# Storage requirements for rainfall runoff from greenfield development sites

R B B Kellagher

Version 2

Report SR 580
March 2002

# Storage requirements for rainfall runoff from greenfield development sites

R B B Kellagher

Version 2

Report SR 580
March 2002

HR Wallingford

**Address and Registered Office:** HR Wallingford Ltd. **Howbery Park, Wallingford, OXON OX10 8BA**
**Tel: +44 (0) 1491 835381 Fax: +44 (0) 1491 832233**

# *Contract - Research*

HR Wallingford Ltd was appointed by the DETR to carry out research and produce an industry guide on the design of stormwater storage for new developments. The DETR reference is CI 39/5/122. The HR Wallingford job number is MAS 0420. The project leader was Richard Kellagher.

Prepared by ..........................................................................................

*(name)*

..........................................................................................

*(Title)*

Approved by ..........................................................................................

*(name)*

..........................................................................................

*(Title)*

Authorised by ..........................................................................................

*(name)*

..........................................................................................

*(Title)*

Date ..........................................

# Foreword

This manual has been brought to the industry by HR Wallingford with financial support from DETR and the steering group members. The steering group has been drawn from all relevant parts of the water industry to tackle a problem which has been in need of resolution for many years. It has been clear for some time that policy on the provision of site storage for urban stormwater runoff is now considered for all developments, but is applied using a range of criteria by the regulatory bodies. This has resulted in confusion and difficulties in obtaining consents and determining the cost of drainage for discharge of rainfall runoff.

In addition there has been some suspicion that the methods normally being used are potentially flawed in achieving their objectives and not cost effective. The subject is extremely important as has been demonstrated by the apparent increasing severity of river flooding and the fact that 4 million new homes are to be built by 2016.

This report should therefore provide a valuable contribution to industry in proposing a more cost effective procedure for stormwater storage design as well as developing better ways of protecting the environment.

This project was led by R Kellagher of HR Wallingford with assistance from P Lo Greco, B Woods Ballard and D Lumbroso. HR Wallingford is also indebted for the guidance that has been provided by the following industry experts:

| | |
|---|---|
| Environment Agency | D Crowson |
| Environment Agency | A Pepper |
| Environment Agency | D Rylands |
| Southern Water Services | P Forster |
| Thames Water Utilities | R Waller |
| Wimpey Homes | S Weilebski |
| Wimpey Homes | M Bowden |
| Wimpey Homes | A MacInnes |
| House Builders Federation | D Baker |

This document is the result of the research carried out by HR Wallingford. The recommended approach for determining storage is HR Wallingford's suggestion for achieving the various objectives of protecting rivers and the development sites. The report is not to be considered as being official national procedure. In the light of this research, the Environment Agency are taking further action to define and produce an official national guidance document.

# Summary

Storage requirements for rainfall runoff from greenfield development sites

R B B Kellagher

Version 2

Report SR 580
March 2002

The aims of the drainage guide for the provision of stormwater storage are:

- Provide guidance on best practice for determining rainfall runoff from greenfield and developed sites.
- Provide a summary of the current requirements and practices for obtaining stormwater discharge consents.
- Test the basis of the philosophy for provision of temporary storage for protecting rivers in flood
- Provide guidance on best practice which is technically effective and meets the concepts of sustainability

At present there is little consensus of opinion amongst regulators and site developers as to the criteria that should be applied to control runoff from developments constructed on greenfield sites. There is also no consistency in defining the limit of the discharge used for sizing detention storage, which has a significant impact on the volume of the storage required and, as a consequence, its cost.

The lack of consistent guidance from regulatory bodies has frustrated developers for a number of years. Runoff control measures for development projects are often subject to extensive periods of discussion and the implemented control measures are often perceived by developers to be unnecessarily conservative. The absence of scientific justification of the use of storage and consistent technical guidance on development runoff storage measures is an issue that has caused considerable concern in the construction industry for over a decade.

The research output for this study was to provide a detailed analysis of the problem and to determine the most appropriate method of approach to achieve the objectives of sustainability and river flood protection. An official guide for use by the regulators, developers and consultants may be produced to provide a national consensus of approach to the provision of storage once the Environment Agency has approved or modified this recommended procedure.

This research document details the information relating to the aims stated above. The tests carried out were based on information from 10 sites where extensive rainfall data exists together with an adjacent river gauge that has recorded flow data for the same period. Analysis of nearly 100 potential sites in England was carried out to select the data sets. This involved assessing the reliability of the data, ensuring a range of catchment sizes, a good geographical distribution and also took into account the soil types.

## ***Summary*** *continued*

These 10 selected sites were then assessed to determine the actual storage volume utilised during rainfall events and also the effectiveness of providing temporary storage in protecting the river during periods of flooding. The length of data sets ranged from 14 to 38 years.

The conclusions of the research indicate that the use of temporary storage is not generally effective in protecting rivers at times of river flooding. Due to the variety and variability of catchments and the limited number of events that could be considered as extreme (for which storage is being designed), a degree of caution should be exercised in interpreting the results. However it does seem clear that current methods are ineffective and that the methodology for design currently used to calculate storage is not appropriate.

The document concludes with a proposed philosophy and procedure for the provision of storage which aims at new developments meeting the concepts of sustainability and that will effectively protect rivers during extreme events.

The philosophy aims to replicate post development runoff from sites to that which occurred prior to development. It is suggested that this is applied to a range of return periods to meet several objectives such as water quality protection, and level of service for the site. To do this, use is made of methods for estimating both peak rates of flow from greenfield runoff, and also calculating volumes of runoff generated by events from both the pre-development and post-development condition of the site.

Two examples are given in an appendix to show the method of approach of applying the proposed procedure, and the advantages it provides over the current approaches generally used at present.

# Contents

## ***Contents*** *continued*

# *Contents* continued

## ***Contents*** *continued*

# *Contents* continued

# *Contents* continued



**Appendices**

# 1. INTRODUCTION TO THE RESEARCH

## 1.1 Background

The British Government has stated that up to four million new houses are required in the south-east of the UK by 2016. Although many of these houses will be constructed on brownfield sites, there will be a large number of developments on greenfield sites. The development of a greenfield site increases the impervious area of a catchment, and this leads to an increase in the peak flow and volume of runoff from the developed site. This increase in the impervious area often increases the risk of flooding from the receiving watercourse or sewer, as well as lowering local groundwater levels, coupled with an increase in the pollution load in the runoff. To reduce these impacts on the receiving environment, developers are often required by the Environment Agency, the sewerage undertaker or relevant planning authority to implement runoff control measures. The current techniques to control runoff from developed sites generally fall into three categories. These are:

- Storage tanks;
- Oversized pipes (tank sewers);
- Open ponds.

The cost of implementing detention systems has been estimated to be between £250 and £500 per cubic metre of detention storage. With the government predicting the need for so many new homes over the next ten to fifteen years, the total cost of providing sufficient runoff control for the proposed developments could cost many millions of pounds.

At present there is no consistent approach amongst regulators as to how to control runoff from developments constructed on greenfield sites. There is also no consistency in defining the limit of the outflow discharge used for sizing detention storage. The outflow discharge rate set for attenuation storage has a significant impact on the volume of the storage required and, as a consequence, its cost. Outflow discharge rates currently used by the Environment Agency usually range from 3 l/s/ha to 8 l/s/ha, but both higher and lower figures have been required. In some Environment Agency areas the discharge consent requirement is to attenuate the 1 in 100 year runoff from the development site to the equivalent 1 in 1 year runoff from the undeveloped site. Such criteria leads to costly runoff control structures which appear to be unjustifiable.

When the runoff from the development site is to be discharged to sewers owned by the sewerage undertaker, there is also limited guidance in the approach to be used. However, this is less of a problem as constraints are a function of the receiving sewer capacity. Runoff control criteria applied by statutory storage undertakers are generally less onerous than those applied by the Environment Agency and criteria is usually based on a 30 year return period.

The lack of consistent guidance from both the Environment Agency and other regulatory bodies has frustrated developers for a number of years. Runoff control measures for development projects are often subject to extensive periods of discussion with limited guidance being provided on control measures to be implemented. The technical aspects involved with meeting Environment Agency criteria often lead to difficulties with both the construction and maintenance of the runoff control measures. Developers value the certainty that is provided by a framework of clear policies rather than the uncertainty that is presented when decisions on the size and type of runoff control are made on a case by case basis. This is because the viability of a development can be affected by the cost of the storage requirements.

## 1.2 The move towards sustainable drainage systems

In recent years the use of sustainable drainage systems (SUDS) as a means of controlling runoff from developments has become increasingly advocated by the Environment Agency as a way of not only attenuating runoff from developed sites but also improving the quality of urban runoff. There are four principal types of Sustainable Urban Drainage Systems. These are:

- Permeable pavements;
- Filter strips and swales;
- Infiltration devices;
- Basins and wetlands.

At present developers are reluctant to use Sustainable Urban Drainage Systems as a form of runoff control owing to the lack of technical information on the design and long term performance of these systems and also the difficulties of getting them adopted and maintained. The Sewers for Adoption document published by the Water Authorities Association, currently the 5th edition, clearly sets out the design requirements for adopting conventional sewerage systems. However, Sustainable Urban Drainage Systems can be considered as either drainage or landscape features and as such there is (currently) no clear indication as to who is responsible for the ownership of such systems. Developers are reluctant to take responsibility for the ownership and maintenance of Sustainable Urban Drainage Systems. Legislation dealing with urban drainage is complex. It is possible that a specific piece of legislation will need to be enacted to ensure that sustainability issues are considered when designing and constructing new drainage systems. However the debate is currently being moved forward by the Environment Agency, SEPA and DETR. DETR produced guidance note PPG25 which strongly promotes SUDS and is funding a number of SUDS research projects. It is therefore likely that the industry will rapidly move towards the position of considering SUDS techniques for most new drainage.

## 1.3 Main objectives of the research

The objectives of providing storage can essentially be summarised as the maintenance of the river regime in its natural state, by minimising the difference between the developed and undeveloped catchment in terms of rainfall runoff, on the receiving watercourse and perhaps groundwater recharge. It is therefore important to develop an understanding of what these “differences” might be, in order that recommendations may be developed for the assessment of future storage requirements and designs.

However, there has been no scientific testing of the efficiency of providing temporary storage in protecting the receiving watercourse.

The main objectives of the project were as follows:

- To develop a consistent method of assessing surface water runoff from development sites;
- To test the philosophy of using storage to reduce flood risks in rivers;
- To produce a set of criteria for assessing storage requirements that might be approved by the Environment Agency and adopted as a standard procedure for the construction industry;
- To provide guidance for determining the allowable surface water runoff rates from greenfield sites to enable the sizing of runoff control structures;
- To produce dissemination material outlining the details of the project to ensure national awareness.

## 1.4 Issues addressed by the research report

The research detailed in this document has been undertaken to address the inconsistencies in the way runoff control measures are sized and to provide guidance to regulators and developers regarding the following:

- Current requirements for storage criteria;
- Methods for the estimation of runoff rates from greenfield sites;
- Methods for the estimation of runoff from developed sites;
- Proposed requirements for storage for developed sites.

In order to fully address these issues, the following actions were carried out:

- A scientific test of ten theoretical development sites to assess the effectiveness of temporary storage;
- A review of rural and urban runoff models;
- A survey of current practices applied by the Environment Agency on the subject of temporary runoff storage for developments.

## 2. AIMS AND OBJECTIVES OF THE RESEARCH

The principal objective of the research carried out was the production of guidelines that can be used by both regulators and the developers to assess storage requirements to control runoff from new developments. This main aim has been achieved through meeting the following objectives:

1. Undertaking a review of current practices for the control of runoff from new developments used in England and Wales.

2. Assessing the influence and effectiveness of a range of limiting discharge rates on storage requirements for ten theoretical development sites across England.

3. Investigating the part played by temporary storage in mitigating flood risk that may occur as a result of development.

4. Understanding how infiltration storage can reduce the risk of environmental damage to the receiving watercourse during periods of low flows.

5. Investigating how development sites, which incorporate temporary storage, infiltration areas, and zones designated to flood in extreme events, can be utilised for reducing the impact of proposed development.

Brief details of how these objectives were achieved are given below.

### 2.1 Review of current practices

A review of current practices used in England and Wales for controlling runoff from new developments was undertaken. A questionnaire was circulated to area offices within each of the eight Environment Agency Regions and also a number of developers and sewerage undertakers. The questionnaire was used to establish the following:

- The reasons and aims for providing detention storage;
- The Environment Agency Regions' policies on storage and runoff control;
- House builders' experience of regulatory requirements for providing runoff control;
- Methods used to assess the limiting discharge rates from proposed development sites;
- Methods used to estimate the amount of storage required;
- Responsibilities for the maintenance of runoff control measures.

A summary of the questionnaire and its findings is given in Chapter 4 of this document. A review has also been undertaken of the methods currently available to estimate the runoff from greenfield sites. This review is given in Chapter 5.

### 2.2 The influence and effectiveness of limiting discharge rates and the use of storage

In order to assess the influence and effectiveness of limiting discharges on storage requirements, ten catchments were selected throughout England and Wales. The potential development sites were assumed to be situated at the sites of rain gauges with extensive recorded data sets that were also located adjacent to rivers where a flow gauge existed. A Hydroworks drainage model representing a 10ha development in each of these catchments was built. The Hydroworks model was used to assess the actual retention volumes required to attenuate flows from the 10ha development sites for a range of throttle limits from 1 l/s/ha to 15 l/s/ha. Runoff was generated using both observed rainfall intensity data and rainfall design events. Autographic rainfall recorded at a rain gauge (tip time data) in each of the catchments was used in conjunction with the available flow data in the adjacent river. The ten theoretical development sites were chosen on the basis of the quantity and quality of flow and autographic rainfall data that were available. After analysis of over 80 data sets of rainfall and river flow, the following ten sites were selected:

- Stoneleigh on the River Sowe;
- Wingfield on the River Amber;
- Drakelow Park on the River Trent;
- Little Driffield on the Elmswell Brook;
- Leeming on Bedale Beck;
- Farnhaxn on the River Wey;
- Albury on Law Brook;
- Ashford Mill on the Isle;
- Kates Bridge on the Glen;
- Fotheringhay on Willow Brook;
- Wennington Bridge on the River Wenning.

The results of the analyses carried out for these catchments are included in Chapters 7 and 8 of this document.

## 2.3 The potential of storage for initial runoff interception

The initial runoff from urban areas caused by rainfall after a prolonged dry spell can, in many instances, have a high pollutant load. Small rainfall events on rural catchments initially generate negligible runoff and generally have no polluting effect.

The use of interception storage was therefore investigated to assess the storage volumes that would be required and also assess the reduction in the number of events that would discharge to the river.

## 2.4 Testing the proposed procedure

Having defined a philosophy, two examples were used to both test the procedure and also illustrate the analysis process to achieve the derivation of the site storage needs. Although SUDS is not explicitly utilised in the solutions, the examples demonstrate how important the consideration of SUDS techniques is to meet the objectives of the proposed methodology.

# 3. THE CURRENT STATUS OF DEVELOPMENT DRAINAGE

## 3.1 Background

Urban runoff control measures need to take into account both technical and legislative constraints. Planning permission for new developments in England and Wales is regulated by the Town and Country Planning Act. This Chapter discusses the following:

- Regulatory authorities responsible for defining the runoff control measures to be employed;
- An overview of the legislative framework and adoption issues;
- The use of sustainable urban drainage systems;

A review of the commonly used runoff control measures currently being employed.

## 3.2 Regulatory authorities

There are currently five organisations in England and Wales that have a role in dictating the type and size of runoff control measures used by developers. These are:

- Environment Agency;
- Sewerage undertakers;
- Local authorities;
- Internal Drainage Boards;
- British Waterways.

The organisation involved in the approval of runoff control measures is dependent upon the classification of the water receiving the discharge. The roles of the various organisations are discussed briefly below.

### 3.2.1 Environment Agency

The Environment Agency is responsible for granting discharge consents for all watercourses designated as "Main Rivers". Watercourses designated as "Main Rivers" are detailed on a map produced by Ministry of Agriculture Fisheries and Food (MAFF) under the Land Drainage Act 1976. On non-main rivers (ordinary watercourses) the Environment Agency's consenting powers are effectively limited to approving the construction of dams and culverts outside Internal Drainage Districts. Before a developer can discharge runoff from a development site to a Main river, Environment Agency approval must be obtained. In many cases the Environment Agency require that some form of runoff control measure is used to attenuate the runoff from the developed site. Although not responsible for consenting discharges to ordinary watercourses, the local authority is required to take account of the Environment Agency's views when considering the issue of discharge consent.

### 3.2.2 Sewerage undertakers

The statutory sewerage undertaker has a responsibility to provide an effective sewerage system for the area that it serves. This includes the prevention of flooding and meeting Environment Agency water quality standards for outflows to watercourses. Where developers discharge runoff from a development to sewers, the degree of attenuation required is dictated by the sewerage undertaker. The criterion usually applied is that the future performance of the sewerage system should not be compromised by the proposed development. Often developers are required to construct attenuation tanks to limit the discharge from their site to a stated maximum rate. These tanks are usually adopted by the sewerage undertaker and are an integral part of the sewerage system.

### 3.2.3 Local authorities

Ordinary watercourses usually fall under the auspices of local authorities. In many cases similar criteria to those applied by the Environment Agency are enforced before a developer is granted a consent to discharge runoff from a new development. The Environment agency is a statutory consultee when local authorities are considering a planning application under the Town and Country Planning Act. It should be noted that in some cases County councils may have regulatory control by default when the local powers have not been exercised by the relevant local authority. The local authority is also the highway authority for non-trunk roads. Highway drainage does not require a consent to discharge in England and Wales.

### 3.2.4 Internal Drainage Boards

Internal Drainage Boards are independent statutory public bodies that are established in the most drainage dependent areas of England and Wales. The districts that Internal Drainage Boards administer are areas defined as those benefiting by drainage or avoiding danger from flooding. In practice this means that Internal Drainage Boards are found in the lowland flat areas of England such as Somerset and the Fens. Internal Drainage Boards exercise similar powers of consent to those of the Environment Agency on Main rivers. Should a developer wish to discharge to a watercourse or drainage channel under the authority of an Internal Drainage Board a discharge consent is required.

### 3.2.5 British Waterways

Navigable canals designated as waterways generally fall under the control of the British Waterways, although local trusts or the National Trust may manage some canals. The British Waterways is not obliged to accept surface water drainage. Any discharges that a developer may wish to make to the waterway network must be made under a licence, which is commercially negotiated and may include a premium, as well as annual payments.

## 3.3 Legislative framework and adoption issues

The legislative framework for the control of urban runoff from sites is complex and only a brief overview is given here. The local authorities have the responsibility to examine developers' planning applications for both strategic and technical competence. Local authorities are also the highway authority for non-trunk roads. The local authority will grant or refuse planning permission in accordance with the drainage advice it receives from statutory consultees. Runoff from development sites is generally discharged to a public sewer or a watercourse. The requirements in terms of runoff control measures that must be provided for each of these is discussed below.

### 3.3.1 Public sewers

In order to discharge development runoff to a public sewer a consent is required from the relevant sewerage undertaker. Runoff control measures are often provided in the form of an attenuation tank.

The issue of adoption of drainage systems of developments as far as public sewers are concerned is covered in the document entitled Sewers for Adoption. This document clearly defines the ownership and maintenance of conventional sewerage systems. Sewerage undertakers generally have a policy of accepting both off-line and on-line underground storage. However, at present, the majority of sewerage undertakers in England and Wales will not adopt surface ponds. The view of many sewerage undertakers is that these structures do not come within the remit of the Water Resources Act 1991 and that therefore they are not obliged to adopt them. The reason for taking this position is also linked to the fact that maintenance of ponds is more labour intensive than for underground storage. However, there are moves to try and encourage sewerage undertakers to soften this position and a range of options are being considered to encourage the greater use of ponds in developments due to their environmental advantages. Some sewerage undertakers will accept soakaways as long as they are linked by a pipe system, normally at high level, and that the final length of pipe from the last soakaway discharges to a watercourse.

### 3.3.2 Watercourses

As outlined at the beginning of Section 3.2 there are four regulatory organisations that are responsible for the control of discharges to watercourses. The most important of these organisations is the Environment Agency who are effectively involved in regulating most of the watercourses in the England and Wales.

The Environment Agency is made up of eight regions. Environment Agency's criteria for runoff control measures from development sites vary from region to region. This is discussed in Chapter 4. The Environment Agency does not adopt runoff control measures such as surface ponds or infiltration trenches that are constructed by developers. The onus is on developers to operate and maintain such features, or passing the responsibility to the local authority or sewerage undertaker.

## 3.4 Sustainable Urban Drainage Systems (SUDS)

Sustainable Urban Drainage is the practice of managing surface water runoff from a point as close to its origin as possible, with a train of techniques before it enters the receiving watercourse or piped drainage system. These comprise a range of different types of structures all of which aim to reduce and/or attenuate rainfall-runoff. One of the main problems with Sustainable Urban Drainage Systems (SUDS) is that under the current legislative framework they are often considered to be landscape features, rather than an integral part of the sewerage system. As a consequence there are legal difficulties for transferring responsibilities for the operation and maintenance of SUDS to any organisation.

At present developers are reluctant to construct SUDS because, without an adoption agreement with the sewerage undertaker or local authority, the operation and maintenance of the SUDS remains the developers' responsibility. A second reason for not building SUDS is the lack of accepted design criteria and proof of long-term adequacy of performance. This opens up the possibility of future failure and risk of litigation and having to renovate or put in an alternative system.

## 3.5 Brownfield sites

All brownfield development sites have site specific requirements that need to be discussed with planners and relevant authorities. In principle any development should be attempting to replicate the greenfield response that would have taken place from the site. However, the environmental benefits of redevelopment of a site compared to the loss of more greenfield areas must be taken into account when considering brownfield development. This means that criteria is usually relaxed to minimise the cost of developing a brownfield site.

In addition, land contamination issues will constrain the options available for use in draining the site if the site is polluted.

## 3.6 Highway drainage

Information on controlling runoff from highways can be found in manuals published by the Highways Agency. Runoff control measures for highways are outside the scope of this document.

# 4. CURRENT REQUIREMENTS FOR STORAGE SPECIFICATIONS AND RUNOFF LIMITS

## 4.1 Background to current practice

It is an essential requirement when studying the development potential of a site for the developer to be able to satisfy the relevant planning authority that the runoff from the site can be suitably disposed of. The most commonly used options for disposal of site runoff are:

- Connection to the sewerage system;
- Discharge to a nearby watercourse;
- Infiltration into the ground.

There is general agreement amongst the Environment Agency and sewerage undertakers that the majority of large developments on greenfield sites require some form of runoff control and storage. The main objective of runoff control is flood defence, although water quality improvements, ecological enhancements and maintenance of the groundwater regime are also important considerations. The regulating authority generally makes decisions as to the size and form of this storage on a site-specific basis. In some cases the regulating authority may state that storage can be avoided if the development area is small (e.g. urban infill) or there is currently sufficient capacity in the receiving system so that there is no additional risk of flooding created by the development.

The current storage techniques to control runoff from newly developed sites generally fall into three categories. These are:

- Open ponds;
- Storage tanks;
- Oversized pipes.

Sustainable Urban Drainage Systems, which provide options for limiting the rate and amount of runoff, are currently not widely used, owing to problems with adoption of these schemes and confidence in their long term performance. Limiting the rate of runoff to a sewer via a method that is not an integral part of the sewerage system (i.e. through the use of infiltration) is also rare owing to problems of split ownership. The relevant authority generally sets the rate to which the runoff is limited from a newly developed site. As outlined in Chapter 3 there are four main organisations that have regulatory powers over the runoff rate from developed sites. The organisation with which a developer has to liaise is dependent upon the nature and type of the receiving waters to which the runoff is being discharged.

The current requirements of the Environment Agency and the sewerage undertaker regarding the control of runoff from newly developed sites are detailed below. The experience of developers and house builders with regard to the provision of runoff control is also briefly discussed.

## 4.2 Review of the current runoff control measures

### 4.2.1 Background to sizing runoff control measures

When sizing runoff control systems for new developments four, basic parameters are required to be established. These are:

- The runoff rate and volume that will reach the control system;
- The return period for which the runoff control system is required to operate;
- The design outflow rate from the runoff control device;
- The volume of storage needed to attenuate the runoff from the site to the required outflow rate.

The volume of runoff from the development site for a particular return period is a function of the topography and climate of the site. The maximum return period for which the runoff control system is required to operate effectively is normally dictated by the regulatory authority. The design outflow rate from the runoff control system is also usually set by the regulatory authority and is some times a function of the capacity of the downstream system. The maximum permitted design outflow rate from a runoff control system can be specified in four main ways as follows:

1. An arbitrary limiting flow rate;
2. An arbitrary limiting flow rate per hectare of the catchment drained;
3. A variable flow rate dependent on storm durations and return periods;
4. A flow rate based on a calculation of the greenfield runoff from the site for a specified return period or range of return periods.

The first three approaches are not commonly used because an understanding of the entire catchment and its associated urban drainage system is required. The use of a fixed flow rate based on the undeveloped site's greenfield runoff rate for a one year return period event is the one most widely used.

### 4.2.2 Runoff control measures

For completeness an overview of all runoff control measures is outlined. The various measures that are available for use to control runoff from development sites are shown in Figure 4.1 below.

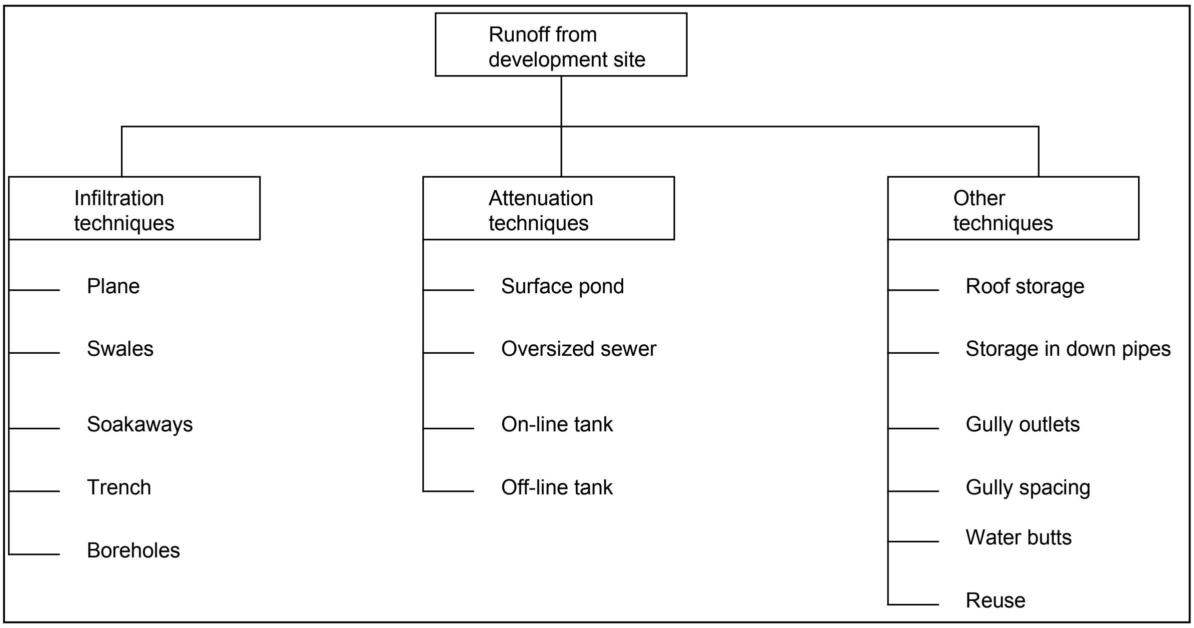



**Figure 4.1 Runoff control measure options**

A brief review of these methods is given below:

**Infiltration techniques**

(i) Plane infiltration generally comprises simple grassed or open-textured surfaces, cellular soil-filled concrete block pavements and porous pavements. To work efficiently these methods require the rate of infiltration to be greater than the design rainfall. These methods are typically used for car parks.
(ii) Basin infiltration consists of grassed swales or dry ponds. The design requirements for sizing a dry pond are similar to those for an attenuation pond.
(iii) Soakaways are commonly used in the UK. They are often built of dry brick work, concrete rings or simple rubble filled excavations.

(iv) Trench infiltration often comprises a porous pipe distribution system within a stone-filled trench.
(v) Boreholes are not commonly used as an infiltration technique in the UK.

**Attenuation techniques**

(i) Surface ponds are often incorporated as a landscaped feature in developments to attenuate and partially treat runoff from developments before it is discharged to a watercourse.
(ii) Oversized sewers or tank sewers are used where surface flooding is restricted and the sewerage system has limited spare capacity. These are commonly used in the UK.
(iii) On-line tanks are constructed as an integral part of the sewerage system and comprise an enlarged flow section that fills when the inflow exceeds the maximum design outflow. These are commonly used in the UK.
(iv) Off-line tanks are physically separated from the basic sewerage system, utilising overflow structures and return flow mechanisms.

**Other techniques**

All of these techniques have value, but, to date, the on-line tank sewer is the option most commonly used for drainage runoff attenuation design.

## 4.3 Environment Agency requirements

The Environment Agency is split into eight separate geographical regions. All have a range of different requirements for the control of runoff from developed sites. These regions are shown in Figure 4.2 below.

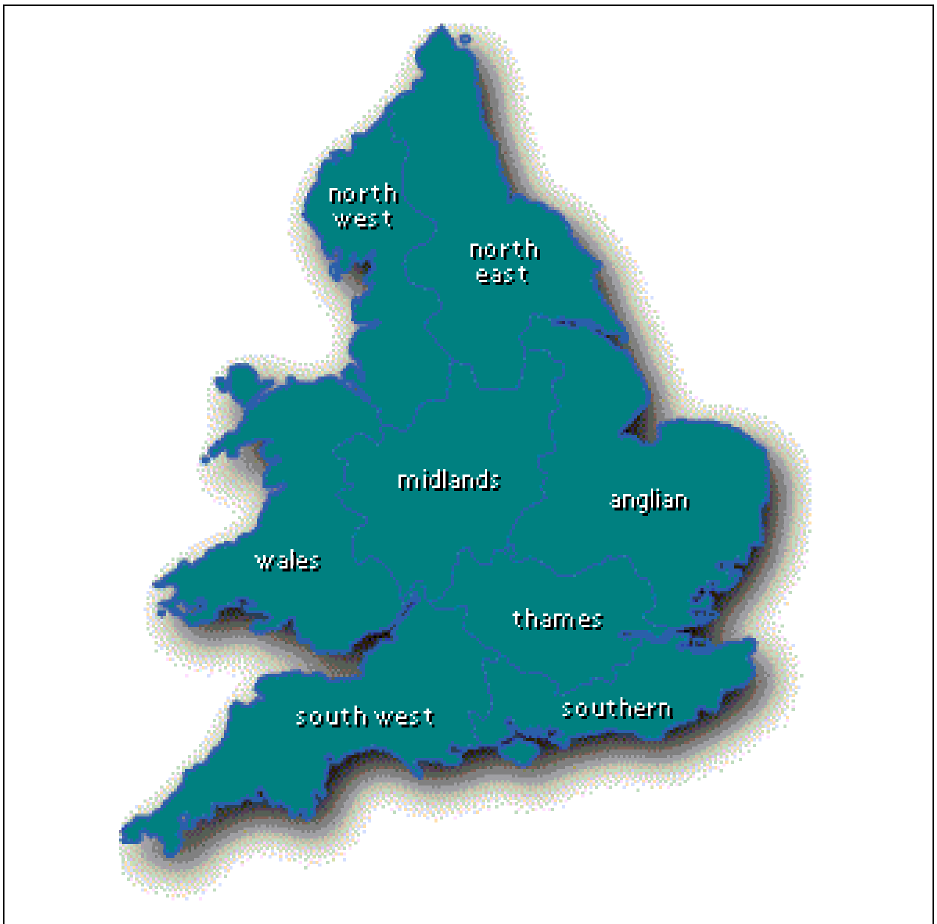



**Figure 4.2 Environment Agency regions**

The majority of Environment Agency regions have maximum allowable runoff values from developed sites, which are usually in the region of 5 l/s/ha to 8 l/s/ha. The recommended method for calculating the storage required to control runoff from new developments also varies considerably. Currently the Environment Agency does not have a nationally consistent method to control runoff from newly developed sites. Within the eight Environment Agency regions, even Area offices apply different criteria for control and storage of runoff.

The main research document SR580 details the results of the questionnaire and provides an overview of the various methods recommended by the eight Environment Agency Regions together with their Area offices. A more extensive description of the questionnaire results is given in report EX 4214 produced by HR Wallingford in May 2000.

## 4.4 Requirements of sewerage undertakers

Sewerage undertakers often require some form of attenuation before runoff flows from a developed site is discharged to a sewer. The size of the attenuation tank provided by the developer is normally dictated by the capacity of the sewerage system and the criterion that the risk of flooding from sewers should not be increased. The requirements of sewerage undertakers for attenuation are usually not as onerous as detention requirements required by the Environment Agency. In most cases the sewerage undertaker requires the 1 in 30 year runoff from the developed site to be attenuated to a rate which can be accommodated by the existing receiving sewer.

## 4.5 House builders and developers experience

House builders and developers experiences in dealing with surface water runoff from new developments is a function of the requirements dictated by the Environment Agency or sewerage undertakers, depending on whether the runoff is to be discharged to a watercourse or sewer. The guidance given to developers by the Environment Agency varies depending upon the location, nature and size of the site. The majority of developers control runoff from newly developed sites through tanks, ponds or oversized pipes. House builders rarely employed the use of Sustainable Urban Drainage System techniques. A consistent message from the questionnaires was that development proposals involved discussions on criteria that differed all over the country. The minimum size of development for which runoff control was required also varied greatly.

House builders use several methods with which to assess the pre- and post- development runoff, as well as storage volumes. Computer packages such as Hydroworks, Microdrainage, or rainfall profiles specified by the Environment Agency are all widely used. Most house builders and developers discuss storage criteria for the site with the local Environment Agency office. However, many developers believe that requirements in many Environment Agency Regions to attenuate the 1 in 100 year development runoff to a 1 in 1 year greenfield runoff leads to balancing ponds and tanks that are significantly oversized. Some developers also believe that owing to lack of investment in the sewerage system they have to provide oversized balancing on site. In some cases developers have stated that they would have preferred to improve the sewerage system as an alternative to constructing storage tanks.

Most developers do not have an explicit policy with regards to runoff control from developments. Policies that do exist include minimising the size and the cost of the storage required and making greater use of plot storage to attempt to keep on-line storage to a minimum. In general the type of runoff control implemented by the developers is dictated by the requirements of the regulating authority.

# 5. REVIEW OF THE COMMONLY USED METHODS FOR ESTIMATING GREENFIELD RUNOFF RATES

## 5.1 Background

There are numerous hydrological techniques currently in use to estimate greenfield runoff rates. This section briefly reviews the techniques that are most relevant for calculating greenfield runoff rates from sites that are to be developed. The methods that have been reviewed include:

- Agricultural Development and Advisory Service (ADAS) Report 345 (1982);
- Rational Method (attributed to Mulvaney in 1850);
- Prudhoe and Young Transport and Road Research Laboratory Method (TRRL Report LR 565, 1973);
- Flood Studies Report, statistical and rainfall runoff methods (National Environment Research Council
- 1975);
- Flood estimation for small catchments, detailed in the Institute of Hydrology report no. 124 (Institute of Hydrology 1994);
- Flood Estimation Handbook (Institute of Hydrology 1999).

A distinction on catchment size can be made with regard to the above techniques. The methods detailed in the Flood Studies Report and the Flood Estimation Handbook are applicable to a wide range of catchment areas ranging from 0.5 $km^2$ to 5000 $km^2$. The other methods listed above should only be applied to estimate the flows from small, rural, ungauged catchments (i.e. generally catchments with areas of 20 $km^2$ or less). Estimation techniques for predicting runoff and flows for small catchments are notably inaccurate and this also applies for small greenfield sites which only form a part of a catchment.

Details of these various techniques commonly used in the UK are outlined below. A comparison of these techniques has been made and the most suitable methods for estimating greenfield runoff rates in the UK are recommended for various situations. The importance of these calculations is due to the fact that the estimation of the greenfield runoff rate for a given return period is often the basis for the outflow rate for the runoff control measure.

## 5.2 Agricultural Development and Advisory Service (ADAS) Report 345 technique

### 5.2.1 Background to the ADAS method

The Agricultural and Development Advisory (ADAS) report number 345 details a technique which is primarily aimed at providing information to determine the size of pipes required for field drainage systems. This method is reported more fully as MAFF report No. 5.

The equation to estimate runoff from a site is of the form:

$Q = S_T F A$

Where:

$Q$ is the 1 year peak flow in l/s.

$S_T$ is the soil type factor which ranges between 0.1 for a very permeable soil to 1.3 for an impermeable soil.

$F$ is a factor which is a function of the following catchment characteristics: average slope; maximum drainage length; average annual rainfall. The F number can be estimated from a nomograph included in the ADAS report.

$A$ is the area of the catchment being drained in hectares.

It should be noted that the “grasslands” line in the nomograph (in MAFF Report No 5) represents the 1 in 1 year flow, but the two other lines (arable and horticulture) refer to the 1:5 and 1:10 year event flow rates. These other higher return period curves are not used in Report 345. Guidance on the values of the above variables is given in the ADAS report, together with a nomograph which can be used to estimate the flow. This method should not be used for catchments that exceed 30ha. This catchment size is given as the limit in the document. Flow rates for other return periods can be calculated using the appropriate Flood Studies Report regional growth curves.

