm/hr. It also provides information at only one location and does not provide information about the whole storm. Radar has a number advantages in providing 2D information, but it also has difficulty in providing enough resolution in both time and space to present an accurate record of the event. It also has limited accuracy in its ability to predict the rainfall that actually falls to the ground, as it is measuring cloud and moisture parameters some distance above the ground.

6.3.2 Seasonal rainfall

The distribution of rainfall through the year has an influence on the runoff response from a catchment. More rainfall in the winter months leads to the ground becoming saturated and this can cause significantly more runoff than might have occurred when the ground is dry. However as events in the summer of 2002 (both in Yorkshire and much of Northern Europe) and summer 2004 (Boscastle) demonstrate, it is not uncommon to have wet periods during summer. These can result in serious flooding, because rainfall intensities in summer tend to be greater than those in winter. Normally, however, rainfall in summer does not result in a large runoff response as much of the rain is absorbed into the soil (to make up the soil moisture deficit).

6.4 Rural runoff

6.4.1 Characteristics of rainfall and rural runoff

Runoff characteristics of rural catchments are quite different from those of urban catchments. With rural catchments, runoff depends primarily on the type of rainfall and the nature of the catchment area, with a runoff hydrograph which is heavily attenuated and a volume which is a function of the wetness of the soil. Table 6.1 summarises the rainfall runoff characteristics of rural catchments.

Table 6.1 *Rainfall runoff characteristics for undeveloped areas*

Rainfall	Catchment characteristics	Catchment response
50 to 85 per cent of rainfall events for all catchments.	Depending on soil type.	No perceptible runoff.
Many events in winter. Large summer events.	Response from less permeable catchments. Response in summer events normally limited to steep catchments.	Small amount of runoff.
Extreme events, particularly after an extended wet period.	Floods in winter. Floods in summer normally limited to steeper catchments.	Large amount of runoff.

However the volume of runoff is only part of the issue. The rate of runoff is also important. Rivers have base flows that are a function of rainfall as well as the hydrogeology of the catchment. Rivers respond to rainfall in a matter of hours where sufficient rain occurs. Runoff is rarely visible as overland flow except in the most extreme conditions, and therefore the response to most rainfall is heavily attenuated. The degree of attenuation is a function of the physical characteristics of the catchment.

The concept of frequency is important in the rainfall-runoff relationship. Frequency of occurrence is related to the return period of the event. For a one in 50 year event occurs on average once every 50 years, and will have an annual probability of exceedance of 0.02. ***Probability of exceedance*** *is the statistical probability of a hydrological event (rainfall or flow) of a given magnitude being exceeded in any individual year*.

Return period *is the average time interval between occurrences of a hydrological event (rainfall or flow) of a given or greater magnitude, usually expressed in years.*

The probability of an event occurring or being exceeded during the system design life can be determined using the following equation (Butler and Davies, 2004):

$$P_r = 1 - [1 - (1/T)]^L \qquad (6.1)$$

where:

P_r = probability of event occurring or being exceeded within design life
T = return period
L = design life (years)

Using this equation the annual probability of exceedance has been calculated for a range of return periods (Table 6.2). The table also includes the probability of an event occurring during a 25 or 100 year design life.

Table 6.2 *Probability of an extreme event happening*

Return period (years)	Annual probability of exceedance	Probability of exceedance during design life of...		
		25 years	50 years	100 years
2	50%	≈100%	≈100%	≈100%
5	20%	≈100%	≈100%	≈100%
10	10%	93%	99%	≈100%
30	3.33%	57%	82%	97%
50	2%	40%	64%	87%
100	1%	22%	39%	63%
200	0.50%	12%	22%	39%

It should be noted that the return period of runoff is not the same as the rainfall event for rural catchments. This is due to rainfall taking place on the days immediately prior to the event making the catchment more liable to produce a flood response. The return period of the flood response becomes proportionate to the rainfall as the catchment becomes more developed (impermeable).

The return period of the events of interest to drainage engineers range from one to 200 years. Consideration of more extreme events is sometimes relevant, but usually only when concerned with risk to life rather than flood damage. Consideration of design exceedance and flood management is an open ended concept, but it should be recognised that there comes a point where design will have limited influence in managing the impact of a flood event. The flooding incident at Boscastle in 2004 illustrates that a rainfall event estimated at between 1000 and 5000 years (annual probability of exceedance of up to 0.0002) cannot be managed directly. However it does provide lessons in managing risk and protecting human life.

6.5 Models for estimating rural runoff

There are a number of tools available for estimating runoff from rural catchments, however it should be recognised that rural areas are not homogenous. Therefore the accuracy of these tools will always be limited when not supported by reliable site measurement of runoff rates.

There are three aspects to be determined when estimating runoff from rural areas:

- the volume of runoff
- the peak rate of flow
- the delay and shape of the runoff.

The next section looks at the methods available for estimating the peak flow rate and this is followed by methods for estimating volumes of runoff. An example catchment is then used to illustrate the differences between these methods.

The following methods are all regarded as "current" and no one method is considered to be "right", while the rest are "wrong". However to avoid confusion a preferred method is recommended as good practice. It is important to stress that using more than one method can provide additional information on the decisions to be made. This can be particularly useful in situations where the consequences of "failure" of a drainage structure would be particularly serious. Table 6.3 summarises the various methods that are currently used to determine peak flow rates and volumes of runoff. Appendix 7 provides a more complete summary of each of the methods and their formulae.

Table 6.3 *Summary of the methods that can be used to estimate the peak rate and runoff volume (the preferred method is shown in bold)*

Method – flow rate	Comment
The Rational Method. Peak flow prediction.	Requires an estimate of the runoff coefficient and a calculated time to peak for which there are a number of empirical formulae. Rarely used.
The Transport Road and Research Laboratory (TRRL) Method. (Young and Prudhoe, 1973). Peak flow prediction.	Correlated against catchment characteristics. All parameters can be derived easily from catchment characteristics. Limited data used for deriving the formula, but considered useful for clay type catchments.
Flood Studies Report (FSR), original formula – (NERC, 1975). Peak flow prediction.	Correlation method for all catchments. All parameters can easily be derived from catchment characteristics. Extensive data set used for deriving the formula. Future efforts to improve on it for small catchments may not improve on it greatly.
FSSR 6 – Flood prediction for small catchments (IOH, 1978). Peak flow prediction.	Correlation method for small catchments. FSR data set for catchment 50 ha to 25 km² used. All parameters can be derived fairly easily from catchment characteristics. Although focused at small catchments, not a big improvement on the standard FSR equation.
Poots & Cochrane – (1979). Peak flow prediction.	Similar to FSSR 6. Produced for small catchments from FSR data. Easy-to-use method.
Report 345 / MAFF Report 5 – (ADAS, 1980). Peak flow prediction.	Correlation method for small catchments. Small data set for catchments up to 30 ha. Aimed at defining agricultural land drainage. All parameters can be derived fairly easily from catchment characteristics.
The SCS Method – (1985-1993). Peak flow prediction.	An empirical method well used in USA. Requires the assessment of a curve number based on vegetation and soil type. Takes account of increasing soil saturation. Rarely used in UK.
Institute of Hydrology Report No. 124 – (IOH, 1994). **Peak flow prediction.**	**Correlation method for small catchments.** **FSR and ADAS data set for catchments up to 25 km².** **All parameters can be derived easily from catchment characteristics.** **No catchment "slope" function.**
Flood Estimation Handbook (FEH) – (IOH, 1999). Peak flow prediction (and hydrograph prediction).	Digitally based method with digital catchments pre-defined. Various methods for predicting peak flow as well as volume of runoff. Data set for small catchments not significantly different to FSR data set. Requires expert use. Requires the FEH software to provide certain catchment parameters.
FSSR 16 – (IOH, 1985). Volume of runoff.	Correlation formula. The final version of the FSR approach to estimating volume of runoff. Simple method, but only applicable to extreme rainfall. Applicable to all catchments.
FEH – (IOH, 1999). Volume of runoff.	Correlation formula. Simple to use, allows for catchment wetness and general use for all rainfall.

Where an estimate of peak flow is required to provide a reasonable basis for producing design criteria for stormwater management of a site, it is recommended that IOH Report 124 is used as the preferred method (good practice), with linear interpolation for flows from areas smaller than 50 ha.

Where an estimate of peak flow is required to provide for sizing of a culvert or other structure where "failure" may have damage implications, appropriate consideration should be given by supplementing the result from IOH Report 124 with some of the other methods. An example of this is shown in Box 6.1. The selection of the other methods depends to some extent on the catchment being considered.

Where a catchment has a significant urban area, the fraction of the catchment that is developed should not be greater than 15 per cent to safely apply these formulae. Where this exists it is advised that a more detailed assessment of the catchment is carried out. This may include explicit modelling of both the rural and urban components.

The prediction of rural runoff volume is becoming more important in setting criteria for stormwater management. For extreme events the FSSR 16 method is the simplest tool to use. Due to the nature of the formula an approximation of the percentage runoff to the standard percentage runoff (SPR) value of the soil type can be made. For small events and time series rainfall analysis the FEH model should be used. This requires the use of a spreadsheet to assess the increasing CWI value and resulting runoff using the hourly net rainfall through the event.

Box 6.1 ***Example of calculating the peak runoff***

The following example is a real catchment in the UK where the assessment of peak flow had to be determined.

Catchment characteristics

The parameters that are needed for the various formulae have been measured or calculated. These are given in Table 6.4.

Table 6.4 ***Catchment characteristics***

Catchment area (km^2)	0.88
Soil SPR	0.5 (type 5)
M5-2day (mm)	95
M5-1hr (mm)	22
URBAN	0
LAKE	0.01
RSMD	65
STMFRQ	3
ARF	0.99
SAAR (mm)	1489
Slope (S1085)	140

It may be useful to carry out a simple "sanity check" on the results, using a simple Rational Method calculation, before going into detail on the various methods of analysis. The critical duration of the catchment of these characteristics is in the region of around four to six hours. If one assumes a constant rainfall of 10mm/hr taking place for four hours (which is approximately a five to 10 year event for this location) and a percentage runoff of 75 per cent for a catchment of $1km^2$, the flow rate arriving at the culvert would be in the region of $2m^3/s$. A high coefficient has been used to take account of the catchment shape and steep slope and the class five soil type. Therefore the various formulae would produce values of this order of magnitude for a 10 year event.

Table 6.5 provides the growth curve values for the region.

Table 6.5 ***Regional growth curve for the catchment***

Return period	0.2	0.5	1	2	5	
Growth factor	0.57	0.73	0.88	0.93	1.21	
Return period	10	25	50	100	500	1000
Growth factor	1.42	1.71	1.94	2.18	2.86	3.19

(This information is available from FSSR reports 2 and 14 produced by the Institute of Hydrology and is based on factoring QBAR, a measure of the mean annual event, to determine other events of greater magnitude).

Table 6.6 shows the results obtained from the formulae for a range of return periods.

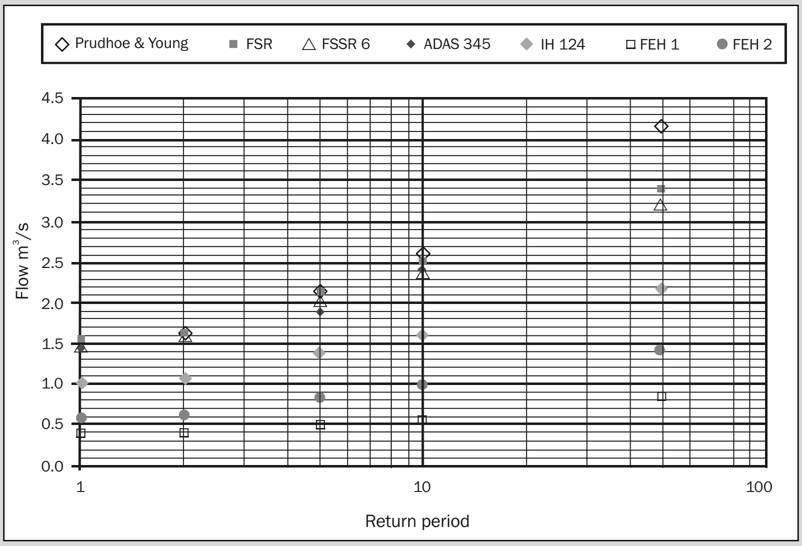
Box 6.1 (cont) Example of calculating the peak runoff

Table 6.6 ***Peak runoff (m^3/s) assessment for the catchment (the preferred method is shown in bold)***

Method	Return period year	Return period 2 year	Return period 5 year	Return period 10 year	Return period 50 year
Prudhoe and Young (TRRL LR565)	–	1.59	2.10	2.58	4.15
Flood studies report (Institute of Hydrology) six parameter method	1.53	1.62	2.11	2.48	3.38
FSSR 6; Flood prediction for small catchments (Institute of Hydrology)	1.45	1.53	1.99	2.34	3.19
ADAS 345 (extract from MAFF report 5) *	1.41	–	1.85	2.38	–
Flood estimation for small catchments (IOH report 124)	0.97	1.03	1.34	1.57	2.14
FEH – Analogy method (CEH, 1999)	0.32	0.34	0.47	0.56	0.82
FEH Statistical method (CEH, 1999)	0.54	0.57	0.78	0.94	1.37

** using the nomograph method*

Figure 6.4 provides this information in a form that allows the variability of the results to be examined. It is clear that for this type of catchment that the FEH methods would appear to be seriously under predicting the peak flow for extreme events (even though they are the result of the most recent work). The implication from this is that although there is a degree of agreement between various methods, the prediction of rural runoff should be treated with caution.



In Figure 6.4 the prediction of the peak flow by IOH Report 124 is relatively low and should not be used exclusively for a situation where an assessment for a receiving pipe size is needed. Conversely as an estimate for assessing peak runoff criteria for setting urban development criteria it provides a conservative estimate. The receiving pipe system, in this case, had a maximum capacity of between 1.25 and 1.5m^3/s

Figure 6.4 ***Predicted peak flows for the catchment for various return periods***

7 Hydrological processes and the effects of urbanisation

7.1 Hydrological processes

7.1.1 Introduction

Rainfall that falls on land can follow a number of different paths depending upon the nature of the rainfall, the soil type, topography and land use (Figure 7.1). Usually only a fraction of the total precipitation produces surface runoff, with the remaining either intercepted before reaching the ground, infiltrated into the ground or lost back into the atmosphere through evaporation and evapo-transpiration.

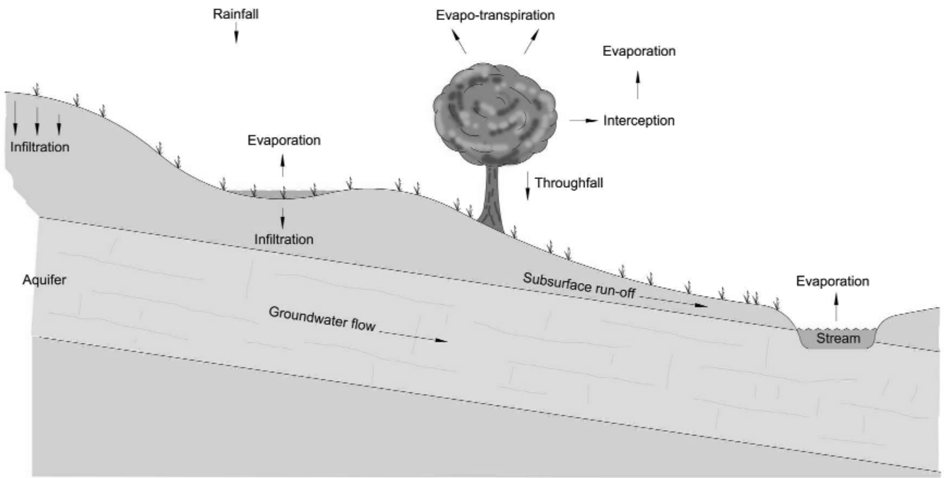


Figure 7.1 *The hydrological process*

7.1.2 Interception

Interception occurs when rain falls and is stored on vegetation, and subsequently evaporates back to the atmosphere through transpiration. The amount of interception depends on the nature of the vegetation, including plant type, form, the density of leaves, branches and stems. Trees often have a high interception capacity compared with grass which is substantially lower. Low rainfall events can be completely intercepted but the proportion of interception for extreme rainfall events may be low.

7.1.3 Depression storage

Rainfall and through-fall below the vegetation cover may be trapped in puddles, ponds, ditches or other depressions in the soil. The water retained in depression storage then evaporates or infiltrates into the soil. The quantity of the depression storage is related to the micro-topography and the properties of the soil surface.

7.1.4 Infiltration

Infiltration is the flow of water into the soil matrix. The infiltration rate depends upon the nature of the soil and the soil moisture. Typically the infiltration rates for clays are low while the infiltration rates for sands and gravels is high. Moist soils normally have higher infiltration rates than dry soils. The water fills the voids between the soil particles and the movement of the water is affected by both surface tension and gravity. Water within the soil may be removed by the root systems of vegetation and subsequently returned to the atmosphere through transpiration. The gravitational force means that the flow of water in the unsaturated zone is predominantly vertical until the water reaches the saturated zone, known as groundwater. Once within the saturated zone the movement of the water tends to be horizontal.

7.1.5 Surface flow

If the rainfall intensity exceeds the infiltration capacity of the soil then water will form a pond on the ground. The infiltration capacity of the soil is affected by the soil moisture, the amount of surface runoff generated by a given rainfall event depends upon the antecedent conditions. During extreme events if the rainfall is intense then a much larger proportion becomes surface runoff than in less extreme events. This is because the rate of rainfall is so great that runoff occurs even though infiltration is still taking place.

The amount of infiltration that can take place may be severely limited if the ground is frozen or baked hard as in the case of clay. The amount of surface runoff may be large.

7.1.6 Evaporation and evapo-transpiration

Water retained on the land surface, either through intercepted rainfall, depression storage or in water bodies, is subject to evaporation. Water absorbed by the root systems of plants returns to the atmosphere through evapo-transpiration. The amount of evaporation and evapo-transpiration depends on the amount of solar radiation on a particular day.

7.2 Runoff

Runoff is that proportion of the rainfall that appears in streams, rivers or drainage systems as a discharge. It can come from surface and subsurface flow and from groundwater. Subsurface runoff arises from water that has percolated through the soil and drained directly to the stream. This subsurface runoff may contribute to stream flow for some time after the rain has ceased. Where a river flows through an aquifer there can be an exchange between the river and the aquifer. The direction of the flow depends upon the relative levels of the water in the aquifer and the stream, and can flow in either direction. Depending upon the size of the aquifer, it may be capable of making a significant contribution to river flow for a long period of time.

The typical stream flow hydrograph for a single-storm event consists of:

- an initial low level of flow representing the base flow in the stream at the start of the storm
- a rising limb resulting from surface runoff that starts a short time after the beginning of the rainfall
- a peak when the discharge reaches a maximum

- a decreasing limb as the rainfall diminishes
- a slow recession produced by groundwater flow after the rainfall ceases.

The recession is typically longer in duration than the rising limb. The shape of the rising limb and the peak discharge depends upon:

- catchment characteristics such as topography, size, shape, stream network development, geology
- initial conditions for example soil moisture and surface retention storage
- characteristics of the rainfall including the spatial and temporal distribution of rainfall or snow melt.

In extreme events a larger proportion of the runoff may be in the form of surface runoff than for less extreme events. This means that a much larger proportion of the rainfall enters the river or drainage system rapidly rather than following slower routes via infiltration. In extreme events, proportionally steeper hydrographs with larger peak discharges may occur.

In discussing flood characteristics a number of parameters are commonly used. The **time of concentration** is defined as the time required for a particle of water to travel from the most distant point in the catchment to the outlet or the point under consideration. The **lag time** can be defined as the time interval between the centroid of effective rain and the peak of the discharge.

The time of concentration is affected by the size of the catchment. Small and steep catchments have shorter times of concentration than large or flatter ones. The time of concentration is important in deciding how a given catchment will respond to a particular rainfall event. For rainstorms of the same probability, the intensity reduces with duration. The intensity that is commonly experienced during a short summer storm of five minutes duration is rarely sustained for periods of hours.

It has been observed that catchments respond differently to storms of different durations. Small catchments respond most to shorter duration rainfall events while larger catchments respond most to longer duration events. There is an interest in estimating the critical storm duration for a given catchment. One of the assumptions of the Rational Method is that a catchment gives the highest peak discharge for a storm with a duration that approximates to the time of concentration of the catchment. If the duration of the rainfall is less than the time of concentration then not all the catchment has started to contribute to the runoff before the rainfall stops. If the duration of the rainfall event is significantly longer than the time of concentration then the average intensity of the rainfall will be less than that of a rainfall event whose duration matches the time of concentration for any given return period. The implication is that for small, steep catchments the highest peak flows are generated by short rainfall events of a few hours while for a large, flat catchment the highest peak flows are generated by extended rainfall events. This analysis assumes that rainfall is stationary relative to the catchment and that the spatial distribution of rainfall is uniform. If the rainfall cell moves relative to the catchment then this may modify the runoff flow rates. If a storm cell tracks down the catchment then the impact of the runoff may be intensified while if it tracks up a catchment then the impact may be diminished. The spatial distribution of rainfall is rarely uniform over the whole catchment and this will also modify the runoff.

7.3 Stream network and channel morphology

The density and pattern of stream channels depends upon the rainfall, local geology and topography, and vegetation. On very steep hillsides the channels are typically straight with long distances between confluences. In flatter areas the distance between stream confluences may be much shorter.

The channel size, shape and pattern are linked to the discharge, the nature of the sediment and the slope of the river valley. The primary determinant on channel size and shape is the discharge. It is a matter of simple observation that channels carrying a small flow are normally small with a small width to depth ratio while channels carrying a large discharge are typically large with a large width to depth ratio. The channel size is affected by the sequence of flows that the channel has to carry. During a major flood, bed and bank erosion may take place modifying the shape of the channel. This may be followed by a period of recovery during which the river returns back towards its pre-flood morphology.

River plan forms are commonly classified as being:

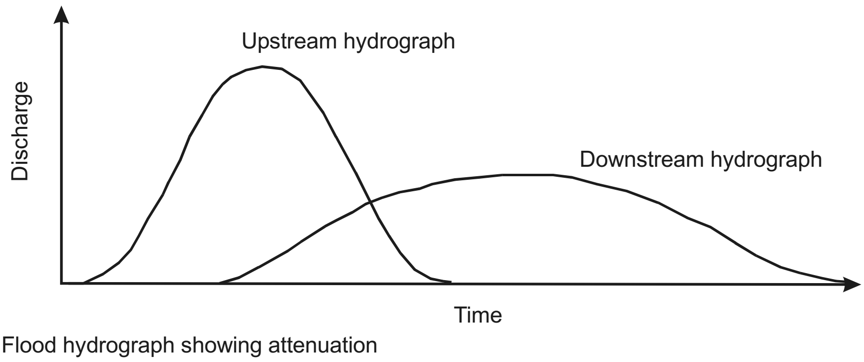
- straight
- meandered
- multi-thread or braided.

In practice there is really a continuum of different plan forms and these divisions are to some extent arbitrary. For example, it is difficult to be precise about how sinuous a channel has to be before it is classified as meandering rather than straight. The plan form is not arbitrary and is linked to the discharge in the river, the sediment and the valley slope.

7.4 Floods in natural catchments

Floods occur when the discharge exceeds the bank-full discharge and the water spreads onto the floodplain. The rarity of the event can be expressed in terms of a probability (the annual probability of occurrence) or in terms of a return period (the n year flood is that flood that is equalled or exceeded on average once every n years).

Typically the shape of a flood hydrograph varies as it passes down a river system. There are different physical processes at work and the consequence is that the response is different in different catchments. In rivers with wide, extensive floodplains water is stored on the floodplain during the rising limb of the hydrograph and then returns to the river during the falling limb. This leads to a reduction or attenuation of the peak discharge and to an extension in the length of the hydrograph. The speed of the flood wave down a river depends upon the magnitude of the discharge with larger discharges having a faster wave speed. This means that a larger discharge can "catch up" with a smaller discharge. This leads to a steepening of the wave front. In flat catchments with extensive floodplains, the process of flood attenuation normally dominates, but in steep catchments with limited or no floodplains, the flood wave may become steeper as it travels down the catchment (Figure 7.2). The process becomes more complicated as the effect of tributaries is included into the analysis. In situations where this occurs, flash flooding can be very dangerous. However these situations are rarely encountered in UK currently, but may become more common place if storage from SUDS storage schemes is not modelled correctly.



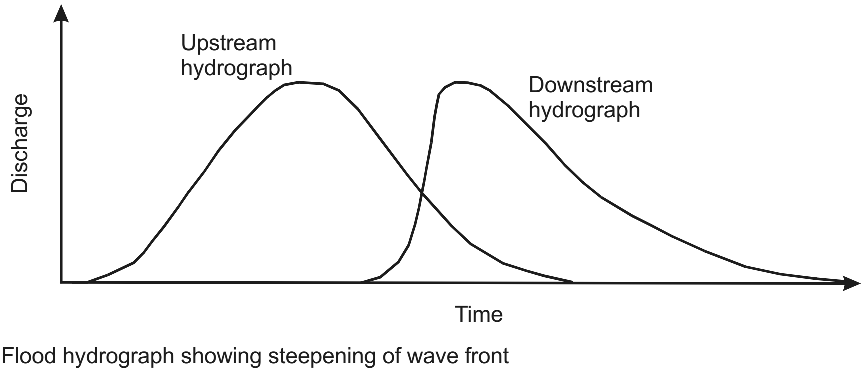


Figure 7.2 *Example of flood wave attenuation*

7.5 The effects of urbanisation

Since before Roman times urban areas have had to address drainage problems due to the construction of impervious surfaces and the refuse associated with man's activities. Through the 19th and 20th centuries, developed countries have succeeded in providing the infrastructure in cities for draining urban areas of both foul and surface water runoff. This has resulted in a dramatic reduction in health problems and minimal impact on the urban community for quite extreme rainfall.

However the solution has only been focused on meeting man's immediate needs. The standard method for draining foul and surface water from built-up areas has been through underground pipe systems, which replicates the conveyance aspects of streams and rivers. These systems are designed to prevent flooding locally by conveying the water away as quickly as possibly. This often becomes a major problem if the system capacity is exceeded.

Urban catchments have a response to rainfall which is very different to rural runoff. The response to rainfall is effectively instantaneous if traditional pipe systems are used and it can differ depending on the level of service provided, the type of drainage designed, and the proportion of rural and urban area in the catchment.

Urbanisation removes the natural processes of absorption and saturation of soil. The soil wetting process, evapo-transpiration and depression storage effects are not replicated and this results in very rapid and unattenuated runoff with large flows occurring in the downstream system and receiving rivers during heavy rainfall.

In addition, mankind, as a by-product of the highly intensive lifestyle and preference for living in high density communities, generates a large number of pollutants. These include sediments, oils, grits, metals, fertilisers, pesticides, animal wastes, salts, pathogens and general litter and can cause extensive environmental damage. These pollutants are collectively termed "urban diffuse pollution". Rainwater mobilises many of these pollutants which are then washed in to rivers and groundwater.

7.6 Urban runoff behaviour

Figure 7.3 shows the effect of three rainfall events (one, 30 and 100 year) and the runoff response obtained from a traditional pipe based system using a simple model. This shows an instant response for the one year event, with the thirty year event showing the surcharge effect with a slightly attenuated peak flow, and the 100 year event being more heavily attenuated as shown by the flat topped hydrograph. The flat topped hydrograph indicates the constraining effects of the capacity of the piped drainage system and the potential for surface flooding (note that the modelling approach does not fully allow for overland flow effects and produces a slightly more "peaky" response).

However it can be seen that the timing of all of them is quick compared with the fourth hydrograph which approximates to the runoff from a rural area of the same size for the one year event. The relationship between rainfall and runoff is very direct and proportional for urban runoff. Greenfield runoff for more extreme events with wet antecedent conditions can result in more rapid runoff, and the figure provides a useful illustration of the difference.

The response of the greenfield site is not only more delayed, but also much reduced in volume of runoff. This shows that a mixed catchment of urban and rural runoff is likely to result in a twin peaked hydrograph. The relative size of each of the peak flows is dependent mainly on the proportion of urbanisation of the catchment, but also on the size of the event and the soil type. This is illustrated by the two schematics in Figure 7.4. In both cases the catchment is assumed to be 10 per cent urbanised, and shows hydrographs of both the rural runoff and paved runoff. The third hydrograph is the combined flow. In the first figure the soil type is assumed to be relatively pervious (sandy), while in the second the rural runoff is assumed to be from a clay catchment.

Figure 7.4 illustrates that once development is in the region of 10 per cent that the response from the paved area may dominate the runoff in terms of flow rate even though the volume of runoff from the pervious area is larger over a period of time.

These figures have used the simple assumption that a paved surface generates 100 per cent runoff for the rainfall that falls on it. Similarly it has been assumed that the percentage runoff from the soils are 30 per cent and 47 per cent respectively. For more information on runoff from natural catchments, see Chapter 6.

The assumption that 100 per cent runoff is generated from paved surfaces is very conservative and much work has been carried out to develop other runoff models which provide a more accurate prediction of runoff. There are several urban runoff models Chapter 8 provides an overview of these models, together with their advantages and limitations.

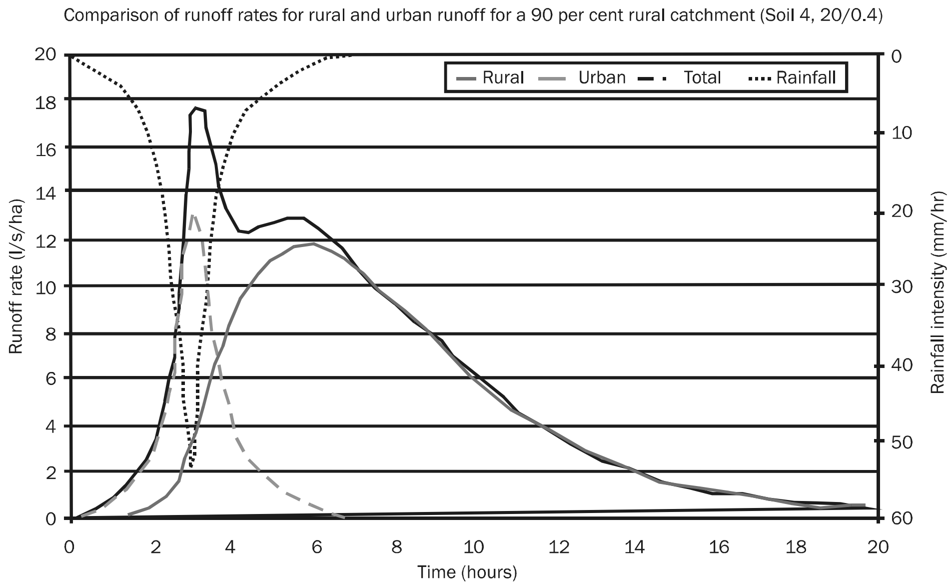


Figure 7.3 *The response of a traditional pipe system to one, 30 and 100 year events compared with a greenfield response*

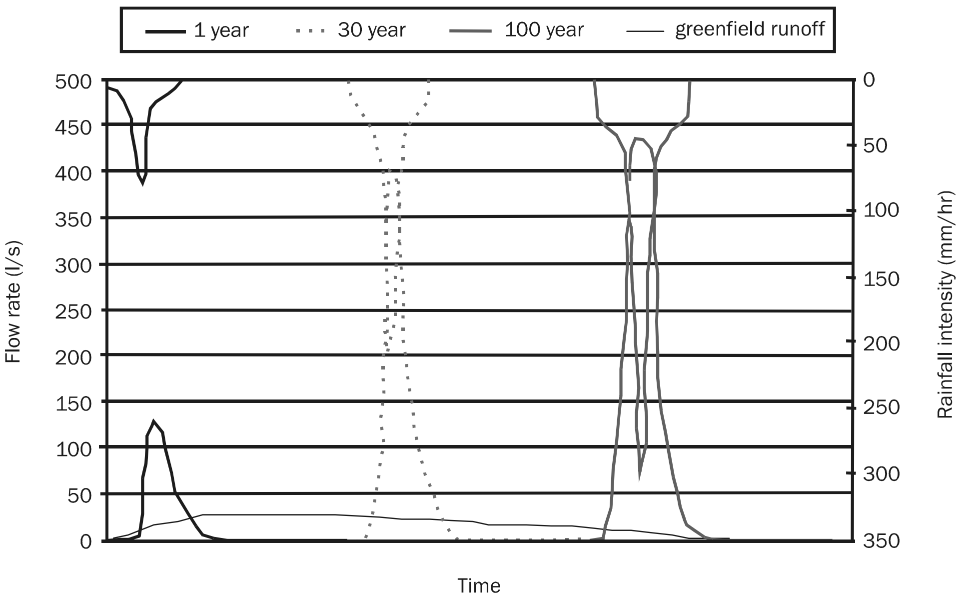


Figure 7.4 *The response from two catchments (soil types 2 and 4) with a development proportion of 10 per cent, for the 100 year event*

7.7 Urbanisation and flooding

The effects of urbanisation can extend beyond the urbanised area. The traditional approach of only considering the local needs of drainage has now been superseded by requirements to account for the wider context. Urbanisation is known to have a significant influence on the flows in river systems to which they discharge. This can have an impact on the distance further downstream, causing rural flooding and the flooding of other urban areas (Figure 7.5). This can be a particular problem where the urban input results in a speeding up of runoff flow in the river resulting in a steepening of the flood hydrograph, as shown in Figure 7.2.

Figure 7.5 *Urban area suffering from river flooding*

Planning Policy Guidance Note 25 and its equivalent in Wales (Tan 15), Scotland (SPP 7) and Northern Ireland (PPS 15), requires the flood impact of urbanisation be mitigated. The Water Framework Directive (Directive 2000/60/EC) may also place greater restraints on urban development. The Environment Agency normally requires that flood impact of new developments are assessed for the 100 year return period event.

The potential for increasing flood risk in receiving systems can be mitigated by the use of more sustainable approaches to drainage, such as SUDS systems. For more details on SUDS systems and best practice drainage design see CIRIA reports C521, (Martin *et al,* 2000a), C522 (Martin *et al,* 2000b), C523 (Martin *et al,* 2001) and C609 (Wilson *et al,* 2004) and CIRIA's SUDS website <www.ciria.org/sud>.

8 Runoff from urban catchments

8.1 Urban runoff models

Urban runoff models are used to estimate the rainfall runoff proportion that is drained by the stormwater system. Historically this was calculated using the Rational Method and it was assumed that in urban environments 100 per cent runoff took place from paved surface and no runoff occurred from pervious areas. In the production of the Wallingford Procedure in 1981, a statistical runoff model was produced, and was referred to as the UK runoff model. This was produced by the Institute of Hydrology and was calibrated against measured flow data. This equation was subsequently modified in 1991. To distinguish it from the first equation this was referred to as the new UK runoff model. For more information, reference should be made to the CIRIA publication X108 *Drainage for development sites – a guide* (Kellagher, 2004).

Many drainage engineers will have experience in using these methods in computer simulation models of drainage networks. The simulation process usually involves the verification of the data and modelling approach using short-term flow surveys (based on the field measurement of sewer flow and rainfall), and subsequent use in simulating the effects of "design" events. How these models work is often misunderstood. The following sections have been developed in order to give the reader a better understanding of the runoff estimation models that are embedded in modern software tools, and enable them to select and apply them more reliably when simulating the effects of extreme events.

The process of constructing a model and the consideration of exceedance is shown in Figure 8.1.

8.1.1 The constant (old UK) runoff model

The first Wallingford Procedure runoff model is now usually referred to as "the constant runoff model" (or old UK runoff model). This runoff model assumes losses are constant throughout a rainfall event (runoff does not increase as the catchment gets wetter) and is defined by the Equation 8.1:

$$PR = 0.829\ PIMP + 25.0SOIL + 0.078\ UCWI - 20.7 \tag{8.1}$$

where:

PR	= percentage runoff
PIMP	= percentage impermeability (contributing)
SOIL	= an index of the water holding capacity of the soil
UCWI	= urban catchment wetness index

Values of SOIL range from 0.15 to 0.5 which are obtained from the winter rainfall acceptance potential (WRAP) map which was produced as part of the Wallingford Procedure. It is also available with the FSR manual. The values are a function of the runoff characteristics of each soil type found in the catchment. There are only five categories of SOIL with soil types being grouped together on the basis of their runoff characteristics.

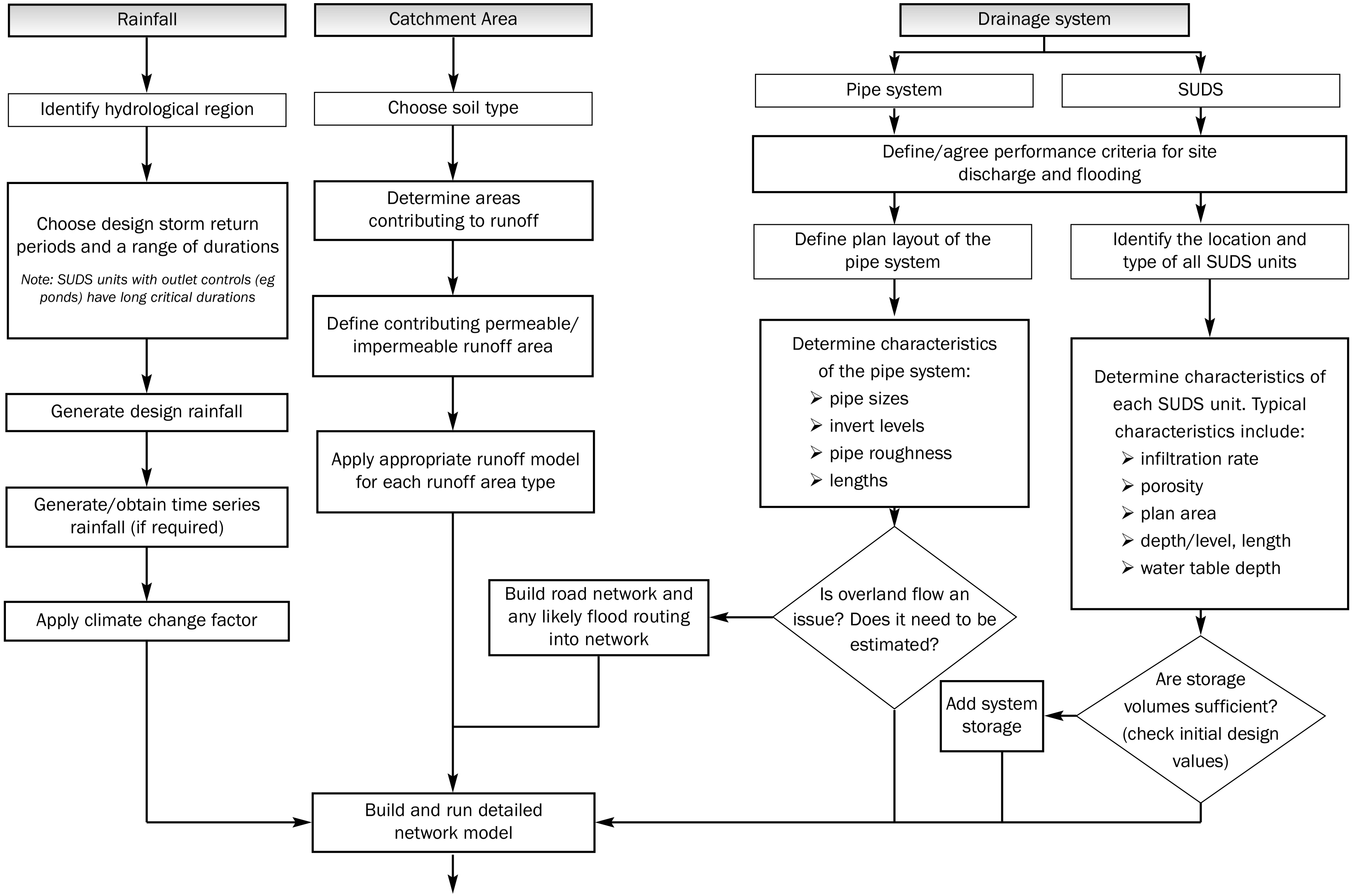


Figure 8.1 *Process of constructing a drainage model*

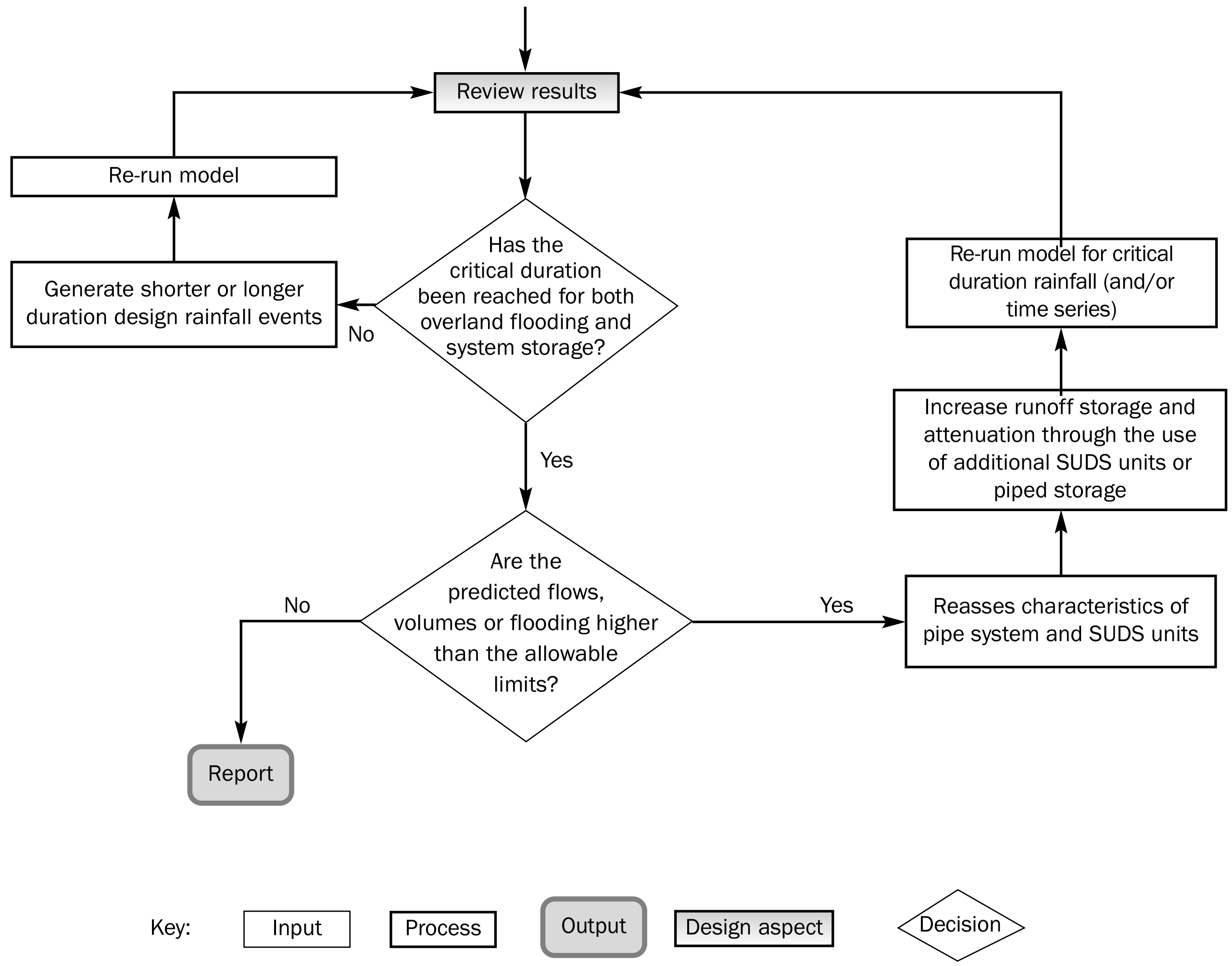


Figure 8.1 *Process of constructing a drainage model (cont'd)*

Values of UCWI, a composite of two antecedent wetness parameters, is given in Equation 8.2:

$$UCWI = 125 + 8\ API\ 5 - SMD \tag{8.2}$$

where:

API5	= five day antecedent precipitation index (mm)
SMD	= soil moisture deficit

It should be noted that the equation is entirely statistical and that it is heavily influenced by the value of PIMP, as expected. If the catchment wetness term and soil term are ignored, then the percentage runoff for a fully (100 per cent) paved catchment would be approximately 62 per cent ($0.829 \times 100 - 20.7$).

Although the model has a constant proportion of runoff, it recognises that the average runoff volume during an event is a function of the catchment wetness. A value of around 100 is used when carrying out network design (a function of season and annual average rainfall depth), but specific values of UCWI are calculated for real events. UCWI can range from 0 to over 300 mm.

There is an assumed distribution of the runoff from the paved and pervious surfaces which assumes that the pervious runoff has a runoff factor which is 10 per cent of the paved rainfall-runoff factor. If the percentage runoff from the paved area is calculated to be 68 per cent, the value for the pervious surface would be 6.8 per cent. The model is applied to urban areas without taking into account topography.

However it became clear that where the equation was applied to areas where the contributing paved proportion was less than 30 per cent, unrealistically low values for PR (percentage runoff) were being produced. Rules were created to try and avoid this problem, but it was recognised that the constant runoff approach, irrespective of storm depth and certain other limitations, needed to be improved. Figure 8.2 illustrates the low values of PR which are produced by the equation. An override was later introduced to ensure that where the PR value for the paved surface could not be less than 20 per cent. However it is obvious that when this rule comes into effect that the equation is being used beyond its limits.

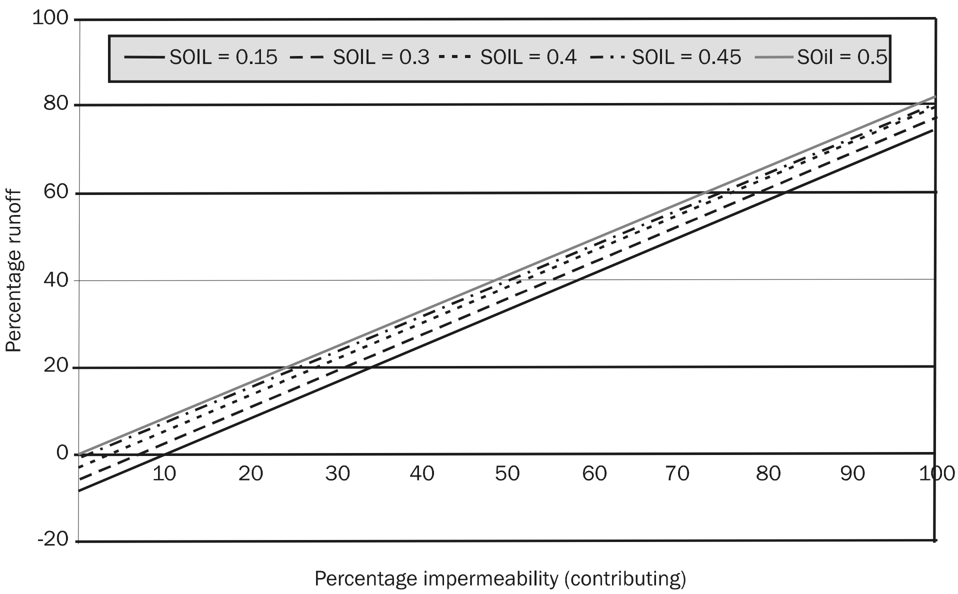


Figure 8.2 *PR as a function of SOIL and PIMP (old UK PR equation)*

8.1.2 The variable (new UK) runoff model

The variable runoff model was developed jointly by HR Wallingford, the Water Research Centre and the Institute of Hydrology with support from North West Water PLC. It was designed as a replacement to the original Wallingford Procedure runoff model. The model was developed in 1990 it is still often referred to as the new UK runoff model. Although it overcomes some of the problems of the original equation, it introduces certain potential problems if used inappropriately. The new equation was designed to overcome some of the difficulties experienced in practical application of the constant runoff model, namely:

- the old equation uses the calculated value of PR as a constant throughout a rainfall event irrespective of increasing catchment wetness. For long duration storms this can have a significant impact on the design of storage systems
- problems have been encountered in applying the original PR equation to catchments with partially separate systems and to catchments with low PIMP, particularly for low SOIL values
- the assumptions of the flow split between paved and pervious runoff is clearly inappropriate for catchments with a significant rural component to the runoff.

The new model is of the form:

$$PR = IF*PIMP = (100 - IF*PIMP)* \frac{NAPI}{PF} \qquad (8.3)$$

where:

IF = effective impervious area factor
PF = moisture depth parameter (mm)
NAPI = 30 day antecedent precipitation index

This equation divides PR into two elements. First, the impervious area runoff is obtained by using an effective contributing area factor, IF (impermeability factor). After initial depression storage losses on impervious surfaces, remaining losses are given as a constant fraction of rainfall volume. Recommended values of IF are indicated in Table 8.1. One of the principal features and possible drawbacks of this equation is that engineers have to choose an appropriate value.

Table 8.1 Recommended values of IF

Surface condition	Effective impervious area factor, IF
Normal roads	0.60
Roofs and well drained roads	0.80
Very high quality roads	1.0

The losses on pervious surfaces and also non-effective impervious areas are represented by the second term of the equation. The first part of this term represents the total percentage of the catchment occupied by pervious and non-effective impervious areas. The losses from this area are dependent on the function NAPI/PF. NAPI (new antecedent precipitation index) is defined as a 30-day API with evapo-transpiration and initial losses subtracted from rainfall. As for API5, API30 is given by Equation 8.4:

$$API_{30} = \sum_{n=1,30} P_{-n}\, C_p^{n-0.5} \tag{8.4}$$

The decay constant value CP of the API has been made dependent on the soil type to reflect the faster reduction of soil moisture on lighter soils. The relationship between CP and soil type is shown in Table 8.2.

Table 8.2 **Relationship between soil type and CP**

Soil Type	C_P
1	0.1
2	0.5
3	0.7
4	0.9
5	(0.99)*

* the value of 0.99 is an uncalibrated value and should be treated with caution

The moisture depth parameter, PF (porosity factor), was calibrated using the data described above. A value of 200 mm was obtained (which compares well with the available water capacity of soils with grass vegetation). It is dangerous to modify this value without careful consideration of the consequences.

Figure 8.3 illustrates the dangers of applying the constant runoff model for low values of PIMP. The analysis assumes a 1 ha paved surface with variable amounts of pervious area to provide a range of PIMP from one to 100 per cent paved catchment area. This is compared with the variable runoff model. Two sets of comparisons are shown, the first for a rainfall depth of 10 mm and a second for an event of 80 mm, both occurring over a 12 hour period. Soil types 1 and 4 have both been plotted. The figure illustrates a number of useful points:

- the total runoff from the use of the constant runoff model does not vary greatly with an increase in permeable area for high values of PIMP. The reduction in runoff volume is not significant until PIMP reduces to 10 per cent for soil type 4, but starts becoming significant at 35 per cent for soil type 1
- the variable runoff model provides very similar results to the constant PR model for small storms down to PIMP values of 20 to 30 per cent, depending on soil type. However for large events the pervious catchment component starts to have a much greater effect as PIMP reduces below 50 per cent
- the variable runoff model will generate large runoff volumes if large pervious catchments are included in the model, although this will still be less than predicted from the same area by the constant runoff model.

The implication of points two and three are that the verification of a system using a small storm with the variable runoff model might appear to be good even though PIMP may be around 25 per cent, but when an extreme event is applied it will probably predict an unreasonable amount of runoff from the pervious area being served by the drainage system. Care is needed in terms of the contributing pervious area with this model, where it is relatively insensitive in the constant runoff model.

The value of NAPI is affected by the decay function for the different soil types and this is illustrated in Figure 8.4. This figure is based on a 50 year 18 hour event of 78 mm with a PIMP catchment value of 50 per cent. In addition to showing the difference in the percentage contribution from permeable areas, it also shows that the maximum contribution from permeable surfaces will not rise much above 30 per cent for large events on fairly impermeable soil types. It can be seen from the total runoff curves that the effective paved area contribution is 60 per cent and that this is a constant contribution.

As with the constant runoff model, it is worth briefly examining the likely maximum value of percentage runoff from this model. For IF of 0.75, this means that for a catchment which is 100 per cent paved, 25 per cent of the area is non-effective paved surface and treated as part of the permeable catchment. If a storm depth of 80 mm is assumed and no decay in the runoff function is allowed for, then the total runoff that takes place is: 75% + 25% × (80/200) which is 85%

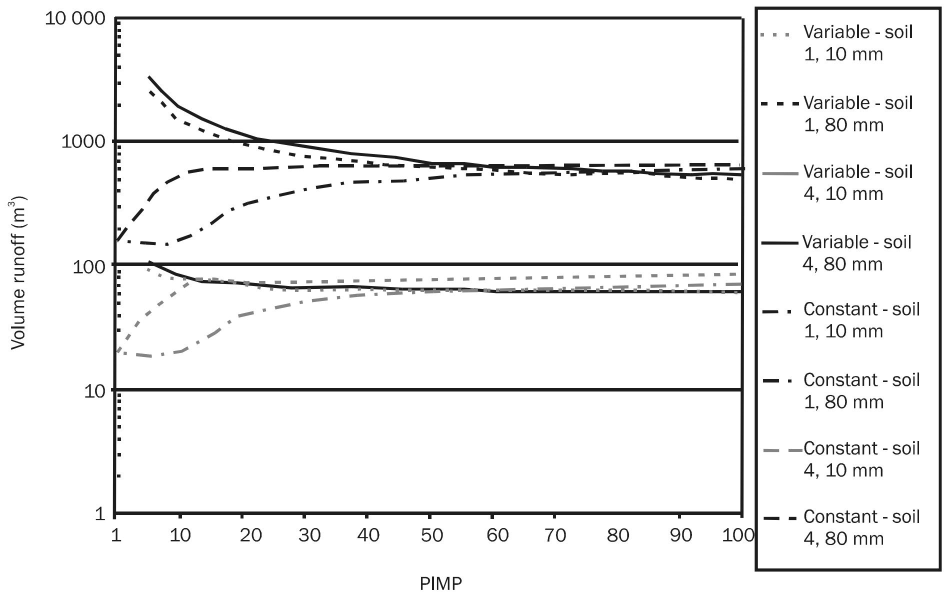


Figure 8.3 *Volume of runoff from a 1 ha paved catchment with a variable amount of pervious area - (constant and variable runoff models)*

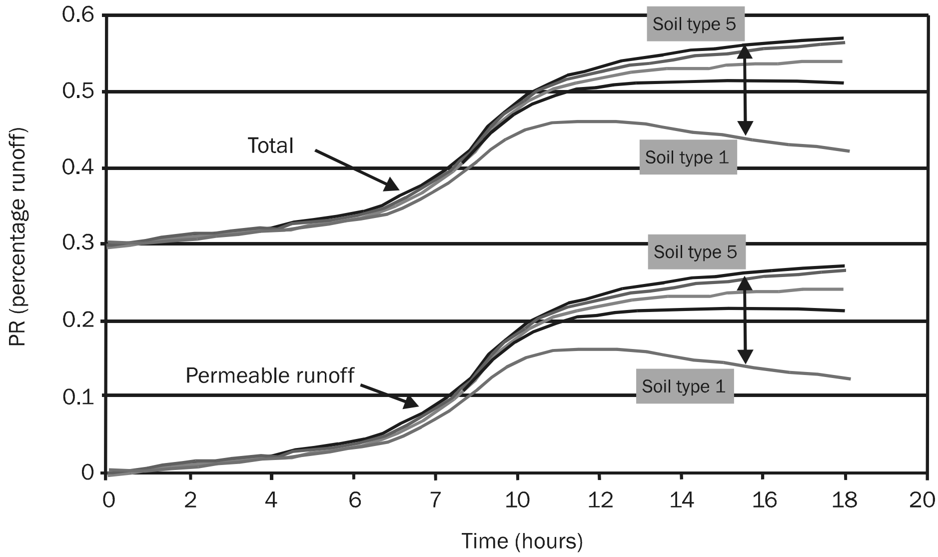


Figure 8.4 *Percentage runoff as a function of rainfall depth using the variable runoff model*

8.1.3 The fixed percentage runoff model

The discussion on the Wallingford Procedure runoff models has highlighted the risks in using these equations even though they are based on calibrated data. It is important to make use of a simpler approach where reasonable approximations can be made without losing too much accuracy.

The fixed percentage runoff model is simple to use and, in appropriate circumstances, can be used without undue concern over its accuracy. The assumption used in *Sewers for adoption 5th edition* that 100 per cent runoff takes place from all paved surfaces and none from pervious areas is generally conservative for urban areas and realistic for short duration and lower intensity events. Variations on this theme exist elsewhere. In Belgium 80 per cent runoff from paved areas and zero from pervious surfaces is commonly used for verifying models, which can involve the use of small storm events. Where an allowance for pervious runoff is believed to be needed, values up to 30 per cent are used depending on the circumstance.

A health check for using this simple approach can be made by comparing the fixed percentage runoff used by *Sewers for adoption 5th edition* with the variable runoff model. Figure 8.5 illustrates the difference in runoff proportion for various levels of PIMP, for soil types 1 and 4, and for four storm events. The results show that, for fairly high urban densities, the assumptions of 100 per cent and 0 per cent runoff, for paved and pervious areas respectively, are cautious but provide a reasonable estimate for predicting volumes of runoff. For large storms some provision for pervious runoff becomes more important as PIMP reduces. However, this assumption is not advisable for PIMP levels which are less than 50 per cent.