### 5.2.2 Advantages and disadvantages of the ADAS method

The method for calculating flows in the ADAS Report No.345 has the following advantages:

- It provides an easily applied cheap, simple and quick method for calculating peak flows from small, rural catchments;
- The method is based on measurements taken on a number of small, rural catchments;
- It requires relatively few variables that can be estimated from Ordnance Survey mapping.

However, the disadvantages of the method are as follows:

- The method should only be applied to catchments which are less than 30ha;
- The method is based on research carried out on a relatively limited number of catchments.
- It is acknowledged that saturated greenfield runoff where the land is not drained can have higher rates of runoff than that predicted by this method.

## 5.3 The Rational Method

### 5.3.1 Background to the Rational Method

The Rational Method, which is some times referred to as the Lloyd-Davies method or Kuichling formula, in its simplest form can be used to estimate peak flows from any catchment. The concept of the Rational Method for determining peak flood flows is based on work first carried out by the Irish engineer Mulvaney in 1850. The Rational Method formula, in its metric form, is as follows:

$Q = 0.278CiA$

Where:

Q is the peak flow in $m^3/s$
C is a non-dimensional runoff coefficient which is dependent on the catchment characteristics. C can vary from around 0.05 for flat sandy catchments to close to 1 for steep slopes and paved areas
i is the rainfall intensity in mm/hour for the time of concentration, Tc, of the catchment
A is the area of the catchment in $km^2$

The time of concentration Tc is defined as the time required for rain falling at the farthest point of the catchment to flow to the catchment outlet where the peak flow is to be calculated. The Rational Method assumes that the rainfall intensity “i" is constant during the time of concentration Tc. and that all the rainfall falling over the catchment contributes to the flow. The rainfall intensity should therefore be calculated for the estimated time of concentration for the required return period from rainfall-duration-frequency curves. These curves can be generated using the Flood Studies Report or from gauged autographic rainfall data. Two methods commonly in use for estimating times of concentration are:

- Kirpich method;
- Bransby-Williams method.

Kirpich's and Bransby-William's methods for calculating the time of concentration are empirical and based on experiments carried out on catchments in the USA and India respectively. The two methods are briefly discussed below.

### 5.3.2 Kirpich's method for estimating time of concentration

The value of the peak flow estimated by the Rational Method is heavily dependent upon the method used to estimate the time of concentration for the catchment. For small natural catchments, a formula derived from data published by Kirpich in 1940 for agricultural areas in the USA, can be used to estimate the time of concentration Tc in hours. This formula is given below:

$$Tc = 0.00025(L/S^{0.5})^{0.8}$$

Where:

L is the length of the catchment along the longest drainage path in metres

S is the overall slope of the catchment in m/m

Kirpich's formula for Tc generally underestimates times of concentrations for catchments and as a consequence values of rainfall intensity are generally overestimated leading to conservatively high peak discharges. Kirpich's formula should only be used as a last resort as in general this method will provide results that are less accurate than more recently developed methods.

### 5.3.3 Bransby-Williams method for estimating the time of concentration

The Bransby-Williams formula is occasionally used in the UK to estimate the time of concentration in hours. This formula is of the following form:

$$Tc = (L/d)(a^2/h)^{0.2}$$

Where:

L is the greatest distance from the edge of the catchment to the outlet at which the discharge is being estimated.
d is the diameter of a circle with an area equal to that of the catchment.
a the catchment area in square miles.
h is the drainage channel slope, as a percentage, along its greatest length.

The Bransby-Williams formula was developed to estimate floods in India and, although applied in the UK, its use is not recommended because it yields times of concentration that are too short. This results in conservatively high peak discharges.

There are several other empirical formulae that have been developed to estimate the time of concentration for rural catchments. However, these formulae are generally only applicable for particular areas of the USA.

### 5.3.4 Advantages and disadvantages of the Rational Method

The Rational Method remains a widely used method for calculating peak flows for small catchments. It has several advantages including:

- It is simple and easy to use;
- It requires relatively few variables.

The Rational Method has several disadvantages. These include:

- The method generally leads to conservative estimates of flows compared with other methods owing to the fact that the time of concentration for the catchment is often underestimated;
- Considerable knowledge of the catchment is needed to estimate a representative value of the runoff coefficient C. The runoff coefficient C also varies for storms of different durations on the same catchment. Using an average value of C only allows a crude estimate of the peak flow which can have a wide margin of error;
- Rainfall intensity duration frequency curves have to be established either from autographic rain gauges or using the Flood Studies Report;
- The Rational Method takes no account of any routing effects;
- The Rational Method is generally less accurate than more recently developed methods.

To conclude, it is recommended that methods other than the Rational Method are used to generate peak discharges for rural catchments. When used with times of concentration generated by the Kirpich or Bransby-Williams equations, this method tends to produce conservatively high peak discharges.

## 5.4 Prudhoe and Young Transport and Road Research Laboratory Report LR 565 method

### 5.4.1 Background to the Prudhoe and Young method

This method is based on the hydrological behaviour of small natural catchments bordering motorways. A multiple regression analysis was carried out based on five experimental catchments located in various parts of England. It should be noted that the percentage runoff is given by a regression equation on average annual rainfall. No account is taken of different soil types and the method is only valid for clay or boulder clay soils. The peak flow for a specific return period is calculated as follows:

The catchment length L in km together with the rise Z in km, from the catchment outfall to the average height of the catchment, is measured from maps. The slope number N is calculated from the equation below:

$$N = \frac{L}{Z}$$

The time of concentration in hours is calculated from:

$$Tc = 2.48(LN)^{0.39}$$

The runoff factor is calculated from the following equation:

$$F_A = 0.00127R_A - 0.321$$

Where:

$R_A$ is the average annual rainfall in mm

The rainfall depth R for a specific return period is given by:

$$\frac{10}{\text{Return period}} = 1.25\,Tc\left(0.0394R + 0.1\right)^{-3.55}$$

The flow generated for a specific return period in $m^3/s$ is calculated from the equation.

$$Q = \frac{F_A AR}{3.6Tc}$$

Where:

Q is the discharge in $m^3/s$
A is the catchment area in $km^2$

### 5.4.2 Advantages and disadvantages of the Prudhoe and Young method

The Prudhoe and Young method has the following advantages:

- This method is quick and relatively easy to apply;
- The method has been developed from data collected from five experimental catchments in the UK;
- The Prudhoe and Young method gives reasonable results for small catchments with predominantly clay soils.

The disadvantages of this method are as follows:

- The method tends to be less accurate for estimating peak flows from upland catchments;
- The method tends to overestimate flows from catchments with predominantly permeable soils and underestimate flows on catchments with shallow soils which overlie rock;
- Prudhoe and Young only carried out a limited amount of research on a relatively small number of catchments.

## 5.5 Flood Studies Report; statistical and rainfall-runoff methods

### 5.5.1 Background

The Flood Studies Report gives a range of methods for predicting flows from ungauged rural catchments in the UK. The Flood Studies Report was originally published in 1975. However, later supplements to the report extended and updated flood flow predictions for catchments with a significant degree of urbanisation (Flood Studies Supplementary Report FSSR No. 5), small catchments (FSSR No. 6), together with updating the Flood Studies Report rainfall-runoff model parameter estimation equations (FSSR No. 16).

The methods detailed in the Flood Studies Report for estimating peak flows and hydrographs for ungauged rural catchments fall into two main categories:

- Statistical methods using linear regression equations and regional growth curves;
- Rainfall-runoff methods.

These methods are detailed below.

### 5.5.2 Flood Studies Report statistical approach

This method allows estimates of peak discharges to be made for ungauged catchments. The method is reliant upon calculating the mean annual flood, $Q_{BAR}$, from which flood discharges for various return periods can be calculated. The recommended general equation to estimate $Q_{BAR}$ in the Flood Studies Report is:

$$Q_{BAR} = C[AREA^{0.94} STMFRQ^{0.27} SOIL^{1.23} RSMD^{1.03} S1085^{0.16} (1+LAKE)^{-0.85}]$$

Where:

AREA is the catchment area in $km^2$.
STMFRQ is the stream frequency in terms of junctions/$km^2$.
RSMD is the net one day rainfall with a five year return period minus the soil moisture deficit.
LAKE is the fraction of the catchment draining through a lake or a reservoir.

S1085 is the stream in m/km measured at distances that are 10 percent and 85 percent of the stream length as measured from the catchment outlet.
C is a regional coefficient.
SOIL is the soil index is a composite index determined from soil survey maps which accompany the Flood Studies Report and is derived from the formula below:

$$\mathrm{SOIL} = \frac{(0.15S_1 + 0.30S_2 + 040S_3 + 0.45S_4 + 0.5S_5)}{S_1 + S_2 + S_3 + S_4 + S_5}$$

Where $S_1 + S_2 + S_3 + S_4 + S_5$ denote the proportions of the catchment covered by each of the soil classes 1 to 5. Soil class 1 has the highest infiltration capacity and hence the lowest runoff potential, while soil class 5 has the lowest infiltration capacity and hence the highest runoff potential. SOIL has a range of possible values between 0.15 and 0.50.

The various catchment descriptors used in the equations are derived manually from 1:25,000 scale Ordnance Survey maps and maps which accompany the Flood Studies Report.

For the Essex, Lee and Thames area only, a different equation was derived. This is:

$Q_{BAR} = O.373AREA^{0.7}STMFRQ^{0.52}(l+URBAN)^{2.5}$

Where:

URBAN is the proportion of the built up area in the catchment.

The estimates of $Q_{BAR}$ are applied to the relevant regional growth curves to estimate the discharge for the relevant return period. The regional growth factor that is applied to $Q_{BAR}$ is dependent upon the hydrometric area in which the catchment is located. The regional growth curve factors for the various regions and hydrometric areas are given in Table 5.1 below.

**Table 5.1 Regional growth curve factors**

| Region | | Hydrometric area | Return period (years) | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | 2 | 5 | 10 | 25 | 50 | 100 |
| NW | 1 | 1-16,88-97, 104-108 | 0.90 | 1.20 | 1.45 | 1.81 | 2.12 | 2.48 |
| | 2 | 17-21, 77-87 | 0.91 | 1.21 | 1.42 | 1.81 | 2.17 | 2.63 |
| | 3 | 22-27 | 0.94 | 1.25 | 1.45 | 1.70 | 1.90 | 2.08 |
| | 9 | 55-67, 102 | 0.93 | 1.21 | 1.42 | 1.71 | 1.94 | 2.18 |
| | 10 | 68-76 | 0.93 | 1.19 | 1.38 | 1.64 | 1.85 | 2.08 |
| SE | 4 | 28, 54 | 0.89 | 1.23 | 1.49 | 1.87 | 2.20 | 2.57 |
| | 5 | 29-35 | 0.89 | 1.29 | 1.65 | 2.25 | 2.83 | 3.56 |
| | 6/7 | 36-44, 101 | 0.88 | 1.28 | 1.62 | 2.14 | 2.62 | 3.19 |
| | 8 | 45-53 | 0.88 | 1.23 | 1.49 | 1.84 | 2.12 | 2.42 |

These values are perhaps more easily shown by the Flood Studies Report figure in a graphical form. This is shown as Figure 5.1.

The hydrometric regions are shown in Figure 5.2.

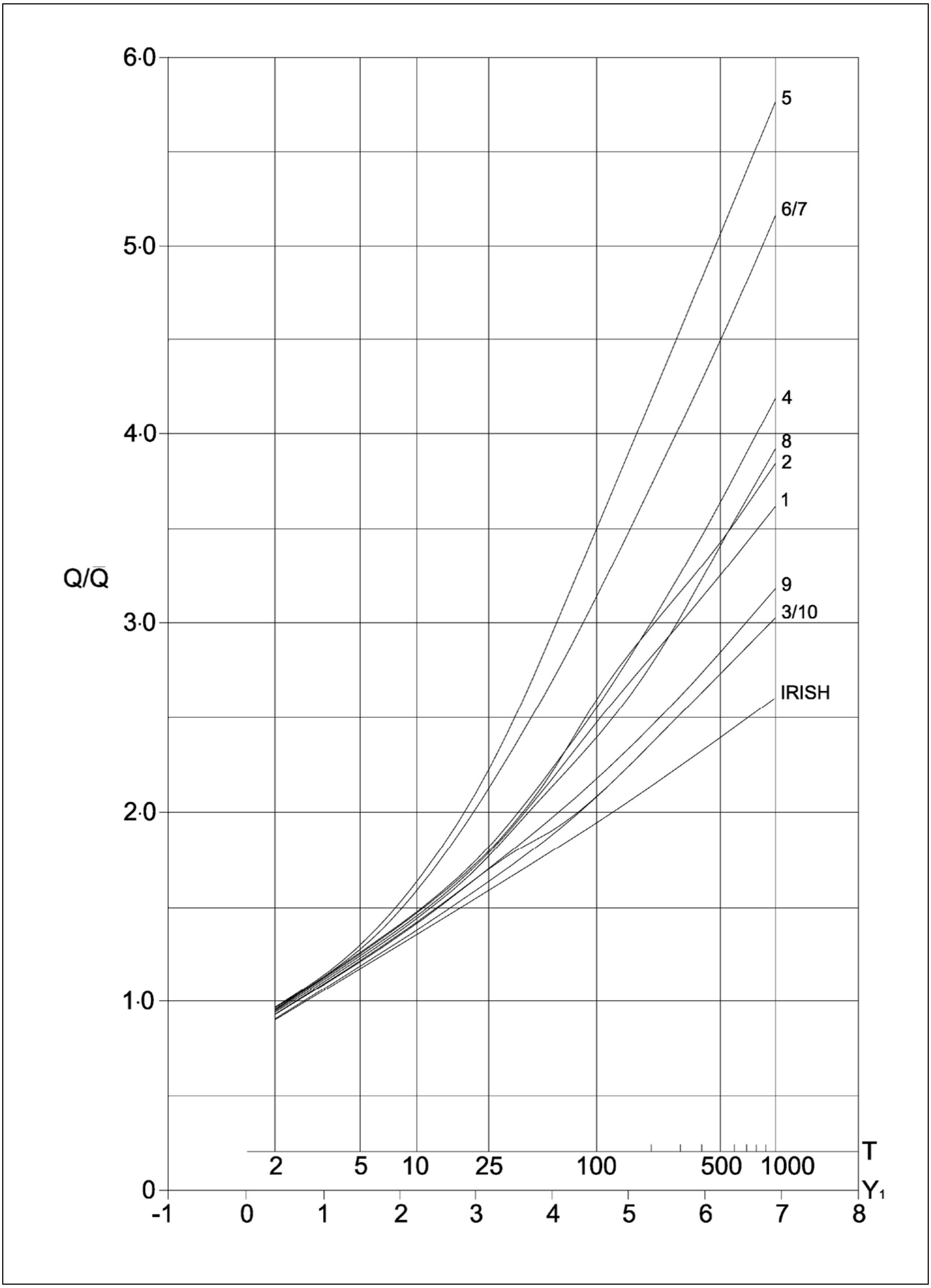



**Figure 5.1 Regional growth curve factor**

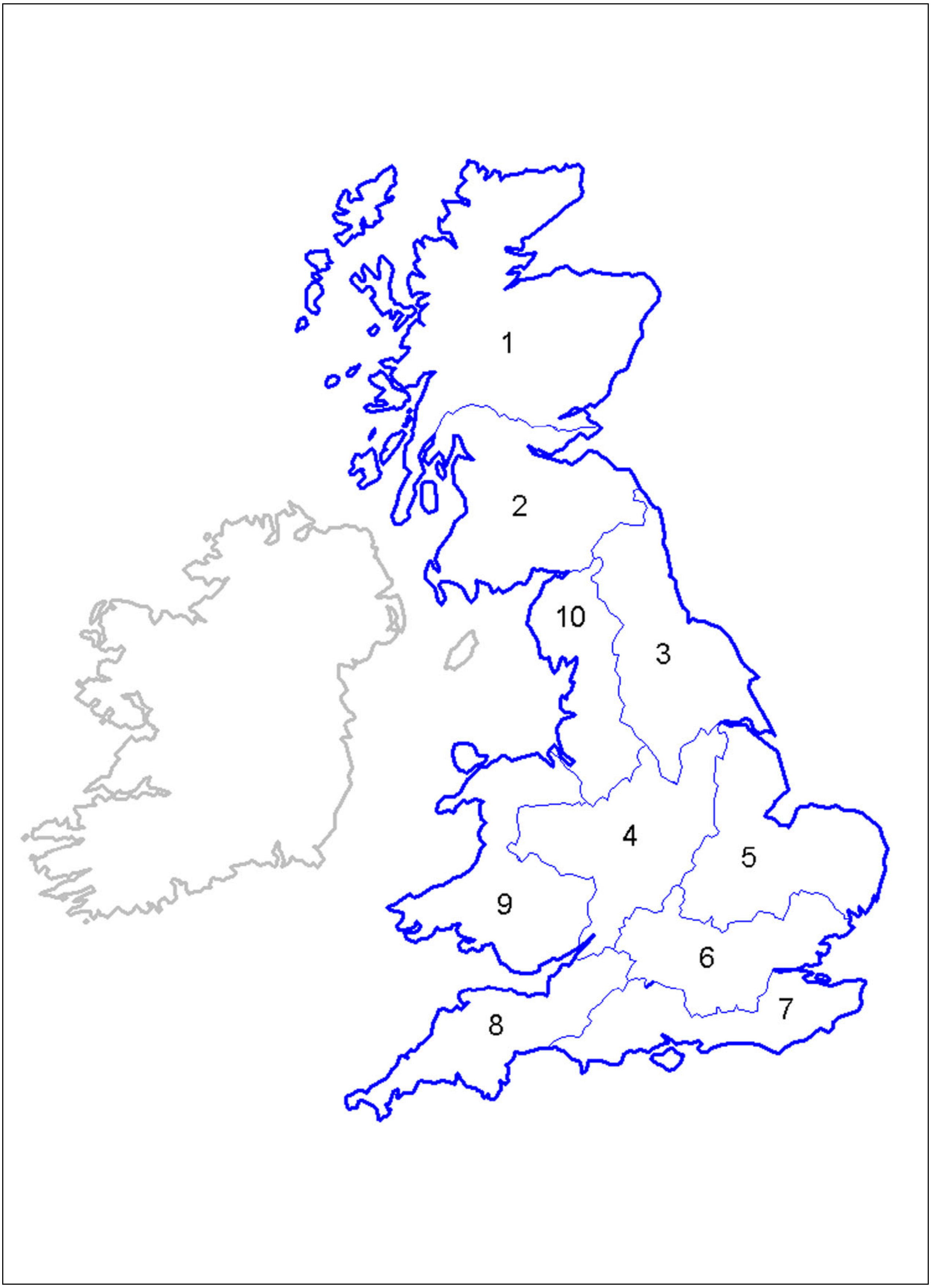



**Figure 5.2 Hydrometric regions of UK**

It should be noted that Flood Studies Supplementary Report No. 14 re-examined the growth curves and found that the country could be covered using only 2 curves, a South East curve and a North West curve.

However it recommended that for events with shorter return periods than 100 years, that the original 10 region curves should be used and only the 'pooled' curves used for more extreme events. It should be noted, in passing, that the Wallingford Procedure drainage software uses these 2 curves for all return periods.

To avoid discontinuities at 100 years, Flood Studies Supplementary Report No. 14 uses the 'pooled' curves to modify the original Flood Studies Report curves beyond the 100 year return period. As this is outside of the normal consideration of urban drainage analysis, this is not reproduced in this report. The determination of the 1 year factor is not possible from Figure 5.1. However a value of 0.7 is often used in the absence of more exact data analysis.

When the initial work for the Flood Studies Report was carried out, a limited number of small catchments (i.e. few catchments with an area less than 20 $km^2$) were considered. In 1978 the Flood Studies Report Supplementary Report No. 6 investigated flood prediction for small catchments i.e. catchments with an area of less than 20 $km^2$. This report concluded that for the Thames, Lee and Essex regions it was better to use the Flood Studies Report mean annual flood equation with the national multiplier.

It should also be noted that Poots and Cochrane studied 42 small catchments throughout the UK and concluded initially that the equation below gave marginally better results statistically than the Flood Studies Report. The Poots and Cochrane equation for $Q_{BAR}$ is of the form:

$$Q_{BAR} = 0.0136 AREA^{0.866} RSMD^{1.413} SOIL^{1.521}$$

The Institute of Hydrology carried out a similar exercise on small catchments, the results of which were published in 1994, which recommended that the equation developed under this research is used rather than the Poots and Cochrane equation. This work is discussed in Section 5.5.4 of this report.

**Advantages and disadvantages of the Flood Studies Report statistical technique**

The advantages of the Flood Studies Report statistical technique are as follows:

- The research carried out was based on a large number of gauged catchments throughout the UK;
- The use of Flood Studies techniques is the most common method of estimating flood flows in the UK;
- Allows a catchment wide estimate of the runoff to be made.

The disadvantages of the Flood Studies statistical technique are:

- Requires considerable knowledge of both hydrology and the Flood Studies Report to apply the techniques correctly;
- Requires defining the catchment and measuring the catchment characteristics from the 1:25,000 Ordnance Survey;
- The Flood Studies Report Statistical technique is regarded as being superseded by the Flood Estimation Handbook;
- The work carried out for the Flood Studies Report included relatively few small, rural catchments.

### 5.5.3 Flood Studies Report rainfall runoff method

The Flood Studies Report rainfall-runoff method uses four parameters to produce a synthetic design hydrograph for ungauged catchments. These are:

- Unit hydrograph;
- Percentage runoff;
- Baseflow;
- Rainfall profile used to produce a design hydrograph for a particular return period.

The unit hydrograph is used to model the transition of the excess rainfall into response runoff. Where less than three years of flow data are available, the synthetic triangular unit hydrograph should be used. The triangular unit hydrograph is defined by its time to peak which is usually calculated from the catchment characteristics from the equations given in Flood Studies Supplementary Report No. 16.

Once the unit hydrograph is defined, the return period of the design flood is chosen and a corresponding design storm assessed. Owing to varying storm profiles combining with differing catchment antecedent moisture conditions, design storms of a given return period do not produce floods having the same frequency of occurrence as that of the rainfall. However, the Flood Studies Report provides a relationship for a recommended storm return period to yield a flood peak for the required return period. The design storm duration is established using a regression equation based on the Standard Average Annual Rainfall (SAAR) and the time to peak of the unit hydrograph.

The design storm depth can be calculated from data and equations in the Flood Studies Report. The key variables include:

- SAAR which is the standard average annual rainfall for the period 1941 to 1970 in mm;
- M5-2 day rainfall which is the depth of rainfall falling in two days once every five years;
- r which is the ratio of the M5-60 minute rainfall to the M5-2 day rainfall.

The percentage runoff from the catchment under consideration is calculated from regression equations and is a function of the soil type, antecedent moisture conditions, the urban fraction of the catchment and the design storm.

A storm rainfall profile is then calculated to apply to the unit hydrograph. The Flood Studies Report compiled a family of storm profiles for different seasons from autographic rainfall records for a number of stations. The rainfall profile calculated is symmetrical. To calculate the effective rainfall, the percentage runoff is applied uniformly to the rainfall amounts throughout the storm. The synthetic unit hydrograph is then convoluted with the synthetic unit hydrograph to produce the design hydrograph. The Flood Studies Report rainfall-runoff method is fairly onerous and time consuming to carry out by hand. The majority of hydrologists use the Micro-FSR software package to implement this method. Micro-FSR is briefly described in Section 5.5.5.

**Advantages and disadvantages of the Flood Studies Report rainfall-runoff technique**

The advantages of the Flood Studies Report rainfall-runoff technique are as follows:

- The research carried out was based on a large number of gauged catchments throughout the UK;
- The use of Flood Studies techniques is the most common method of estimating flood flows in the UK;
- It allows the production of a design hydrograph.

The disadvantages of the Flood Studies rainfall-runoff technique are:

- It requires considerable knowledge of both hydrology and the Flood Studies Report to apply the techniques correctly;
- It requires the catchment and the catchment characteristics to be defined manually on the relevant 1:25,000 Ordnance Survey mapping;
- The work carried out for the Flood Studies Report included relatively few small, rural catchments.

### 5.5.4 Flood estimation for small catchments Institute of Hydrology report no.124

The Institute of Hydrology Report No. 124 was published in 1994 and describes research for flood estimation for small catchments. The research was based on 71 small rural catchments (i.e. catchments with an area of less than 25 $km^2$). A new regression equation was produced to calculate $Q_{BAR}$ the mean annual flood for small rural catchments. $Q_{BAR}$ is estimated from the three variable equation shown below:

$$Qbar_{rural}=0.00108AREA^{0.89}\ SAAR^{1.17}\ SOIL^{2.17}$$

Where:

| | |
|---|---|
| AREA | is the area of the catchment in $km^2$. |
| SAAR | is the standard average annual rainfall for the period 1941 to 1970 in mm. |
| SOIL | is the soil index, which is a composite index determined from soil survey maps which accompany the Flood Studies Report and is derived from the formula outlined in Section 5.5.2. |

The above equation should be used in preference to the equivalent equation detailed in Flood Studies Supplementary Report No. 6 for use on small rural catchments. The above equation can also be used as an alternative to the six variable equation in the Flood Studies Report.

The $Q_{BAR}$ can be factored by the UK Flood Studies Report regional growth curves to produce peak flood flows for a number of return periods.

**Advantages and disadvantages of the Institute of Hydrology 124 method**

The advantages of the Institute of Hydrology Report no 124 method are as follows:

- The research carried out to produce the three variable equation was based on small, rural catchments;
- The research was based on data from 71 small, rural catchments, significantly more than either the ADAS or TRRL research;
- Peak flows for various return periods can easily be calculated by applying the Flood Studies Report regional growth curves;
- The method is simple and relatively easy to use.

The disadvantages of the method are as follow:

- In theory the method should only be applied to a catchment drained by a well defined watercourse, not to a small greenfield site;
- It does not have a catchment slope component which on small steep sites is likely to limit its accuracy
- It advises that the formula should not be used for catchments less than 50ha.

### 5.5.5 Micro-FSR software

Micro-FSR is a computer package, produced by the Institute of Hydrology, for flood estimation in the UK. The Micro-FSR software incorporates the flood estimation techniques in the Flood Studies Report and the Flood Studies Supplementary Reports. Micro-FSR is currently widely used by hydrologists in the UK to implement the methods for flood estimation in the UK, especially for ungauged catchments. In 1999 the

Institute of Hydrology published the Flood Estimation Handbook which updates and supersedes the Flood Studies Report and the Flood Studies Supplementary Reports. The techniques employed by the Flood Estimation Handbook are discussed in Section 5.6. However, it should be noted that the Flood Estimation Handbook rainfall-runoff technique is broadly similar to the Flood Studies Report rainfall runoff technique. The latest version of MicroFSR can be used to calculate the Flood Estimation Handbook's rainfall-runoff method.

## 5.6 Flood Estimation Handbook

In 1999 the Institute of Hydrology produced the Flood Estimation Handbook. The methods of flood estimation detailed in the Flood Estimation Handbook are generally considered to supersede the Flood Studies Report and the Flood Studies Supplementary Reports as standard practice for rainfall and runoff analysis in the UK. The Flood Estimation Handbook provides two main approaches for flood frequency estimation. These are:

- Statistical methods;
- Flood Studies Report rainfall-runoff method.

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 details of the Flood Studies Report rainfall-runoff method and Micro-FSR have been discussed in Sections 5.5.4 and 5.5.5 respectively. 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 for small catchments.
The general philosophy behind flood frequency estimation in the FEH is as follows:

1. Flood frequency is best estimated from gauged data;
2. Where gauged data are not available, data transfers from nearby or similar catchments are useful;
3. 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 $km^2$ (50ha) or greater based on a 50m resolution DTM model. The lower limit reflects the facts 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.

### 5.6.1 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 to estimate the 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. It should be noted that this method should only be used as a last resort.

### 5.6.2 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.

### 5.6.3 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 (i.e. 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.

## 5.7 Comparison of methods for assessment of runoff rate

A comparison was made of the following greenfield runoff rate estimation methods:

- Agricultural and Development Advisory (ADAS) report number 345;
- Rational Method using Kirpich's time of concentration;
- Rational Method using Bransby-William' s time of concentration;
- Prudhoe and Young's Transport and Road Research Laboratory Report LR 565 method;
- Institute of Hydrology Report No. 124 flood Estimation For Small Catchments.

A theoretical greenfield site was used to compare the methods. The above five methods were initially compared with areas ranging from 10 ha to 100 ha. Details of this comparison are available in the main research document SR580. A direct comparison between all the methods is not easily made as the equations use a range of different parameters. In general terms the following results were found.

The Rational Method produces high greenfield runoff rates because Kirpich's and Bransby-Williams time of concentration equations significantly underestimate the time of concentration. It is therefore recommended that Rational Method is not used to estimate greenfield runoff rates.

A more detailed comparison was made between the ADAS, Prudhoe and Young and Institute of Hydrology 124 methods for catchment areas between lha and 100ha. The results indicate that the smaller the greenfield site, the greater the discrepancy between the three methods. For catchment areas of 100ha the difference between the three methods is of the order of 20% to 30%.

As stated in Section 5.4, the Prudhoe and Young method is based on research carried out on a limited number of catchtnents and the method tends to overestimate flows from catchments with predominantly permeable soils. For these reasons it is recommended that the Prudhoe and Young method is not used to estimate greenfield runoff rates. The differences between the ADAS and Institute of Hydrology 124 methods, depending on selection of catchment slopes, were found to be quite small.
It is clear that direct comparison of like with like is impossible owing to the various assumptions that have to be made. Therefore it is recommended to always check the predicted runoff values using both methods to assist in decision making and get an understanding of the relative differences in the predicted peak flow rates.

### 5.7.1 Recommendations and conclusions

There are a variety of techniques for estimating runoff rates from greenfield sites. The recommended methodology which should be applied for estimating greenfield runoff rates is given in Figure 5.3. The methodology is as follows:

- •Should a runoff rate be required for a sizeable catchment area (i.e. greater than 500 ha) rather than just a greenfield site itself, the Flood Estimation Handbook should be used;

- Where only the greenfield runoff rate is to be calculated for areas less than 500 ha, the Institute of Hydrology 124 method should be used unless the use of FEH is thought to be particularly appropriate for that location. For sites less than 30ha the Institute of Hydrology 124 method should be compared with the results produced by the ADAS method. To estimate runoff for return periods other than QBAR, the Flood Studies Report Regional growth curves should be applied to the mean annual runoff produced.

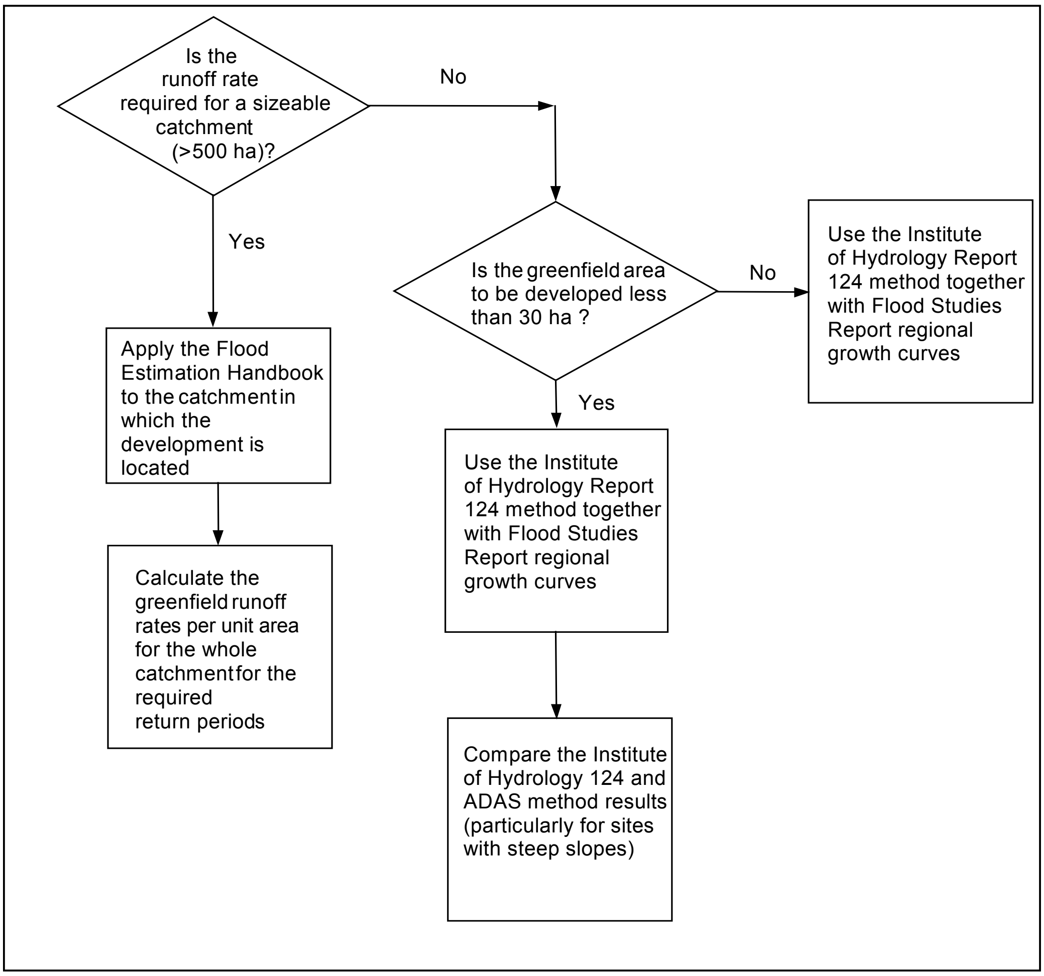



**Figure 5.3 Flow chart to establish which method to use to estimate greenfield runoff rates**

## 5.8 Calculation of greenfield runoff volume

In addition to attenuating the peak discharge from a development site, it is also important that there is some consideration given to the increase in runoff volume caused by the developed area. This requires an estimate to be made of the volume of runoff from the greenfield site. The method for estimating this volume is discussed below.

The greenfield runoff volume for a particular rainfall depth can be estimated by calculating the percentage runoff from the site. The volume of the runoff per unit area from the greenfield site can be calculated from the product of the rainfall depth and the percentage runoff. The percentage runoff from a greenfield site can be calculated using the technique detailed in Flood Studies Supplementary Report No 16. The percentage runoff for a rural area ($PR_{RURAL}$) is given by the following equation:

$$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$$

Where:

SPR is the standard percentage runoff which is a function of the five soil classes $S_1$ to $S_5$

$$SPR = 10S_1 + 30S_2 + 37S_3 + 47S_4 + 53S_5$$

$DPR_{CWI}$ is a 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. The relationship of CWI to SAAR is shown in Figure 5.4.

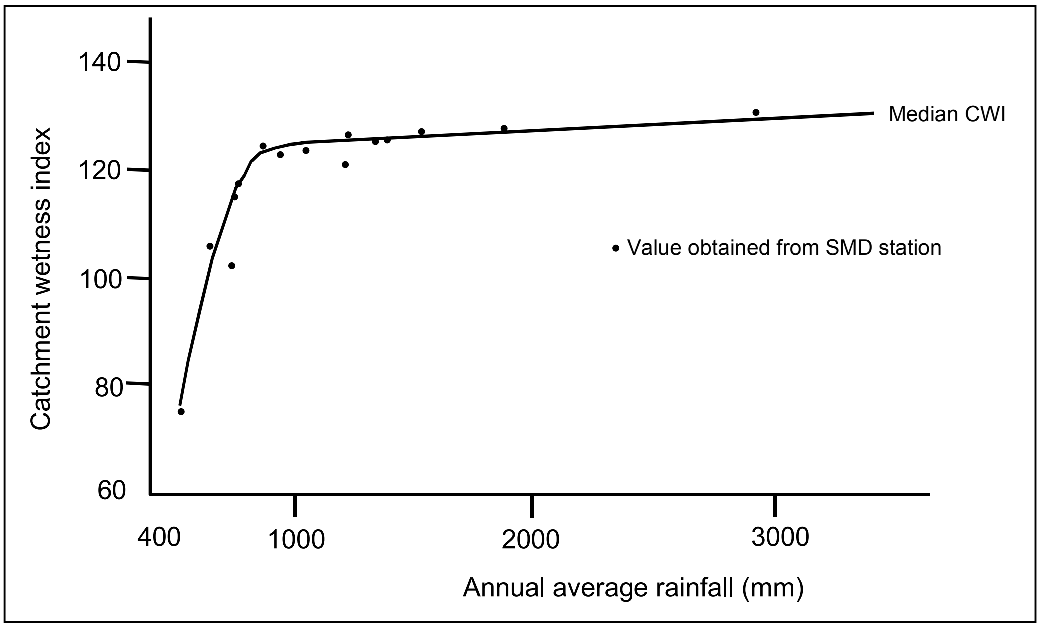



**Figure 5.4 Relationship between the catchment wetness index and the average annual rainfall**

$DPR_{CWI}$=0.25 (CWI-125)

It can be seen that $DPR_{CWI}$ generally has little effect for most locations in the country.

The $DPR_{RAIN}$ is the third dynamic component that increases the percentage runoff from large rainfall events.
$DPR_{RAIN} = 0.45(P - 40)^{0.7}$ for P >40 mm

$DPR_{RAIN} = 0$ for P less than 40 mm

Where:
P is the depth of rain that falls during the design storm

This method is very simple. Its level of accuracy is variable and supporting information should be sought if it is available. The estimate of percentage runoff should be checked against the runoff values in neighbouring catchments. Figures 5.5, 5.6, 5.7 and 5.8 show the relationships between percentage runoff for a rural area and rainfall depth, and runoff per unit area and rainfall depth for mean annual rainfalls of 700mm and 1000mm or greater for the five soil types used in the Flood Studies Report.