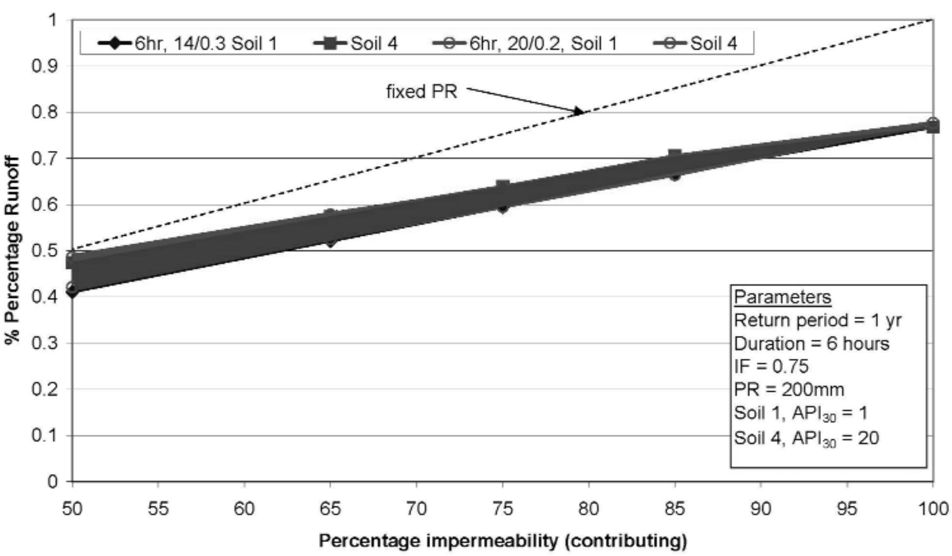


Figure 8.5a *Comparison of percentage runoff between the variable runoff model and fixed percentage model used in* Sewers for adoption 5th edition *(2001) for a one year return period storm of six hour duration*

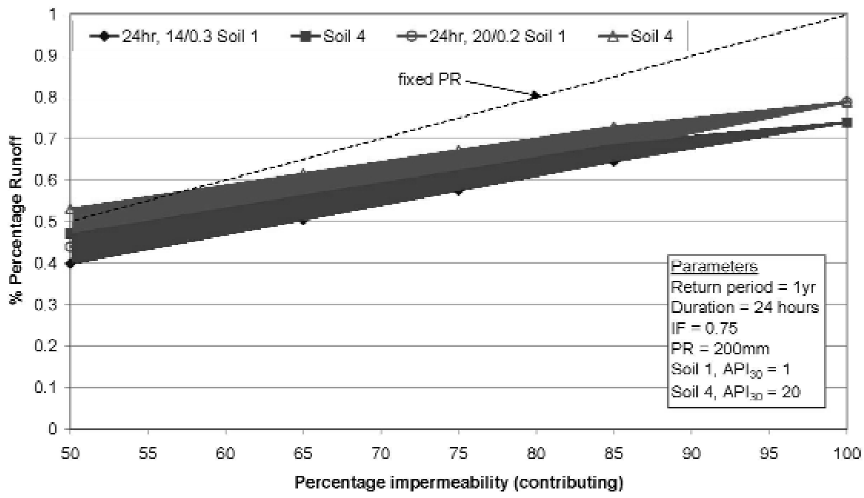


Figure 8.5b *Comparison of percentage runoff between the variable runoff model and fixed percentage model used in* Sewers for adoption 5th edition *(2001) for a one year return period storm of 24 hour duration*

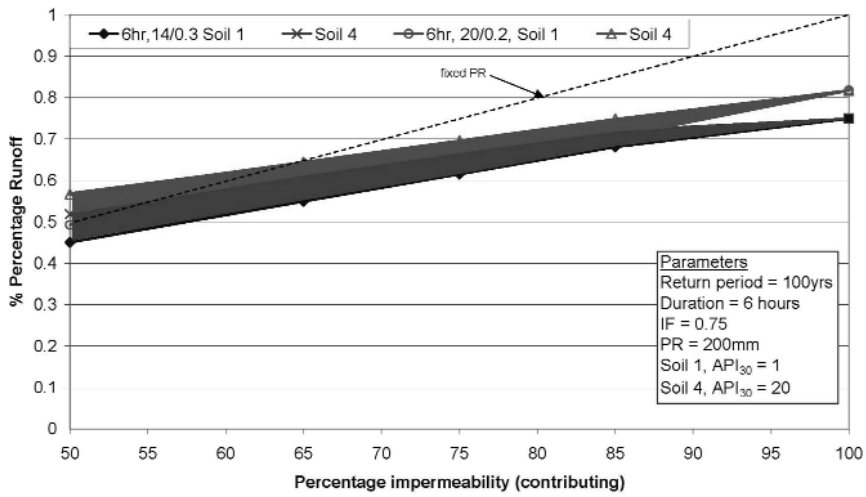


Figure 8.5c *Comparison of percentage runoff between the variable runoff model and fixed percentage model used in* Sewers for adoption 5th edition *(2001) for a 100 year return period storm of six hour duration*

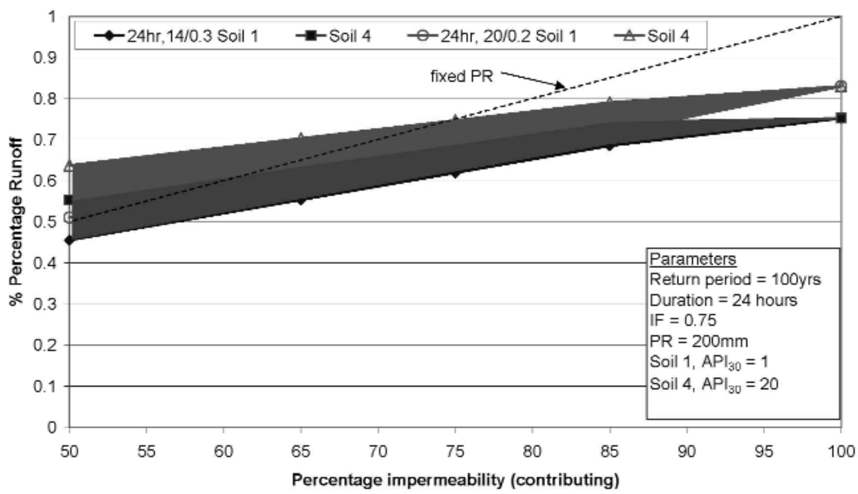


Figure 8.5d *Comparison of percentage runoff between the variable runoff model and fixed percentage model used in* Sewers for adoption 5th edition *(2001) for a 100 year return period storm of 24 hour duration*

8.2 Estimation of the difference between greenfield and development runoff

Section 6.4 and Appendix 6 on rural runoff provide the tools to enable an assessment to be made of the difference between a site before and after development. This is important if design criteria are based upon the measurement of these differences. The difference in runoff rates has been shown to be significantly so the rate of runoff from a paved area needs to be physically controlled to achieve flow rates equivalent to greenfield runoff. It is not necessary to determine the unrestrained runoff rate for the paved area. However the same is not true of runoff volume.

Examination of the formula in FSSR 16 demonstrates that an approximation to SPR is valid for extreme events for the respective soil type. Unfortunately this means that soil types 4 or 5 which have SPR values around 50 per cent can be larger than the predicted runoff from the catchment after development. Examination of Figure 8.3 shows that for a catchment with a PIMP of 50 per cent, it is possible to get a percentage runoff predicted from a large event of around the same amount. This intuitively is incorrect as the calibrated models of the Wallingford Procedure suggests that paved areas have a runoff proportion in the region of at least 60 per cent and up to 85 per cent. It would seem inappropriate that less runoff is predicted for the development scenario than pre-development.

Careful consideration of the built environment provides some support for this result. Developments involve the construction of not only buildings and roads, but also involve the re-contouring of the area. Runoff from pervious areas may not be possible from back gardens or low areas created by the development process. Therefore where the pre-development situation reasonably assumed that the whole catchment contributed to the runoff, this may no longer be the case once development has taken place. Figure 8.6 shows a good recent housing development plan where much of the green area is to be found in back gardens behind terraced houses.

It is important to be able to differentiate between areas that can and cannot contribute runoff to the drainage system. Similarly, when using infiltration systems, the proportion of the paved area that drains to a watercourse may only be a proportion of the total hard surface area. These issues together with the rather complex and sometimes awkward issues of using the appropriate runoff equation, means that a simple and easy to use approach to quantify the difference between runoff volumes before and after development is desirable.

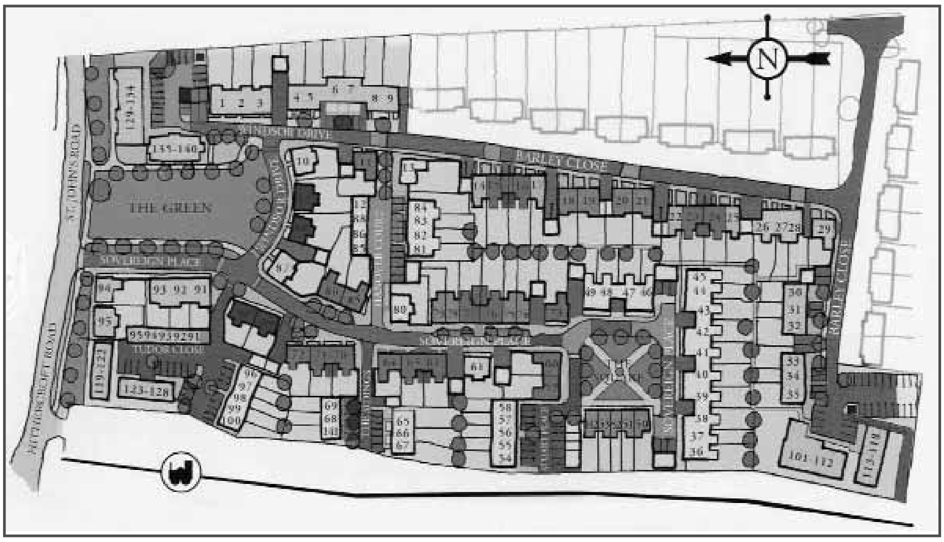


Figure 8.6 *Development plan of a modern high-density housing estate (courtesy Linden Homes Chiltern Limited)*

Equation 8.5 has been derived to achieve this. It assumes that extreme events are being considered, as SPR is only a recent assumption for the soil runoff factor for this situation. It also assumes that only 80 per cent runoff occurs from paved areas as it is generally recognised that 100 per cent is a cautious assumption which aims to take into account some pervious runoff element.

$$Vol_{xs} = 10.RD.A \left[\frac{PIMP}{100} (\alpha 0.8) + \left(1 - \frac{PIMP}{100}\right) (\beta.SOIL) - SOIL \right] \qquad (8.5)$$

where:

Vol_{XS}	=	the extra runoff volume (m^3) of development runoff over greenfield runoff
RD	=	the rainfall depth for the 100 year, six hour event (mm)
PIMP	=	the impermeable area as a percentage of the total area (values from 0 to 100)
A	=	the area of the site (ha)
SOIL	=	the “SPR” value for the relevant FSR soil type
α	=	the proportion of paved area draining to the network or directly to the river (values from 0 to 1)
β	=	the proportion of pervious area draining to the network or directly to the river (values from 0 to 1)
0.8	=	the runoff factor for contributing paved surfaces

If all the paved area is assumed to drain to the network and all the pervious areas are landscaped not to enter the drainage system or river, this formula simplifies to:

$$Vol_{xs} = 10.RD.A\left(0.8\frac{PIMP}{100} - SOIL\right) \qquad (8.6)$$

But where all pervious areas are assumed to continue to drain to the river or network the formula becomes:

$$\mathrm{Vol}_{xs} = 10.\mathrm{RD.A}\left(0.8\frac{\mathrm{PIMP}}{100} - \frac{\mathrm{PIMP}}{100}.\mathrm{SOIL}\right) \qquad (8.7)$$

Figures 8.7 and 8.8 illustrate the difference in runoff volume for these two extremes (fully disconnected/fully connected pervious surfaces) for the five different soil types for any development density. To obtain a volume all that is required is to multiply the X-axis value by the catchment area and the rainfall depth.

These graphs demonstrate the difference in soil type, the importance of using infiltration to disconnect impermeable areas from the drainage network and the need to be efficient in designing the general landscape to disconnect pervious areas.

This provides a rapid and robust easy-to-use method for assessing the additional volume of runoff generated by any development.

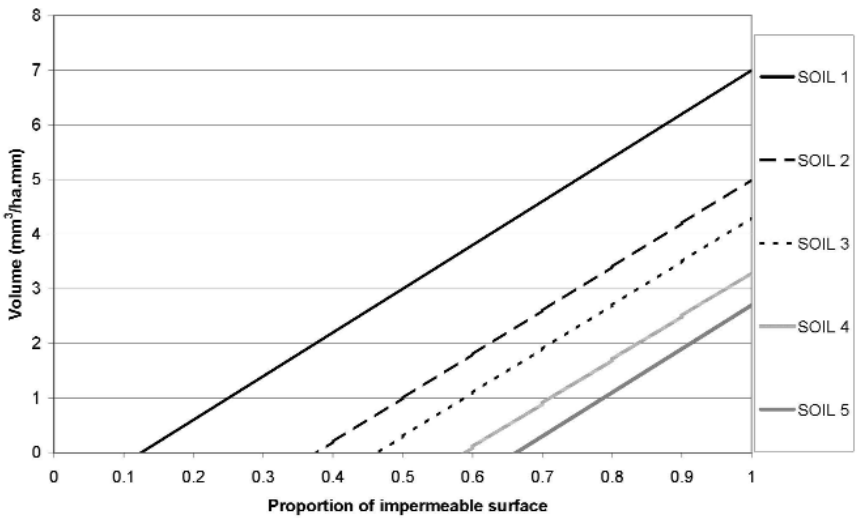


Figure 8.7 *Difference in runoff volume for developments where all pervious areas are assumed not to drain to the drainage network*

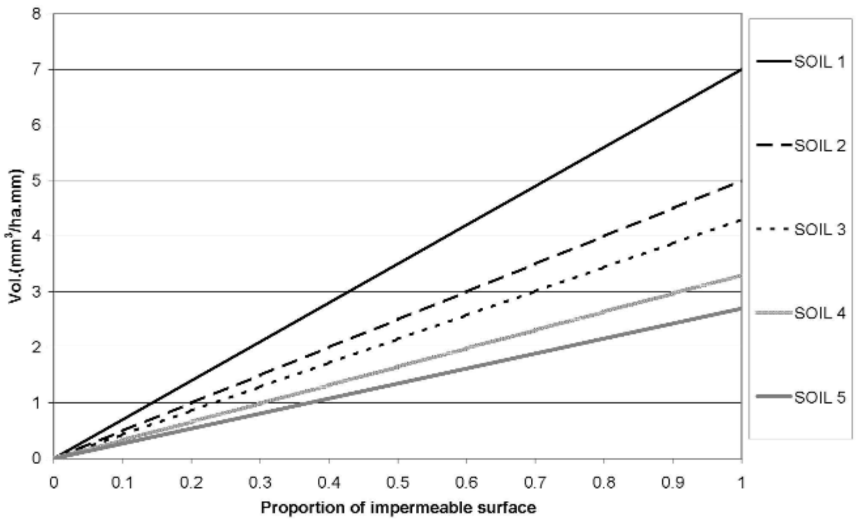


Figure 8.8 *Difference in runoff volume for developments where all pervious areas are assumed to drain to the drainage network*

9 Interaction between major and minor systems

9.1 Principles of interaction

This chapter defines the mechanisms of interaction between the major and minor drainage systems. It identifies when the major system comes into operation and gives guidance as to how the user may determine the flows and volumes conveyed on the surface by the major system, in specific circumstances. Interaction between the minor and major drainage systems is complex. Above ground flow that causes the major system to come into operation is known as the exceedance flow, and may be generated from four sources:

- flooding from manholes and other connections to the minor system resulting from a lack of capacity in the minor drainage system, or blockage, collapse or other service defects
- excess surface runoff that cannot enter the minor system due to the limited capacity of drainage inlets
- surface runoff from permeable areas that have no direct connection to the sewerage network
- flooding caused by high levels in receiving waters preventing sewerage systems from draining effectively.

During an extreme event, exceedance flows will travel on the surface in flood pathways. These may consist of existing roads, paths and natural depressions in the ground. Where they are not specifically designed as surface flood pathways (designed pathways) they are known as default pathways. Such pathways can transfer flow over significant distances so that flooding can occur at locations remote from the point of discharge from the drainage system.

Although sewerage undertakers are required to keep records of sewer flooding incidents, such records are often not sufficiently detailed to enable the cause of the flooding to be reliably determined. When surface flooding is observed it is often very difficult to ascertain the underlying cause by observation alone (Figure 9.1). In some cases flooding can be the result of all four of the causes set out above which may complicate the situation.

The processes governing the various interactions between the major and minor system are complex and require a suitable level of analysis and supporting data if reliable results are to be obtained. Interactions between inputs, processes and outputs are illustrated in Figure 9.2 and described in the subsequent text.

Figure 9.1 *Urban flooding illustrating the difficulty of identifying the precise cause of flooding*

9.1.1 Flooding from manholes and other drainage connections

This condition describes the case where the flow in a piped drainage system becomes such that it exceeds the capacity of that part of the system. This causes flow to back up into manholes and gullies, and flooding can then occur when the level in the manhole or gully rises above ground level. Flooding can also occur where a property, or part of a property such as a cellar, lies below the level of the hydraulic gradient in the drainage system. For this to occur there should be a pathway between the drainage system and the property. This does not have to be an actual pipe connection, as there is plenty of evidence of property flooding where no formal connection to the drainage system exists. Cellared properties are particularly vulnerable to this form of flooding.

Modern computer simulation software can accurately replicate these conditions *provided that the connections between the minor and major systems are modelled*. The volume of flooding on the surface at a particular node (manhole or other point of connection) can be predicted, and a depth of flooding can be deduced if the area of flooding is known.

How such surface flooding may be conveyed above ground by the major system (Figure 2.1), and what the consequential effects might be, has not been assessed historically. This is because of the lack of guidance about how this should be done, and limitations in the available terrain data (see Appendix 1). However, evidence of urban flooding shows that overland flow is very important in determining the risk of flooding of individual property.

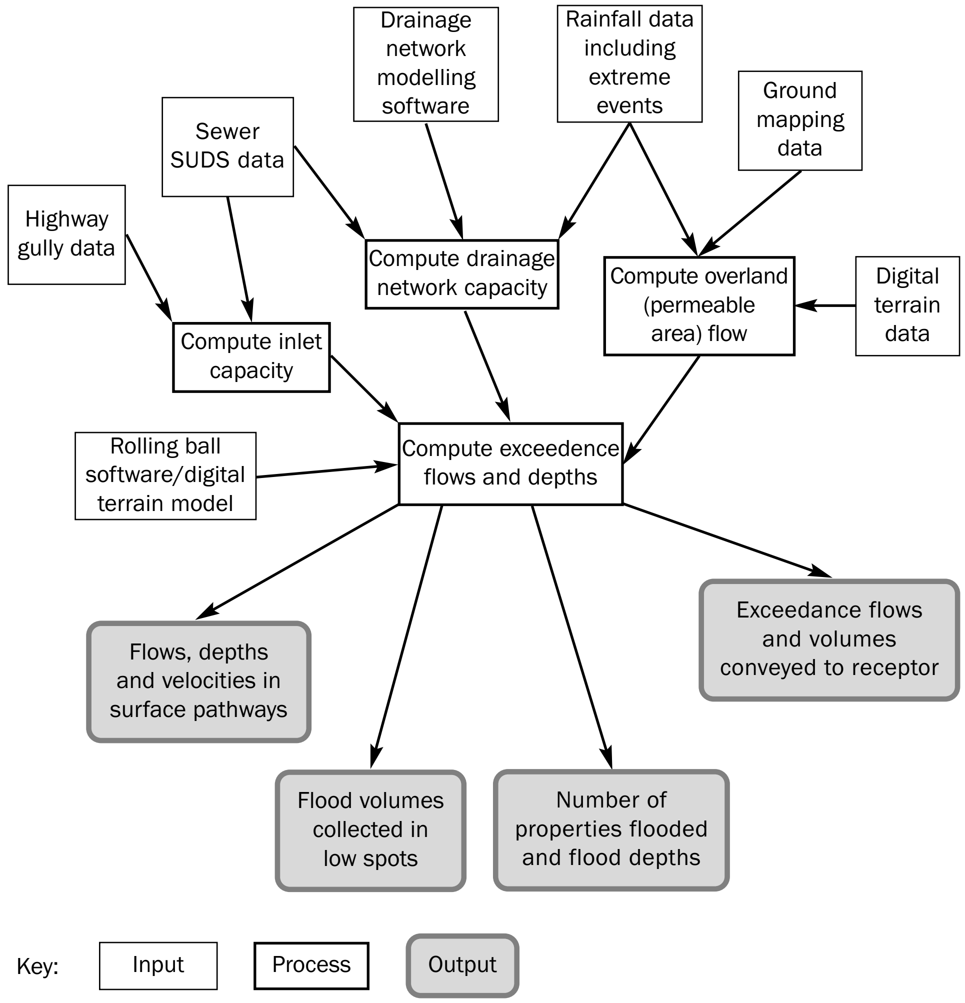


Figure 9.2 *Computing exceedance flows and volumes*

9.1.2 Limitation of inlet capacity

During very intense rainfall events the rate of runoff may be sufficient to exceed the capacity of the drainage inlet. This may be a road gully, yard drain or roof gutter for example. The hydraulics of drainage inlets can be complex, however their capacity can be simplified as shown in Figure 9.3.

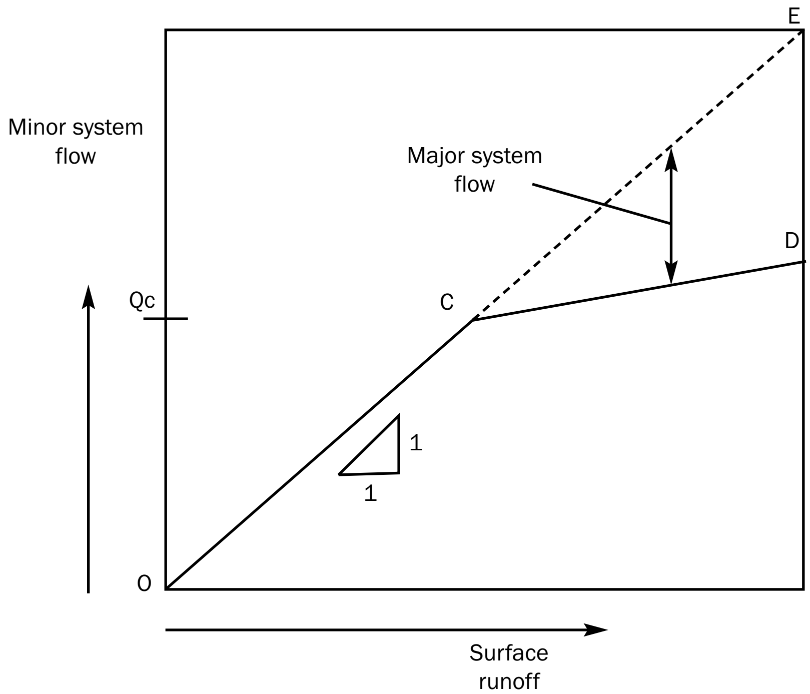


Figure 9.3 *Conceptual model of exceedance due to inlet capacity*

Point C in the figure shows the capacity of the inlet, defined by the flow Qc. Up to this point all the surface runoff is passed into the minor system, as represented by the 1:1 relationship of the line OC. Beyond this point excess flow (exceedance flow) is diverted to the major system. As depths of flow on the surface increase, it is possible that further flow may be forced into the inlet, so that there is a further increase in minor system flow, as indicated by the line CD. The slope of CD will be largely governed by the hydraulic characteristics of the inlet.

The exceedance flow in the major (above ground) system will be the difference between the surface runoff and the inlet flow, as illustrated in the figure. Some inlets such as roof gutters may restrict the inflow such that the line CD is almost horizontal, and the excess flow beyond C is diverted to the major system. Others, such as highway gullies, may allow almost all the additional flow to enter as depths and flows increase, being limited only by the capacity of the connection to the piped system. In practice the transition may not be as abrupt as it is shown in the figure, the part CD may not be linear, and the characteristic may have more than two stages.

The simplified approach in Figure 9.3 has the advantage that it can easily be built up for standard inlet components. Also, by aggregating the effect of sub-components, the overall characteristic of a sub area can be built up. This will be useful in modelling large catchment areas where the model detail does not extend to the head of the drainage system. This is explained further in the case studies in Part C.

When reviewing records of flood events great care should be taken in interpreting evidence of flooding. Photographic and video evidence may at first sight appear to show inflow being restricted by the capacity of the inlet whereas the flow may not be able to enter the piped system because it is already surcharged to ground level.

9.1.3 Surface runoff from pervious area

Runoff from pervious areas adjacent to drained paved areas is known to contribute to drainage flows (Figure 4.3). Some modern software tools explicitly allow for this.

The impact of overland flow from pervious surface runoff is increasingly seen as a major factor in urban flooding. In the subsequent analysis of flooding in Glasgow East, runoff from adjacent pervious areas was shown to contribute up to 34 per cent of the total flood volume (Figure 5.3).

Where significant pervious areas are known to drain onto developed areas, their impact during extreme events should always be assessed. When modelling the effects of extreme events, these contributing areas should be explicitly included in the simulation model with an appropriate runoff model (see Chapter 6).

9.2 Calculating exceedance flow

The interaction between the minor and major drainage systems is complex and accurately assessing exceedance flow can also be complex and sometimes expensive. The degree of resources in calculating exceedance flow should match the needs of a particular project. In some cases a more approximate method can be justified whereas in others (where the risk and/or impact is higher) a detailed analysis will be necessary. This guide recommends a three level approach to calculating exceedance flow, as illustrated in Box 9.1.

The user is encouraged to evaluate which level of study is appropriate in individual circumstances based on the perceived level of flood risk. Further guidance on this is given in Section 10.3. The levels of analysis may be applied progressively, and to different parts of a drainage area as required. The overall process is illustrated in the flowchart in Figure 9.4.

Box 9.1 Levels of detail for calculating exceedance flow

Level 1 study

Design of new drainage in small developments (say up to 50-100 dwellings).

Simple dendritic drainage layout without complex ancillaries. This level is not suitable for analysing existing systems.

Minor drainage system designed in conventional manner using Rational Method or suitable software tool.

Exceedance flow calculated on basis of minor drainage system at capacity ie all surface runoff from the extreme event conveyed by the major system. Surface conveyance replicated by simplified dendritic layout of open channels. Peak flows and volumes calculated using Rational Method or suitable software package. Contributing pervious areas included as equivalent paved areas when using the Rational Method.

Level 2 study

Analysis/design of medium to large (>200 properties say) drainage systems with some degree of complexity.

Minor drainage system analysed/designed using sewer network modelling software capable of replicating surcharging and surface flooding. For existing areas the sewer network model should be verified by comparison with short term flow survey data or historic flooding data.

Surface conveyance represented by pathways identified by site inspection, or in conjunction with a “rolling ball” model using digital terrain data. Low spots where floodwater may pond identified by site inspection or in conjunction with a digital terrain model. Contributing pervious areas modelled explicitly. No allowance for inlet capacity included.

Properties may be grouped into areas for the assessment of risk.

Level 3 study

Analysis/design of large and complex drainage systems.

Minor drainage system analysed/designed using sewer network modelling software capable of replicating surcharging and surface flooding, including backwater effects. For existing systems the sewer network model should be verified by comparison with short term flow survey data and historic flooding data.

Surface conveyance replicated by explicit modelling of known surface flood pathways with full interaction between major and minor networks. Subsequent flooding analysed using “rolling ball” model. Contributing pervious areas explicitly modelled. Allowance made for inlet capacity on an area or individual basis.

Property flood risk assessed on an individual property basis, including explicit allowance for floor levels and cellars.

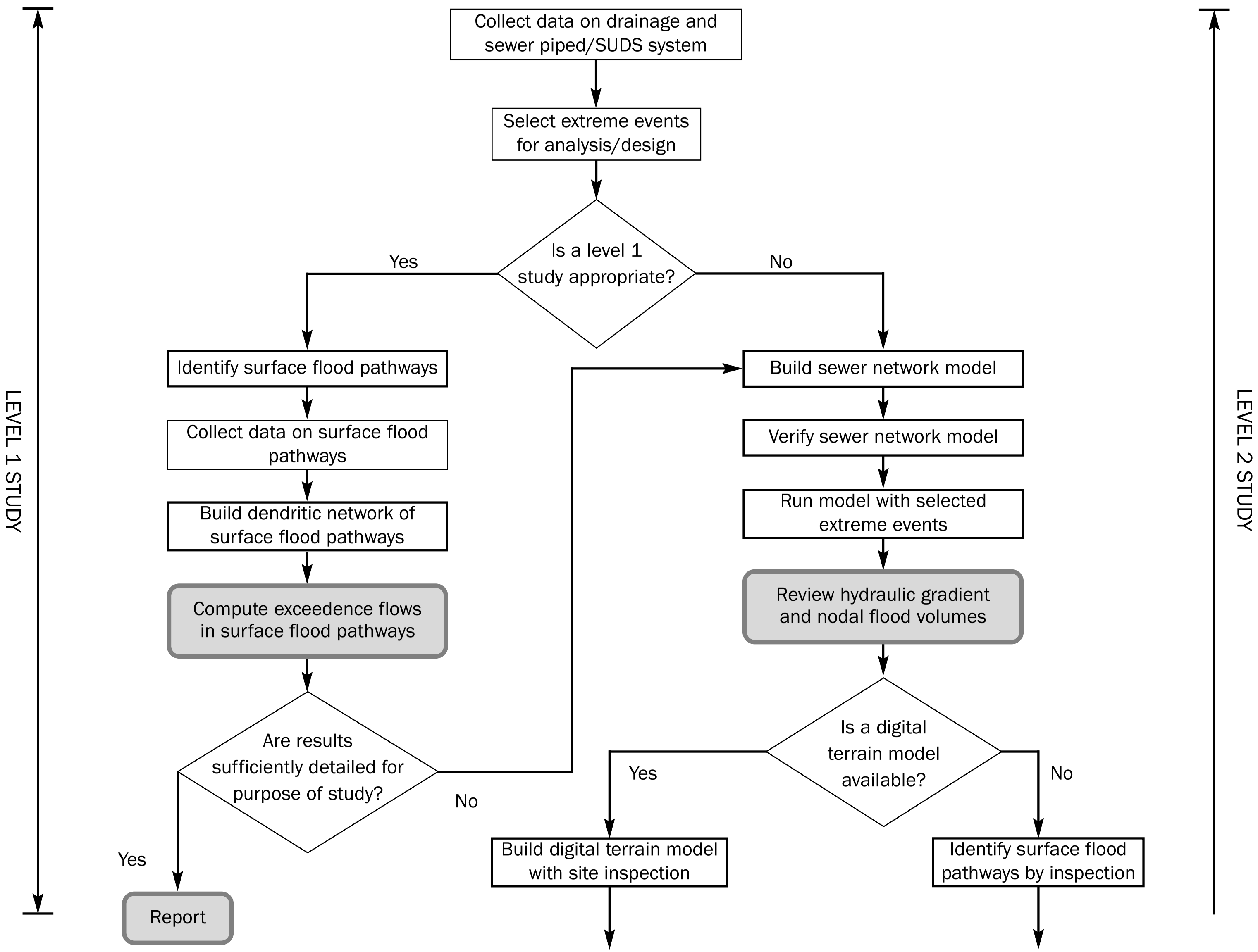


Figure 9.4 *Flowchart for computing exceedance flows and volumes*

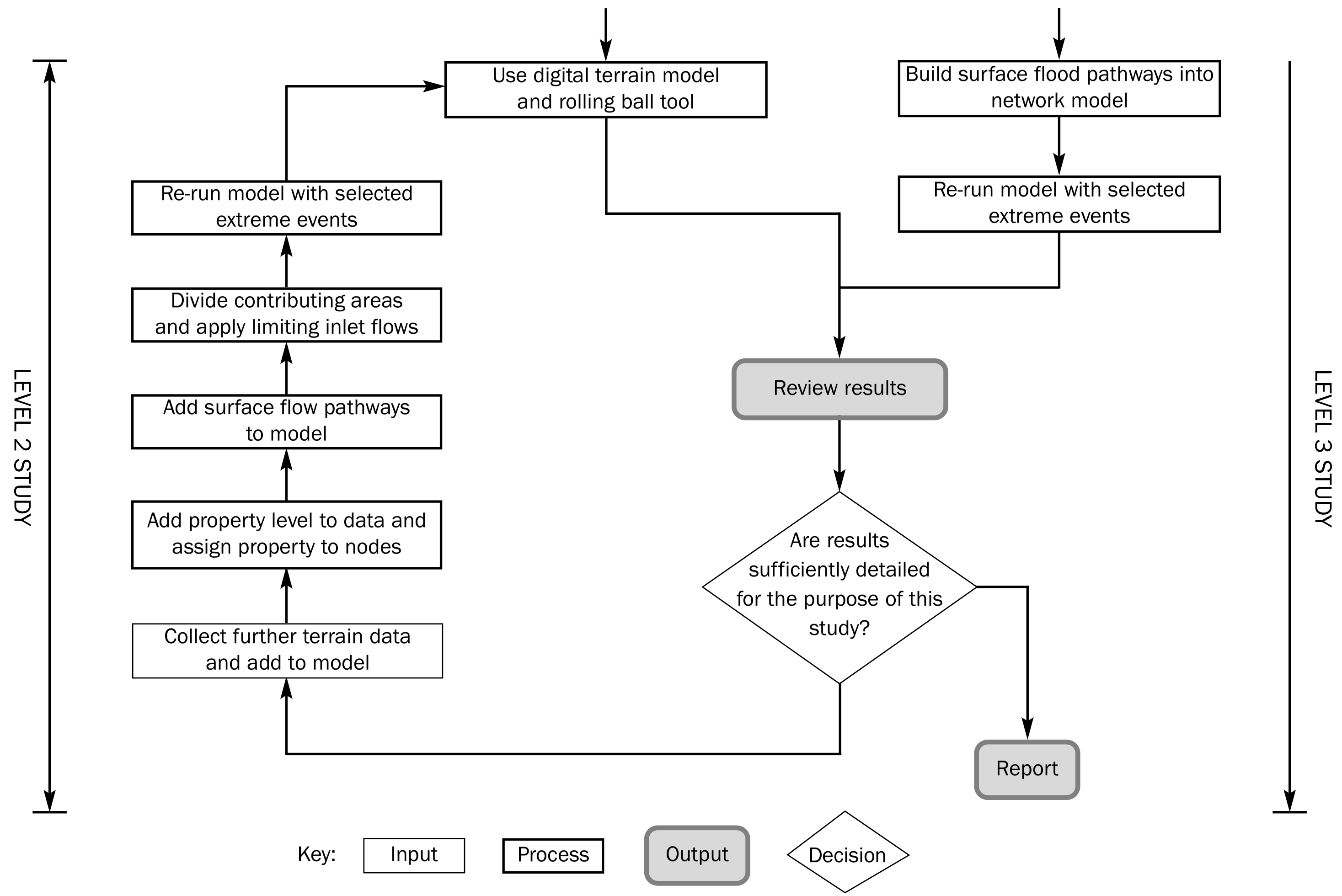


Figure 9.4 *Flowchart for computing exceedance flows and volumes (cont'd)*

9.3 Calculating flows in surface flood pathways

9.3.1 Surface runoff

Surface runoff may be calculated using the Rational Method (see Chapter 6), however this method has significant limitations, especially in generating correct flow volumes. Given the availability and ease of use of modern simulation software tools the Rational Method is only recommended for use with very small areas and where significant major system flow is not anticipated.

The above ground flood pathways may be defined as rectangular or vee section open channels, with nodes at key junctions. When using the Rational Method these should form a dendritic network ie avoiding loops. However loops are permitted with most simulation software.

Contributing areas should be assessed in the usual way by drawing boundaries around the curtilage of property. Contributing areas of different types should be distinguished (eg roofs from ground level paved, and paved from permeable). Each area type should be assigned a suitable percentage runoff value. Further details are given in Chapter 8.

If using the Rational Method each contributing area A can then be converted to an equivalent impermeable area A_e, using the percentage runoff PR:

$$A_e = \frac{PR}{100} \times A \tag{9.1}$$

The peak rate of surface runoff may be obtained using the Rational Method. This calculates flow using the following equation.

$$Q = 2.78.A_i.i \text{ litres/sec} \tag{9.2}$$

where:

A_i	=	equivalent impermeable area in hectares
i	=	average rainfall intensity in mm/hr, based on a critical duration equal to the time of concentration

The rainfall intensity i should be chosen for an appropriate return period of extreme event, and for the critical duration (equal to the time of concentration). Further guidance on this may be found in the Wallingford Procedure (Department of Environment, 1981).

Simulation software tools normally allow for variations of rainfall intensity during storm events and will assign specified PR values to contributing areas directly.

9.3.2 Adding runoff from permeable areas

Current drainage design practice is only to allow for runoff from permeable areas where these are immediately adjacent to paved areas or where they are drained directly to the sewerage system (eg through land drains). However, recent studies such as those in Glasgow have shown that in extreme events these areas can significantly contribute to urban flooding and so should be included in any computations of surface runoff for extreme events, as described in Chapter 8.

When using the Rational Method, a permeable area may be represented by an equivalent impermeable area using the PR value for conversion. However, it is important to note that by representing permeable areas as equivalent impermeable areas, the time of concentration will not be properly replicated. When using simulation software, permeable areas should be properly specified and not represented as equivalent impermeable areas. Further guidance on this is given in Chapter 8.

9.3.3 Surface conveyance

In **level 1 studies** the below ground sewer system is assumed to be full to capacity so that all surface runoff is conveyed above ground. This is a significant simplification of the real scenario and normally will lead to an over estimate of above ground flows. This approach is only suitable for use in small developments where a conservative allowance for surface conveyance is acceptable (eg new sewerage for small housing and industrial developments). It should not be used for assessing exceedance conditions for existing systems or medium to large sized new systems.

Major system flows may be determined either by the Rational Method or preferably by computer simulation, by representing surface flood pathways as drainage channels.

Most software simulation tools will allow the user to specify the shape of the channel cross-section as one of a number of standard types. Rectangular, vee and trapezoidal are usually sufficient to replicate most surface pathways. Figure 9.5 shows how a typical road cross-section may be represented by two vee section channels.

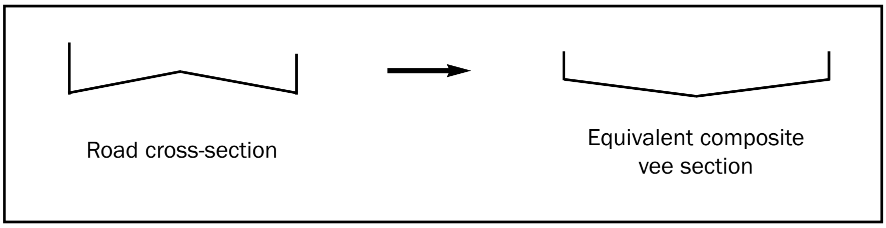


Figure 9.5 *Road flood pathway represented by vee section channel*

In specifying the equivalent vee section in Figure 9.5 the cross-sectional area of flow below top of kerb level should remain the same. The slope of the road surfaces will also be the same. The only change to the equivalent section will be the reduction to the wetted perimeter on the vertical sides of the section. As this is small in terms of the overall cross-section, its effect will be minimal.

$$Q = \frac{A_c}{n} R^{2/3} s^{1/2} \qquad (9.3)$$

Conveyance may be represented by the Manning Equation 9.3.

where:

Q	=	discharge, m^3/s
n	=	Manning roughness value
R	=	hydraulic radius = A/P
s	=	slope (decimal)
A_C	=	cross-sectional area of flow, m^2
P	=	wetted perimeter, m

Further information on the application of this equation is given in Chapter 11, including recommended values for the Manning roughness coefficient n.

The Manning Equation may be solved by trial and error to determine the depth of conveyance flow in an above ground pathway for a given discharge. It can also be used for determining the velocity of flow and time of travel. In theory it can be used with the Rational Method to compute flows and depths in above ground flow pathways. However the method is tedious, and the engineer is advised to use an appropriate software simulation tool instead (see Appendix 1). Some software simulations tools give the user a choice of equations for replicating conveyance, usually between the Manning Equation and the Colebrook-White Equation. This is discussed further in Chapter 11.

The engineer should carefully check values of computed depth in such channels to ensure that the flow will remain within the assumed pathway. Where depths of flow exceed the height of a drainage channel (eg kerb height) then the channel section should be revised (eg extending the road section to back of footpath). Further information on this can be found in Chapter 11 and Appendix 2.

In level 1 studies the above ground flood pathways may be represented by a dendritic network. In the case of road junctions, this may require a decision as to which direction forms the major flood path. This is not always obvious from a plan view and a site visit is advisable where possible to ascertain direction of surface flows. Other flood pathways may be included such as paths and grass lined channels designed especially to convey above ground flood flow.

In **level 2 studies** it is assumed that there is no limit on drainage inlet capacity so that all surface runoff is drained to the below ground system it is full. At this point any additional runoff will induce surface flooding from manholes. In simpler models this surface flooding may be distributed over known flooded areas adjacent to the manhole, and the depth of flooding determined. This may in turn be used to estimate damage cost to property etc and further information on this is contained in Chapter 10. However, experience shows that flood water discharged from manholes often does not remain in the vicinity of that manhole, and can travel some considerable distance, affecting people and property remote from the point of discharge. The engineer should account for such potential surface transmission of flood flows. In its simplest form this can be done by site inspection, identifying potential flood pathways and low spots where flood water might accumulate. This is a particularly useful method when combined with records of known flooding. However unrecorded locations of flooding can be missed and the method is time consuming when applied to larger areas.

Where topographical data is available this can be used to build a digital ground terrain model, and the flood volume can then be transmitted along the line of maximum ground slope, and accumulated in low spots. This technique is often referred to as a "rolling ball" model and this can give a more accurate assessment of the location and volume of flooding. Light detection and ranging (LiDAR) data for many of the major cities is available based on 1 m grid sets with ±150 mm vertical accuracy. If this data is not available, it can be obtained by commissioning a survey (Allitt, 2004).

The accuracy of rolling ball models depends on the accuracy and resolution of available digital terrain data. These tend to give better results in undeveloped areas rather than urban areas, because surface flood pathways in urban areas are often defined by artificial features, often no more than some 50 mm in height. A good example is the role of roads with kerb heights of no more than 100 mm. Currently most terrain level data is not available to such resolution. This means that rolling ball models need to be interpreted with care, and the engineer may need to supplement them by modelling known major flood pathways explicitly.

Local watercourses in extreme events can significantly affect the performance of urban drainage systems. High water levels can restrict the discharge from outfalls from surface water sewers and combined sewer overflows. Where local water courses are thought to interact with the sewerage system in this way an allowance should be made in the model either by applying an appropriate level hydrograph to the outfalls or by explicitly modelling the watercourses as part of the drainage network. Particular attention should be paid to representing culverted sections and watercourse screens, both of which may become blocked during extreme events. Where outfalls are known to be affected by tide levels, this should modelled.

For level 3 studies, full interaction between major and minor systems may be simulated, by modelling surface flood pathways linking nodes, as shown in Figure 9.6. How surface flood pathways are represented will depend on the specific software used. Further information on this is given in Appendix 1. Surface flooding may still occur in such models when the flood pathways are overtopped, and the subsequent passage of such flow to low spots may be simulated with a rolling ball model.

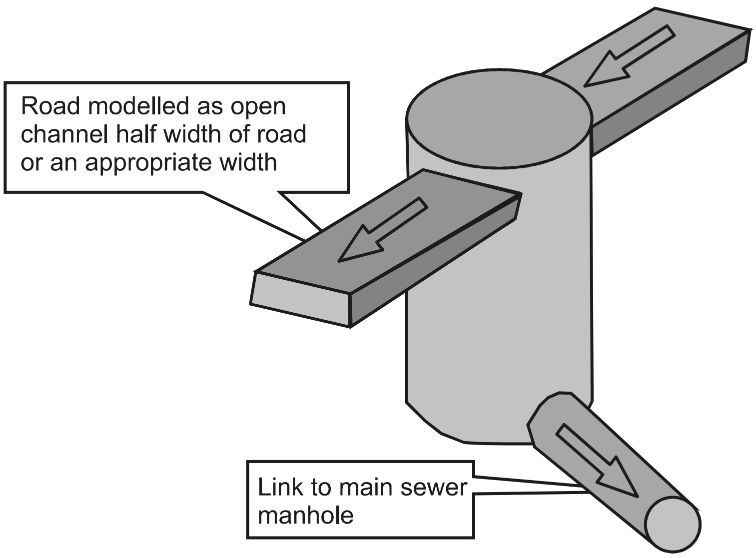


Figure 9.6 *Conceptual representation of fully interactive major/minor system modelling (after Allitt, 1999)*

In **level 3 studies**, individual property may be assigned to nodes so that flood risk can be determined on an individual property basis. In assessing flood risk, due account should be taken of the level of the hydraulic gradient with respect to property floor levels, especially where properties have cellars. It should be remembered that a significant proportion of property flooding occurs without flood flow being discharged from manholes in the locality (ie before the hydraulic gradient reaches ground level). Also, the inlet capacity of drainage should be simulated where it is likely to affect system performance.

Highway gullies, yard gullies and roof gutters can have limited capacity and that including these effects may be important in representing exceedance conditions in large complex drainage systems. Further details of this are given in Section 9.4.

The effects of local watercourses and tide locked outfalls should be represented in the same way as in level 2 studies.

When planning a level 3 study it will be more productive to undertake a level 2 study first. This will enable the engineer to identify locations of major flooding and likely surface flood pathways that can then be explicitly modelled in the level 3 study. In many cases it is not necessary to use a level 3 approach throughout the whole of a drainage area. Level 3 detail may be applied to specific areas of importance with the rest of the area represented at level 2.

Developers should allow for more time in the programme planning for larger developments as modelling and design of sewer networks may take longer than traditionally has been the case.

9.4 Calculating drainage inlet capacity and exceedance

Different drainage inlet devices exhibit different hydraulic characteristics. As represented in Figure 9.3 there will be a particular inflow at which the capacity of the inlet is exceeded. Beyond this not all the flow can be accommodated. The capacity and subsequent ability of an inlet to accept additional flow is described in the following sections for the three main inlet types. Contributing drainage areas will exhibit a characteristic that is a combination of these components.

9.4.1. Highway gullies

The discharge into a highway gully may be limited either by the capacity of the inlet grating, the capacity of the gully pot (normally defined by the size of the outlet) or the capacity of the connecting pipework downstream. Drainage design for highway gullies under normal operating conditions recognises that a proportion of the flow passes over the gully grating (depending on its efficiency), so its performance in extreme events becomes an extension of the conditions assumed in design (Figure 9.7).

Flow draining off the highway surfaces and other paved areas draining onto the highway is conveyed in the channel adjacent to the kerb. Normal design limits the width of flow allowable under design storm conditions. Gully spacing is determined by allowing for the efficiency of the gully grating, and any by-pass flow continues on to subsequent gullies downstream.

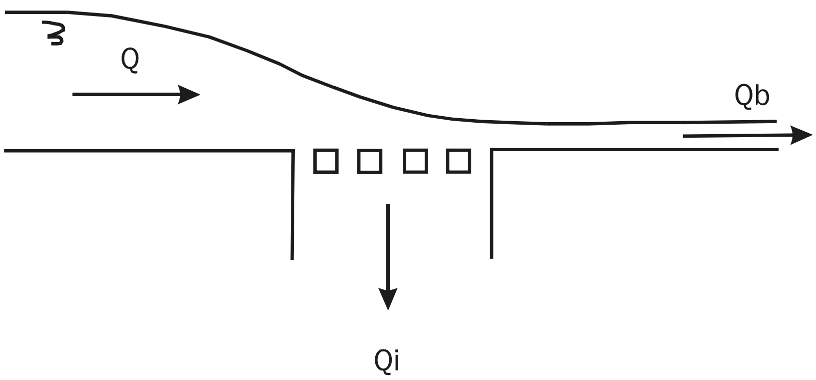


Figure 9.7 *Flow pattern in highway channel at gully inlet*

There will be surface flow on the highway as part of the normal drainage function, and that exceedance flow conveyance will be in addition to this, as illustrated in Figure 9.8. When allowing for the effects of exceedance flow the depth for normal drainage on the surface should be accounted for. The full cross-section of the highway will not be available for exceedance conveyance, as illustrated in the figure.

The efficiency of the gully grating depends on its geometry and the rate and depth of flow in the highway channel (Highways Agency, 2000). Provided that a gully grating is well maintained, its efficiency will not normally be adversely affected by increased flow under extreme events. The channel depth and flow will increase such that the additional runoff from contributing areas is accommodated. This is illustrated in Figures 9.9a and b. However there is evidence that where maintenance is poor, or the outlet from gullies including downstream pipework is limited or defective, then increased flows will not be accommodated. In such cases flow depths and widths on the highway may increase significantly.

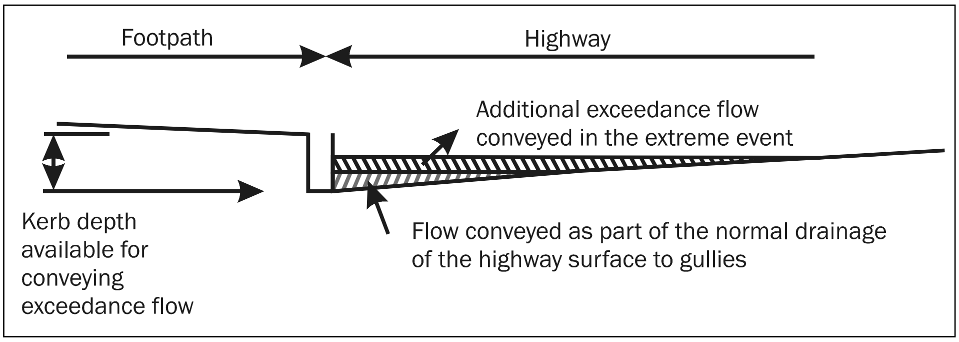


Figure 9.8 *Illustration of conveyance flows on a highway surface (see case study one for further information)*

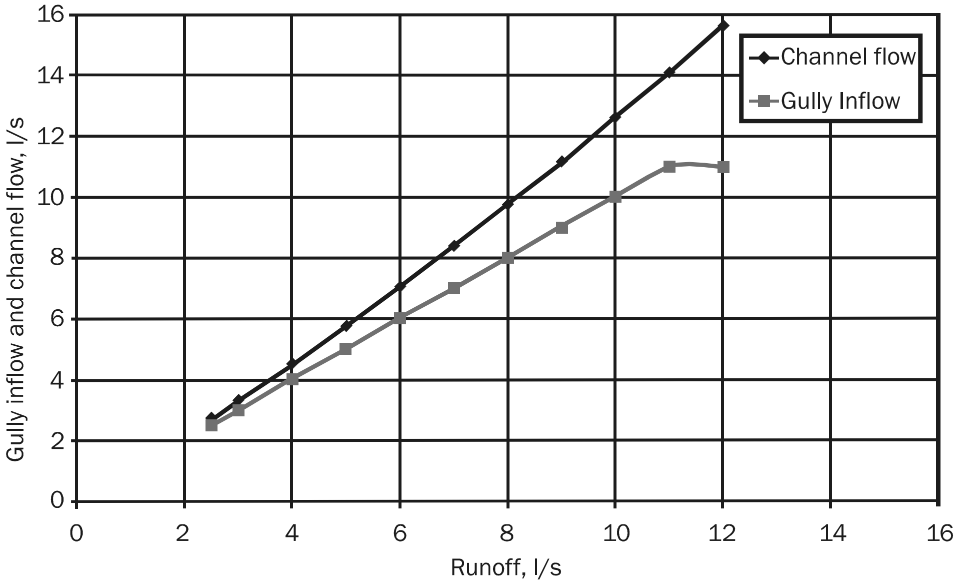


Figure 9.9a *Flow characteristics of a gully inlet, maintenance factor = 1.0*

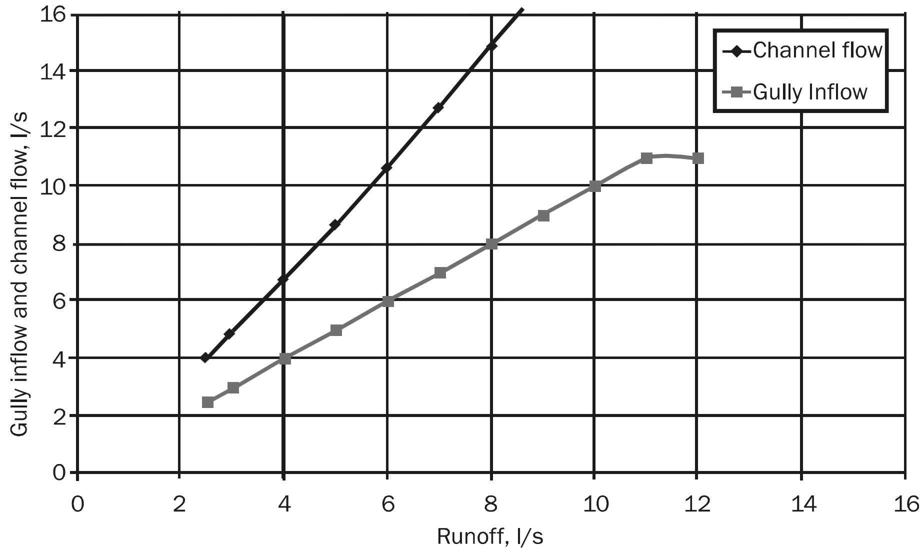


Figure 9.9b *Flow characteristics of a gully inlet, maintenance factor = 0.7*

Exceedance due to gully capacity is likely to occur in the following circumstances:

- the maintenance of all the inlet gratings in the area is so poor as to obscure a significant part of the grating
- there are insufficient gullies for effective drainage of the area (ie less than the normally recommended spacing)
- the runoff discharged to each individual gully is greater than the gully outlet or downstream pipe capacity
- the depth of flow in the highway channel is sufficient for flow to be diverted over the kerb top
- the width of flow in the carriageway is sufficient that a substantial part of the flow is not presented to the gully.

The engineer should undertake a survey of gully inlets and outlets to determine if their capacity is likely to restrict flow from extreme events. Since systems vary widely, only a local inspection can determine the capacity.

For the normal range of contributing areas per gully, it is also unlikely that depth of flow in the highway channel will exceed 100 mm for rain intensities less than about 200 mm/hr. Where the surface cross-fall is less than one in 40, some flow may by-pass the gully on the surface. It is not possible to quantify this as it is beyond the range of gully performance data available, but it is not considered to be significant. Thus the risk of flows being diverted out of the highway channel is only likely where there are drop kerbs or side access roads with a gradient falling away from the channel. In such cases the whole of the channel flow can be diverted. This may be calculated numerically using the method set out in Appendix 2. When using modelling simulation software the effect of such flow diversion may be replicated by reassigning the contributing areas to an appropriate node in the network, or by modelling the surface pathway explicitly. Flow widths can be significant during exceedance events, but they will not have a significant effect on gully efficiency until the flow width exceeds 2 m in most cases. Again, this is unlikely with all but the most extreme events.

It may be concluded that in all but the very exceptional circumstance that the capacity of the gully grating will not restrict the inflow to the gully pot. Tests have shown that capacity of a 100 mm and 150 mm gully outlets is 11 l/s and 19 l/s respectively (Escarameia and May 1996), however limits in the capacity in the connecting pipework can significantly reduce this. It will be these flows that limit the capacity of typical gully inlets. For the larger contributing areas per gully (≈ 0.02 ha) the lower limit (11 l/s) is reached with a rainfall intensity of about 200 mm/hr, though there is some limited evidence to show that exceedance can occur in practice when intensities are greater than 100 mm/hr. Some knowledge of the types, number and condition of gullies and associated pipework serving the drainage area is a clear advantage in assessing the limiting flow at which exceedance will occur. Where this information is available, the limiting discharge per hectare may be calculated from Equation 9.4.

$$\text{Limiting discharge} = G_{out} \times n_{gull} \text{ l/s/ha} \tag{9.4}$$

where:

Gout	=	gully outlet discharge capacity = 11 l/s for 100 mm outlets and 19 l/s for 150 mm outlets. Note that these values should be reduced if there are limitations in the downstream pipework
ngull	=	number of gullies per hectare

Further information on the effects of extreme events on highway flow, together with a worked example, is given in Appendix 2, which is based on workings set out in the Highways Agency Guide HA 102/00 (Highways Agency, 2000).

9.4.2 Roof drains

Most roof drainage consists of a gravity driven system of gutters and rainwater pipes. Rainwater pipes may connect directly to the underground drainage system or via a surface drainage gully. Occasionally large roof areas are drained by syphonic drainage systems that allow large quantities of flow to be drained through relatively small diameter rainwater pipes. Current best practice is described in the Good Building Guide GBG 38 *Disposing of rainwater* (BRE, 2000), Building Regulations Approved Document H (DTLR, 2002) and Standard BS EN 12056-3. For historic reasons many roof drainage systems will not achieve the performance set out in these standards. Others may achieve better performance, especially where the roof areas drained are smaller than the maximum allowed by the standard.

It would not be realistic to account for all the possible variations of roof drainage practice, and there is little data available to establish the actual performance of roof drainage on an area wide basis. For the purposes of this guide it has been assumed that a design complying with GBG 38, but with the maximum roof area allowed to drain to a single rainwater pipe, will represent the average of roof drainage practice (ie an equal distribution of flows drained by systems below standard and above standard).

There are three potential limits on the flow capacity of roof drainage systems.

- the capacity of the roof gutter
- the capacity of the inlet to the rainwater pipe
- the capacity of the drain receiving flow from the rainwater pipe.

The capacity of a rainwater gutter is influenced by the brink depth of flow as the water spills into the rainwater pipe (Figure 9.10). The brink depth determines the depth of flow in the gutter. For roof drainage designed to GBG 38 (BRE, 2000) a gutter will carry the flow from a peak rainfall intensity of 75 mm/hr before overtopping. Shortly after the limit is reached, overtopping occurs over a long length of gutter and little additional flow is conveyed into the drainage system.

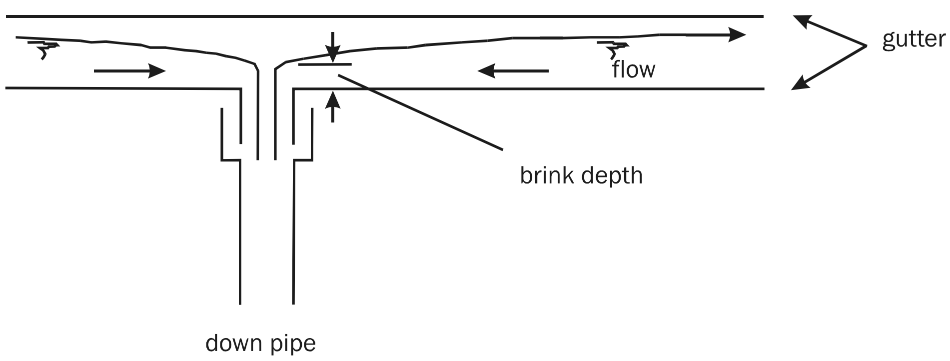


Figure 9.10 *Flow of water from gutter into rainwater pipe*

The capacity of the rainwater pipe is determined by the horizontal flow area at entry to the top of the pipe. Where the rainwater pipe matches the gutter the capacity of this inlet will be in excess of the discharge capacity of the gutter. Rainwater pipes do not normally limit the inlet capacity of roof drainage.

Similarly, flow conveyed by rainwater pipes discharging directly to the below ground drainage system will not normally be limited at the point of connection. Rainwater pipes connected via a surface gully may have their capacity limited by debris restricting the capacity of the gully gratings or where several rainwater pipes discharge to a single gully.

The inlet capacity of roof drainage is best represented on average by the flow that results from 75 mm of rainfall falling on the effective roof area. As a first approximation the contributing roof area may be taken as the plan area of roofing, though if a significant proportion of contributing roofs are pitched, this will lead to some underestimate of the limiting flow. Further information may be found in GBG38 (BRE, 2000) and Building Regulations Part H (DTLR, 2002). The limiting flow may be calculated using the Rational Method assuming that the runoff for the contributing roof area is 100 per cent. For example, if the contributing roof area is 250 m^2 in plan, then the limiting flow will be $2.78 \times 0.025 \times 75 = 5.4$ l/s. This amounts to a value of 0.216 l/s/m^2 of contributing area or 216 l/s/ha which can be used as a suitable default value. Where detailed information of roof drainage exists, it should be used to calculate the actual limiting discharge appropriate to the contributing roof area. In the absence of this, the default value should be used. It should not be necessary to represent each individual roof area and roof areas of a particular type may be grouped together for the purpose of this calculation. In exceptional cases where the default value is used resulting in a significant exceedance flow then a sensitivity analysis should be undertaken on the final design/analysis of the drainage system using different default values.

9.4.3 Yards and other paved area drainage gullies

The capacity of gullies for draining paved areas is set out in BS EN 1253-1. Beyond the guidance for highway drainage set out previously, there is little information available for sizing ground level drainage areas to connect to gullies. Practice varies widely and is often determined by the needs of individual property owners.

For car parks, access roads and other areas drained by gullies, it is suggested that the same principles are applied as for highway gullies (see Section 9.4.1). Data for the hydraulic performance of slot drainage systems for exceedance conditions is not available. It is possible that lack of maintenance will cause the slots to block thus limiting inlet flow. However, in common with other drainage inlets, the capacity of such systems when clean is normally in excess of the required inlet capacity. It is more probable that the actual limit on inlet flow will occur at the connection to the underground pipe system. It should be assumed that the limit on inlet capacity is similar to that which occurs with a gully inlet system.

For smaller domestic areas such as yards and drives the situation is likely to be far more variable. These gullies are also known to be badly maintained and gully pots are sometimes broken. There is no known data available for establishing guidance for calculating limiting flows. However, Part H of the Building Regulations (DTLR 2002) recommends the use of 50 mm/hr for design. This equates to 139l/s/ha of flow, and it is recommended that this be used as the limiting inlet capacity for yard drainage. As this value is significantly less than that for highway drainage, the consequence of this is that exceedance flow from yard drainage may be localised, with the excess flow draining to a local highway drain during extreme events.

If the default values recommended above result in a significant exceedance flow then a sensitivity analysis should be undertaken on the final design/analysis of the drainage system using different default values.

9.4.4 Applying limiting inlet capacity to calculate exceedance flows

From the preceding sections it can be seen that for practical purposes the limitation of inlet capacity to the drainage system can be expressed in terms of either a limiting rainfall intensity or a limiting discharge per unit area. The procedure for determining the division between minor system flow and major system flow (exceedance) is as follows:

1. For each contributing area, divide the area up into different types eg roads, car parks, roofs, paved areas, permeable areas.
2. For each type determine a suitable limiting discharge per unit area.
3. Group together areas sharing the same unit limiting discharge.
4. Identify each group as separate contributing areas in the drainage network model. Depending on the software used this may require the specification of a separate node for each group to connect to. These nodes should be joined together with dummy pipes.
5. For each group, calculate the limiting discharge in l/s. This should be done by multiplying the contributing area for the group by the corresponding unit limiting discharge. For example if roof and yard areas combine to a total group area of 0.02 ha, then the limiting discharge will be $0.02 \times 216 = 4.32$ l/s using the default value.
6. Apply this limiting discharge to the outlet from the connecting node in the model. How this is done will depend on the software simulation tool used. Where the software requires this to be specified as a whole number, the value should be rounded down.

9.5 Inlet capacity of SUDS systems

Where SUDS receive inflows from an upstream piped system, the inlet capacity will be determined by the inlet capacity of the piped drainage system, as described earlier. SUDS units falling into this category includes soakaways, storage ponds and basins. Other SUDS, however, receive surface runoff directly. These include swales, infiltration trenches and pervious pavements. Filter strips and green roofs fall into this category but they are currently rare in the UK.

When SUDS receive runoff directly, the capacity will be limited by either the inflow, or by the volume of storage. For example, the inlet capacity of a pervious pavement will be exceeded when the rainfall intensity exceeds the capacity of the permeable surfacing or when the storage capacity is exceeded. The latter will be depend on exfiltration or outflow rate.

The inlet capacity of a particular SUDS unit will depend on the characteristics of the rainfall, the contributing area and the SUDS unit itself. It can only be fully assessed by detailed analysis or modelling. However, this will not be feasible at outline design stage and may not be cost effective at detailed design.

To assist the designer a generic analysis of pervious pavements, swales and infiltration trenches has been undertaken on a number of "standard sized" units, each draining a unit contributing area of one hectare. Details of the analysis and further results are given in Appendix 5 and the results are shown in Table 9.1 to 9.8. Results are quoted separately for the south and the north of England to allow for difference in climate. Values may be interpolated for intermediate locations.

Table 9.1 *Overflow rate (l/s) from pervious pavement system – south*

Additional paved area (ha)	0				1				2			
Limiting discharge rate l/s / Return period	1	2	5	10	1	2	5	10	1	2	5	10
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0
100	0	0	0	0	0	0	0	0	27.1	25.7	22.0	7.826
200	0	0	0	0	4.4	3.1	0.0	0	44.6	43.2	39.4	33.6

Table 9.2 *Overflow rate (l/s) from pervious pavement system – north*

Additional paved area (ha)	0				1				2			
Return period \ Limiting discharge rate l/s	1	2	5	10	1	2	5	10	1	2	5	10
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0
100	0	0	0	0	0	0	0	0	684.6	616.2	464.8	287.3
200	0	0	0	0	103.1	56.2	12	0	1116	1044	878.3	674.7

Table 9.3 *Overflow volume (m³) from pervious pavement system – south*

Additional paved area (ha)	0				1				2			
Return period \ Limiting discharge rate l/s	1	2	5	10	1	2	5	10	1	2	5	10
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	18.9	17.5	13.7	7.8
100	0	0	0	0	11.7	10.3	6.3	0.8	57.2	55.8	51.9	46.0
200	0	0	0	0	24.1	22.7	18.9	13.0	80.1	78.7	74.8	68.8

Table 9.4 *Overflow volume (m³) from pervious pavement system – north*

Additional paved area (ha)	0				1				2			
Return period \ Limiting discharge rate l/s	1	2	5	10	1	2	5	10	1	2	5	10
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	466	366	262	111
100	0	0	0	0	283	222	101	5.5	1401	1315	1155	940
200	0	0	0	0	610	542	391	219	1899	1810	1642	1413

Table 9.5 *Overflow rate (l/s) from 100m long swale system – south*

Swale outflow rate (l/s)	0				1				2			
Return period \ Gradient (m/m)	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	2.7	0.8	0	0
10	0	0	0	0	0	0	0	0	14.4	5.6	1.1	0
30	0	0	0	0	6.5	0	0	0	27.0	14.3	3.5	0
100	15.4	0	0	0	31.7	8.7	0	0	44.5	30.8	11.8	0.6
200	29.3	0	0	0	41.0	20.9	0	0	51.4	39.8	18.8	1.7

Table 9.6 *Overflow rate (l/s) from 100m long swale system – north*

Swale outflow rate (l/s)	15				7.5				0			
Return period \ Gradient (m/m)	0.05	0.020	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	8.7	4.9	0	0
10	0	0	0	0	0	0	0	0	18.8	15	6.9	0
30	0	0	0	0	0.7	0	0	0	27.9	24.1	16.1	0
100	1.3	0	0	0	5.4	1.5	0	0	41.1	37.3	29.2	5.8
200	3.4	0	0	0	8.1	4.4	0	0	48.4	44.5	36.5	13.1

Table 9.7 *Overflow rate (l/s) from an infiltration system designed to a 10 year return period – south*

Return period	Infiltration rate (mm/hr)		
	1.8	3.6	36
10	1.3	2.4	0.0
30	10.8	14.15	9.6
100	20.0	18.8	17.6
200	22.3	24.7	22.5

Table 9.8 *Overflow volume (m^3) from an infiltration system designed to a 10 year return period – south*

Return period	Infiltration rate (mm/hr)		
	1.8	3.6	36
10	9.9	8.4	0
30	61.4	53.6	12.7
100	136.5	123.3	57
200	176.7	161.3	84.1

The results give the maximum flow rate and volume diverted to the major (above ground) system for different return period events for one hectare of contributing area. The results show that pavements constructed as pervious pavements throughout do not generate any exceedance flow or volume, even for the 200 year return period events. Only where additional impermeable paving is added is major system flow generated.

Swales of less than 100 m with no outfall (or blocked) can cater for virtually any event less than 12 hours if the gradient is equal to or less than 1:200, increasing to one in 50 when the infiltration capacity is 15 l/s. For swales of less than 100 m with no outfall (or blocked), flood flows of up to 50 l/s can be generated on steep catchments for storms greater than 12 hours duration. Swales with check dams at 20 m intervals will effectively prevent any flooding from taking place even on steep catchments. Flood volume is unlikely to be a problem unless the swale outlet is blocked or is extremely small.

The overflow rate for infiltration trenches is virtually identical for different climates (north and south). The maximum overflow rate is very insensitive to infiltration rate as the design of the unit relates the two issues of volume and infiltration rate. The maximum overflow volumes are more sensitive to infiltration rate and climate with the northern climate causing greater flooding. The maximum flood volume per house for a 200 year return period event is 5 m^3

The designer may use Tables 9.1 to 9.8 to estimate exceedance flow and volumes for outline design. At detailed design stage the designer should compare the details of the proposed design with the "standard" arrangements used in the analysis, as summarised in Appendix 5. Where the design differs significantly from these, the exceedance flow and volume should be determined using the method set out in Appendix 6 or by using the appropriate network model.

An example of using the graphs for a permeable pavement design are provided in Appendix 7.

10 Developing a risk assessment

10.1 An introduction to exceedance flood risk assessment

Assessing the risk of flooding to human life and property is very important. Flooding has the potential to cause serious harm or death to human life or can have serious socio-economic, financial and psychological effects. Surface flooding may be considered in the very least to be inconvenient, however if flood water enters the property, then significant damage can be caused to the internal fabric and fittings. Post flooding, a substantial clean up operation is usually required to restore the property and its contents to its former state. The ultimate cost of this clean up is passed back to individual property owners or their insurers.

Assessing flood risk has become an important process. For example, the EA (Murphy, 2003) have adopted a more strategic approach by focusing on flood risk reduction rather then just on flood defence. CIRIA's recent publication C624 *Development and flood risk – guidance for the construction industry* (Lancaster *et al*, 2004) provides a method to address flood risk as part of the planning process. Other documents such as *Sewers for adoption 5th edition* (Water UK and WRc, 2001) clearly highlight the need to understand what happens with exceedance flow during extreme events.

Flood risk can be assessed by calculating the probability of an event occurring and the subsequent impact that it has on a receptor. It is essential to consider risk in terms of probability and consequence rather than one of these components in isolation. A common misinterpretation of risk is that it is the probability of an event occurring only, yet the consequences as a result of the event are just as important. For example, flooding may regularly occur in a given location (hence having a high probability) but the consequence of this flooding may be very limited so that the overall risk would be low. Alternatively flood impact could be quite high, but the likelihood of such flooding is so small that the overall risk is considered to be low. The following sections on exceedance flood risk assessment (EFRA) will help to quantify this and offer principles and methods that enable the user to determine a risk value. This flood risk assessment guidance has been developed to apply to exceedance flooding from urban sewerage systems, rather than other forms such as fluvial flooding.

10.2 Components of the EFRA

Many different components need to be considered when assessing exceedance flood risk. Some of these are critical to quantifying risk, while other (softer) components such as social, health and psychological impacts are less easy to quantify. Nevertheless, these are important and a summary of their impact is included in this guidance. Currently further research is required to quantify their effect (Evans *et al*, 2004).

The key components in the exceedance and flood risk assessment related to probability and consequence are shown diagrammatically in Figure 10.1. In determining risk there are three component groupings:

- inputs
- processes
- outputs.

There are three main groups of inputs that feed into the processes. Firstly there is the determination of the exceedance flow, depth, velocity, volume and duration. This may be determined through catchment modelling involving the use of computer simulation models or hand calculations in the simple cases. The information contained in Chapters 6, 7 and 8 should be used to identify the most appropriate method for arriving at these values. These will usually be determined for a particular rainfall return period, specifying the probability of occurrence.

The second group of inputs that feed into the consequence part of the assessment are measurable, and include damage to property or the health and safety of the public.

The third group are those which are more difficult to quantify and include the environmental, socio-economic impacts and loss of facility.

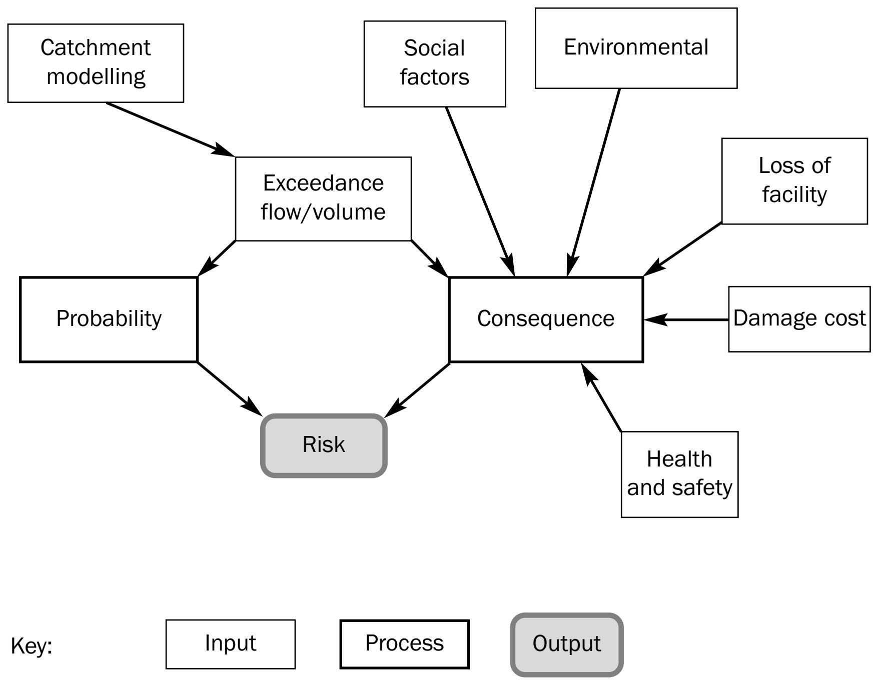


Figure 10.1 *Summary of the inputs, processes and outputs in the exceedance flood risk assessment*

10.3 Determining the risk value

10.3.1 EFRA process

The process to determine the risk is illustrated in the two flowcharts in Figures 10.2 and 10.3 and is divided into two parts. The level of detail used in the risk assessment should match the level of detail of the study, as set out in Box 9.1. This is discussed further in Section 10.3.

The following sections describe each step of the process in detail.

Step 1: Determine the level of risk assessment using the criteria set out in Box 9.1.

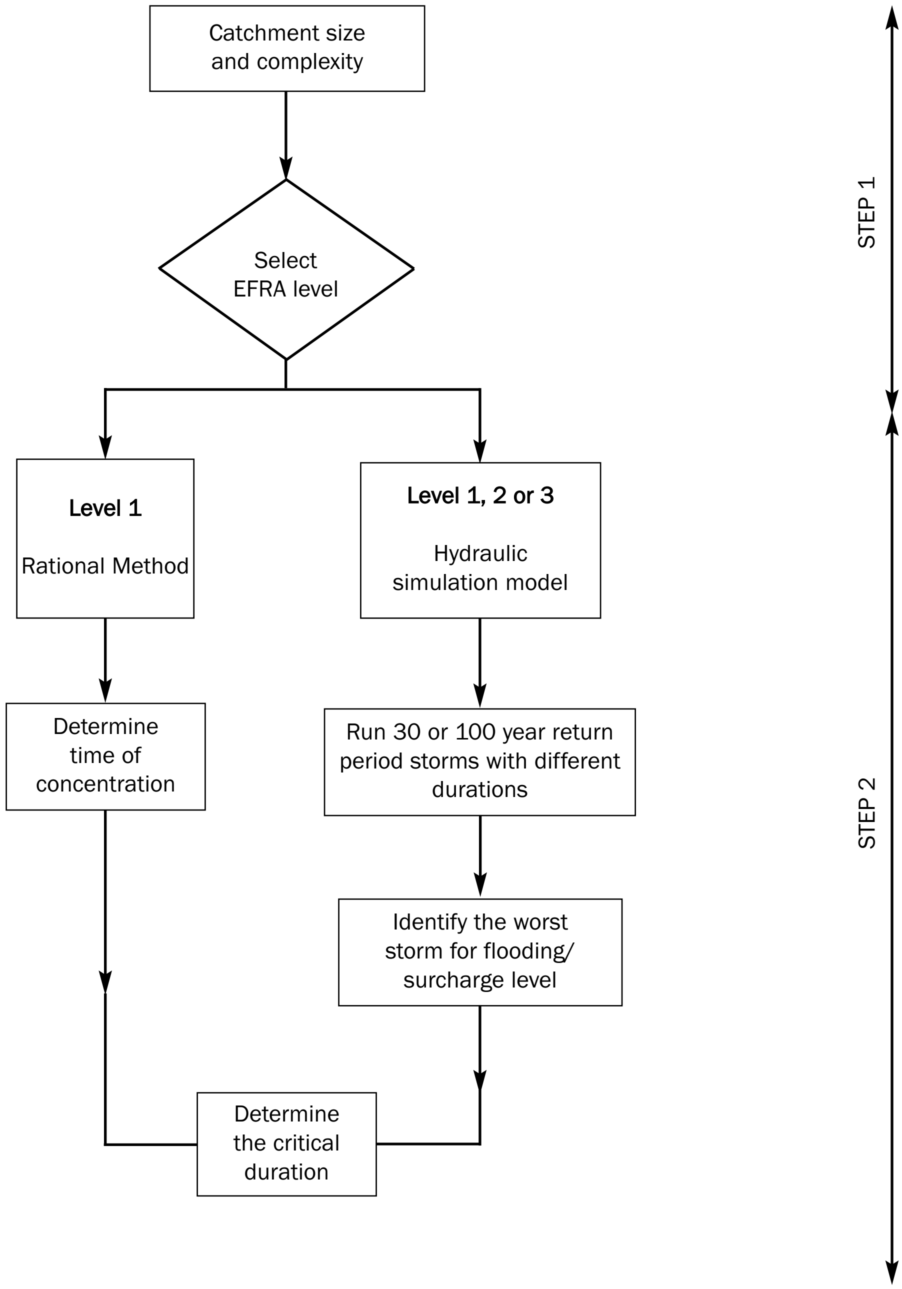
Step 2: This involves selecting the critical (worst case) duration for the rainfall event used in the assessment. When using the Rational Method (level 1 studies only, see Appendix G) the critical duration should be equal to the time of concentration for the drainage area and this value should be used for all storm return periods. If a hydraulic computer simulation tool is used then 30 year return period design events should be selected and run with the hydraulic model (can be used for all levels of study). The critical duration can then be determined from the storm that creates the most significant flooding or highest surcharge levels.

Step 3: The 30 year return period design storm is used to initially identify if flooding occurs and the location. The method of calculation (or modelling) will be determined by the level of study (level of risk assessment) as set out in Box 9.1 and discussed in Section 10.3 below. If no flooding occurs in the area, then the return period is increased appropriately, until flooding appears.

Step 4: This step involves calculating the consequence of any flooding. The level of detail used in the calculation and in quantifying consequence will again depend on the level of risk assessment being used (see Section 10.3). Once the consequence has been determined, this is combined with probability (obtained from the rainfall return period) to determine the risk score (using the risk matrix in Figure 10.4).

Step 5: If necessary the process may be repeated for other return period events.

Revising the assessment with a storm of higher or lower return period should be determined by the level of flooding and risk from the previous assessment. It is expected that the risk rating determined for a range of storms is unlikely to increase or decrease by more than one value. If a wide range of values is obtained, this will indicate that the area may be sensitive to minor changes or that some of the input values may be incorrect.



to Figure 10.3

Figure 10.2 *Part 1 Generic EFRA process to determine the critical duration for the area being assessed*

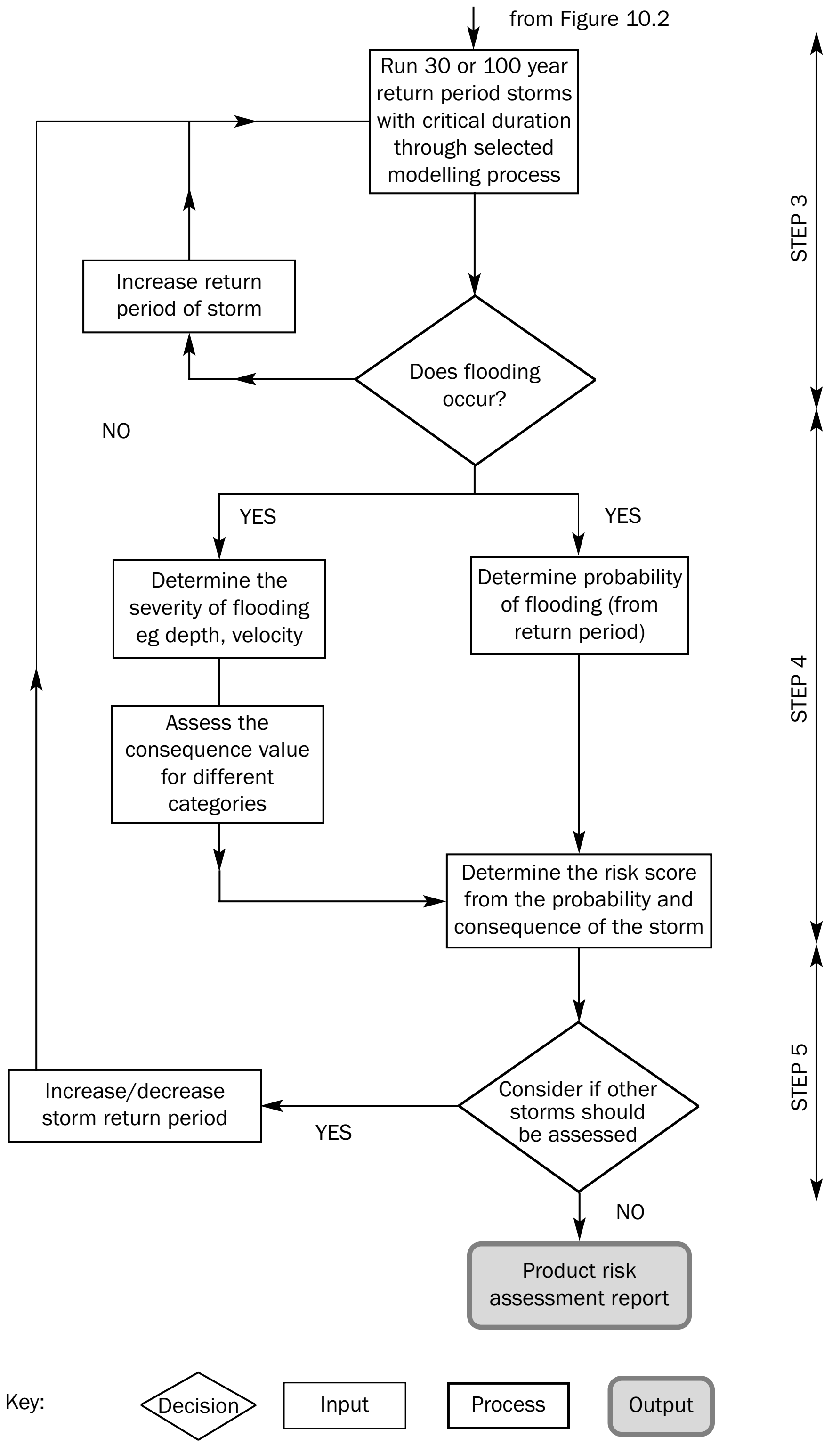


Figure 10.3 *Part 2 Generic EFRA process to determine the risk rating for different storm return periods*

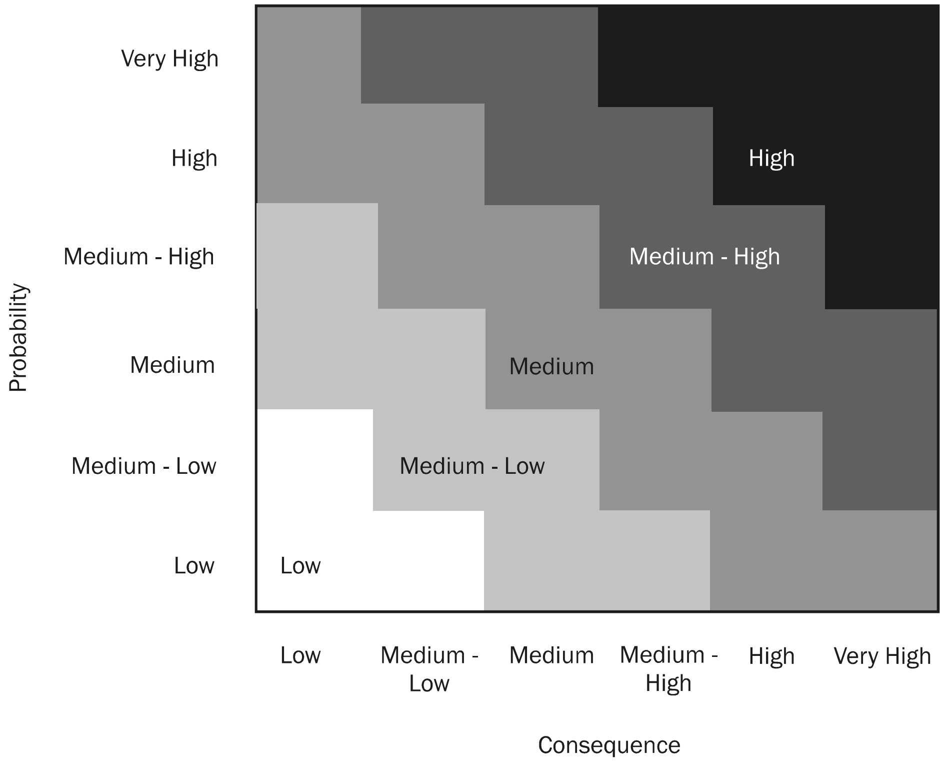


Figure 10.4 *Showing the risk value depend on the probability and consequence value determined.*

Note: This figure is only indicative, and the scoring of probability and consequence, and risk should be adjusted to suit particular circumstances

10.3.2 Selection of the appropriate EFRA level

Any risk assessment undertaken should be appropriate to the size and complexity of the area being assessed. The EFRA has been split into three levels, and relates to the three levels of study discussed in Section 9.2. A level 1 study will be applicable to small drainage areas that have a simple dendritic drainage layout without complex ancillaries. A level 2 study will be applicable to large or complex drainage systems. A level 3 study will be applicable to large and complex drainage systems. The information required to complete the different levels of EFRA are described in the following sections.

An initial scoping study can be used to identify the appropriate extent and detail of the final study. Scoping studies should be conducted as level 1 studies with consequence assessed from Table 10.6. Further detail can then be added as necessary to build up the study to level 2 or 3 in certain areas.

There are similarities in the information requirements to complete an EFRA for existing and newly designed drainage systems. An existing system will make use of current information. Any existing flooding data should be considered and used to verify modelling and calculations produced to identify existing flood paths prior to developing solutions. New designs will make use of development and works drawings. The proposed information and the assessment could provide extra evidence to the flood risks in any proposed development plan.

An example of undertaking a risk assessment at a variety of levels is considered in the case study in Part D.

10.3.3 Level 1 EFRA – simple small areas

Simple small areas are treated as having dendritic drainage layouts without complex ancillaries. This level is applicable to property flooding rather than health and safety risks. The various phases of the Level 1 EFRA is detailed in Table 10.1 which includes a summary of the information, inputs and outputs (see Sections 9.2 and 9.3). In a level 1 study the minor system is ignored and all the flow is assumed to be conveyed by the surface pathways. The surface pathways form the drainage network.

Table 10.1 *Summary of the phases used to undertake each level of study*

Phase	Description	Level of study		
		1	2	3
Desktop study	Collect OS mapping/development plans.	●	●	●
	Collect sewer system data/locations.	●	●	●
	Collect topographical data (preferably in digital form).		●	●
	Collect available information on previous flooding incidents (eg photographs, videos).			
	Identify low spots (including location of properties with cellars).		●	●
	Identify and map potential above ground flow paths.	●		
	Digitally map above ground flow paths.	●	●	●
	Identify and map known/potential flooding locations.	●	●	●
	Assess known flood paths if existing site to be re-developed.	●	●	●
On-site study	Site visit to confirm desktop assessment.	●	●	●
Flow calculation	Select an appropriate method for calculating surface runoff (Chapter 8).			
	Select appropriate rainfall return period (Chapter 6) and determine critical duration (step 2 above).			
	Compute peak rate of runoff from contributing areas.			
	Calculate flows in surface flood pathways.			
	Take off contributing areas from mapping, taking care to include pervious areas that drain to the system.	●	●	●
	Select a suitable runoff model (Chapter 8).	●	●	●
	Select suitable sewer network modelling software capable of replicating surcharging and surface flooding.	●	●	●
	Commission a short term flow survey to verify sewer model (*large and more complex areas on level 2).	●	●	●
	Add in surface pathways to model as appropriate.		●	
	Model above ground pathways.			●
	Model inlet capacity and minor/major interaction.			●
	Run model with different duration storms to determine the critical duration.		●	●
Flooding location/ volume	Compare computed flows with capacity of surface flood pathways.			
	Identify flooding location and approximate volume.			
	Run model with appropriate return period to compute location and degree of surface flooding.	●	●	●
	Identify conveyance in surface pathways and locations and depths of surface flooding.	●	●	●
	Identify the depth and velocity in above ground flow paths.			●
	Identify depth and velocity surrounding property.			●

Determine probability	Identify the probability that a location will be flooded (Section 10.5).		●	●
	Identify the likely flood depth banding that causes:			
	● basement flooding	●		
	● external flooding			
	● ground floor and above flooding.	●		
	Identify flood depth at locations with critical storm.		●	●
	Identify internal flooding frequency.		●	●
Determine consequence	Identify initial property consequence from Table 10.7.			
	Identify property type(s).			●
	Determine adjusted consequence value based upon flood depth banding.	●		
	Determine consequence rating dependent upon depth property depth and adjusted for velocity.	●	● ●	●
	Determine psychological consequence.			●
	Determine loss of business consequence (if applicable).			●
	Determine health and safety consequence.			●
Determine risk	Using the risk matrix (Figure 10.4), determine risk value for:	●	●	●
	● groups of property	●	●	
	● individual property			●
	● loss of business		●	●
	● limited health and safety.	●	●	●

10.3.4 Level 2 EFRA – large or complex areas

A level 2 study is appropriate for the analysis/design of medium to large or complex systems (for further information see Section 9.2 and 9.3). In this study the minor drainage system will be analysed/designed using sewer network modelling software capable of replicating surcharging and surface flooding. Surface flooding is represented by ponding in low spots, which are identified and sized by local interpretation or using a topographical model. Surface conveyance is represented either by modelling surface flood pathways explicitly, or by using a "rolling ball" software with a digital terrain model. The drainage network may consist exclusively of the minor (below ground) drainage network or a combination of minor and major drainage systems.

Contributing pervious areas are modelled in the sewer network model, however no allowance is made for inlet capacity. The volume and depth of flooding is calculated but the velocity of surface flow is not. A level 2 study is applicable in determining the risk of property flooding based upon depth (Table 10.1).

10.3.5 Level 3 EFRA – large and complex areas

A level 3 study is appropriate for the design and analysis of large and complex drainage systems (see Sections 9.2 and 9.3). The minor drainage system is analysed/designed using sewer network modelling capable of replicating surcharging, backwater effects and surface flooding. Surface conveyance is replicated by the explicit modelling of known surface flood pathways with full interaction between the major and minor networks. Contributing pervious areas are explicitly modelled and an allowance is made for inlet capacity on an area or individual basis. A level 3 study enables the flood risk on property (depth and velocity), loss of business and health and safety consequences to be assessed (Table 10.1).

10.4 Assessing the probability

If flooding occurs from a rainfall event, the probability of the event occurring should be based upon the return period. The return period can also be expressed as the likelihood that the event will occur within one year. A number of storm return periods are given in Table 10.2 and their related probability rating. These should be used in Part 2 described in Section 10.3.1.

Determining the frequency of the event is necessary to determine the probability rating. However the "critical duration" as identified in step 1 in Section 10.3 is also important to assess the flood risk. The critical duration is that which causes the worst case of flooding and surcharging in the area being assessed. In some extreme cases (and most likely to occur where large catchments are being assessed) there may be more than one critical duration.

Table 10.2 *Probability rating for a storm event*

Return period (1 in n years)	Probability of being equalled or exceeded in any one year	Suggested probability rating
1	1	Very high
2	0.5	Very high
5	0.2	Very high
10	0.1	High
20	0.05	High
30	0.033	Medium – high
50	0.02	Medium
100	0.01	Medium – low
200	0.005	Low

10.5 Assessing the consequence

The consequence of flooding can be very wide ranging from minor overland flooding through to deep and high velocity flow. The types of consequences include danger to life and the health and safety of people, damage to property and its internal contents, psychological impact, loss of business or trade and preventing normal services from operating. Many of these impacts are combined due to the nature of flooding. Table 10.3 identifies the consequences considered for each level of assessment.

Table 10.3 *Type of consequence considered for each EFRA level*

EFRA Level assessed consequence	EFRA level 1	EFRA level 2	EFRA level 3
Property damage (volume)	YES	YES	YES
Property damage (depth)	NO	YES	YES
Loss of facility	NO	YES	YES
Property damage (velocity and depth)	NO	NO	YES
Health and safety	YES*	YES*	YES

* Health and safety should be considered during risk assessment for EFRA levels 1 and 2.This is only possible using the depth and velocity data produced during a level 3 assessment, however the magnitude of flows and depths should be considered for locations where flows may collect or be routed.

10.5.1 Consequence hierarchy for building types or land use as a result of flooding

Damage to property is regularly reported by the media and can be a significant consequence of flooding. Table 10.4 gives an initial indication of the consequence rating for property flooding. It has been developed using information available in PPG 25, and identifies a hierarchical approach to flooding. The table may be used in level 1 studies and as an initial assessment of risk in identifying appropriate levels of study.

Table 10.4 *Initial hierarchy of the consequences of flooding certain properties/locations*

Potential impact zones or structures	Initial consequence rating
• hospitals • junior/infant school and nurseries • senior citizen housing • emergency services • telecommunication centres • high value manufacturing • temporary domestic dwellings (mobile home/pre-fabs) • major shopping areas • any facilities located in a tunnel (London Underground, subways etc) • major stormwater pumping stations • power supplies • water and wastewater treatment works • road/railway cuttings • underground car parks • access for emergency services and to these areas.	High
• major highways/transport routes • medium/low value manufacturing • permanent domestic dwellings • other schools • commercial/business areas • local shopping areas • major sports facilities.	Medium
• playing fields and open space • minor highways/transport routes • car parks and minor sports facilities • derelict buildings • brownfield sites • canals.	Low

10.5.2 Damage to property

Damage can be caused in the short term and the property will need to be dried out with the contents replaced, refer to C623 *Standards for the repair of buildings following flooding* (Garvin *et al,* 2005). More severe flooding can cause long term structural damage and this has more significant implications. An indication of the typical consequences as a result of flooding to residential properties is described in Table 10.5. A number of factors are involved in property flooding including the depth of water, duration of flooding, wave effects, external pressures, velocity of the water and water quality.

The form of construction of a property can have an effect on the amount of damage caused and this commented on further in Section 13.3.1.

Table 10.5 *Flood damage for a typical residential property (ODPM, 2003)*

Depth of floodwater	Damage to the building	Damage to services and fittings	Damage to personal possessions
Below ground floor level.	Minimal damage to the main building. Floodwater may enter basements, cellars and voids under floors. Possible erosion beneath foundations.	Damage to electrical equipment and other services in basements and cellars. Fittings in basements and cellars may need to be replaced.	Possessions and furniture in basements and cellars damaged.
Up to half a metre above ground level.	Damage to internal finishes, such as wall coverings and plaster linings. Wall coverings and linings may need to be stripped to allow walls to dry out. Floors and walls will become saturated and will require cleaning and drying out. Damp problems may result. Chipboard flooring likely to require replacement. Damage to internal and external doors and skirting boards.	Damage to electricity meter and consumer unit. Damage to gas meters and low-level boilers and telephone services. Carpets and floor coverings may need to be replaced. Chipboard kitchen units are likely to require replacement. Washing machines, free standing cookers, fridges and freezers may need to be replaced.	Damage to sofas, other furniture and electrical goods. Damage to small personal possessions. Food in lower kitchen cupboards may be contaminated.
More than half a metre above ground level.	Increased damage to walls, possible structural damage.	Damage to higher units, electrical services and appliances.	Damage to possessions on higher shelves.

10.5.3 Damage due to depth

The final cost of property flooding depends upon the depth of water. Data from the *Multi coloured manual* (Penning-Rowsell *et al,* 2003) has been used to determine depth-cost damage relationships for different property and area types. These relationships have then been used to develop consequence ratings for different property types. This has been converted into a chart to enable a quick assessment of the consequence to be assessed based on depth (Table 10.6). If foul sewage is present in the flood water, the consequence rating should be increased by one.

Table 10.6 *Consequence ratings for property dependent upon flood depth*

Property type \depth (m)	-1	0.0-0.25	0.25-0.5	0.5-0.75	0.75-1.0	1.0-1.25	1.25-1.5	1.5-2.0	>2.0
Farm/parkland	n/a	L	L	L	L	L	L	L	L
Bungalow	n/a	L	ML	ML	M	M	M	M	MH
Detached	n/a	L	ML	ML	M	M	M	M	MH
Semi-detached	n/a	L	ML	ML	ML	M	M	M	MH
Terrace**	n/a	L	ML	ML	ML	M	M	M	MH
Flat	n/a	L	ML	ML	ML	M	M	M	MH
Retail warehouse*	n/a	L	L	ML	ML	ML	ML	M	M
High street shop*/**	n/a	L	L	ML	ML	ML	ML	M	M
Warehouse*	n/a	L	ML	M	M	MH	MH	MH	MH
Office*	n/a	ML	M	MH	H	VH	VH	VH	VH
Super/hyperstore*	n/a	ML	MH	H	VH	VH	VH	VH	VH

For all categories, if the flooding contains foul sewage then increase the consequence rating by two.
* If the duration causes the property to be unoccupied or limits trading then the consequence value will change (see Section 10.5.6)
** Some of these properties may have basements. If these are lived in, then increase the consequence rating by one.
L = Low, ML = Medium low, M = Medium, MH = Medium high, H = High, VH = Very High

10.5.4 Damage due to depth and velocity

Where velocities are high, extra property damage can occur. Depth velocity relationships produced by Clausen and Clark (1990) have been used to develop Table 10.7. This uses the existing depth damage consequence above and the velocity × depth (DV) value to determine the consequence. Only when the DV relationship exceeds a value of three does the consequence rating change.

Table 10.7 *Consequence rating when velocity and depth are considered*

Velocity * depth value \ Existing consequence rating depth only	Low	Medium – low	Medium	Medium – high	High	Very high
0.0-3.0 m²/s	L	ML	M	MH	H	VH
3.0-5.0 m²/s	ML	M	MH	H	VH	VH
5.0-7.0 m²/s	M	MH	H	VH	VH	VH
> 7.0 m²/s	MH	H	VH	VH	VH	VH

10.5.5 Health and safety

Health and safety impacts are wide ranging and include:

- storm water mixed with foul sewage presenting a health risk
- pedestrians being at risk from drowning due to the depth of the flood water
- the combination of the depth and velocity could knock a pedestrian off their feet
- vehicles could be carried away by a combination of depth and velocity
- blown manhole covers leaving an exposed entry could become a trip or fall hazard to pedestrians and vehicles.

When considering the consequence of the flow depth and velocity to **pedestrians**, the following chart can be used to determine the consequence rating (Table 10.8). This is based upon research undertaken by Helsinki University of Technology (2001) and Defra/EA (Ramsbottom *et al,* 2003). The rating is dependent upon the depth of the DV relationship and the external conditions. Three external conditions are considered and the two extremes, good and bad are described in more detail in Table 10.9. Further reading is available in reports on Flood Risk to People (Phase 1 and 2 of a Defra/EA project) by Ramsbottom *et al* (2003) and HR Wallingford *et al* (2004).

When considering the consequence of the flow DV relationship to **cars and their passengers**, Table 10.10 can be used to determine the consequence rating. This is based upon research reported by Reiter (2000).

Table 10.8 *Consequence rating for pedestrians in flood water. For children and the elderly, increase the consequence rating by one*

Surrounding conditions / Depth or depth* velocity value	Good conditions	Normal conditions	Poor conditions
0.5 m	L	ML	MH
1.0 m	ML	M	H
0.5 m²/s	M	H	VH
1.0 m²/s	MH	VH	VH
1.5 m²/s	H	VH	VH
> 1.5 m²/s	VH	VH	VH

Table 10.9 *Description of the two condition extremes for pedestrians*

Criteria \ Conditions	Good	Poor
Ground surface	Smooth, not slippery and no obstacles	Uneven, slippery, obstacles
Water	No moving debris, warm, good visibility	Moving debris, low temperature, poor visibility
Human subject	Not carrying any additional load, in good health	Carrying additional load, disabled, elderly or a child
Lighting	Good lighting, daylight	Poor lighting, night time

Table 10.10 *Consequence rating for humans in cars and damage to the car itself*

Depth × velocity range \ Criteria	Damage to car	Adult in a car	Child in a car
	ML	M	H
< 0.1 m²/s	L	L	L
0.1 – 0.3 m²/s	L	LM	M
0.3 – 0.6 m²/s	M	M	H
> 0.6 m²/s	H	VH	VH

10.5.6 Loss of facility/business

Loss of facility occurs when flooding limits or prevents a service or function from operating correctly and is likely to have an economic impact. This could be, for example, if a shop is flooded and limits or prevents customers entering. Alternatively, electricity supplies could be stopped which can cause further impacts beyond the immediate area of flooding. The cost of this can however be estimated.

Financial implications from the interruption is very important and could cause a company to cease trading due to loss of custom in the short or long term. If the duration causes the property to be unoccupied or limits trading the consequence rating identified in Table 10.8 should be increased by one. If the duration causes the property to cease trading for a considerable period of time, the consequence rating should be increased by two.

10.5.7 Emergency services

Some areas have higher consequences as a result of flooding compared with others and they should be protected from "any conceivable event". These would include, for example, the headquarters and depots of emergency services, high security installations and certain medical facilities. This classification should be used sparingly. For practical purposes "any conceivable event" may be interpreted as the 1000 year event.

10.5.8 Social implications

Flooding can cause significant stress to individuals, particularly where loss is not insured and the impact may be greater in poorer areas of society. The impact is likely to be affected by frequency and duration of flooding as well as extent. Frequent minor flooding can have long term psychological effects though this can often be overlooked when compared with the less frequent but greater impact flooding.

In some cases the public will adapt to frequent minor flooding by implementing their own remediation measures such as temporary flood barriers. The response to and impact of flooding will therefore depend on the social and economic background of the people affected.

Currently there is very limited information available on the social implications of flooding and therefore the consequences have not been quantified. This lack of data has been highlighted in the recently published *Foresight report – future flooding scientific study* (Evans *et al*, 2004).

10.6 Calculation of risk

Once the probability and consequence has been assessed, the risk can be calculated. This is done by combing probability and consequence as set out in Figure 10.5. Where there is uncertainty in either the consequence or probability assessment (or both) then this may be represented by a fuzzy area on the figure. If the probability rating is assessed to be between medium and medium high and the consequence rating is between high and very high, then the risk value will fall within a band as demonstrated in Figure 10.5. This results in a risk value within the shaded box in the figure, giving a risk of between medium-high and high.

This EFRA process should be repeated following any new design or changes to a design. Any mitigation affects can then be assessed and identify the new risk value. The aim should be to reduce the risk value to an appropriate value.

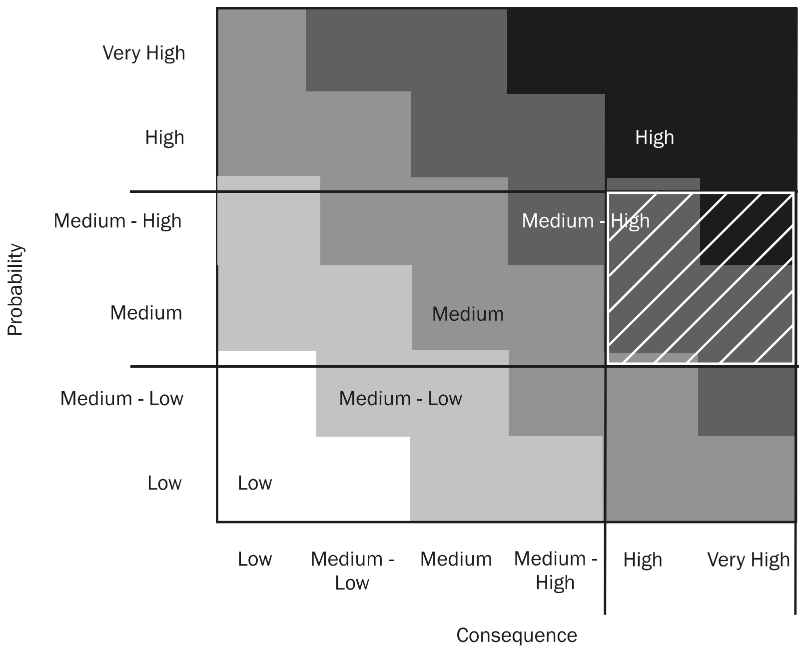


Figure 10.5 *Example of a possible risk band when there is low confidence in the probability and consequence values*

11 Designing for surface conveyance

11.1 Principles of design

This chapter looks in detail at how surface channels for flood conveyance may be specifically accommodated in new and existing development, and how they should be designed to convey the exceedance flow. It also deals with some of the health and safety issues raised in Chapter 10.

Channels designed to function as surface flood pathways during extreme events may, on a day to day basis, serve as:

- highways
- footpaths
- ditches and swales
- car parks
- vegetated channels formed naturally or artificially.

The main use of such pathways is referred to as the primary function, with flood conveyance becoming the secondary function. In exceptional circumstances pathways may be defined where flood conveyance is the primary function.

When designing surface flood pathways for extreme events, the engineer should be aware of the primary function of the proposed pathway. Engineers should not compromise the primary function and due care should be taken of the safety implications of infrequent flooding of a facility normally used for another purpose. For example, the risk of drowning in areas that only occasionally flood may be greater than in areas where water is retained permanently. An integrated approach is required when planning and designing surface flood pathways with building position and street furniture, to prevent flooding caused by such obstacles. This is discussed further in Chapter 13.

Surface flood channels for extreme events should:

- not detract from the primary function except during extreme events
- convey the required exceedance flow
- provide a freeboard to allow for wave action and any uncertainties in design
- limit the depths and velocities so as not to pose undue risk to the primary function, property or the public
- provide a smooth transition from the primary to secondary function and back, ie sudden rises in flood flow/depth/velocity should be avoided
- minimise the possibility of sediments or trash accumulated during extreme events to hinder the proper operation of the flood pathway
- not intercept or block pathways that the public may need to use to escape from flooded areas.

Figure 11.1 shows the inputs, processes and outputs involved in designing the surface flood pathways for conveyance. Figure 11.2 sets out the design procedure in a flowchart. The following text describes the design procedure in more detail. This chapter will focuses primarily on designing surface flood channels in new developments, and it is relevant to designing such channels to retrofit into existing developments. It will also assist in the proper specification of surface channels (designed and default) when modelling surface flood pathways (Chapter 9).

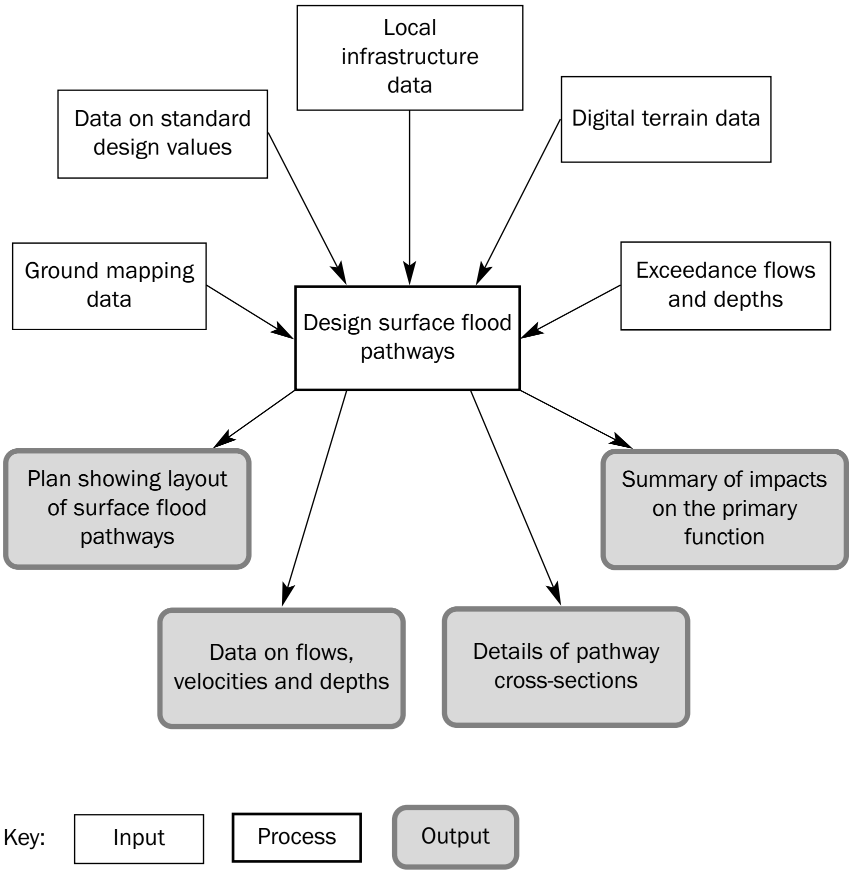


Figure 11.1 *Designing for surface conveyance*

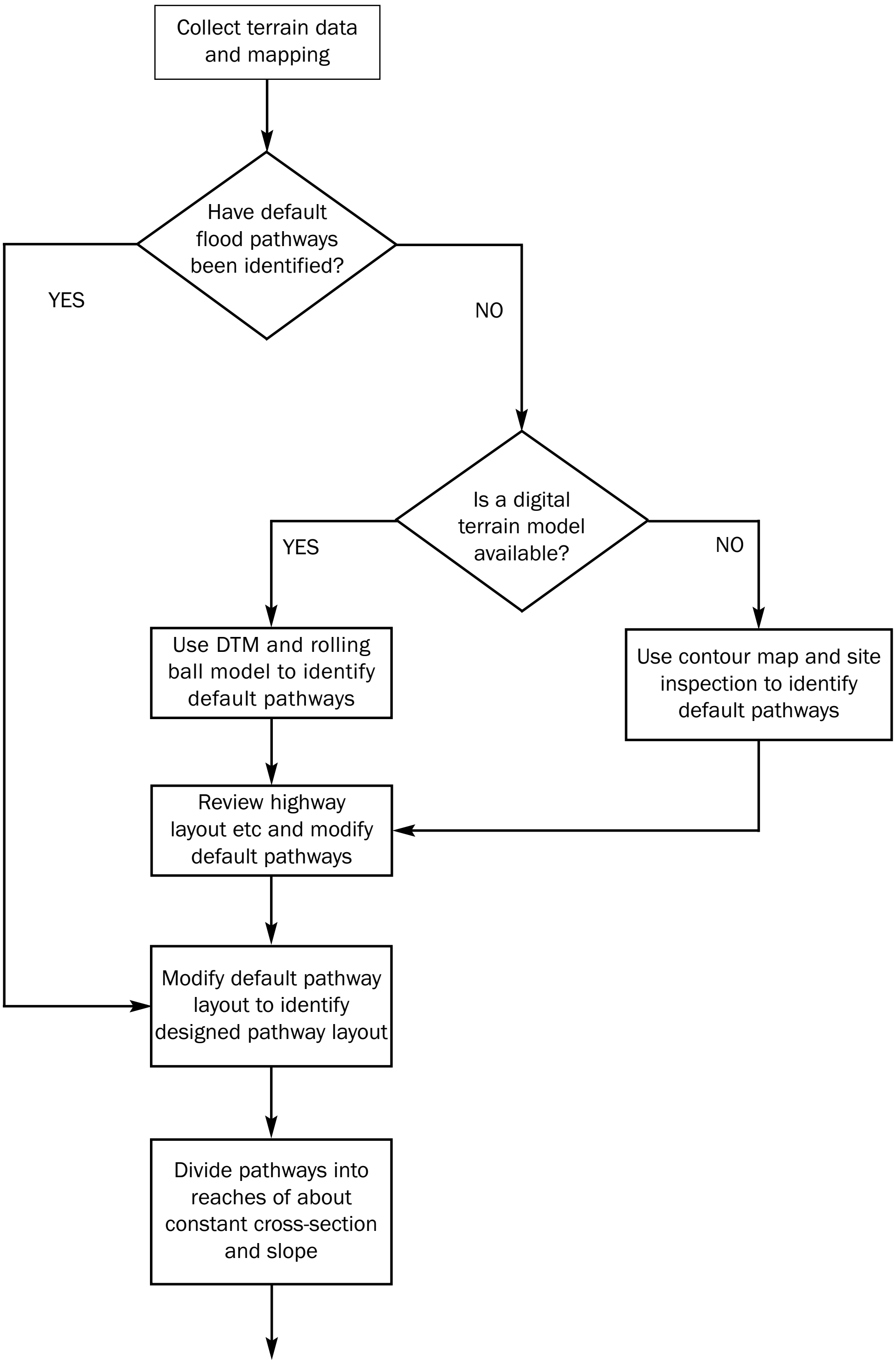


Figure 11.2 *Flowchart for designing surface flood channels for extreme events*

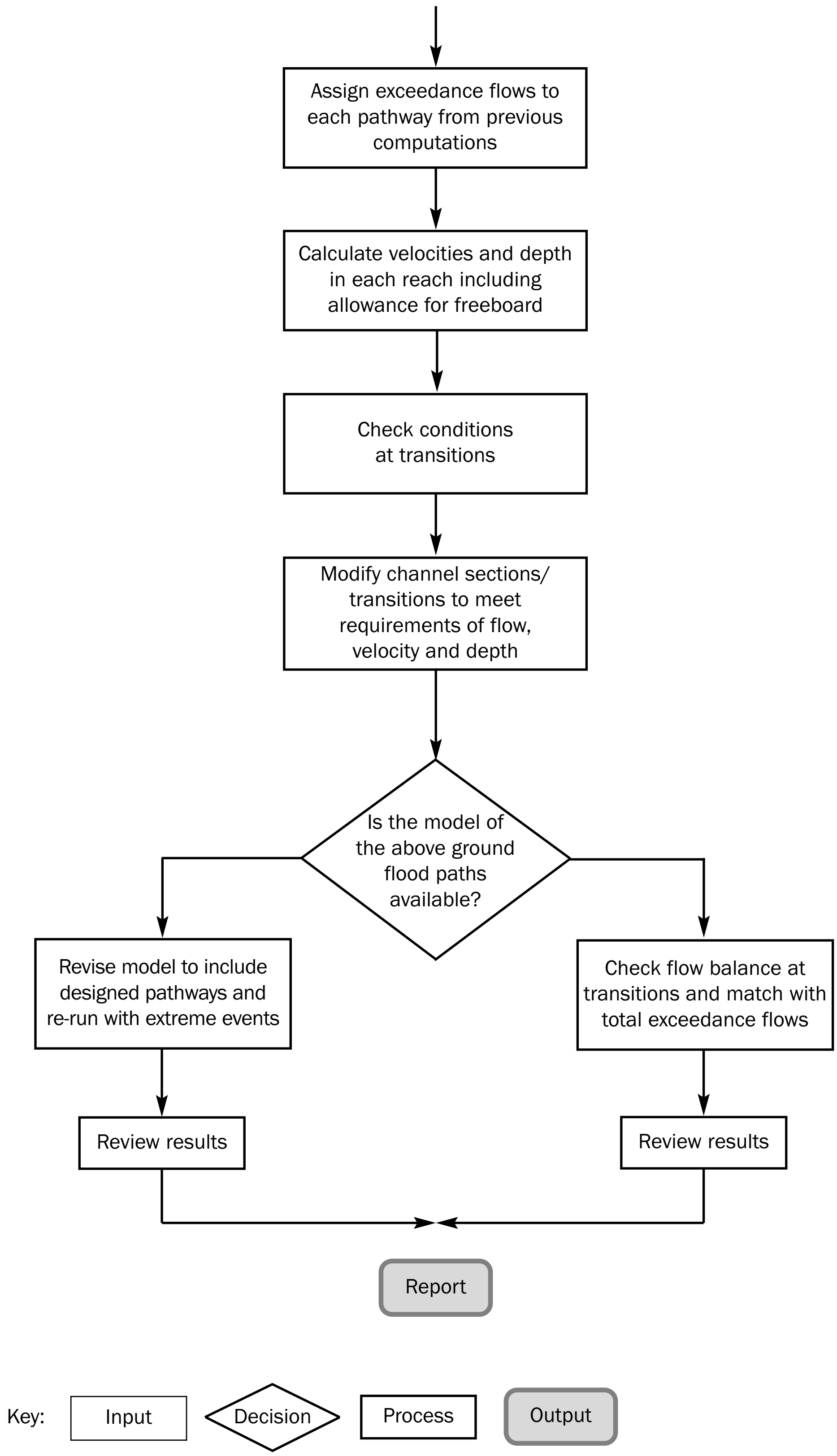


Figure 11.2 (cont'd) *Flowchart for designing surface flood channels for extreme events*

11.2 Identifying flood pathways

On undeveloped sites, natural drainage channels are defined by topography, with water draining to low spots and being conveyed at the bottom of natural valleys. The use of DTMs can aid the identification of existing flood pathways in undeveloped areas. When developing greenfield sites, designed surface flood channels can be most effective when they follow the natural drainage paths. Since these may differ considerably from the infrastructure layout of the proposed development, early consideration of exceedance conveyance in the planning process is desirable.

On developed sites, retrofitting surface flood channels can prove more difficult, with the engineer largely being restricted to adapting existing highways, paths and spaces between buildings. In either case, building layout, road design and other barriers or channels will influence the resulting network of surface flood pathways. This is dealt with in more detail in Chapter 13.

Where exceedance flows have been calculated using a drainage network model, an initial assessment of flood pathways (default pathways) will have been made. When designing new or modified surface flood pathways, the layout and cross-sections should be based initially on these assessments. It may be necessary to make changes to the direction and capacity of pathways in order to provide the desired level of protection to property and the public (ie reducing the flood risk to a desirable level). This may result in the identification, and subsequent modification and construction of new flood pathways and the abandonment of some default pathways.

Where default flood pathways have not been previously identified, an initial assessment of the site should be made. This should include a review of the topography of the site, and if digital terrain data is available, a rolling ball model can be used to make an initial assessment. However, flood pathways may be extensively modified by highway layout, paths and other artificial landscape features, and default pathways should account for this. Ideally these will have been confirmed by observation during wet weather and through meetings with local residents.

11.3 Designing flood channels

11.3.1 Channel conveyance

The channel forming the flood pathway may be designed using the Manning Equation set out in Equation 9.3. This equation is based on the assumption that uniform flow occurs in the channel. This requires a constant cross-section, surface roughness and slope. In practice all three of these will vary somewhat, and average values have to be used. However there will be cases where significant changes in any of these variables make an averaging process inappropriate. An example would be a significant change in channel slope, such as occurs when a road changes direction from running with the contours to cross them. Before calculating channel capacity the design surface flood channels should be divided into separate reaches where the cross-section, surface roughness and slope can be considered to be sensibly uniform.

$$Q = \frac{1}{n} A_C R^{\frac{2}{3}} s^{\frac{1}{2}} \tag{9.3}$$

where:

Q	=	discharge, m^3/s
n	=	Manning roughness value
R	=	hydraulic radius = A/P
s	=	slope (decimal)
AC	=	cross-sectional area of flow, m^2
P	=	wetted perimeter, m

The exceedance flow to be conveyed is represented by the discharge Q. For a given cross-section shape, the depth is determined using Equation 9.3. The equation is solved by trial and error (ie various depths are chosen), the corresponding section properties calculated, and the discharge determined. This is repeated until the discharge calculated matches the exceedance flow to be conveyed.

When using the Manning Equation the roughness value n should be chosen with care. Typical values of n for different types of channel surface are given in Table 11.1.

Table 11.1 *Values of Manning roughness coefficient 'n' for use with Equation 9.3*

Surface type	n
Rough concrete (unfinished)	0.014 – 0.020
Smooth concrete (float or slip formed finish)	0.009 – 0.020
Paving flags (well laid with mortar joints)	0.015 – 0.020
Hot rolled asphalt	0.013 – 0.016
Surfaced dressed	0.017 – 0.025
Well formed setts	0.018 – 0.030
Mowed grass (in artificial grass lined drainage channels)	0.057 – 0.061
Unmown grass (artificial grass lined drainage channels)	0.067 – 0.083

Note: When choosing an appropriate n value the engineer should assess the smoothness of surface finish against the norm that can be expected for the surface type, using higher n values for rougher finishes. For grass lined channels the lower values refer to rye grass and the higher values to fescues (Chow, 1959; Escarameia and May, 1996; Escarameia *et al*, 2002).

Calculated depths may also be affected by local disturbances. Evidence suggests that flood flow depths temporarily increase significantly due to wave action from moving vehicles for example (Figure. 11.3). The calculation of conveyance capacity also carries a significant amount of uncertainty and when designing flood channels, the engineer should include some freeboard to allow for these factors. Freeboard equal to 25 per cent of the required flood depth is considered to be reasonable. If the flood pathway is contained entirely within a highway bounded by 100 mm kerbs, the maximum design depth of flow would be 80 mm. Even with this allowance, occasional overtopping of the channel due to flood waves from moving vehicles can be expected. Where this is likely to cause a significant increase in flood risk (such as where property doorways open directly onto the footpath at footpath level), then consideration should be given to restricting vehicle movements during extreme events by reducing vehicle speed.

Figure 11.3 *Flood wave caused by moving vehicle*

11.3.2 Velocity and depth of flow

Achieving sufficient conveyance capacity is not the only criteria for designing flood channels. Surface flood channels should operate so as not to expose the public and their property to undue risk. This requires the depth and velocity of flow to be limited, see Chapter 10 for more information. For the purposes of design, the following limits (Nania and Gomez, 2002) should be applied:

- flooding over property thresholds and minimisation of traffic disruption. The depth of flow in a surface flood channel is limited to 0.3 m (300 mm) or 0.2 m where a highway forms part of the flood channel
- risk of the flow pushing pedestrians over. The product of depth × velocity shall be limited to 0.5 m^2/s (Nania *et al,* 2002)
- risk of pedestrians slipping. The product depth × $velocity^2$ shall be limited to 1.23 m^3/s^2 (Nania *et al,* 2002).

The depth of flow is determined when designing the conveyance capacity of the channel (see preceding section). For the purposes of this section, the depth should include the freeboard. The velocity is determined by dividing the discharge by the cross-sectional area of flow. For the purposes of this section, the cross-section area of flow should not include the freeboard.

Where the depth, velocity or their product fail to meet the design criteria, the channel section should be re-designed so as to meet the above criteria. Since velocity is very dependent on channel slope, it may be necessary to re-route the channel, avoiding particularly steep topography, and meet the depth × velocity criteria.