These graphs show the importance that soil type plays in the expected volume of runoff and emphasises the need to get corroborative evidence if possible. However, the volume of runoff for large catchments is affected by spatial rainfall issues that will tend to over-predict the runoff fraction. Care must therefore be taken in this regard.

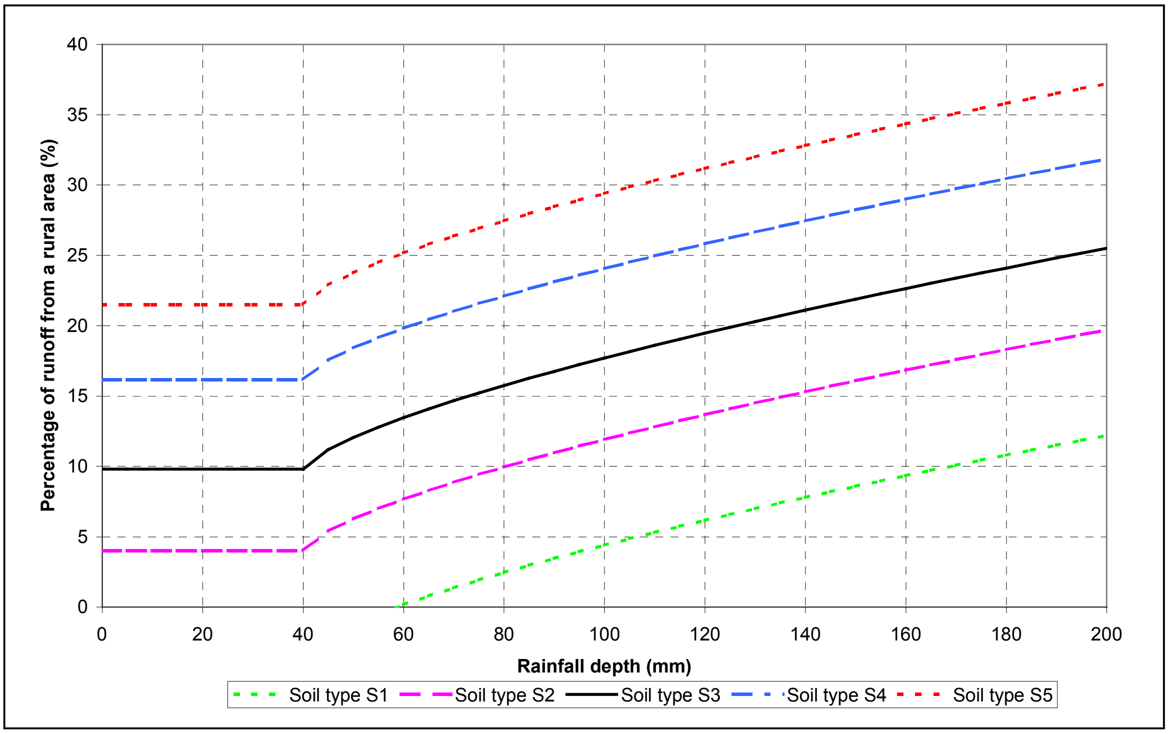



**Figure 5.5 Percentage of rural runoff versus rainfall depth for a mean annual rainfall of 700mm**

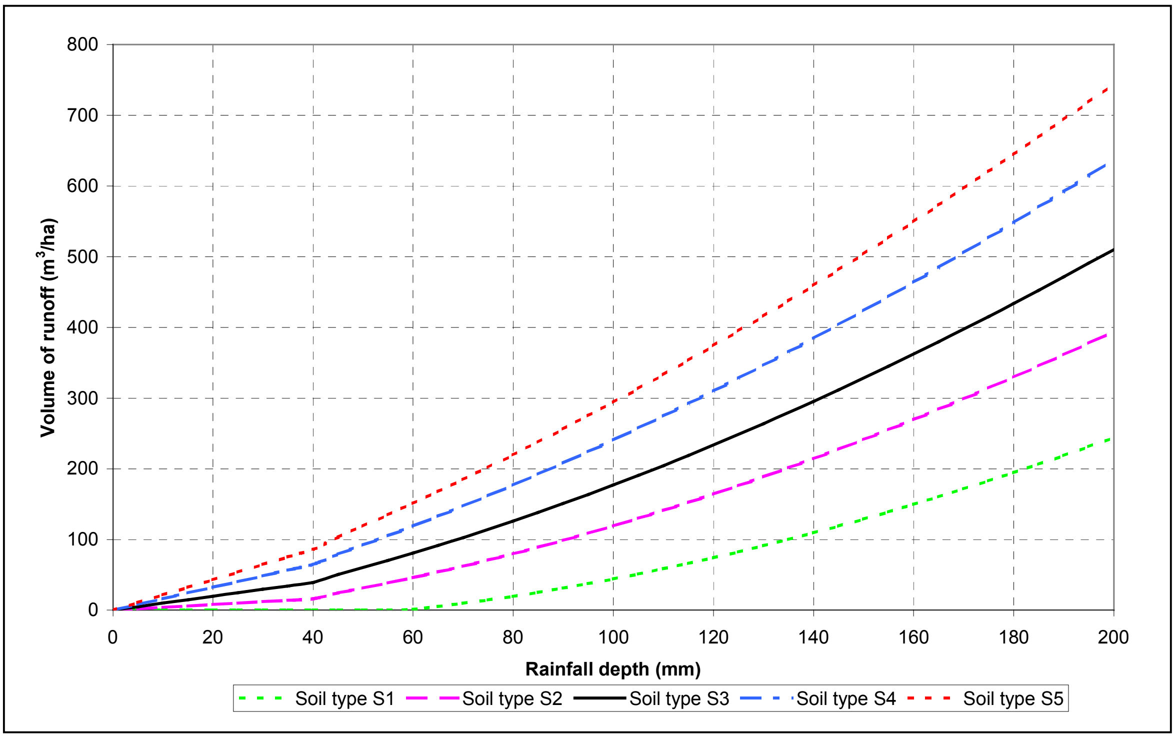



**Figure 5.6 Volume of rural runoff versus rainfall depth for a mean annual rainfall of 700mm**

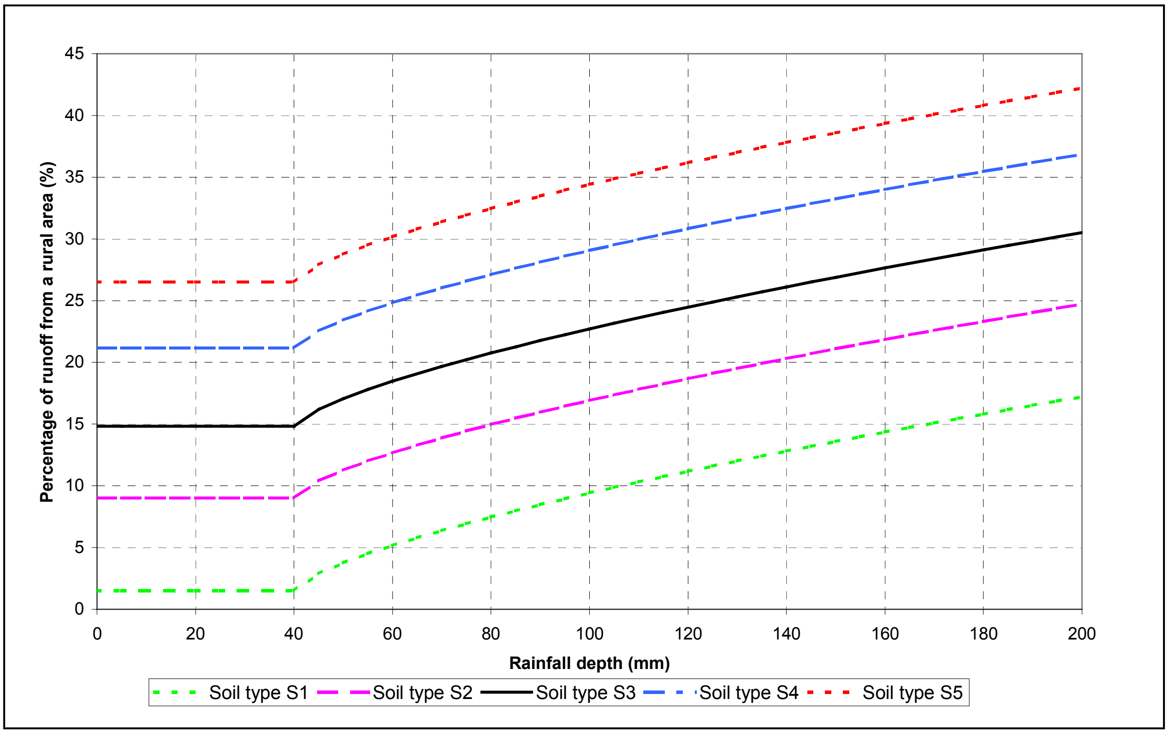



**Figure 5.7** **Percentage of rural runoff versus rainfall depth for a mean annual rainfall equal to or greater than 1000mm**

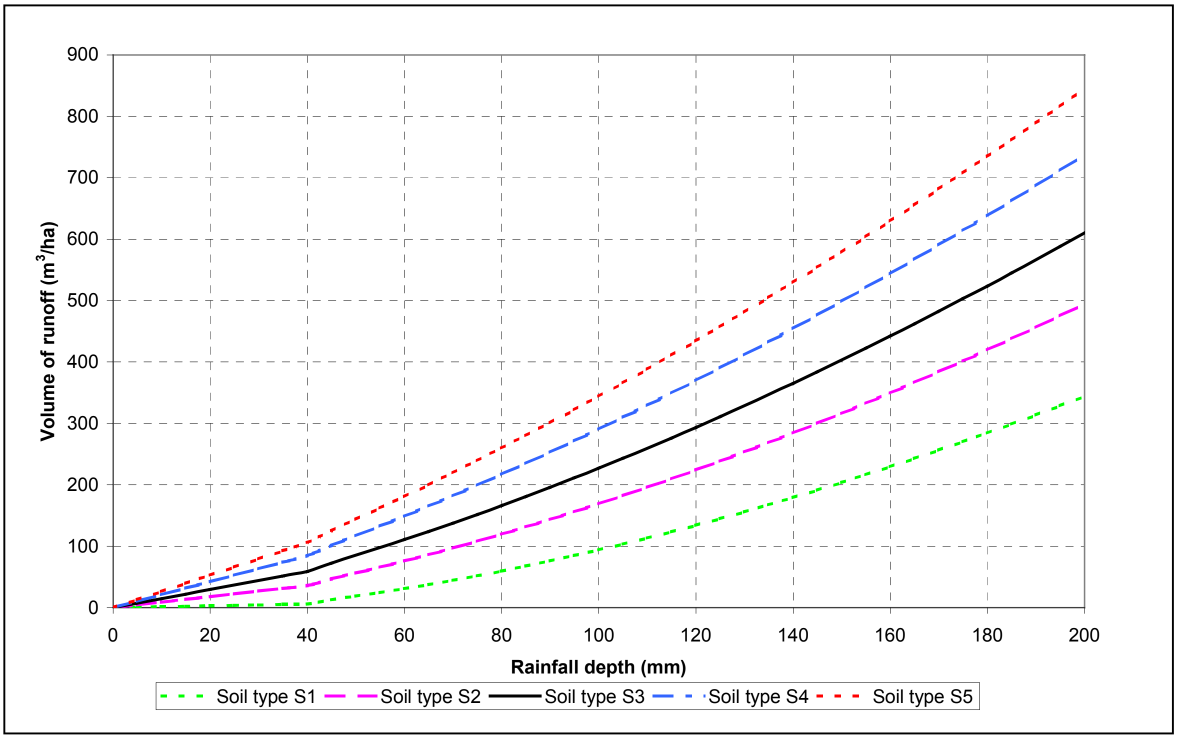



**Figure 5.8** **Volume of runoff versus rainfall depth for a mean annual rainfall of equal to or greater than 1000mm**

## 6. RUNOFF FROM DEVELOPMENT SITES

Over the last decade it has become evident that the positive drainage of new developments without consideration of the impact of the increasing rate and volumes of runoff on rivers downstream is not sustainable. Due to commercial pressures, developers tend to produce sites with a high proportion of paved area that significantly increases the runoff from the site compared with the undeveloped greenfield site. Often little consideration is given to ameliorating the impact this has downstream by providing facilities to counter this effect. In principle there are two features that have to be considered. The first is to reduce the runoff rate to protect the river from environmental damage (erosion and water quality impact) and the second is to consider the increased volume of flow being generated. This later aspect becomes more critical as the proportion of the paved area increases especially for catchments which are naturally porous (sandy soils).

### 6.1 Rate of runoff

Runoff from positively drained paved areas is effectively instantaneous by comparison to greenfield runoff. The runoff rate therefore reflects the intensity of rainfall with attenuation being provided by the filling of depression storage, surface runoff routing and pipe routing. This is true for all rainfall up to around 20 to 30 year return period events. Thereafter, short duration "summer" type storms have intensities which are so great that temporary flooding takes place owing to the inadequate capacity of the pipe system to cope with the volume of water.

From a site storage point of view, it is therefore relatively unimportant to determine peak flow rates from the site except to be aware that it is rapid. Information relating to the routing processes of rainfall runoff used in models is available in the Wallingford Procedure (1981) or Wallingford Procedure for Europe (2000). The important feature in determining any storage volume is the percentage rainfall that directly drains and runs off into the drainage system, and the shape of the storm profile being used.

### 6.2 Runoff volumes

The determination of storage is a function of runoff volume and the time taken which effectively approximates to the duration of the rainfall. The following are the most commonly applied methods for determining storage requirements.

- Wallingford Procedure, Old Percentage Runoff equation;
- Wallingford Procedure, New Percentage Runoff equation;
- The Modified Rational Method;
- COPAS.

**The Wallingford Procedure methods** are hydrograph based methods. They are based on determining flow volumes and therefore take account of volumes of pipework and flooding and are therefore most suitable for analysis of storage related issues.

**The Modified Rational Method**, as previously discussed, is a discharge rate based method, and therefore is less suitable. It can be used for determining storage, but cannot be used to simulate performance for various criteria. It is important to note that the difference between the standard Rational Method and the Modified version proposed as part of the Wallingford Procedure in 1981 is that the volumetric coefficient is no longer 1.0, but 0.7. This is important in that it recognises that the runoff from paved surfaces is never 100%.

**The Copas method** and other empirical methods are now considered to be obsolete.

The following section details the Wallingford Procedure methods, how they should be used and their limitations.

## 6.3 The Old PR Wallingford Procedure equation

The old PR runoff model is a statistically based regression equation that was calibrated against a large number of events recorded in UK. The most important parameter is the contributing paved percentage of the catchment (PIMP) followed by a factor referred to as SOIL (soil characteristic) and UCWI (Urban Catchment Wetness Index). The old percentage runoff (PR) Wallingford Procedure equation is given below.

$$PR = 0.829PIMP + 25.0SOIL + 0.078UCWI - 20.7$$

where:

| | |
|---|---|
| PR | percentage runoff; |
| PIMP | percentage impermeability (0 to 100); |
| SOIL | an index of the water holding capacity of the soil (0.15 to 0.50); |
| UCWI | Urban Catchment Wetness Index. (0 to 300). |

Inspection of the equation indicates that for low values of PIMP, SOIL and UCWI low or even negative values of PR can be calculated. Consequently, a minimum value of PR of 20% is used for impervious surfaces in Wallingford Software models. The reason for this apparent inadequacy is that the correlation was based upon fully urbanised catchments that were drained by fully combined or fully separate systems. Therefore catchments with PIMP values much below 30% are likely to underpredict the volume of PR.

**PIMP**
The range for which PIMP is usually considered to be acceptable is 30 to 100. The higher values tend to slightly over predict runoff while lower values under predict. This has implications for use in partially separate systems (roofs draining to soakaways) and phased development where the areas for subsequent development are included as pervious areas.

**SOIL**
SOIL is a parameter which takes account of the soil type in the catchment. Soil types are given indices from 1 to 5 to represent the range of characteristics from sandy through to clay/rock soils, and their coefficients used in the equation are as shown in Table 6.1.

**Table 6.1 Soil type parameters**

| SOIL type | Coefficient |
|---|---|
| 1 | 0.15 |
| 2 | 0.3 |
| 3 | 0.4 |
| 4 | 0.45 |
| 5 | 0.5 |

**UCWI**
UCWI is calculated based upon antecedent rainfall conditions and the soil moisture. For real events therefore UCWI is individually calculated. This is not detailed here as it unlikely to be a feature of storage volume assessment. Reference should be made to the Wallingford Procedure (1981) for details of its derivation. For design events (often used for storage design) there are values provided for both Summer and Winter which are linked to the Standard Annual Average Rainfall (SAAR) value for that location. These curves were originally devised as part of the Wallingford Procedure research. It is now generally recognised that owing to the increased wetness of catchments in the long events which tend to be critical

for storage design, that the winter UCWI should be used and increased by around 20 percent. Figure 6.1 shows the relationship between SAAR and UCWI.

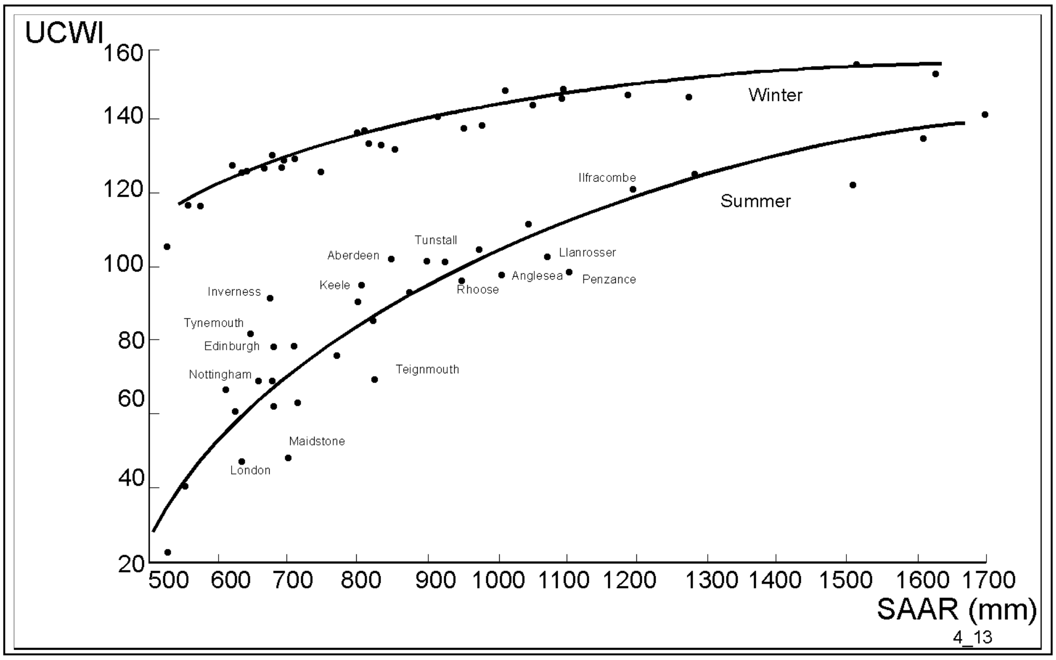



**Figure 6.1 Seasonal UCWI relationship with SAAR**

The Old PR model predicts a value for the total runoff from all surfaces in the sub-catchment; usually defined as roads, roofs and permeable. Runoff for the catchment is distributed between the different surfaces using weighting coefficients. All surfaces can therefore contribute some runoff even at low runoff rates, provided that initial losses have been satisfied. The weighting is usually apportioned with 10% of the runoff coming from the permeable area and this is routed at a slightly slower rate than the paved runoff. Thus if the percentage runoff is say 73% from the paved surface a value of 7.3% would be allocated as coming from the permeable surface. This immediately highlights the apparent contradiction between the Wallingford Procedure model and the FSR predictions where figures of up to 50% and more can be predicted as coming from the permeable catchment for extreme events. The equation for calculating surface type distribution of percentage runoff is given below.

$$PR_j = \frac{f_j A_j}{\sum_{k=1}^{3} f_k A_k} PR$$

where:

$f_j$ is weighting coefficient for surface j
$PR_j$ is percentage runoff for surface j
$A_j$ is area for surface j

Table 6.2 below shows the default parameters that are used in the above equation.

**Table 6.2 Default parameters for the weighting coefficients**

| Weighting coefficient | Surface | Value |
|---|---|---|
| $f_1$ | Paved | 1.0 |
| $f_2$ | Roofed | 1.0 |
| $f_3$ | Pervious | 0.1 |

Although this is currently the more commonly used model when the Wallingford Procedure is applied, the model has two serious drawbacks. The first is that the runoff losses are assumed to be constant throughout a rainfall event and secondly there is no linkage between runoff factor and return period. In practice catchments have increasing runoff as it gets wetter. This is the main reason why there is a trend towards using the New PR equation by drainage engineers. Figure 6.2 illustrates the differences in the percentage runoff (PR) between low and high values of PIMP and SOIL using a constant value of 100 for UCWI.

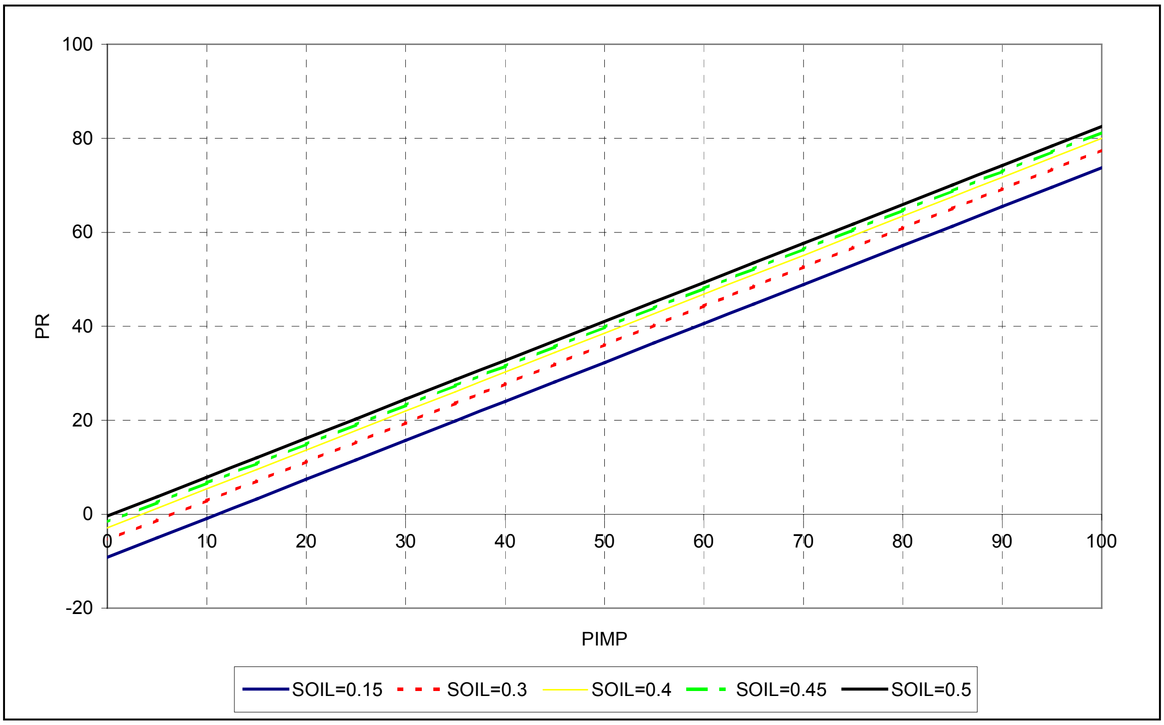



**Figure 6.2 Percentage runoff for the old PR equation**

## 6.4 The New Wallingford Procedure runoff model

The New UK PR equation, developed jointly by HR Wallingford, the Water Research Centre and the Institute of Hydrology with support from North-West Water PLC, has been designed as a replacement to the familiar Old PR equation. The new equation was designed primarily to overcome some of the difficulties experienced in applying the old PR equation, namely:

1. PR remained constant throughout a rainfall event irrespective of catchment wetness. For long storms reducing losses during the event may be significant in terms of urban drainage design, particularly for storage.
2. Problems have been encountered in applying the Old PR equation to partially separate catchments and to catchments with low PIMP and low SOIL values.
3. The increasing runoff associated with permeable areas should be treated more realistically.

To overcome these problems various new model forms were investigated using a subset of the original data that was used to derive the original PR equation. The New Wallingford Procedure runoff equation is given below.

$$PR = IF \times PIMP + (100 - IF \times PIMP) \times NAPI/PF$$

Where:

IF is the effective impervious area factor
PF is the moisture depth parameter (mm)
NAPI is derived from net antecedent rainfall (mm).

This equation divides percentage runoff (PR) into two elements. First, the impervious area PR that is obtained by using an effective impervious area factor, IF. IF should not be regarded as the proportion of runoff (the remaining proportion of rainfall being lost), but as the proportion of the impermeable area which has 100% runoff while the remaining impermeable area is treated as permeable area. Values of IF are usually calibrated against recorded data or selected based upon a knowledge of the surface condition. For unverified models, recommended values of IF are indicated in Table 6.3 below. These values are introduced as a constant runoff coefficient. A figure of 0.70 or 0.75 is recommended for new developments.

**Table 6.3 Recommended values of IF**

| Surface condition | Effective impervious area factor IF |
|---|---|
| Poor | 0.45 |
| Fair | 0.60 |
| Good | 0.75 |

The losses from pervious surfaces and non-effective impervious surfaces are represented by the second term of the equation. The first part of it represents the total percentage of the catchment occupied by pervious and non-effective impervious areas which has a response that is dependent on the function NAPI/PF. NAPI is defined as a 30 day (new) antecedent precipitation index (NAPI) with evapotranspiration and initial losses subtracted from rainfall. NAPI is given by equation below.

$NAPI_{30} = \Sigma_{n=1.30} P_{-n} C_p^{(n-0.5)}$

**NAPI**

The value of NAPI is calculated for the start of the event and is continuously updated during the rainfall event, thus increasing runoff contribution from permeable surfaces. P is the net precipitation each day. Typical values of NAPI at the start of an event are usually between 0mm and 30mm. It is usually considered reasonable that a value of zero is used when carrying out an analysis looking at extreme event assessment. However, there has been little analysis, and no official advice exists on this issue.

**C**

The constant value C of NAPI has been made dependent on the soil type to reflect the faster reduction of soil moisture on lighter soils. The relationship between C and soil type is shown in Table 6.4.

**Table 6.4 Decay coefficients for calculation of NAPI**

| Soil type | Coefficient "C" |
|---|---|
| 1 | 0.10 |
| 2 | 0.50 |
| 3 | 0.70 |
| 4 | 0.90 |
| 5 | 0.99 |

The main problem with the choice of decay coefficient is the sensitivity of its effect. The table advises on the likely best value, but care should be taken in being aware of the impact of the choice that is made.

**PF**

The moisture depth parameter, porosity factor (PF), was investigated and the research suggested that a value of 200 mm should be used. (This compares well with the available water capacity of soils with grass vegetation). It is recommended that 200 mm is not changed unless there is good data against which it can be calibrated.

### 6.4.1 Limitations of the New PR equation

The limitations of this equation for design use relate to the following points:

- The choice of IF;
- Selection of the initial value of NAPI;
- Maximum permeable area runoff;
- The choice of soil depth PF;
- The decay constant C.

**The choice of IF**

Engineers tend to choose values ranging from 100% down to 70% depending upon the situation and personal views. The problem is that the New PR equation requires a decision by the engineer, whereas the Old PR equation left no room for choice. However in practice engineers have little difficulty in making such a decision and a figure of around 70% is generally chosen. However it can be seen from the table above that much lower values for IF might be appropriate for existing "old" systems where runoff from all paved surfaces are not effectively drained. A quick comparison with the Old PR equation can be carried out to determine $PR_{paved}$ and $PR_{pervious}$ values for the start of the event.

**Selection of the initial value of NAPI**

The value of NAPI for design conditions has not been properly researched or officially agreed. Analysis of antecedent conditions related to extreme events would provide the information necessary for a specific location, but this is impractical in most situations and a pragmatic choice is usually made. It is suggested that a value is calculated for the pervious area which provides an initial runoff of 10 percent of the value of IF, which would therefore relate fairly closely to the Old PR equation. However, as the basis for this assumption for the Old PR equation is an averaged allowance for the duration of the event, this is probably being a little conservative.

A more important consideration is that the maximum value of permeable percentage runoff for pervious areas should probably never exceed the equivalent percentage runoff predicted by a Flood Studies analysis and should probably be considerably less. The logic behind this suggestion is that the landscaping that takes place on a site would disrupt the natural greenfield runoff flow paths to the river. However, there is a counter argument which is that permeable flow paths are much shorter to any point of direct drainage.

For large rainfall events, the proportion of runoff from the pervious catchment can increase dramatically during the event. This is due to the increase of NAPI with rainfall depth particularly in the case of clay soils. Figure 6.3 illustrates the increasing value of Percentage Runoff for a catchment with 50 percent paved area with a value of IF set at 0.6 and the five soil types. An 80mm 18 hour design event has been applied. The figure also shows the increase in only the permeable contribution for this same event. The distinctive shape is a function of the design rainfall profile and the decay constants for the various soil types. It should be noted that the pervious runoff fraction is still below 30% even for rock/heavy clay catchments for this example, which rather belies the industry concern that the New PR equation over-predicts pervious runoff.

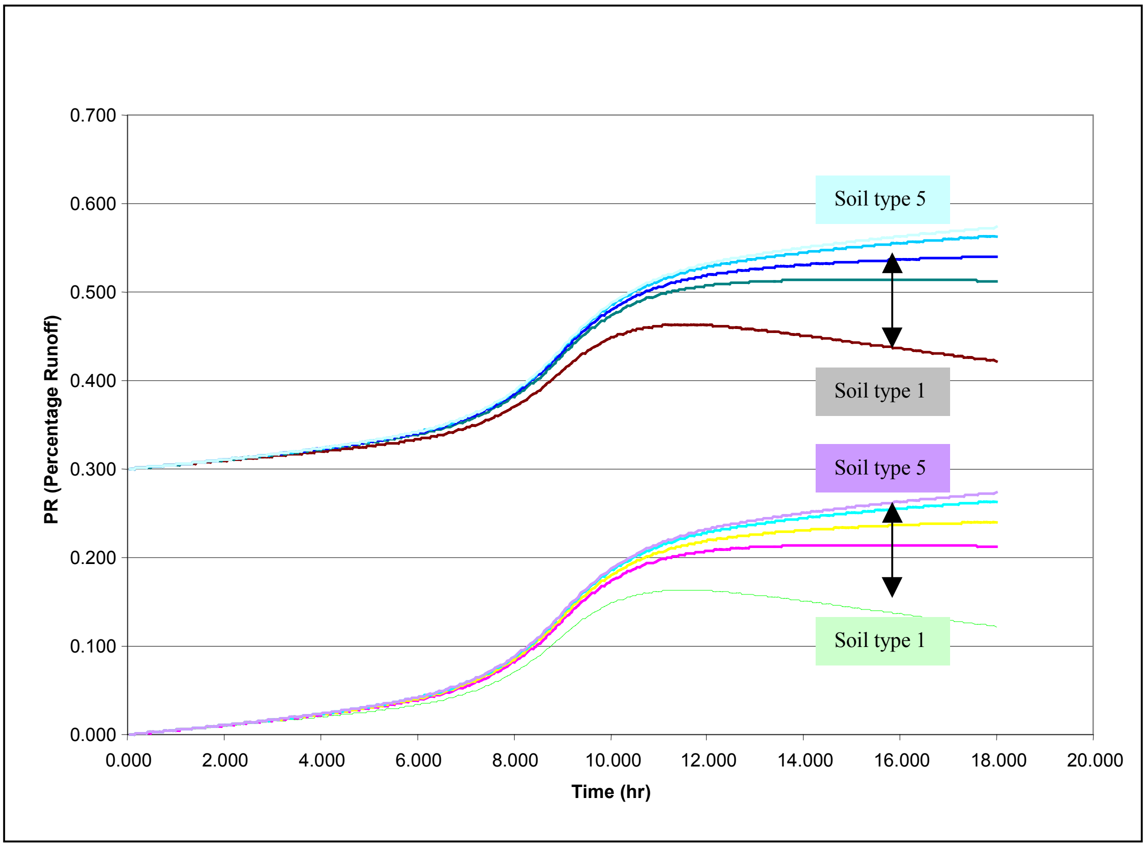



**Figure 6.3 Increasing runoff with the New PR equation.**

If storage is being designed, volumes can be significantly greater than those derived by the Old PR equation. In particular with high return period analysis, the critical duration event can increase significantly. In the worst case a critical duration may seem unreasonably long, for instance a 24 hour storm. In this circumstance it might be appropriate to look at using a rainfall time series as the frequency of such events may be rare. This would only occur where very severe discharge limits are being applied.

**The choice of soil depth PF**

The value of PF is one of the most important imponderables. Although defined as the soil depth, it is difficult to relate the soil characteristics of the catchment to this parameter. In pragmatic terms it controls the rate of increase of runoff from the permeable surface. Therefore arriving at a figure, if changed from the advised 200 mm, (which provided the best average correlation for the research data used) it is best considered by looking at the final runoff proportion at the end of the event. As a guide it is suggested that the Flood Studies Report could be used to determine figures for percentage runoff which accord with the relevant soil type and return period to provide an indication of the maximum value that would take place off a rural catchment. This would provide an upper bound value in the region of 50 to 55% for heavy soils. Unfortunately urban environments are not like rural catchments. Drainage paths are short, but much of the permeable area may not have direct access to the drainage system after landscaping. It is suggested that the maximum value of the permeable runoff for developed catchments should not be greater than 65% of the FSR value for the undeveloped site. However, this is clearly related to landscaping and also very much a matter for personal engineering judgement.

**The decay constant C**

The decay constants will allow a continuous time series to be analysed. However there is no seasonal term and the coarse choice of parameters varying from 0.99 to 0.1 in 5 soil classes, allows considerably opportunity for NAPI to diverge from the most appropriate value. Manipulation of C will modify NAPI and so control the maximum percentage contribution of permeable runoff. In most instances it is suggested that C should not be modified from its advised soil related value.

### 6.4.2 Other modelling issues for determining runoff from permeable areas

**Steep or compact surfaces**
There are always special circumstances that can be relevant. Care should be taken where local steep permeable areas will respond to summer storms with up to 70% or 80% runoff. Another example of non-standard runoff is reinstated land. Over-compacted re-instated coal tips (as grassed hillocks) have had runoff measured in excess of 60% in winter with very low intensity continuous rainfall in winter. These situations need to be treated individually and should not be confused with the general argument of determining increasing runoff from permeable areas for longer and bigger events.

**Disconnected permeable areas**
The effect of landscaping during development is to alter the natural flow path of water to the lowest point of the site. This can be used to the advantage of the design engineer as careful attention to landscaping can effectively reduce the contributing permeable area that is drained. These areas might intentionally pass to a soakaway system or some wetland area. The advantage of disconnecting areas is that contributing runoff volumes passing to the receiving stream can be reduced and so minimise the storage needs for the catchment.

**Rate of runoff of permeable areas**
The default routing rate for permeable runoff is set at four times that of directly paved runoff in the Wallingford Procedure software. This is rarely challenged as the effect of paved runoff is so dominant in terms of peak flow rate and volumes generated. However, there are indications that the correct representation of permeable routing rates is probably closer to a value between 10 and 15 times slower than the paved coefficient. In practice permeable catchments range from strips of grass adjacent to roads to large football fields and therefore routing factors would also range widely. For simplicity it is suggested that the default value is used (a conservative assumption) unless a change is justified.

**Permeable pavements**
Permeable pavements can be represented in hydraulic models. However, there is little official guidance at present in the approach that should be taken in modelling them accurately. The impact on both the runoff rate and volume is very significant and this will require more attention in the future as Sustainable Urban Drainage Systems get implemented.

### 6.4.3 Comparison of the Wallingford Procedure new PR equation with FSSR 16

The derivation of the two methods for estimating runoff volumes is based upon very different event types and environments. The research in this report indicates that the excess runoff generated by a development is an important factor and needs to be specifically catered for. Both equations are accepted as best practice, but a comparison highlights the limitations of relying completely on the values obtained. Figure 6.4 illustrates the importance of the soil type in generating additional runoff and shows the excess volume of runoff for a range of soil types for a range of impermeable catchment fraction.