11.3.3 Cross-section details

Experience of surface flood flow shows that the capability of flood pathways to convey flow is dependent on the detail of the channel section. Details that appear unimportant to the inexperienced eye, can have a significant impact on flood risk. The more detail that is incorporated into the modelling or design process, the better the results will be. However, it is recognised that collecting data and building complex models can be time consuming and costly, and as with other parts of the process, the level of detail should be tailored to the overall accuracy of the results required. Less detail could be used with level 2 studies, where surface pathways might initially be represented by simple rectangular channels, while in level 3 studies the actual cross-section shapes might be represented.

An illustration of floodwater entering property situated below carriageway level is shown in Figure 5.5. Flood water is inadvertently diverted over a dropped kerb onto the footpath at the top of the picture. The flow then travels downhill, contained between the boundary wall and the raised kerb, and diverts over the footpath back edge where the boundary wall finishes. Such flooding can be avoided with careful design of the highway and pathway sections, so they contain the flood flow. In particular, dropped kerbs and surface cross falls need to be carefully specified. Examples of suitable channel sections, together with explanatory notes, are contained in Appendix 3.

11.4 Channel transitions

11.4.1 General principles

Transitions occur at nodal points in surface flood network. They occur at junctions between individual channels, at road intersections and other similar junctions, and at inlets and outlets. The hydraulic conditions at these transitions can significantly affect the performance of surface flood pathways. To understand the hydraulics of transitions it is first necessary to distinguish between the two types of flow that can occur in open channels. These are:

- subcritical flow
- supercritical flow.

Subcritical flow occurs when the velocity of flow is less that the critical value given by Equation 11.1.

$$V = \sqrt{g\frac{A}{W}} \tag{11.1}$$

where:

V	=	critical velocity of flow, m/s
A	=	cross-sectional area of flow, m^2
W	=	width of the water surface, m
g	=	gravitational acceleration = $9.81 m/s^2$

Subcritical flow is characterised by tranquil flow with low velocities and larger depths. When a channel carries subcritical flow its slope is said to be mild.

Supercritical flow occurs when the velocity of flow is greater than that given by Equation 11.1. Supercritical flow is characterised by shooting flow with small depths and large velocities, and often entrains air to give a “white water” effect. When a channel carries supercritical flow, its slope is said to be steep.

The geometry of the flow boundaries at transitions significantly affects their performance. Changes of section should be made with sweeping curves. Sharp radius bends and abrupt changes of section should be avoided, especially where the flow is expanding (Chow, 1959). Failure to do this will result in unnecessary energy loss and this in turn will restrict the overall conveyance capacity of the system. Channel surfaces in transitions are prone to erosion, and consideration should be given to the erosion resistance of materials used to form the transition (Hall *et al,* 1993).

11.4.2 Transition between single channel reaches

Where a single channel reach changes directly into a second reach (the reaches being distinguished by change in cross-section and/or slope) there are four possibilities.

Both reaches are mild sloping (flow subcritical). Providing the recommendations on transition geometry given above have been followed, the water surface and velocity will change smoothly and progressively at the end of the upstream reach. There will be minimal energy losses. The depth at the transition will be determined by the depth of flow in the downstream reach.

Both reaches are steep sloping (flow supercritical). Providing the recommendations on transition geometry given above have been followed, the water surface and velocity will change smoothly and progressively at the beginning of the downstream reach. There will be a small energy loss. The depth of flow at the transition will be determined by the depth of flow in the upstream channel.

Upstream channel mild, downstream channel steep. The flow changes from subcritical in the upstream channel with the surface drawing down and the flow accelerating as it becomes supercritical in the downstream reach. The depth at the transition is defined by the "brink" depth, that is the critical depth.

The critical depth dc is given by:

$$dc = \left(\frac{Q^2}{gb^2} \right)^{\frac{1}{3}} \qquad (11.2)$$

where:

dc	=	critical (brink) depth, m
Q	=	discharge in channel, m^2
b	=	average width at entry to downstream channel, m
g	=	gravitational acceleration = $9.81 m/s^2$

The flow accelerates further, reducing in depth along the downstream channel until it reaches its normal depth as defined by the Manning Equation (9.3).

Upstream channel steep and downstream channel mild. The faster flowing supercritical flow meets the slower flowing subcritical flow at the transition and a standing wave is formed. This is known as a hydraulic jump. There is considerable energy loss in the jump with the energy level dropping to that in the downstream channel (Chow, 1959).

11.4.3 Road junctions

Flows at road junctions can be extremely complex, being governed by the relative flow rates and slope of each channel entering and leaving the junction and the junction layout (eg cross roads, T junctions). Flows do not follow the transition principles in Section 11.4.2 because momentum also plays a part in determining the flow split.

In order to simplify the analysis, road junctions may be represented as 90° four way junctions (Figure 11.4) and the area of the intersection assumed to be horizontal. Previous research on four way junctions (Nania *et al*, 1999) showed that the ratio of outgoing flow to incoming flow could be related to the relative channel gradients. The results are reproduced in Figures 11.5a and b, and are limited in the range to which they apply. In the absence of other information they provide a useful starting point for determining the division of flow at road junctions. At important or more complex junctions, engineers may use field observations or 2D analysis by computational fluid dynamics (CFD) to determine more accurate information.

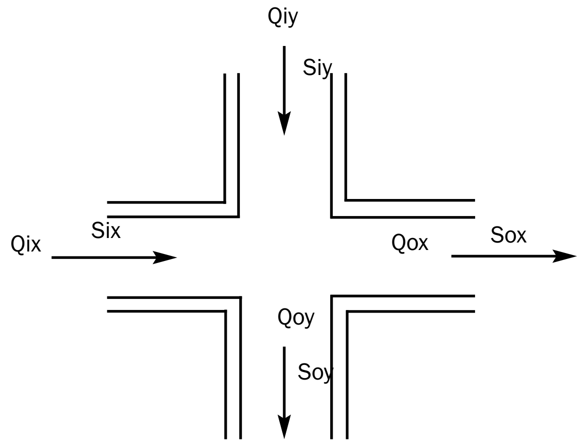


Figure 11.4 *Definitions of flows and slopes at four way road junction, after Nania* et al *(1999)*

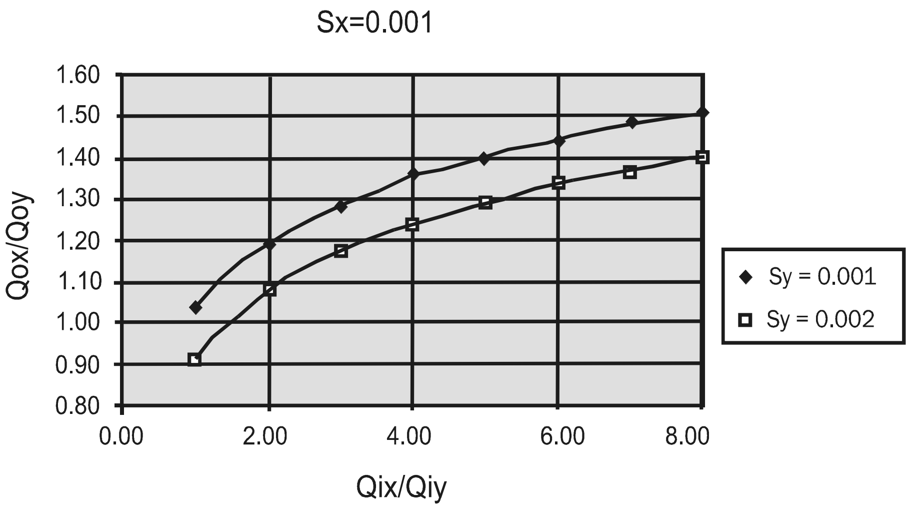


Figure 11.5a *Flow split at four way junction, subcritical flow, after Nania* et al *(1999)*

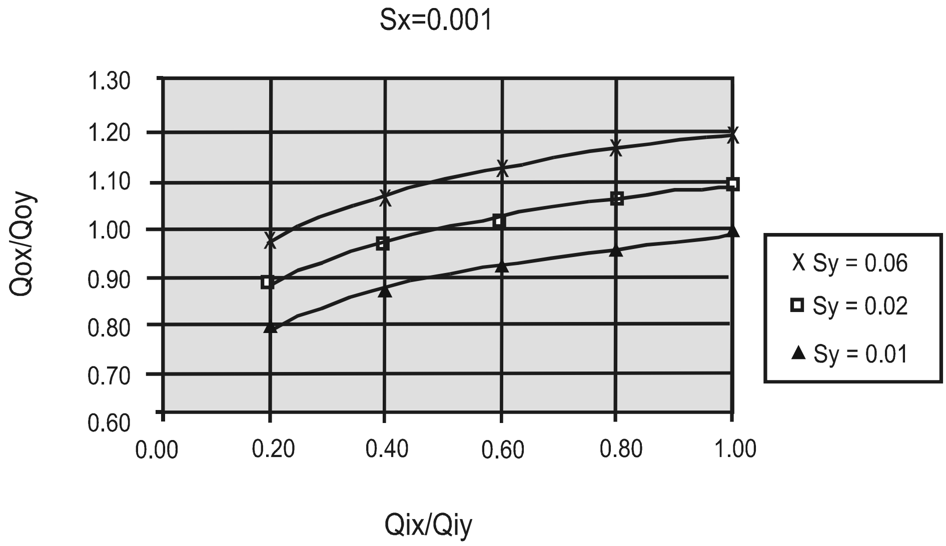


Figure 11.5b *Flow split at four way junction, supercritical flow, after Nania* et al *(1999)*

11.4.4 Inlets

Flood flow may accumulate on the surface before it is discharged to a surface flood channel. This will occur, for example, where flow is discharged from a manhole onto a paved surface area that is near horizontal and surrounded by higher ground. There is a risk in such cases that the level of the flood water will rise above thresholds of adjacent property and cause flooding, or the depths created will present a risk to pedestrians and vehicles using the area (see Section 11.4.3). The depth of water that will accumulate depends on the relative level of the invert of entry to the flood channel(s) and the flow discharged to the channel.

The water level required to discharge the excess flood flow into the flood channel is dependent on whether the flood channel is mild or steep. For a mild sloping channel, the water surface will be determined by the depth of flow in the flood channel. 1.5V2/2g (V= velocity of flow in flood channel) should be added to this to allow for changes in energy level as the flow enters the channel. This figure includes 0.5V2/2g of energy loss as the flow passes to the flood channel. This is illustrated in Figure 11.6a.

For a steep sloping flood channel, the depth at entry to the channel will be the brink or critical depth as defined by Equation 11.1. The water level will be 1.5dc above the invert (see Figure 11.6b). This analysis assumes that there is no energy loss as the flow passes into the flood channel.

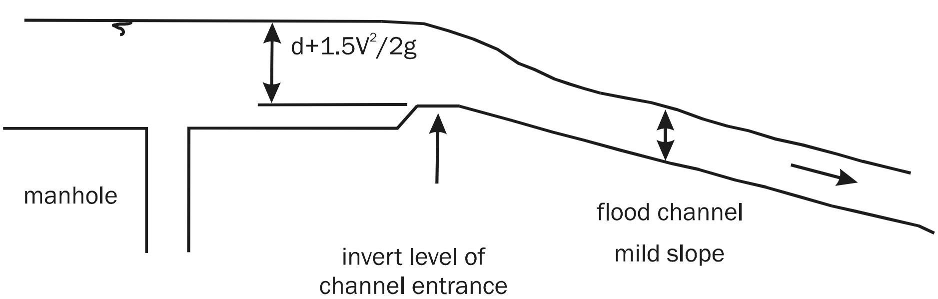


Figure 11.6a *Depth of ponding governed by discharge to a mild sloping flood channel*

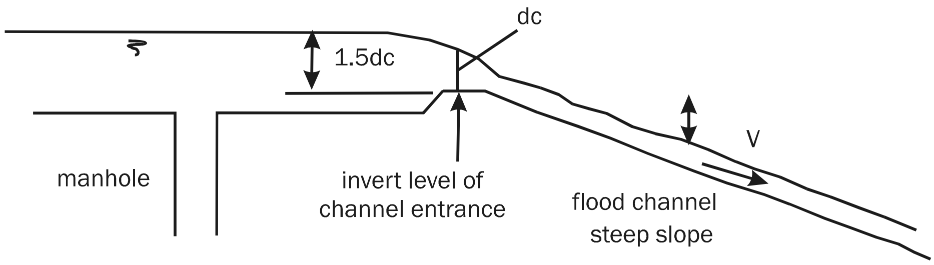


Figure 11.6b *Depth of ponding governed by discharge to a steep sloping flood channel*

The more sophisticated drainage network simulation software will automatically allow for these effects. Where the user is required to specify energy loss coefficients, care should be taken to chose suitable values so that the simulated results are consistent.

Where more than one outlet exists, the flow split to each channel can be calculated by assuming a ponded water level, calculating the flow passed to each flood channel using the above methods, and then checking the total flow with the total exceedance flow at that node. The calculations are reworked with different levels until the flows balance. This is a tedious process and in such circumstances the engineer is advised to use an appropriate simulation software tool.

11.4.5 Outlets

The velocities in surface flood channels will normally be limited by the criteria set out in Section 11.3.2. They may be discharged directly into storage ponds, for example, without any particular measures to dissipate the velocity energy in the flow, though a Reno mattress protecting the bed of the pond at the outlet from the channel would be prudent. Where the flood channel is mild sloping then the transition into the pond will be smooth, with the water surface slowly increasing to the level of the pond as the flow approaches.

With steep flood channels however, a hydraulic jump will occur at the entry to the pond. Consideration should be given to additional protection on the ground surface in such cases, perhaps with the provision of a concrete apron or rip-rap.

Greater care needs to be taken when discharging directly to a watercourse. With small watercourses, flood flows with velocities as low as 1 m/s can still cause significant damage to the stream bed. A properly designed outfall structure is recommended in such circumstances. Further guidance on this may be found in B14 *Design of flood storage reservoirs* (Hall *et al*, 1993). Consideration of the potential effects of the additional flood flow on the watercourse is considered in Chapter 14.

Surface flood channels used only to convey extreme events may operate infrequently and are likely to convey considerable quantities of silt and trash. This can block outlets and have detrimental effects on storage pond outlet structures, culvert screens and downstream watercourses in general. It will also affect maintenance requirements. Further information on this is provided in Chapter 12.

12 Designing for surface storage

12.1 Principles of design

During extreme events, excess flow (exceedance flow) will be conveyed on the surface. In a managed major drainage system, as advocated by this guide, this flow will be conveyed in surface flood channels specifically designed for that purpose. However, it may be necessary to store some of the flow above ground for two reasons: because the capacity of designed flood channels is limited by economic constraints, or there is a requirement to protect and not overload the downstream receptor system (see Figure 12.1 and Chapter 14):

- it will always be above ground
- it will be utilised infrequently
- the area set aside for storage may normally have a different use (primary use) eg a playing area
- the primary use may not be available during storage operation.

Examples of areas that may be set aside for exceedance flood storage as a secondary use are car parks, parkland and minor highways. Further information on this is given in section 12.4. Figure 12.2 shows the inputs, processes and outputs involved in designing the surface flood pathways for surface storage. The recommendations in this chapter refer to both new development and retrofit applications in existing developments.

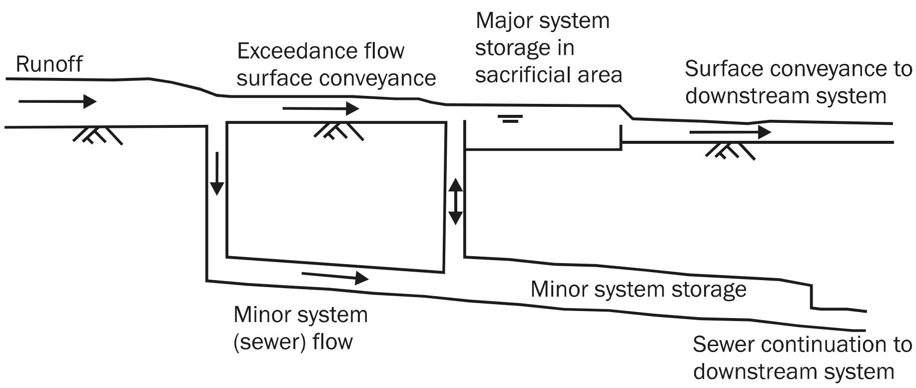


Figure 12.1 *Concept of how exceedance flows are managed*

12.2 Storage area design process

12.2.1 Size

The design of surface storage areas is primarily controlled by the size required and where storage can be utilised. The flowchart in Figure 12.3 sets out the design procedure. Due to the complexity of knowing the volume and flow conveyed away from the area, it is likely that in all but the simplest of cases a hydraulic computer simulation model will be used to determine the volume required. This will be used to replicate at least the above ground major system, and preferably the minor system and the interaction between them. The volume that needs to be stored and conveyed is demonstrated conceptually in Figure 12.4. The design may include a provision to drain the area after the event, and the drain can be manually or automatically operated. The capacity of the drain should be based on the maximum permissible flow that can be discharged into the downstream system during the exceedance event.

In some cases it will not be possible to accommodate a drain to the downstream drainage system. This is usually because the storage area is below the invert of the downstream system. In such cases stored water will drain slowly into the ground and also evaporate. Storage areas that perform in this way are known as sacrificial areas as their primary function (if any) may not be available for a considerable period. Sacrificial areas of this nature should only be considered on low value land which is not readily accessible to the public. Large storage depths (above say 500 mm) should be avoided. Sacrificial areas should not be used where flood water is expected to contain foul flow.

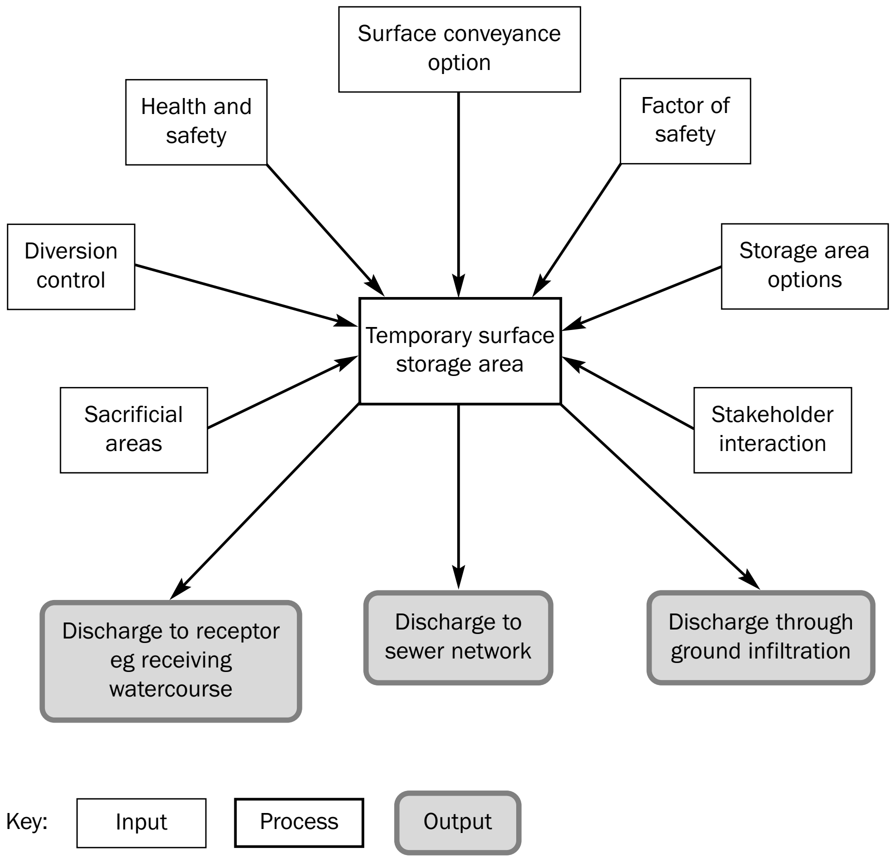


Figure 12.2 *Designing for surface storage*

The storage volume required will depend on the available conveyance capacity and the volume of surface runoff, the latter depending on the design storm used. Guidance for suitable return periods for exceedance design is given in Chapter 3.

Where surface storage is being provided as part of the conventional drainage system, such as in a surface storage pond, then it is often cost effective to provide the necessary additional capacity for storing exceedance flows through increasing its capacity. Further information on this is given in Section 12.3.2 and in the *Interim code of practice for SUDS* (National SUDS Working Group, 2004).

Storage volumes should be calculated for various duration events to determine the critical duration for the design, ie the duration generating the maximum storage volume requirement.

Design return periods for when storage options may be utilised are identified in Section 12.3.1. The upper limit advocated by this guidance is the one in 200 year return period event. However it may not be possible for a single storage area to have this capacity. In this case the required storage volume should be distributed over more than one area.

In general the design of surface storage areas should be in accordance with relevant local and national practice/guidance and regulations in particular the Reservoirs Act, 1975 (where applicable). Further useful guidance can be found in B14 *Design of flood storage reservoirs* (Hall *et al*, 1996). Each individual solution type will have some common aspects and these are indicated in Figure 12.3.

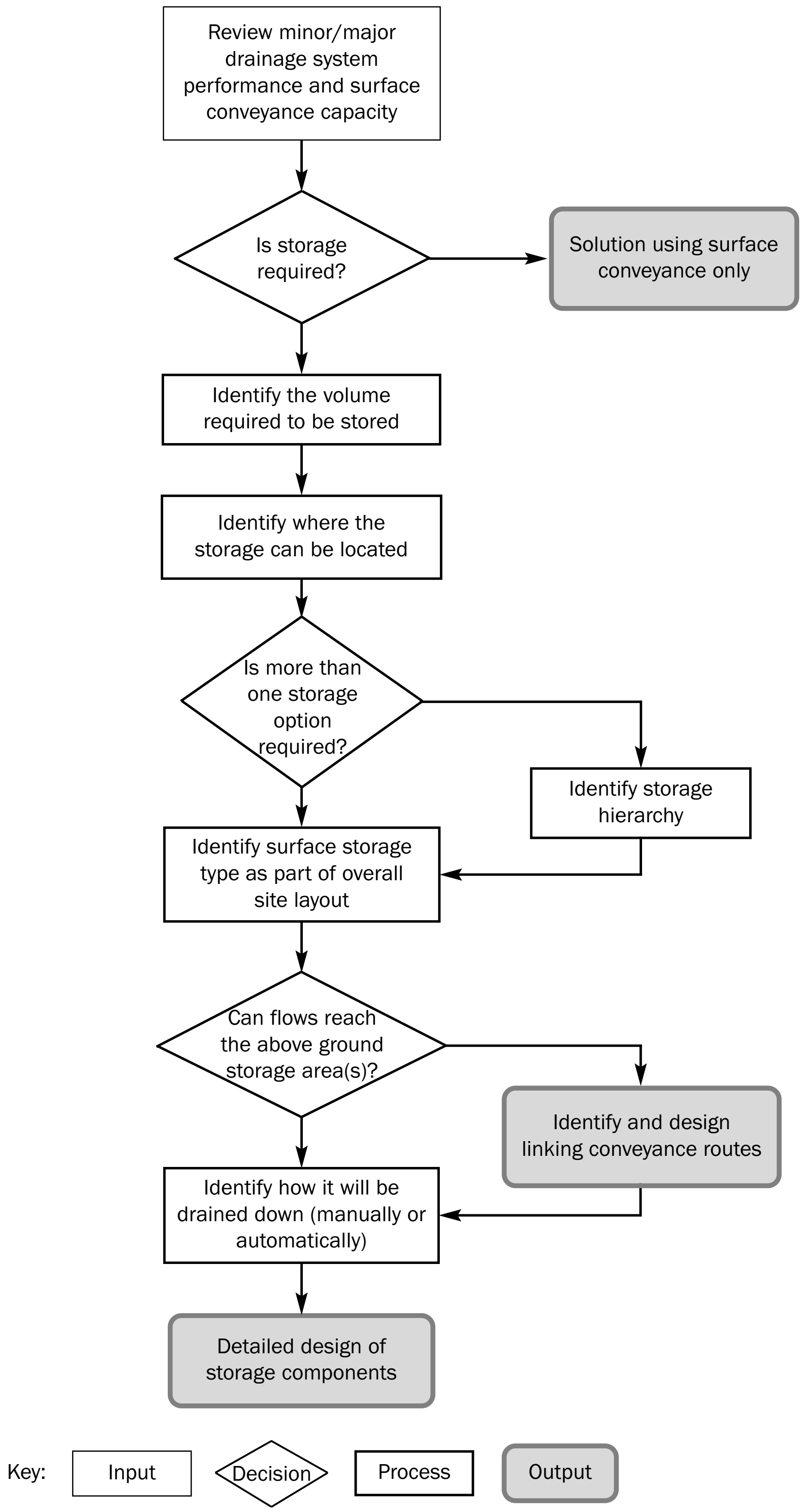


Figure 12.3 *Flowchart of the above-ground storage design process*

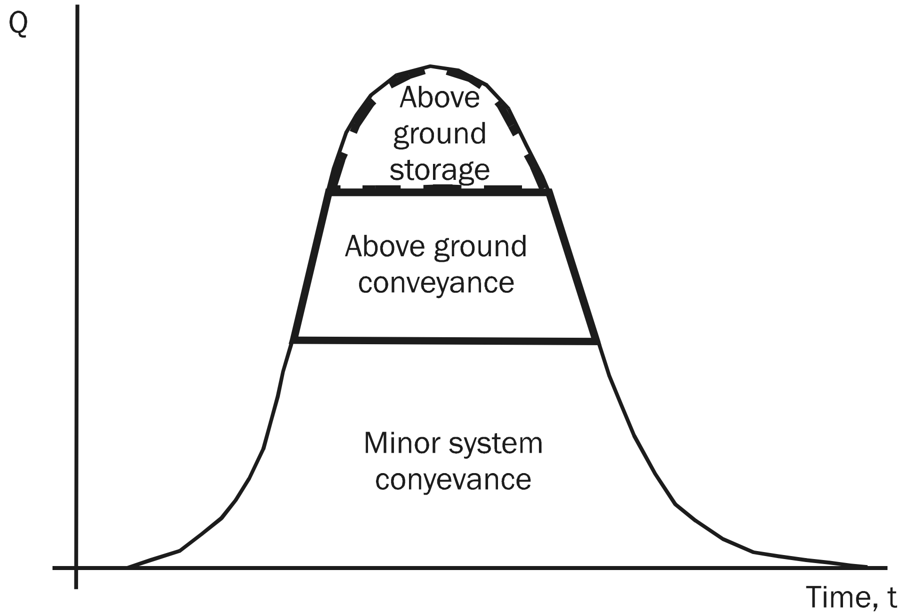


Figure 12.4 *Conceptual demonstration of the exceedance rainfall event hydrograph*

12.2.2 Health and safety

Above-ground storage structures should be designed to minimise risk especially to the general public. This will apply to those constructing, operating and maintaining the structure as well as the public who may use the area. Careful design can eliminate or reduce such risk. Particular attention should be given to areas such as playgrounds that may be used by children. Table 12.1 summarises the potential hazards and gives guidance on best practice for managing risk.

Table 12.1 *Key health and safety areas that should be considered in the design of exceedance storage areas*

Hazard	Description
Access/egress	This should be through designated points and where practical are separate from where water enters or leaves the area. Where applicable sloping banks may be used with a maximum of one in four slope, with specific areas for escape, and access for maintenance vehicles at 1 in 12 such as in playing fields. Land should be graded so as not to leave islands where people can be stranded.
Water depth	Water depth will depend upon the area selected to retain water (see Table 12.3). It is important to set a maximum depth and where applicable have a specific overflow point that is utilised once the storage is full. This may only be applicable for areas that are designed to operate between fixed return periods (eg >1 in 30 to 1 in 75). Marker posts could be positioned in the areas to indicate the depth of the water.
Water velocity (entering/ draining)	Flow velocity may be high, particularly at those points where flows enter into or are drained from the area. These areas should be partitioned off if the velocities are found to be high. High velocities may also contribute to scouring of areas. Although its operation will be infrequent, the area should be checked after it has operated to determine if any remedial works are required. Where possible all velocities should be designed to be within those stated in Chapter 11.
Tripping hazards	Small tripping hazards that become submerged and unseen when flows are stored should be avoided, for example drop kerbs in a car park. Ground should be even under foot wherever possible, with only gradual changes in level. Objects that project above the maximum water depth will remain visible and will not become a hazard.
Change from primary use	Signs indicating that areas have a dual purpose should be clearly displayed to highlight that areas may flood in very heavy rainfall. Designated access and egress points from the area should be clearly signed in case of an emergency. Any overland flow paths feeding into the areas should also be signed.
Sediment and trash	After an event where storage has been utilised, a clean up operation should be undertaken. Significant sediments may have been deposited in an area and these will need to be removed. Repairs to the areas may be required if high velocities or moving debris within the flow has caused damage. Areas that have contained foul sewage may need to be treated to reduce the risk to public health.
Inlet/outlet structures	Ideally, risks associated with inlet/outlet structures should be designed out. Where an outlet forms part of the bank, it should slope with the bank. If penstock controls are used, these should be located within inspection chambers.

Further guidance on the design of storage may be found in CIRIA publications *Sustainable urban drainage systems* design manuals (Martin *et al,* 2000a, b, 2001), C609 *Sustainable drainage systems. Hydraulic, structural and water quality advice* (Wilson *et al*, 2004) and B14 *Design of flood storage reservoirs* (Hall *et al,* 1996).

12.2.3 Maintenance

Regular maintenance of surface storage areas should be undertaken as part of the primary drainage function. The maintenance regime should reflect the secondary (storage) function requirements. However best practice storage design should minimise maintenance requirements. As with any form of emergency facility, maintenance will be limited to regular inspection on operability (eg to ensure inlets and outlets are not obstructed), and post operation inspection, clean up and remedial works. A summary of key maintenance items is linked in Table 12.2. Further guidance on the maintenance of storage areas may be found in CIRIA publications on SUDS and B14 *Design of flood storage reservoirs* (Hall *et al*, 1996). This applies to the maintenance of the whole area including the outfall and any control structures.

Table 12.2 *Key maintenance considerations required in the design of exceedance storage areas*

Requirement	Description
Drain down	Due to the infrequent nature of operation, a manual drain down of areas may be required, eg through pumping. If this is necessary an agreement as to when and who will carry out the work is necessary and should be determined prior to the designation of such areas. This responsibility is likely to rest either with the sewerage undertaker, local authority or the environmental regulator. Areas that naturally drain down will need to be checked to determine if this has been achieved. The rate of drain down should be checked to minimise the risk of stagnation or septicity of the water.
General inspection	Although some areas such as playing fields and parkland may be routinely inspected, further inspection may be required after such areas have been used for storage. This should include checking for damage to the structure likely to be caused by high velocities or floating debris in particular to dedicated access/egress points, inlet/outlet controls and sloping banks. General inspections should also take into account impacts that may arise during its primary use.
Inlet/outlet controls	Inlets and outlets should be checked on a regular basis as part of the general maintenance of an area and also after an event has occurred where storage has been utilised. Any blockages or debris positioned in and around the controls should be removed. Velocity of flow entering or leaving the area should not exceed the values suggested in Section 11.3.2.
Cleaning	Significant sediment deposition is likely in areas used for storage after an event, therefore a post clean up operation may be required. The removal of litter, vegetation, sewerage debris and larger objects may be necessary Where areas have been used to contain combined sewage, they will need to be washed down and disinfected after use.

12.2.4 Outfall design

The practical means of draining exceedance storage areas are:

- direct gravity connection to the sewerage system
- automatic gravity connection to receiving water
- manually operated gravity connection to receiving water
- emptying water by pump or pumped emptying
- infiltration to groundwater
- evaporation (sacrificial areas only).

The simplest method of draining surface storage areas is automatically and by gravity. Outflow may be connected directly to the downstream sewer system or receiving water, but in either case the impact on these systems should be assessed, as described in Chapter 14. This is likely to be the preferred method when considering car parks, minor roads, recreational areas, industrial areas and SUDS. Draining of a storage area may also be achieved through infiltration, however the drain down time using this method is likely to be large. Where there is a high risk of combined sewage flooding occurring, draining to a combined or foul sewerage system is preferable to draining to a watercourse.

A manually operated gravity connection to a receiving water may be used for draining parkland and playing fields. A manual penstock housed within an inspection chamber is likely to offer the best control while being protected from vandalism. An operating protocol should be agreed with the body responsible for operating the receptor system.

Pumping of stored water may occur if the likelihood of the area being used is remote and no direct route to a receiving water or sewer is possible. If combined sewage is stored and flow cannot be returned to a combined or foul sewerage system, it should be pumped or tankered directly to a treatment works.

The draining down of such an area should be carefully considered. The receiving system should have the capacity to receive the flows, and drain down should be achieved ideally within a 48 hour period (eg 5.8 l/s over a 48 hour period will pass 1000 m^3). Further guidance on this is given in Chapter 14.

In designing the outfall control, the receiving sewer or water should be studied to check that reverse flow into the structure is not possible. If the outfall control is permanently closed during operation, and only opens to drain down, then this is unlikely to be problematic. Further advice to design outfalls is available in C609 *Sustainable drainage systems. Hydraulic, structural and water quality advice* (Wilson *et al,* 2004) and B14 *Design of flood storage reservoirs* (Hall *et al,* 1996).

12.2.5 Diversion control design

Although storage areas will be designed to accommodate extreme events, it is possible that the designed area is itself exceeded. This is likely to occur if a hierarchical storage approach is taken where specific areas are designed to operate before others (identified in Table 12.3). In this case some areas will fill to capacity and then spill through a diversion control to other conveyance pathways and storage areas downstream, illustrated in Figure 12.5. The design of such diversion controls should remain as simple as possible and be compatible with the general design of the structure.

When designing a hierarchical system of storage structures, care should be taken that the network of major system conveyance and storage adequately handles flow up to the design return period.

Uncertainties involved in estimating exceedance flows and designing storage facilities, means that an adequate allowance for freeboard should be incorporated into all storage and diversion structures. For exceedance events a value of 100 mm is recommended.

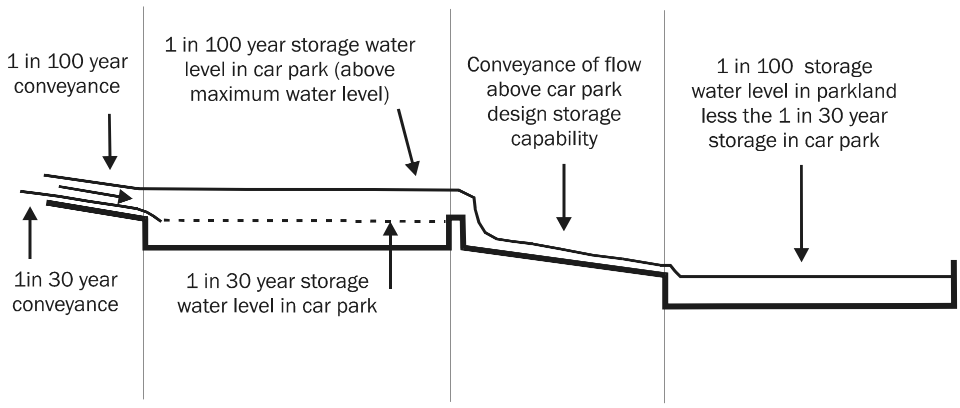


Figure 12.5 *Conceptual process of diverting excess flow*

12.3 Types of storage areas

12.3.1 Storage options hierarchy

Although a number of options exist for storage of above ground flows, certain areas should be utilised before others. Table 12.3 summarises the different type of storage facility and gives guidance on acceptable frequency of flooding. This allows the designer to identify a hierarchy of operation where more than one type of area is available. When selecting a storage area type, consideration should be given to the length of time that the area may not be available for its primary function. This period may be longer than the storm duration due to drain down and clean up operations. More detailed descriptions of each type appear in subsequent sections.

Table 12.3 *Summary of different storage options available*

Storage area type (primary use)	Description	Maximum water depth	Acceptable flooding hierarchy
SUDS – detention/ retention ponds, infiltration basins etc	Additional storage used to attenuate peak flows for all storms up to normal design events. Volume of such structures could be increased to retain exceedance event volumes depending upon available area.	Varies depending upon storage area design	> 1 in 30 y SW > 1 in 100 y CS
Car parks	Used to temporarily store exceedance flows. Depth restricted due to potential hazard to vehicles, pedestrians and adjacent property. Could be residential, commercial or industrial.	0.2 m	> 1 in 30 y SW > 1 in 30 y CS
Recreational areas	Hard surfaces used such as basketball pitches, five-a-side football pitches, hockey pitches, tennis courts.	0.5 m unless area can be secured, then 1.0 m	> 1 in 30 y SW only
Minor roads	Minor roads typically where maximum speed limits are 30 mph. Depth of water can be controlled by design.	0.1 m	> 1 in 30 y SW > 1 in 30 y CS
Playing fields	Used for sport such as football and rugby. Set below the ground level in the surrounding area and may cover a wide area and hence offer large storage volume.	0.5 m unless area can be secured, then 1.0 m	> 1 in 20 y SW only
Parkland	Has a wide amenity use. Often may contain a watercourse. Care needed to keep water separate and released in a controlled fashion to prevent sudden downstream flooding.	0.5 m unless area can be secured, then 1.0 m	> 1 in 30 y SW > 1 in 100 y CS
School playgrounds	Hard standing area of schools could provide significant storage. Extra care should be taken when designing such areas due to high number of children.	0.3 m	> 1 in 30 y SW only
Industrial areas	Low value storage areas. Care should be taken in the selection as some areas used could create significant surface water pollution.	0.5 m	> 1 in 50 y SW > 1 in 100 y CS
Major roads/ motorways	Due to their primary function and importance only used for severe events.	0.1 m	> 1 in 100 y SW > 1 in 100 y CS

Key: SW = surface water flooding CS = combined sewerage system flooding y = year

12.3.2 Additional storage in SUDS

When considering the type of areas used to store exceedance events, SUDS arrangements should not be overlooked as these can be very effective in attenuating exceedance flows (Evans *et al,* 2004). The use of such systems to retain and attenuate exceedance flows is becoming more common. Maintenance, ownership of and the legal framework issues surrounding the design and use of such systems is now being resolved using model agreements (Shaffer *et al,* 2004, National SUDS Working Group, 2004).

Much work has already been completed and published to aid practitioners in designing such systems. In terms of above ground storage though, basins and ponds are the most appropriate types. More technical guidance is given in CIRIA publications C522 *Sustainable urban drainage systems – design manual for England and Wales* (Martin *et al* 2000a) and C609 *Sustainable drainage systems. Hydraulic, structural and water quality advice* (Wilson *et al,* 2004).

The design of ponds, detention basins and infiltration basins can be undertaken using the current guidance listed above. SUDS are normally designed to work and accommodate runoff generated from more frequent occurring storms. This needs to be taken into account if such areas are to be upgraded to accommodate additional storage during extreme events.

The calculation of the extra volume available will enable the design of these systems to be more appropriately sized. The likely changes are to be to the plan area, depth and levels of surrounding site. The normal size of a pond or basin may only need a small increase in depth around the perimeter of the normal operating design to provide significant extra storage. This increase in size should be located in the safety bench or in the main area (Figure 12.6). The design of ponds and basins may require an extreme event overflow. This may need to be redesigned if the system is to accommodate exceedance flow. There may still be the need for a diversion control if the volume exceeds the additional amount provided (eg in a hierarchy of surface storage systems).

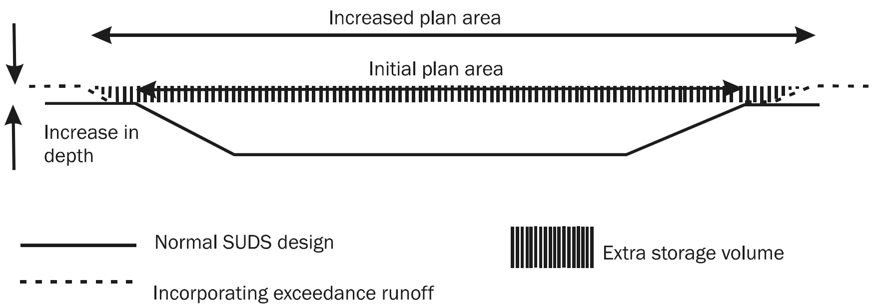


Figure 12.6 *Cross-section through an infiltration or detention pond designed to accommodate exceedance*

12.3.3 Car parks

Car parks may become storage areas for exceedance flows by default during an extreme event. Surface ponding can regularly be seen on car parks that are poorly finished due to uneven surfaces. Discharge from a car park into the minor system can be limited and this can also contribute to surface ponding (Figure 12.7). The volume stored will depend on the perimeter design and in particular the kerb height at the lowest point. Traditionally, the kerb height is designed to be 100 mm to avoid damage to vehicles that overhang the kerb.

Figure 12.7 *Temporary ponding in car park showing the storage potential during an extreme event*

It is possible to increase the storage volume by raising kerbs at specific locations on the perimeter or by grading the surface to increase the kerb level relative to the low point of the car park. A long profile berm (Walesh, 1999) can also be positioned across the entrance to control the stored water. Berms are considered to be similar to speed humps (although their aim is to not act as a speed control), however they may be longer to avoid the discomfort experienced with speed humps. These long profile berms have been successfully used in Chicago in the United States (Walesh, 1999) to temporarily store water during normal and exceedance rainfall events. Health and safety fears such as freezing or standing water has proved not to be a problem to date (Walesh, 1999). An example of a long profile berm is shown in Figure 12.8.

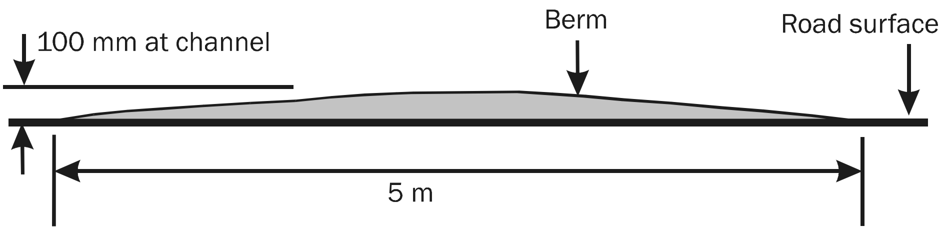


Figure 12.8 *Example berm profile (adapted from Walesh* et al, *1999)*

Due to the nature of car parks it is unlikely that there will be any significant flow velocity, however it is recommended that the depth of storage be limited to 0.2 m (see Chapter 11). This is lower than other figures in use elsewhere with 0.3 m being a typical value (Clark Countym, 1999). However by restricting the depth of water to 0.2 m, damage to motor vehicles should be minimal.

Any surrounding buildings should be set with thresholds above top water level. This offers protection if the water level overtops the kerb level if a flood wave from a moving vehicle is produced. The depth of flood wave is primarily dependent upon the depth of flow and velocity of the vehicle. An initial flood wave created by a moving vehicle reduces and dissipates as the distance increases. Therefore if parking bays are located either side of the access road, any flood damage will be minimal (Figure 12.9). If building thresholds are close to where a flood wave might be generated, the freeboard should be increased accordingly, or vehicle movements restricted during extreme events.

The greatest potential for utilising such storage will be on business and industrial parks, and commercial shopping locations. There may also be potential with communal parking bays in residential areas. The design should take into account the likely use of such a facility. Large car parks offer significant storage volume potential, and a staged or zoned approach could be taken. Areas furthest away from the premises should be designed to flood first with different areas segregated by berms.

Consideration should be given to keeping disabled bays above the top water level.

12.3.4 Minor roads

The use of minor roads for the secondary function of storm conveyance is now open to consultation (Defra, 2004). The use of minor roads for this function in other countries has taken place for a number of years (Walesh, 1999; Clark County, 1999; Nania *et al,* 2002). Minor roads have also been used to store exceedance volume. A recent study in Bangkok indicated that the cost of implementing storage to receive all runoff could be substantially reduced if street storage was employed (Boonya-aroonnet *et al,* 2002).

Utilising storage within the minor road network requires careful design. Consideration should be given to the position of storage locations. If a large number of such locations are selected, the potential storage volume is large. The use of long profile berms in the minor road network, where vehicles should be travelling slowly, may enable additional storage to be provided. An example of a long profile berm to enable storage is shown in Figure 12.10.

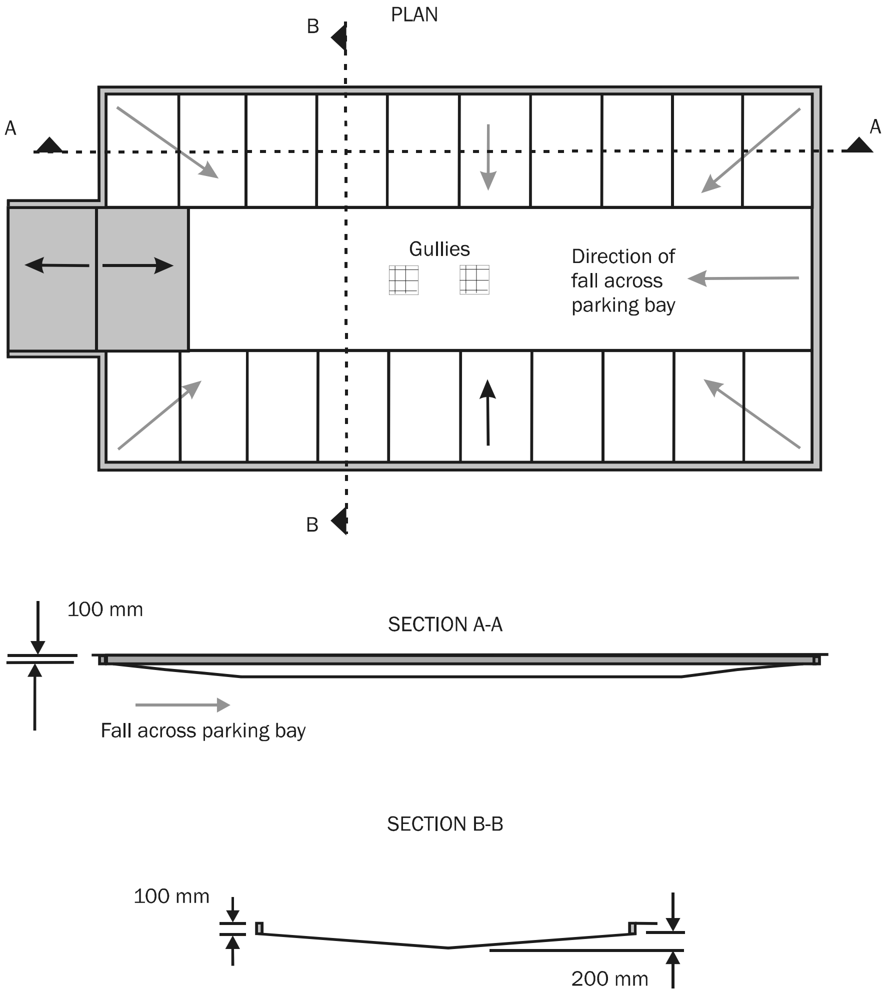


Figure 12.9 *Conceptual arrangements to temporarily store exceedance runoff in a car park*

The positioning of the storage and any long profile berms should be agreed during the layout and design stage and when a known storage volume has been identified. Highway storage should be restricted to residential/business park locations where the speed limit is 30 mph or less, and their position effectively signed. Typical signage used around the world indicates that the road is liable to flooding during heavy rainfall periods and caution should be taken when driving through the area (ie reduce speed). Provided that berms have sufficiently long profile they can made compatible with horizontal alignment designed to control speed. The creation of ponded areas by artificially raising the kerb height is not recommended as this may mislead the motorist or pedestrian as to the depth of water on the highway. Surface storage may be filled and drained from the low points using conventional gullies, though kerb inlet gullies may be more suitable for this purpose.

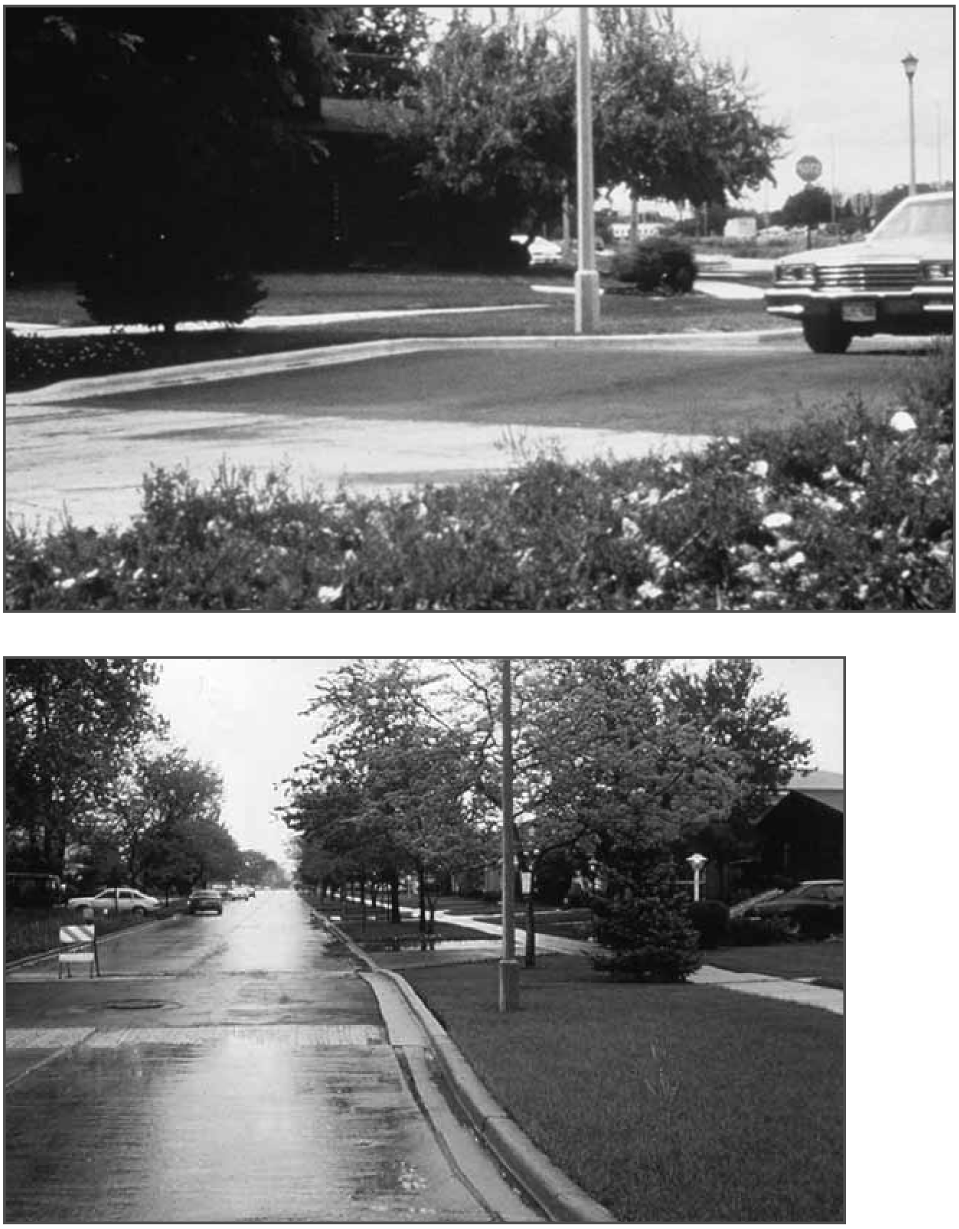


Figure 12.10 *Example of the storage that can be achieved and examples of berms (courtesy Walesh)*

12.3.5 Playing fields, recreational areas and parkland

The use of recreational areas such as playing fields to control and temporarily store surface water flows has been acceptable practice for a number of years (Wisner and Kassem 1982; Hall *et al,* 1996).

By design, the frequency of flooding in such areas will be low. However during an extreme event, significant quantities of water may be stored. The design of conveyance pathways to such areas should be carefully considered to ensure that flows can reach them and not cause flooding elsewhere in the catchment. It is crucial that any parkland, recreational area or playing field used for storage is considered during the early stages of the development process. An example of this is shown in Figure 12.11. Sign posting of such areas is very important and should state that the area is a dual drainage-recreational area and as such could be subject to flooding during wet weather conditions. Recreational areas could include outdoor five-a-side football or basketball pitches that are lower than the ground level and could provide additional storage relatively cheaply.

As these areas are often larger than other flood storage areas, careful consideration should be given to filling and emptying. Public using such areas should have an obvious and direct means of escape when such areas fill. Escape pathways should be kept separate from flood pathways. Velocities during filling and emptying may be significant and the areas should have relatively flat or gently sloping bases. Ground levels should be graded to avoid islands during filling that may leave people stranded.

Figure 12.11 *A dual-purpose recreational baseball pitch/storage area in Japan. This is significantly lower than the surrounding ground level (courtesy Shoichi Fujita)*

13 Building layout and detail

13.1 Design principles

There are numerous pressures applied to property developers and many different design considerations required during the design process. This chapter focuses on issues that relate to property flooding and how small changes in layout and design may improve the level of protection from flooding.

When deciding on the drainage of a new development, the designer should identify and consider the use of natural flood pathways in the undeveloped area. As a general principle, the closer artificial drainage and above ground flood pathways follow the natural layout, the more effective the drainage system will be.

Planning Policy Guidance Note 3 *Housing* (PPG3) places pressure on housing developers to increase the building density to between 30 and 50 dwellings per hectare. This is against a traditional number of 23 dwellings per hectare for suburban estates (CIRIA, 2003c). Increased pressure could lead to a reduction in available space for some SUDS components and above ground storage areas. The guidance provided in PPG25 Development and flood risk (DTLR, 2001) where it encourages the incorporation of measures for effective flood control, may create tension. Other publications such as *Sewers for adoption 5th edition* (Water UK and WRc, 2001) highlights that above ground pathways should be considered during design. BS EN 752-3:1996 also identifies that during the planning stage overland flow paths may influence the site design. CIRIA's publication C624 *Development and flood risk – guidance for the construction industry* (Lancaster *et al,* 2004) addresses this by promoting a consistent approach to planning in relation to flood risk.

Drainage and flood control issues should be considered at the start of the development process when site layout is first set out. Fixing building layout prior to drainage considerations significantly limits the potential for effective control of flood water in extreme events. The factors that should be considered are summarised in Figure 13.1.

Careful planning and consideration of development layouts can facilitate the management of flood risk. Development layout can significantly affect how flows will be conveyed and may even contribute to property flooding. In particular, channels and areas that may act as conveyance pathways and storage areas should be identified early in the process. The flowchart in Figure 13.2 simplifies the overall design process.

13.2 Building type and layout

13.2.1 Layout and flood pathways

Identifying the route of flood pathways in a development is a key design process that can assist with managing flood risk. Once flood pathways have been identified, the subsequent effect on downstream receptor systems should be considered. Conveyance pathways, storage areas and the downstream receptors all interact with the conventional drainage system, so the design process tends to be heavily interactive, with the final design often differing significantly from the outline proposal. Adequate time should be allowed to complete this stage of the site development process.

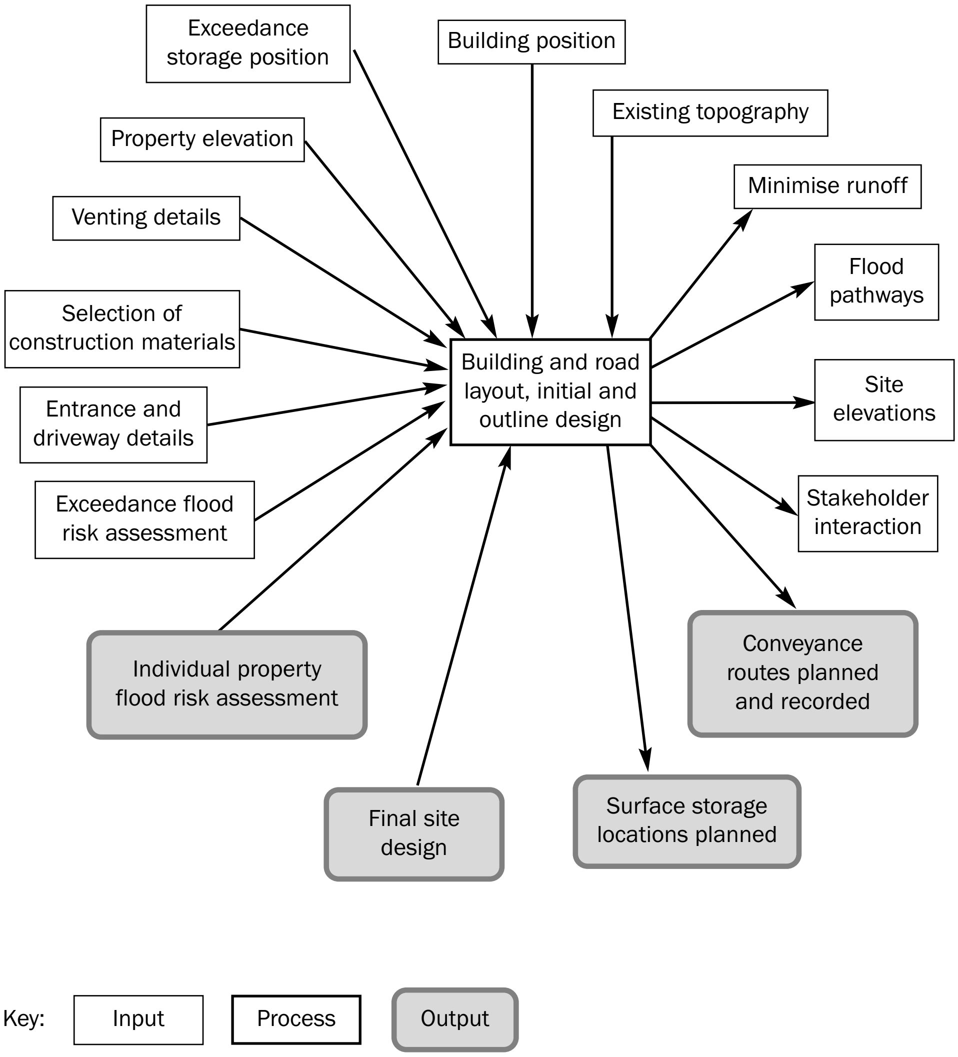


Figure 13.1 *Processes involved with designing building layout for exceedance events*

An example of the route of above ground flood pathways in a development is shown in Figure 13.3. This example also indicates how temporary above ground storage has been incorporated into the design. Consideration should be given at initial layout design stage of any site features that might be incorporated, such as playing areas, fields, detention ponds, infiltration basins, car parks and minor roads. Normally the use of dual-purpose areas will be the most cost effective and the greatest benefit to the community.

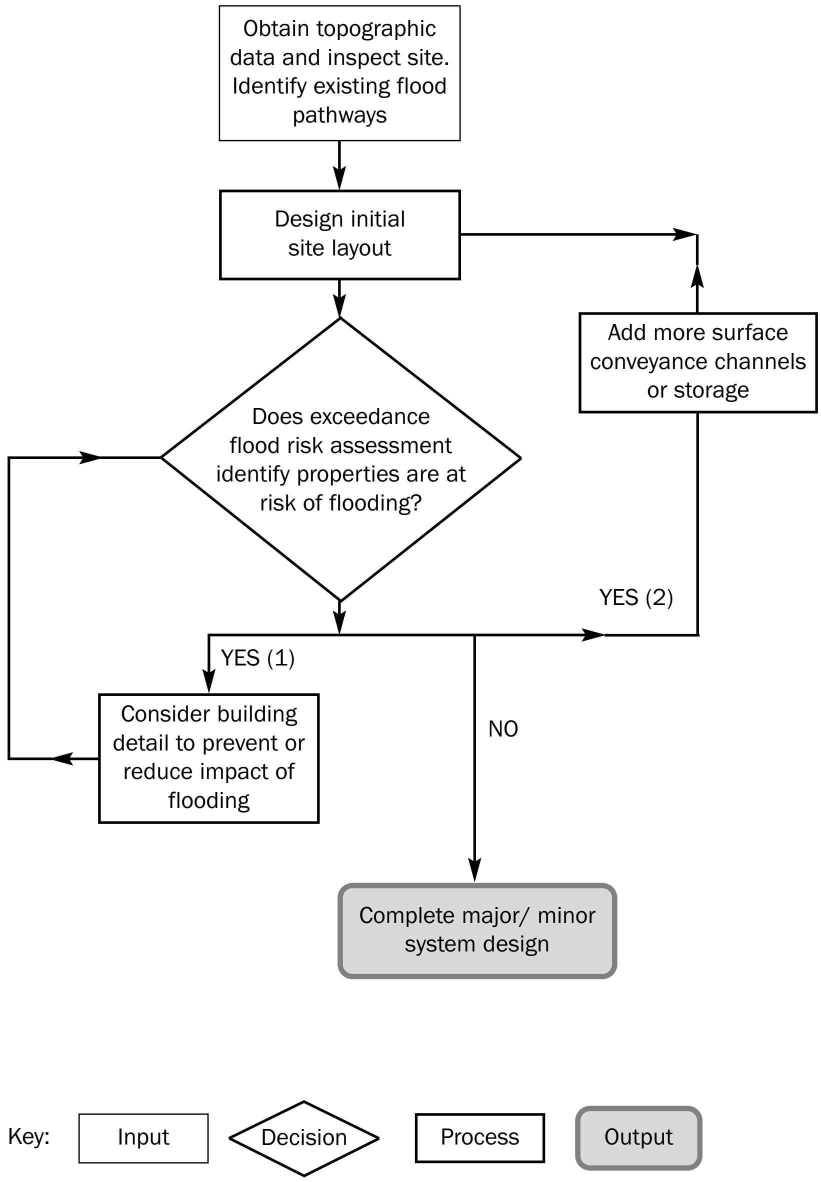


Figure 13.2 *Flowchart showing the process of including building detail and layout in the design process when considering the exceedance flood pathways*

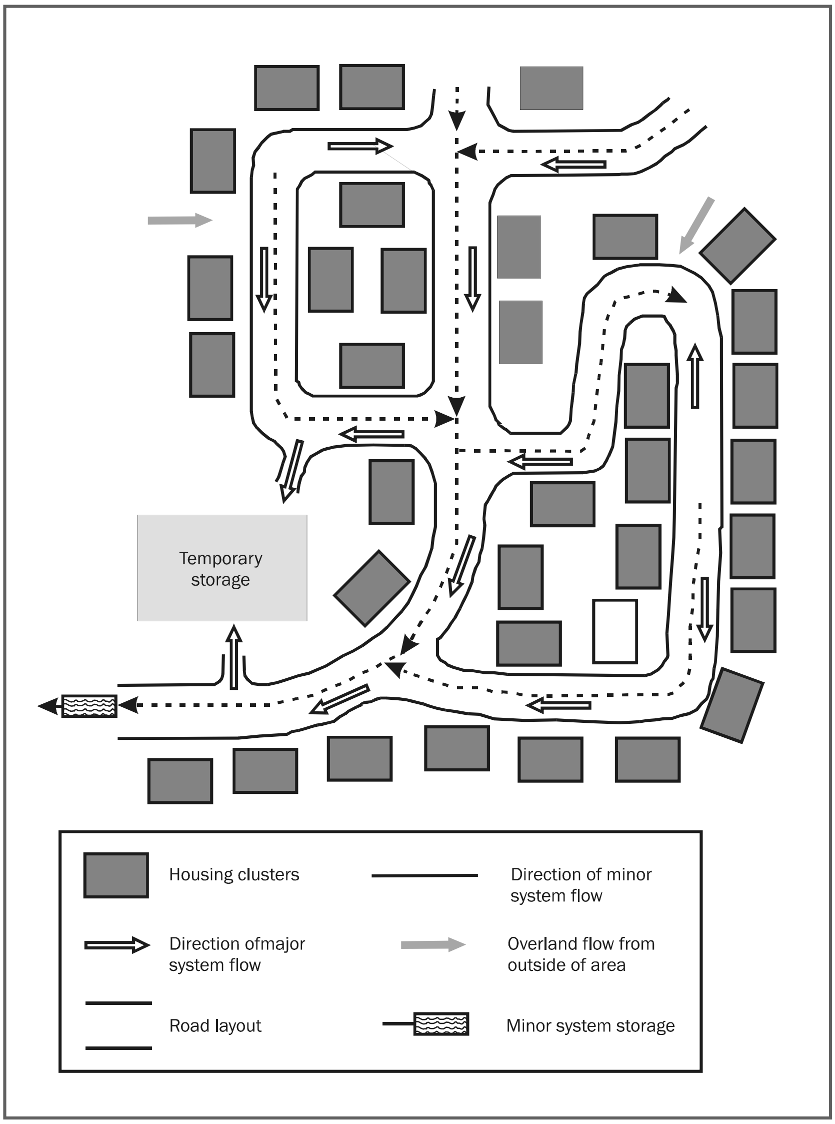


Figure 13.3 *Example of major and minor systems in a site layout*

Note: The pathways created between housing clusters to allow overland flow to drain from undeveloped areas behind property.

13.2.2 Utilising existing features of the site

It is important to consider numerous aspects in the design from the outset to produce an integrated solution that satisfies all stakeholders. Auckland's Regional Council technical publication *Stormwater management devices: design guide manual* (Auckland Regional Council, 2003) identifies four techniques that can be used during the initial design layout to minimise runoff:

- retain natural drainage site features
- good runoff control practices
- clustering of properties together
- minimise earthworks and ground disturbance.

It is important that these techniques are considered at the start of the design process.

Where possible the natural drainage site features should be retained and the position of houses built around these. In particular, the location of watercourses and dry valleys should be recorded, as these will be the primary routes for draining runoff from extreme events in the natural catchment. Retaining these pathways in the developed site through the careful positioning of roads and buildings will greatly enhance the capability of the drainage site to handle exceedance flow without significant impact.

In extreme events a considerable volume of runoff can be generated from permeable areas. Buildings are often located between such areas and the main drainage system or highways. Paths need to be created so that overland flow from such areas can escape during extreme events without impacting on adjacent property. Existing above ground pathways can potentially be retained by positioning buildings in clusters or as a perimeter block, and can be designed to cross roads using long profile berms where necessary. Alternatively these flows may be diverted onto a new above ground pathways. This similarly applies to the elevation of the site layout and where possible the new development should mimic the existing terrain. Where this is not possible flood pathways should be re-assessed.

When planning any earthworks on the site, it should be noted that earth moving equipment can seriously compact ground material, reducing its infiltration capacity. Measures should be taken to minimise this effect, and if this is not possible, the potential of reduced infiltration to increase surface runoff should be accounted for in the design of the drainage system.

Cul-de-sacs have traditionally been a common feature in housing estate design and should be given particular attention. If the entrance to the cul-de-sac is higher than the end then flows will be directed to an area likely to flood unless a flood pathway is incorporated into the design. Any such pathway needs to be appropriately recorded. More importantly, any drive or pathway around a property that has been designed as a flood pathway should be maintained during any future development.

13.3 Building detail

13.3.1 Building in protection measures

The detailed design of buildings can significantly influence their resistance and resilience to flooding. Further information can be obtained from CIRIA's flooding website: <www.ciria.org/flooding>. A number of reports are also available including SP155 *Reducing the impacts of flooding – extemporary measures* (Elliott and Leggett, 2002) and *Preparing for floods* (ODPM, 2003). Such measures could be included in most building design, however care should be taken to ensure that they meet or exceed the standards set out in the Building Regulations 2000. These are currently being reviewed and are likely to change in the future to account for the impact of flooding (Defra, 2004). The latest guidance on this can be found in C623 *Standards for the repair of buildings following flooding* (Garvin *et al,* 2005).

There are potentially a large number of small changes in property detail that can offer a small increase in protection to flooding. It is likely that the combination of a number of measures will result in a more substantial increase in the level of protection against flooding. Choosing which measures are best suited to each development will depend upon the level of risk of flooding and the acceptability of the changes.

The use of such measures may be applicable to both new build and retrofit scenarios. They will be of particular benefit if flood pathways are to pass close to buildings. This may be through the design of roads to act as above ground flood pathways and the raising of elevation levels above the predicted flood pathway water level.

13.3.2 Property elevation/threshold levels

Increased property threshold levels will provide an increase in flood protection. Local authorities, during the planning process, have the power to impose minimum ground floor levels. A simple increase of one or two brick courses may offer the necessary protection required. Alternatively, and depending on the site, the levels of the ground itself could be raised as demonstrated by the house on the left in Figure 13.4.

Figure 13.4 *Shows that increasing the ground level of the property can prevent flooding (courtesy Scottish Water)*

There is the potential to increase ground floor levels significantly and building on brick piers has previously been used with good effect. However, a modest increase in level can usually afford the level of protection required. Figure 13.5 shows the importance of the threshold level. Even where the threshold is only a little above back of kerb level, a considerable cross-sectional area for exceedance flow can be created between the highway boundaries. It is important that any changes in property threshold level meet or exceed the standards set out in the Building Regulations (2000) Part M and advised in *Access to and uses of buildings – Approved Document M* (ODPM, 2004c). This allows for ramped access enabling threshold levels to be raised without the design compromising disabled access.

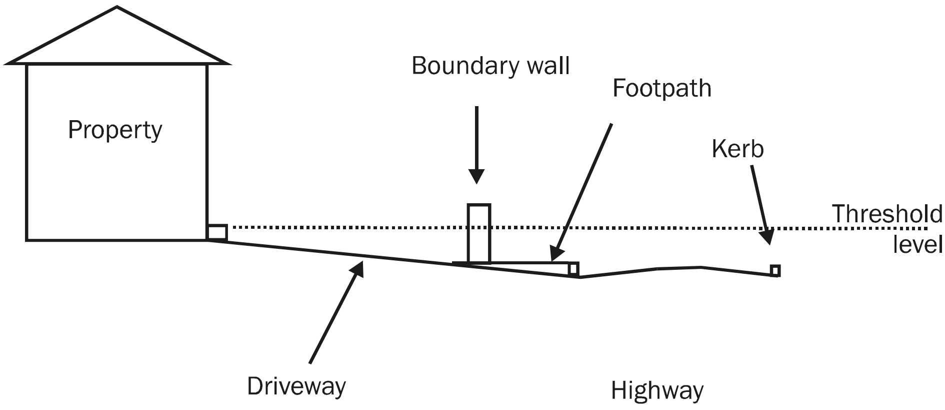


Figure 13.5 *Theoretical position of property which has a high threshold level and therefore protection against flooding*

13.3.3 Selection of the building materials

There are four main areas in which the selection of appropriate building materials can substantially improve flood resistance. An added benefit is the lower drying out and re-build costs in the event of a flood. This is applicable to new and existing property. The four areas are:

- floors
- external walls
- internal walls
- fixtures and fittings.

The construction materials and techniques are not described in detail here, however more detailed guidance can be obtained from advice sheets produced by CIRIA <www.ciria.org/flooding> and *Preparing for floods* (ODPM, 2003). Detailed guidance on properties that have been flooded and are to be repaired can be found in C623 *Standards for the repair of buildings following flooding* (Garvin *et al*, 2005).

13.3.4 Venting

Flow may enter into a property through a variety of venting systems. The most common of these will be through an air brick and its subsequent vent channel due to its relatively low position. In older properties air bricks may be positioned lower than those in newer properties with the vent channel passing straight through the wall. Newer properties may have periscope venting channels which are higher at the air brick and lower internally (Figure 13.6). It is important when detailing buildings that any buildings that may be subject to flooding have the air vents above the likely flood depth. Alternatively, an automatic flood seal can be fitted to each vent.

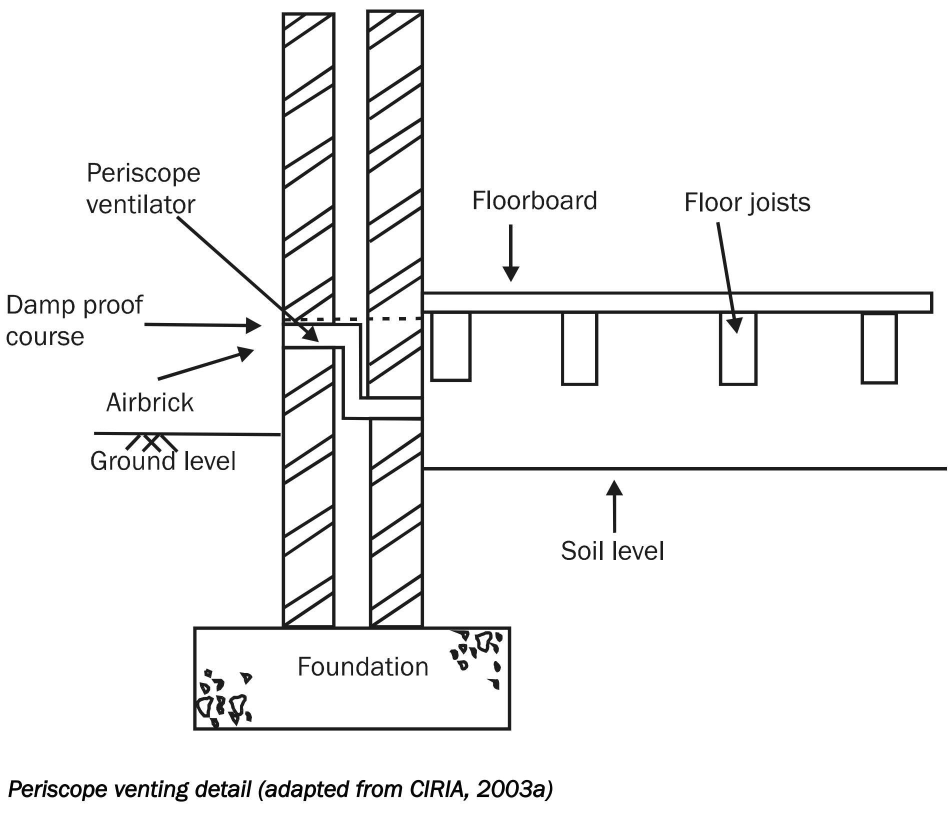


Figure 13.6 *Periscope venting detail (adapted from CIRIA, 2003a)*

13.3.5 Entrance details

The elevation of the entrance to a property can act as a line of defence to prevent flooding. This will be effective if the threshold is above the top water level of any flood. Where possible the entrance level should be set with a 50 mm free board allowance above the expected top water level. Where this is not possible, the risk of the curtilage and property being flooded should be assessed. This will depend upon the property elevation and access slope and this risk can be mitigated with the design of other elements. Where the property threshold is below back of footpath level, the back of footpath level should not be lowered to ease the driveway gradient, as this would encourage exceedance flow conveyed within the highway to divert into the property. Further information on this is given in Appendix C.

13.3.6 Driveways and curtilage

The slope and the surrounding land within the curtilage will define whether water will flow towards or away from a property. Where possible, gardens and drives should slope away from the property. Where this is not possible, greater consideration should be given to increasing the ground level of the property and possible entry points for flood water.

The area in the immediate vicinity around a building should be sloping. Approved Document H, *Drainage and waste disposal* (DTLR, 2002) identifies that a reverse gradient from the property should be created for at least 500 mm with a cross fall recommended at one in 60. However for exceedance flows, this may need to be increased. If a flood pathway is designed to pass close to a building the size will greatly depend upon the calculated water depth.

Gardens at the front and rear of the property should be lower than the property itself. Case studies show that properties with gardens at a higher level that slope and drain towards the house can direct substantial quantities of flow towards the property which can lead to flooding if no flood pathway away from the building is provided (Figure 4.3).

13.3.7 Siting of services

The position of services should be kept above possible flood water levels. This particularly applies to electrical and gas appliances which needs to be inspected by a qualified engineer following flooding before they are re-used. Any meters should also be positioned as high as possible. Where services are located in the basement, they should be at ceiling height to offer the maximum protection possible. Further advice on the position of services and appliances is given in *Preparing for floods* (ODPM, 2003), SP155 *Reducing the impacts of flooding – extemporary measures* (Elliott and Leggett, 2002) and Advice sheet 7 *Flood-resilient services* (CIRIA, 2003b).

13.3.8 Inadvertent modifications to existing flood pathways

Flooding by default can often occur because of the inadvertent modifications to existing flood pathways. This may be through an extension to a property such as garages, conservatories or even the construction of new fence or wall on the boundary that block or re-routes a flood pathway (Figure 13.7). Property A has a driveway that enables exceedance flows to pass and not cause flooding. Property B shows the same property but with an extension. Exceedance flows, having no flood pathway, initially cause surface ponding followed by property flooding. This could even occur during minor storms. It is important that all overland flow paths are recorded and any proposed changes to a property take into account these flow paths.

The challenge of keeping the pathway clear in the future is difficult, although a number of measures can be set in place in an attempt to preserve such designed pathways. Designed flood pathways should be recorded on the approved drawings of a planning consent. This will alert planning officers to the existence of the flood pathways when they consider a new application. Where a development is disposed of in a number of separate lots, such as with a typical housing development, restrictive clauses should be included in the transfer or assignment titles to protect the rights of adjacent plots to the conveyance of flood flow.

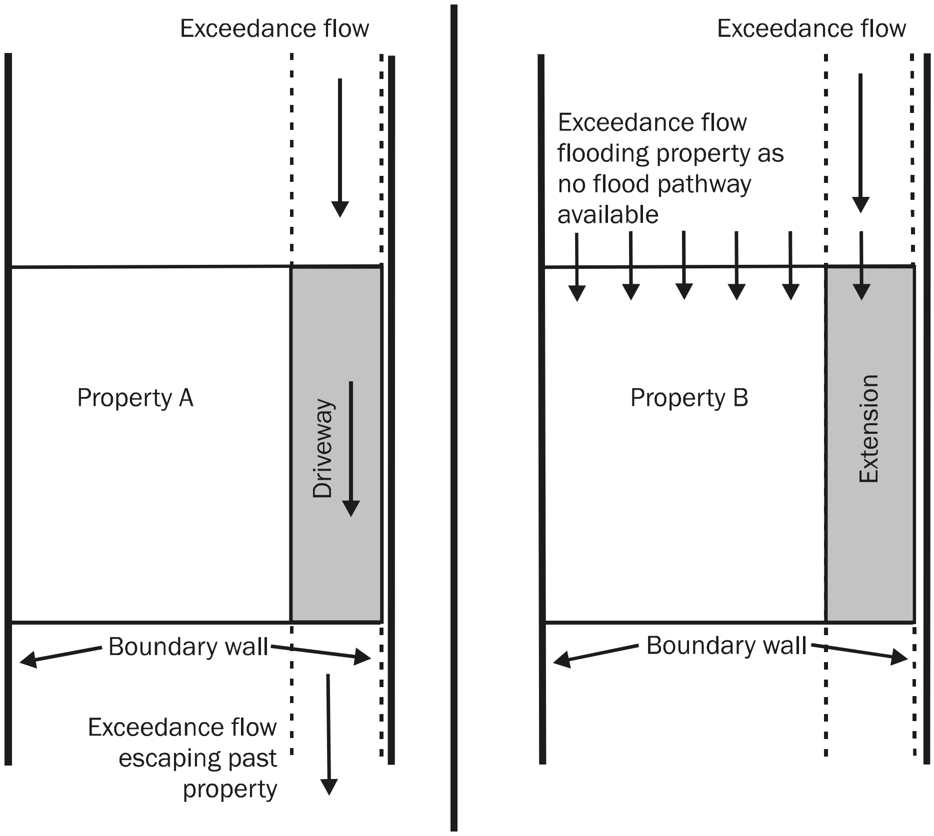


Figure 13.7 *Plan view of how extensions to property can cause flooding by removing an existing flood pathway*

13.3.9 Under building flood paths

Under building flood paths may exist by design or by default. If it is known that a flood pathway exists, where practical, the building should be repositioned or the flood pathway re-routed. However if neither of these options are viable, there may be the need to incorporate the design of the flood pathway into the design of the property. This is likely to mean that an open area is created, for example at a car park that could be used to control the flows or alternatively provision is made to pass the flow under the building. If the flow is to pass underneath the building, settlement and protection of the foundations should be considered, as well as ensuring there is sufficient capacity for the flow. Approved Document C of the Building Regulations (ODPM, 2004b) addresses this issue. If the flow is to be routed above ground, the structure should be designed to allow a free flow into, through and away from the structure. Obstructions to the flow should be avoided.

14 Downstream impact assessment

Creating additional conveyance by utilising above ground flood paths could have significant adverse effects on downstream systems, for example by transferring flooding from one area to another. When a solution is implemented it is important to identify the consequences downstream and undertake any mitigation measures that are necessary.

When assessing the downstream impact the approach should be to consider the assimilative capacity of the receiving system and the spatial extent to which the assessment is necessary. This should then be applied to how this affects the various stakeholders.

14.1 Conveyance and storage

The downstream implications of a proposed solution will vary depending on the receiving system and the proposed solution itself. It is likely that conveyance solutions will be predominantly used to transfer flows from one area that suffers from a risk of flooding, to another. The implications of this for the receiving system may be significant, with high flow rates and volumes being transferred over a short period of time. Alternatively where storage forms part of the solution, the impact on the receiving system may be less due to the attenuation effects of the storage volume.

In either case, the assimilative capacity of the receiving system should be adequately assessed in order to determine if further storage is required to mitigate adverse effects.

14.1.1 Flood conveyance impacts

Conveyance of flood exceedance flows commonly makes use of road surfaces and other surface pathways, depending on the topography and land use of the site. The impact downstream may be significant if flood water is passed on to other areas which may also be suffering from flooding. Additional flow conveyed from the upstream system may add to an already overloaded downstream system significantly increasing flood risk in that area. The flood extent and risk can be exacerbated when areas receive high upstream flow and an increase in conveyed flood flow rates.

Assessment of the spatial extent of potential impact is very difficult in an urban environment. In the situation where the flow discharges to an open channel or river that serves an area which is at least 20 times greater than the area causing the flooding, it is unlikely that the impact downstream will be significant. However where flow is discharged to another urban drainage system, where streets confine flood flows and pipe systems can convey large volumes of water to hydraulic low points elsewhere, a more detailed assessment is necessary.

Consideration of the overland flow paths and also the sewerage system capacity is probably necessary to determine the impact of the flood flow. Drainage simulation modelling is likely to be needed in many instances to carry out this an accurate assessment.

14.1.2 Conveyance with storage

Where storage has been provided within the upstream area, in order to manage flood risk, the downstream impacts are likely to be less than with a conveyance only solution. This is because the storage attenuates the runoff, reducing peak flow, and spreading the flow hydrograph over a longer time period. However there will be occasions where storage can be detrimental to downstream systems. This will occur in cases where the storm response time in the downstream system is significantly longer than that in the urban system, such as occurs when a small to medium sized urban area discharges to a river system fed from a large upstream rural catchment.

In such cases, the effect of storage can be to delay the runoff so that its peak flow occurs around the same time as the peak response in the receiving river. Without storage, the peak in the urban runoff will discharge before peak flow occurs in the river. Without storage it may be possible to accommodate the additional urban flow within the capacity of the existing river system.

14.2 Procedure for assessing and mitigating impacts

The procedure for assessing the impacts of exceedance flow and then for designing and implementing any required mitigation measures is set out in Figure 14.1.

14.3 Assessing the impact on downstream systems

To avoid an incomplete assessment of impacts it is useful to consider all the various stakeholders that might be affected by the impact from upstream exceedance flows. Such stakeholders may be affected because:

- their property or land is damaged by flood flows resulting from the impact
- there is an increased risk to their personal health or safety
- there is an increase in the liability for the statutory or non-statutory function they are required to fulfil.

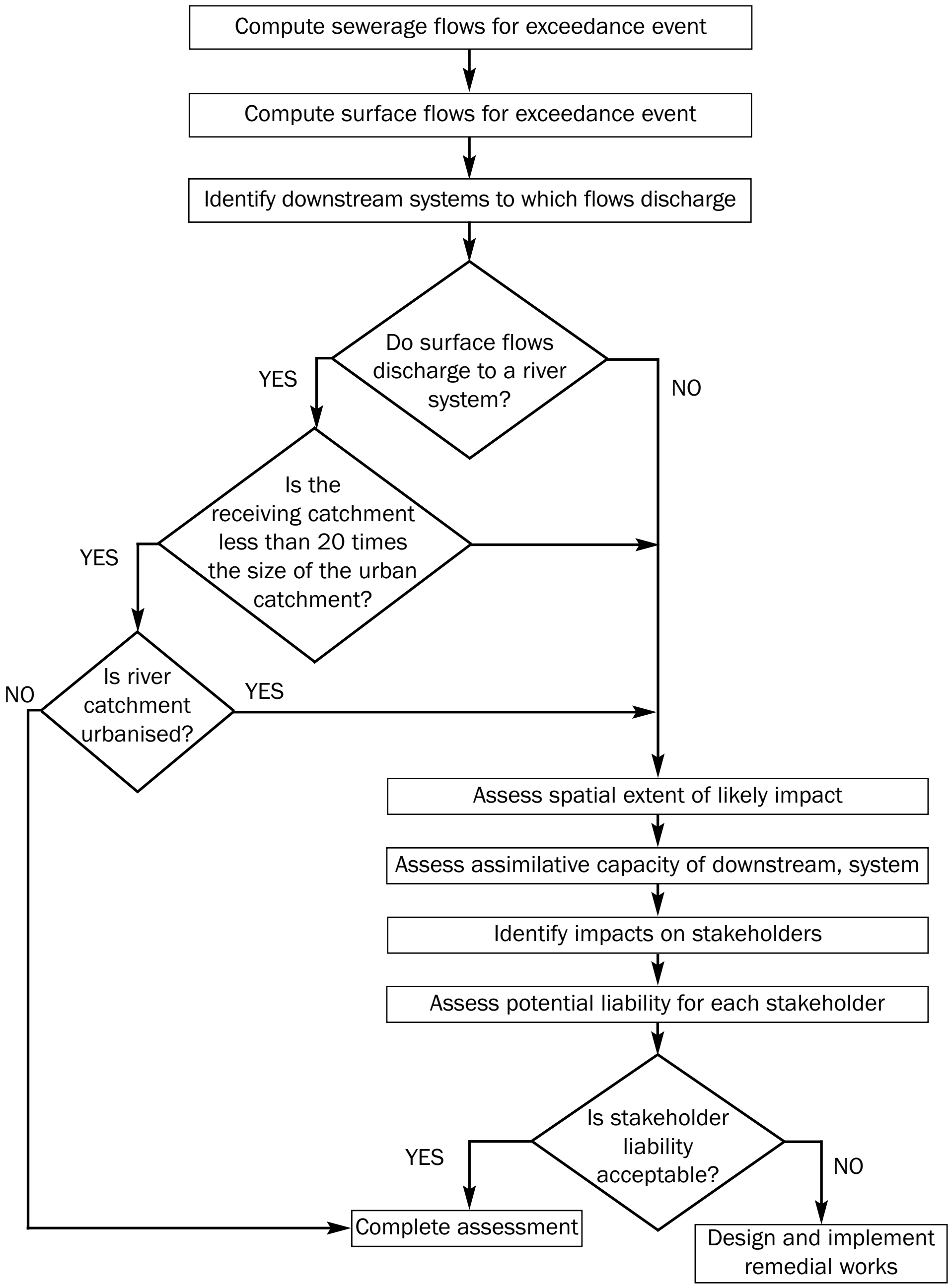


Figure 14.1 *Procedure for assessing the impact of exceedance flow on receptor systems and developing appropriate mitigation measures*

The roles and responsibilities of various stakeholders involved in the process are summarised in Part A Chapter 3 and in more detail in Part B Chapter 5. These stakeholders are listed below, together with a summary of their respective potential impacts. It is important to note that this assessment refers to the quantity of the flow from an extreme event and not water quality aspects.

Environmental Regulator

The Environmental Regulator (Environment Agency/local authorities in Scotland and SEPA/DOE Northern Ireland) will primarily be concerned with any extra flow and volume that may enter watercourses that they are responsible for (note that this does not preclude the discharger from obtaining the necessary permission to construct any new outfall, see below). The assimilative capacity of the river downstream will determine whether or not the discharge will have a significant impact. In assessing the impact, account should be taken not only of the peak rate of discharge, and volume discharged, but also the timing of the event. As explained previously, where the timing of the peak of the discharge differs significantly from the peak in the river, it may be possible to accommodate the additional discharge without significant impact.

Potential impacts may stretch for a considerable distance downstream and in some cases upstream. The EA will advise on the spatial extent of any required assessment.

A second concern may be the quality of the water entering the watercourse. Although water quality is not considered as part of the guidance, it should be noted that if the new discharge to the watercourse occurred frequently this might become an issue.

Environmental regulators are responsible for flood control and flood mitigation for watercourses within their control, but are not liable for consequential damage.

Land drainage authorities (including this function in local authorities)

Land drainage authorities will have similar concerns to the Environmental Regulator for any additional flow being discharged to a watercourse that is their responsibility. Any increase in flow into these watercourses may cause flooding further downstream, therefore identifying the spatial extent is very critical.

There are no simple rules for determining the spatial extent in such cases, as this depends on local conditions. In the first instance the route for draining exceedance flows through the receptor system should be traced out on a suitable plan. Limitations to the drainage capacity of the various downstream sections should be identified. In particular the capacity of culverts, bridge openings and key conveyance channels should be assessed. The critical sections should be marked on a plan and potential areas of flooding identified by inspection of existing ground levels. This will identify the extent of the area that may require more additional assessment for flood impact.

Riparian owners and other owners of receiving waters

The consent of the relevant riparian owner is required where a new outfall is to be constructed. In granting consent, the owner will assess the likely impact and may require additional works to mitigate against perceived impacts.

Highway authorities

Highway authorities (Highways Agency/local authorities) are responsible for highway drainage, highway maintenance and the safety of highway users. They will have a particular concern that any additional surface flow that discharges onto a highway surface may have an impact not only on highway drainage capacity, but also on the structural condition of the highway and/or the safety performance. The former is unlikely to be significant if the occurrence of the extreme event is rare (which will normally be the case). The latter is likely to be more important as highway authorities are becoming increasingly exposed to litigation following accidents where the cause has at least in part been attributable to standing water. Adhering to the limitations of velocity and depth recommended in Chapter 11, should mitigate these effects. At the very least an agreement to discharge onto a highway surface would normally be required between the highway authority and the stakeholder proposing to discharge the flow.

Sewerage undertakers

The conveyance of surface flows into a downstream sewerage system may cause hydraulic overloading and subsequent flooding for which the sewerage undertaker is liable. As explained in Chapter 5, a sewerage undertaker only has a responsibility to effectually drain property, and will argue that it is not their responsibility to drain other areas, nor to handle extreme events. Where the additional surface flow is shown to reduce the existing level of service of a sewerage system, normally determined by computer simulation, the undertaker may require mitigation measures to restore the original level of service.

Insurance industry

The insurance industry will be interested in the transfer of flood risk from one area to another as a result of the implementation of a solution. This will be critical when a property owner's liability changes and their own liability is affected. Insurers will normally try to offset any perceived increase in risk through an appropriate increase in insurance premiums, imposed excess and/or limitations to benefit.

Land owners

Where surface water is discharged onto private property in such a way as to deliberately cause flooding (as would be the case in discharge from a surface flood channel), a right to flood agreement would be necessary between the landowner and the stakeholder wishing to discharge. In reaching such an agreement, the following should be taken into consideration.

1. Who has the right to flood at the moment?
2. What compensation is payable in exercising the right to flood?
3. Whose land is affected?
4. How would the land used for flood conveyance and storage be protected?
5. How will the property owners insurance be affected?

Consideration would also have to be given to the future maintenance of such areas and any necessary clean-up following an extreme event. Further details of this are given in Chapter 12.

Care should be taken to fully evaluate the health and safety risks to the public caused by any agreement to flood. The relatively short times of concentration in urban areas means that it may not be possible to give adequate flood warnings and therefore it will not be possible to actively manage such flooding. Consideration should be given to appropriate signage and the education of local community groups.

14.4 Mitigating the effects of downstream impacts

Mitigation measures will be needed where there is insufficient assimilative capacity in the receptor system. Mitigation can be achieved by providing a storage facility close to the point of discharge. If this is not possible, additional storage may be distributed in the area upstream.

The amount of storage volume required will depend on how the flow is to be limited. As mentioned previously, the analysis should extend not only to look at the effects on peak flow and volume, but also to compare the timing of the peak flow in the upstream system with the peak flow in the receptor.

Receptor systems may often be hydraulically overloaded at the time of peak discharge, because they also are suffering from the effects of the same extreme event. However, there may be other causes of overloading in the receptor system. The effects of a sudden extreme event in summer, when river levels are low, may be different from winter when levels are higher. Where discharges are to coastal areas, impacts may be affected by tide levels. Some form of joint probability analysis may be required.

Addressing the needs of flood exceedance is unlikely to result in standard drainage solutions for extreme events. Storage is much more likely to be provided by mobilising flooding of certain roads, car parks, playing fields and other temporary flood areas where the consequences and capital costs are limited.

Consideration should be given to creating sacrificial areas in areas of low land value, where flood water from rare extreme events can be stored over long periods. Water from such areas could then infiltrate slowly into the ground and/or evaporate. Care should be taken to fully address the health and safety aspects of such measures, as explained in Chapter 12.

Part C Case studies

15 Case study 1: Bishopbriggs South

15.1 Introduction

The Bishopbriggs South catchment lies to the north of Glasgow. The catchment covers an area of some 350 hectares and has a population of 18 000. In 2002 the area suffered from an extreme rainfall event of between 30 and 100 year return period. This caused extensive flooding of property. Subsequent analysis showed that in addition to the effects of runoff from the developed areas, the sewerage system also suffered from additional flow from permeable areas adjacent to the developed area, and from backing up of outfalls from local watercourses that were themselves overloaded.

The catchment is largely made up of medium density housing in the form of semi-detached and terraced property. The minor drainage system consists primarily of combined sewers, though part of the area is separately drained. Trunk sewers mostly run parallel to the Bishopbriggs Burn, which is the principal watercourse. The minor system drains to the Kelvin Valley trunk sewer, the minor drainage system is shown in Figure 15.1.

Little is known of the history development of the catchment. The majority of the development is post-war and it is likely that the sewerage system is contemporary with this. The system is thought to be structurally sound and infiltration minimal.

15.2 Stakeholder involvement

The flooding of 2002 which affected a large number of properties caused a public outcry. There was considerable press coverage with adverse commentary largely being aimed at Scottish Water and East Dunbartonshire Council, the local unitary authority. In the weeks following, these two bodies agreed to collaborate to find suitable solutions to managing flood risk in the area.

Scottish Water are responsible for draining developed areas provides an agreed level of protection against flooding, which in this area is the 30 year return period rainfall event. Flow from highway drains are the responsibility of East Dunbartonshire Council and the Council is also responsible for land drainage and the maintenance of local watercourses. Scottish Water agreed to address their responsibilities with respect to the performance of the sewerage system while East Dunbartonshire Council agreed to address any deficiencies in draining permeable areas and ensure adequate capacity in local watercourses, especially the culverted sections.

The following sections illustrate how the recommendations in this guidance can be applied to the Bishopbriggs catchment to understand the performance of its drainage systems in extreme events and to assess proposed remedial measures.

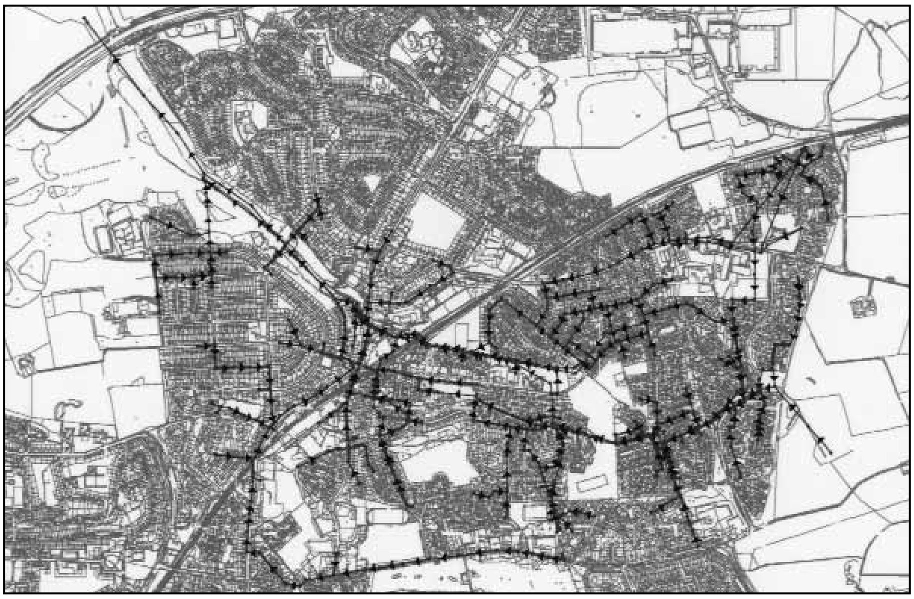


Figure 15.1 *Bishopbriggs South sewerage system*

15.3 Calculating exceedance flow

The process for calculating exceedance flow is set out in Chapter 9. The network is not particularly large, but is complex because of the interactions with local watercourses and the input of overland flow from permeable areas. A level 2 study (Box 9.1) is selected initially. Figure 9.2 summarises the various inputs to the process and Figure 9.5 sets out the different steps.

15.3.1 Collecting data

The following data and information was available from earlier studies:

- a HydroWorks model of the sewerage system used in a previous drainage area study. This model had been verified by short-term flow survey data
- LiDAR digital ground level data for the catchment
- records of the flooding for the extreme event of 2002
- ordnance survey background digital mapping.

The HydroWorks model had been built solely to assess the performance of the sewerage system. This did not allow fully for the interaction with local watercourses or for the input from permeable areas. The contributing areas had been modelled using the fixed parameter UK runoff model (Equation 8.1).

The model was extended to include runoff from permeable areas and to model the natural water courses and their contributing areas. The variable parameter or new UK runoff model was used (Equation 8.3) to model contributing areas, and a further flow survey commissioned to verify the parameters used in the model.

15.3.2 Using models to assess system performance

The model was run with rainfall of different durations to determine the critical event. This was found to be the 120 minute duration winter event. Subsequent analysis was performed using this event. The model was run with a 30 year event and the surface flooding at nodes recorded. Figure 15.2 shows the results of the 30 year event and highlights significant flooding at a number of nodes. The model was also run with the 100 year event, and the results are shown in Figure 15.3. The locations of flooding are the same though the volume of flooding is increased substantially.

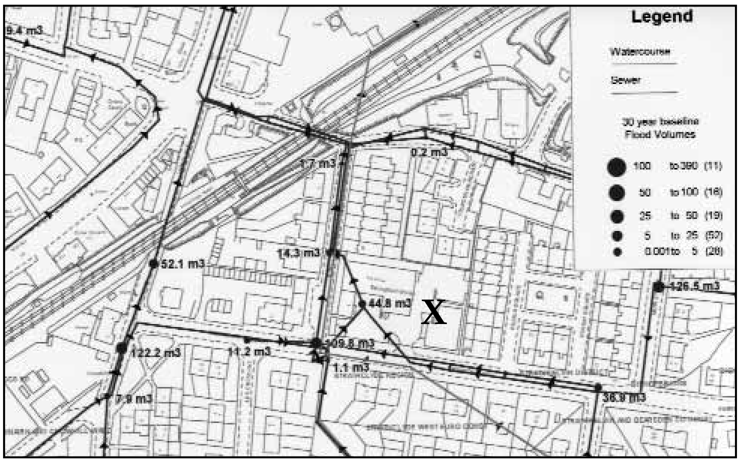


Figure 15.2 *Nodal flooding for the 30-year event, central area only*

15.3.3 Verifying against historic flooding

The results in Figures 15.2 and 15.3 were compared with records of flooding for the July 2002 extreme event. Records showed that significant flooding only occurred in the vicinity of Springfield Works, at the junction of Emerson Road and Arnold Avenue (point X in the figures). This is a natural low spot in the catchment (Figure 15.4)

Reports from the public showed that there was considerable surface flow along highways during the July 2002 event. It is clear that considerable overland flow occurs during extreme events. The study was upgraded to a level 3 study to better replicate prototype performance.

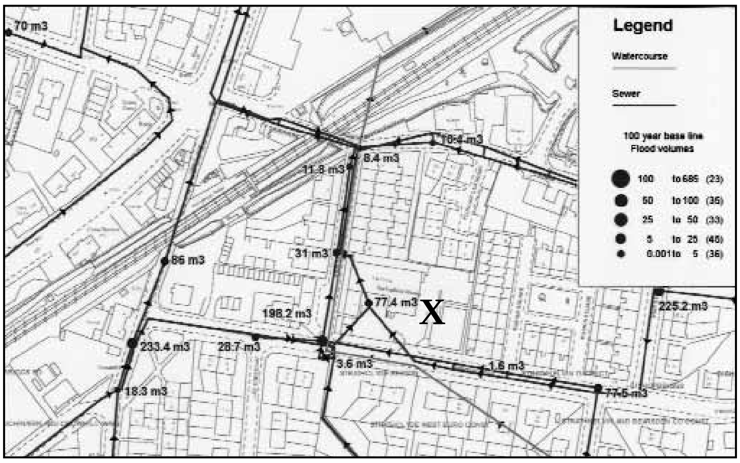


Figure 15.3 *Nodal flooding for the 100 year event, central area only*

Figure 15.4 *Springfield Works: location of flooding in July 2002*

15.3.4 Upgrading to a level 3 study

The first step was to assess the likely surface flood paths. A digital terrain model was built using the LiDAR data, and a rolling ball software applied to identify potential pathways. These are shown in Figure 15.5.

These clearly showed that any overland flow would drain to the low spot at Springfield Works. However, these pathways do not exist in practice. Inspection of the ordnance survey background shows that the pathways were heavily modified by artificial features such as roads and buildings. The most likely pathways will be along highways (as observed during the July 2002 event). These were identified by comparing the rolling ball pathways with the road layout, and then explicitly modelled into the HydroWorks model using the duplicate node method described in Appendix A. When modelling these pathways an initial curb depth of 40 mm was assumed to allow for the effects of drainage flow from the immediate contributing areas, as described in Section 9.3.3. This was achieved by reducing the effective kerb height to 60 mm. The courtyard at Springfield Works was also modelled as a flooded area. The modified modelled network is shown in Figure 15.6. The simulations were then re-run with the 30 year and 100 year events and the results are shown in Figures 15.6 and 15.7 respectively.

The results for the modelled 30 year and 100 year events are consistent with the observations during the July 2002 event. Flood flow discharged from manholes and gullies is conveyed along highway surfaces and accumulates in the courtyard at the Springfield Works.

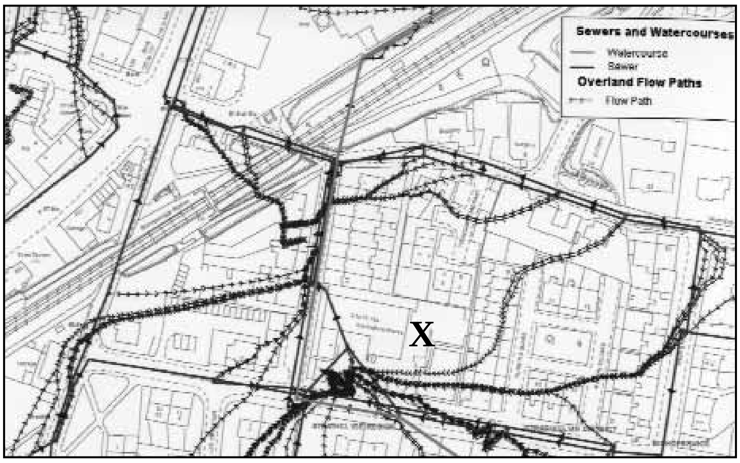


Figure 15.5 *Surface flood pathways identified by rolling ball model*

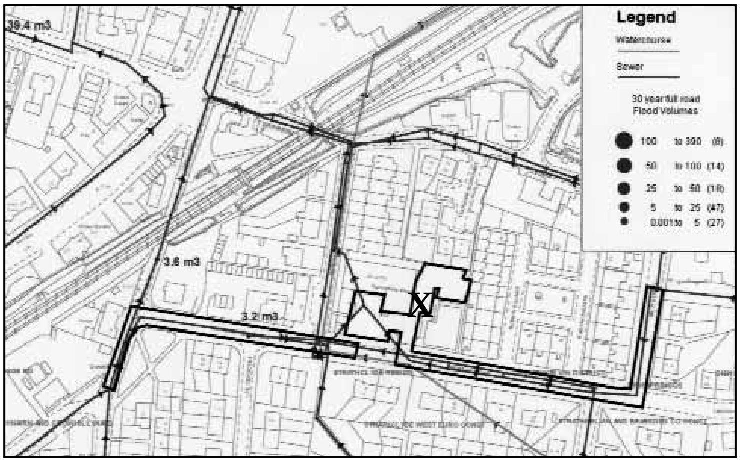


Figure 15.6 *Flooding for the 30 year event with surface pathways modelled*

Note: The duplicate drainage network representing the surface flood pathways (shown dotted). The areas bounded by the heavy black line represent surface flooding.

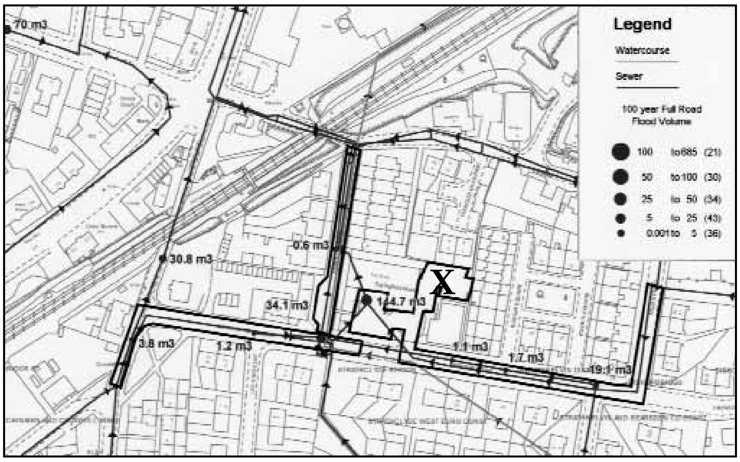


Figure 15.7 *Flooding for the 100 year event with surface pathways modelled*

Note: The duplicate drainage network representing the surface flood pathways (shown dotted). The areas bounded by the heavy black line represent surface flooding.

The capacity of modelled inlets was reviewed by inspecting the rainfall intensities in the 30 year and 100 year rainfall event files. This identified that the capacity of inlets was not exceeded during the one in 30 year event. The one in 100 year events exceed the roof drainage inlet capacity of 75 mm/h for four minutes with a maximum of 81 mm/h, and exceeds the 50 mm/h capacity for yard gullies for 18 minutes. Some localised flooding of yard gullies can be expected for the 100 year event, but due to the limited period of exceedance this is not considered to be significant.

The depths and velocities of flow conveyed in the highway channels were also obtained from the model output. In HydroWorks this data is produced in the hydrograph output files. Table 15.1 summarise the results for the 100 year event. Figure 15.8 shows the courtyard area to Springfield Works which is used as a car park. This area is enclosed with a 100 mm kerb in practice, though this limitation was not applied to the model. The depth generated by the model in the courtyard is also given in Table 15.1.

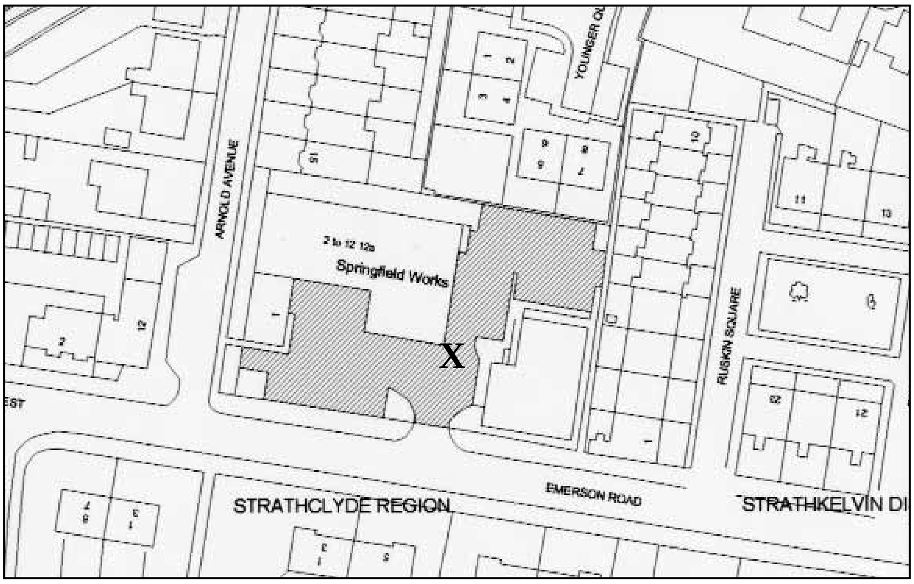


Figure 15.8 *The courtyard to Springfield Works showing the location of the most significant flooding in the drainage area*

Table 15.1 *Summary of maximum surface velocities and depths for the 100 year event*

Events	Max channel depth (mm)	Max velocity (m/s)	Depth in car park (mm)
100 year design events	82	1.5	222 mm

Note: The maximum channel depth includes the 400 mm base depth assumed.

The results show that at no point does the conveyance flow exceed kerb height. An inspection of the actual footpath profile in the area indicated that flow was unlikely to be diverted out of the highway by footpath crossings or dropped kerbs.

15.4 Exceedance risk assessment

The flood risk assessment was undertaken in stages, following the procedures set out in Chapter 10, and fully explained in Appendix A. The consequence of property flooding was assessed first. All the property in the sub area used in the case study is domestic residential with the exception of Springfield Works. The consequence of flooding property situated below the level of the adjacent highway or with cellars was assessed first. This was done by comparing the modelled levels of sewer surcharging with the property base level. Each individual property was assigned a consequence level using the level criteria set out in Table 10.8. This was done for both the 30 year and 100 year events.

The probability of flooding different property was assessed for each event based on the proximity to an identified flood pathway by inspection from the plans. Combining probability and consequence using a matrix similar to that in Figure 10.13 enabled a comparative risk score for property to be established. None of the properties gave a high risk score, with the medium high only being assigned to a small number or properties in Muir Street and Arnold Avenue.

A review of maximum velocities (see Table 15.1) showed that there was no additional risk associated with velocities. The associated risk scores for property are shown in Figure 15.9 for the 100 year event.

Health and safety aspects were also reviewed by comparing modelled flow velocities and depths in surface pathways (see Table 14.1). However none of the values recorded were significant enough to warrant a risk score.

The most important result from the modelling and risk assessment were from the Springfield Works compound. This showed a flood level of 232 mm for the 100 year event, which overtops the kerb level and would cause flooding to the works. This was confirmed by the event of July 2002 when there was considerable flood damage to the works.

15.5 Solution development

Flooding in the Springfield Works courtyard causes a problem for both the 30 year and 100 year event as levels exceed the 100 mm kerb height of the car park and enter the works. Options for flood alleviation focused on two key alternatives:

- providing additional storage in the sewer system to protect property from flooding for the 30 year event
- increased capacity through the provision of a new sewer connected to the Kelvin Valley trunk sewer.

The storage option proved impractical due to the unavailability of land in the vicinity of Springfield Works to construct a storage tank. The relief sewer option was created as a potential solution.

To connect to the Kelvin Valley trunk sewer and alleviate the 30 year flood event, a 450 mm new sewer is required along the line shown in Figure 15.10. Surface conveyance along highways will still occur with this solution. For surface flooding to be removed for the 100 year event, a 750 mm sewer is required. However, modelling the effects of the 100 year event shows that the water level in the car park at Springfield Works remains below kerb level and drains away at the end of the storm. The area acts as temporary surface storage for exceedance flows without risk of flooding the works. Although velocities and depths in surface pathways increase for the 100 year event, they still remain within the design requirements.

The 450 mm new sewer to convey flows to the Kelvin Valley trunk sewer so remains the preferred option.

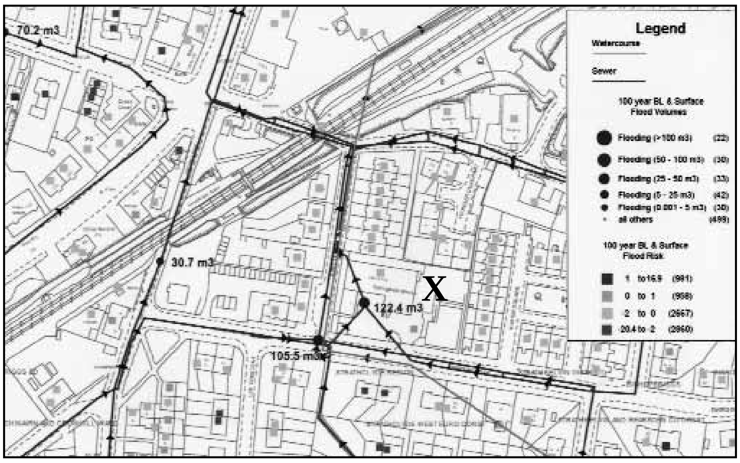


Figure 15.9 *Exceedance flood risk assessment for 100 year event*

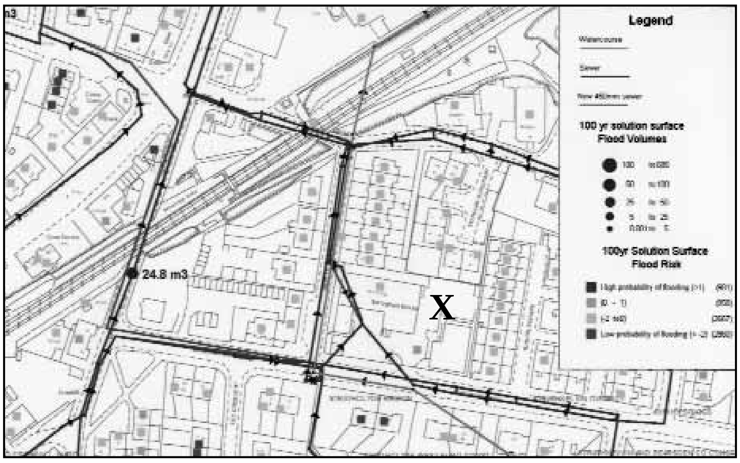


Figure 15.10 *Solutions model with 100 year event showing flood risk*

15.6 Impact on downstream systems

For the 100 year event all exceedance flows are conveyed on the surface to the courtyard of the Springfield works, as described above. Flows are stored temporarily at this point and are discharged to the new 450 mm sewer that transfers flow to the Kelvin Valley trunk sewer to the north west of the area, as shown in Figure 15.10. A model of the Kelvin Valley trunk sewer was available, and the additional flows were input into this. The results show that they do not cause a significant deterioration in the performance of the trunk sewer. A further solution was tested, using a 600 mm sewer, which was the size required to convey the 100 year flood flow. This did cause some noticeable deterioration in the Kelvin Valley trunk sewer performance.

16 Case study 2: Upton, Northampton

16.1 Introduction

The Upton development lies to the south west of Northampton. It is a greenfield sustainable urban extension developed in a partnership between English Partnerships and Northampton Borough Council. Once completed, the development will include approximately 6000 dwellings. It is being constructed in phases, with phase 1 covering 37 ha and comprising of some 1400 homes and associated infrastructure such as schools, work units, retail and community development. It is located immediately north of the River Nene floodplain, which provides the outfall for surface water runoff. The first advanced infrastructure contract started in 2003 and house building commenced in 2004.

This case study is concerned with a sub area of phase 1, known as catchment D. This consists of 16 ha of development of which 8 ha is impermeable, illustrated in Figure 16.1.

Upton's key development principles relate to promoting sustainable growth and an enduring, distinctive environment. Sustainability was embedded in both the Upton design code and urban framework plan. From the drainage perspective this was expressed by the following requirements.

- stakeholders are to be involved at an early stage
- surface drainage shall be by means of sustainable drainage systems (SUDS)
- the drainage of the site from extreme events and impact on downstream systems should be explicitly allowed for.

It is important to note that these requirements are entirely consistent with the recommendations of *Sewers for adoption 5th edition* (Water UK and WRc, 2001). For the purposes of this case study, it was necessary to amend part of the design to create an exceedance problem and demonstrate how it should be considered.

16.2 Stakeholder involvement

The key stakeholders in the development are:

- English Partnerships (developer)
- Northampton Borough Council (land drainage and planning)
- Northamptonshire County Council (highway authority and planning)
- Anglian Water (sewerage undertaker)
- Local residents (potential occupiers).

An "enquiry by design" took place for phase 1 in 2001, which allowed the residents, local stakeholders and key decision makers in the area to become involved with the development of the design.

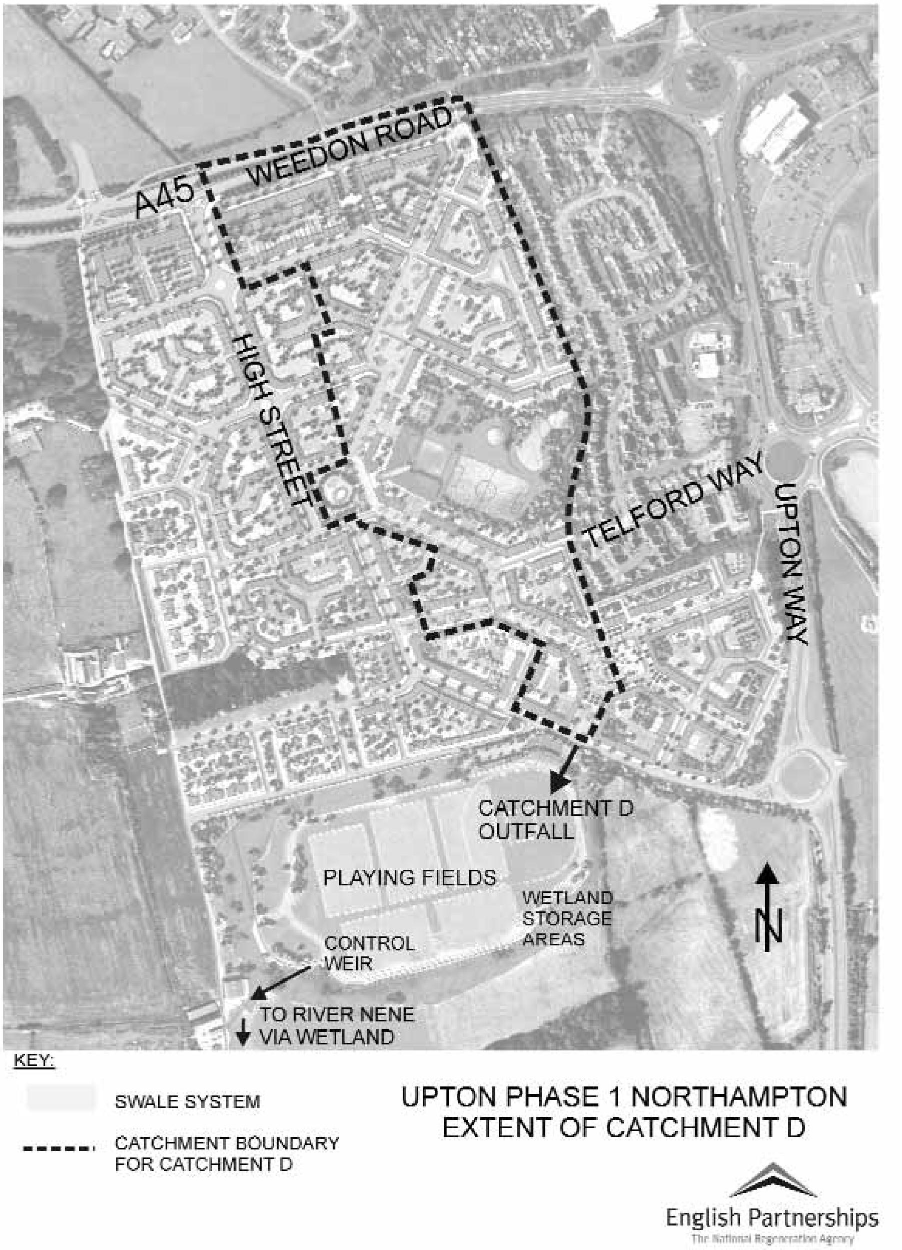


Figure 16.1 *Overview of catchment D including details of the outfall from the surface water drainage system (courtesy English Partnerships)*

The scheme had support from the Environment Agency who approved a flood risk assessment for Upton phase 1 in 2003. Northamptonshire County Council, Northampton Borough Council and Anglian Water were also supportive of the sustainability approach and the principles involved, but difficulties emerged in early consultations regarding the adoption of swales and the linking pipework and flow controls. Legal and statutory challenges outweighed a shared appreciation of the potential benefits and hampered progress of the scheme.

Surface water drainage was to be delivered primarily through a SUDS scheme. The SUDS scheme consisted of a system of linked swales that convey runoff to wetland storage areas around playing fields adjacent to the River Nene as well as having a storage and infiltration function. Permeable paving, rainwater harvesting and water butts were to be provided by developers of individual sites. This led to difficulties over adoption of the surface SUDS elements.