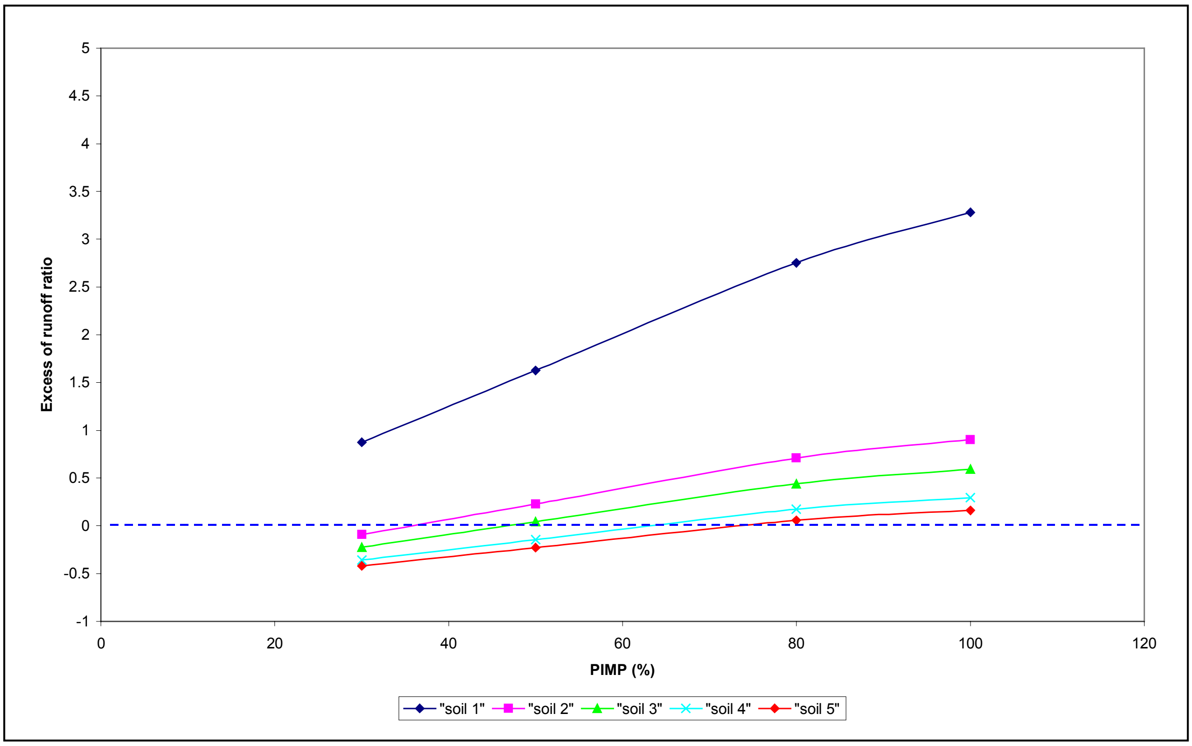



**Figure 6.4 Excess runoff volume based upon New PR and FSSR 16**

It can be seen that there is an apparent anomaly with a development generating less water than that produced before construction. As stated before, this might be explained by the effects of landscaping effectively isolating or "switching off" some areas. However, an alternative (conservative) method of approach would be to assume that all hard surfaces generate approximately 80% runoff (see Figure 6.2) and that some or all of the permeable surfaces would contribute as they did prior to the development. This could be represented as shown in the following equation:

$$PR = (SPR + DPR_{CWI} + DPR_{RAIN})(1\text{-}PIMP) + 0.8PIMP$$

Figure 6.5 illustrates this alternative method of determining excess runoff and should be compared to Figure 6.4. It can be seen that this is more conservative. An intermediate alternative is also possible where it is known what permeable areas contribute or do not contribute to drained runoff.

Finally it must be pointed out that many developments take place on steep sites; steeper than the sites for which the FSSR 16 formula should be used. It might therefore be reasonably assumed that the pre-development runoff might be higher in these circumstances than values predicted by FSSR 16. Overall, it is therefore recommended that the standard forms of the equations are generally applied in most circumstances in determining excess volumes. Unfortunately the simple differentiation between permeable and hard surfaces is complicated by SUDS techniques with impermeable surfaces contributing much less runoff and landscaping specifically preventing runoff from permeable areas. All these aspects will need to be taken into account in determining excess runoff volumes in the future.

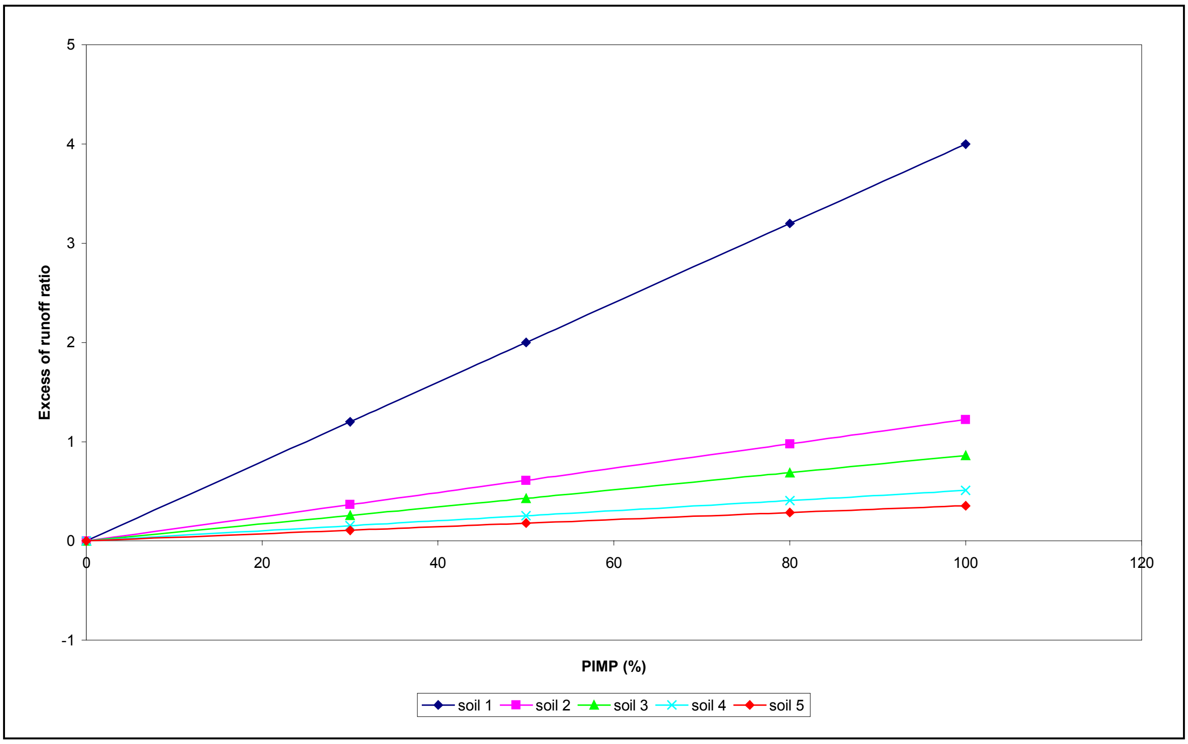



**Figure 6.5 Excess runoff volume using FSSR 16 and PR of 80% for paved surfaces only**

## 6.5 Rainfall profiles

Rainfall profiles used in UK are designed as either summer or winter profiles. These are symmetric and are respectively defined as being 50 percentile and 75 percentile storms. The summer profile provides a maximum intensity which 50 percent of real storms exceed for that specific return period and duration. Similarly this applies to the winter profile for 75 percent of events.

The design rainfall profiles were derived from the Flood Studies work in 1975. The recent work carried out for the Flood Estimation Handbook has shown that the current design events have volumes and intensities that can be significantly different to figures currently used based on FSR. This is likely to be addressed in due course, but until this takes place, care should be taken in defining design storms. However, FEH has taken the view that storm profiles are the same.

The critical duration event for any specific limiting discharge may either be a winter or summer profile depending on the relative volumes above the effective runoff intensity threshold for that event. The more restricted the limiting discharge, the longer will be the critical duration event. Figure 6.6 illustrates the effect of rainfall profile and limiting discharge in determining storage volumes.

## 6.6 Rainfall parameters

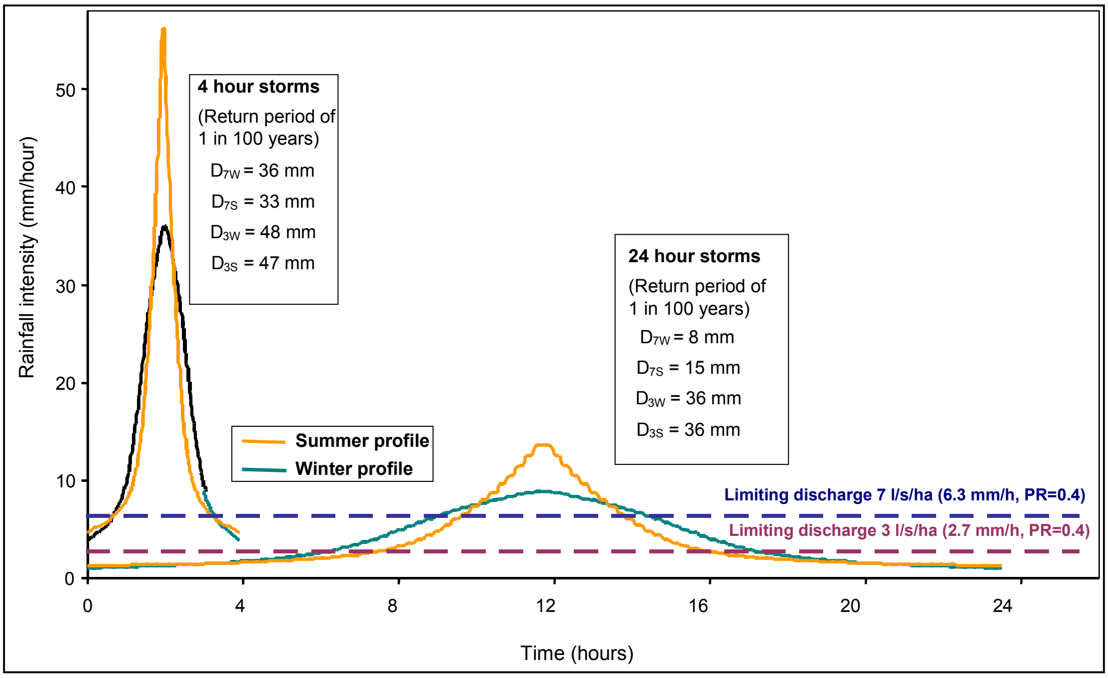
The analysis of UK rainfall used in the Wallingford Procedure is described in the Flood Studies Report. Volume 2 of the Report gives details of the analysis that was carried out, and Volume 1 gives details of some of the statistical basis of the work. The analysis was based on the very extensive rainfall records held by the Meteorological Office in the UK. This is still the basis for most sewer network assessments.

The relationship between depth of rainfall and storm duration and return period was derived by analysing rainfall records for a large range of durations to determine the largest depth of rain for each duration in each year of the record. These formed an annual maxima series from which the depth of rain for various return periods could be derived. Relationships were then found so that the rainfall depth in storms covering the whole range of durations and return periods could be defined by two parameters and the use of standard equations. The two parameters are:

- M5-60, the depth of rain in 60 minutes for a return period of 5 years.
- r, the ratio between M5-60 and M5-2 day rainfall.

It should be noted that M5-60 refers to a 60 minute period starting in any minute rather than to a clock hour (for a 1 hour period starting at the top of the hour). Conversely M5-2 day refers to a period of 2 calendar days and would normally start at 9 am Greenwich Mean Time (GMT). M5 refers to a return period of 5 years. These two parameters, together with the choice of event duration and return period, allow design rainfall profiles to be generated. These two parameters are given on maps covering the whole of the UK. Small scale versions of these maps are given in Figures 6.7 and 6.8, but larger ones at a scale of 1:1,000,000 are provided separately in Volume 4 of the Wallingford Procedure, or in electronic form in the more recent Wallingford Procedure for Europe (2000). Alternatively the same information is available from the maps that accompany the Flood Studies Report.

The method gives rainfall depths for durations from 5 minutes to 48 hours, and for return periods from 1 year to 100 years. It first calculates the depth of rain for a 5 year return period for the required duration (M5-D), then calculates from this the depth of rain for the required return period (MT-D).




Note: $D_{7W}$ refers to the effective depth of storage in terms of rainfall depth for a winter profile for a throttle of 7 l/s/ha

**Figure 6.6 Rainfall profile effects on the critical duration**

M5-D i.e. the 5 year rainfall depth for any duration is given by the equation below.

$$\ln M5\text{-}D = \ln D + \ln\left(1.06\,\frac{M5\text{-}60}{48\,r}\right) + \frac{\ln\left(\frac{721}{1+15\,D}\right)\ln\left(\frac{48\,r}{1.06}\right)}{\ln\left(\frac{721}{16}\right)}$$

The calculation of MT-D depends on the return period and region. The country is divided into two parts; England and Wales, and Scotland and Northern Ireland. This was done to take account of the differences in the growth curve characteristics. For return periods greater than 5 years rainfall depth is given by;

$$\ln\left(\frac{MT\text{-}D}{M5\text{-}D}\right) = C_r\,(\ln(T) - 1.5)$$

where $C_r$ is a constant varying with geographical location and with the value of M5-D.

Further information on rainfall parameters is available from the Wallingford Procedure documents as it is not particularly relevant to go into detail in this document.

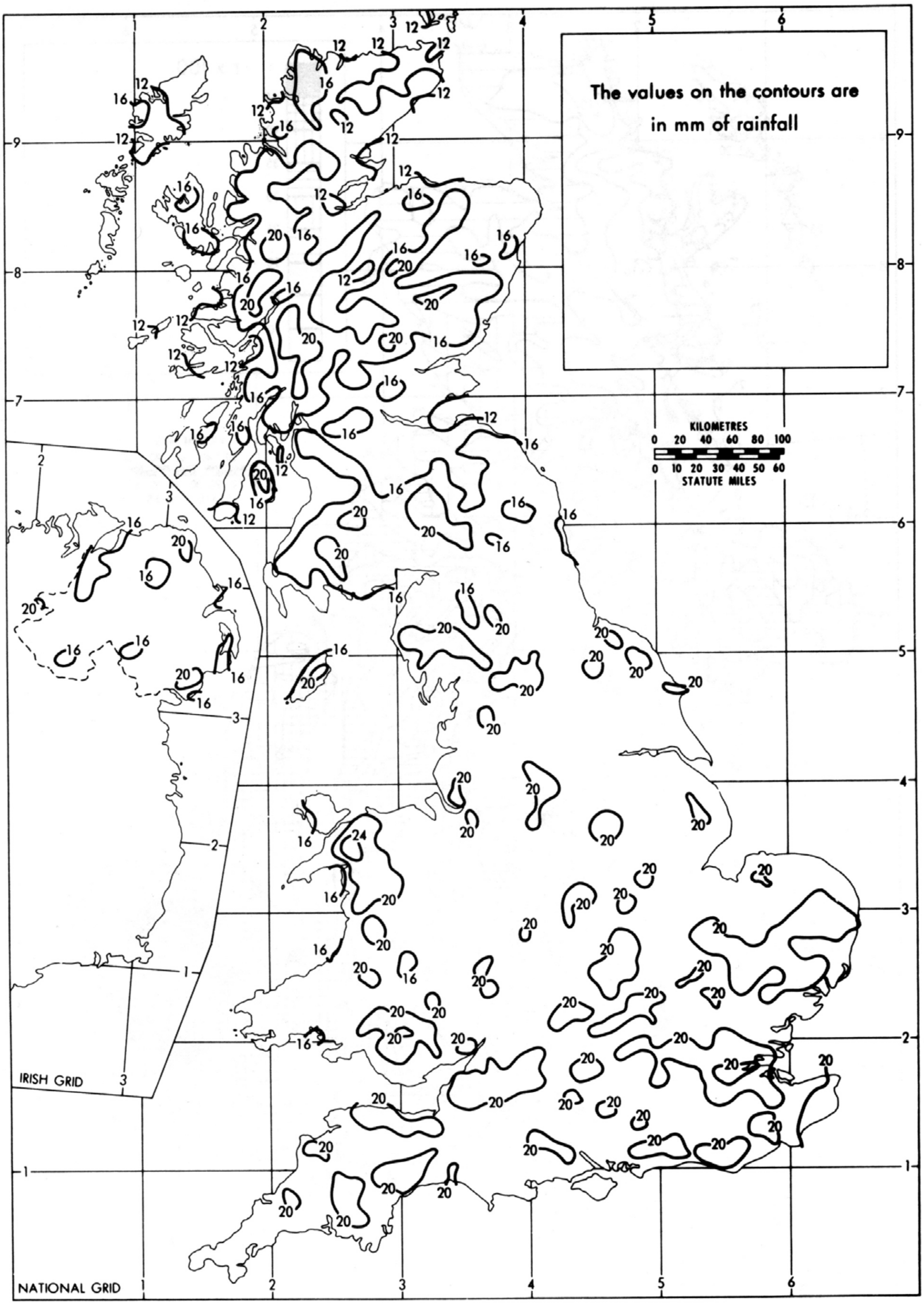



**Figure 6.7 Rainfall depths of five year return period and 60 minutes duration ($M_5 60$)**

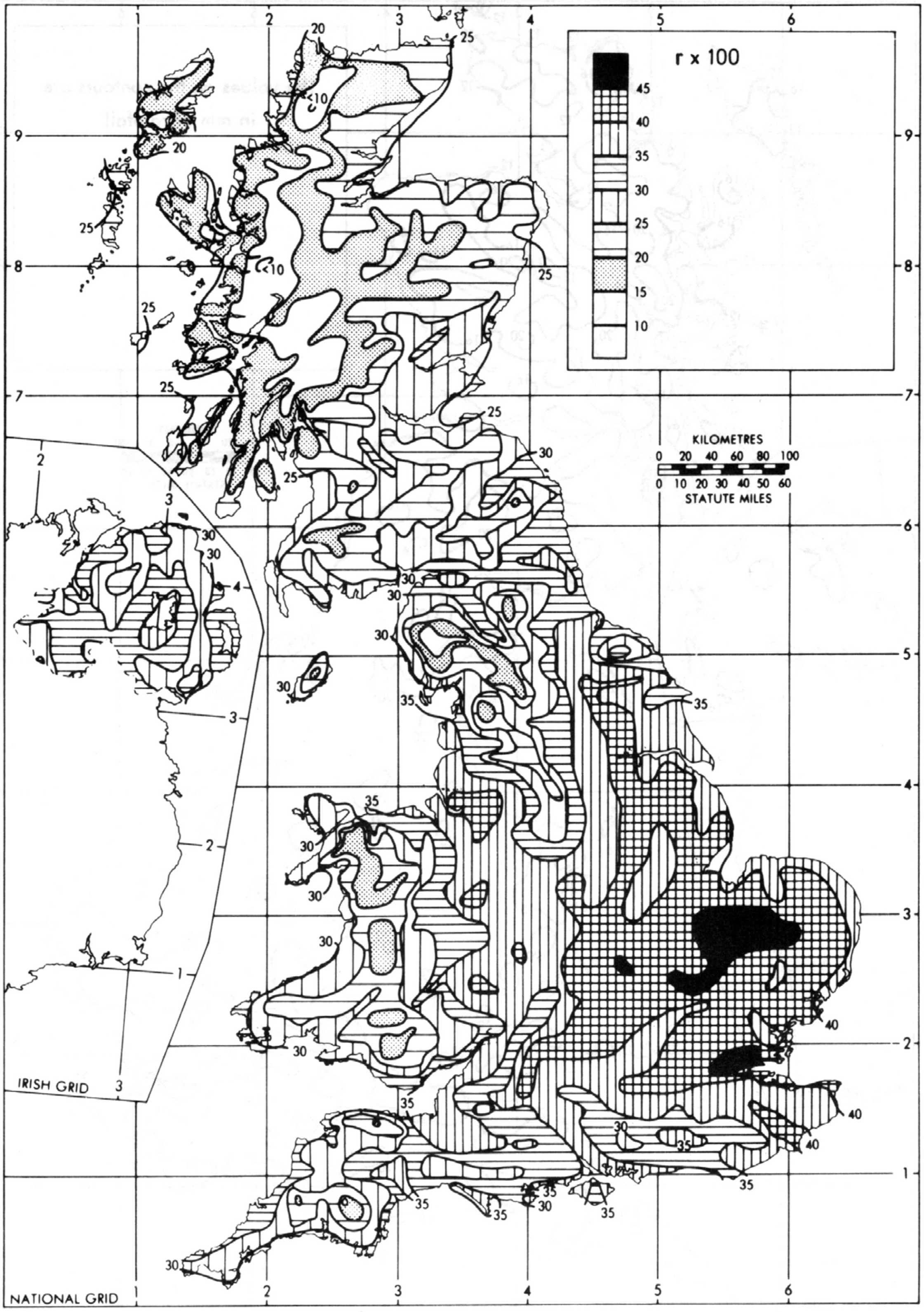



**Figure 6.8 Ratio of sixty minute to two day rainfalls of five year return period (r)**

# 7. STUDY OF THEORETICAL SITES USING OBSERVED AND THEORETICAL DATA SETS

One of the main reasons for this research was to test the philosophy of using temporary storage to satisfy a limiting discharge constraint to protect the river, mostly with the aim of protecting it during periods of flood risk. The methods of testing this assumption are described in this chapter.

## 7.1 Background

This chapter provides details of the analysis carried out for theoretical development sites using observed and design rainfall data sets to establish the storage required to attenuate the runoff from sites using a range of discharge limits. In order to assess the influence and effectiveness of limiting discharges on storage requirements, ten sites were selected throughout England and Wales. These were assumed to exist at the location of a rain gauge which was also located close to an adjacent river flow gauge. A theoretical 10ha development site was assumed for each of the ten catchments. Autographic rainfall data recorded at a rain gauge in each of the catchments were used in conjunction with the available river flow data to assess the storage volume to attenuate runoff from the development site for various discharge limits. The sites were chosen on the basis of the quantity and quality of flow and autographic rainfall data that were available. Details of the sites together with the rain and flow gauges used are given in Tables 7.1 and 7.2 below.

**Table 7.1 Details of the flow gauges used**

| Site number | Name of development site | Receiving watercourse | Location of flow gauge | Catchment area draining to gauge ($km^2$) | Length of flow data set |
|---|---|---|---|---|---|
| 1 | Hinckley | River Sowe | Stonleigh (Grid ref: SP332731) | 262.0 | 1978 to 1999 |
| 2 | Ogstone | River Amber | Wingfield Park (Grid ref: SK376520) | 139.0 | 1971 to 1999 |
| 3 | Overseal | River Trent | Drakelow Park (Grid ref: SK239204) | 3072.0 | 1974 to 1999 |
| 4 | Bordon | River Wey | Farnham (Grid ref: SU838462) | 191.1 | 1979 to 1999 |
| 5 | Cranleigh | Law Brook | Albury (Grid ref: TQ045468) | 16.0 | 1979 to 1999 |
| 6 | Crowland | Glen River | Kate Bridge (Grid ref: TF 106149) | 341.9 | 1977 to 1999 |
| 7 | Oundle | Willow Brook | Fotheringhay (Grid ref: TL067933) | 89.6 | 1977 to 1998 |
| 8 | Richmond | Bedale Beck | Leeming (Grid ref: SE306902) | 160.3 | 1985 to 1999 |
| 9 | Maundown | River Isle | Ashford Mill (Grid ref: ST361188) | 90.1 | 1962 to 1999 |
| 10 | Wennington Bridge | River Wenning | Wennington Bridge (Grid ref: SD615701) | 142.0 | 1990 to 1999 |

**Table 7.2 Details of the rain gauges used**

| Site number | Name of development site/Rain gauge | Mean annual rainfall (mm) | Length of rainfall data set |
|---|---|---|---|
| 1 | Hinckley | 682 | 1963 to 1999 |
| 2 | Ogstone | 765 | 1964 to 1999 |
| 3 | Overseal | 724 | 1974 to 1999 |
| 4 | Bordon | 854 | 1979 to 1999 |
| 5 | Cranleigh | 817 | 1979 to 1999 |
| 6 | Crowland | 620 | 1977 to 1999 |
| 7 | Oundle | 616 | 1979 to 1999 |
| 8 | Richmond | 689 | 1985 to 1999 |
| 9 | Maundown | 890 | 1967 to 1989 |
| 10 | Wennington | 1334 | 1991 to 1999 |

The location of the selected catchments is shown in Figure 7.1.

## 7.2 Selection of the test catchments

Rainfall data were collected from the Environment Agency for 59 autographic rain gauges throughout England. In addition to the autographic rainfall data, 41 daily rain gauge data were obtained that were in the vicinity of the autographic rain gauges. These gauges were used as a check for the 59 autographic rain gauges. In addition to the rainfall data, flow data for the 66 flow gauging stations closest to the 59 autographic rain gauges were also collected. Figures 7.2 to 7.7 show the positions of the raingauges and their associated flow gauges.

An analysis of the data from each of the autographic rainfall gauges was carried out, in order to establish the reliability and accuracy of the rainfall data. Tables were developed that compared the daily totals from the autographic rainfall gauges with corresponding daily check gauges. The results of these analyses were summarised on a monthly basis. The Tables showing these analyses are given in Appendix A. Monthly differences between the autographic rainfall gauge and the daily rainfall check gauge of less than 10mm are highlighted in green, differences of more than 10mm are highlighted yellow and differences of greater than 20mm are highlighted in red. Where daily rainfall data from a check gauge were not available, the table is left uncoloured.

The procedure for the selection of the test catchments was based on establishing the most useful data sets for both autographic rainfall and flows. The following criteria were used:

- Duration of the data set;
- Reliability of the data set;
- Size of the associated catchment;
- Location of the rain gauges.

A total of ten catchments were selected as detailed in Table 7.1.