Permeable paving to courtyards is a significant SUDS element. In addition, Anglian Water applied the condition that infiltration devices could not be connected to the adoptable piped system. This meant that permeable paving connected to the piped system had to be tanked. Furthermore AW required that hydraulic design for the adoptable sewers had to demonstrate self cleansing with the permeable paving operating as designed (ie discharge attenuated) while having capacity to receive additional runoff, should the permeable paving fail (ie discharge unattenuated).

None of the stakeholders would agree to adopt the surface SUDS components. To resolve this, it was agreed that the Upton Management Company, which has English Partnerships and Northampton Borough Council backing, would undertake the necessary maintenance.

16.3 Drainage of developed areas

The surface water drainage system consists of a combination of SUDS elements and a conventional below ground piped system.

At the building level, water butts are to be provided by the developer for water to be stored for use in gardens. Rainfall is collected by conventional rainwater systems where it is passed on to a piped system. From there it is discharged to a series of swales. Car parking courtyards and some adoptable mews or lanes will have permeable paving (subject to agreement of details with the highway authority). The overall drainage layout is shown in Figure 16.2.

The general nature of the immediate subsoil within the development area is slightly sandy clay (Glacial Lake Deposits) that has variable permeability. The water table is also variable and there is the possibility that groundwater will affect infiltration at certain times of the year. If the groundwater is high the swales act as ditches. Although infiltration will be significant during most rainfall events, the design of the swale system does not assume infiltration and allowance has been made in the hydraulic design for groundwater inflow. The swale system design has been based on conveyance/storage. The swales are predominantly about 10 m wide and 1.2 m deep with side slopes varying from one in three to about one in five. Flow controls are orifice plates located in chambers or slots/steps in weirs.

In order to ensure that flow could discharge into swales, some pipes had to be laid at shallow depth, with a minimum cover of 800 mm. The highway authority required the use of minimum 300 mm diameter ductile iron pipes beneath the highway in such cases.

16.4 Interaction between the minor and major systems

The minor drainage system was designed such that no surface flooding occurred for rain events more frequent than the one in 30 year event (annual probability 0.033), though in practice the detailed design of the swales led to them achieving conveyance for larger events without overtopping.

The SUDS elements provide a series of "green corridors" through the development. The corridors also form a natural pathway for exceedance flows to be conveyed. In addition, some of the highways are available to act as above ground flood channels. Because of the range of drainage elements used in the development (including SUDS), interactions between the minor and major drainage systems are complex. They may be broken down into four categories:

- surface flow generated by limited inlet capacity. This includes drainage from conventional surfaces such as highways
- flow from permeable surfaces when infiltration is inhibited by high groundwater levels
- flow discharged from manholes and gully inlets due to surcharged sewers
- flow overtopping the banks of swales.

The design for conveyance and storage for exceedance conditions was carried out in stages. The first stage was to complete a conventional design for the building drainage, infiltration surfaces, pipe sewerage and swales, to meet the one in 30 year level of protection. An outcome of this was a hydraulic model of the drainage system capable of simulating its performance. This model was then used to perform a risk assessment of the drainage proposals for more extreme events. Finally any additional design for surface conveyance and storage of exceedance flow was undertaken. Details of the risk assessment and subsequent additional design are explained in the following sections.

16.5 Risk assessment

A level 2 study of the performance of the designed drainage system was chosen based on the size of the drainage area (Figure 16.1) and the complexity of the drainage system (Box 9.1). The model was used to simulate the performance of the drainage system. This allows the contributing areas, piped drainage components, and SUDS systems to be accurately represented. It also has the advantage of displaying outputs in GIS and 3D formats, so that results can be easily visualised.

16.5.1 Collecting data and building a hydraulic model

The landowners and consultants provided the outline design information (as listed below) to facilitate the modelling to commence.

- digital terrain data for the proposed development
- highway design details for major highways, with the centre line, camber, levels and kerb information
- routes of the swales and piped sewerage system.

Details of infiltration pavements were not available, and minor highways were not included. The swales were modelled without infiltration.

Figures 16.2 and 16.3 show details of the drainage model. Potential above ground flood pathways were identified by inspection using the model and digital terrain data, and included parts of the highway system and pathways formed by the swale system. These pathways were included in the model as open channels, as described in Section 9.3.3 and Appendix 3. The piped sewerage system was modelled with a free outfall.

16.5.2 Assessing system performance (one in 30 year return period – 0.033 annual probability)

To test the performance of the drainage system (outline design), the model was used to assess what flooding would occur for the 30 year return period storm. Rainfall durations from 15 minutes to 24 hours for both summer and winter events were simulated.

No flooding was detected in the pipe system nor in the SUDS structures, and no flow was conveyed in the above ground flood channels. No properties or surface areas were affected. The design met the requirement for a 30 year level of protection from flooding.

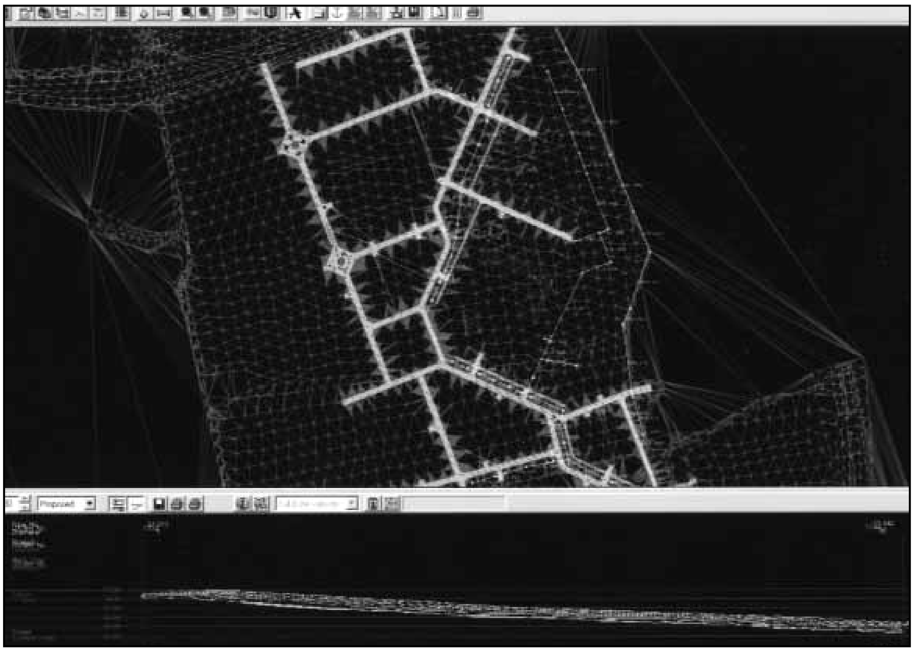


Figure 16.2 *Plan and section of ground model including the position of the highways, swales and surface water pipe system (courtesy Micro Drainage Ltd)*

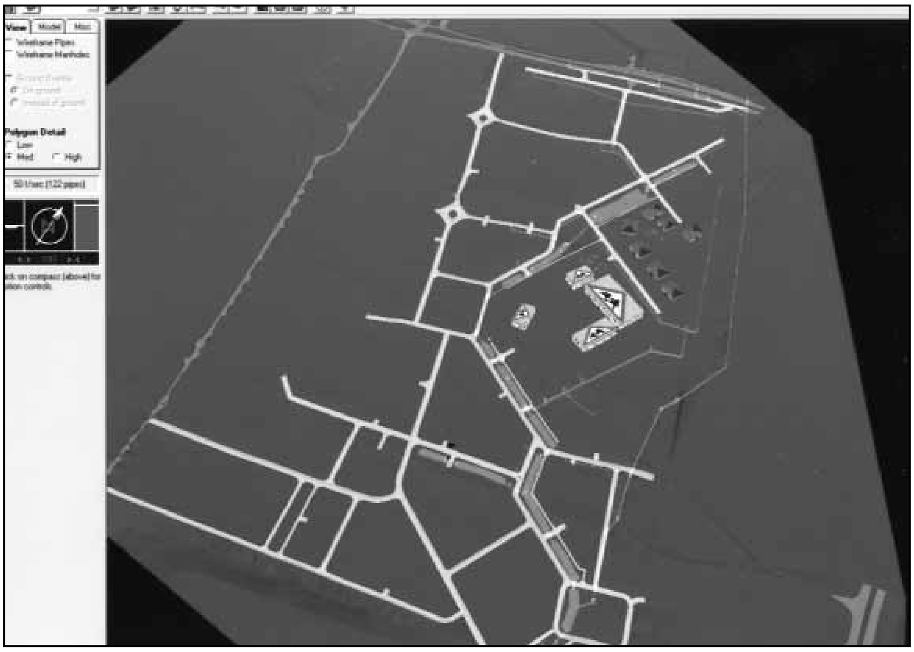


Figure 16.3 *3D view of ground model with conceptual view of the school and some housing (for illustrative purposes (courtesy Micro Drainage Ltd)*

16.5.3 Assessing system performance (one in 100 year return period – 0.01 annual probability)

The network model was then run with the 100 year return period storm event (probability of occurrence of 0.01). A range of durations was used for both summer and winter events to determine the most critical. Further simulations were undertaken with an allowance for climate change. Rainfall increased by 10 per cent as recommended in CIRIA publication C609 *Sustainable drainage systems – hydraulic, structural and water quality advice* (Wilson *et al,* 2004).

In this case surface flooding was identified, with surface flows being conveyed in the above ground flood channels included in the model. The flow paths indicated that flooding would occur around the school and at the housing indicated on the right of Figure 16.4. It should be noted for the purposes of demonstration in this case study, the school's position had been moved to illustrate flooding implicitly and in its designed location would not have been affected. In addition to this, the terrain data was amended to produce flooding to the property as shown in Figure 16.4.

16.5.4 Assessment of risk outside school

The highway (indicated by 'A' in Figure 16.4) acts as part of the major system for overland flow. It was necessary to confirm that the velocities and depths of flow did not pose a risk to traffic and pedestrians, particularly young children (as described in Section 10.5.5).

For the critical rainfall event, the rate, depth and velocity of the exceedance flow conveyed by the highway were computed. Details of the flood flow in this highway are shown in Figure 16.5. The surface flow was due to a combination of the minor system capacity being exceeded from the manhole upstream and surface flow that was designed to be conveyed in the highway as part of the highway drainage design. The maximum velocity was found to be 0.98 m/s at a depth of 50 mm (half kerb height). This was felt not to pose a significant risk to either parked or moving vehicles, or pedestrians.

The site is gently sloping and it is unlikely that the velocity and depth of flow in the landscaped areas would pose a high risk to pedestrians (see Chapter 10 and 11).

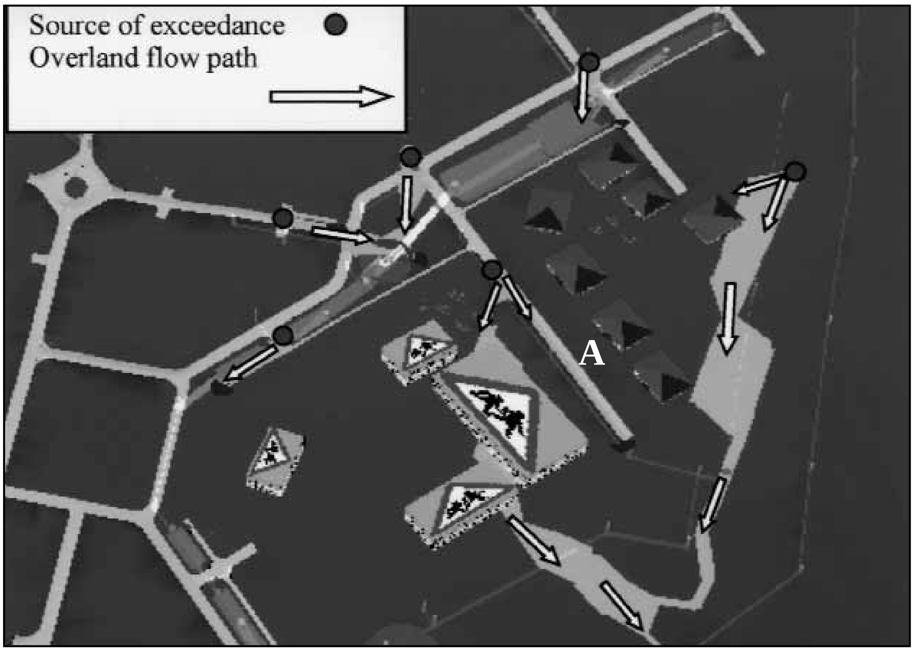


Figure 16.4 *Flood flow paths for 100 year event with additional flooding due to climate change indicated by the lighter colour (courtesy Micro Drainage Ltd)*

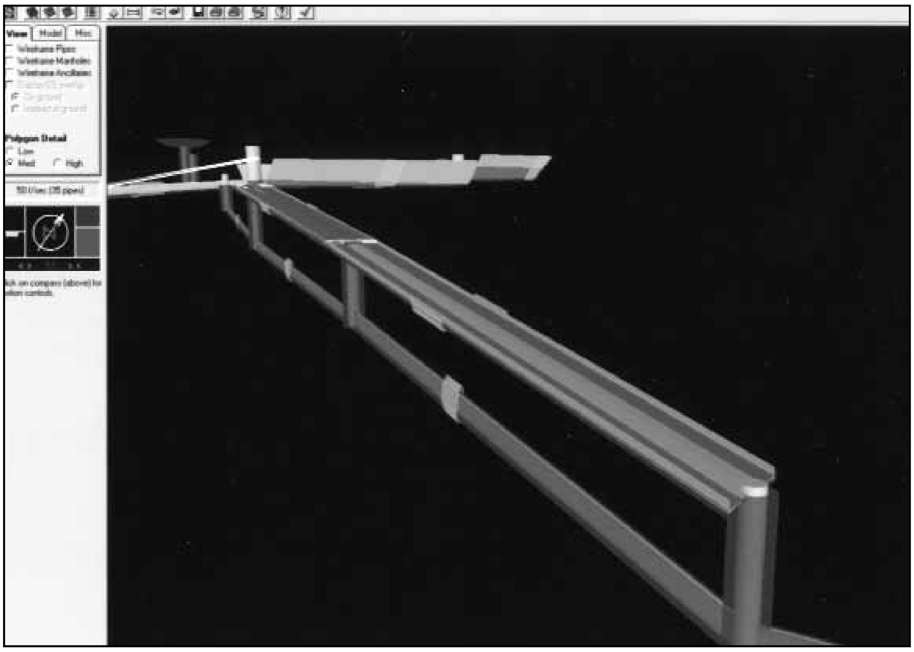


Figure 16.5 *3D view of the modelling of surface flood paths and the inter-connections with the below ground sewerage system, in the vicinity of the school (courtesy Micro Drainage Ltd)*

16.6 Building layout and detail

The "green corridors" provided by the swales and the principal highways provide the network of above ground flood pathways around the development. Local detailing of building layout and landscape is also important to ensure that when these systems operate in extreme events, individual property remains protected from flooding. Two potential problems had been identified in the risk analysis, the school and a single property in the vicinity, as illustrated in Figure 16.4. The following sections describe the remedial measures undertaken to manage this risk.

16.6.1 Amending building layout and threshold levels

A number of options were available to prevent the flooding of the school and housing. The proposed solution was to raise the threshold levels of the school. This forced the above ground flow to be retained within the highway, passing it safely downstream towards the outfall. The flooding of the individual property was tackled by re-profiling the ground locally, to create an above ground channel to convey flow away from the housing area. Other options included locating storage in the area, raise ground levels, and an above ground conveyance channel such as a swale or highway, but these were considered to be less practical.

The option of raising threshold levels at the school would be combined with raised pathways enabling a safe means of escape from the building if it was occupied during an extreme event.

The model was then re-run with the modifications described and these were shown to have successfully alleviated the flooding (Figure 16.6).

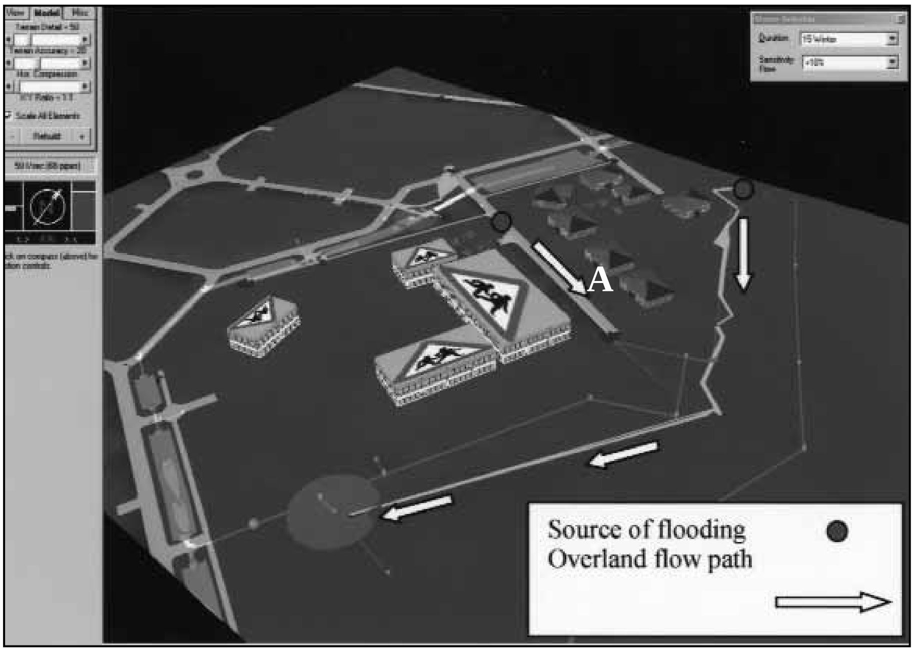


Figure 16.6 *Modification to building layout with school threshold levels raised and ground re-profiling adjacent to housing. (courtesy Micro Drainage Ltd)*

16.7 Impact on downstream system

Following a review of the Upper Nene catchment after the April 1998 flooding, which resulted in loss of life in Northampton, a development requirement was that there should be no impact on the downstream system resulting from events up to the one in 200 year return period (0.005 annual probability). An initial assessment of the site, following the procedures set out in Chapter 4, established the greenfield runoff. The proposed development, even allowing for effects of source control in the SUDS, would have had a significant impact on flood flows in the River Nene for the 200 year event.

To mitigate this effect, surface storage was planned, to attenuate exceedance flows on the surface. A suitable location for this was found in the grounds of the school. By reviewing the ground topography, and the level in the receiving river, the maximum depth of storage was determined. From this a suitable outlet control was designed to limit the discharge.

The storage pond was then incorporated into the model with an initial estimated volume. The data was entered to allow for the varying surface area with depth that would occur with the real pond. By successive trial and error the required volume that prevented overtopping for the 200 year event was determined. The whole of this volume could be accommodated below the level of the surrounding ground, so no special measures were required to protect an embankment in the event of overtopping. The location of the storage pond is shown in Figure 16.6. Similar storage ponds were provided in other phases of the development.

As the storage has been provided in an open space, stores only surface runoff, and operates infrequently, there will be few requirements for clean up following an extreme event. Care needs to be taken in detailing such designs to ensure they do not pose a safety hazard when in operation. This means careful attention to side slopes and the detailing around the flow control.

16.8 Conclusion

The case study demonstrates the importance of considering drainage design early in the development process, and also justifies early stakeholder involvement. It also shows that although SUDS can mitigate the effects of flooding, they are insufficient on their own to alleviate the risk of flooding from extreme events.

By carefully identifying the paths for exceedance flood routes through a development, the damaging effects of flooding from extreme events can be relieved. In this case the conveyance of the large resulting flood volumes is more cost effective than local storage. However, discharging these flows into the receiving river would have proved unacceptable due to the potentially damaging effects of consequential flooding downstream. The provision of local surface storage in a dual use area has in this case helped to mitigate these effects.

The use of suitable modelling software in this case considerably aided the design process. Although reference is made here to a particular software product, other products are available that can provide a similar function, and users should make their own judgement as to the most suitable.

The staged approach to modelling and flood risk assessment provides a useful framework by matching the level of effort and cost to the perceived risks. In this case it was relatively straightforward to add additional detail to the level 2 model in parts where additional information was required.

Overall the resulting design delivers a level of flood protection substantially above that provided in many new developments with little additional cost. This was highlighted by the need to amend the design to create a flooding problem. This design also demonstrates how the general recommendations of *Sewers for adoption 5th edition* (Water UK and WRc, 2001) can be delivered in practice.

Part D Appendices

A1 Modelling exceedance

A1.1 Surface flood pathways

There are various methods used to represent surface flood pathways in network models. Where the software tool does not explicitly include an algorithm for modelling such pathways, it is common practice for the user to specify additional links between nodes using an open channel link. There are two possible alternatives:

- surface channel links of the major system connect between the existing nodes of the minor system
- a separate system of nodes and links are used to specify the major system, with appropriate interconnecting links between the nodes of the two systems.

The former has the advantage of requiring the minimal modification to the minor system network. However it constrains the major system in that it can only connect between the minor system nodes (though some applications will allow the creation of additional nodes where necessary). Also it is not usually possible to assign separate contributing areas to the major system with this method.

The latter, which is illustrated in Figures 9.6 and 15.7, has the disadvantage that additional network modelling is required. However, the major system layout is no longer constrained to that of the minor system, and the links between the two sets of nodes can be used to accurately model transfers of flow between the two systems. Care should be taken with the dummy manholes specified in this method as they may confuse the algorithms in the software into performing additional and inappropriate calculations in relation to manhole head losses and additional storage volume.

The accuracy of modelling the surface pathway section will also depend on the algorithms used to define the open channel model. Some software allows compound channel shapes to be represented, so that flow within and without the highway can be represented. Others restrain the user to simple rectangular sections, which should be used with caution.

The facilities for viewing output also vary between different software packages. Where the results can be shown graphically and in real time, in 3D format, the user can gain a greater understanding of the movement of floodwater in the drainage area (Figure A1).

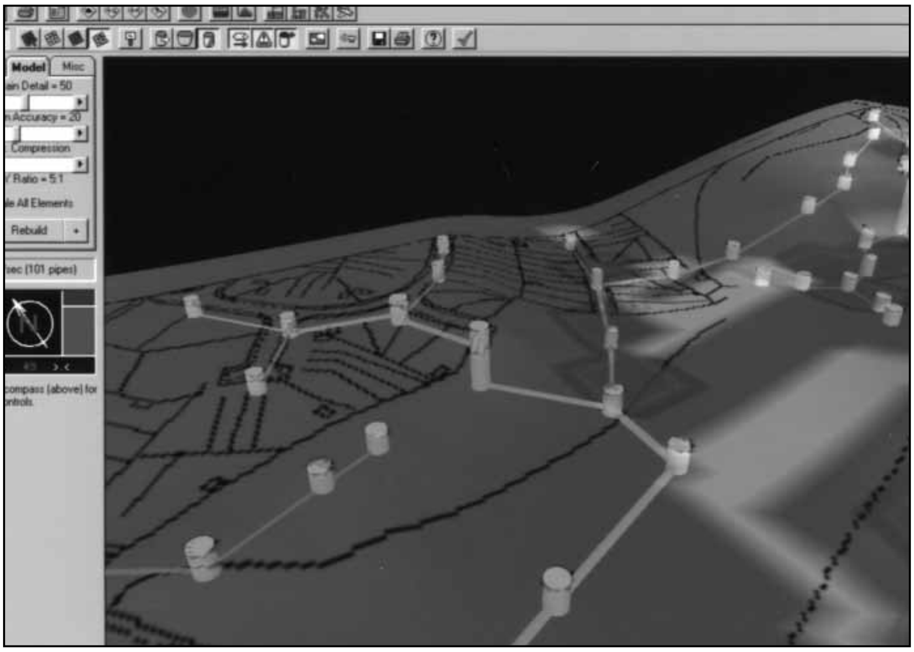


Figure A1.1 *3D graphical representation of surface flooding (courtesy Micro Drainage Ltd)*

A1.2 Surface flooding

Most software packages will simulate flooding from manholes (nodes) in the network. The extent and surface depth of flooding will depend on the area over which the flood volume is distributed. There are a number of possible options available to the user, depending on the software specification:

- flood water lost from the system
- user specified flood area
- user specified flood volume
- flood routed through surface flood pathway
- flood distributed in 3D according to ground terrain.

Where flood water is lost from the system there is no two-way interaction with the minor network. This can result in an overestimation of flood volume at the flooded node and an underestimation of flood volume downstream. However it is a simple model and useful for a level 1 study or as a first pass to a more complex study.

Where a flood area or volume is specified, floodwater will be returned to the system at the point of flooding. In the former case, the floodwater is distributed uniformly over the flooded area. In the latter case the floodwater may be unevenly distributed according to the user specified shape of the flood volume. In either case, floodwater can only be returned at the same point from which it is discharged.

Where flood pathways are modelled, floodwater may be returned to a different node from which it originated. Floodwater will be transferred over the surface. However, the overland flow model is constrained by the software and the data available, and surface pathways are usually a simplification of reality.

3D surface flood modelling is still in its infancy, though a few research models have been developed using finite element analysis. This approach requires significant and detailed digital terrain data and substantial computing power. However it sets out the likely route for future software development. A simpler approach to allowing for three dimensional ground effects is to use a "rolling ball" algorithm to identify a surface flood pathway. The "rolling ball" algorithm works with a digital terrain model and identifies the line of greatest slope in each grid. These are then joined together to form a surface flood pathway. This will convey flow to a low point, or to another part of the minor system, as shown in Figure A1.2.

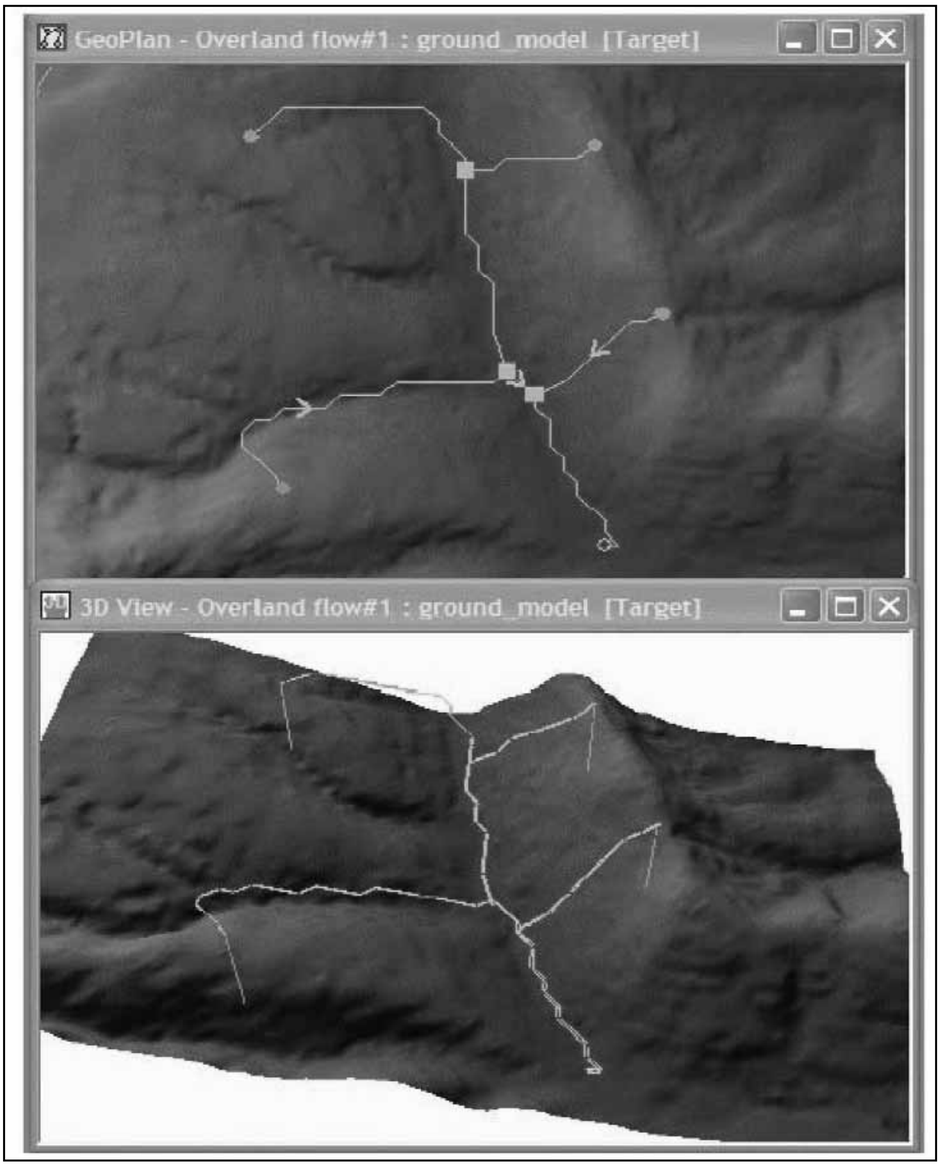


Figure A1.2 *Overland flow paths identified with "rolling ball" algorithm (courtesy Wallingford Software)*

Some software enables the user to assess the direction where above ground flows may split when flow reaches a crest. This accounts for flow that may split in several different directions from a single source.

A1.3 Modelling inlet capacity

As explained in Chapter 9, the capacity of different drainage inlets is limited, and this can be important in extreme events. Some software explicitly allows for modelling the restrictive effects of inlet capacity. This may be done by applying a throttle to the inlet, and including a notional reservoir to allow for subsequent storage on the surface upstream. This is illustrated in Figure A1.3, where the algorithm for a flooded node is adapted to restrict the flow passing in from the surface.

Normally, each individual area contributing to a gully would not be represented, rather they would be grouped together to represent a number of areas draining to a notional gully.

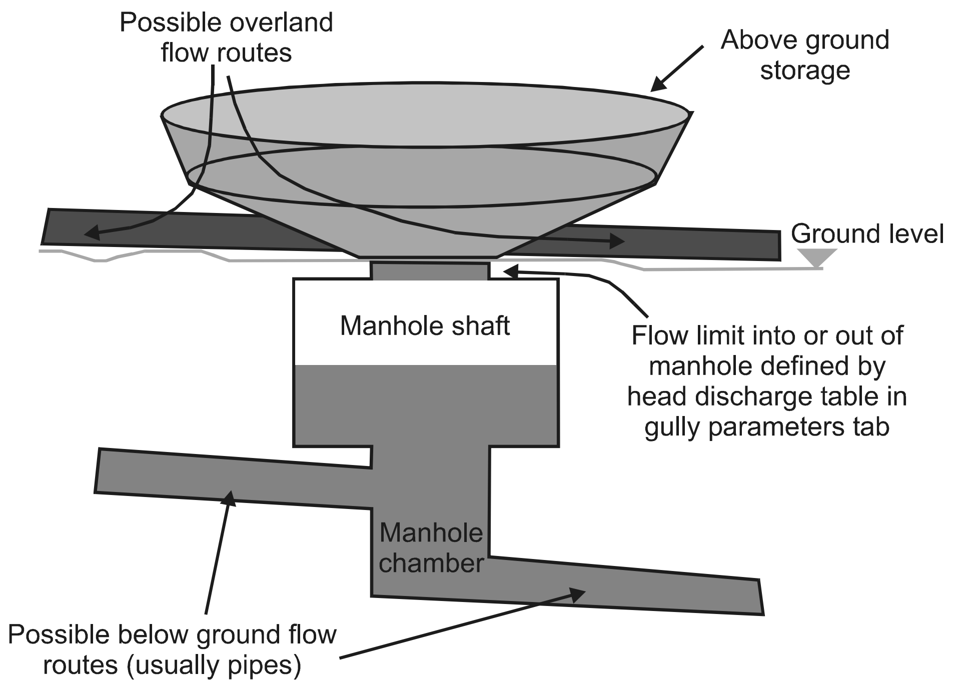


Figure A1.3 *Model of gully inlet (courtesy Wallingford Software)*

A1.4 Modelling flood risk

Some software packages will perform additional calculations to estimate the direct flood risk to property. Risk will depend on the likelihood that property will flood, and the consequence if it floods. The likelihood of flooding depends on two factors: the level of the property in relation to the level of surcharging in the minor system, and the proximity of the property to surface flood paths in areas of flooding. This is illustrated in Figure A1.4. The figure also illustrates that property below the level of the minor system, or with a cellar, may flood even where no surface flooding is indicated at the node of the minor system.

The consequence will depend on the depth to which the property is flooded which is usually determined by the local depth of surface flooding, as illustrated in Figure A1.4. A risk score can be developed by combining these two factors. Results can be indicated on background mapping using GIS, as illustrated in Figure A1.7.

Some software adopts a sensitivity analysis approach that includes the potential of climate change combined with the ability to model watercourses and rural, and urban runoff. This gives a holistic approach to the prediction of flood pathways and flood levels.

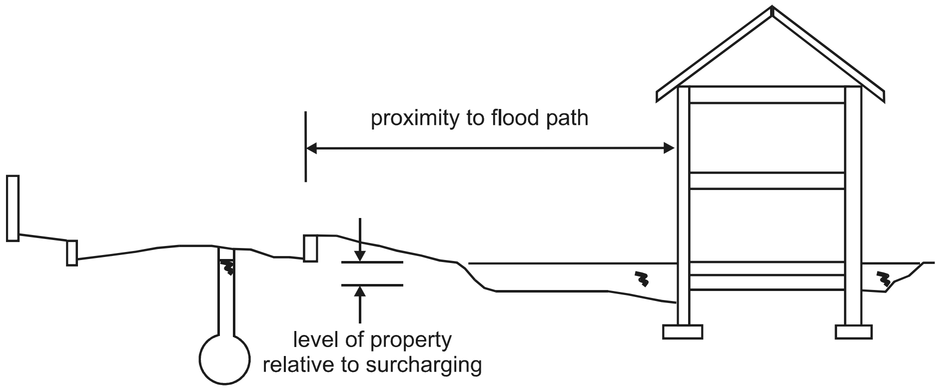


Figure A1.4 *Property below level of minor system, showing flooding due to sewer surcharging*

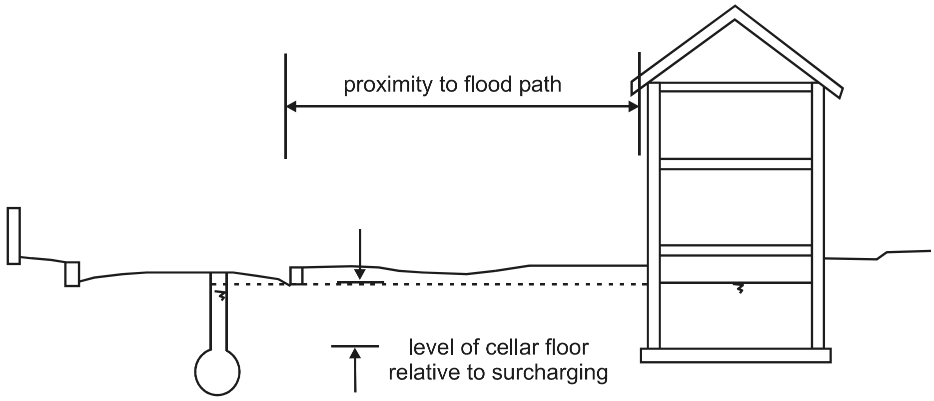


Figure A1.5 *Property with cellar, showing flooding due to sewer surcharging*

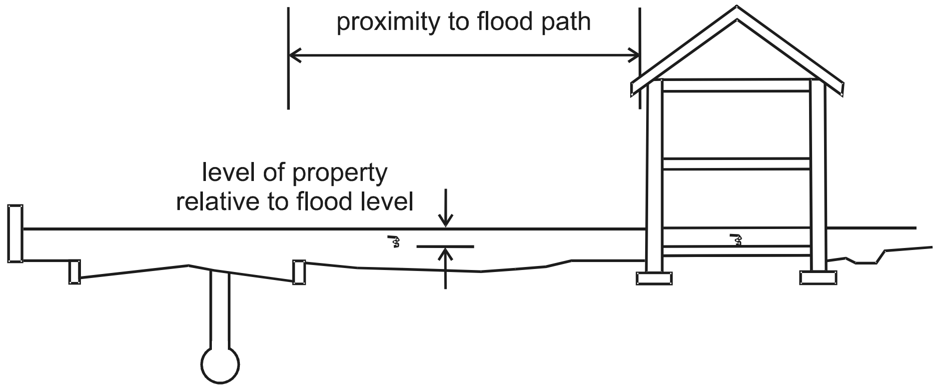


Figure A1.6 *Property above level of minor system showing flooding due to local surface flooding*

A1.5 Further guidance

It is not practical to give detailed guidance on individual software packages here. Practitioners are advised to seek further guidance from the suppliers of such software or to use help functions provided.

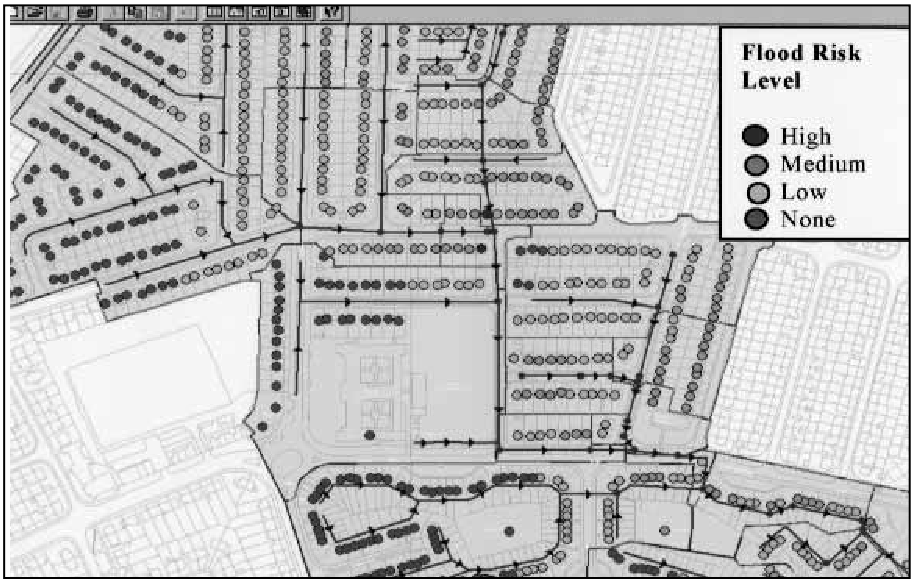


Figure A1.7 *Flood risk scoring at the level of individual property (courtesy MWH)*

A2 Exceedance flow at highway gully inlets

The contents of this appendix are based on Highways Agency report HA102/00, *Spacing of highway gullies* (Highways Agency, 2000)

The design of gully inlets is based on the knowledge that a proportion of flow bypasses the gully grating under design conditions and continues along the channel of the highway. A gully receives flow from its local contributing area plus a proportion of the flow from upstream areas. It passes a proportion of this total flow onto the next gully downstream. Ultimately the excess flow is collected by an additional gully at the low spot. It is appreciated that highway drainage is based on the premise that the highway acts as a drainage channel conveying major system flow.

With extreme events the flow in the highway channel will increase considerable. However, this does not significantly affect the efficiency of the gully grating (see Table A2.1), providing that the width of flow in the highway channel is not excessive (not >1.5 m). Even where efficiencies are reduced due to poor maintenance highway gullies can still achieve a steady state condition where the inflow to a group of gullies equals the runoff from the contributing areas.

When assessing the effect of extreme events on highway drainage, the engineer needs to consider the major system flow in the highway channel to determine whether the depth or width of flow is such that a significant proportion of flow may be diverted from the channel and ultimately not collected by the highway gullies. Such diverted flow would become the exceedance flow.

However, using the methodology set out in Appendix 2, even with rainfall intensities up to 300 mm/hr, and using typical contributing areas, it is unlikely that depth of flow will be greater than 100 mm.

The flow bypassing a highway gully is shown in Figure A2.1 and represented by Equation A2.1.

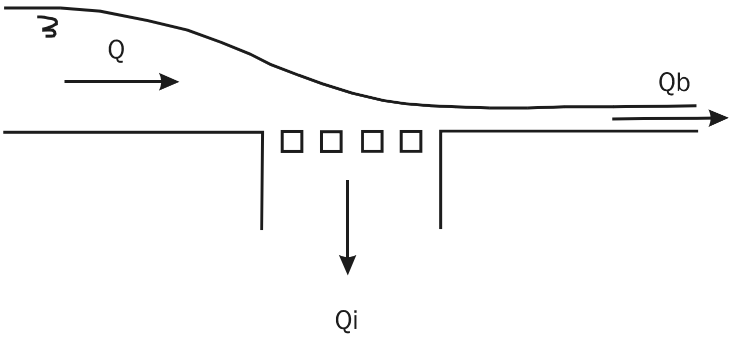


Figure A2.1 *Flow split at highway gully inlet*

The flow entering the gully, Qi (m^3/s), is defined by the gully efficiency ç, and is given by the equation.

$$Qi = \frac{ç}{100} Q \qquad (A2.1)$$

where: $\eta \quad = \quad m\left(100 - Gd\left(\frac{Q}{H}\right)\right)$

Q = flow n highway channel approaching the gully
G*d* = gully factor defined by the gully grating geometry
H = average depth of flow in the gully channel
m = factor to allow for maintenance

Note that a suitable maintenance factor for design would be 1 or 0.9, and a minimum value used for exceedance conditions 0.7 (assumes worse maintenance).

It follows that the flow by-passing the gully is

$$Qb = (1 - ç)Q \tag{A2.2}$$

Qr is the flow (m^3/s) collected from the contributing area to each gully, and is given by the Rational equation,

$$Qr = 2.78Ae.i \tag{A2.3}$$

where:

Ae = contributing area as defined by Figure A2.2 (ha)
i = mean rainfall intensity (mm/hr) for 5 min duration

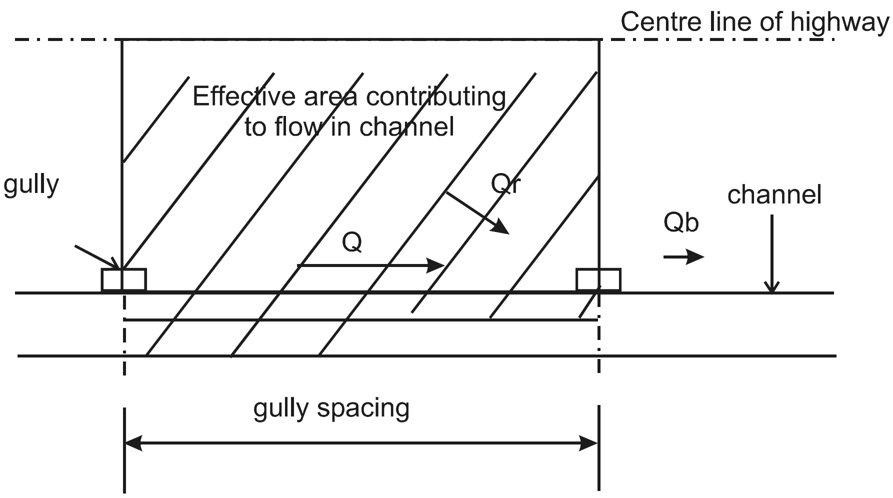


Figure A2.2 *Effective area contributing to flow in channel - note that ç is calculated at the design flow*

Q is also the flow conveyed in the highway channel (m^3/s), made up of Qr + Qb, and is related to the width and depth of flow by the Manning Equation

$$Q = \frac{1}{n} A_F R^{\frac{2}{3}} s_L^{\frac{1}{2}} \tag{A2.4}$$

where:

A_F = cross-sectional area of flow in channel (m^2)
R = hydraulic radius = AF/P (m)
P = wetted perimeter of flow section (m)
s_L = average longitudinal slope of highway
n = Manning roughness for highway surface

Figure A2.3 shows the cross section of flow in the channel.

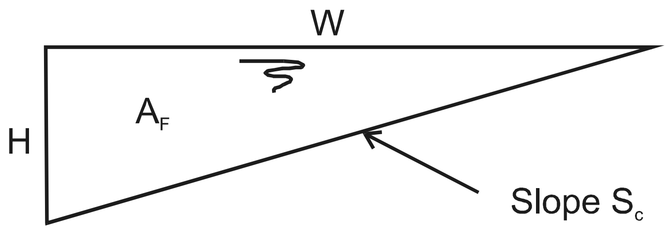


Figure A2.3 *Cross-section of flow in the channel*

It can be shown that:

$$A_F = \frac{1}{2}\frac{H^2}{s_C}$$

$$R = \frac{H}{2(s_C + 1)}$$

$$Q = \frac{H^{\frac{8}{3}} s_L^{\frac{1}{2}}}{2^{\frac{5}{3}} n.s_C (s_C + 1)^{\frac{2}{3}}}$$

and $H = K_1 Q^{0.375}$ (A2.5)

where: $K_1 = 1.54\ (n.s_c)^{0.375}\ (s_c + 1)^{0.25}\ S_L^{-0.188}$

Example: For an urban road with gullies designed for a one year return period rainfall, the exceedance flow for the 30 year rainfall may be calculated as follows.

Data:
Gully spacing = 12.4 m
Effective width of contributing area = 6.5 m
Contributing area = 0.00806 ha
Longitudinal slope sL = 1 in 40 (0.025)
Crossfall = 1 in 33 (0.030)

Maintenance factor for design = 1

Maintenance factor for exceedance calculation = 0.7

Design rainfall intensity for five min duration and annual probability of 1.0 = 55 mm/hr

Exceedance rainfall intensity of five min duration and annual probability of 0.02 = 148 mm/hr

Runoff from contributing area per gully $= 2.78 \times 0.00806 \times 55 = 1.23$ l/s for the design event and $2.78 \times 0.00806 \times 148 = 3.32$l/s for the exceedance event.

For the design conditions make a first estimate of gully efficiency using a channel flow equal to the runoff.

From Table A2.1, $K_1 = 0.173$ so that the depth of flow in the channel is:

$$H = 0.173 \times 0.00123^{0.375} = 0.014 \text{ m} = 14 \text{ mm}$$

so that the gully efficiency

$$ç = 1.0\left(100 - 60\left(\frac{1.23}{14}\right)\right) = 95\%$$

Under steady state conditions the gully inlet flow Qi equals the runoff from the contributing area Qr = 1.23l/s.

The channel flow $Q = \frac{1.23}{0.95} = 1.29$ l/s and the by-pass flow is 0.06 l/s

Reworking the above calculation with a flow of 1.29 rather than 1.23 l/s leaves the gully efficiency unchanged at 95 per cent.

For the exceedance conditions, assume initially that the gully efficiency can be derived from the design value as:

$$\frac{0.7}{1} \times 95 = 66\ \%$$

As explained above, the inflow to the gully equals the runoff = 3.32l/s so that the channel flow becomes

$$Q = \frac{3.32}{0.66} = 5.03\ \text{l/s}$$

For a channel flow of 5.03 l/s the depth of flow in the channel now becomes

$$H = 0.173 \times 0.00503^{0.375} = 0.024\text{m} = 24 \text{ mm}$$

The gully efficiency is now:

$$ç = 0.7\left(100 - 60\left(\frac{5.03}{24}\right)\right) = 61\ \%$$

The channel flow is now reworked as:

$Q = \dfrac{3.32}{0.61} = 5.44 l/s$ and the by-pass flow is 2.12 l/s

The depth of flow in the highway channel may now be calculated using Equation A2.5.

$H = 0.173 \times 0.00544^{0.375} = 0.025m = 25\ mm$

For a cross-fall of one in 33, the width of flow becomes 0.825 m, which is within the required 1.5 m.

The flow entering each gully is 3.32 l/s which is well below the limiting capacity of 10 l/s for a gully with a 100 mm outlet. It follows therefore that even where drop kerbs exist, it is extremely unlikely that any of the extreme event runoff will result in exceedance flow due to the capacity of the gully inlets.

Table A2.1 *Values of K1*

n	sL	Sc	K1	n	sL	Sc	K1
0.015	0.010	0.005	0.104	0.015	0.035	0.005	0.081
0.015	0.010	0.010	0.135	0.015	0.035	0.010	0.105
0.015	0.010	0.015	0.157	0.015	0.035	0.015	0.122
0.015	0.010	0.020	0.176	0.015	0.035	0.020	0.136
0.015	0.010	0.025	0.191	0.015	0.035	0.025	0.148
0.015	0.010	0.030	0.205	0.015	0.035	0.030	0.159
0.015	0.010	0.035	0.217	0.015	0.035	0.035	0.168
0.015	0.010	0.040	0.229	0.015	0.035	0.040	0.177
0.015	0.010	0.045	0.239	0.015	0.035	0.045	0.186
0.015	0.010	0.050	0.249	0.015	0.035	0.050	0.193
0.015	0.015	0.005	0.096	0.015	0.040	0.005	0.079
0.015	0.015	0.010	0.125	0.015	0.040	0.010	0.102
0.015	0.015	0.015	0.146	0.015	0.040	0.015	0.119
0.015	0.015	0.020	0.163	0.015	0.040	0.020	0.133
0.015	0.015	0.025	0.177	0.015	0.040	0.025	0.144
0.015	0.015	0.030	0.190	0.015	0.040	0.030	0.155
0.015	0.015	0.035	0.201	0.015	0.040	0.035	0.164
0.015	0.015	0.040	0.212	0.015	0.040	0.040	0.173
0.015	0.015	0.045	0.222	0.015	0.040	0.045	0.181
0.015	0.015	0.050	0.231	0.015	0.040	0.050	0.188
0.015	0.020	0.005	0.091	0.015	0.045	0.005	0.077
0.015	0.020	0.010	0.119	0.015	0.045	0.010	0.100
0.015	0.020	0.015	0.138	0.015	0.045	0.015	0.116
0.015	0.020	0.020	0.154	0.015	0.045	0.020	0.130
0.015	0.020	0.025	0.168	0.015	0.045	0.025	0.141
0.015	0.020	0.030	0.180	0.015	0.045	0.030	0.151
0.015	0.020	0.035	0.191	0.015	0.045	0.035	0.161
0.015	0.020	0.040	0.201	0.015	0.045	0.040	0.169
0.015	0.020	0.045	0.210	0.015	0.045	0.045	0.177
0.015	0.020	0.050	0.219	0.015	0.045	0.050	0.184
0.015	0.025	0.005	0.088	0.015	0.050	0.005	0.075
0.015	0.025	0.010	0.114	0.015	0.050	0.010	0.098
0.015	0.025	0.015	0.133	0.015	0.050	0.015	0.114
0.015	0.025	0.020	0.148	0.015	0.050	0.020	0.127
0.015	0.025	0.025	0.161	0.015	0.050	0.025	0.139
0.015	0.025	0.030	0.173	0.015	0.050	0.030	0.149
0.015	0.025	0.035	0.183	0.015	0.050	0.035	0.158
0.015	0.025	0.040	0.193	0.015	0.050	0.040	0.166
0.015	0.025	0.045	0.202	0.015	0.050	0.045	0.174
0.015	0.025	0.050	0.210	0.015	0.050	0.050	0.181
0.015	0.030	0.005	0.085	0.015	0.055	0.005	0.074
0.015	0.030	0.010	0.110	0.015	0.055	0.010	0.096
0.015	0.030	0.015	0.128	0.015	0.055	0.015	0.113
0.015	0.030	0.020	0.143	0.015	0.055	0.020	0.125
0.015	0.030	0.025	0.156	0.015	0.055	0.025	0.136
0.015	0.030	0.030	0.167	0.015	0.055	0.030	0.146
0.015	0.030	0.035	0.177	0.015	0.055	0.035	0.155
0.015	0.030	0.040	0.186	0.015	0.055	0.040	0.163
0.015	0.030	0.045	0.195	0.015	0.055	0.045	0.170
0.015	0.030	0.050	0.203	0.015	0.055	0.050	0.178

A3 Conveyance in surface flood pathways

A3.1 Introduction

The design of surface flood pathways is set out in Chapters 9 and 11 in Part B of the guidance. The purpose of this appendix is to give supplementary information on typical surface flood channel sections.

A3.2 Flood pathway channels

The following figures show some typical surface flood pathways encountered in urban areas. They are only intended to be indicative since the type, size and detail is infinite. However they will give a good indication of typical dimensions and surface roughness.

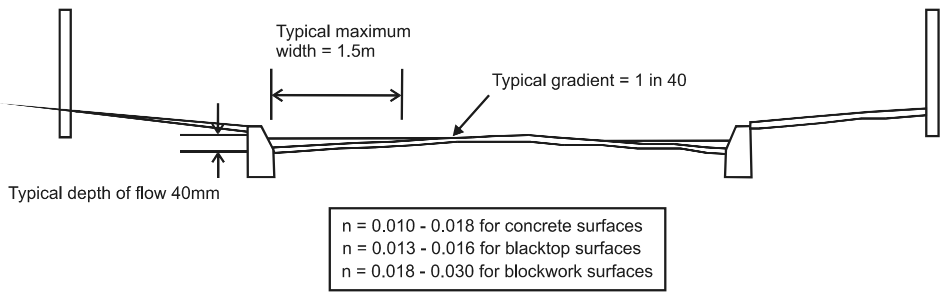


Figure A3.1 *Typical cross-section of flood pathway in a minor highway, showing surface conveyance for non-extreme events*

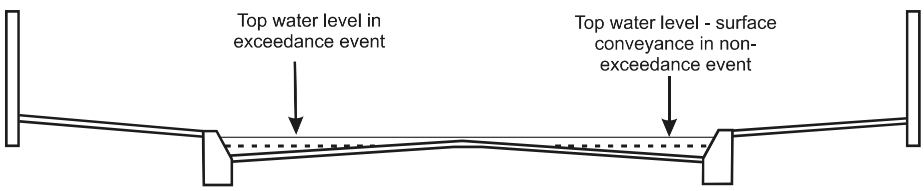


Figure A3.2 *As* Figure A3.1 *but with exceedance flow contained within the highway channel.*

Note: The exceedance flow occupies the space between the free surface and the surface of the normal drainage flow (shown dotted)

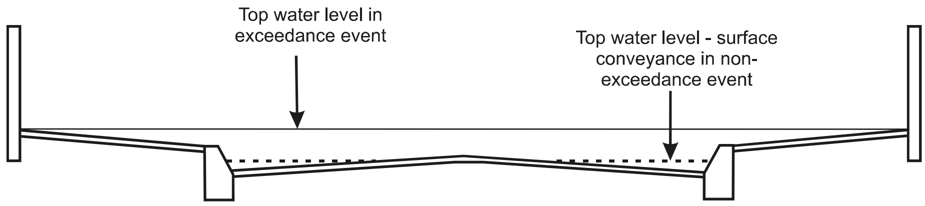


Figure A3.3 *As* Figure A3.2 *but with exceedance flow above top of kerb level*

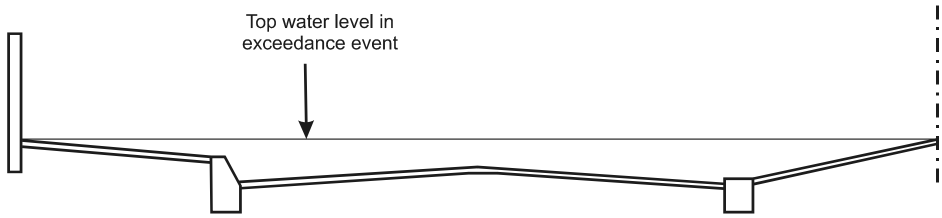


Figure A3.4 *As* Figure A3.3 *but showing dropped kerb detail on right hand side.*

Note: The back of footpath level is retained to prevent the surface flow being diverted off the highway

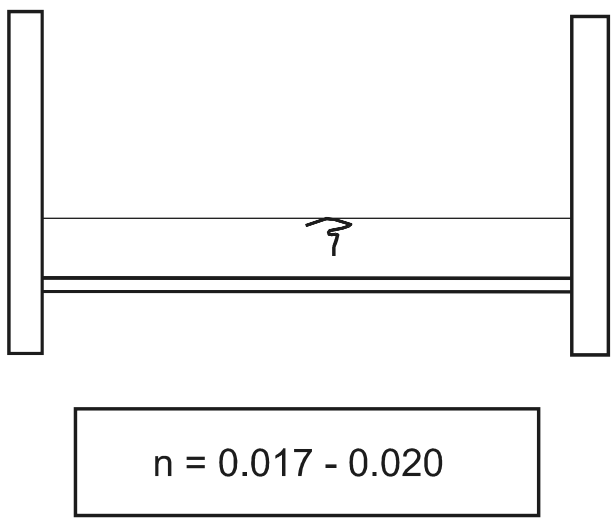


Figure A3.5 *Flood pathway on footpath between walls*

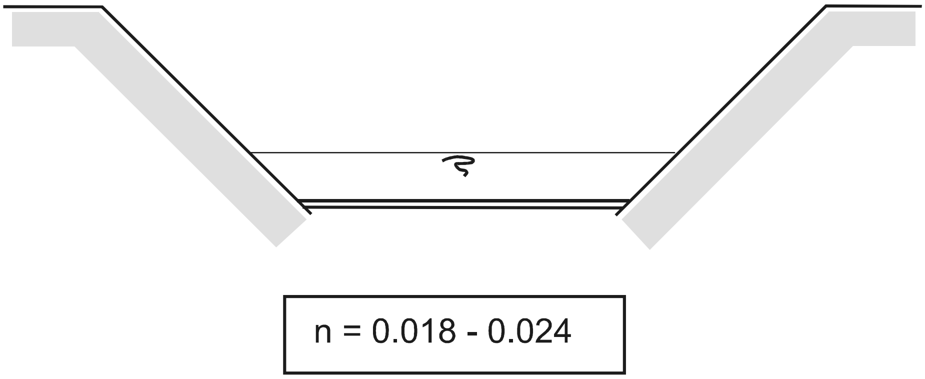


Figure A3.6 *Flood pathway on footpath between grass banks*

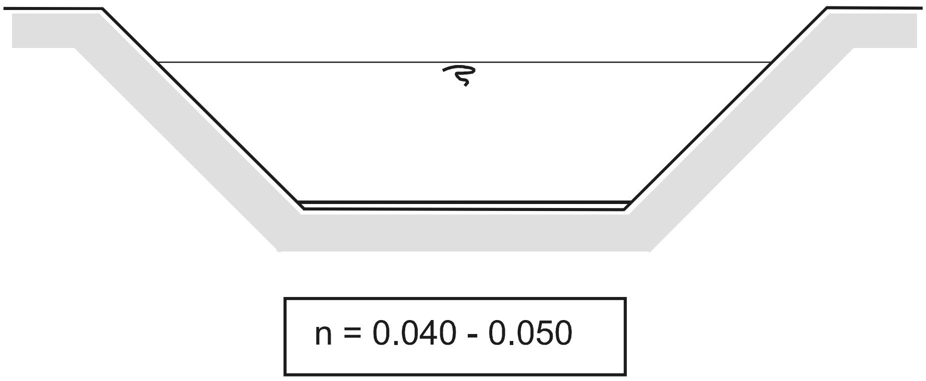


Figure A3.7 *Flood pathway in rough cut ditch*

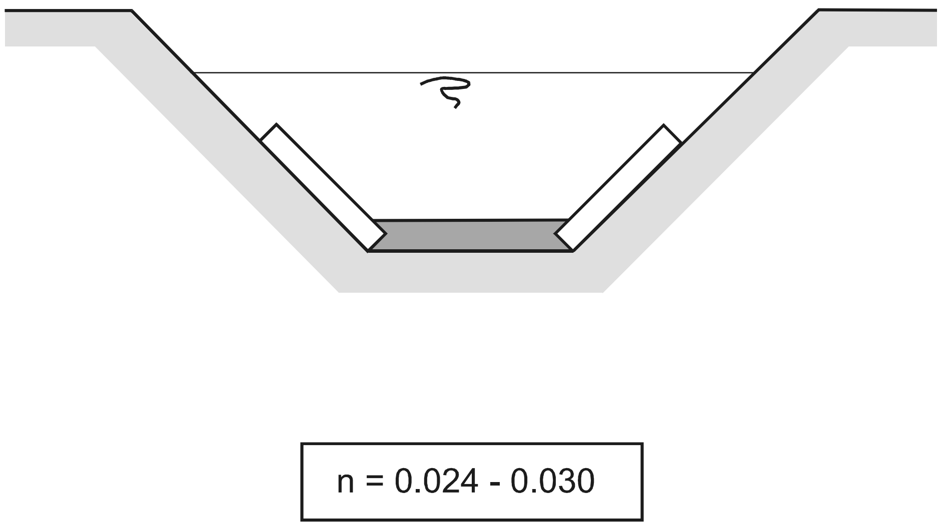


Figure A3.8 *Flood pathway in lined ditch*

A4 Assessment approach to determine flood volumes and rates from SUDS

A4.1 Assessment approach

An analysis was carried out on the most relevant SUDS units to evaluate their exceedance characteristics. The systems assessed were:

- pervious pavements
- swales
- infiltration systems.