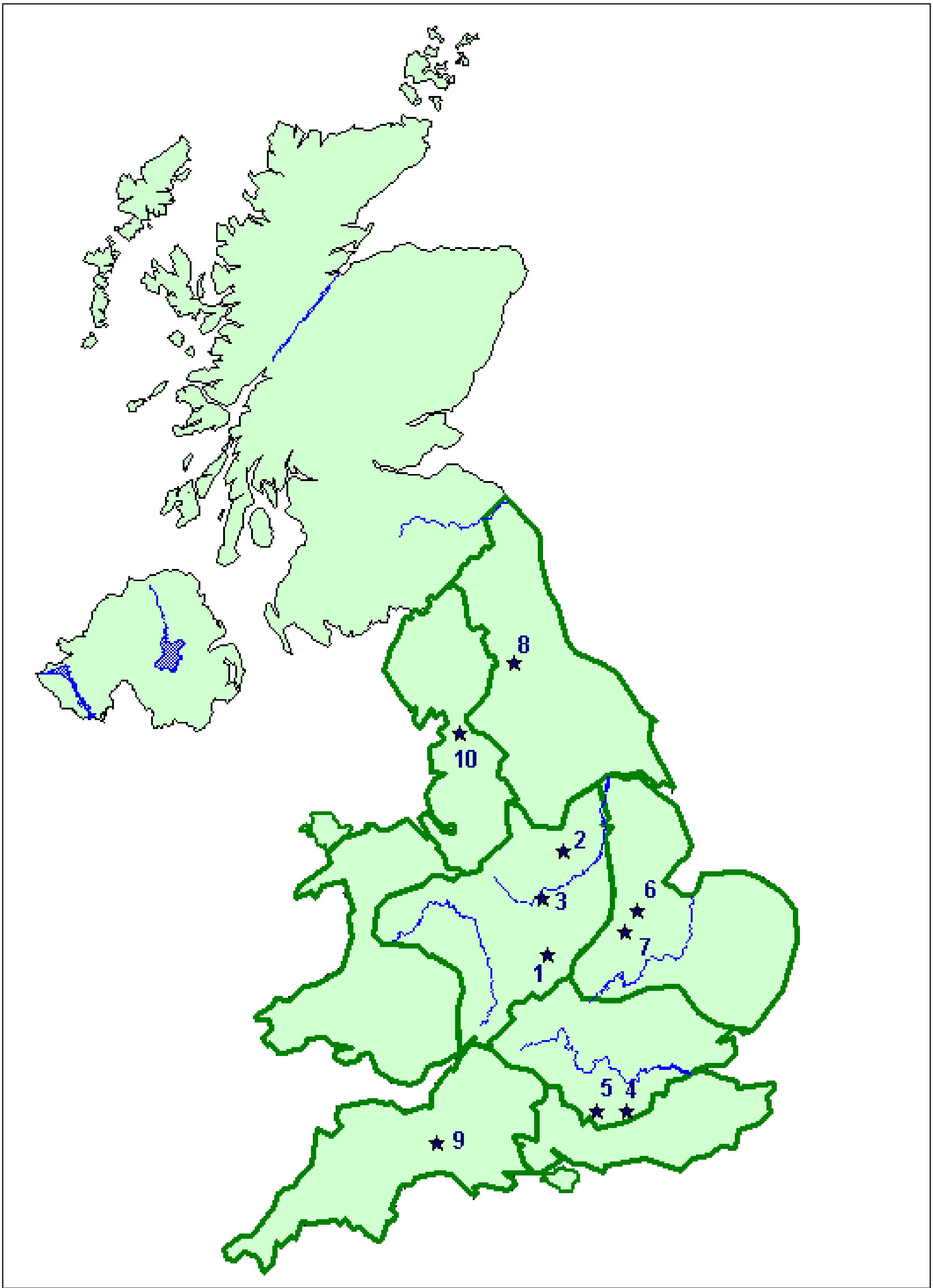



**Figure 7.1** Selected test sites and river catchments

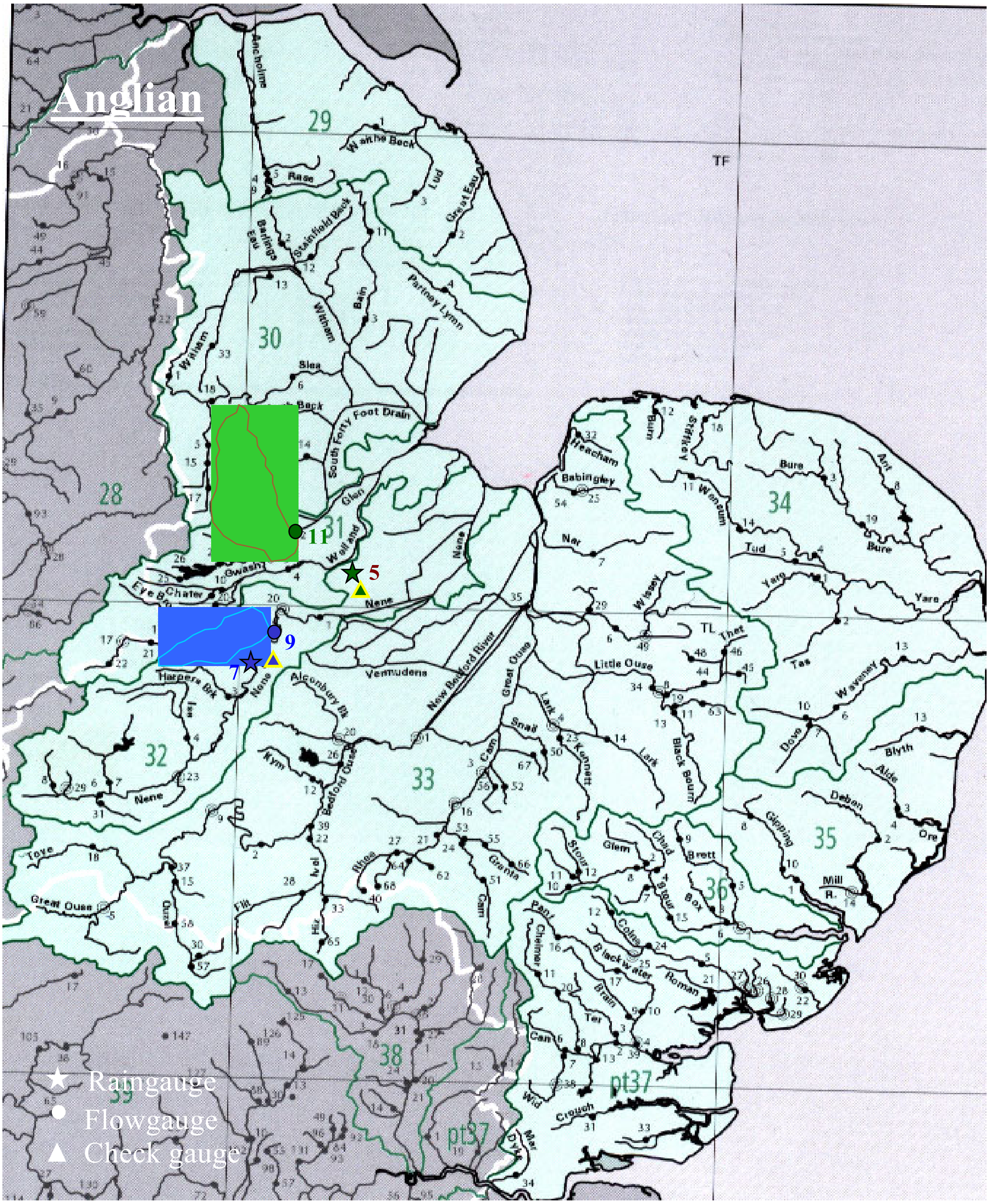



**Figure 7.2** Raingauges at Crowland (5) and Oundle (7) for River Glen (11) and Willow Brook (9) catchments

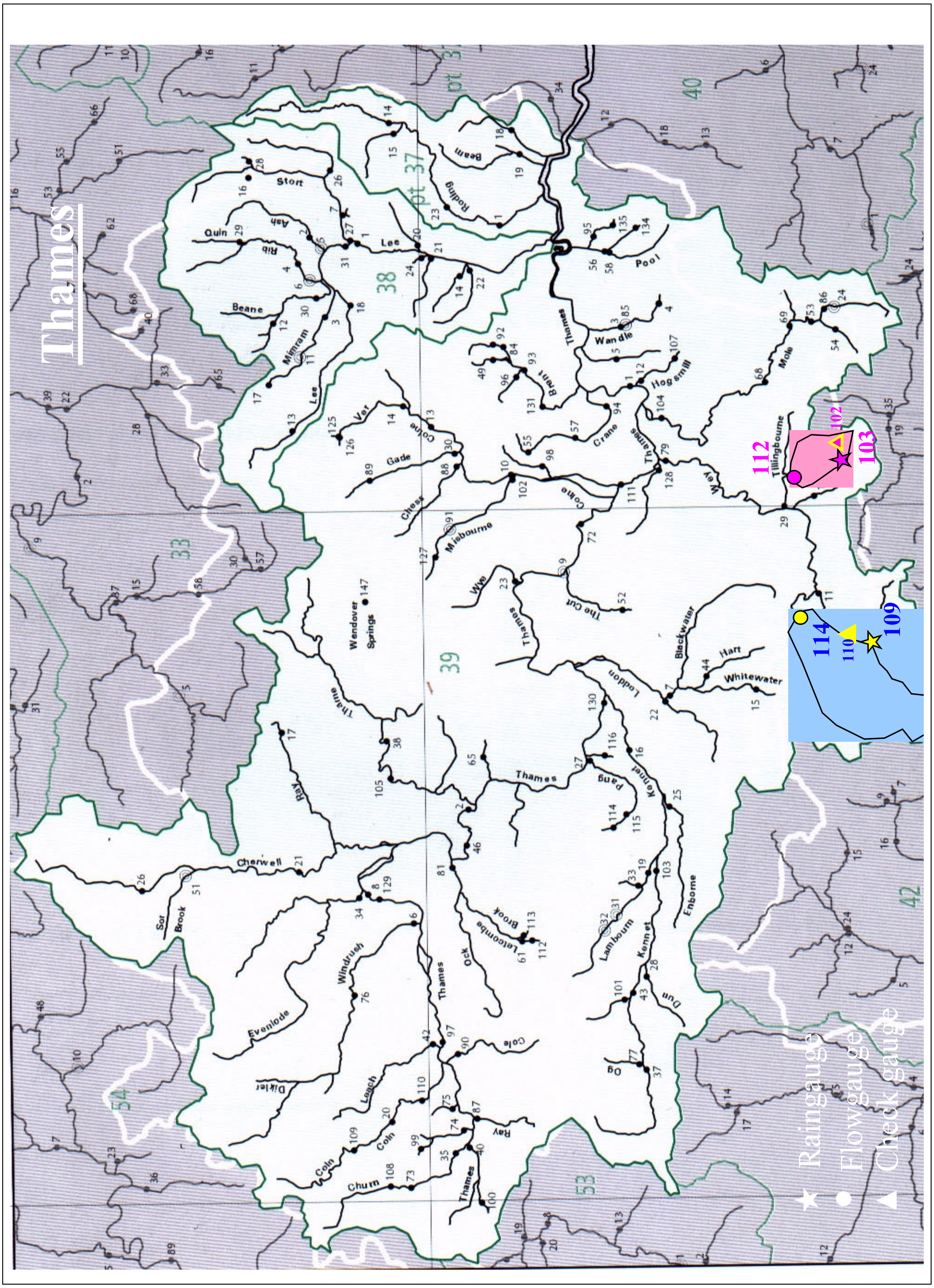



**Figure 7.3** **Raingauges at Bordon (109) and Cranleigh (103) for River Wey (114) and Law Brook (112) catchments**

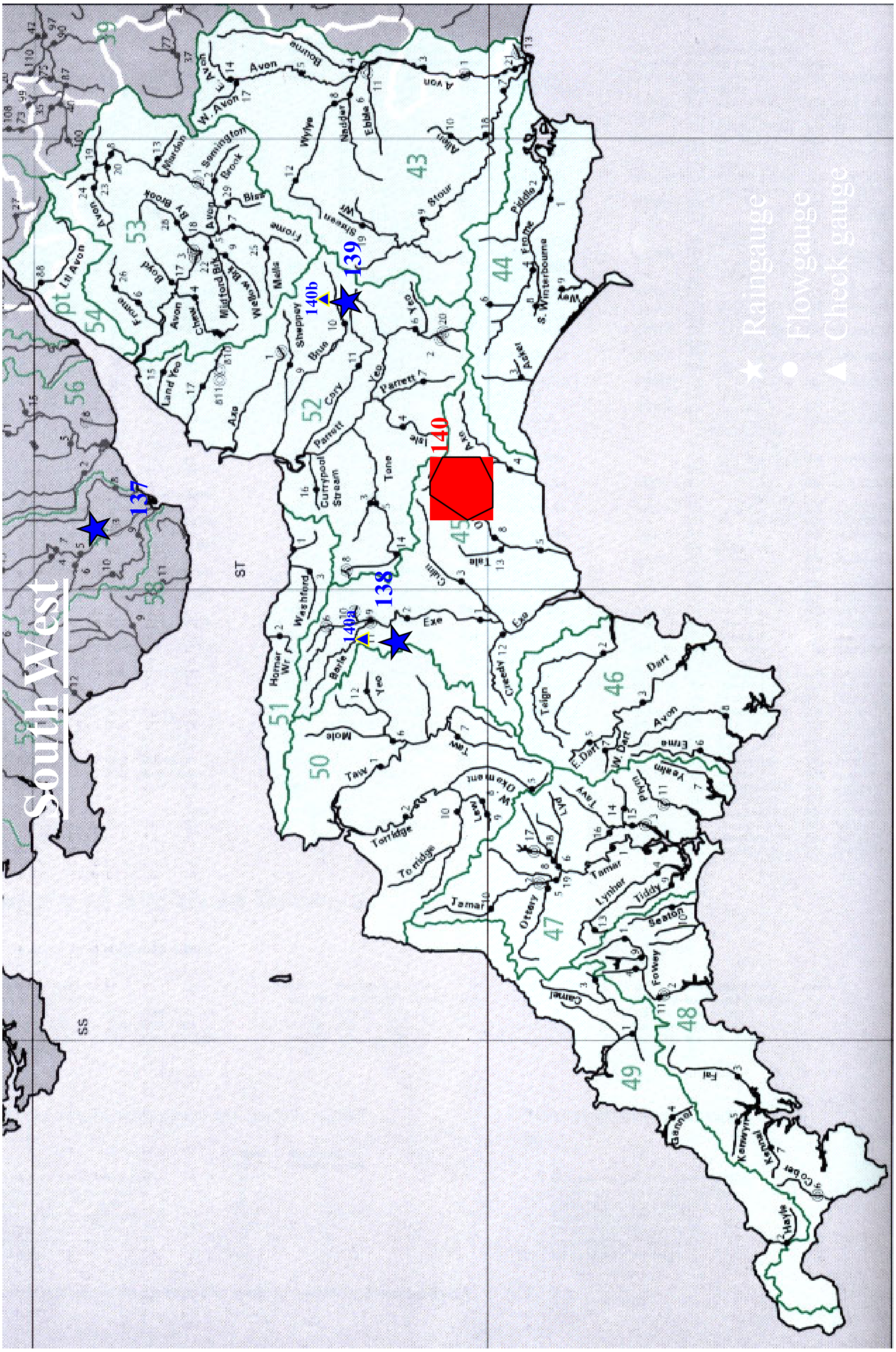



**Figure 7.4 Raingauge at Maundown (138) for River Isle (140) catchment**

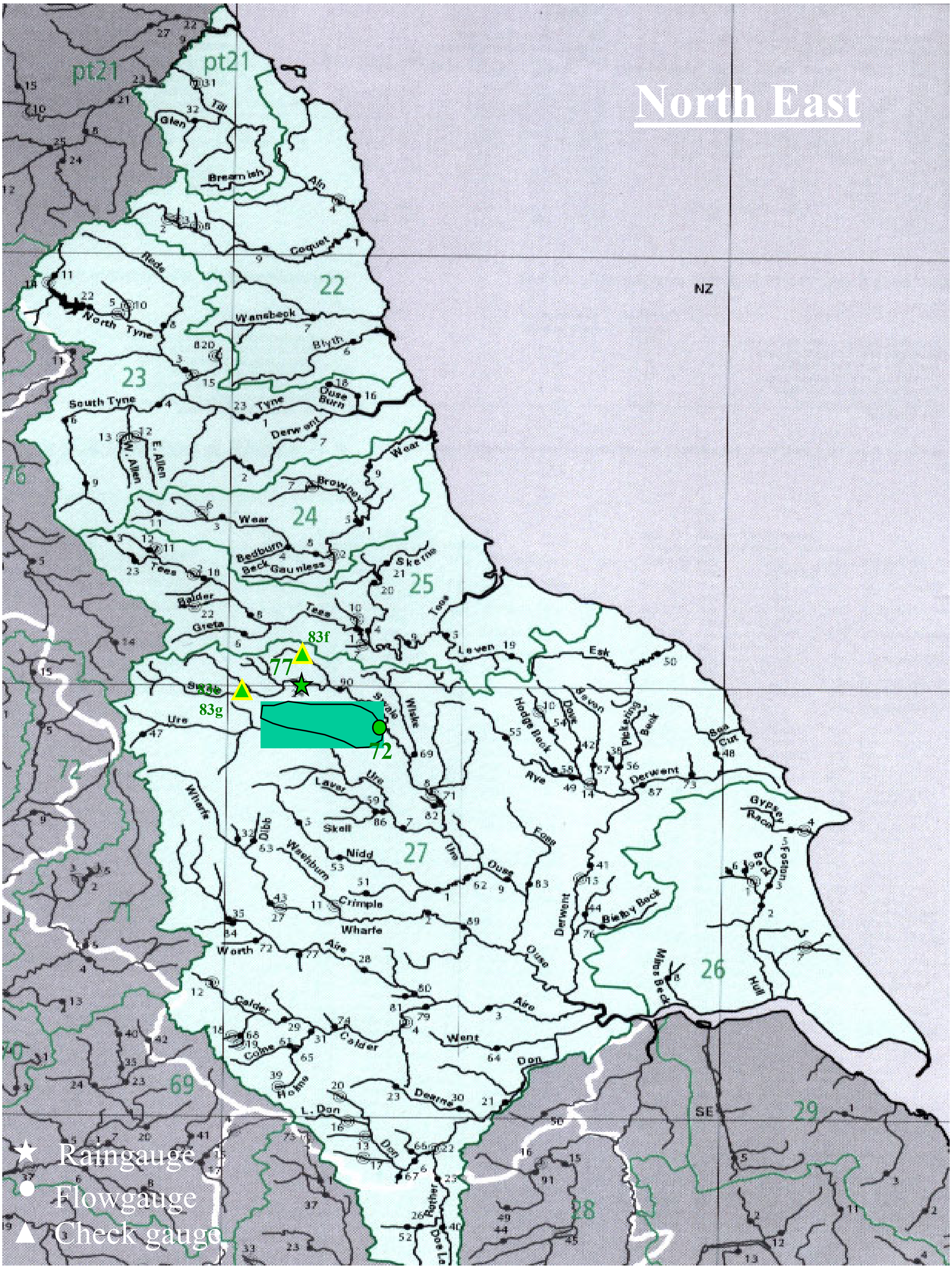



**Figure 7.5 Raingauge at Richmond (77) for Bedale Beck (72) river catchment**

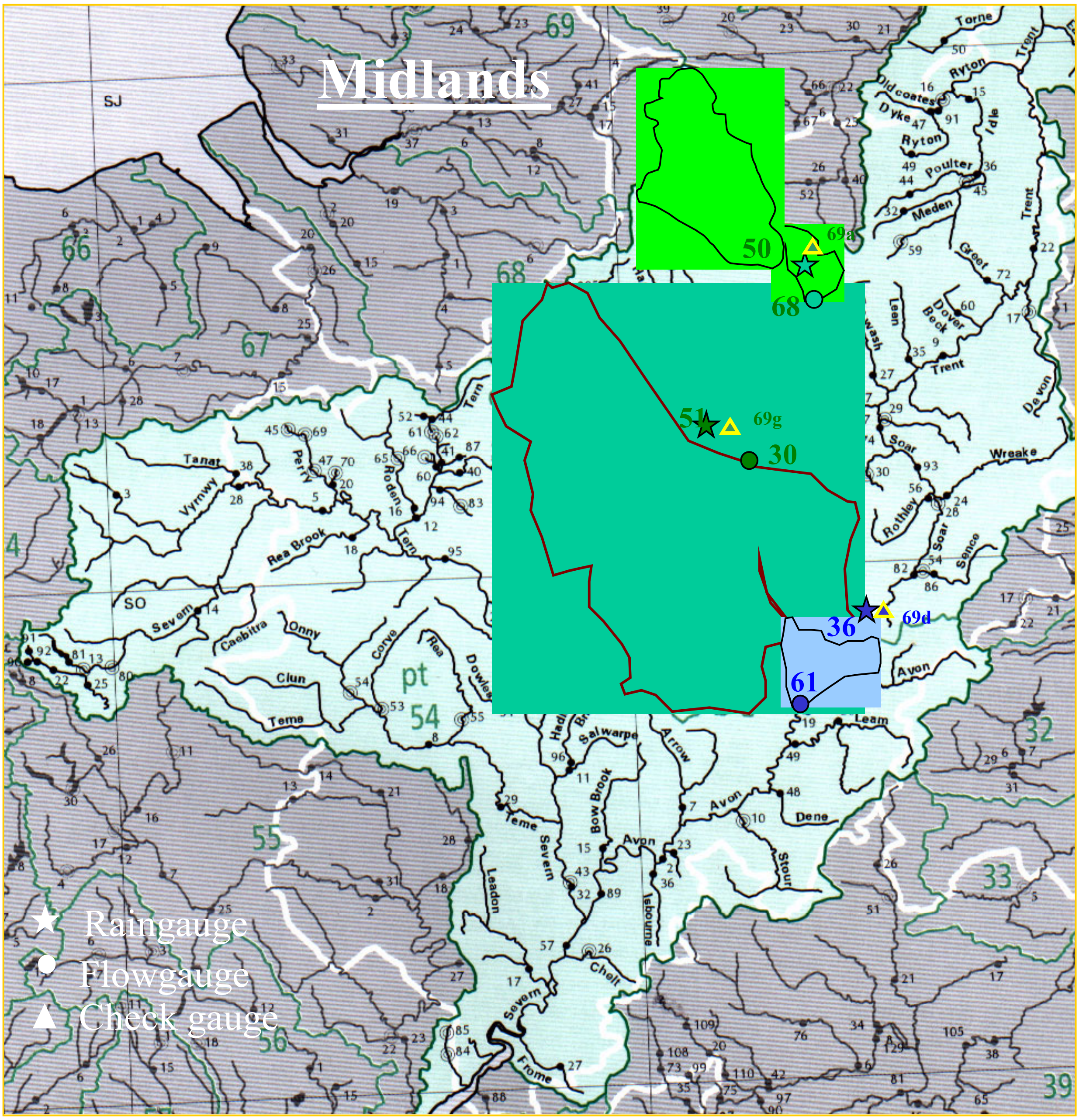



**Figure 7.6** **Raingauges at Hinkley (36), Ogstone (50) and Overseal (51) for River Sowe (61), Amber (68) and Trent (30)**

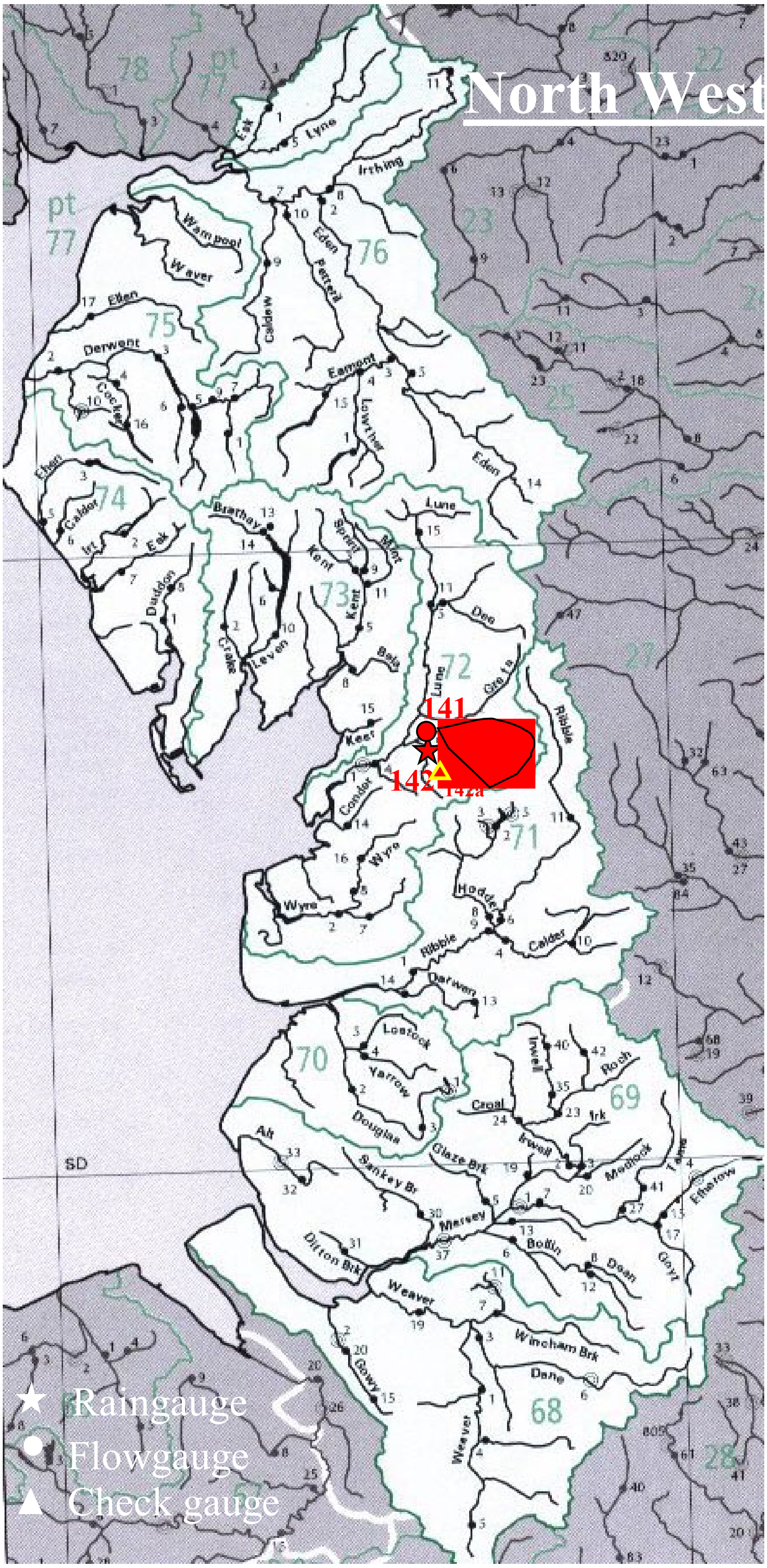



**Figure 7.7 Raingauge at Wennington Bridge (142) for River Wenning (141) catchment**

## 7.3 Approach used for determining the actual runoff control storage volumes

In order to be able to carry out an assessment of storage requirements, a realistic model of a theoretical development needed to be built together with the development of both the design and time series rainfall events.

### 7.3.1 Details of the Hydroworks model

For each of the ten selected catchments a Hydroworks computer model representing a simple drainage system for a 10ha development was produced. The Hydroworks models were used to run both observed and theoretical rainfall data for each of the ten catchments. The Wallingford Procedure new percentage runoff (New PR) model was used to generate runoff from the site. The Hydroworks model was used to assess the runoff volumes retained to attenuate runoff from the 10ha development site generated from the observed and design rainfall events for a range of throttle limits from 1 l/s/ha to 15 l/s/ha. The Hydroworks model for each site had the following characteristics:

- Pipe sizes were designed to run full for a 1 in 2 year 60 minute rainfall;
- The 10ha development site was divided equally into nine distinct areas each served by a separate
- manhole;
- The gradient of the drainage network was set at 0.0033 m/m;
- The pipe roughness k was set at 1.5 mm;
- The percentage impermeable area for the site was set to 50%;
- The new percentage runoff (New PR) equation was used;
- NAPI, the moisture depth parameter at the beginning of the storm was set to zero;
- PF was set to 0.20 m;
- The impermeability factor IF was set to 0.70
- The soil class for each of the ten sites was taken from the Wallingford Procedure maps.

The Hydroworks model was constructed with an orifice at the downstream end of the model. This orifice was used as a throttle to limit the discharge to the receiving watercourse. For each of the ten theoretical development sites, four different orifice sizes were used to produce throttle limits of 1 l/s/ha, 3 l/s/ha, 7 l/s/ha and 15 l/s/ha. The Hydroworks model was constructed with a side spill weir, with a crest level equal to the soffit level of the outlet pipe. This allows discharges exceeding the throttle value to overflow into a storage tank and the actual storage volume required to attenuate the runoff for the four throttle limits to be calculated. Figure 7.8 shows a schematic diagram of the Hydroworks model used to represent the theoretical development sites.

### 7.3.2 Derivation of time series rainfall events

Theoretically the model could be run with one continuous rainfall event of the data duration (up to 38 years for one location). However, the analysis required the assessment of storage volumes utilised during specific flooding events when the river water level was high (see Section 7.3.3). Running individual events also enabled the model to run more quickly, reduced computer storage requirements and made analysis of storage volumes easier.

Care had to be taken in defining what an "event" was. Where a storage tank has a limiting discharge of 1 l/s/ha, the time taken in draining the tank takes many hours. A check was made to establish that a 24 hour dry period would effectively empty a tank except for the most extreme of events. The assumption was therefore made that an "event" constituted all the rainfall between the start of an event and the first dry period of 24 hours. This resulted in quite long events but ensured that storage volumes were not affected by previous events. Secondly, to reduce the number of events, it was assumed that a rainfall event of less than 1.0mm was not worthy of assessment. Very little runoff, if any, would take place for an event of less than 1.0mm and storage would not be activated.

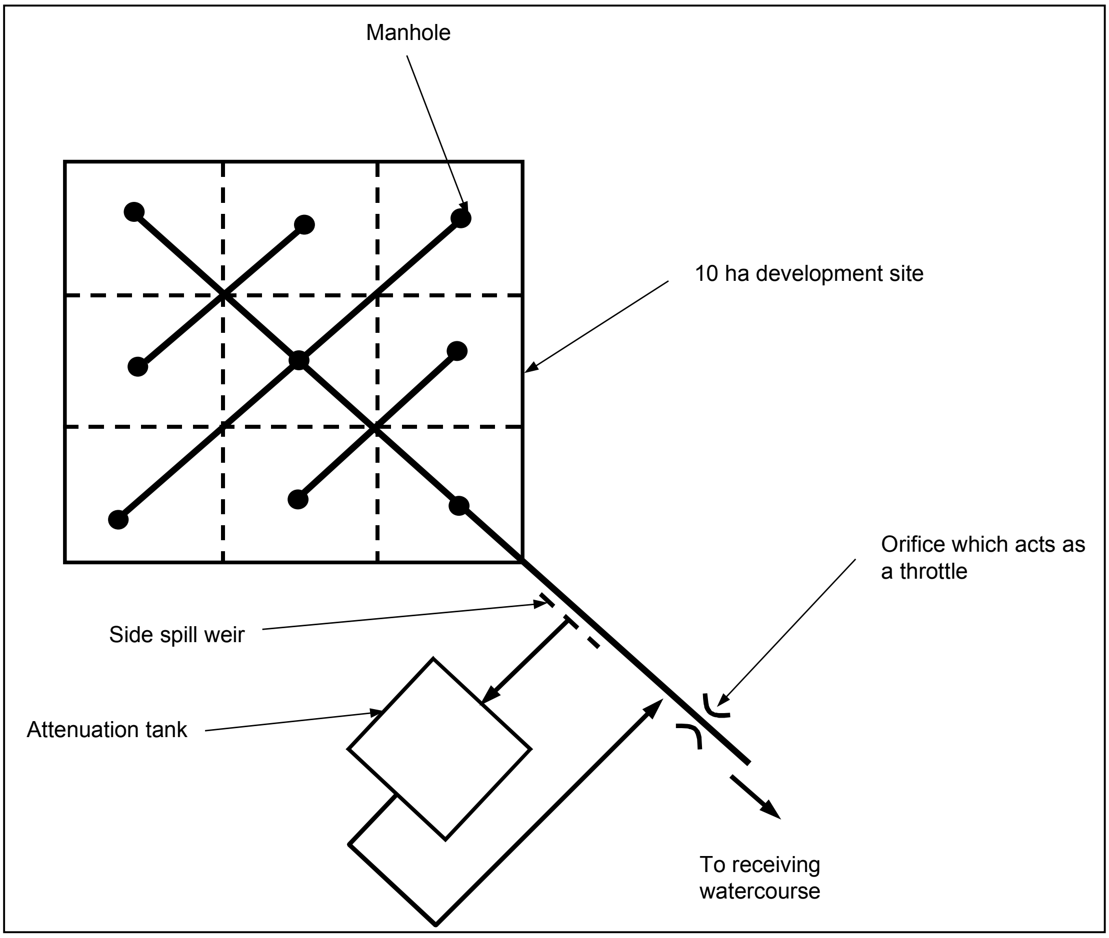



**Figure 7.8 Schematic diagram of the Hydroworks model for the 10 ha development site**

### 7.3.3 Selection of flood events

One of the main aims of the research was to test the effectiveness of providing throttled storage to protect the river during times of flood. To test this required the selection of periods when the river was in flood. The river hydrograph for the period available for each site was examined and the $Q_{10}$ value was determined. Initially all events which exceeded $Q_{10}$ were selected. However, as this resulted in a very large numbers of events, a higher threshold was used to limit the average number of events to between two and three per year. Table 7.3 provides a summary of the flood events selected and the flood threshold used.

**Table 7.3 Data sets used for each development site**

| Site number | Name of development site | Number of events from time series rainfall | Number of selected flood events | Period of flow data | $Q_{10}$ ($m^3/s$) | $Q_{90}$ ($m^3/s$) | Flow threshold ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| 1 | Hinckley | 2171 | 85 | 1978 to 1999 | 5.62 | 1.53 | 11.83 |
| 2 | Ogstone | 2139 | 114 | 1971 to 1999 | 27.47 | 5.49 | 39.01 |
| 3 | Overseal | 1763 | 79 | 1974 to 1999 | 64.67 | 15.87 | 119.69 |
| 4 | Bordon | 1077 | 49 | 1979 to 1999 | 1.36 | 0.21 | 2.81 |
| 5 | Cranleigh | 1015 | 44 | 1979 to 1998 | 0.14 | 0.07 | 0.18 |
| 6 | Crowland | 1503 | 92 | 1977 to 1998 | 0.50 | 0.001 | 1.08 |
| 7 | Oundle | 1225 | 51 | 1979 to 1998 | 1.39 | 0.42 | 3.10 |
| 8 | Richmond | 891 | 32 | 1985 to 1999 | 3.54 | 0.36 | 28.15 |
| 9 | Maundown | 1101 | 84 | 1967 to 1989 | 2.86 | 0.31 | 8.80 |
| 10 | Wennington | 557 | 29 | 1991 to 1999 | 10.20 | 0.33 | 33.67 |

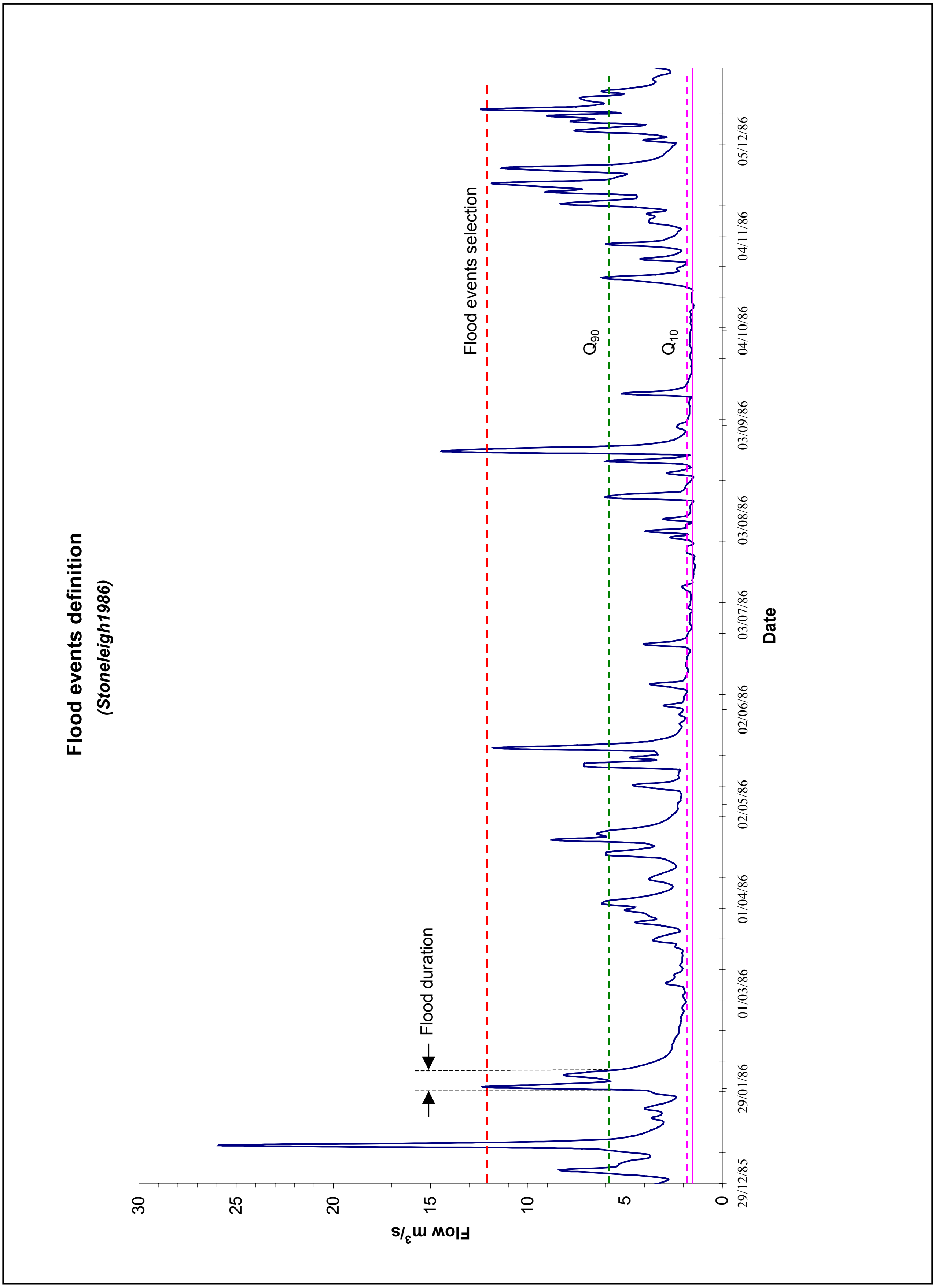



**Figure 7.9 Selection of flood events and flood periods**

Figure 7.9 illustrates the process of selection of flood events and the start and end of a flood based on the period for which flows are above $Q_{10}$.

Rainfall events were then related to each of these flood periods. If more than one rainfall file existed for the period of the $Q_{10}$ exceedence flow, these files were combined to provide a single event. If rainfall was taking place at the time when the flow in the river rose above $Q_{10}$ then the event was assumed to start at the beginning of that rainfall event. If it was not raining, the analysis commenced at the time of the start of rainfall after the start of the flood.

### 7.3.4 Derivation of design events

A secondary part of the analysis was to find the relationship between the storage volumes generated using design events compared to observed data. There were several reasons for doing this including:

- To carry out a comparison between the design storage volumes generated using observed and design rainfall;
- To estimate the storage volumes actually needed for extreme flood events;
- To investigate the effect of using critical duration events for assessing storage volumes.

A brief description of these analyses is given below. The design rainfall events for each theoretical development site were derived using the parameters obtained from the Wallingford Procedure maps. The various parameters are detailed in Table 7.4.

**Table 7.4 Site characteristics used in the analyses**

| Site number | Flow gauge name | Rain gauge name | Catchment time to peak (hours) | Soil type | M5-60 rainfall (mm) | r | Winter urban catchment wetness coefficient (UCWI) |
|---|---|---|---|---|---|---|---|
| 1 | Stonleigh | Hinckley | 14.2 | 4 | 20 | 0.34 | 125 |
| 2 | Wingfield Park | Ogstone | 5.0 | 1 | 20 | 0.35 | 125 |
| 3 | Drakelow Park | Overseal | 26.3 | 2 | 19 | 0.40 | 110 |
| 4 | Farnham | Bordon | 15.6 | 4 | 19 | 0.40 | 124 |
| 5 | Albury | Cranleigh | 7.7 | 4 | 19.5 | 0.35 | 125 |
| 6 | Kate Bridge | Crowland | 8.8 | 4 | 19 | 0.35 | 125 |
| 7 | Fotheringhay | Oundle | 20.1 | 1 | 19 | 0.42 | 122 |
| 8 | Leeming | Richmond | 27.9 | 4 | 19 | 0.40 | 124 |
| 9 | Ashford Mill | Maundown | 12.9 | 2 | 19 | 0.35 | 125 |
| 10 | Wennington Bridge | Wennington | 6.6 | 4 | 19 | 0.15 | 130 |

**Comparison between observed and design rainfall**

Theoretically the predicted storage requirements for a particular return period for a design event should relate to the exceedence probability of the results of an analysis of real data. The fact that they do not always agree is perhaps more a reflection on the accuracy of the design rainfall parameters than any intrinsic inappropriateness of the use of design storms (assuming the statistical basis of the comparison is reasonable). It therefore draws attention to the fact that using design events is only an approximation of the rainfall for a particular location.

**Extreme events**
Unfortunately, as for most recorded real data, this is only available for a limited period of time. The design events allow a theoretical prediction of storage needs for events up to 200 years return period. This means that design for extreme events will always need to be used for this type of analysis.

**Critical duration events**
A range of both winter and summer design events was created. The durations ranged from 1 hour to 20 hours for return periods ranging from 1 in 1 year to 1 in 200 years. In practice the winter event produced larger volumes, so the summer events were not used. In addition, an event with a duration equal to that of the flow hydrograph's time to peak, Tp, was derived for each catchment as an additional comparison to the results of storage required by the flood events.

**Derivation of the catchment time to peak, Tp**
As stated in 7.3.4, rainfall events of duration Tp were produced to provide data to compare with storage required by the flood events. The logic for carrying out this comparison was based on the principle that the type of event which caused the river to flood would be of a similar duration to Tp. The value of Tp is not quite the same as the time of concentration for designing drainage, but it is sufficiently similar to investigate the relationship that might exist. The calculation of Tp was carried out as defined in the Flood Studies Report. The values of Tp derived for the ten catchments are given in Table 7.4.

## 7.4 Overview of the hypotheses tested

In general the currently accepted methodology for arriving at a storage volume for a site is to choose a return period for the level of protection required and then carry out a critical duration analysis on the site network using design storms. This enables the derivation of the maximum storage needed to achieve a particular limiting discharge.

The principal reason for asking for this requirement is that it is assumed that the reduced flow rate attenuated over a longer time would protect the river, particularly in preventing flooding from getting any worse. This has recently been taken to extremes to try and reduce flood peaks by requiring post-development 1 in 100 year runoff to be attenuated to 1 year pre-development runoff. However, testing of this hypothesis has not been carried out until now to try and justify and measure the value of using limiting discharges.

This research is therefore aimed at trying to measure the benefit of this requirement. The key features of the differences between pre- and post-development of an area are the changes in the rate of runoff and volume of runoff. In addition the number of events that have a 'response' to the river increases from only 'big' events or events in wet periods to effectively all events. Using time series to assess storage is usually avoided due to the effort involved and limitations of data sets, and therefore design events are used.

The following aspects were therefore investigated for a range of limiting discharges:

- Storage volumes using "all year" time series rainfall compared to "flood event" series;
- Storage volumes using "all year" and "flood events" compared to critical duration design storms;
- Storage volume requirements for "flood event" series compared to Tp critical duration design events;
- Effectiveness of storage retention during flood events; at the peak of the flood and at the end of the flood;
- Measurements made in terms of proportion of runoff retained and compared to predicted FSSR 16 volumes;
- Comparison between the use of the Old PR equation and the New PR equation;
- Comparison between the use of summer and winter profile design events;
- An investigation was also carried out on the number of events per month by rainfall depth.

### 7.4.1 Storage volumes using "all year' compared to "flood events"

The hypothesis for this analysis was that flood events in rivers were due to long low intensity rainfall events. It was thus expected that the results of the "all year" series of rainfall should demonstrate higher volumes of storage than the "flood event" series. The "all year" series of rainfall were expected to generate only marginally higher volumes for very heavily constrained limits of discharge (1 l/s/ha) and larger differences for more relaxed limits of discharge (7 l/s/ha).

If this hypothesis were proven, it would indicate that the use of the critical duration event to determine site storage volumes to protect the river in flood would only be appropriate if the critical duration was long. It would also demonstrate that storage volumes demanded by the regulator were inappropriate for river flood protection except where throttle limits were very tight.

Owing to the computational effort required to do "all year" series analysis, this test was only carried out at 3 l/s/ha. However, the close relationship that should exist between all year and critical design storms allows this comparison to be made at other throttle rates by inference.

### 7.4.2 Storage volumes using "all year' and "flood events" compared to critical duration design storms

The hypothesis made is that design storms generated using the Flood Studies Report rainfall parameters should closely reflect the "all year" series and predict greater volumes than the "flood events". This difference would be small for tight throttles and very much larger for higher throttle rates that would have critical durations of only two to four hours.

This test was used to support Test 1 (as discussed in 7.4.1) to reduce the number of runs of the "all year" series. In addition it allows an assessment to be made on the accuracy of using Flood Studies Report rainfall parameters to predict the same storage volumes for an equivalent return period of a real series. Owing to budgetary constraints, the finer detail of analysing the exact values of NAPI for each event and what it should be when using design events was ignored. The differences demonstrated in the results are more a function of other issues such as the accuracy of the Flood Studies Report parameters, climate change and the statistics of making comparisons.

### 7.4.3 Storage volume requirements for "flood events" series compared to Tp critical duration design events

The hypothesis made is that the "flood events" should closely reflect the rainfall events that caused the river to flood which are generally caused by longer events. Again this is the same argument being tested as discussed in 7.4.1, but provides added support to the argument of using Tp duration events rather than site critical duration events if river flood protection is the principle driver for site storage of surface water runoff.

### 7.4.4 Effectiveness of storage retention during flood events

The principle reason for demanding storage retention is try and prevent runoff from entering the river to exacerbate flooding. To achieve this, sufficient water should be retained in storage during the flood period. A measure of what is "sufficient" can be done in two ways:

1. The proportion of runoff retained in storage compared to total runoff.
2. The proportion of runoff in the river is no greater than the greenfield runoff volume which would have been generated.

These measures can be made at the peak of the flood flow in the river and also at the end of the flood event, If these tests demonstrate that "sufficient" on site water retention is not being achieved, then the concept of temporary storage by using throttles should be reconsidered.

A significant limitation of the measurements made for part two of this test is the assumption that the greenfield runoff volume as predicted using FSSR 16 and the developed runoff volume are directly comparable. In practice the rate of runoff is very different and due allowance should be made in evaluating the results. Therefore if the test shows that no more water is in the river after development, the result would seem to indicate that the development was no worse than prior to construction. In practice the rate of runoff from greenfield sites is an order of magnitude slower. Thus the test favours the development in the analysis and is not conservative. However, if they fail the test of "sufficiency" posed above, the concept of throttled storage is very effectively proven as being inadequate. The effort needed to carry out a greenfield hydrograph volume analysis for specific points in time, although desirable, would be both time consuming and not particularly accurate.

### 7.4.5 Comparison of the Wallingford Procedure Old and New PR equations

This test is a relatively minor element of the research. This analysis was only carried out using design rainfall events. The reason for doing this comparison is that the Old PR equation is still the most established runoff model in spite of its limitations as discussed in Chapter 6. However, there is no consensus yet in the water industry of the design parameters that should be used in the New PR equation, particularly the design (starting value) of NAPI. This test is not systematic in being applied to all sites and it is not relevant to spend too much time on it in this research. However, the value of doing it is the demonstration of the fact that very similar runoff volumes are generated by the two equations, but with runoff volumes increasing by a small amount for the larger events with the New PR equation.

This analysis was only carried out at the 3 l/s/ha throttle rate. The Old PR equation uses the term UCWI and this was applied as a value between 110 to 130 depending on the location of the site due to its relationship with SAAR.

### 7.4.6 Comparison between summer and winter design events

Similar to the test on the runoff models, a check was made on determining whether the winter event generates more storage than the summer event. The Old PR equation calculates a higher value of PR as UCWI is around 20% greater. However, it is not immediately obvious why the New PR equation would generate greater volumes as NAPI is assumed as being zero at the start of either the winter or summer event (even though it is likely to be higher in winter). The difference is shown to be marginal.

### 7.4.7 Number of rainfall events

An analysis of the number of rainfall events was made by month. On the basis that the greenfield site does not produce runoff for smaller events, particularly in summer, an investigation was made into the effectiveness of providing interception storage to replicate this behaviour. It was hypothesised that summer events are generally smaller and shorter and that the provision of interception storage (prevention of runoff getting to the river) might dramatically reduce runoff, particularly in summer. This is particularly desirable due to the higher pollution loads in the runoff and the lower flows occurring in the river. It is recognised that the basis of the definition of an event as described in 7.3.2 is not necessarily the most suitable for this test, but it is considered acceptable to carry out this brief analysis.

### 7.4.8 Greenfield runoff rates for sites and catchments

The philosophy of approach in interpreting the concept of sustainability in this context is to try and ensure pre and post development runoff is the same. However, there is a trend in some areas to try and use average total catchment flow rates per unit area as a basis for limiting storage off a site into that catchment. A brief analysis of the ten sites has been made to compare the predicted site runoff rate with the catchment upstream for each of the ten locations.

# 8. RESULTS OF THE ANALYSIS

The results of the various tests carried out are described in this chapter. Extensive supporting graphical representations of the results for each site are provided in the appendices.

## 8.1 Results of storage volume requirements analysis

The Hydroworks model was run for all the rainfall series and design events as described in the Section 7.4 of this report. The model results were processed to summarise the following:

- Total runoff from the development site;
- Maximum volume of water stored in the attenuation tank;
- Runoff discharged to the receiving watercourse;
- Volume of water in the tank at the peak of river flow and at the end of the rainfall event;
- Date of the flow peak in the receiving watercourse;
- Date of the end of the rainfall event.

The return period analysis for the storage volumes for each site was based on a volume exceedence frequency. Graphs showing the attenuation volume required for each observed rainfall event against time were produced for each of the four throttle limits. The complete set of graphs comparing all the storage analysis requirements are presented in Appendix B. It should be noted that for many of the sites analysed with a throttle value of 15 l/s/ha that these did not yield any attenuation storage volume i.e. the runoff from the development site was less than 15 l/s/ha.

### 8.1.1 Storage volumes using “all year’ compared to “flood events”

By inspection of the results it can be seen that the hypothesis that the “flood events” require less storage for any given exceedence probability (or return period) than the “all year” rainfall proved to be true. This result is seen to be consistent across the 6 sites analysed. It is unfortunate that the “all year” series was not run for the other three throttle limits as it would have been interesting to note how the volume relationship between the two series changes for different discharge limits. However, as design events should relate closely to the “all year” series, this comparison can be made indirectly. Figure 8.1 summarises the relationship of all the test catchments showing the ratio of “flood event” storage to the “all year” series set of events. The variability of the data is partly due to the smaller number of the flood events data set compared to the all year set. However it is clear that at 3 l/s/ha, storage utilisation is around 75 to 80 percent. It is surmised the results would converge towards 100 percent for tighter throttles and the higher the return period of the event. Conversely the ratio would significantly reduce for higher throttle rates and lower return periods.

A detailed analysis of the results looking at the effect of other parameters such as network configuration, catchment size, Tp and regional rainfall characteristics has not been done. It is suspected that the smallest differences in storage volume ratio are achieved in westerly catchments where the 5 year, 60 minute to 2 day rainfall ratio is low. However there is insufficient information to be able to carry out this type of analysis. The results do show however that “flood events” require less storage, for any return period, than an equivalent protection against site flooding for “all events”.

### 8.1.2 Storage volumes generated by “flood events” compared to design storms

One of the key objectives of this research was to compare actual storage utilisation during “flood events” to that which would have been designed using design storms based on the critical duration event. As with any data set of a number of years, there is a high probability that the return period of one or two of the events is actually significantly more extreme than that of the duration of the data set. However, if a frequency exceedence analysis is used, an approximate comparison can be made between the results of the “peaks over threshold” data set of the “flood events” and the results of the design events.

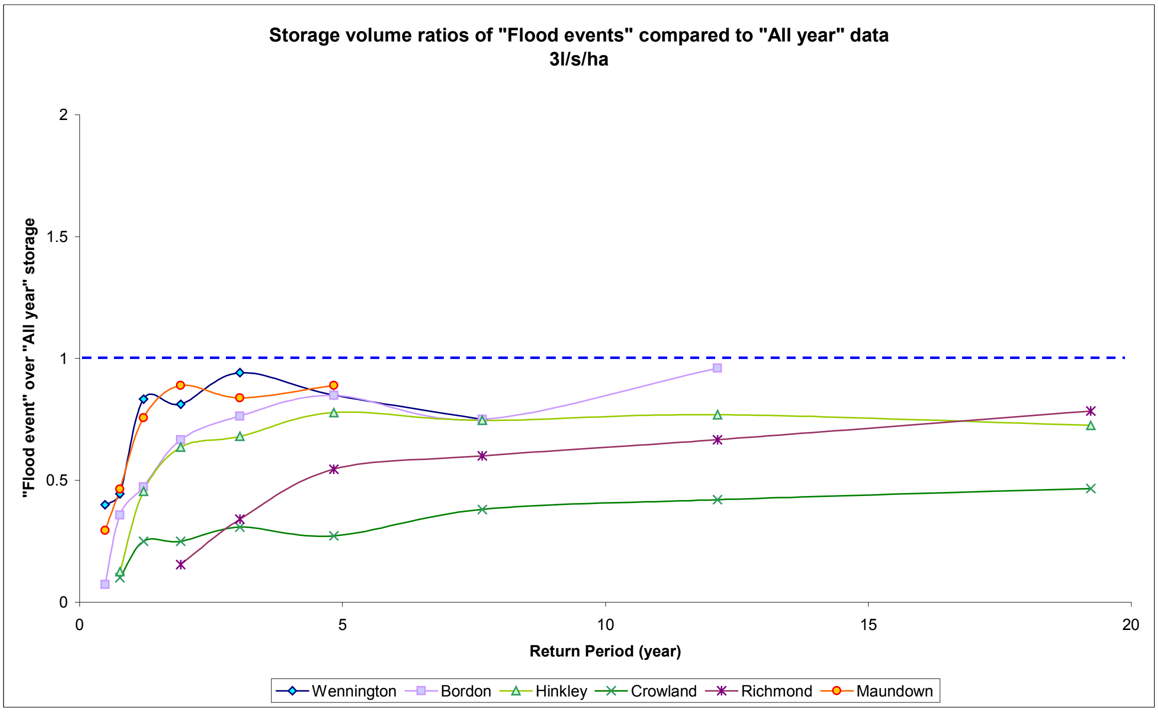



**Figure 8.1 Storage volume ratios of "flood events" compared to "all year" data**

The relationship between the results can be seen to be a function of the throttle limit used. It can be seen (allowing for the variability of the various catchments) that design storms produce approximately the correct volume of storage for very restrictive throttles, but as throttles become less restrictive, the use of critical duration design storms are not appropriate and seriously over estimate the storage volume required. The assessment was also carried out for 15 l/s/ha but no storage was found to be needed for the flood events.

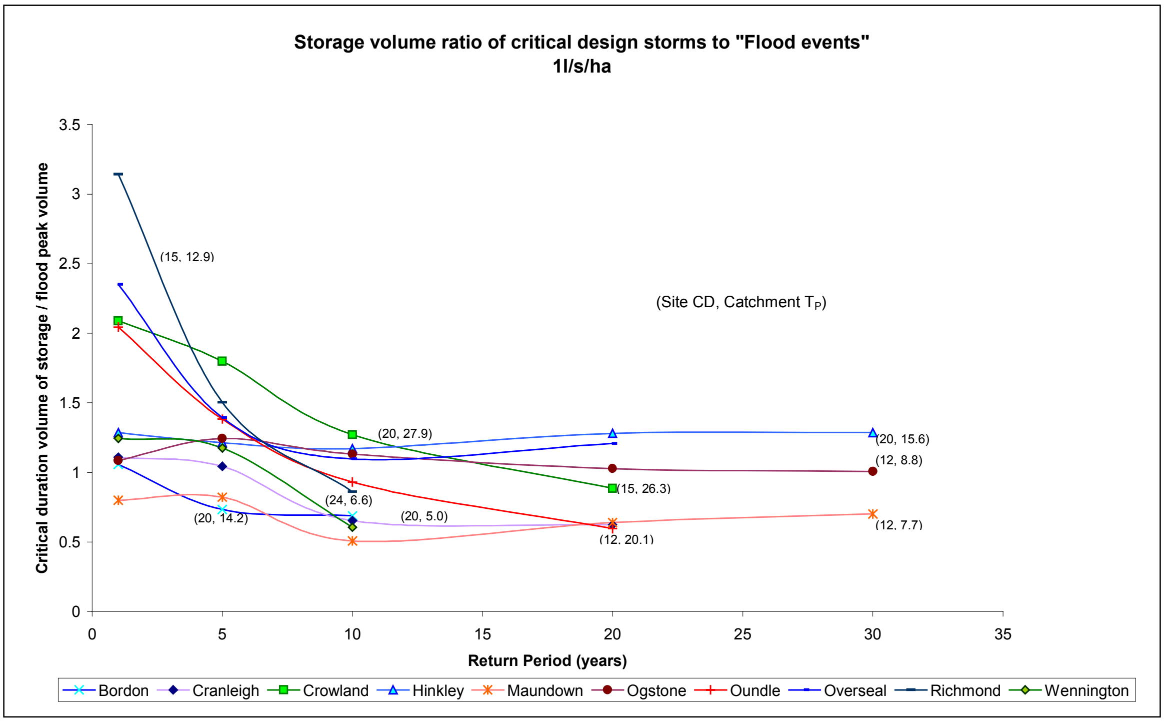



**Figure 8.2a Storage volume ratio of "flood events" to critical design storms - 1 l/s/ha**

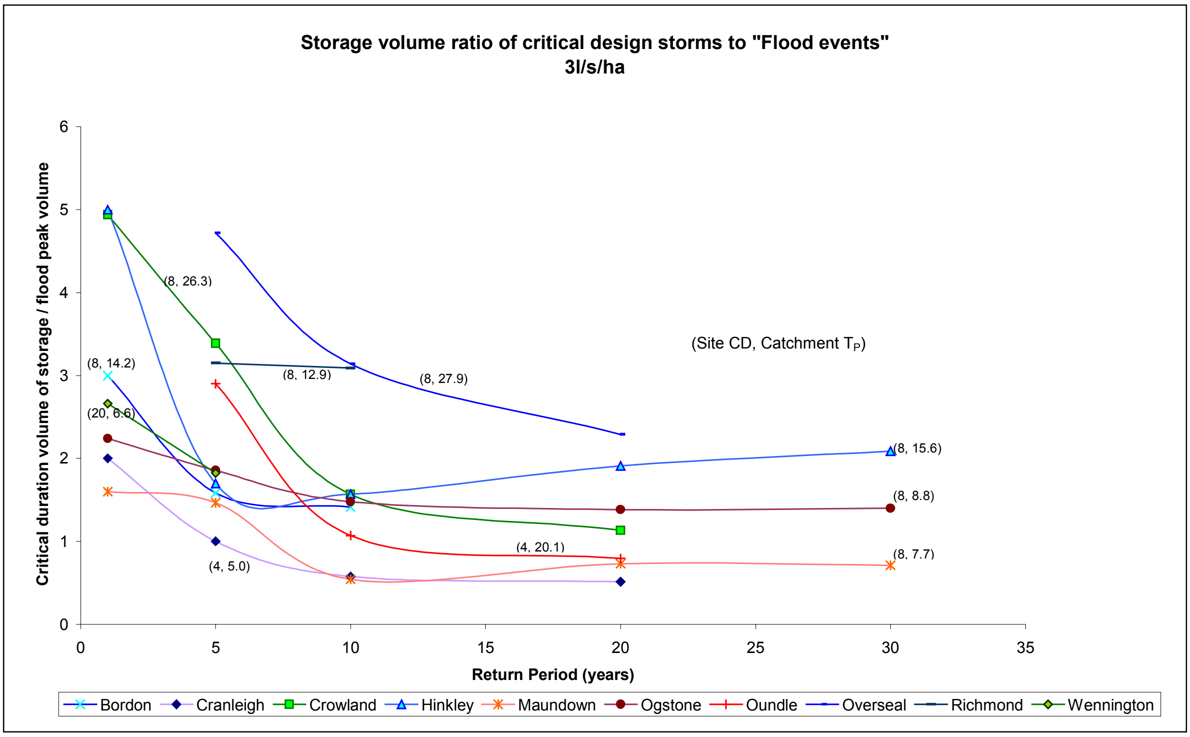



**Figure 8.2b Storage volume ratio of "flood events" to critical design storms - 3 l/s/ha**

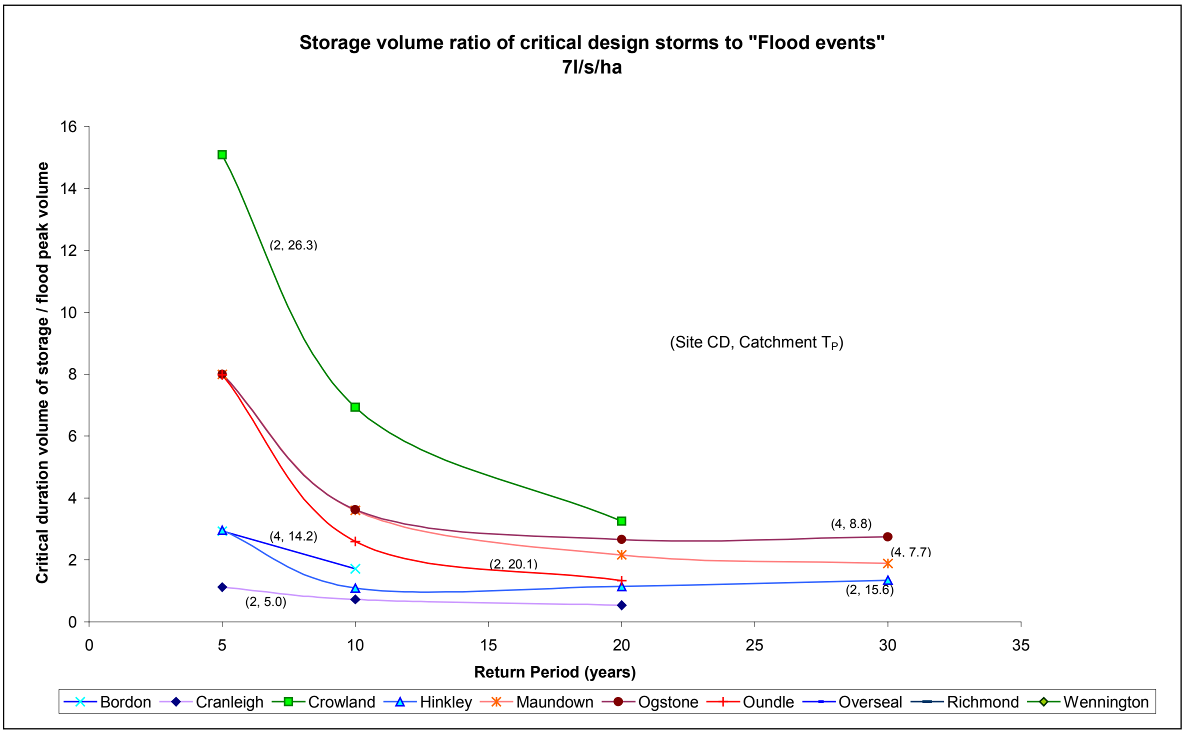



**Figure 8.2c Storage volume ratio of "flood events" to critical design storms -7 l/s/ha**

Each of the plots gives the site location, the critical duration and the duration of Tp for the catchment.

It can be seen that the critical durations for the low limit of discharge are long and are similar to the value of Tp. However, for larger limits of discharge, the critical durations are much shorter.

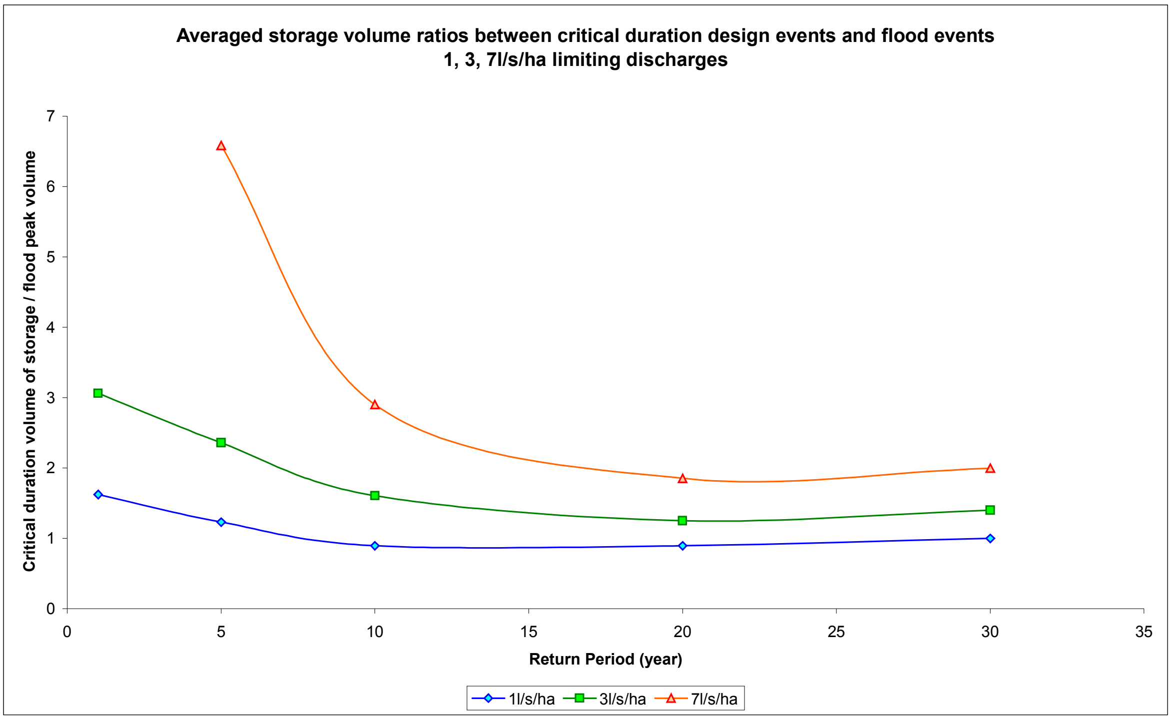



**Figure 8.3 "Averaged storage volume ratios between critical duration design events and flood events - 1, 3, 7 l/s/ha limiting discharge"**

It is fairly meaningless to try and derive a general relationship between the storage ratio and throttle limit as catchment size, its flood response characteristics and the accuracy of rainfall parameters and other issues affect the storage volume required. However, as an indication of the influence of throttle size, an average of the test site results has been produced and it provides a remarkably consistent visual illustration of the findings. Figure 8.3 gives the average storage ratio of design events to flood events as a function of throttle rate. This result suggests that the use of critical duration storms for assessment of storage to protect the river is only valid for very restrictive throttles and that an alternative approach would be more appropriate for less onerous throttles.

### 8.1.3 Storage volumes for flood events compared to Tp critical duration design storm events

Section 8.1.2 demonstrated that critical duration events should not be used to design for storage to protect the river against flooding. A test using Tp duration events was considered as possibly being more accurate. Figure 8.4 shows the relationship between the storage volume ratio of "flood events" to the Tp design storm.

As expected, the approximation of design storage using TP duration events more closely reflects the storage utilised by the "flood events" particularly for the larger throttle rates. As Tp is a fixed duration, it was found that the critical storage volume for the larger throttles was produced using the summer rather than the winter profile due to the higher rainfall intensity of summer events. As in Section 8.1.2, an average for all test sites is also plotted. Again it must be emphasised that Figure 8.5 is not to be used as a definitive relationship to be used for any catchment. However, it dramatically supports the idea that design events using Tp will more closely represent storage needs for any throttle rate if river flood protection is the objective by using temporary storage.

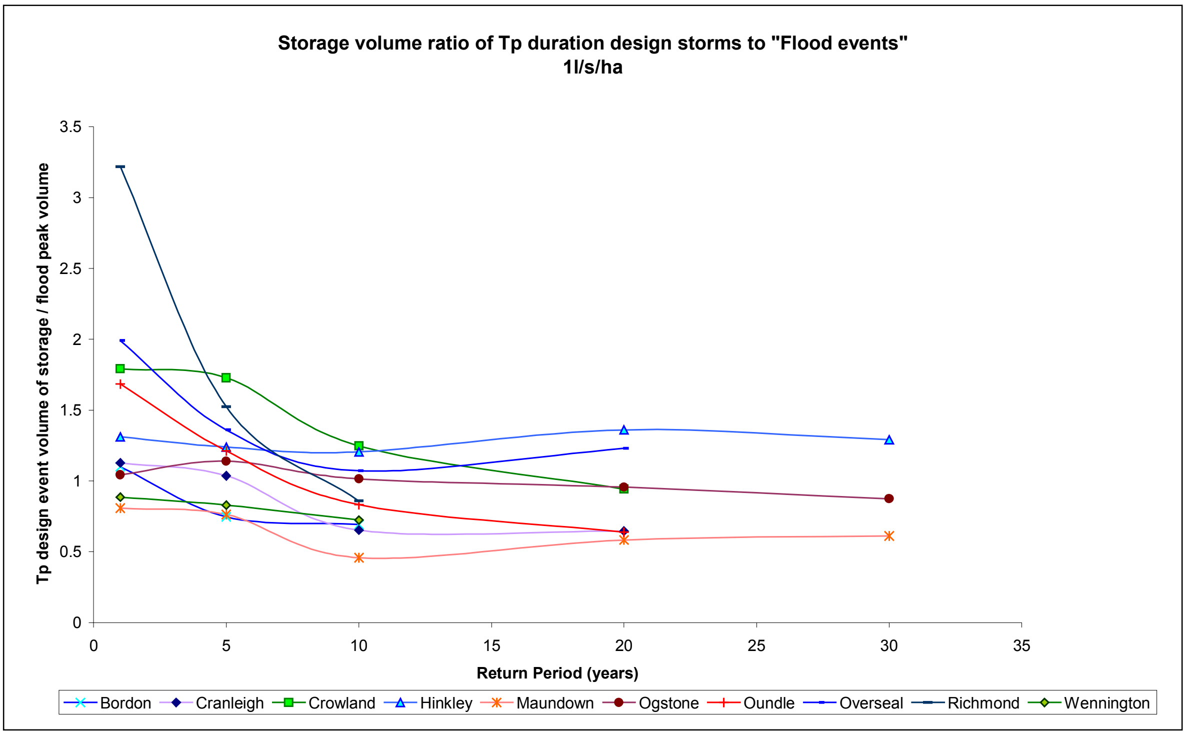



**Figure 8.4a** Storage volume ratio of 'flood events' to Tp duration design storms – 1 l/s/ha

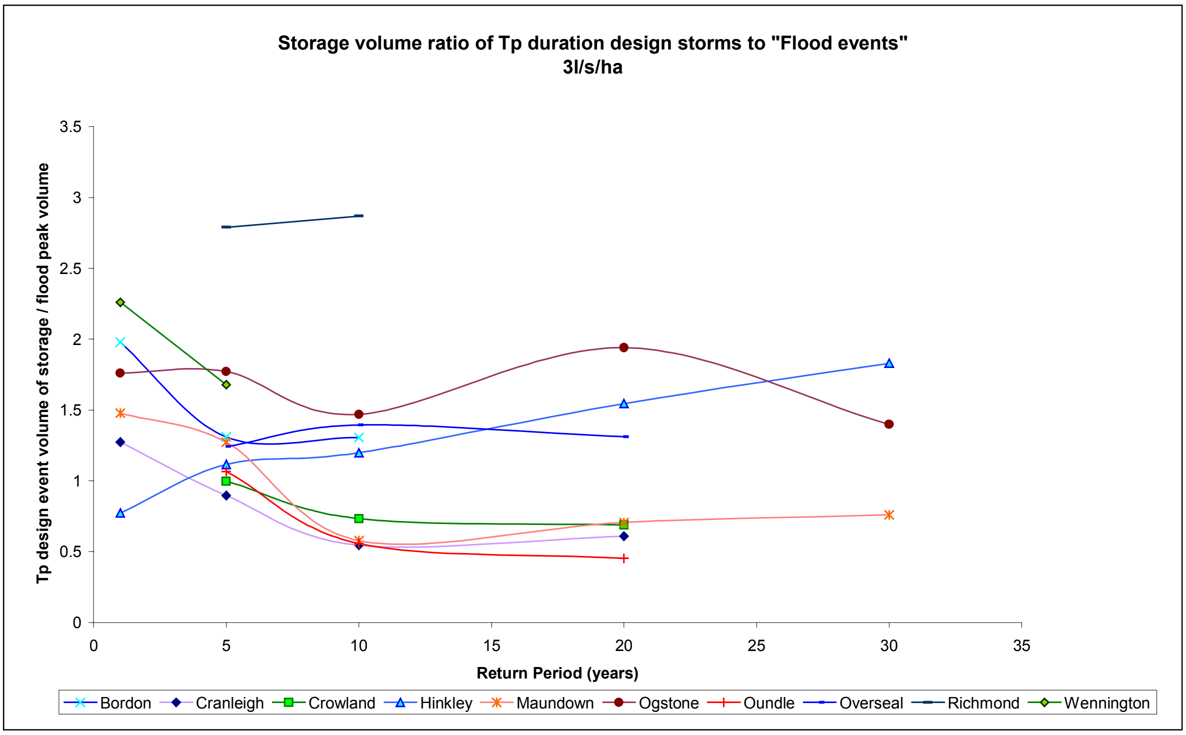



**Figure 8.4b** Storage volume ratio of 'flood events' to Tp duration design storms – 3 l/s/ha

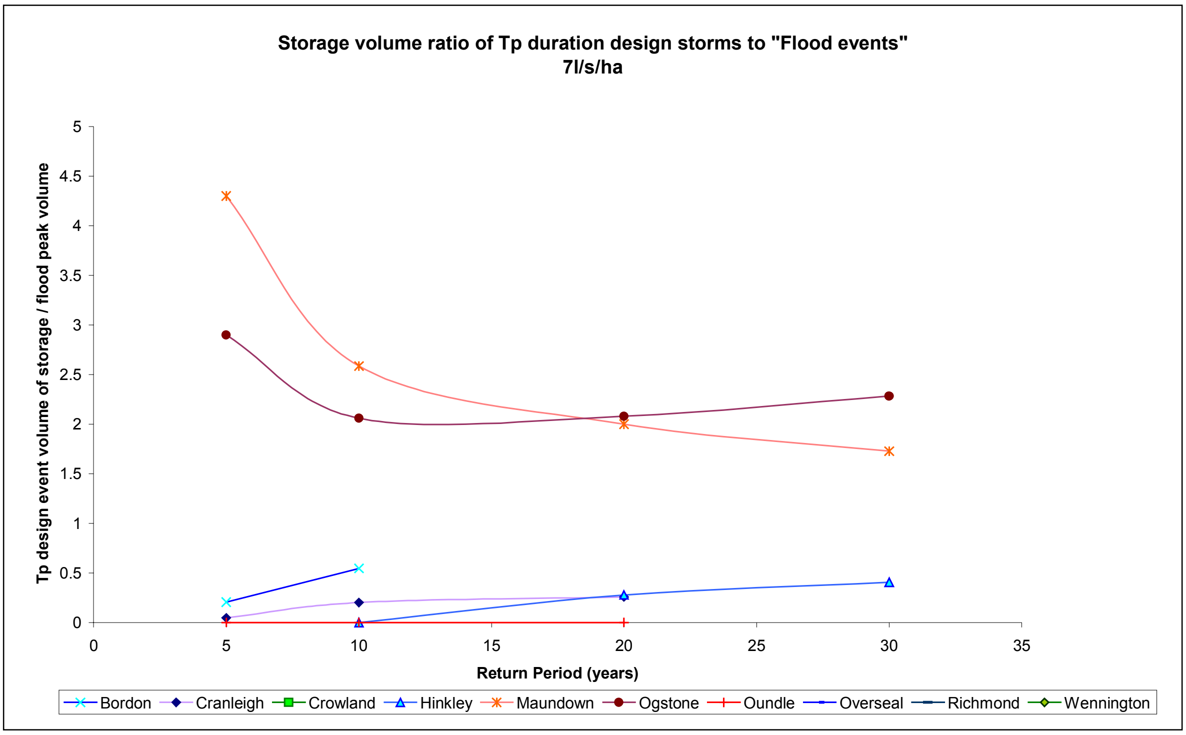



**Figure 8.4c** Storage volume ratio of 'flood events' to Tp duration design storms – 3 l/s/ha

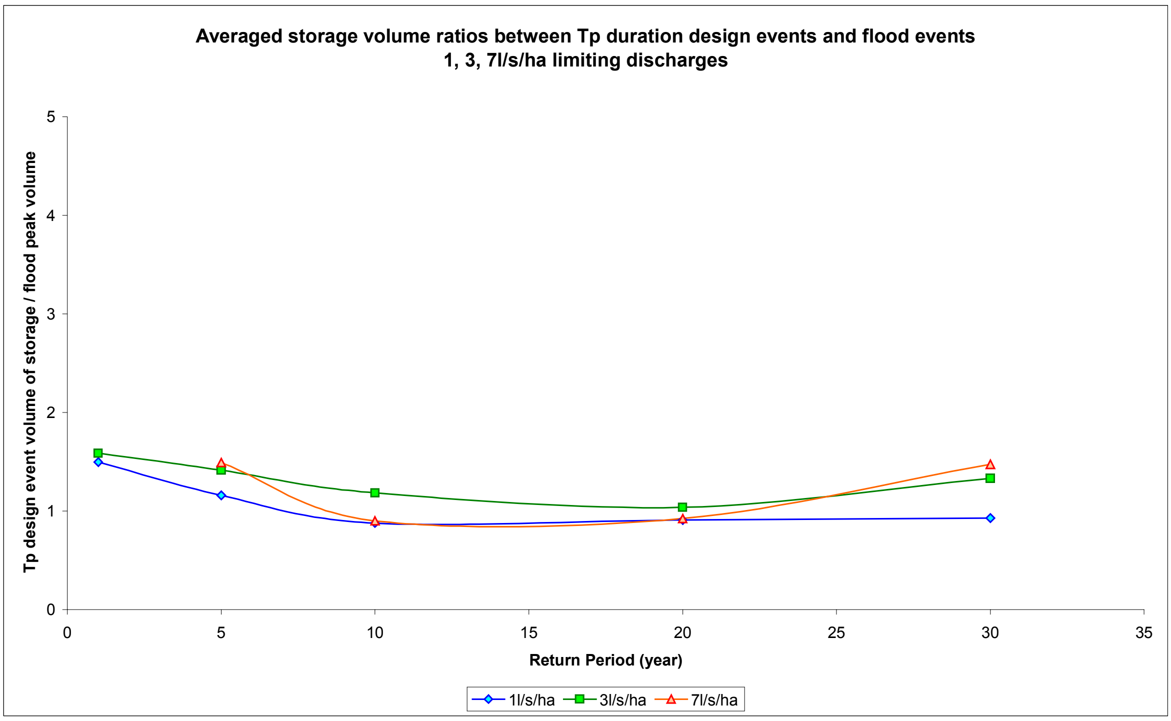



**Figure 8.5** "Averaged storage volume ratios between Tp design events and flood events – 1, 3, 7 l/s/ha limiting discharge"

A brief summary of the values of Tp comparing them to the critical duration events is given in Figure 8.6. This illustrates the fact that the critical duration is generally similar to Tp for very small throttle rates.

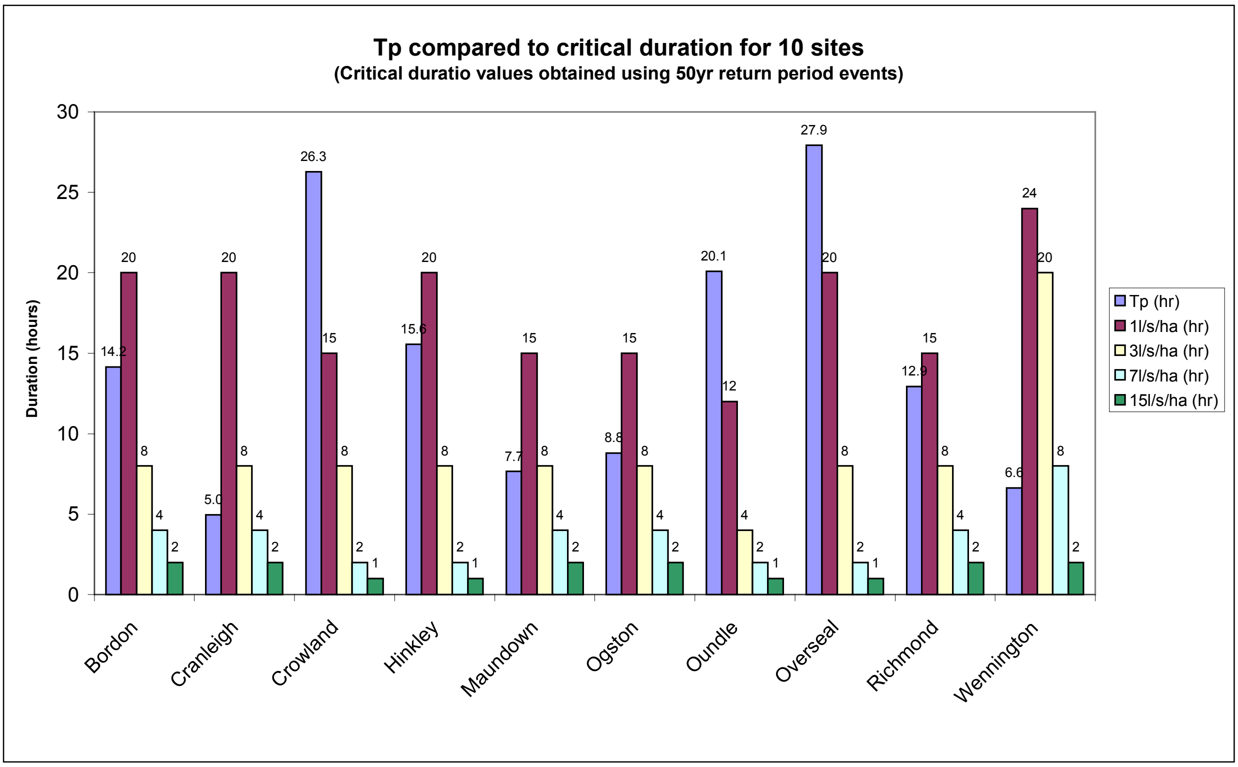



**Figure 8.6 Tp compared to critical duration for 4 throttle rates for ten sites**

## 8.2 Effectiveness of storage retention during flood events

Measuring the volume of storage needed for temporary storage to protect rivers from runoff from sites during flood events does not in fact test the effectiveness of the storage, but just determines what volumes would be required to meet a throttle criteria. The tests carried out to check the effectiveness of temporary storage are defined in Section 7.4.4. These were:

- The proportion of runoff retained in storage compared to total runoff; both at the peak and at the end of the flood events in the river;
- Relating the runoff volume in the river to the equivalent volume that would take place from the greenfield site.

The basis of the analysis is shown graphically in Figure 8.7.

### 8.2.1 Proportion of runoff in the river

The results of the analysis are shown in Appendix D. The information is ranked in two ways; the first in terms of the proportion of the volume stored compared to total runoff and secondly by magnitude of event. The result is startling in that the proportion of runoff stored at either the peak or the end of the flood event is effectively zero for the throttles of 3, 7 and 15 1/s/ha for all sites. At 3 1/s/ha around 10 percent of events utilise some storage at these times.

Only when 1l/s/ha is being used are there a significant number of events that show a useful amount of storage is being utilised at these key points in time. However, even with this tight throttle it is evident that the proportion of water being stored compared to the total runoff is often quite small. The reason for this is that the time delay of flood peaks together with the spatial effects and intermittent nature of rainfall is resulting in very little effective storage in terms of protecting the river.