The approach taken was to model the SUDS unit using a drainage modelling tool and apply rainfall to a drainage area. Examination of the model results enables an assessment of the hydraulic performance of the unit to be made, with peak flow rates and volumes quantified when the capacity of the unit was exceeded.

The performance of a SUDS unit is a combination of rainfall event and the volume of storage provided, so this analysis involved running a range of models. For example a swale of the same shape and length receiving runoff from the same storm and contributing surface area, will perform very differently when built with a slope of 1:20 compared with one of a slope of 1:200. The volume provided by the latter is greater and provides a corresponding higher level of service.

The results provide an indication of the exceedance performance of the SUDS systems, based on the various assumptions used in the analysis. Where these assumptions apply fairly closely with the drainage system being designed, these results can be used to provide an indication of the impact of rainfall exceedance and appropriate design carried out to address these excess flows.

There are too many variables to provide this information for all drainage situations. A generic indication of exceedance performance is provided in Appendix 5. This appendix is aimed at providing a first order indication of performance as a number of significant assumptions are made to enable this guidance to be developed.

A4.2 Hydrology

The hydrology of the country can be divided into eight categories by using the FSR parameters $M_5 60$ and rainfall ratio "r". These parameters measure the 60 minute rainfall depth of the storm with the probability of occurrence of 0.2 per year and the ratio of the 60 minutes to the two-day event of the same probability of occurrence. Figure D.1 illustrates these eight areas. Although FSR is generally accepted as having been superseded by FEH (*Flood estimation handbook*, 1999), this figure provides the simplest method of an overview of the hydrology across the country.

To minimise the analysis, rather than running eight different sets to represent all the hydrological regions, two representative regions were chosen. The FSR parameters selected for $M_5 60$ and rainfall ratio "r" were 17 mm and 0.2 which generally describes areas to the north and west of the country, and 20 mm and 0.4 which generally describes the south and east.

However it is recognised that FEH and climate change together means that use of the FSR rainfall would not be a conservative assumption for doing this assessment, particularly for extreme events. The rainfall has been factored up to take account of this.

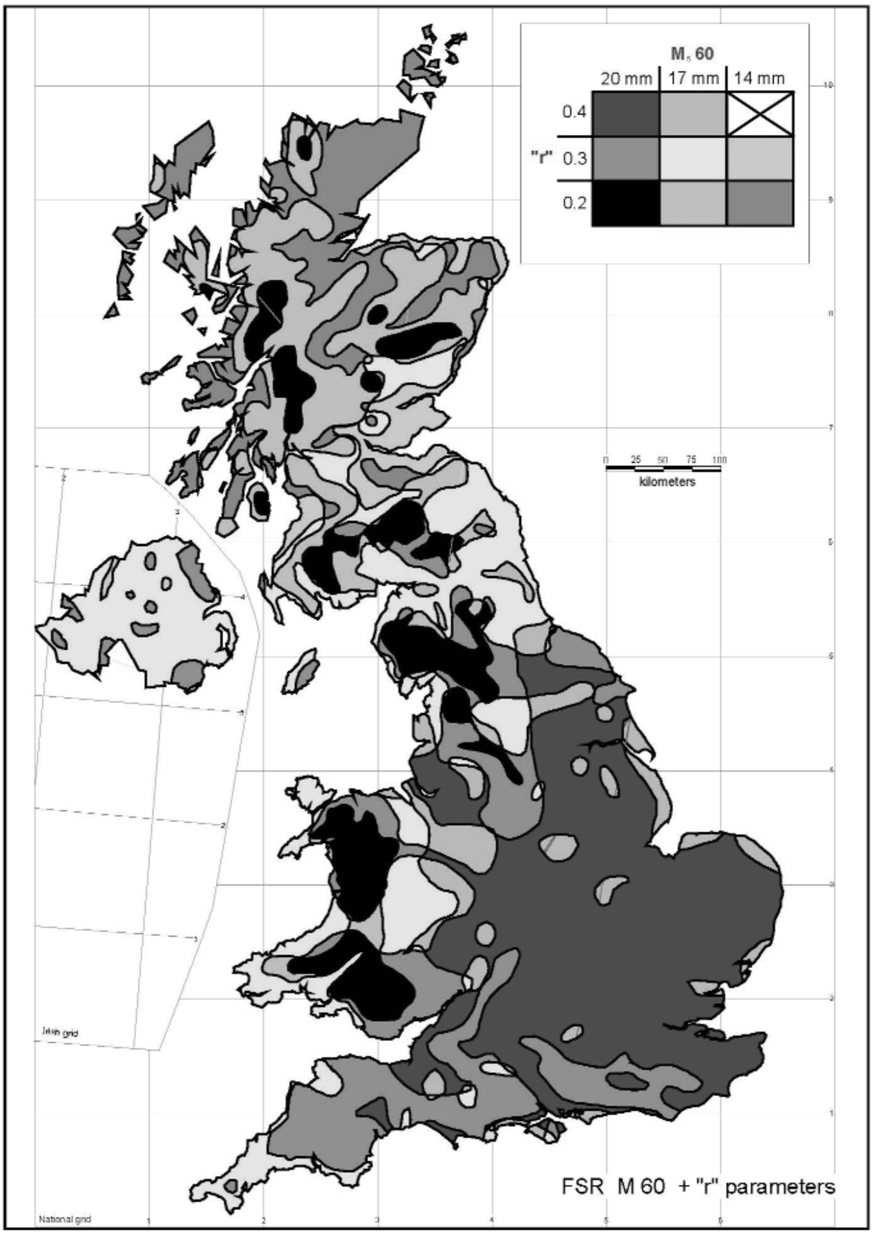


Figure A4.1 *FSR regions across the UK (courtesy HR Wallingford)*

A4.3 Pervious pavement performance

Pervious pavements are normally built with 350 mm of granular fill to a specific stone size mix. This provides around 30 per cent voids for water storage. High voids storage, around 95 per cent, can also be constructed using plastic crate type systems. These can be used for both smaller depths than 350 mm as well as depths of up to a few metres. The standard granular construction with 30 per cent voids has been selected for analysis.

The volume of storage is large even with respect to large rainfall events, and so the analysis also looked at the use of the pavement to provide storage for adjacent impermeable areas, and reflect common practice. The throttle rates used were not modified in assessing the extra demands made by these areas, though in practice hydraulic discharge criteria would probably increase proportionately.

Table A4.1 Summarises the assumptions used for assessing the hydraulic performance of pervious pavements and their impact on the major system.

Table A4.1 *Pervious pavements – modelling assumptions*

Pervious pavement	Characteristics and scenarios
Physical pavement construction characteristics	1 ha area 350 mm deep 30 per cent voids Horizontal
Outflow from the pavement *	1, 2, 5 and 10 l /s/ha (0.36 – 3.6 mm/hr)
Inflow to the pavement	100 per cent runoff with 5 mm depression storage a) Car park area only b) 1 ha of additional paved area c) 2 ha of additional paved area
Hydrology and climate change factors	20/0.4 (south) and 17/0.2 (north) 1 to 10 year (factor 1.1) 30 and 50 year (factor 1.2) 100 to 200 year (factor 1.3) 1 – 12 hour events design storms

* These outflows could be considered to be either throttles on the outlet or an infiltration capacity below the car park. These infiltration rates are well below the values which would be considered appropriate for use of soakaway devices.

A4.3.1 Results

The results of the analysis were evaluated for both the maximum "spill" volume and the peak rate of flow when it became full and overflowed. The maximum duration selected was a 12 hour event, though it should be noted that these flow rates and volumes sometimes increase for longer duration events. Figures 9.11 and 9.12 summarise the results and shows that overflow never takes place for any event even with the smallest throttle rate when the car park does not serve additional impermeable areas.

When it serves an addition of 1 ha it becomes surcharged and overflow takes place for extreme long duration events. The northern climate with its greater depths of long duration storms shows a significant reduction in performance compared with the southern climate. Where the additional area is limited to 1 ha, only the 200 year event in the south spills, while in the north, the maximum overflow rate is 61 l/s for a 200 year event. However it should be noted that the 30 year event is catered for without flooding for all scenarios in the south, but causes a small amount of exceedance flow in the north.

This trend is extended when 2 ha of additional area is drained to the pavement. The maximum overflow rate for this situation is 95 l/s which is from a 12 hour 200 year event.

Similarly, Figure 9.15 and Figure 9.16 show the maximum volumes of spill. The additional 2 ha of impermeable area for the 200 year event generates an overflow volume of over 1100 m^3 in the south rising to nearly 2000 m^3 in the north. These large volumes reflect the very low limit of discharge applied in this case.

A4.3.2 Application of results and conclusions

The four Figures of 9.11 to 9.14 provide a simple way to assess the exceedance flow from a pervious pavement which has a storage depth of around 120 mm for a range of limiting outflow discharge rates and impermeable area served. The following general advice can also be derived from these results.

- pervious pavements with this depth of storage will never cause a rainfall exceedance problem for events with a probability of occurrence less than 0.005 in any year until it is used to provide storage for an additional impermeable area that is at least equal in extent to that of the pervious pavement
- where the extent of the additional surcharge area is significantly greater than the area of the pervious pavement, the volume of runoff, which cannot be catered for within the unit, becomes large and would need specific consideration for long duration extreme events (12 to 48 hours). The limiting discharge would need to be significantly greater than the range used in this analysis to eliminate this flood volume
- the rate of "overflow" is unlikely to ever cause a significant flood flow depth in terms of the conveyance in the major system as the events that cause exceedance are not short sharp high intensity storms.

To obtain values for other arrangements of storage provision, hydrological regions, outflow rates and contributing areas, it is necessary to produce a model of the system. However a generic "look-up" method is provided in Appendix 5. It has been derived based on a number of simplifying assumptions which enables a first order estimate of the exceedance characteristics for any SUDS system.

A4.4 Swale performance

Swales are still not frequently used in UK and current design criteria are only related to erosion control, pollutant removal and maintenance needs. There are a number of different types of swale configurations (under-drained, standard conveyance, partial storage with overflow etc) which means that there are endless possibilities for the hydraulic design of these units. Hydraulic criteria relating to hydraulic performance has yet to be defined and accepted nationally. Current design guidance suggests that swales would not provide a level of service greater than a return period of 10 years and that road drainage design only requires around a one year level of service. Slopes across a site and the different lengths of swale, which are needed to fit into the road layout, makes the performance of each swale length unique. This means that simple guidance on exceedance behaviour is very difficult to derive.

The common cross-section often used to describe the requirements of a swale is shown in Figure A4.2. The performance of a swale is related to the outflow characteristics (outlet size and infiltration along its length), the inflow (the area generating runoff from rainfall) and the volume of storage in the swale before it fills and starts spilling down the road. This volume is not a function of the length of the swale, but the proportion which is below the level of the top of the swale at the downstream end. For steep swales this can be quite a short length. At 1:20 gradient, the bottom of the swale eight metres from the downstream end is the same level as the top of the swale if the depth is 400 mm.

Guidance on exceedance design is through the provision of information on performance relating for a range of combinations of slope and length of swale assuming a standard cross-section. This provides a good understanding of the level of service provided by swales and the typical range of flow rates and volumes that can occur during extreme events. General guidance advice is drawn from the results.

Assumptions are needed for deriving an understanding of level of service and exceedance behaviour for the following elements:

- contributing area
- gradient of the swale
- length of the swale
- outflow control from the swale.

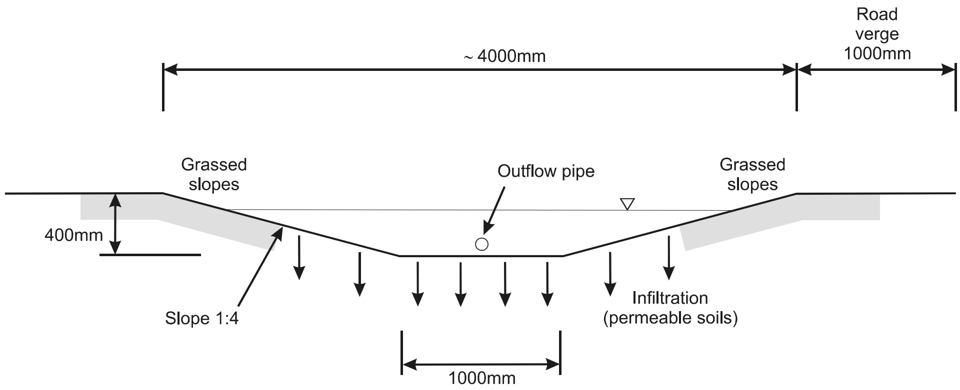


Figure A4.2 *Swale cross-section*

A4.4.1 Contributing area

The effective area contributing runoff is assumed to be a half road width (3.5 m) together with an allowance of 1.5 m to allow for the bottom of the swale and some adjacent runoff. Runoff is assumed to be 100 per cent. The length of road is assumed to be the same as the length of the swale.

A4.4.2 Gradient of the swale

Gradients analysed range from 1:20 to 1:250. Design guidance (Martin *et al*, 2000a) suggests that 1:17 should not be exceeded, though figures ranging from 1:10 to 1:28 are also suggested in other SUDS literature. This maximum gradient is set to reduce the risk of erosion, ensure a degree of treatment and reduce the collection of sediment at the bottom end of the channel. Swales flatter than 1:100 may have problems draining. However the current trend towards bio-retention means that in many cases, flatter gradients may be considered to be quite desirable.

A4.4.3 Length of the swale

Although pollution design criteria includes a suggested minimum length of 30 m with a preference for 60 m, in a residential estate the length of a swale would range between 20 m and 100 m before it has to terminate. At this point it would either pass under the obstruction/road into another swale or connect to a carrier pipe in the road. This range has been used to assess swale performance.

A4.4.4 Outflow control from the swale

In this analysis infiltration is assumed not to take place on the basis that the reduction in volume lost by infiltration from an extreme event is small relative to the total runoff volume. Swales are normally served by an outfall pipe which passes to a carrier sewer. Based on the normal water company requirements of a minimum pipe size of 150 mm diameter, an outflow rate of 15 l/s has been assumed. At the other extreme the worst case scenario is for a blocked outlet. These two outflow rates have been used together with an intermediate flow rate of 7.5 l/s to provide additional information (see Table A4.2).

As for the pervious pavements, the assessment provides peak "overflow" rates when the swales are filled and spills take place, and also the maximum volume spilled. Design events ranging from one to 200 year return period have been used.

Table A4.2 *Swale – modelling assumptions*

Swale	Characteristics and scenarios
Swale shape	1:4 side slope 400 mm deep 1 m base width
Lengths	20 m, 50 m, 100 m
Gradients	1:20, 1:50, 1:100, 1:250
Outflow from the swale	0, 7.5 and 15 l/s
Inflow to the swale	100 per cent runoff 5.0 m road width
Hydrology and climate change factors	20/0.4 (south) and 17/0.2 (north) one to 10 year (factor 1.1) 30 and 50 year (factor 1.2) 100 to 200 year (factor 1.3) one to 12 hour events design storms six extreme events from a time series

A4.4.5 Application of results and conclusions

As for pervious pavements, the maximum duration selected was a 12 hour event. Tables 9.5 and 9.6 summarise the output on maximum spill rates and total volumes respectively for the 100 m swale for southern UK rainfall. Tables A4.3 to A4.14 illustrate the results for northern and southern rainfall for shorter swales.

These graphs can be used by interpolation to determine approximate flood flow rates and flood volumes. Where swales are proposed to be built with different profiles or with longer runs, either models will need to be built to predict their exceedance performance or Appendix 5 can be used to obtain a first order estimate of their flood characteristics.

Table A4.3 *Overflow rate (l/s) from 20 m long swale system – south*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0.1	0	0	0
30	0	0	0	0	0	0	0	0	0.4	0	0	0
100	0	0	0	0	0	0	0	0	1.6	0.1	0	0
200	0	0	0	0	0	0	0	0	2.7	0.3	0	0

Table A4.4 *Overflow rate (l/s) from 20 m long swale system – north*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0
100	0	0	0	0	0	0	0	0	0	0	0	0
200	0	0	0	0	0	0	0	0	0	0	0	0

Table A4.5 *Overflow rate (l/s) from 50 m long swale system – south*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	0.2	0	0	0
10	0	0	0	0	0	0	0	0	2.0	0.4	0	0
30	0	0	0	0	0	0	0	0	6.2	1.7	0	0
100	0	0	0	0	0	0	0	0	14.8	5.9	0.9	0
200	0	0	0	0	0	0	0	0	20	9.7	1.7	0

Table A4.6 *Overflow rate (l/s) from 50 m long swale system – north*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	0.8	0.1	0	0
10	0	0	0	0	0	0	0	0	2.2	1.2	0	0
30	0	0	0	0	0	0	0	0	4.9	2.4	0.6	0
100	0	0	0	0	0	0	0	0	9.6	5.2	2.0	0
200	0	0	0	0	0	0	0	0	12.5	8.6	2.9	0

Table A4.7 *Overflow rate (l/s) from 100 m long swale system – south*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	2.7	0.8	0	0
10	0	0	0	0	0	0	0	0	14.4	5.6	1.1	0
30	0	0	0	0	6.5	0	0	0	27	14.3	3.5	0
100	15.4	0	0	0	31.7	8.7	0	0	44.5	30.8	11.8	0.6
200	29.3	0	0	0	41	20.9	0	0	51.4	39.8	18.8	1.7

Table A4.8 *Overflow rate (l/s) from 100 m long swale system – north*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	2.6	1.8	0.2	0
10	0	0	0	0	0	0	0	0	9.3	4.9	2.4	0
30	0	0	0	0	0	0	0	0	15.6	10.6	5	0.4
100	0	0	0	0	9.6	0	0	0	26.3	20.1	10.6	2.6
200	0	0	0	0	18.2	0	0	0	31	24.9	16.8	4

Table A4.9 *Overflow volume (m³) from 20 m long swale system – south*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0.4	0	0	0
30	0	0	0	0	0	0	0	0	2.2	0	0	0
100	0	0	0	0	0	0	0	0	4.8	0.7	0	0
200	0	0	0	0	0	0	0	0	6.3	2.1	0	0

Table A4.10 *Overflow volume (m³) from 20 m long swale system – north*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	1.9	0	0	0
30	0	0	0	0	0	0	0	0	4.1	0	0	0
100	0	0	0	0	0	0	0	0	7.2	3.1	0	0
200	0	0	0	0	0	0	0	0	8.9	4.7	0	0

Table A4.11 *Overflow volume (m³) from 50 m long swale system – south*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0.4	0	0	0
30	0	0	0	0	0	0	0	0	2.2	0	0	0
100	0	0	0	0	0	0	0	0	4.8	0.7	0	0
200	0	0	0	0	0	0	0	0	6.3	2.1	0	0

Table A4.12 *Overflow volume (m³) from 50m long swale system – north*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	4.2	0.6	0	0
10	0	0	0	0	0	0	0	0	10.4	6.7	0	0
30	0	0	0	0	0	0	0	0	15.9	12.3	4.3	0
100	0	0	0	0	0	0	0	0	23.8	20.2	12.1	0
200	0	0	0	0	0	0	0	0	28	24.3	16.3	0

Table A4.13 *Overflow volume (m³) from 100m long swale system – south*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	8.7	4.9	0	0
10	0	0	0	0	0	0	0	0	18.8	15	6.9	0
30	0	0	0	0	0.7	0	0	0	27.9	24.1	16.1	0
100	1.3	0	0	0	5.4	1.5	0	0	41.1	37.3	29.2	5.8
200	3.4	0	0	0	8.1	4.4	0	0	48.4	44.5	36.5	13.8

Table A4.14 *Overflow volume (m³) from 100m long swale system – north*

Swale outflow rate (l/s)	15				7.5				0			
Gradient (m/m) / Return period	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.00 4	0.05	0.02	0.01	0.004
1	0	0	0	0	0	0	0	0	14	10.2	2.2	0
10	0	0	0	0	0	0	0	0	26.3	22.5	14.4	0
30	0	0	0	0	0	0	0	0	37.4	33.6	25.6	2.1
100	0	0	0	0	1.8	0	0	0	53.1	49.3	41.3	17.9
200	0	0	0	0	4.5	0	0	0	61.5	57.7	49.6	26.2

A4.5 Infiltration system performance

Infiltration trenches and soakaways can be designed accommodate rainfall up to a 10 year return period. A design methodology is outlined in CIRIA Report 156. They are often filled with granular material, with the system designed to infiltrate surface water into the ground.

The critical issue for infiltration systems (as with all outlet rate assumptions for these SUDS units) is the rate of infiltration into the subsurface material. Clay-like soils allow very little infiltration whereas a sandy soil allows high levels of infiltration. Infiltration rates normally range from 5×10^{-4} mm/s for a soil with marginal infiltration capacity to 1×10^{-2} mm/s for a soil with a relatively good infiltration capacity.

As for the pervious pavement and swales, the analysis provides an assessment of peak flood flows when the infiltration system has filled and the maximum volume generated. The test characteristics are shown in Table A4.15.

Table A4.15 *Infiltration system – test characteristics*

Infiltration system	Characteristics and scenarios
Design criteria	10 year return period 30 per cent void ratio
Outflow	No pass forward flow
Inflow to the system	50 m^2 roof area × 50 houses per hectare (Total catchment = 2500 m^2 per hectare)
Infiltration rates	5×10^{-4} mm/s – marginal 1×10^{-2} mm/s – good
Hydrology and climate change factors	20/0.4 (south) and 17/0.2 (north) one to 10 year (factor 1.1) 30 and 50 year (factor 1.2) 100 to 200 year (factor 1.3)

A4.5.1 Application of results and conclusions

Table A4.16 and A4.17 summarise the maximum flood flow rates and maximum flood volumes respectively for the infiltration trench for southern UK rainfall. Figures for the northern UK rainfall are not provided as the design of an infiltration trench would largely take into account the difference in performance for the various return periods.

These graphs can be used by interpolation or limited extrapolation for any infiltration rate and return period rainfall to determine approximate flood flow rates and flood volumes. Where infiltration units are used with very different infiltration rates or designed to a criterion other than a 10 year return period, either models will need to be built to predict their exceedance performance or Appendix 5 can be used to obtain a first order estimate of their flood characteristics.

Table A4.16 *Overflow rate (l/s) from infiltration system*

Location	North			South		
Return period \ Infiltration rate (mm/hr)	1.8	3.6	36	1.8	3.6	36
10	1.3	2.4	0	8.1	9.6	1.1
30	10.8	14.5	9.6	17.1	16.1	16.5
100	20	18.8	17.6	19.5	21.5	18.2
200	22.3	24.7	22.5	25.2	26.5	22.5

Table A4.17 *Overflow volume (m^3) from infiltration system*

Location	North			South		
Return period \ Infiltration rate (mm/hr)	1.8	3.6	36	1.8	3.6	36
10	9.9	8.4	0	73.8	66.5	0.5
30	61.4	53.6	12.7	144.4	131.6	31.7
100	136.5	123.3	57	240.4	225.1	86
200	176.7	161.3	84.1	287.2	271.2	117.3

A5 Generic guidance on assessing flood volumes and rates from SUDS

The results of the analysis of the various SUDS units in Appendix 4 provide an outline for the performance of SUDS systems. There is difficulty in comparing the units because the performance is a function of the assumptions made regarding the design of the unit. The results provide a guide as to their performance and they can be used to determine the maximum flood flows if the assumptions used apply to the case being considered.

A generic method has been developed to enable the performance of a SUDS unit to be predicted. This method allows the peak flood flow rate and total volume of flooding to be estimated. Unfortunately the various aspects of head-discharge characteristics, variable infiltration rates and attenuation of runoff makes the derivation of a generic approach only approximate. It should be stressed that the range of assumptions that has to be made in using this method, makes this approach only suitable as a first order assessment of the situation. The estimate of flood volume will probably be more accurate than the prediction of the peak flood flow rate.

In principle all drainage systems have a storage component together with an outflow rate. Pipe systems have a relatively small storage component. However, in the case of SUDS there is more emphasis on the storage element and specific allowance for this should be made.

A5.1 Assumptions

A range of assumptions have been made in developing this assistance tool and these are placed into three categories:

- inflow
- outflow
- storage.

These assumptions generally result in a conservative estimate of the storage required or flood volume which will take place for any event. The results are too conservative for units such as pervious pavements where the lag time needs to be taken into account. Two versions have been produced; lagged and un-lagged. Units which have flow passing through granular media, such as infiltration trenches and pervious pavements, are based on the lagged method.

A5.1.1 Inflow

The assumptions used are that:

- runoff volume is generated assuming 100 per cent of the rainfall is effective
- rainfall intensity occurs at a constant intensity for each duration event
- runoff into the drainage unit occurs immediately.

The inflow of rainfall as runoff into a drainage unit is the combined process of wetting, runoff along the surface and finally entering into the unit. Times of entry of three to eight minutes are commonly assumed when using the Rational Method of design.

This process provides storage and attenuation and an assumption of an immediate arrival of runoff without attenuation may be cautious and over predicting the impact of the rainfall and the requirements needed to address it.

A5.1.2 Outflow

The assumption being made in this method is that outflow occurs immediately at a defined and fixed discharge rate.

Outflow from a drainage unit is a function of the head-discharge relationship of the structure at the outlet(s). Wetting of the storage medium, infiltration and even evaporation might also provide an outflow in some units. As infiltration may be the main outflow route, the method provides the outflow rate in units of l /s and mm/hr/m^2. In addition to the head-discharge aspect affecting outflow, flow through the medium in the unit may also affect the discharge. In the case of a pervious pavement, granular fill provides significant attenuation of the flow within the structure.

The assumption used is not conservative in assessing storage requirements and subsequent overflow impact.

A5.1.3 Storage

The difference between the inflow and outflow into a structure represents the storage and there is no significant approximation made for this element. A difficulty for the application of the method in practice is the difficulty of estimating what the storage provided actually is for certain structures, so that the flooding values can be estimated. For example, not only is water being stored as it passes down a swale, the ability to estimate the volume of storage below the level at which overflow commences is a function of the shape and the gradient of the channel.

A5.2 The principles of the flood estimation method

Figure A5.1 illustrates the basis of the estimation of flooding. Curves can be produced for any return period for any climate region. Assuming a proportion of runoff and a contributing area, these become the runoff volume and inflow into the drainage system.

A second family of curves can be produced for the outflows from any structure if the flow rate is known. If a constant outflow is assumed, these curves are straight lines.

The difference between any outflow curve and the inflow curve represents the storage/ flood volume for that structure. The location of the maximum difference between these two curves gives the critical duration of the design event and the storage required for any return period.

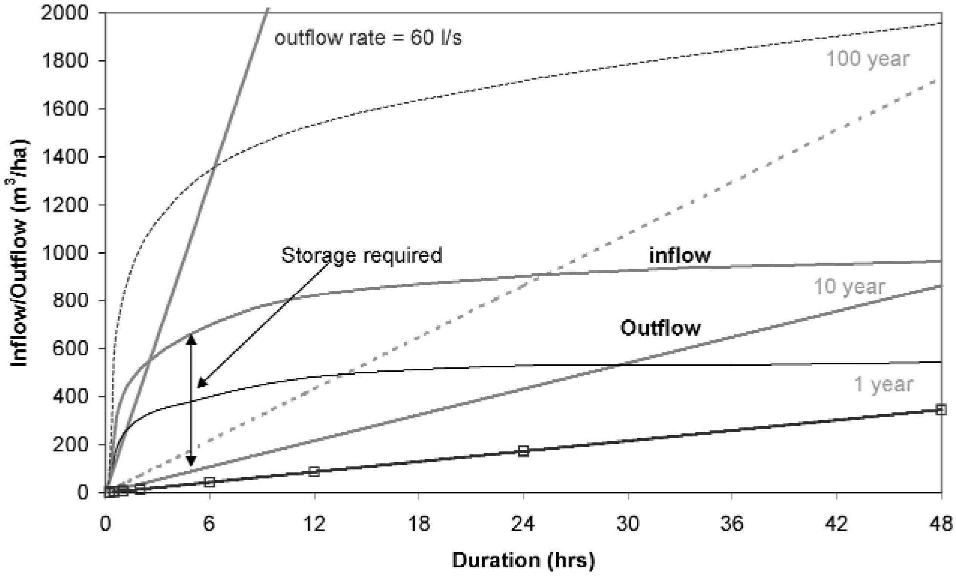


Figure A5.1 *Estimation of storage required*

Figure A5.2 builds on these assumptions by showing an additional line which is above and parallel to the outflow curve. The difference between these two lines is in the storage provided in the system and from this various aspects can be derived. The first is that if the line is always above an inflow line, then no flooding should occur at the location of that drainage structure for that return period. In Figure A5.2 this means that there should not be any flooding for a one year event, but some flooding will take place for a 10 year event which has a duration from around one hour through to 27 hours. This can either be taken to be the predicted flood volume or the storage needed to prevent flooding.

In addition to the estimate of flood volume, it can be assumed that the intersection of the inflow curve and the storage curve represents the point at which flood flows will commence. As the gradient of the inflow line is related to rainfall intensity, this can be used to predict the peak rate of flood flow from the unit using intensity/duration/frequency rainfall information. As the inflow curve is very steep and no allowance has been made for initial wetting and routing of runoff, values of rainfall intensity for periods less than 15 minutes can be produced. This can result in unrealistic flood flows being derived and it has been decided to cap the minimum duration to 15 minutes.

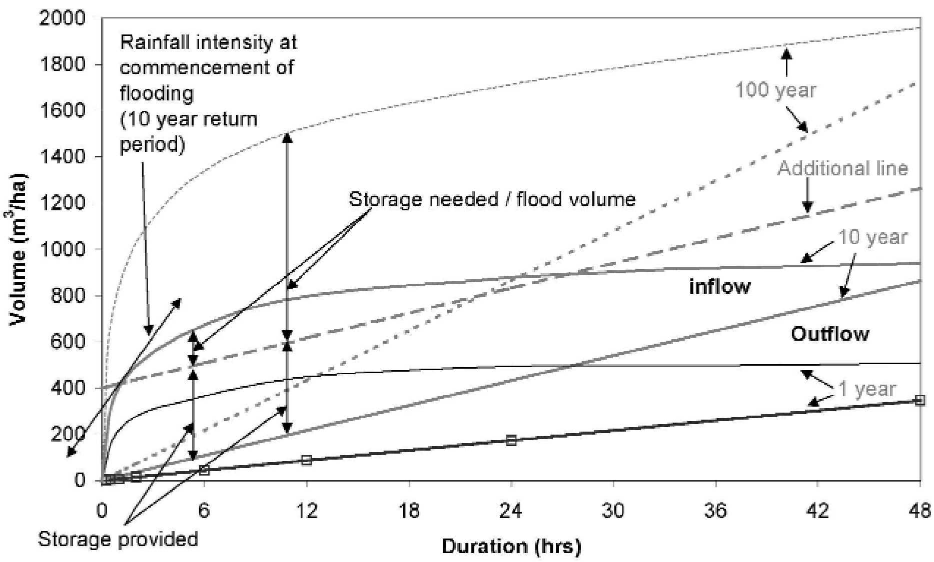


Figure A5.2 *Estimation of flood volumes*

In addition, application of the method has shown that the lag that takes place in units such as pervious pavements must be taken into account otherwise predictions become unacceptably high, particularly flood flow rates due to the gradient of the inflow line in the first two or three hours.

A5.3 Method of application

The flowchart in Figure A5.3 shows how the method is applied.

Figures A5.1 and A5.2 are provided in the appendix, but these are not needed in the method as the difference between the inflow and outflow curves has been evaluated and is shown in Figures A5.4 and A5.5. These figures assume a range of outflow rates ranging from 1 l/s/ha through to 60 l/s/ha. It can be seen from Figure A5.1 that 60 l/s/ha is a steep line and produces very small flood volumes (subject to the area being drained) even for the 200 year event. It should be noted that although any flood volume is small, the maximum flood flow rate can still be very high if inadequate storage provision is made. Due to the nature of the two curves (inflow and outflow) it is likely that the 15 minute rainfall intensity would be used to determine the peak flood flow impact.

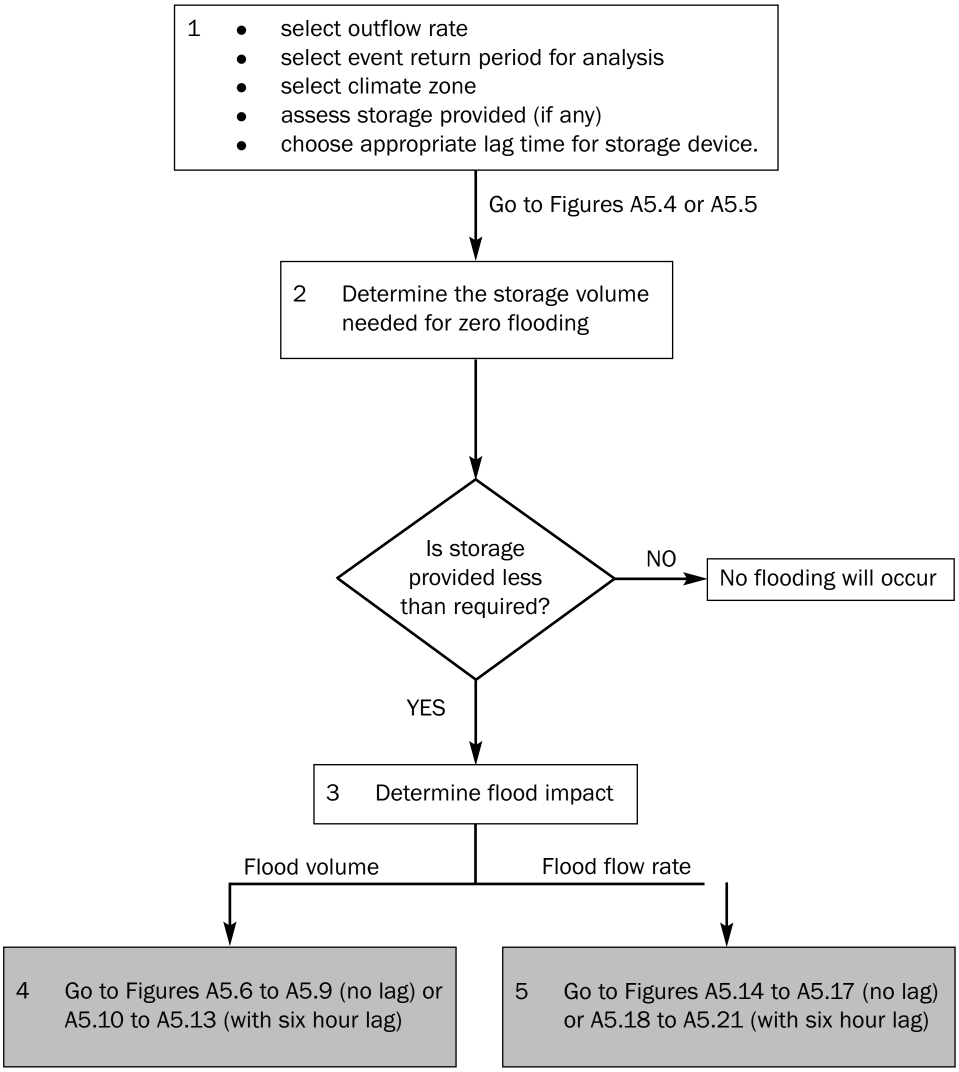


Figure A5.3 *Flowchart for estimating flood volumes and flood flow rates*

The following clarification on its application is provided.

1. The outflow rate can be specified as a flow rate (l/s/ha) or an infiltration rate (mm/hr/ha). The lag time relates to the process of runoff into and flooding out of a unit. Systems such as swales have a limited response time, whereas research has demonstrated that lag times in units such as pervious pavements are measured in hours. It should be understood that the performance of pervious pavements to extreme floods have not been recorded, though one event in Scotland was measured which still showed a significant delay. It is believed that the use of the six hour lag series of graphs is used for pervious pavements and similar units, but recognising that there is limited knowledge currently available as to the behaviour of these devices under very extreme events. Charts are provided for the no lag situation and the six hour lag.
2. The flood storage is determined using Figure A5.4 or A5.5, depending on whether a lag time has been specified.
3. Flood impact can be assessed as both the flood volume or the peak overflow rate.
4. Figures A5.6 to A5.9 provide the flood volume associated with different levels of storage for the 10 year, 30 year, 100 year and 200 year return period rainfall

events respectively. Figures A5.10 to A5.13 provide similar information for the situation where the storage device has a six hour time lag associated with it.

5. Figures A5.14 to A5.17 provide the flood flow rate associated with different levels of storage for the 10 year, 30 year, 100 year and 200 year return period rainfall events respectively. Figures A5.18 to A5.21 offer similar information for the situation where the storage device has a six hour time lag associated with it.

A5.4 Check against modelling results

To illustrate the application of the method and also to indicate the order of accuracy provided, a comparison is made with the analysis carried out for the pervious pavement, the infiltration trench and swale. The same assumptions used in the model are applied to this method where appropriate.

A5.4.1 Pervious pavement

The predicted flood volume from the pervious pavement for the 200 year event with an outflow of 10 l/s when it is serving 3 ha in the south of the country is 675 m^3 as illustrated in Figure 9.13.

The pervious pavement provides a storage volume of 1050 m^3. To prevent flooding from occurring a total storage volume of 1050 m^3 + 675 m^3 = 1725 m^3 is required. This is equivalent to a storage volume of 575 m^3/ha for zero flooding.

Following through the procedure:

Step 1 Using Figure A5.5 (as pervious pavements can be expected to have a significant time lag), for a flow rate of 3.3 l/s/ha the required storage volume for zero flooding is approximately 900 m^3/ha (or 2700 m^3 for 3 ha).

Step 2 As the pavement provides a storage volume of 1050 m^2, the flood volume is predicted to be 2700 m^3 – 1050 m^3 = 1650 m^3.

Step 3 The storage volume provided is 39 per cent (=1050/2700*100) of the required storage volume.

Step 4 Figure A5.21 is used to calculate the peak overflow rate for the 200 year return period. For a storage provision of 39 per cent the peak overflow rate is 41 l/s/ha (123 l/s for 3 ha).

The maximum flood volume predicted of 1650 m^3 and peak flood flow rate of 123 l/s are much larger than the value of 34 l/s obtained from the model. This indicates that the method is approximate and treated with caution, and that outline solutions should be checked using more accurate methods. Much of the difference is that the modelling tool assumes a significant attenuation process for the runoff entering and passing through the granular medium. Current methods of modelling are based on a limited data set of rainfall events to calibrate the model. Unfortunately, the size of events which have been recorded and available for calibration are relatively small. The flooding process during an extreme event may short circuit much of this attenuation and the values shown in Figure 9.13, although based on best available techniques, may not be conservative. This means that uncertainty over the accuracy of the prediction to the extent of flooding may be as much to do with the assumptions in the model as the precautionary assumptions of the simple method.

A5.4.2 Infiltration trench

Assuming a group of 50 infiltration trenches serving 1 ha of housing designed for a 10 year event in the south of the country which has an infiltration rate of 10 mm/hr, the predicted cumulative flood volume for a 200 year event is 150 m^3 (Figure 9.18) with a maximum flood flow rate of around 22 l/s (see Figure 9.17). It should be noted that this procedure can be applied to an area of SUDS units assuming that the appropriate inflow, outflow and storage is cumulated. The analysis will result in a prediction for an area, but individual units, which vary significantly in their hydraulic characteristics, may also need to be addressed individually.

Following through the procedure:

Step 1	Using Figure A5.5 (as infiltration trenches can be expected to have a time lag), for a flow rate of 10 mm/hr the required storage volume for zero flooding is approximately 200 m^3/ha.
Step 2	As the infiltration trenches provide a storage volume of 106 m^3, the flood volume is 200 m^3 – 106 m^3 = 94 m^3.
Step 3	The storage volume provided is 53 per cent (=106/200*100) of the required storage volume.
Step 4	Figure A5.21 is used to calculate the peak overflow rate. For a storage provision of 53 per cent the peak overflow rate is 41 l/s/ha.

The maximum flood volume predicted of 94 m^3 and peak flood flow rate of 41 l/s/ha are reasonably close to the values obtained from the model.

A5.4.3 Swale

Assuming a swale of 100 m serving an equivalent road width of 5 m at a gradient of 1:20 and an outflow rate of zero l/s results in the model predicting a flood volume for a 200 year event of 50 m^3 and a maximum flood flow rate of around 50 l/s (Figures 9.15 and 9.16). The storage in the swale is approximately 3.5 m^3 before flooding can occur.

As the procedure does not make provision for a zero outflow rate, a value of 1 l/s/ha is used. This approximation should not be significant.

Following through the procedure:

Step 1	Using Figure A5.4 (no lag time), for a flow rate of 1 l/s/ha the required storage volume for zero flooding is approximately 1141 m^3/ha.
Step 2	As the swale provides a storage volume of 70 m^3/ha (3.5 / 0.05 ha) the flood volume is 1141 m^3 – 70 m^3 = 1071 m^3/ha (54 m^3 per swale).
Step 3	The storage volume provided is six per cent (=70/1141*100) of the required storage volume.
Step 4	Figure A5.17 is used to calculate the peak overflow rate. For a storage provision of six per cent the peak overflow rate is 1020 l/s/ha (51 l/s per swale).

The maximum flood volume predicted of 54 m^3 per swale and peak flood flow rate of 51 l/s per swale are very similar to those predicted by the model.

The model was run for a maximum duration of 24 hours for the Figures in 9.15 and 9.16. However a zero outflow would have a maximum flood volume which will always be greatest for the longest storm duration. A value of 1 l/s/ha results in a critical duration event of the order of 50 hours. The predicted volume for the model may be slightly smaller than the volume from the simple method. The difference in rainfall depth between a 24 hour event and 50 hour event in Figure A5.1 is not great and this difference is unlikely to be significant in comparing the two results.

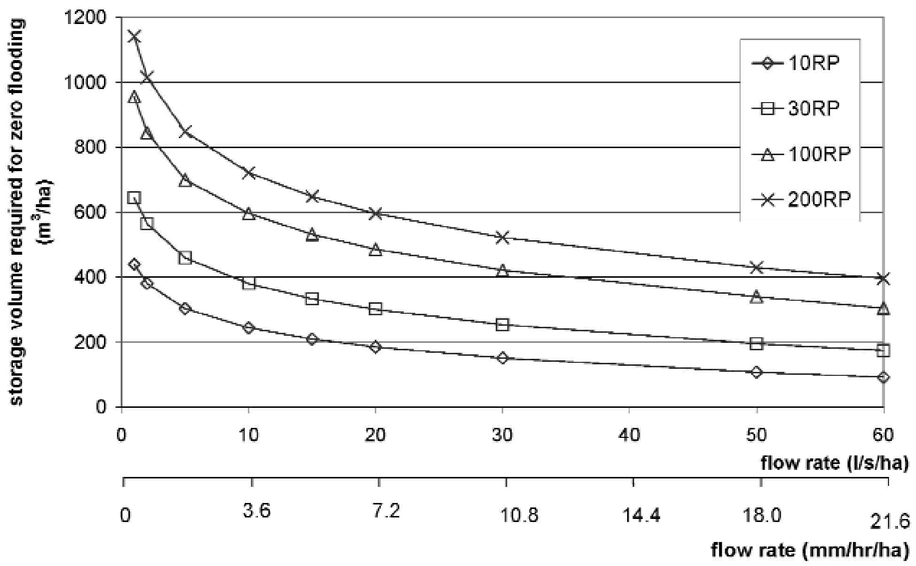


Figure A5.4 *Estimation of storage/flood volume for different return period events (no lag)*

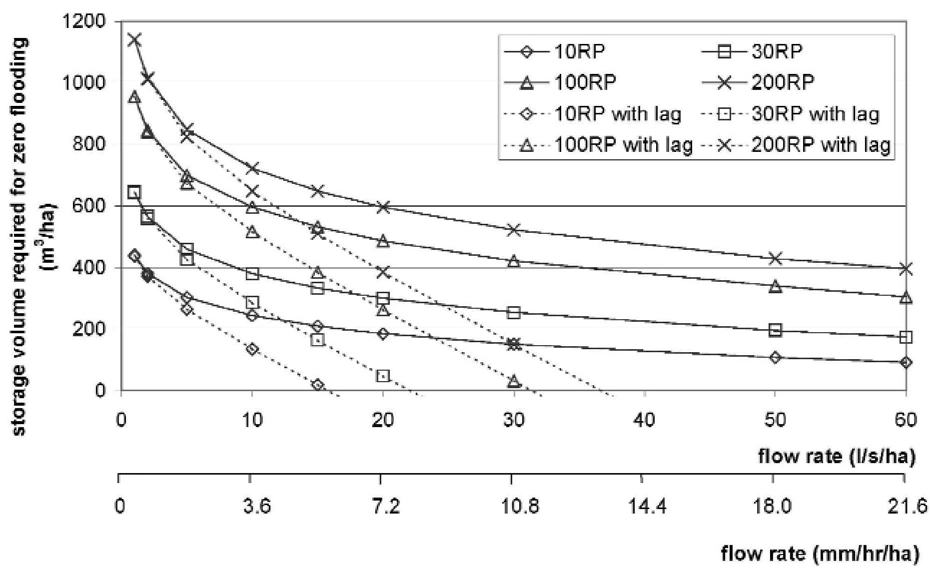


Figure A5.5 *Estimation of storage/flood volume for different return period events (six hour lag)*

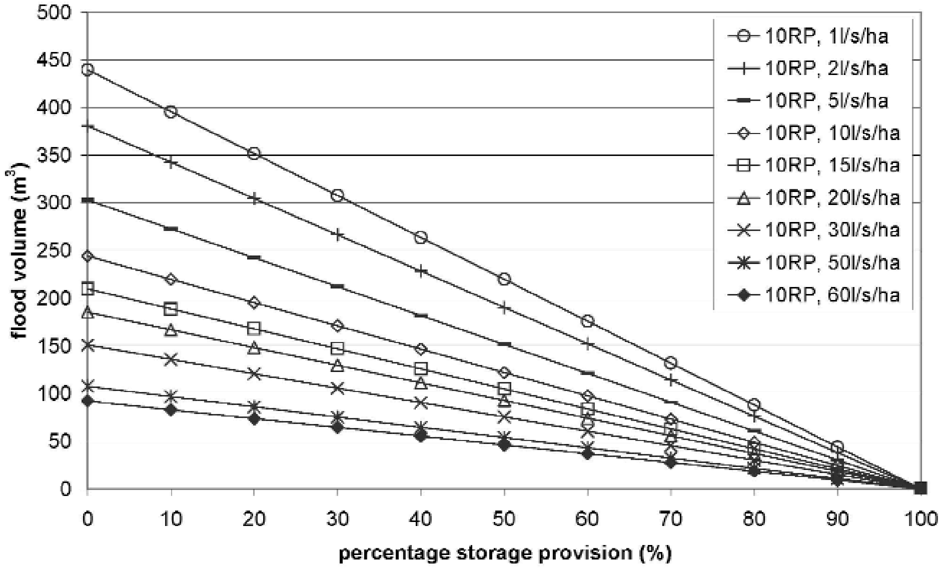


Figure A5.6 *Flood volume for different levels of storage provision for 10 year return period events (no lag)*

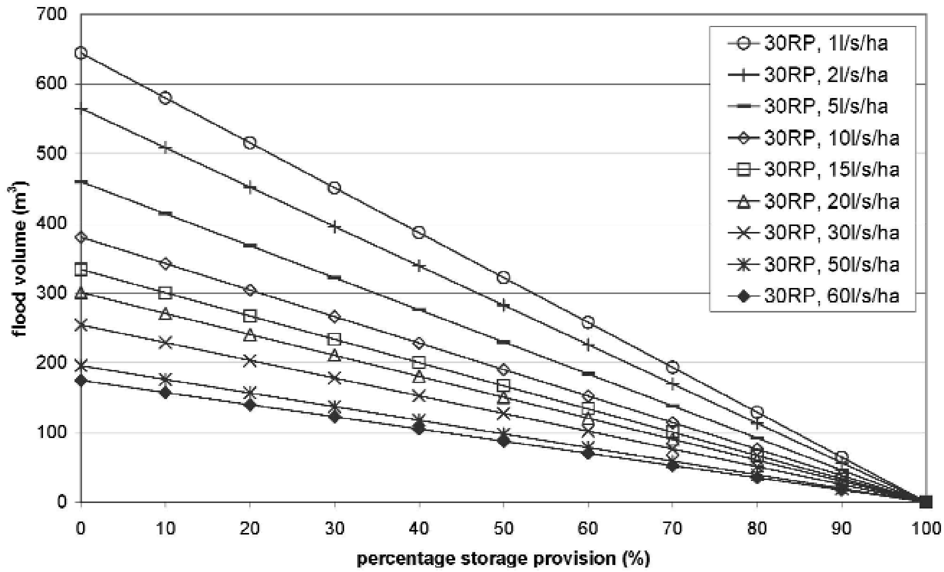


Figure A5.7 *Flood volume for different levels of storage provision for 30 year return period events (no lag)*

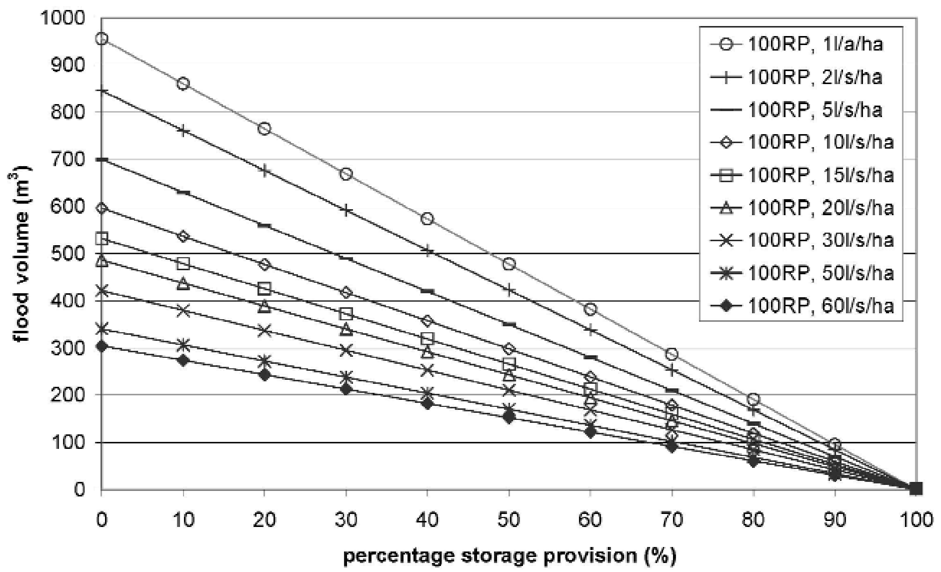


Figure A5.8 *Flood volume for different levels of storage provision for 100 year return period events (no lag)*

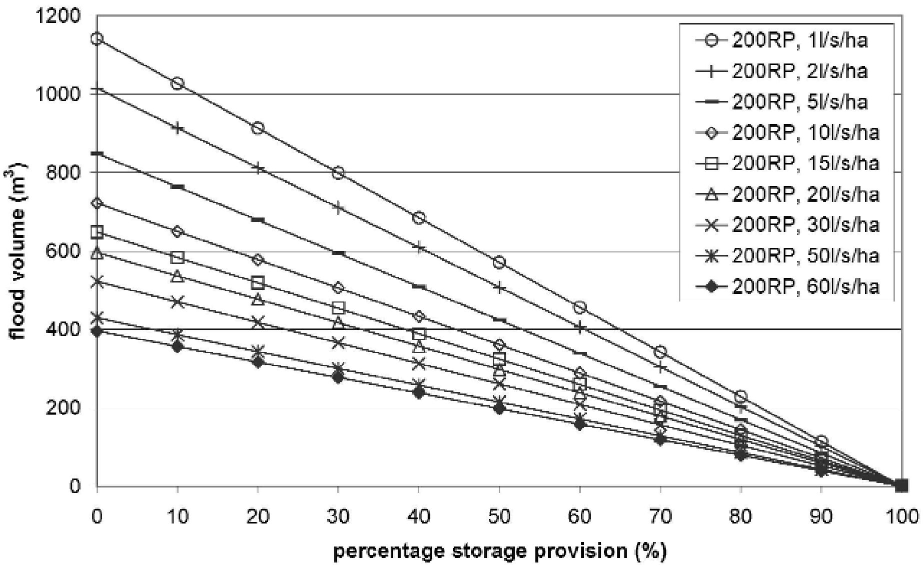


Figure A5.9 *Flood volume for different levels of storage provision for 200 year return period events (no lag)*

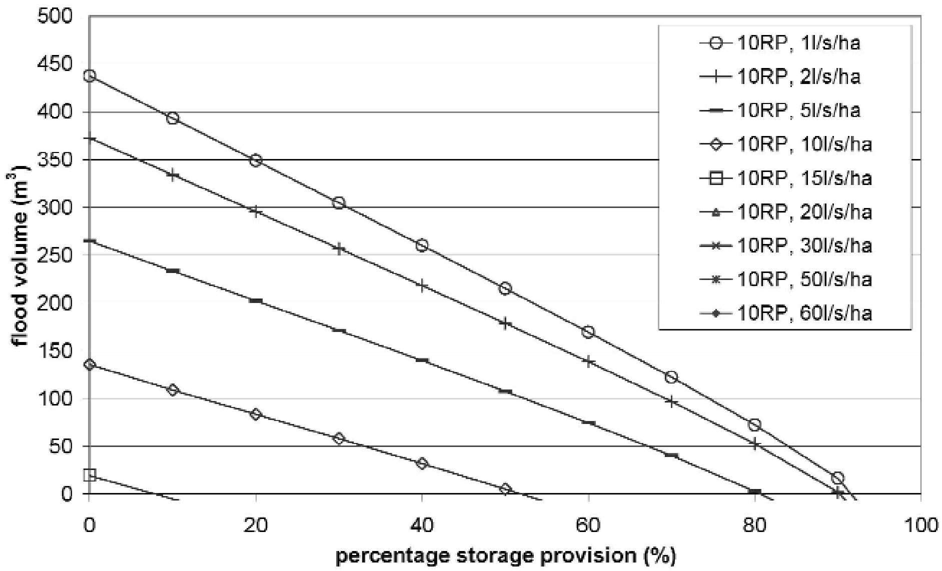


Figure A5.10 *Flood volume for different levels of storage provision for 10 year return period events (six hour lag)*

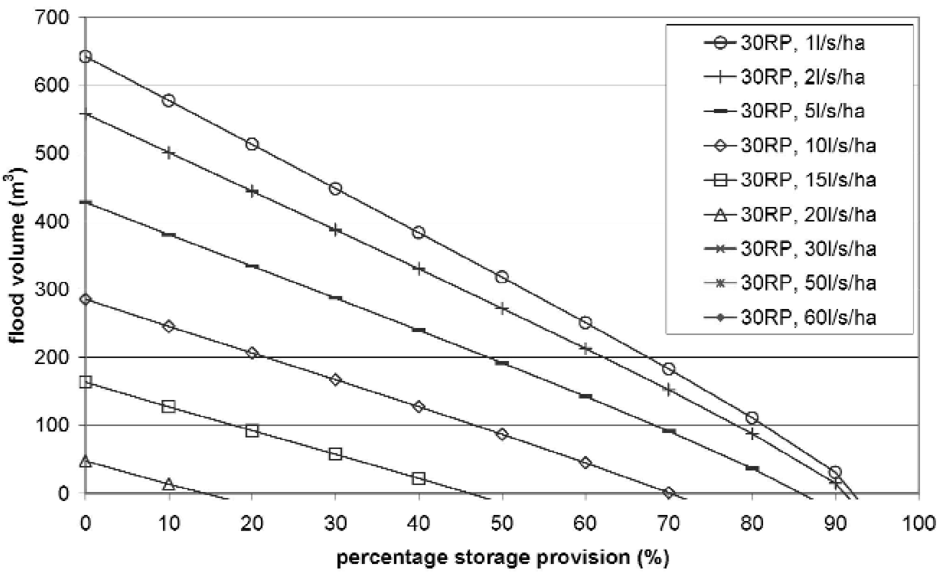


Figure A5.11 *Flood volume for different levels of storage provision for 30 year return period events (six hour lag)*

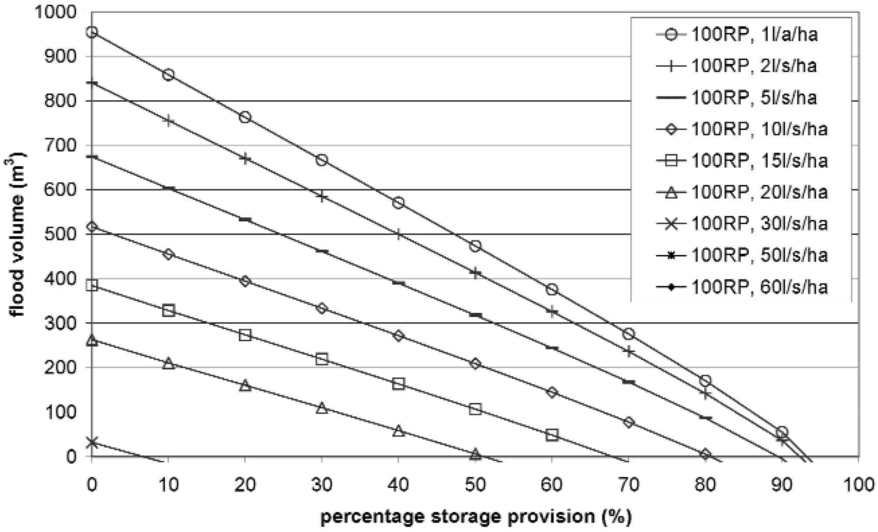


Figure A5.12 *Flood volume for different levels of storage provision for 100 year return period events (six hour lag)*

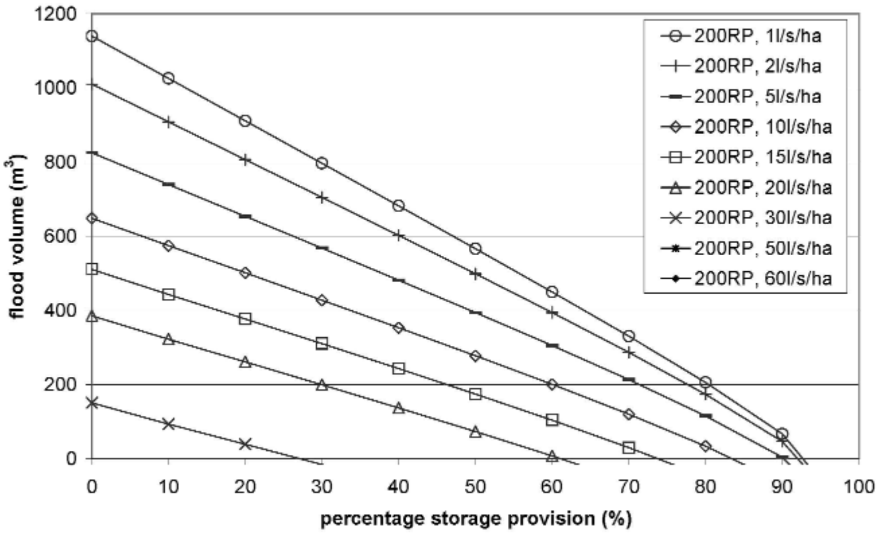


Figure A5.13 *Flood volume for different levels of storage provision for 200 year return period events (six hour lag)*

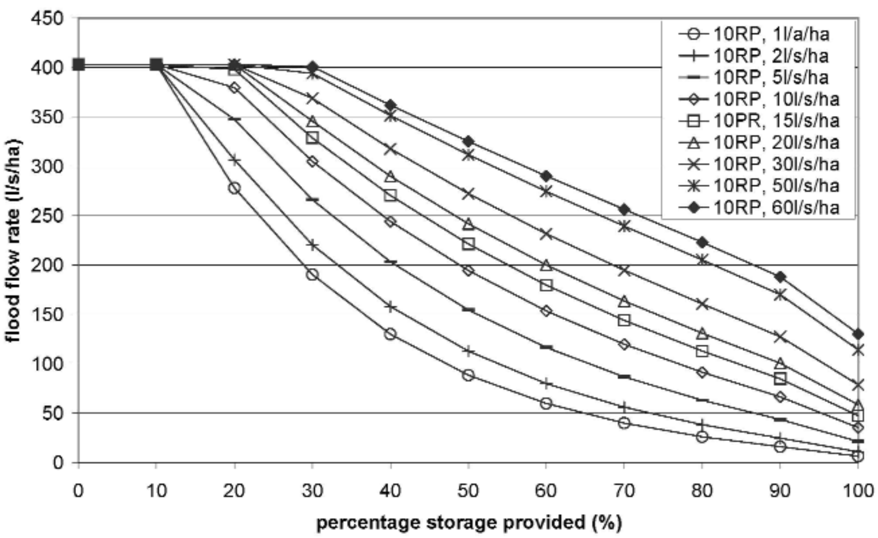


Figure A5.14 *Flood flow rate for different levels of storage provision for 10 year return period events (no lag)*

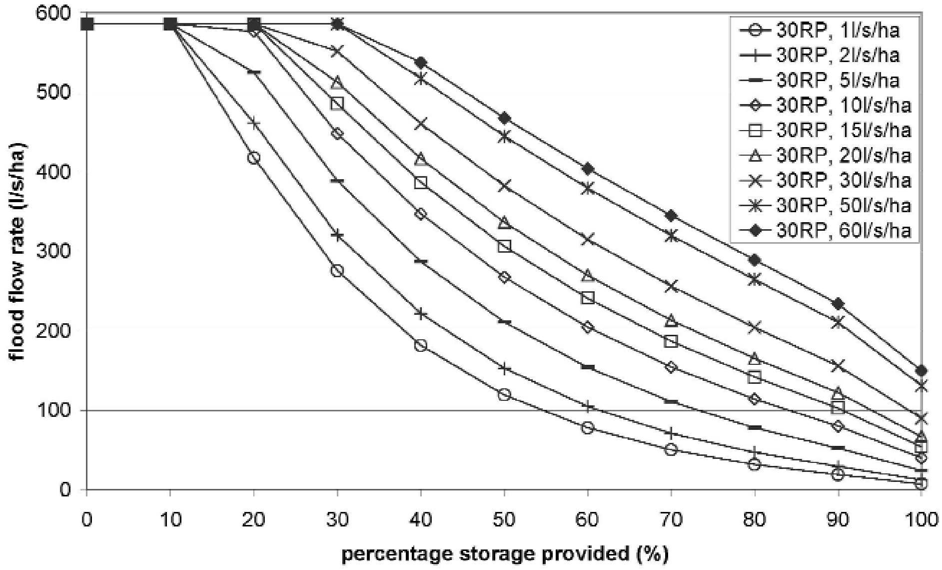


Figure A5.15 *Flood flow rate for different levels of storage provision for 30 year return period events (no lag)*

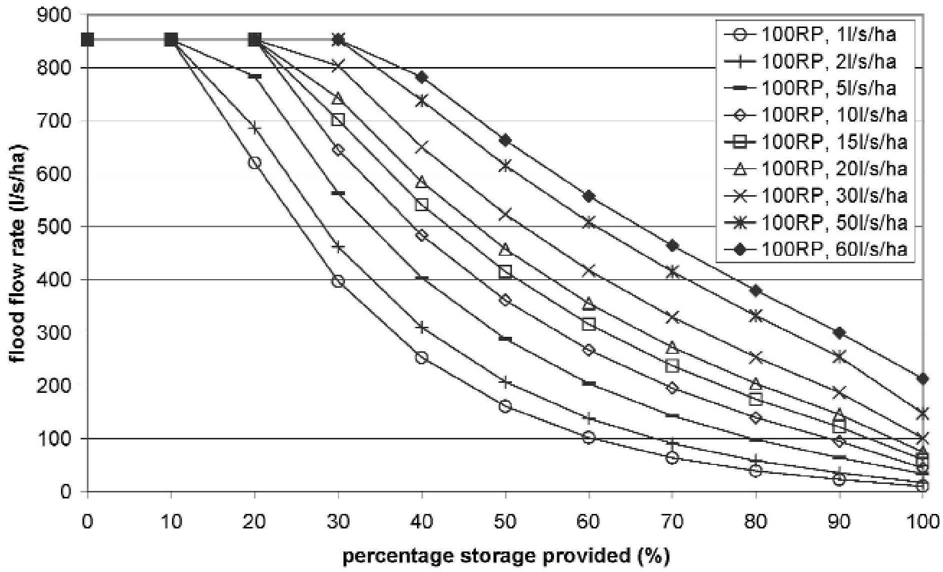


Figure A5.16 *Flood flow rate for different levels of storage provision for 100 year return period events (no lag)*

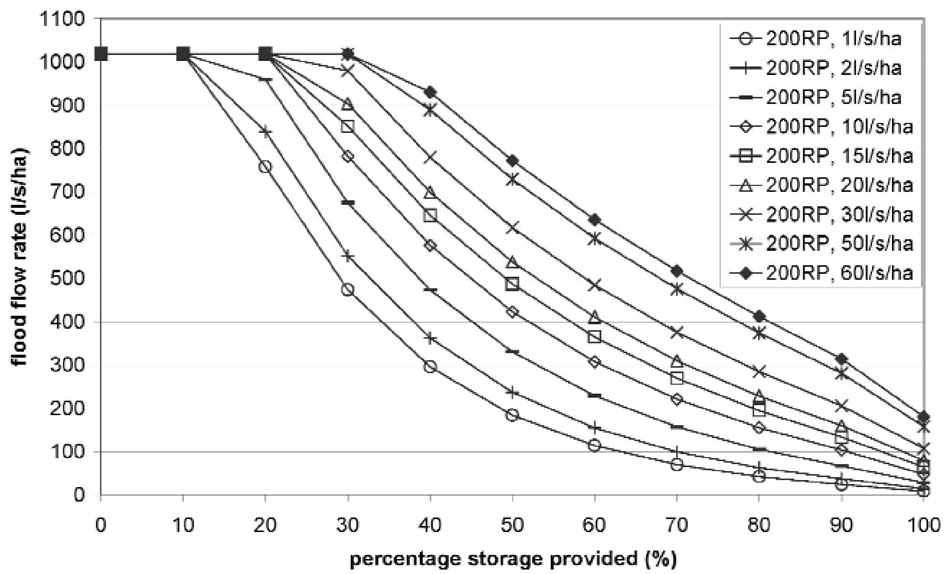


Figure A5.17 *Flood flow rate for different levels of storage provision for 200 year return period events (no lag)*

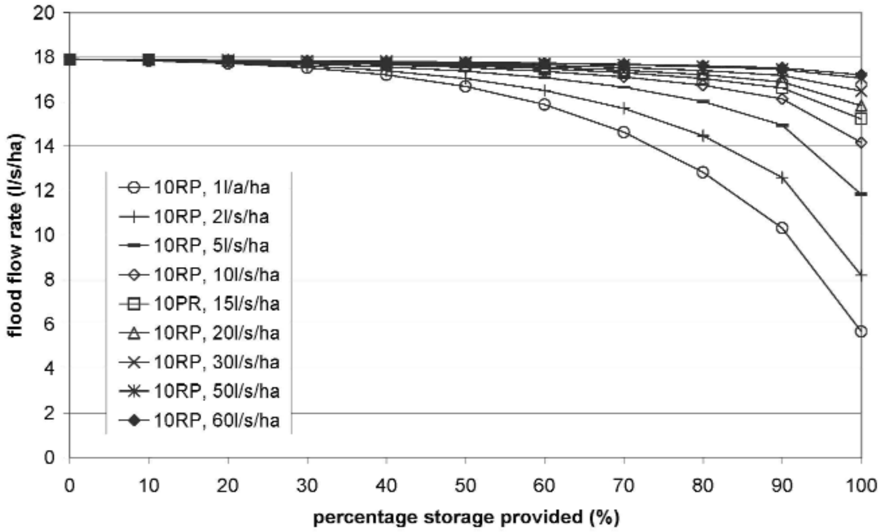


Figure A5.18 *Flood flow rate for different levels of storage provision for 10 year return period events (six hour lag)*

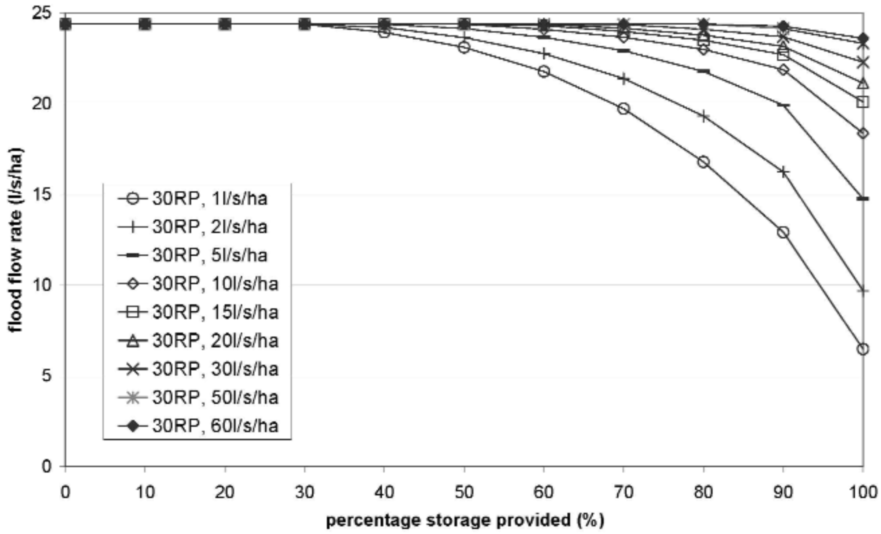


Figure A5.19 *Flood flow rate for different levels of storage provision for 30 year return period events (six hour lag)*

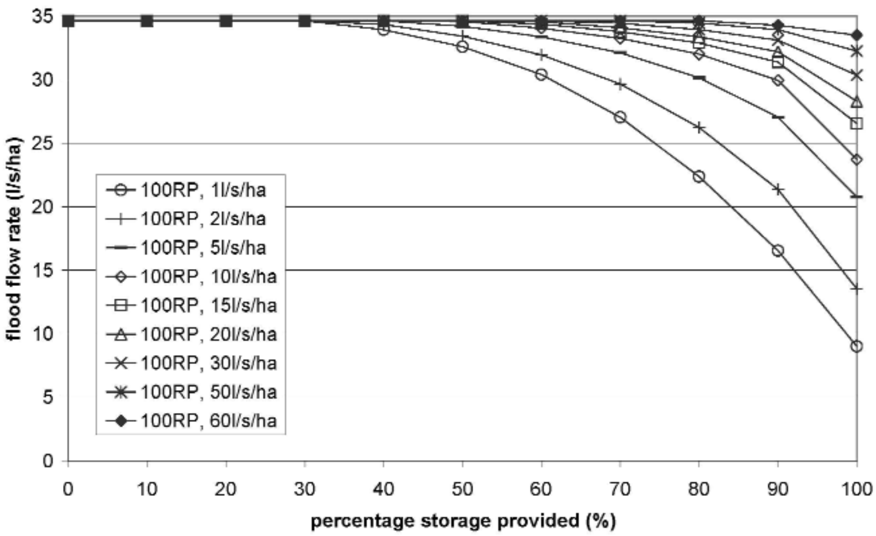


Figure A5.20 *Flood flow rate for different levels of storage provision for 100 year return period events (six hour lag)*

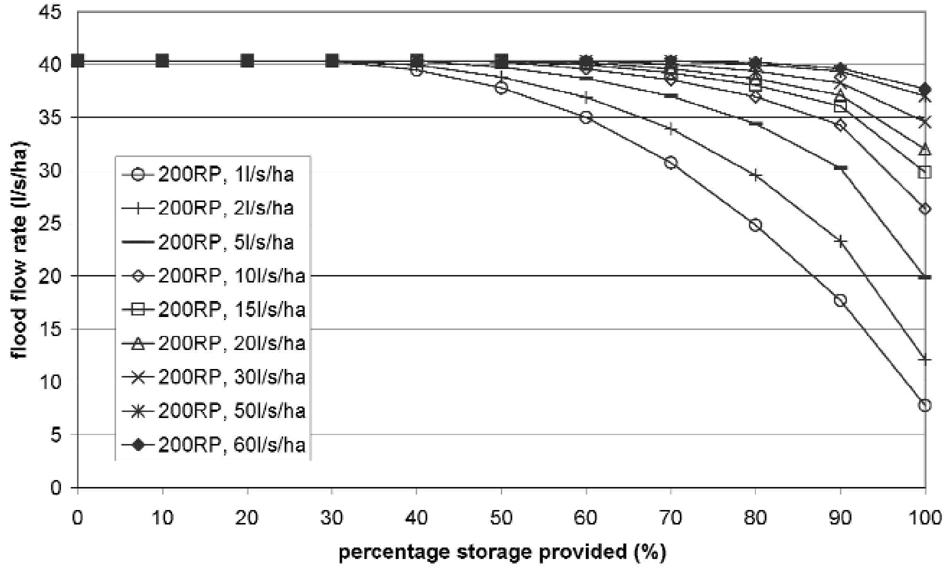


Figure A5.21 *Flood flow rate for different levels of storage provision for 200 year return period events (six hour lag)*

A6 Design example of a permeable pavement

Design example for a car park in the north with a one in 200 year return period design storm. Determine the peak exceedance overflow rate and volume.

Table A6.1 **Example of design data**

Impermeable area =	1.7 ha
Pervious pavement area =	0.85 ha
Infiltration rate =	1.2 mm/hr

Convert infiltration rate to a flow rate:

$$Flow\ rate = \frac{1.2 \times 100 \times 100 \times 1000}{1000 \times 3600} = 3.3 \ \text{l/s/ha}$$

Permeable pavement also drains the impermeable surface area. For every 1 ha of permeable surface there is 2 ha of impermeable surface.

$$impermeable\ to\ permeable\ ratio = \frac{1.7}{0.85} = 2.0$$

Use Figure 9.12 to determine peak exceedance flow rate from the permeable pavement.

Peak flow rate is 77 l/s/ha for a 200 year event with surface area ratio of two.

Exceedance volume for the pervious pavement system can be calculated using Figure 9.14. An infiltration rate of 3.3 l/s/per hectare of pavement and an area ratio of 2.0, produces an exceedance volume of 1750 m^3/ha.

Calculate exceedance overflow rate and volume for the car park which is 0.85 ha in size.

Exceedance flow rate = 77 × 0.85 = 65.45 l/s

Exceedance volume = 1750 × 0.85 = 1488 m^3

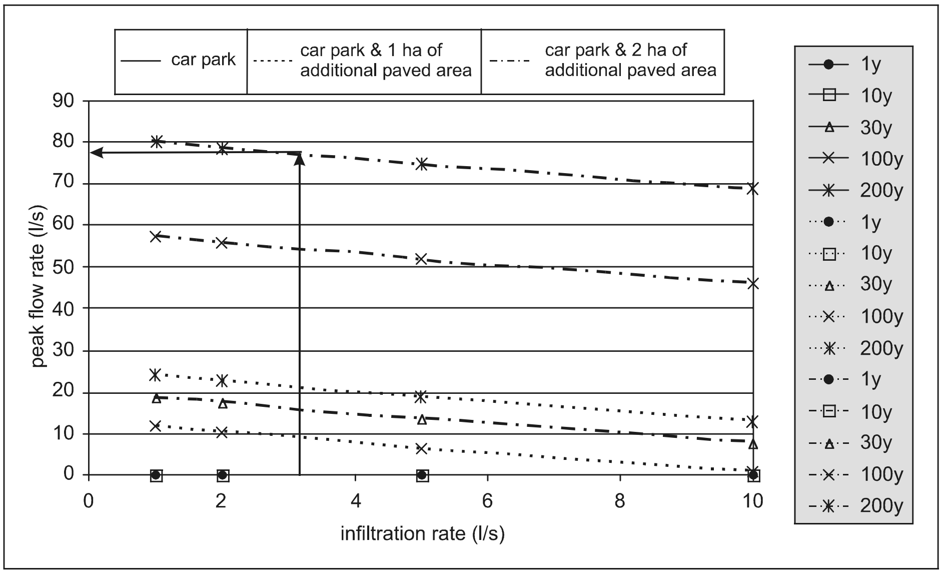


Figure A6.1 *Flood flow rate from pervious pavement system – north (developed from Table 9.2)*

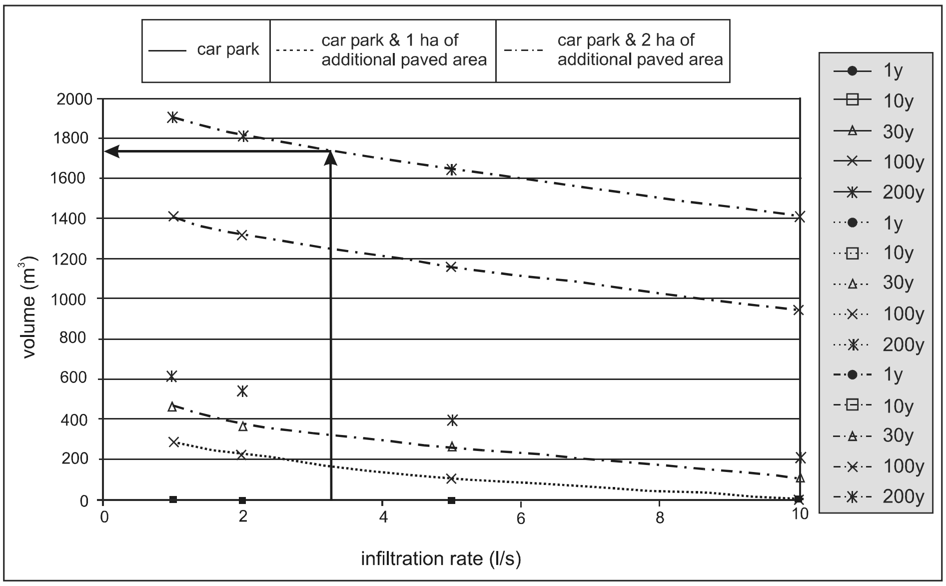


Figure A6.2 *Flood volume from pervious pavement system – north (developed from Table 9.4)*

A7 Methods of estimating rural runoff

A7.1 Rational Method

The Rational Method has its origins in the mid-nineteenth century with Mulvaney in Ireland and since its application to the design of sewers in England in the beginning of the twentieth century, this method has also become known as the Lloyds-Davies Equation. Due to its simplicity, this method is still widely used in the design of small storm sewer systems but can also be applied to the modelling of pervious catchments. The method predicts the design peak runoff rate using the following formula in metric units:

$$Q = 0.278\, C\ i\, A \tag{A7.1}$$

where:

Q	=	design peak runoff (in m^3/s)
C	=	runoff coefficient
i	=	rainfall intensity for the design return period (in mm/hr) for a specific duration related to the "time of concentration" of the catchment
A	=	catchment area (in km^2)

The estimate of the time of concentration can be made by a number of methods (Kirpich, Bransby-Williams) which use catchment slope, area and length as parameters.

The Kirpich formula (1940) is:

$$TC = 0.063\ (L/S^{0.5})^{0.8}$$

where:

L	=	length (km)

The Rational Method is recognised as oversimplifying the rainfall runoff process but is considered sufficiently accurate for runoff estimation for small contributing areas. The main assumptions for the method are:

- rainfall occurs at uniform intensity for a duration at least equal to the time of concentration of the catchment
- rainfall occurs uniformly over the entire area of the catchment
- the rainfall return period is the same as the runoff return period
- the factor C incorporates both the percentage runoff factor, an element for attenuation and also accounts for the routing effect.

Refinements to the Rational Method (the Modified Rational Method of the Wallingford Procedure) have been made to improve the accuracy of results through the split of the runoff coefficient C into two coefficients; a routing and a volumetric coefficient. The product of Cv and Cr for urban drainage is suggested as 0.6 and 1.3 respectively, this value approximates close to one. However in a rural context Cv (percentage runoff) would range from 0.1 through to 0.5 or so.

Due to the difficulties of estimating the percentage runoff and the time of concentration, the Rational Method is rarely used for estimating rural catchment peak flows.

A7.2 The TRRL method (Young and Prudhoe, 1973)

This method was developed specifically for small natural catchments to allow the estimation of peak flows for the sizing of road culverts. It was developed from rainfall and runoff data over several years of monitoring. The method used data collected from four catchments plus one other catchment from a previous study. The TRRL method was published before the Flood studies report (1975) and provides a procedure for the determination of peak flood flows for any return period from natural catchments using key catchment parameters. The method resembles a "rational" type formula. The following two equations define the time of concentration and the resulting peak flow rate:

$$T_C = 2.48\left(L_C N\right)^{0.39}$$

where:

T_C	=	the time of concentration when all the catchment area is contributing to the flood flow (hr)
L_C	=	the catchment dimension, measured from outfall to upstream divide (km)
N	=	dimensionless slope number equal to the ratio L/Z, where Z is the rise from the outfall to the average height of the upstream divide

From this value for T, the peak flow Qc can be predicted using the following formula:

$$Q_c = \frac{F_A\, A\, R_B}{3.6T}$$

where:

A	=	catchment area (km^3)
F_A	=	dimensionless annual rainfall factor (=0.00127 R_A – 0.321), and RA is the average annual rainfall (mm)
R_B	=	expected rainfall depth, which is given in tabulated form in Young and Prudhoe (1973) based on values from the Bilham formula (mm). The value for R_B can be derived as follows for any value of T and return period: 10/Return Period = 1.25 T$(0.0394R_B + 0.1)^{-3.55}$

This procedure is only considered suitable for natural catchments with an underlying soil type of clay as this was the predominant nature of the catchments that were measured. The authors state that the method is also applicable to catchments having shallow soil overlying rock. It is generally recognised that the limited number of catchments and period of measurement makes the equations application of limited value. However for small catchments of with these type of soil characteristics, it is a useful tool with which to carry out a check on the results produced by other methods.

A7.3 Flood studies report (FSR) – (NERC, 1975)

The *Flood studies report* provides a number of different ways of predicting flows and volumes for a given return period event. For the application to flood estimation from ungauged natural catchments the most applicable procedure is the statistical approach based on catchment descriptors.

The characteristic discharge adopted is the mean annual flood. This is defined as QBAR and is given by:

$$QBAR = 0.201 * AREA^{0.94}\ STMFRQ^{0.27}\ S1085^{1.23}\ SOIL^{1.23}\ RSMD^{1.03}\ (1 + LAKE)^{-0.85}$$

where:

QBAR	=	mean annual flood (m^3/s)
AREA	=	catchment area (km^2)
STMFRQ	=	stream frequency in terms of the average number of stream junctions per km^2
S1085	=	representative channel slope (in m/km) defined by points 10 per cent and 85 per cent upstream from the outflow point from the catchment
SOIL	=	index determined from five soil types defined by the winter rainfall acceptance potential (WRAP) map, RSMD is the net one day five year rainfall minus the Soil Moisture Deficit (mm)
LAKE	=	index of lake area as proportion of total area

Once the value of QBAR has been determined, design flood values for other return periods are obtained using an appropriate regional growth curve. Growth curves have been determined for the different areas of the United Kingdom and are contained within the FSR. The growth curves were revised in FSSR 16 and are shown in figure G1 as is the map of the hydrological regions for UK (Figure A7.2).

For completeness the WRAP map of UK is included in Figure A7.3 to provide an indication of soil type across the country.

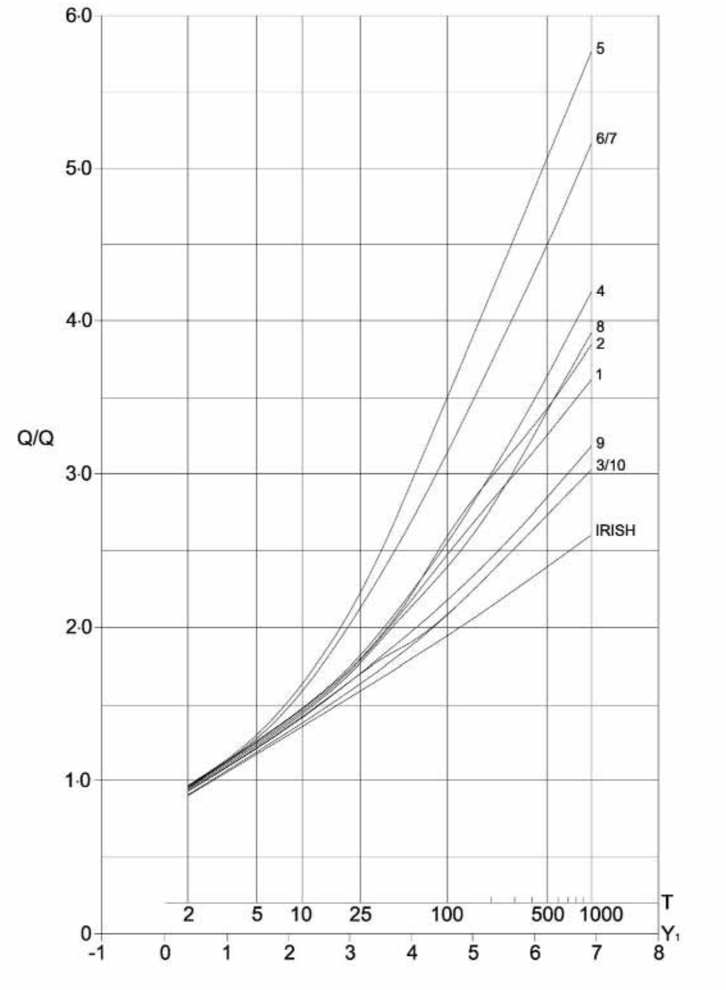


Figure A7.1 *Growth curve factors for UK hydrological regions (courtesy Institute of Hydrology 1983)*

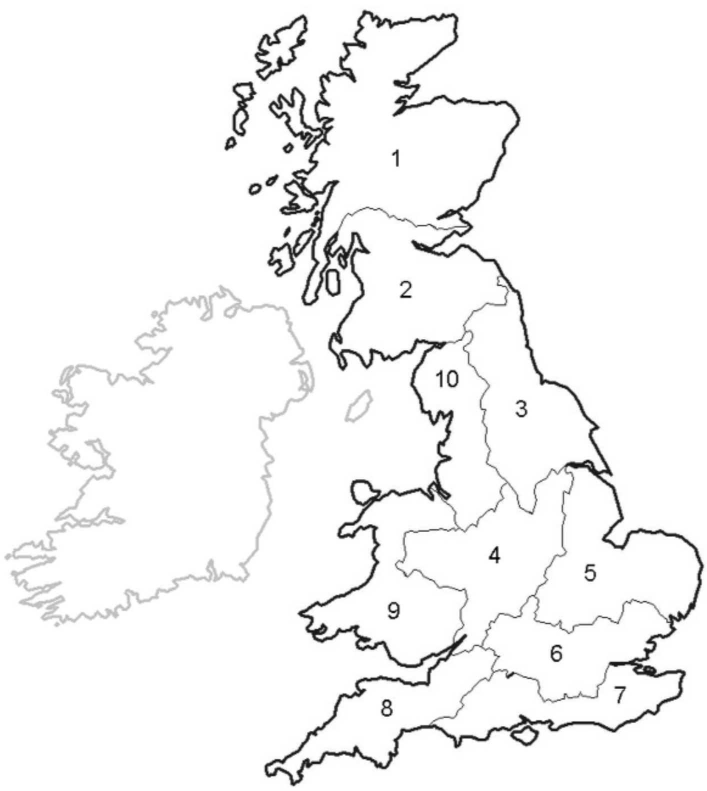


Figure A7.2 *The growth curve regions of UK (courtesy Natural Environment Research Council 1975)*

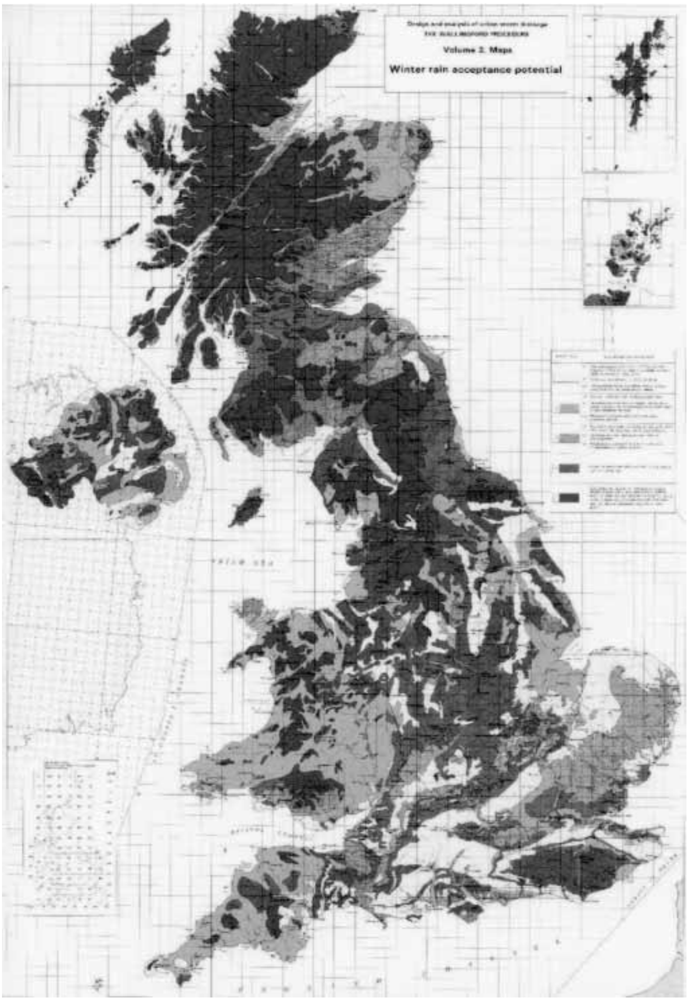


Figure A7.3 *WRAP map of SOIL type for UK (courtesy Department of Environment and National Water Council 1981)*

A7.4 FSSR 6 – flood prediction for small catchments

Following on from the *Flood studies report* in 1975, a number of other mini studies were carried out which refined various elements of the report. FSSR 6 was issued in 1978 to try and provide a better prediction of runoff estimation for small catchments although it was still limited to a minimum size of 50 ha, but less than 20 km^3.

The mean annual flood was derived for new three and four parameter equations. These are:

$$QBAR = 0.00066\ AREA^{0.92}\ SAAR^{1.22}\ SOIL^{2.0}$$

and:

$$QBAR = 0.0288\ AREA^{0.90}\ RSMD^{1.23}\ SOIL^{2.0}\ STMFRQ^{0.23}$$

Although they have the advantage of fewer parameters, particularly the three parameter equation, where parameters can be very easily found, in practice it was established that the accuracy of the predictions was not significantly improved from the general six parameter equation for all catchments.

The prediction of peak flows for small catchments was again revisited in the early '90s and is detailed below in IOH report 124 in Section A7.8.

A7.5 Poots and Cochrane method, 1979

A similar equation to FSSR 6 was derived by Poots and Cochrane from FSR data.

$$QBAR = 0.0136\ AREA^{0.866}\ RSMD^{1.413}\ SOIL^{1.521}$$

It is thought to provide similar levels of accuracy to FSSR 6.

A7.6 The ADAS method (Agricultural Development and Advisory Service, 1980)

The ADAS report was produced primarily for agricultural drainage and is aimed at providing information for the sizing of pipes for field drainage systems. The method is based on measurements taken on a number of very small, rural areas measuring flows collected by field drains. There are two forms of the ADAS method:

- basic method (which was derived for entirely clay type catchments)
- modified method (which was developed to take account of soil type and was based on work for the *Flood studies report*, 1975 and the Soil Survey of 1978).

The description below refers to the modified method in numerical form, (a graphical procedure is also available). This method takes into account the design storm rainfall and time of concentration for the required return period by using the Bilham formula. The equation to estimate runoff from a site is of the form:

$$Q = 13.73\ S_m^{\ 2.0}\ F_A\ A\ \frac{R_B}{T}$$

where:

Q	=	peak flow in l/s
S_m	=	soil type factor
A	=	catchment area in ha
F_A	=	annual rainfall factor (note that F_A is the same factor used in the TRRL method in LR565) where: $F_A = 0.00127\ R_A - 0.321$
R_A	=	average annual rainfall in mm
R_B	=	rainfall depth (as defined by the Bilham formula)
TC	=	time of concentration (as defined in LR565)
S_m	=	soil index, which is based on five soil classes that are dependent on their winter rainfall acceptance potential, and is defined as:

$$S_m = \frac{(0.10S_1 + 0.30S_2 + 0.37S_3 + 0.47S_4 = 0.53S_5)}{(1 - S_u)}$$

$S_{1,\ 2}$....denote the proportions of catchment covered by each of the soil classes 1 to 5 and S_u is the unclassified area of the catchment covered by water or pavement. Tables for the determination of the soil types are given in ADAS (1980). Soil class 1 has a low runoff potential and soil class 5 has a high runoff potential. The parameter S_m for a natural earth catchment can vary between 0.10 (very low runoff) to 0.53 (very high runoff).

The predicted peak flow rate resulting from the ADAS equation should be taken as the one year peak flood for the catchment. Flow rates for higher return periods can be calculated using the appropriate regional growth curve in the *Flood studies report*. Care should be taken in using the curves shown in the graphical method as they represent the one, five and 10 year return periods.

The ADAS procedure is relatively simple to apply but a limitation of the procedure is that the method should not be applied to catchments larger than 30 ha (or 0.3 km²). It should also be recognised that it represents the measured soil inter-flow collected by field drainage and therefore does not incorporate any over-land flow. The general consensus is that it probably underestimates stream runoff for flows in streams for extreme events.

A7.7 The SCS method (Soil Conservation Service, 1985 – 1993)

A number of different procedures for applying the SCS method exist. The simplest case, the TR-55 graphical method, is described here. In this method the peak discharge is determined based on catchment characteristics (time of concentration, drainage area, curve number) and a 24 hour rainfall amount and distribution. The method applies where:

- the catchment is accurately represented by a single curve number
- time of concentration is from 0.1 to 10 hours
- the catchment has only one main watercourse
- no valley or reservoir routing is required.

The curve number is a land use coefficient that dictates the relationship between total rainfall depth and direct storm runoff. The time of concentration and travel time can be computed to include sheet flow, shallow flow and channel flow. The method adjusts the catchment response as the amount of rainfall increases, to take account of greater saturation of the soil.

The SCS method is widely used in the United States for estimating peak runoff rates on small drainage basins. Application of the method is simple and direct; however, variation in results occurs in practice due to the choice of procedure to determine the time of concentration and to choose the curve number. The TR-55 procedure is suggested for drainage areas up to 50 km².

As is apparent from the above, a limitation of the SCS procedure for UK applications is that all the data used to generate the curve number parameter is from the United States. Equivalent rainfall types and other terms are required in order to apply this method to the UK.

A7.8 Institute of Hydrology report No.124 (1994)

The Institute of Hydrology Report No.124 was published in 1994 and describes research for flood estimation in small catchments. The research was based on 71 small rural catchments. An adjusted regression equation was produced to calculate the mean annual flood for small rural catchments. This methodology should preferably only be applied to a catchment drained by a well-defined watercourse and where the catchments are larger than 50 ha (ie 0.5 km²). This is the latest and the current most accepted method for predicting runoff. However as there is no slope function, in some instances it is still worthwhile checking the result obtained against one or more of the other methods.

From the analysis undertaken the three variables that were found to be most significant were SOIL, SAAR and AREA. The following equation was developed to predict the Mean Annual Flood ($QBAR_{rural}$) for the small rural catchments in the study. No information about the effect of other catchment parameters, such as slope, is given.

where:

$$QBAR_{rural} = 0.00108 \cdot AREA^{0.89} SAAR^{1.17} SOIL^{2.17}$$

$QBAR_{rural}$	=	mean annual flood for small rural catchments (m^3/s)
AREA	=	catchment area (km^2)
SAAR	=	standard average annual rainfall in mm for the period 1941 to 1970 in mm and particular location
SOIL	=	soil index which can be calculated as follows:

$$SOIL = \frac{(0.10S_1 + 0.30S_2 + 0.37S_3 + 0.47S_4 + 0.53S_5}{S_1 + S_2 + S_3 + S_4 + S_5}$$

S_1, …S_5 denote the proportions of the catchment covered by each of the soil classes 1-5.

Soil class 1 has a low runoff potential and soil class 5 has a high runoff potential. SOIL can vary from 0.10 (very low runoff) to 0.53 (very high runoff). SOIL may be estimated from the SOIL maps given in FSR or from knowledge of the catchment. The soils are classified according to runoff potential (RP).

QBAR can then be scaled according to the FSR regional growth curves to produce peak flood flows for the required return period and duration event.

A7.9 Flood estimation handbook (FEH, 1999)

The methods of flood estimation in the *Flood estimation handbook* unofficially supersedes the *Flood studies report* and the *Flood studies supplementary reports* as standard practice for assessing river behaviour in UK. The Environment Agency has officially endorsed FEH as the method that should be used for rainfall and runoff estimation.

Digital terrain and thematic data are provided for all catchments in the UK down to an area of 0.5 km². The FEH method cannot be specifically applied to very small catchments due to the inability to define digital catchment areas other than those given in the FEH tool. In these instances interpolation of results is required. The whole tool is digital and provides parameter values for every kilometre square across the UK. As in the case of FSR, the minimum advised catchment area is 50 ha.

The *Flood estimation handbook* is supported by three software packages:

- WINFAP-FEH to support the statistical procedures for flood frequency estimation
- Micro-FSR to apply the *Flood studies report* rainfall-runoff method
- FEH CD-Rom which presents catchment descriptors for four million UK sites and implementing the rainfall frequency estimation procedure.

For the estimation of runoff from greenfield, sites, the Micro-FSR package and FEH CD-Rom are applicable. The FEH CD-Rom includes a digital terrain model of the whole of the UK produced by the Institute of Hydrology from 1:50 000 scale maps.

The CD-Rom allows the delineation of catchment boundaries to be carried out automatically. The catchment boundaries based on the terrain map are inevitably approximate. However, it should be noted that discrepancies are most likely to arise in small catchments.

The general philosophy behind flood frequency estimation in the FEH is as follows:

- flood frequency is best estimated from gauged data
- where gauged data are not available, data transfers from nearby or similar catchments are useful
- estimation of floods from catchment descriptors alone should be used as a method of last resort.

It should be noted that the FEH provides catchment descriptors for all sites draining an area of 0.5 km^2 (50 ha) or greater based on a 50 m resolution DTM model. The lower limit reflects that:

- very small catchments are poorly represented in the data sets used to calibrate the models for estimating flood frequency from catchment descriptors
- digital terrain and thematic data may not be well resolved on very small catchments.

The statistical procedures and rainfall-runoff methods used in the FEH are outlined below.

A7.10 Statistical procedures for flood estimation

The major changes to the statistical procedures for flood frequency estimation compared with the *Flood studies report* are as follows:

- the median annual flood Q_{MED} rather than Q_{BAR} is used as the index variable
- where no gauged data are available, Q_{MED} is estimated from catchment descriptors based on digital data rather than derived manually from maps
- the derivation of the growth curve for the catchment is flexible rather than fixed and is generally based on pooling of relevant flood peak data, or in a few cases, the catchment descriptors
- catchment similarity is initially judged in terms of size, wetness and soil.

The index flood represents the typical magnitude of a flood expected at a given site. The FEH uses the median annual flood Q_{MED} as the index flood. Q_{MED} is the median value of the annual maximum flow series. There are several methods for estimating Q_{MED}. These are described below.

If the catchment is gauged then Q_{MED} can be estimated from the annual maximum flow data. This method is recommended if there are 14 or more years of records. Where there are more than two years and less than 14 years of annual maximum flows, the peaks over threshold data should be used.

The recommended procedure for Q_{MED} at sites where there are no flood peak data is to transfer data from a nearby donor site or from a more distant analogue catchment. A prerequisite for the data transfer is that the donor/analogous catchment is hydrologically similar to the catchment of interest. Further details of these data transfer techniques are given in the FEH.

For ungauged rural catchments where data transfer is not possible, $QMED_{rural}$ can be estimated from five catchment descriptors. These are:

- drainage area (AREA)
- average annual rainfall (SAAR)
- soil drainage type represented by SPRHOST and BFIHOST
- storage attenuation due to reservoirs and lakes (FARL)
- baseflow index (BFI).

FEH catchment descriptors are based on drainage boundaries defined by a digital terrain model. Inconsistencies may arise on small catchments, this method should only be used as a last resort in this situation.

A7.11 Rainfall – runoff method for flood estimation

The rainfall-runoff method remains similar to that described in the *Flood studies report*. However, where no gauged data are available, catchment descriptors should be based on the FEH digital data, rather than derived manually from maps. The unit hydrograph method used by the Micro-FSR software package is still applicable.

The FEH rainfall analysis was carried out for the period 1971 to 1990 whereas the FSR analysis was based on data between 1941 and 1970. The results show quite significant differences when compared with the FSR rainfall characteristics. Although there may be an element of climate change, the view is generally held that the difference is more a function of the statistical methods used for analysing the data. These differences can be as high as 40 per cent in parts of the south east of the UK, while other areas have very similar predicted depths. The difference between FSR and FEH for more extreme rainfall nearly always show a greater rainfall in the FEH assessment of rainfall depth. The main exceptions are in east Scotland and Northern Ireland where FSR tends to predict higher rainfall depths. For small return periods the difference between FSR and FEH is generally less though there are hot-spots, such as the Reading area, which still has a significant increase in rainfall depth for any return period. Figure A7.4 illustrates the differences in rainfall depths between FSR and FEH across the UK.

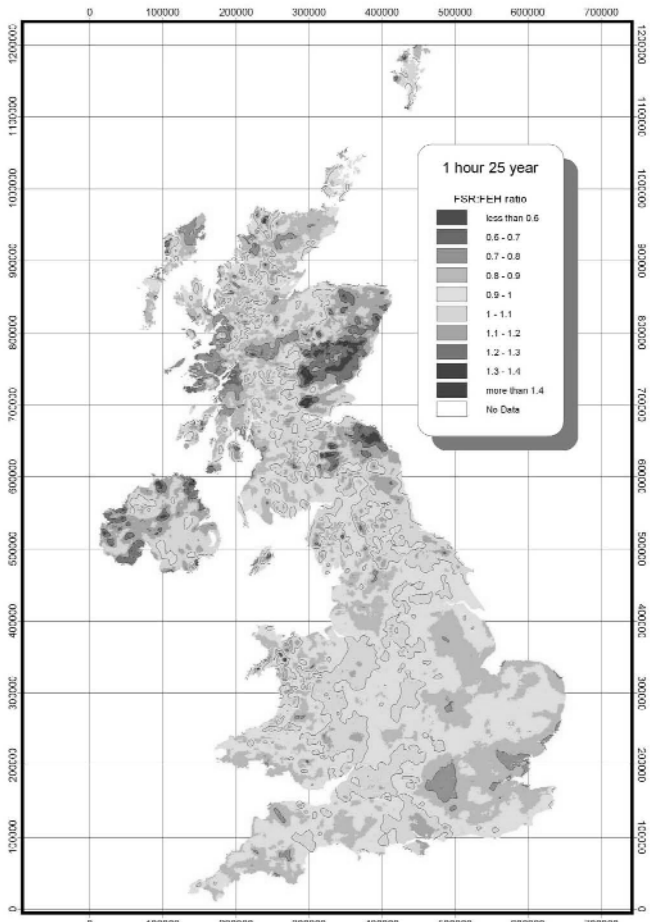


Figure A7.4 *FSR/FEH rainfall depth ratios for the one hour 25 year event (courtesy UKWIR)*

Estimation of QMED (median annual maximum flood) from catchment descriptors is produced using the following equation:

where:

$$QMED_{rural} = 1.172 AREA^{AE} \left(\frac{SAAR}{1000}\right)^{1.560} FARL^{2.642} \left(\frac{SPRHOST}{100}\right)^{1.211} 0.0198^{RESHOST}$$

FARL	=	factor to account for the presence of reservoirs and lakes
SPRHOST	=	soil parameters
RESHOST	=	soil parameters
AE	=	area exponent the other factors are defined by FSR

FEH uses a 29-class Hydrology of Soil Types (HOST) classification and the database gives the percentage of each class present in a 1 km × 1 km grid. Currently there is no paper map version of the country showing the HOST soil classes.

Rainfall growth factors in FSR were considered to be over general, masking local and regional variations in rainfall. The FEH procedure takes more account of local data, both in constructing the focused growth curves and in mapping the standardising variable, RMED.

The following advice is given in the FEH manual as to the applicability of QMED (median annual maximum flood) from catchment descriptors:

- the model is only applicable to rural UK catchments
- the model should not be applied to unusual catchments
- the model should not be relied on if the runoff is strongly influenced by lakes and reservoirs
- QMED is likely to be less accurately estimated for permeable catchments.

A7.12 Advantages and disadvantages of the Flood estimation handbook techniques

The advantages of the *Flood estimation handbook* are as follows:

- uses pooling and transfer of flow data to estimate flows from ungauged catchments
- the catchment area and descriptors are calculated digitally from a digital terrain model
- the pooling of the flood flow data for defining the growth curve is flexible and tailored to fit the subject site.

The disadvantages of the *Flood estimation handbook* are as follows:

- requires detailed hydrological knowledge to apply the techniques correctly
- requires access to and detailed knowledge of the use of the Flood estimation CD-Rom and WINFAP_FEH software
- catchments below 0.5 km^2 cannot be defined on the *Flood estimation handbook* digital terrain model
- definition of small catchments (ie catchments with an area less than 1 km^2) may be inaccurate in some cases particularly adjacent to urban areas where contours are not well defined
- small, partially urbanised catchments, where the urban fraction is significant, should not use the catchment descriptor method.

A7.13 FSR and FSSR 5, and 16 percentage runoff estimation

The original FSR produced a formula for predicting the volume of runoff from a catchment. The correlation coefficient R^2 was quite low at 0.43. The formula was revisited in FSSR 5 (1979) *Design flood estimation in catchments subject to urbanisation* and also in FSSR 6 (1978) *Flood prediction for small catchments* and finally in FSSR 16 (1985) *The FSR rainfall – runoff model parameter estimation equations updated*. Percentage runoff was not addressed again in IH report 124.

In general, the formula are all of the same form using SOIL, a function of CWI and SAAR and rainfall depth. There is also a final term for the additional runoff which would take place from any urban areas within the catchment. None of the models included a slope term. The SOIL term is a weighted SOIL value based on the amount of area of each soil type in the catchment. As FSSR 16 was the final version this is the only one reported here, and it is the presumed that it relates to flood response and therefore relates to extreme events only.

The FSSR 16 formula is:

$$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$$

where:

SPR	=	standard percentage runoff, which is a function of the five soil classes: $10S_1 + 30S_2 + 37S_3 + 47S_4 + 53S_5$
DPR_{CWI}	=	dynamic component of the percentage runoff. This parameter reflects the increase in percentage runoff with catchment wetness. The catchment wetness index (CWI) is a function of the average annual rainfall, shown in Figure A7.5. $DPR_{CWI} = 0.25\ (CWI - 125)$
DPR_{RAIN}	=	second dynamic component that increases the percentage runoff from large rainfall events. $DPR_{RAIN} = 0.45(P - 40)^{0.7}$ for P > 40mm (where P is rainfall depth) $DPR_{RAIN} = 0$ for $P \leq 40$mm

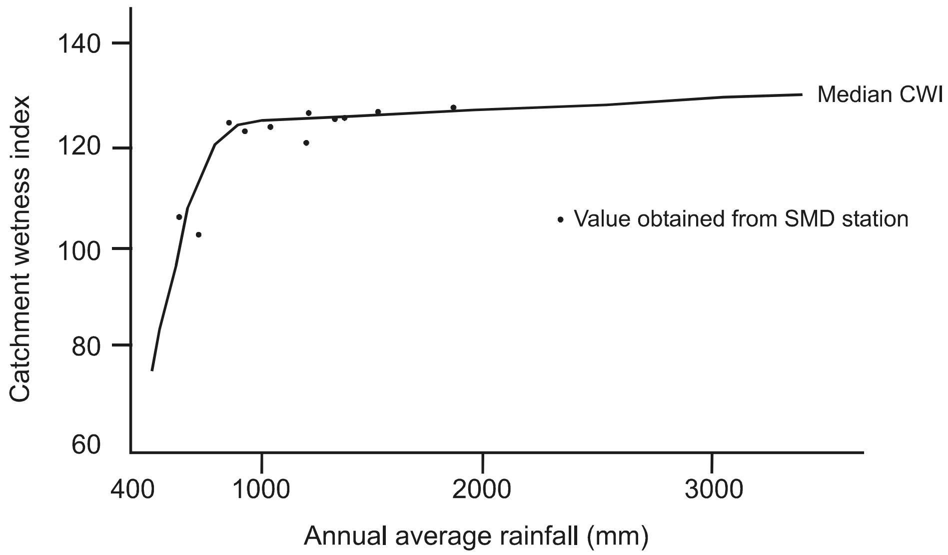


Figure A7.5 *CWI versus SAAR (Flood studies report)*

It can be seen from the formula that the runoff proportion is only slightly greater than the value of SPR for all areas where the AAR value is greater than 800 mm, however SPR is by far the most dominant factor. The key feature of this formula is the important influence of soil type. In practice it indicates that developments on sandy soils create massive additional runoff compared with the pre-development condition, but development on clays much less so. This is obvious, but it has very significant implications for the cost of developments in terms of the storage provision if a criterion is applied relating to the greenfield runoff volume. The other parameters have very little influence.

To calculate runoff from a greenfield site two options are available. Either the full formula is applied or a simple assumption is made that the runoff from a greenfield site is equal to the SPR (standard percentage runoff) value for the soil type. The assumption that SPR is the runoff proportion is a reasonable approximation for extreme events even though runoff is clearly related to catchment wetness.

A7.14 FEH runoff model – variable percentage runoff

The FSSR 16 model is also used in the *Flood estimation handbook* runoff model. There is however, an alternative to the calculation of the DPR_{CWI} value compared with that given in the *Flood studies report* to allow the percentage runoff to be related to the increasing catchment wetness through an event. Instead of CWI factor being a constant related to SAAR, it is a function of the API5 and SMD and is given by the following equation:

$$CWI = 125 + API5 - SMD$$

where:

API5	=	five day antecedent moisture precipitation index
SMD	=	pre-event soil moisture deficit

SMD values are a function of the soil type, antecedent rainfall and evaporation. In winter months and in very wet conditions, SMD will usually be close to zero, which represents saturation of the soil field capacity. SMD values of 100 mm or more can occur in summer.

This equation allows the prediction of greenfield runoff of an event including those which are not extreme (that would not cause river flooding). This has considerable advantages in that runoff from a catchment can be assessed for all events for a site. This means that the design of drainage systems can assessed against a prediction of the greenfield runoff for a range of rainfall and runoff criteria.

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Glossary

Annual probability	The estimated probability of a flood of given magnitude occurring or being exceeded in any year. Expressed as, for example, 1 in 100 chance or 1 per cent.
Attenuation	A reduction of the rate of flow with a consequent increase in the duration of the flow.
Catchment	The area contributing flow or runoff to a point on a drainage or river system. Can be divided into sub-catchments.
Climate change	Long term variations in global temperature and weather patterns both natural and as a result of human activity, primarily greenhouse gas emissions.
Conveyance	The movement of water from one location to another.
Conveyance capacity	The capacity of a system to convey flow. In piped systems, the conveyance capacity will exceed the pipe full capacity due to flow backing up in manholes.
Default pathways	Surface flood pathways which have not been designed to convey flows.
Depression storage	The depth of water retained on the ground surface in puddles or other depressions.
Design criteria	A set of standards agreed by the developer, planners and regulators that the proposed system should satisfy.
Design event	A historic or notional flood event of a given annual flood probability, against which the suitability of a proposed development is assessed and mitigation measures, if any, are designed.
Designed pathways	Surface flood pathways which have been designed to convey flows.
Detention basin	A vegetated depression that is normally dry except following storm events constructed to store water temporarily to attenuate flows. May allow infiltration of water to the ground.
Development	The carrying out of building, engineering, mining or other operations in, on, over or under land or the making of any material change in the use of any buildings or other land.
Discharge	Rate of water flow.
Downstream system	The system to which the defined area which is subject to exceedance discharges to eg an urban drainage system or watercourse.
Evapo-transpiration	The process by which the earth's surface or soil loses moisture by evaporation of water and its uptake and then transpiration from plants.

Exceedance flood by risk assessment	A study to assess the risk of a site or area being affected exceedance flow, and to assess the impact that any changes made to the areas or site will have on the exceedance flood risk.
Exceedance flow	Excess flow that appears on the surface once the conveyance capacity of the minor system is exceeded.
Extreme events	Rainfall events that are sufficient in size to produce exceedance flows.
Fluvial flooding	Flooding from a river or other watercourse.
Freeboard	The difference between the level of where flooding or overtopping may occur and the designed water level.
Greenfield runoff rate	The rate of runoff that would occur from the site in its undeveloped, and undisturbed, state.
Groundwater	Water in the ground, usually referring to water in the saturated zone below the water table.
Gully	Opening in the road pavement that allows water to enter conventional drainage systems, often covered by a metal grate.
Hydrograph	A graph, that shows the variation with time of the level or discharge in a watercourse.
Hydraulic gradient	In an open channel, the gradient of the water surface; in a pressurised pipe, the gradient joining points to which water would rise in pressure tappings.
Impermeable surface	An artificial non-porous surface that generates a surface water runoff following rainfall.
Infiltration	The passage of water through a surface, either the pervious surface or the underlying ground.
Infiltration system	A system designed to aid infiltration of surface water to the ground.
Internal drainage board	Body with powers and duties relating to ordinary watercourses within an internal drainage district.
Level of protection	A target level in which no flooding should occur eg a 0.02 probability of a property being flooded.
Level of service	A level of water, usually specified as a return period to which an urban drainage system is designed to provide as a minimum, referred to as a target level.
Local planning authority	Body responsible for planning and controlling development, through the planning system.
Main river	A watercourse designated on a statutory map of main rivers, maintained by Defra.
Major system	The system of above ground flood pathways, including both open and culverted watercourses.
Minor system	The formal or designed drainage system.
Ordinary watercourse	A watercourse which is not a private drain and is not designated a main river.

Overland flow	Overland flow is water flowing over the ground surface that has not entered or has escaped from a natural drainage channel or artificial drainage system (other commonly used terms for this phenomenon are pluvial flooding or surface water runoff flooding).
Percentage runoff	The proportion of rainfall that runs off a surface.
Pervious surface	A surface that allows inflow of rainwater into the underlying construction or soil.
Pluvial flooding	Flooding caused by overland flow that has not entered into a natural drainage channel or artificial drainage system.
Pond	Permanently wet basin designed to retain stormwater.
Probability of exceedance	The statistical probability of a hydrological event (rainfall or flow) of a given magnitude being exceeded in any individual year.
Public sewer	A sewer that is vested and maintained by the sewerage undertaker.
Return period	The frequency with which an event occurs. A one hundred year storm is one that occurs on average once every 100 years, ie its annual probability of exceedance is 1 per cent (1/100).
Runoff	The flow of water, caused by rainfall, from an area over the ground surface to the drainage system. This occurs if the ground is impermeable or is saturated.
Runoff co-efficient	A measure of the amount of rainfall that is converted to runoff.
Sacrificial areas	Low value land to which exceedance flood volumes can be discharged and retained for limited periods of time.
Sewer	A pipe of channel with a proper outfall that takes domestic foul and/or surface water form buildings and associated paths and hardstandings from two or more curtilages.
Sewerage undertaker	An organisation with the legal duty to provide sewerage services in an area, including the disposal of surface water from roofs and yards of premises.
Sewers for adoption	A guide agreed between sewerage undertakers and developers (through the House Builders Federation) specifying the standards to which private sewers need to be constructed to facilitate adoption.
Soakaway	A subsurface structure into which surface water is conveyed to allow infiltration into the ground.
Stakeholders	A single person or group of people, company or organisation with an interest in managing exceedance.
Surface flood pathways	Routes in which exceedance flows are conveyed on the ground.
Sustainable development	Development which meets the needs of the present without compromising the ability of future generations to meet their own needs.

Sustainable drainage systems	A sequence of management practices and control structures, often referred to as SUDS, designed to drain surface water in a more sustainable manner than some conventional techniques. Typically, these techniques are used to attenuate rates of runoff from development sites. Sometimes called sustainable urban drainage systems.
Swale	A shallow vegetated channel designed to conduct and retain water, but may also permit infiltration.
Target level	A defined level of protection which is satisfied when failure occurs less frequently than the specified level.
Time of entry	Time taken for rainwater to reach an inlet into the drainage system after hitting the ground.
Trigger level	When failure occurs more frequently than the identified level of protection.
Watercourse	Any natural or artificial channel that conveys surface and/or groundwater.

Abbreviations

á	Proportion of paved area draining to the network or directly to the river
â	Proportion of pervious area draining to the network or directly to the river
A	Catchment area
AC	Cross-sectional area of flow
AE	Area exponent
Ae / Ai (ch9)	Equivalent Impermeable Area
AF	Cross-sectional area of flow in channel
ADAS	Agricultural drainage advisory service
API5	Five day antecedent precipitation index
AREA	Catchment area
ARF	Areal reduction factor
b	Average width at entry to downstream channel
BFI	Baseflow index
BFIHOST	Soil parameters
C	Runoff co-efficient
CP	Decay constant
CEH	Centre for Ecology and Hydrology
CFD	Computational fluid dynamics
CIRIA	Construction Industry Research and Information Association
CWI	Catchment wetness index
D	Depth
dc	Critical depth
Defra	Department of Environment Food and Rural Affairs
DoE(NI)	Department of Environment (Northern Ireland)
DPR	Dynamic component of the percentage runoff
DTLR	Department of Transport, Local Government and the Regions
DTM	Digital terrain model
EA	Environment Agency
EFRA	Exceedance flood risk assessment
FA	Annual rainfall factor
FARL	Factor to account for storage in reservoirs and lakes
FEH	Flood estimation handbook
FLAG	Flood Liaison and Advice Group
FSR	Flood studies report

FSSR	Flood Studies Supplementary Report
g	Gravitational acceleration
GBG	Good Building Guide
Gd	Gully factor defined by gully grating geometry
Gout	Gully outlet discharge
H	Average depth of flow in the gully channel
HA	Highways Agency
i	Rainfall intensity
ICE	Institution of Civil Engineers
IF	Effective impervious area factor
IOH	Institute of Hydrology
L	Design life (years)
LAKE	Index of lake area as proportion of total area
LC	Catchment dimension, measured from outfall to upstream divide
LDF	Local development framework
LiDAR	Light detection and ranging
M	Factor to allow for maintenance
MAFF	Ministry of Agriculture Fisheries and Food
n	Manning roughness value
N	Slope number
NAPI	30 day antecedent precipitation index
NERC	Natural Environment Research Council
ngull	Number of gullies per hectare
NHBC	National House Building Council
ODPM	Office of the Deputy Prime Minister
Ofwat	Office of Water Services
OS	Ordnance Survey
OST	Office of Science and Technology
P	Wetted perimeter
PR	Probability of event occurring or being exceeded within design life
PAN	Planning Advice Note
PF	Porosity Factor
PIMP	Percentage of impermeable area
PPG	Planning Policy Guidance
PPS	Planning Policy Statement
PR	Percentage runoff
Q	Flow/discharge
Qb	Flow by passing gully
QBAR	Mean annual flood (m^3/s)
QC	Peak flow

Qi	Flow entering gully
QMED	Median annual flood
Qr	Flow collected from the contributing area to each gully
R	Hydraulic radius
RA	Average annual rainfall (mm)
RB	Rainfall depth (as defined by Bilham formula)
RD	Rainfall depth for the 100 year, six hour event
RSMD	Net one day, five year rainfall minus the Soil Moisture Deficit (mm)
RSS	Regional Spatial Strategy
s	Slope
SAAR (mm)	Standard annual average rainfall
SEPA	Scottish Environment Protection Agency
SL	Average longitudinal slope of highway
SMD	Soil moisture deficit
Sm	Soil index
SOIL	An index of the water holding capacity of the soil
SPP	Scottish Planning Policy
SPR	Standard percentage runoff
SPRHOST	Soil parameters
STMFRQ	Stream frequency in terms of the average number of stream junctions per km^2
SUDS	Sustainable drainage systems
S1085	Representative channel slope (m/km) defined by points 10 and 85 per cent upstream from the outflow point from the catchment
T	Return period
TAN	Technical Advice Note
Tc	Time of concentration
TRRL	Transport Road Research Laboratory
UCWI	Urban catchment wetness index
UKWIR	United Kingdom Water Industry Research Limited
V	Velocity
DV	Velocity*depth
VolXS	Extra runoff volume of development runoff over greenfield runoff
W	Width of water surface
WRAP	Winter rainfall acceptance potential