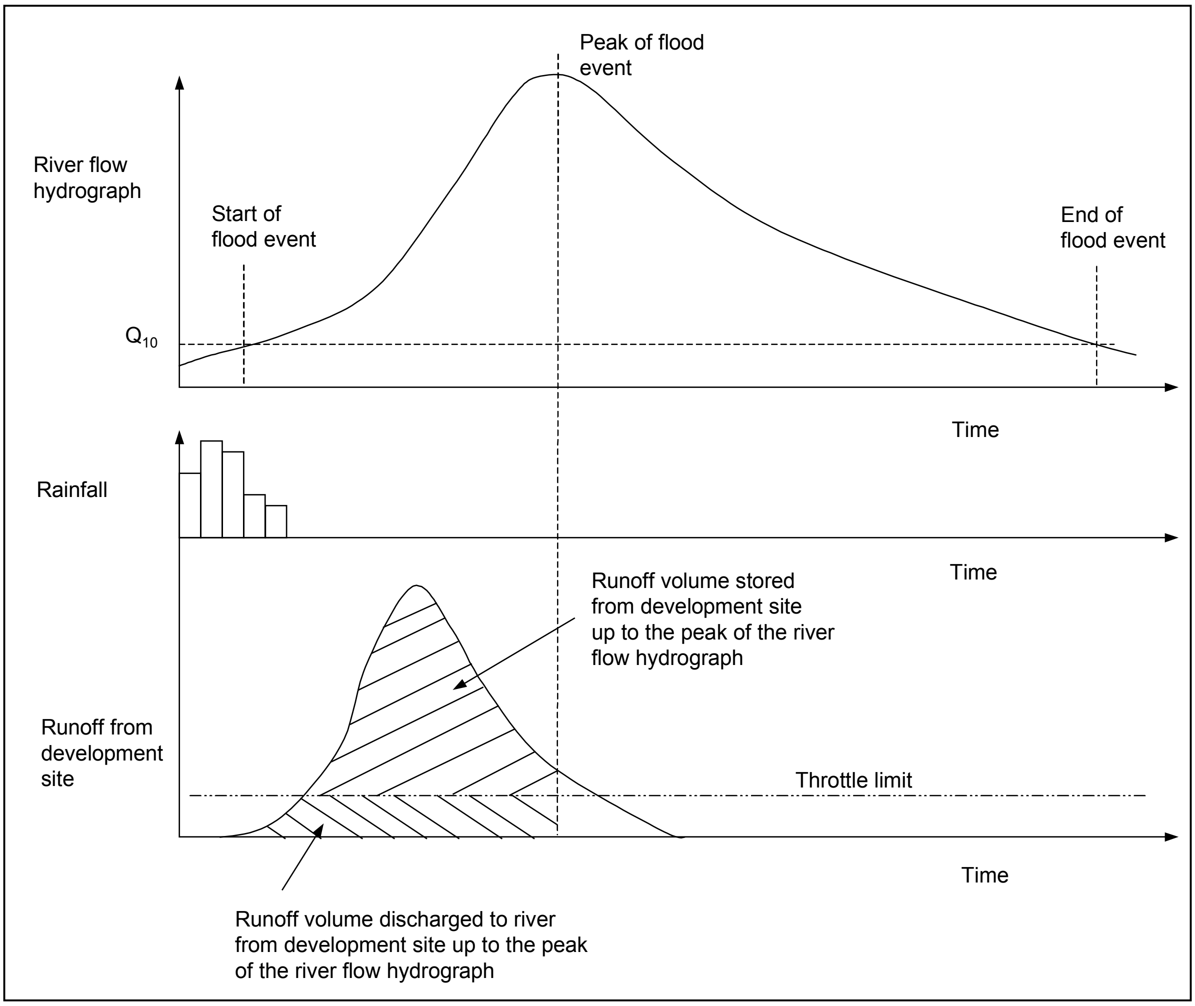



**Figure 8.7 Analysis carded out to estimate the runoff generated from the development site up to the peak of the river flow hydrograph**

A closer inspection of the results was made by looking at the top three flood events of each catchment. These are also shown in the plots in Appendix D. This shows that these do not correlate with either the greatest effective storage or even the largest runoff volumes, though they tend to be among the largest.

A summary of this result is provided by taking the average, 10 percentile and 90 percentile values for the proportion of runoff in storage. Figure 8.8 gives the values of the runoff percentage stored at the peak of the river flow and Figure 8.9 gives the values for the end of the flood events. It can be seen that the effectiveness of storage at the peak of the flood event that the proportion of utilised storage is greater than at the end of the event, but that this does not alter the conclusion that temporary storage provision is largely ineffective.

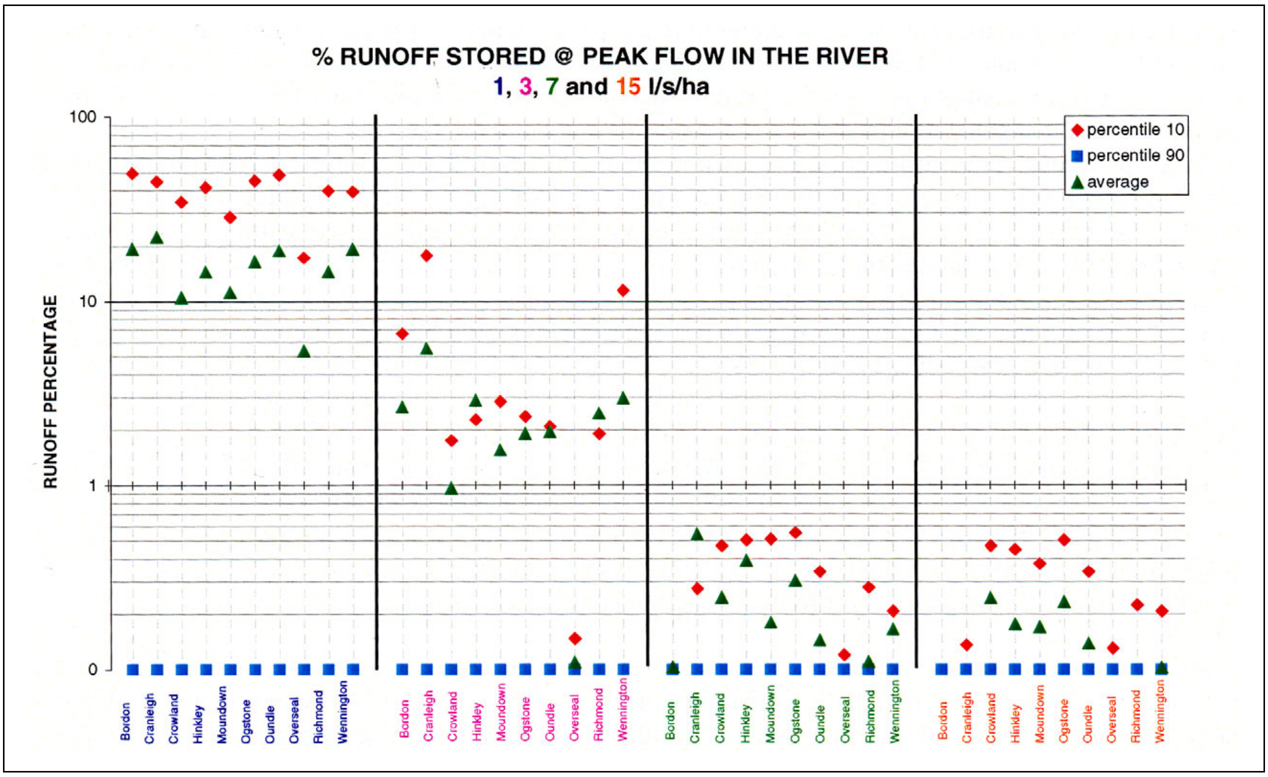



**Figure 8.8** **Proportion of runoff in storage at peak of flood events**

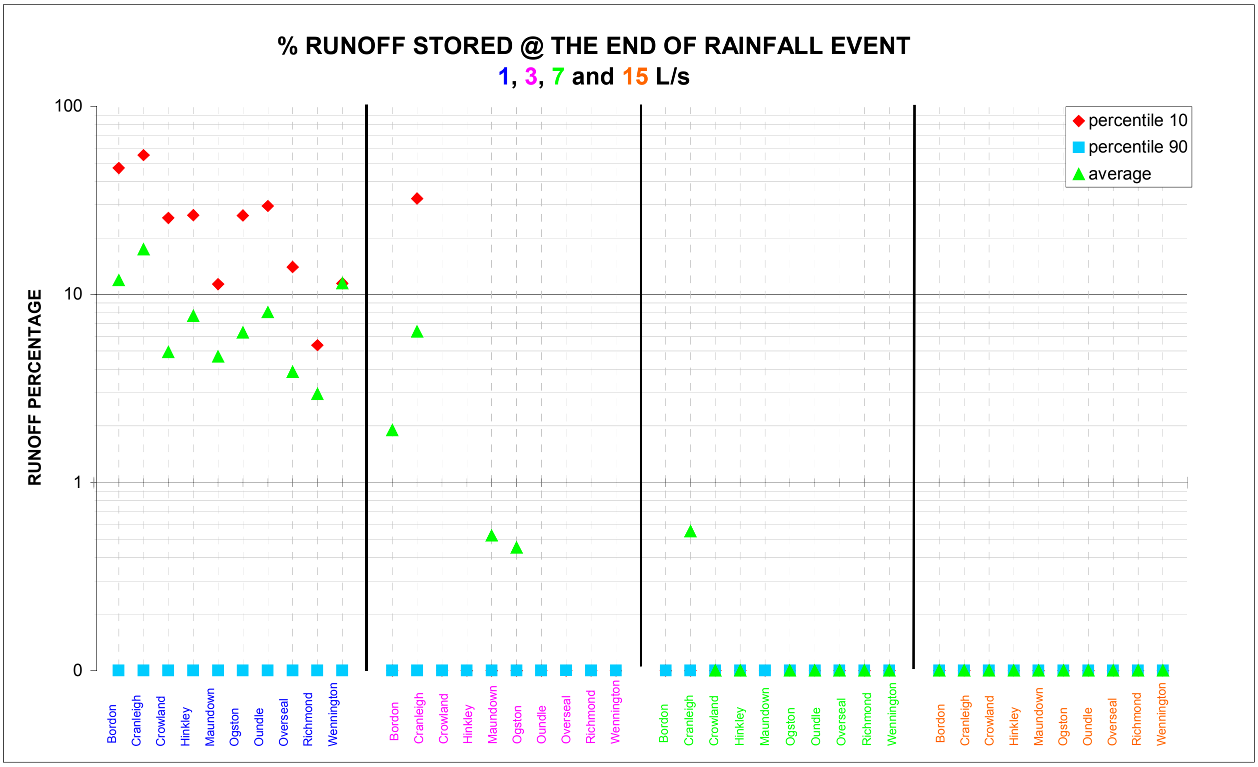



**Figure 8.9** **Proportion of runoff in storage at end of flood events**

### 8.2.2 Runoff in the river compared to green-field runoff

Although the previous analysis shows how ineffective temporary storage appears to be, it really needs to be related to the greenfield condition to define whether it meets the philosophy of restraining the site runoff characteristics to that of the greenfield state. An analysis was therefore carried out to calculate the volume of runoff that is generated in greenfield conditions and compare the volume of runoff in the river for the developed state. This information is shown in Figures 8.10a - 8.10d. Unfortunately it highlights the limitation which was discussed in Section 6.4.3 where it would appear that the volume of runoff generated by the greenfield condition can be greater than that after site development particularly for clay soils. It also emphasises the problem faced by sites with sandy soils. This was discussed earlier. It is clear that for "ordinary" events not only would one expect the development runoff volume to be greater, but also the rate of runoff prior to development would be very much slower which has not been taken into account in these graphs.

This set of plots therefore seems to indicate that catchments with soil type 4 (with 50% paved area) do not require storage as runoff in the river is never greater than the greenfield state. Catchments with soil types 1 and 2 generate far more water and the storage achieved even at 1 l/ha is not very effective in maintaining the status quo. However, bearing in mind the limitations of FSSR 16 for predicting volumes for "ordinary" events and the fact that greenfield attenuation has not been taken into account, taken together with Appendix D, these results indicate that temporary storage for river flood protection is not effective.

A point of caution now needs to be given. These sites represent the results for a limited data set of flood events from 10 to 40 years. Although the probability that a few of the events might be of the order of, or greater than, a 100 year event, the majority will be "ordinary" floods of a low return period. By definition, rainfall depths for flood events averaged for a period of two to five days will be very low, but may double for extreme events. This will still be low, but are more likely to store a larger proportion of water than shown by these graphs. However, the results obtained, which taking into account the real effects of spatial variability, do indicate that only very tight throttles would be effective in protecting rivers during times of flood.

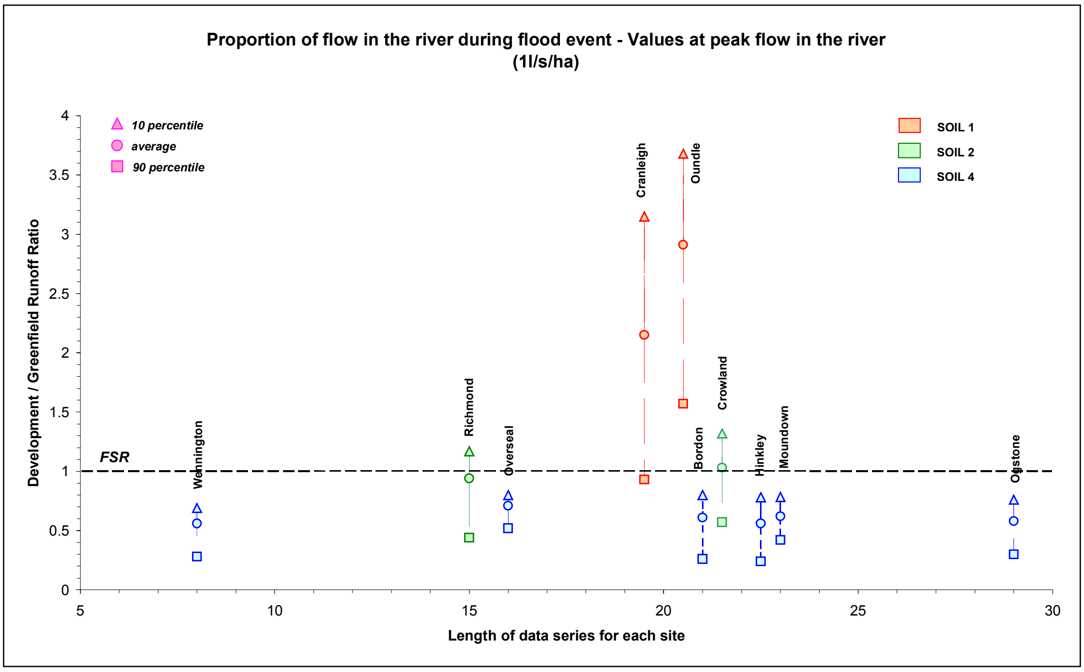



**Figure 8.10a** **Comparison of runoff in the river at the peak of the flood event; pre and post development – limiting discharge 1 l/s/ha**

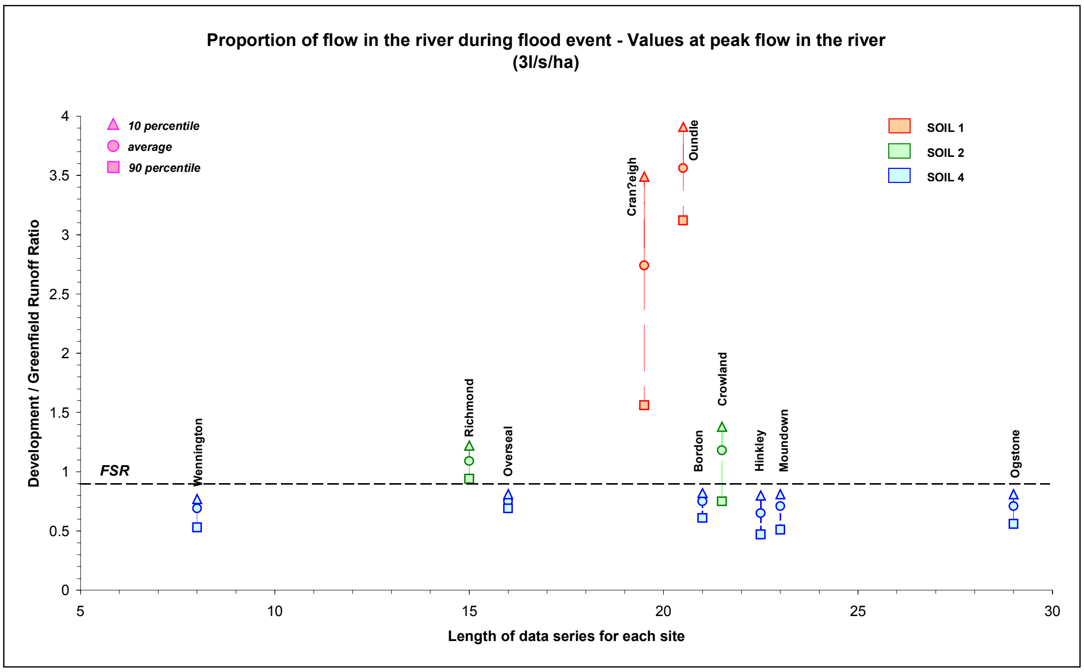



**Figure 8.10b** **Comparison of runoff in the river at the peak of the flood event; pre and post development – limiting discharge 3 l/s/ha**

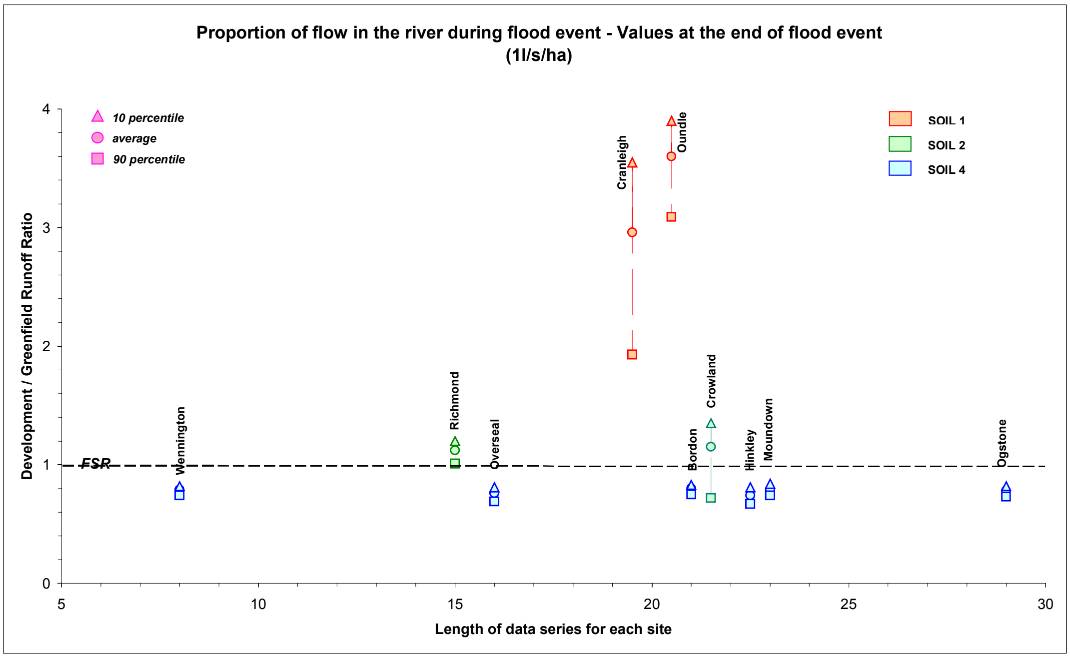



**Figure 8.10c** Comparison of runoff in the river at the end of the flood event; pre and post development – limiting discharge 1 l/s/ha

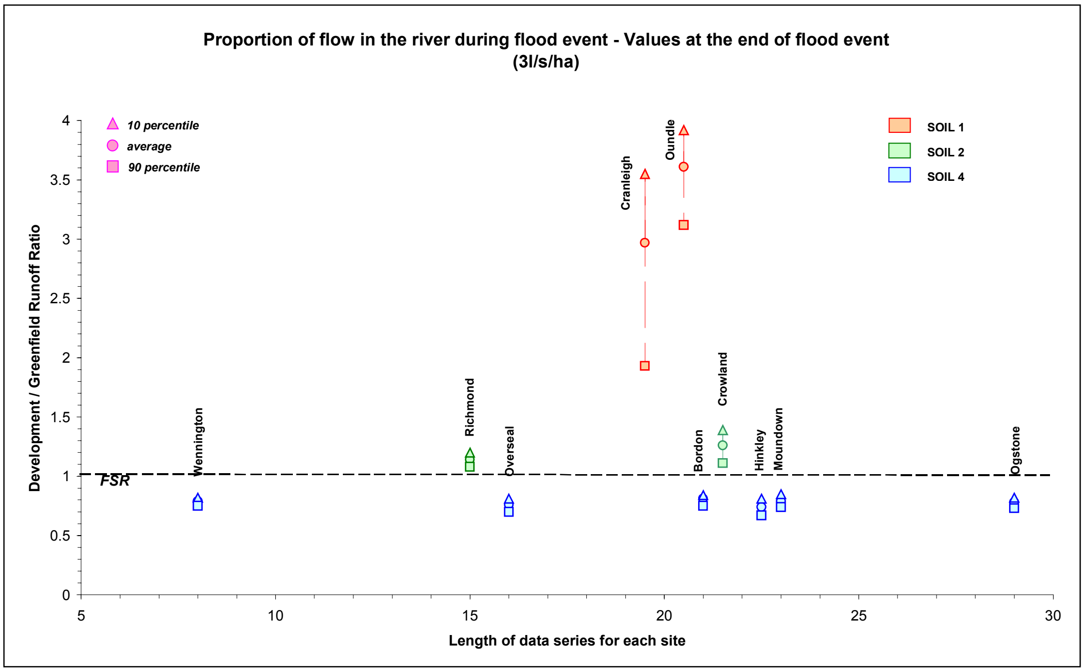



**Figure 8.10d** Comparison of runoff in the river at the end of the flood event; pre and post development – limiting discharge 3 l/s/ha

## 8.3 Comparison of Old and New PR equation of the Wallingford Procedure

Comparison of Old and New PR equations was considered a useful additional element of the study. Currently the Old PR equation is generally applied, but it has a number of limitations as discussed previously. However, as also stated, the New PR equation has not been generally accepted for use in design as values such as NAPI have not been researched and agreed.

To provide confidence in the use of the New PR equation, both runoff models were used for determining storage volumes in the design events for the 3 l/s/ha limiting discharge analysis. The results demonstrate that the New and Old PR equations produced virtually the same results and that the New equation is slightly more conservative for larger events as one might expect. This is intuitively correct in that additional runoff would be expected for extreme events as permeable surfaces get wetter. The results are not plotted on the graphs in the appendix as the differences are small and would make viewing the plots more confusing. Table 8.1 shows the parameters used in the Old and New PR equations. Figure 8.11 summarises the differences between the Old and New PR equation for all soils assuming the following parameters (used in the study for the 10 theoretical sites). The drop off of the curves for 1 year events is caused by depression storage effects reducing the amount of net rainfall.

**Table 8.1 Parameters used in the Old and New PR equations**

| Old PR | New PR | Events | |
|---|---|---|---|
| UCWI = 125 | NAPI = 0mm | Return period | 1 in 1 year, 1 in 10 year, 1 in 100 year |
| PIMP = 50% | PF = 200mm | Duration | 4 hours, 24 hours |
| | PIMP = 50% | M5-60 | 20mm |
| | IF = 70% | | |

## 8.4 Comparison between summer and winter design events

Similarly a check was carried out to investigate the difference between summer and winter events for the use of determining storage volumes using critical duration storms and the New PR runoff model. Winter was found to be marginally worse for any given return period. This information is also plotted on the graphs of Appendix B for the 3 1/s/ha limiting discharge. The following table illustrates the differences found for a range of return periods and throttles. In all cases the same parameters were used as those for Figure 8.11.

**Table 8.2 Storage volumes for summer and winter design events**

| Volume of storage ($m^3$) | | | |
|---|---|---|---|
| Return Period | Throttle discharge limit | Summer | Winter |
| 1year | 1 l/s/ha | 404 | 426 |
| | 3 l/s/ha | 233 | 246 |
| | 7 l/s/ha | 74 | 32 |
| | | | |
| 10year | 1 l/s/ha | 978 | 1022 |
| | 3 l/s/ha | 737 | 785 |
| | 7 l/s/ha | 441 | 449 |
| | | | |
| 100year | 1 l/s/ha | 1996 | 2078 |
| | 3 l/s/ha | 1728 | 1814 |
| | 7 l/s/ha | 1254 | 1344 |

It can be seen that the volumes are virtually identical. In practice NAPI in winter could be expected to be slightly higher for design than summer in a similar way to UCWI. However, a design value for NAPI has yet to be derived.

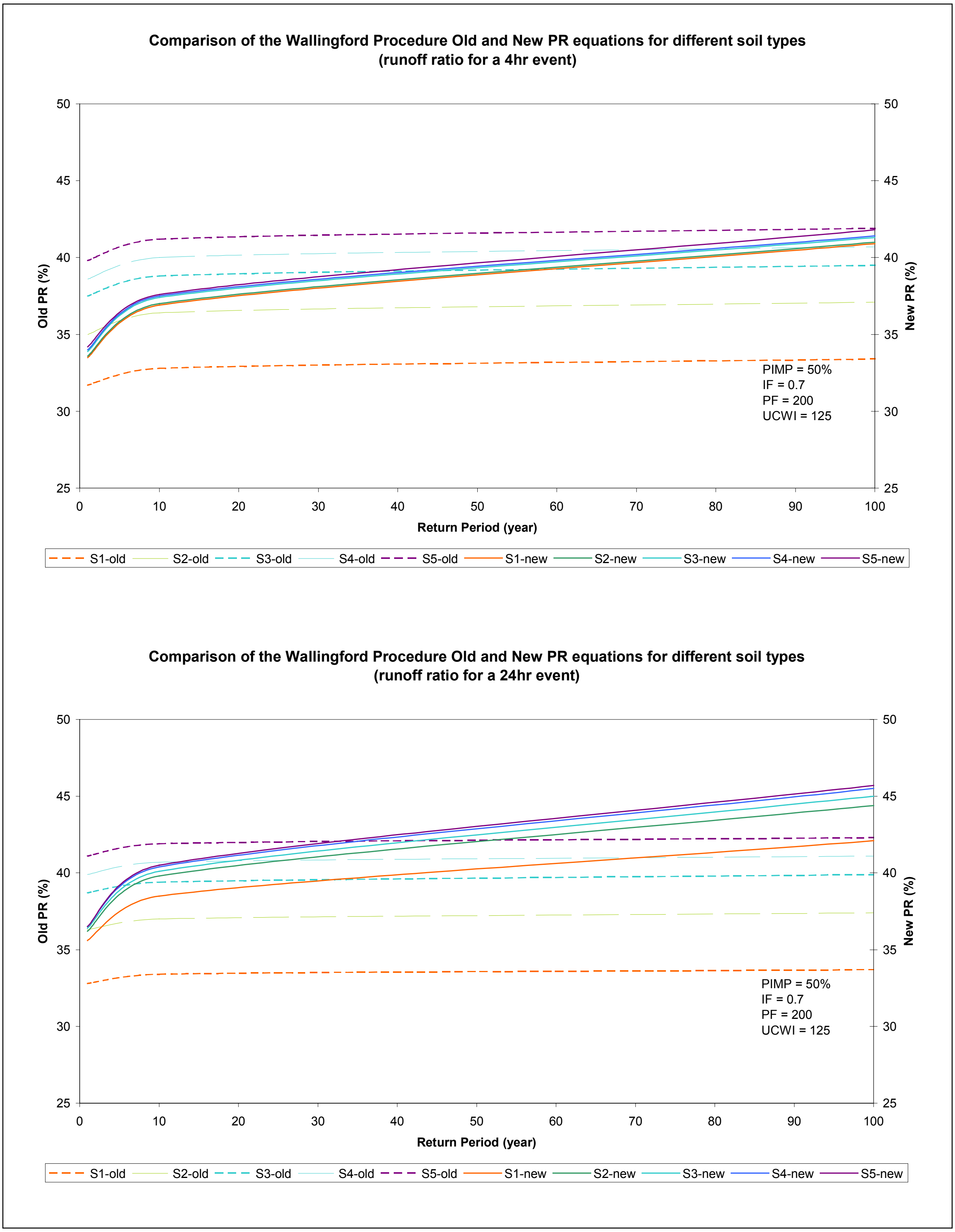



**Figure 8.11 Comparison of the Wallingford Procedure Old and New PR equations**

## 8.5 Number of rainfall events

A brief analysis was carried out to investigate storm depths by month. The concept of interception storage was investigated to try and replicate greenfield runoff behavior where river response in summer was virtually nil from smaller events. The rainfall events for all sites were grouped by month and categorised by depth in the ranges 0 mm to 5 mm, *5* mm to 10 mm and greater than 10 mm. The results are shown in Appendix E.

The results show that the average number of events/month is reduced significantly if interception storage equivalent to 5mm or 10mm is provided. What was a little surprising was that the number of events in summer was not markedly different from other months. It should be noted that the basis for an event was a 24 hour dry weather period and a different result might well be expected if a one or two hour dry weather period was used. In practice there are difficulties in designing for interception storage (to prevent water getting to the river), particularly for the larger volumes that would be generated by 10 mm of rainfall.

This analysis therefore tends to indicate that it is not possible to replicate greenfield response in this aspect. It should however not be forgotten that interception of initial washoff, particularly after a long dry period, has very considerable benefits in reducing the pollutant load passing to the river. Figure 8.11 gives an average of the events analysis for all ten locations. This is used only for illustration as the value of averaging the rainfall characteristics for a range of different locations means little. Appendix E should be referred to, to see the differences between locations.

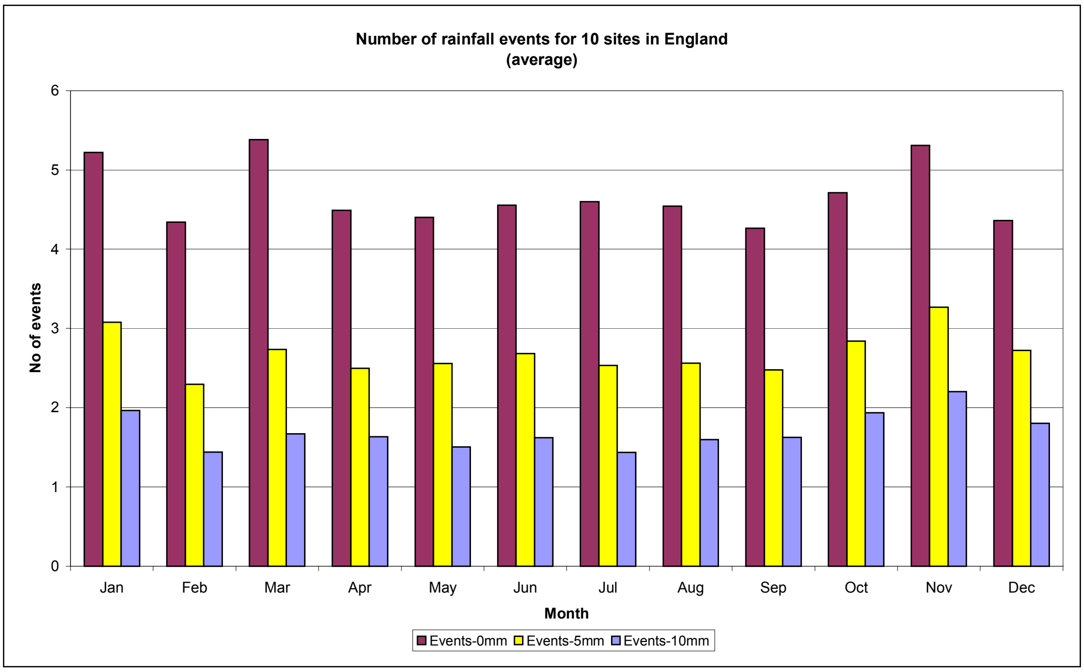



**Figure 8.12 Number of annual rainfall events for the average of the ten sites in England**

## 8.6 Greenfield runoff rates for sites and catchments

An analysis of peak flow rates from sites and their respective river catchments was done using Institute of Hydrology report 124. Figure 8.13 summarises the flow rates for the sites and colour coding helps highlight the importance SOIL plays in this calculation. Figure 8.14 shows how much greater the runoff might be expected to be from a small site compared to the upstream catchment. The flow rate is usually two to three times the catchment rate. This demonstrates that there is little scientific basis for using catchment flow rates for defining limiting discharges from sites.

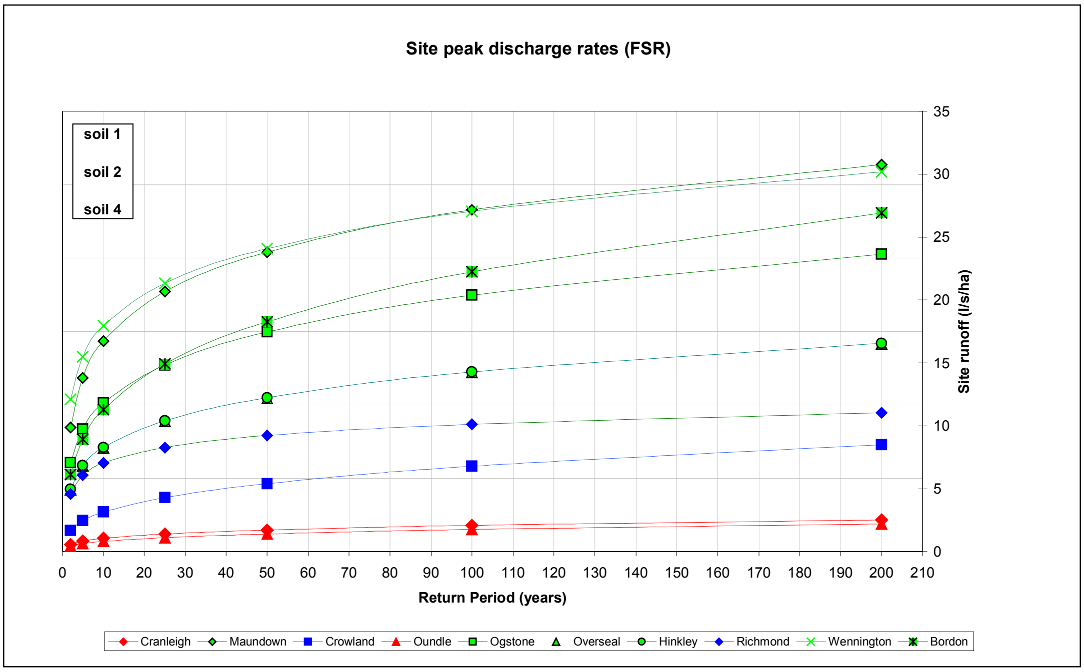



**Figure 8.13 Site greenfield peak discharge rates**

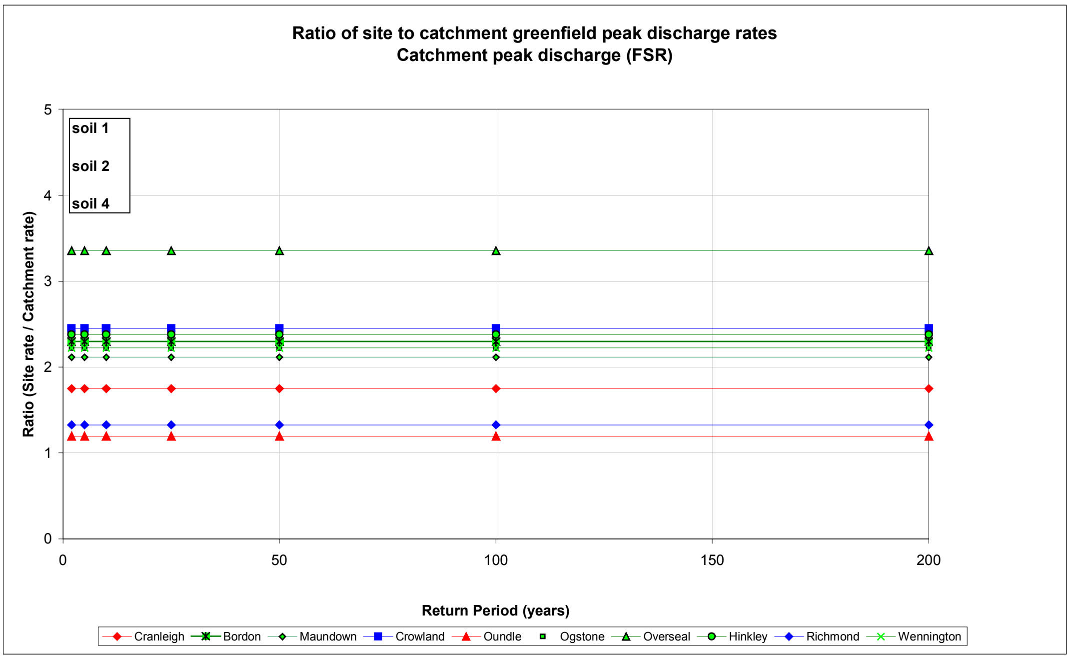



**Figure 8.14 Ratio of greenfield site to catchment peak discharge rates**

# 9. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are the outcome of this study. The extent of the analysis has been limited to assessment of ten sites around England. The depth of the analysis has also been limited by the extent of the data sets. It would have been desirable to carry out further processing of the information, but it is clear that them should be cause for concern over the current methods of approach used for determining temporary storage volumes and their effectiveness for protecting rivers in flood. It is felt that the findings are sufficiently robust to allow the formulation of an alternative approach to achieve a sustainable runoff control philosophy for development sites.

## 9.1 Conclusions

### 9.1.1 Existing storage requirements by regulators

The work undertaken for this research document highlighted the lack of consistent guidance provided by regulatory bodies such as the Environment Agency concerning measures to control runoff from developments. In addition the research suggests that the methods being applied are ineffective for river flood protection.

### 9.1.2 Philosophy of sustainability

Due to the findings, it would appear that a radical change in the philosophy of approach is needed to ensure the concept of sustainability is applied. The most obvious interpretation of sustainability, with regards to developments, is that site runoff should reflect the pre-development state as closely as possible. This should consider both water quality and hydraulic behavior, although this study has only briefly touched on the issue of water quality.

### 9.1.3 Effectiveness of temporary storage

The analysis indicates that temporary storage volumes requested, principally to protect the river during periods of flooding, are oversized (for periods when rivers are in flood) and that nearly all the runoff from the development actually enters the river during flooding periods. Throttle rates need to be set to levels of around 1 l/s/ha to provide effective retention for river flood protection.

### 9.1.4 The use of critical duration events for storage design

The analysis in the research indicates that the use of critical duration design storms for assessing storage to protect the river at times of flooding provides an over-estimate of volume. This storage volume ranges from being approximately correctly sized at very low throttle limits (1 l/s/ha) to being significantly over-sized for throttles with more generous limits. It would appear that the use of catchment Tp duration events is a more accurate and appropriate method for predicting these volumes. However, the value of the storage (for these more generous limits) is limited due to the rapidity of the runoff passing to receiving stream.

The use of critical duration events for assessing the site level of service (for storms all year) is still considered appropriate.

### 9.1.5 The need for permanent storage

The concept of “permanent” storage needs to be considered to take account of additional volumes of runoff generated by development. This concept is necessary to make storage effective in protecting the river. Drainage of permanent storage needs to be delayed until after the flood wave has passed and therefore should utilise land drainage techniques or other indirect runoff systems.

The importance of “permanent” storage is very much a function of the soil type of the site in question. SOIL type 5 is theoretically unlikely to require “permanent” storage, whereas SOIL type 1 will require extensive storage. However this is based on assuming the validity of using the two equations FSSR 16 and the New PR equation for determining volumes. In practice a pragmatic equation needs to be derived to overcome the apparent contradictions between these two equations. In addition the rate of runoff also needs to be catered for in deciding "permanent" storage needs.

### 9.1.6 Interception storage

The concept of interception storage to replicate green-field conditions in summer (when little or no runoff takes place) would be difficult to replicate cost effectively. The concept of initial runoff interception to minimise pollutants washoff passing to the river is extremely desirable, but may only be effectively implemented in the more pervious catchments by ponds when storage provision is being considered.

The use of intercepted storm runoff passing to the foul system in some way, apart from being technically difficult to achieve, also has political consequences and is likely to run into resistance from the Sewerage Undertakers.

### 9.1.7 Brownfield developments

The research initially intended to deal with brownfield as well as greenfield sites. However, the political and environmental issues surrounding brownfield sites requires that each site should be assessed individually. The principle of sustainability (for a greenfield response) should be aspired to, but this would only be taken as the starting position for site evaluation.

### 9.1.8 Throttle sizes

The requirement of using small limiting discharges conflicts with practical operational and maintenance issues for smaller sites. Adoption of storage systems using pipe orifices of less than 150mm diameter is generally not acceptable. This results in a minimum flow rate of around 10 l/s. This means that sites which are less than 3ha in size are likely to be constrained by throttle size rather than green-field runoff flow rates.

This leads to the obvious requirement to plan for regional flood protection rather than working on an individual development basis.

## 9.2 Recommendations

The following recommendations are proposed for defining storage requirements of sites.

### 9.2.1 Sustainability

The concept of sustainability should be applied. In this case this might mean that post-development site runoff should be as similar as possible to the pre-development situation. This should preferably be applied for a range of return periods and not just one criterion.

### 9.2.2 The analysis of storage volumes for developed sites

The New Wallingford Procedure runoff model is preferred for determining storage volumes for development sites using summer or winter design storms. If the Old PR runoff model is used, it should be applied within the constraints for its correct application. Previous research suggests that the value of winter UCWI should be increased by 20% above the curve in the relevant WaPUG user note (as shown in Figure 6.1) for prediction of storage volumes.

Parameters for use in the New PR equation are open to engineering judgement, but it is suggested that new developments would use a value of IF of 0.75 and NAPI of zero. NAPI could be raised as high as 30mm if it was considered that wet winter conditions were appropriate for the design storm being considered. There is no official guidance available to industry at present on design values of NAPI.

### 9.2.3 The analysis of the rate of runoff from greenfield sites

The Institute of Hydrology Report No. 124 is recommended for use to determine peak rates of runoff from the site for use in determining throttle limits for discharge rates. Cross checks against the ADAS report 345 can be carried out to compare derived values.

### 9.2.4 The analysis of volume of runoff from greenfield and developed sites

The FSSR 16 formula should be used for determining volumes of runoff from green-field sites. The runoff volumes from applying FSSR 16 can be greater than the values derived by the New PR equation for post development runoff, particularly for Soil types 4 and 5, so appropriate care should be taken.

Due to the apparent mismatch between the FSSR16 formula and the permeable runoff generated by the New PR equation, an alternative equation is proposed for those who wish to minimise any apparent anomaly in the use of these two equations. It is suggested that 80% runoff could be assumed to take place from all paved surfaces and that all pervious surfaces which can be drained by the drainage network would be assumed to contribute in the same manner as that prior to development. This is defined in Chapter 10. It should be clearly understood that this would only be applicable for long duration events where catchment wetness can be expected to be high.

### 9.2.5 The provision of permanent storage

The concept of throttled temporary storage to limit rates of flow off the site to replicate greenfield response should be supported by the concept of "permanent" storage to take account of the additional runoff generated by development hard surfaces. This "permanent" storage would be designed to drain indirectly and over many days rather than hours.

Evaluating the volume to be stored should be based on a design event with a critical duration of Tp for the catchment. The basis for this suggestion is that events of length Tp or greater will be of significance in terms of river flooding for that location.

However, as Tp will vary from 4 hours to 48 hours, depending on the location of the development and the river it drains to, this will result in very large differences in storage volumes for sites which are otherwise identical. This would appear to be unfair even though the criterium has a logical basis. A pragmatic alternative is therefore to select a conservative value of Tp which ensures compliance with this philosophy for the majority of sites. It is suggested that a value of 12 hours is sufficient to achieve this objective.

### 9.2.6 The use of techniques to minimise permanent storage

Due to the extensive volumes of permanent storage required for sites with soil type 1, it is advised that sustainable drainage techniques and landscaping of the site are considered to try and minimise the land requirements needed for storage.

The method of mobilising this storage is not easy when using traditional storage and throttle techniques. Very tight throttles are likely to be needed, and therefore storage will start to be mobilised at relatively low return periods in some cases to enable the required volumes of storage at the 100 year event to be achieved. This therefore indicates that other methods such as careful contour planning of the site and (SUDS) might be more appropriate in some circumstances to achieve compliance with this concept.

### 9.2.7 Implementation of the philosophy of development storage

In order to ensure that the runoff control philosophy suggested by this research is acceptable to industry and the Regulators, it is recommended that the following activities are carried out:

- A national guide is produced and promoted by the Environment Agency;
- A methodology and consistent set of criteria are agreed for use in all Area offices for local authorities and the Environment Agency;
- A simplified guide together with supporting software is produced to assist developers in estimating

storage requirements for sites.

## 9.3 Recommended further work

The analysis carried out is limited by the scope of the project and the extent of the data available. Although the results are considered robust, it is felt that all the implications of the proposed change of approach and methods of analysis have not been completely explored. The following are a couple of aspects where it is felt that there is particular need for investigation.

### 9.3.1 Analysis of All year data

The limited analysis of the "all year" rainfall seemed to indicate that design storms may be slightly under sizing volumes of storage. This could be related to climate change effects or, more likely, slightly incorrect characteristics used by FSR for those sites. However design event profiles, which have been specifically designed for determining flow rates rather than storage volumes, do not necessarily accurately reflect real rainfall and the storage they mobilise for any given throttle limit. This implies the use of time Series rainfall is the most accurate method for determining storage volumes. This is probably the case, but it increases the complexity and cost of analysis. There is therefore a need to continue to take the pragmatic approach of continuing to use design storms with appropriate correction factors. Alternatively, it may be shown that use of FEH rainfall resolves this apparent inconsistency. The only way to check this is to carry out an extensive programme of comparing TSR results against the use of design storms.

This analysis is particularly needed for ensuring that adequate "permanent" storage is mobilised where throttles are being used. This is because the intensities during flood events are generally quite low and therefore appropriate throttle limits are very difficult to determine accurately.

### 9.3.2 Mobilising of "permanent" storage

As the concept of permanent storage now appears to be necessary, the whole area of delayed return flow, the ability to mobilise the storage of runoff and cost effective method of meeting these requirements needs to be examined. This is particularly important in the light of PPG25 and PPG3, where in the case of the latter, there may be inadequate space to implement SUDS techniques in high density developments. In particular there is a need to classify SuDS units in terms of the rate of runoff to establish whether they meet the aspiration of providing sufficient attenuation to be classed as providing "permanent" storage.

### 9.3.3 Site characteristics

All the analysis for this research has been carried out using a theoretical 10ha site. Developments actually range from a few houses to 100ha and greater. The implications of this on the findings are not obvious. It is considered advisable to carry out a study on a range of sites, preferably based on real developments.

## 10. RECOMMENDED PHILOSOPHY FOR STORMWATER STORAGE

Developers at present have difficulty in defining whether they need to provide storage to attenuate runoff and, if so, in determining what storage volumes must be provided. This is partly due to the fact that each site and receiving waters are different and also due to a variety of criteria within the different regions of the Environment Agency regarding storage of stormwater runoff.

Although HR Wallingford has carried out this research and has suggested a way forward, the Environment Agency policy is still under review while they consider HR Wallingford's recommendations. This chapter is written in two parts. It gives a summary of the current status and most commonly requested requirements, followed by the suggested recommended approach for the future. Examples are provided in Appendix F which illustrate both the existing and proposed methodology and also the advantages provided by the new procedure.

Part A - Current Regulatory requirements for storage
- Regulatory basis for requiring site storage
- Current requirements for storage design

Part B - HR Wallingford's proposed procedure for storage design
- Philosophy of approach for using site storage
- Proposed storage design criteria

This is followed by a summary of methods for assessing runoff rates and storage volumes together with recommendations as to which are most appropriate.

### 10.1 Part A - Current Regulatory requirements for storage

#### 10.1.1 Regulatory basis for requiring site storage

The involvement responsibilities and requirements of the various planning authorities has already been detailed in chapter 4. However the subject is worth re-visiting briefly. Theoretically Local Authorities are responsible for discharges to non-Main river (ordinary watercourses), while the Environment Agency are responsible for designated rivers referred to as Main River. Although in practice the Environment Agency are deferred to for advice in all planing applications with respect to river discharges, the Environment Agency is not able to control the runoff from new development without the support of the Local Authority who are able to influence this matter by way of planning agreements and conditions.

The Environment Agency comprises many regional offices and, although national guidance is provided, these act (at present) autonomously which has resulted in a range of approaches to storage specification. Over the last few years the requirement for storage provision has tightened. The provision of storage is largely aimed at trying to protect against spate flow effects and flood mitigation. This has led to the generally held view that storage lower down the catchment is less desirable (to discharge ahead of the hydraulic wave coming down the river), and to store water otherwise, particularly if known flood locations exist downstream.

The criteria used to date have generally been to request a throttle to limit discharges to around 5 7 l/s/ha (though this can range from Ito more than 10 l/s/ha), though this limit is tending to get lower. This throttle value is currently arrived at using a range of methods. These range from the use of the Flood Studies Report to determine a site runoff rate, a selection of an arbitrary value, the use of the ADAS runoff method, through to the calculation of the average unit rate of runoff of the catchment for a river discharge in flood. The return period requirements are usually for the 100 year event and using the site critical duration storm.

This research by HR Wallingford has shown that throttling and the provision of temporary storage is unlikely to make much difference to the flood impact in the river for many locations as storm events that cause river flooding are usually long events with very low average rainfall intensities. This is diametrically opposite to the way site storage is currently being assessed and results in very little of the storage volume being utilised during these events. This does not discount the fact that rivers must also be protected from high intensity events as well (which are usually spatially limited) which can temporarily cause high peak flows in the river locally, particularly where large developments discharge to small water courses.

The research recently carried out indicates that there is discrepancy between current practice and its effectiveness in providing the intended benefits and a suggested methodology has been put forward to help determine a more suitable approach.

### 10.1.2 Overview of criteria used for storage

It must be stressed that the requirements for storage should now always be considered in the context of the national emphasis now being made of using SUDS to serve new developments. Storage is just one component of the SUDS techniques. In practice, until recently, storage has been the main mechanism used for protecting the receiving waters.

There is a general strategy that storage may only be avoided if the development area is particularly small (there is likely to be a minimum practicable Hydrobrake or orifice control size), or there is currently sufficient capacity in the receiving waters so that there is no additional risk of flooding created by the development. The approach to brownfield development is generally case specific and rarely involves consideration of equivalent greenfield conditions. The position of the site in the catchment is usually an important aspect and is sometimes a reason for the Agency positively requesting that storage is not used (usually for developments which discharge to lower reaches of rivers).

Representations regarding surface water disposal are generally made in pre-draft Local Plan stage in terms of flood defence (in line with DoE C30/92), conservation, landscape, and water quality issues. PPG25 is a guidance document on development and flood risk (2001 DETR) and this replaces DoE C30/92. PPG25 emphasises the need to use SUDS techniques for drainage of developments and the Environment Agency has already included reference to SUDS in local plans in a number of Regions.

The following is a non-exclusive list of the principal criteria generally used for specifying storage. In nearly all cases the limit of discharge chosen is then used with a 100 year rainfall critical duration event to determine the storage volume. In all instances, peak flow rates is the controlling criteria and volumetric aspects of the increased volumes of runoff caused by development is not explicitly addressed.

<u>Flood Studies Method</u>
The Flood Studies methods are recognised as having limitations as most sites are smaller than 50ha which is the officially recommended limit for using the FSR approach. The FSR approach is used to determine the peak runoff from the greenfeld site. This is often determined for a return period of only 1 year, or the mean annual flood. In the case of Thames Region they have introduced the concept of varying limits of discharge in line with runoff for any return period. Although difficult to implement, it results in low limits of discharge for frequent events and quite generous discharge rates for rare events (100 years). Although FSR predictions of peak flow is not advised for catchments less than 50ha, the Institute of Hydrology method in report 124, when compared to ADAS, usually provides comparable values.

<u>ADAS method</u>
The ADAS method is an alternative approach to determining peak runoff from a greenfield area. Its application to development sites is largely due to the 50ha limit mentioned earlier. The advice given is that the ADAS method should not be applied to areas larger than 30ha.

Total catchment analysis
There has been a move recently to analyse peak flow rates in rivers and, on a proportional basis, determine the unit rate of discharge for the catchment. This is then used to arrive at the limit of discharge for the development site. This has been done using either FSR or gauged records.

Arbitrary limits of discharge
The majority of Agency regions quoted maximum allowable runoff figures in the region of 5 to 10 1/s/ha. Higher values are used with a value of 801/s/ha being used in one instance for a steep site in the north of England. The basis for choosing values is often indirectly linked to typical figures derived by using Flood Studies or other analytical methods.

For most regions, the majority of storage is implemented as oversized pipes or tanks, despite the Agency advocating open ponds for ecological and water quality reasons, wherever possible. Historically there has been a lack of willingness to adopt (operate and maintain) such features by local authorities / sewerage undertakers.

SUDS approach
The Environment Agency is now strongly promoting the use of SUDS techniques with the aim of mimicking greenfield behavior in the rainfall runoff response from developments. The techniques allow volumetric runoff generation and the rate of runoff to be dealt with by using a variety of infiltration and storage techniques, some of which are relatively new while others are well established with a proven performance record.

## 10.2 Part B - HR Wallingford's proposed procedure for storage design

This research project involved a steering group which included the Environment Agency, who are now examining the results of study and its recommendations. It must therefore be stressed that the following recommendations, although meeting the general objectives for protecting the environment from urban stormwater runoff, has not been ratified by the Regulator, but may provide the basis for a future national method.

### 10.2.1 Philosophy of approach for using site storage

The philosophy of approach on drainage planning for greenfield sites is to use site storage to provide a mechanism by which the river regime can be maintained in its natural state, by minimising the differences between the developed and predevelopment catchment runoff. Storage is utilised to limit both the flow **rate** and **volume** of surface water runoff that drains directly off the site thus mimicking the pre-development runoff state. Although the proposal does not explicitly use the word "SUDS" it is clear that some of the objectives might best be met by using SUDS techniques. The following section briefly summarises the findings of the research and the implications that it has with regard to storage design criteria.

### 10.2.2 Findings of the research on stormwater storage

The scope and detailed findings of the research has already been reported.

A summary of the findings are as follows:

Design storage volume
A comparison was made of the storage requirements that were needed for a site by using standard design storm events with the recorded rainfall for that location. The storage volume predicted using the standard approach of the critical duration event of a design storm was closely reflected by the recorded data for the rainfall for the whole year for any given limit of discharge. This is exactly what should have been found.

However the design storage requirements for rainfall events that took place during river flood events were correctly predicted by critical design events for only throttle rates of 1 and 3 l/s/ha. More generous throttles (7 and 15 1/s/ha) resulted in significantly less storage being predicted than that needed for the critical design storm. This result is not unexpected as the critical duration event for these higher limits of discharge is very short (1 or 2 hours) while an event which is critical for a river catchment is often 12 hours or more. As throttles become tighter, the critical duration extends and is therefore closer to that of the river.

Retention of runoff in storage

This analysis checked on the validity of the assumption that the storage system was effective in reducing the flood impact of the urban runoff. A flood was considered as occurring when flow rates were above the $Q_{10}$ flow rate value. The analysis checked to see what proportion of the runoff volume passed to river by the time of the peak flow in the river and also the proportion that passed to the river by the end of the flood event.

The findings indicated that unless the throttles were very tight, in the region of 1 l/s/ha, the volume of runoff passing to the river was close to 100 percent and that there was rarely any effective retention on the site during the flood event.

The implication of this result is that temporary storage using throttles either has to be draconian or another method of retention is needed if the additional runoff generated from urbanised areas is to be mitigated. The concept of "permanent" or longer term storage has therefore been introduced in the proposed criteria to cater for the additional runoff that urban development creates.

### 10.2.3 Proposed storage design criteria

If the basic premise is that greenfield conditions prior to development is to be replicated by post development runoff, these results imply that current methods of trying to protect the rivers are neither effective environmentally or technically efficient. The following philosophy of approach proposed by HR Wallingford tries to take account of these findings. Although SUDS is not explicitly stated in the method, it is implicitly implied as the concept of "permanent" storage requires reduction (infiltration) or longer term retention of part of the volume of runoff which might only be achievable using certain SUDS techniques.

The criteria have been broken down into 6 parts. This is not to add complication to the process of development, but is the logical approach based on the recognition that there are a range of implications for stormwater runoff for site requirements as well as river protection. It also takes account of the fact that different runoff rates occur from the greenfield site for different return periods. Therefore to design and implement this concept to best effect, the following six elements should be considered.

These criteria are listed in the likely order of frequency of return period.

- **River water quality protection**. This criterion aims to protect the water quality of the receiving watercourse;
- **River regime protection**. This criterion aims to protect against ecological and physical damage by minimising changes to the receiving watercourse's regime;
- **Level of service protection**. This criterion aims to protect the site from flooding from the drainage system;
- **River flood protection**. This criterion ameliorates the risk of flooding in the receiving watercourse downstream of the development site;
- **Site flood protection**. This criterion controls flooding of the site during extreme events;
- **Catastrophic protection**. This criterion is not explicitly a regulatory issue, but more a CDM requirement. It should only be employed where flooding of the site could lead to loss of life or damage to property is likely to be excessive.

These criteria also subdivide into the two categories of site design and river protection. Although explicit consideration of all six elements should be made, there will be many situations where application of some, or even all, of the criteria may not be needed.

In practice it is quite difficult to implement all these criteria by just using storage units and throttles to achieve the variable discharge limits to mimic greenfield response and the volumetric reduction or retention of "permanent" storage. The examples in appendix F show how this procedure might be modelled. In practice, careful planning of the site together with the use of SUDS and a drainage model will be needed to implement all of these criteria. It must be stressed that pragmatic solutions may be needed which approximately achieve the objectives of the criteria required for the site development. The main principle behind the philosophy is that the developer should be entitled to discharge predevelopment runoff volumes at a rate equal to, or similar to, green-field runoff from the site for any particular return period.

Table 10.1 summarises the criteria that are suggested for applying site storage for development. It should be stressed that currently there is no nationally agreed approach and even when there is an agreed procedure, it is essential that considerable flexibility is provided in its implementation for any specific site.

**Table 10.1 Suggested criteria for limiting green-field site runoff**

| Element | Return period (years) | Description |
|---|---|---|
| River water quality protection | 0 to 1 | • Minimum discharge rate 10 - 15 l/s<br>• Interception of between 0 mm and 10 mm of rainfall: 2 to 5mm desirable |
| River regime protection | 1 to 5 | • Minimum discharge rate 10 - 15 l/s<br>• 1 in 1 year greenfield site discharge rate<br>• 1 in 1 to 2 year site critical duration storm<br>• No flooding on site |
| Level of service for the site | 20 to 30 | • Minimum discharge rate 10 - 15 l/s<br>• 1 in 20 year greenfield site discharge rate<br>• 1 in 30 year site critical duration storm<br>• No flooding on site except where specific planned flooding is approved |
| River flood protection | 50 to 100 | • Minimum discharge rate 10 - 15 l/s<br>• "Permanent" flood storage for development runoff volume greater than greenfield runoff<br>• 1 in 100 year, 12 hours, or catchment critical duration storm Tp, for volume assessment of additional runoff |
| Site flood protection | 50 to 100 | • Minimum discharge rate 10 - 15 l/s<br>• 1 in 50 year greenfield site discharge rate*[1]<br>• 1 in 100 year site critical duration storm<br>• Temporary controlled surface flooding / storage |
| Catastrophic protection | 200 or greater | • Unusual circumstances<br>• Embankment breaches<br>• Very high value property<br>• Loss of life |

NB*[1] In many instances it is unlikely that the 1:50 year peak greenfield runoff rate can be utilised fully in draining the site for "site flood protection" if compliance is achieved with provision of "permanent" storage and the other criteria. This is because the critical duration event for the site is normally short, and therefore the runoff is volumetrically relatively small, compared to the volumes involved in "river flood protection". This will depend on the SOIL characteristics of the catchment and the usage of SuDS units or just storage and throttle systems.

The suggested return periods generally reflect current criteria used in the water industry. The minimum discharge rate is based on the premise that the minimum orifice and pipe size needs to be 150mm and self cleansing flows need to be at least 101/s or greater. To achieve a lower rate would require either a smaller orifice or techniques other than the use of a hydraulic throttle.

**Water quality**
Rainfall on rural catchments initially has no runoff, except in very wet winter periods. Hence it is desirable to intercept this initial runoff 'first flush' from a development site and prevent it reaching the receiving watercourse. This will not always be feasible, but it is an issue that should be considered and designed for, if possible. The objective of the water quality element is therefore to minimise runoff of polluted water from development sites thereby protecting the water quality of the receiving watercourse, especially during periods of low flow. Initial runoff from urban surfaces from rainfall is often highly polluted. It is suggested that the first 2 to 5mm is very much more cost effective than trying to intercept 10mm.

**River regime protection**
The objective of river regime protection is to minimise the impact of short response times and high flows of the runoff from a development site that discharge to a watercourse. This minimises erosion, high concentrations of suspended solids and other ecological damage. Return periods (and flow rates) should therefore be lower rather than higher and cater for frequent events.

**Level of service for the site**
The developer should aim to provide a level of service that guarantees against flooding for a certain level of service as well as complying with the philosophy of limiting runoff from the site to green-field flow rates. This applies to both storage design as well as the drainage network performance.

**River flood protection - volumetric limitation**
Watercourses should be protected from extra runoff during extremely wet event periods. Analysis has shown that temporary flood storage has limited benefit in protecting a river with most of the runoff entering the river during the period of the flood. Effective discharge limits that reduce volumes of water to the watercourse during the period of flooding, have to be very tight and this results in very large storage volumes being designed. The alternative approach is to provide a mechanism for storing excess rainfall runoff compared to that which would have taken place on the site before development. This storage would be "permanent" and be retained for several days and have an indirect drainage mechanism. This is not easily achieved in practice as rainfall intensities are generally very low for long duration events when river flooding takes place.

An alternative to local site protection schemes for each development, is the use of large ponds serving a group of development areas, such as those found at Milton Keynes, which allow the construction of throttle systems using very low unit rates of discharge.

**Site flood protection for extreme events**
Although the level of service for drainage should be in the region of 30 years, it is generally considered important to consider the implications of the impact of a 100 year event. The provision of "permanent" 1 in 100 year storage for the protection of the watercourse and the "level of service" storage in the region of 20 to 30 years, is not necessarily sufficient protection for the site for extreme events which are critical for the development drainage system. These events are usually summer thunderstorms. This could involve temporary flooding of car parks or other areas that would cause limited disruption. The main objective would be to ensure that floor levels of houses and other important units would be at a level that is well

above predicted flooding water levels. In practice, where "permanent" storage is provided (particularly for sites with sandier soils), calculations will generally show that there is usually no need for additional site flood protection storage. It should be noted that the designed pipework will be heavily surcharged and therefore consideration will need to be given to the impact of flood flows and their routing, which will occur all over the site.

**Catastrophic events**
The possibility of extraordinary rainfall and flooding and its implications should always be considered. It is rare that designing for such return periods can be justified on economic grounds, but where a risk might cause life threatening situations to develop, it is important to demonstrate that consideration has been given to these circumstances occurring. For instance rivers or dams which might breach their embankments and inundate urban areas would justify investigating the extent and implications of such an event occurring. Particular emphasis will need to be placed on topography and the routing of flood flows to try and protect against loss of life.

Figure 10.1 illustrates the various aspects of storage design for a site

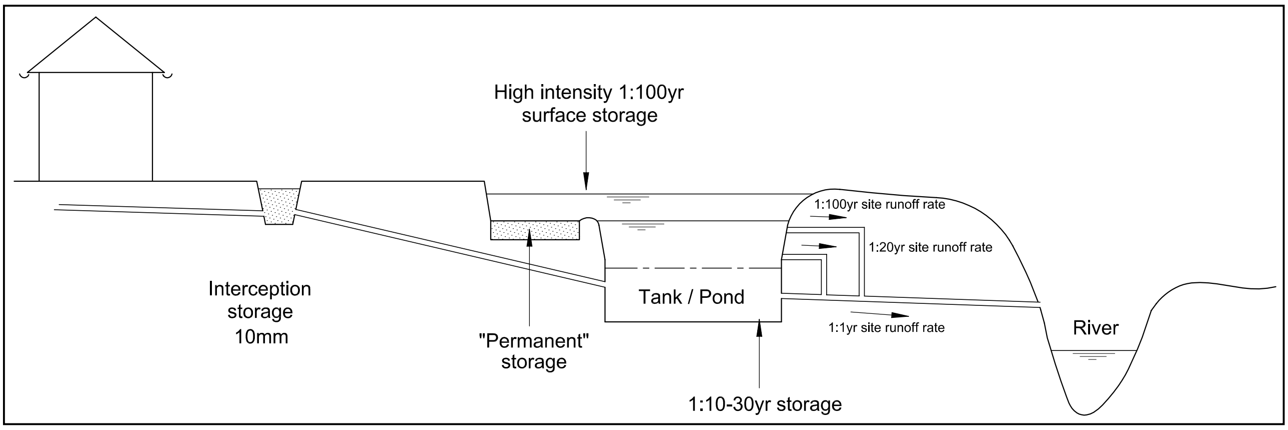



**Figure 10.1 Schematic of site storage design**

Two examples are used to illustrate both the existing and the proposed new method of approach. These are given in Appendix F.

## 10.3 Assessment of Greenfield site runoff

There are numerous hydrological techniques currently in use to estimate green-field runoff rates and these have been detailed earlier.

The principal methods used for determining peak flow rates used by the industry, and which are recommended in this report, are ADAS report 345 and Flood estimation for small catchments, Institute of Hydrology Report 124. A direct comparison between these methods is not easy due the different parameters used, but in general the difference is relatively small for most typical catchments. The recommended method for determining the volume of runoff is FSSR16.

The Flood Studies methods are applicable to a wide range of catchment areas ranging from some 0.5 $km^2$ to 5000 $km^2$. The ADAS method should be applied to estimate the flow from small, rural, ungauged catchments (i.e. generally catchments with areas of 30 ha or less). It should be noted that estimation techniques for predicting runoff and flows for small catchments are not going to be very accurate and this particularly applies to small greenfield sites for development.

The choice of method of assessing greenfield runoff is important as the predicted peak runoff and volume of runoff is used to determine the runoff control measures for the developed catchment which have very rapid runoff characteristics (effectively instantaneous) by comparison. It is therefore important to make use of all local information and compare the predictions of ADAS 345 with Report 124 on small catchments to derive pre-development runoff characteristics for the site.

## 10.4 Assessment of Development runoff and storage volumes

### 10.4.1 Rate of runoff

Runoff from positively drained paved areas is effectively instantaneous by comparison to greenfield runoff. The runoff rate therefore reflects the intensity of rainfall with a little attenuation being provided by the filling of depression storage, surface runoff routing and pipe routing. This is true for all rainfall up to around 20 to 30 year return period events when short duration “summer” type of storms have intensities which are so great that brief temporary flooding takes place due to the inadequate capacity of the pipe system to cope with the volume of water.

From a site storage point of view, it is therefore relatively unimportant to determine peak flow rates from the site except to be aware that it is rapid. Information relating to the routing processes of rainfall runoff used in models is available in the Wallingford Procedure (1981) or Wallingford Procedure for Europe (2000). The important feature in determining any storage volume is the percentage rainfall that directly drains and runs off into the drainage system, and the shape of the storm profile being used.

### 10.4.2 Runoff volumes

The determination of storage is a function of runoff volume and the critical duration needed to fill it which effectively approximates to the duration of the rainfall. The Wallingford Procedure runoff models allow this storage to be determined as it is a volumetric method using rainfall hyetographs. The Wallingford Procedure runoff equations are given in Chapter 6.

The design rainfall profiles that are in current urban drainage software is derived from the Flood Studies Report, 1975. The recent work carried out for the Flood Estimation Handbook, 1999, has shown that the current design events have volumes and intensities which can be significantly different to what is currently computed in the Wallingford Procedure software which is based on FSR data.. This is likely to be addressed in due course, but until this takes place, care should be taken in defining design storms.

The critical duration event for any specific limiting discharge may be either a winter or summer profile depending on the relative volumes above the effective runoff intensity threshold for that event. The more restricted the limiting discharge, the longer will be the critical duration event. Figure 10.2 illustrates the effect of rainfall profile and limiting discharge in the determination of storage volumes.

Comparison of the urban runoff models (Wallingford Procedure) and the runoff calculation method of FSSR 16 can show that a SOIL of type 4 (clay) actually has slightly more runoff than post development runoff volumes at the 100 year return period, though at lower return periods less runoff is predicted. This is intuitively incorrect as paved surfaces have around 80 percent runoff whereas the FSR analysis cannot give values much greater than around 55 to 60 percent. Care has therefore to be taken in applying this procedure using these two equations for determining runoff volumes. The reason for this apparent discrepancy is due to the runoff assumptions for the permeable surfaces of the development catchment which in the Wallingford Procedure equations are lower than the FSSR 16 values. It can be argued that this is correct due to landscaping and other relevant development effects. This might be true, but these issues should be specifically considered.

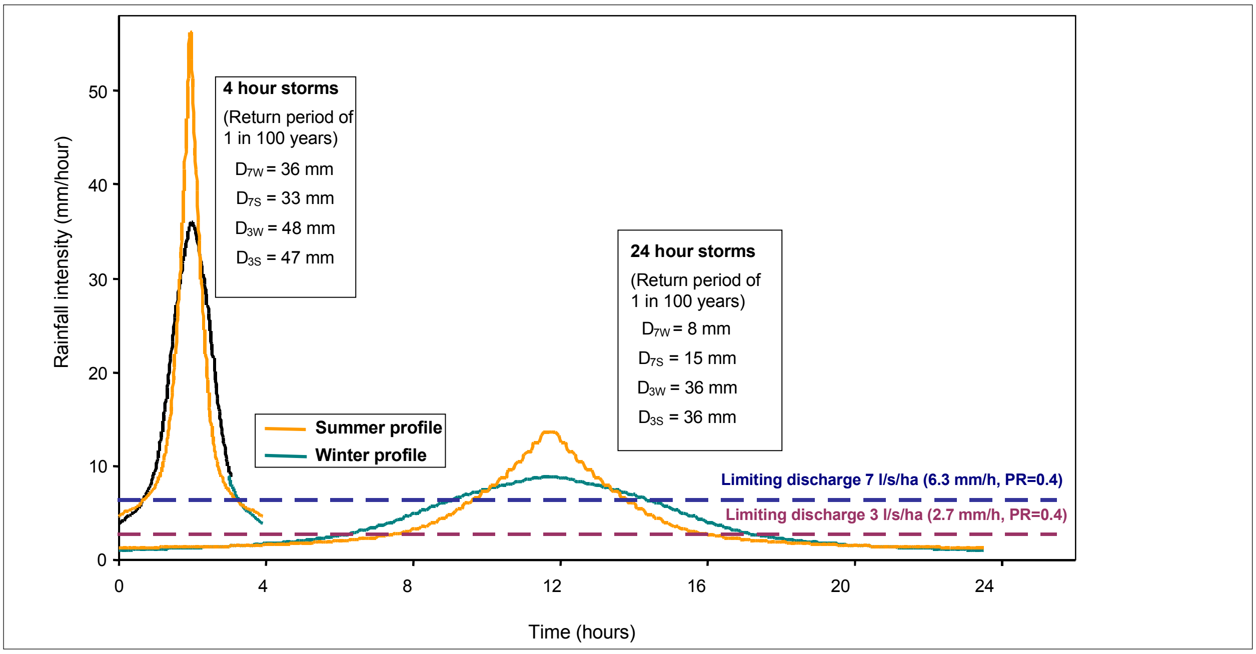



**Figure 10.2 Rainfall profile effects on critical duration events**

NB. $D_{7W}$ refers to the effective depth of storage design in terms of rainfall depth for a winter profile for a throttle of 7 l/s/ha

Therefore an alternative more conservative option is suggested that assumes pervious areas continue to generate the same runoff after development as before and a constant runoff of 80% from paved surfaces. An equation can therefore be developed which reflects this. This equation can be modified further by including terms for those paved and pervious areas which are specifically designed not to drain to any point on the network or the river. This would be developed as follows:

Additional Volume of Runoff = Paved areas draining to the network
\+ Pervious runoff draining to the network or river
— Pervious area of the greenfield site.

Therefore using the formula

$$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$$

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

Where:

| | |
|---|---|
| $Vol_{xs}$ | is the extra runoff volume ($m^3$) |
| RD | is the rainfall depth usually defined by using duration Tp~ or 12 hours (mm) |
| PIMP | is the impermeable area as a percentage of the total area (0-100) |
| A | is the area of the site (ha) |
| $PR_{RURAL}$ | is the rural runoff which uses the “soil” index from the SPR indices (values from 0.1 to 0.5) |
| $\alpha$ | is the proportion of paved area draining to the network or directly to the river (0-1) |
| $\beta$ | is the proportion of pervious area draining to the network or directly to the river (0-1) |

However if one examines the equation $PR_{RURAL}$ and in particular the indices used in the term SPR which uses constants which are virtually the same as the values of SOIL, this equation simplifies to:

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

α and β are terms defining the proportion of paved and pervious areas draining to the network. Therefore 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:

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

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

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

The following figure illustrates the excess storage volume needed for developments for different soil types and development density, assuming all pervious areas are specifically prevented from draining directly to the network or river and all paved areas are positively drained. This shows that even with SOIL type 1 that careful use of landscaping results in very little storage being needed.

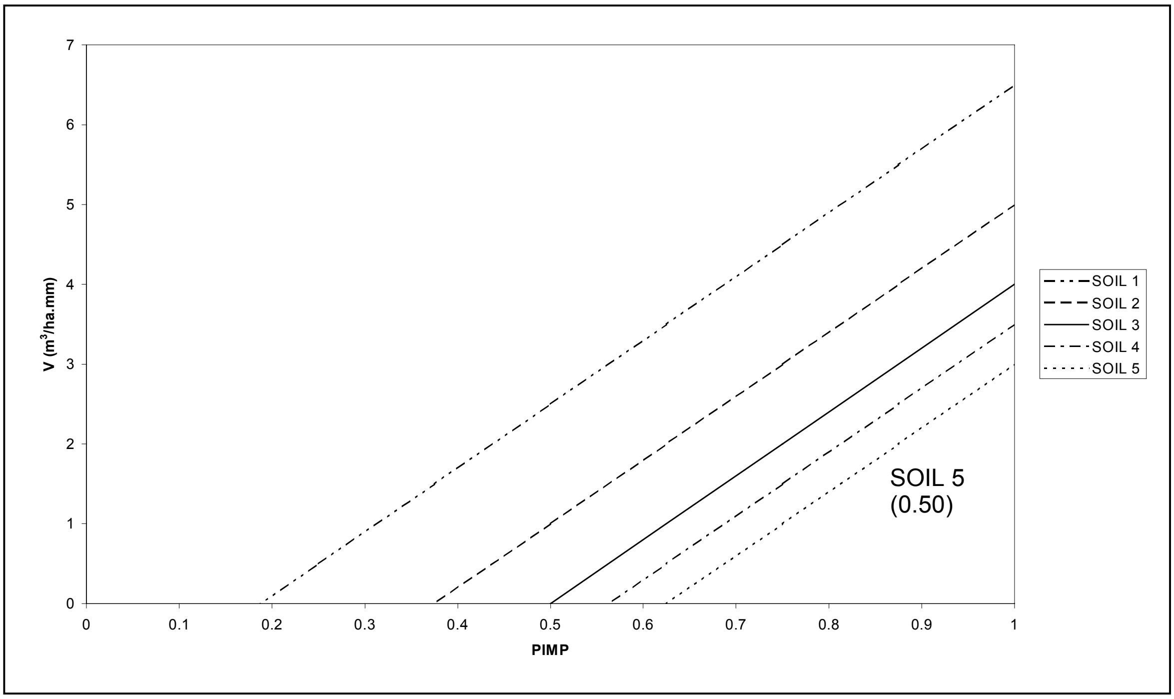



**Figure 10.3 Additional runoff caused by development with all pervious areas positively not drained**

Conversely the following graph (Figure 10.4) shows that if all of the pervious area continues to drain directly to the river, storage volumes need to be much greater for developments with low volumes of PIMP. As the development tends towards being 100 percent paving, the storage required will tend to the same value using either method.

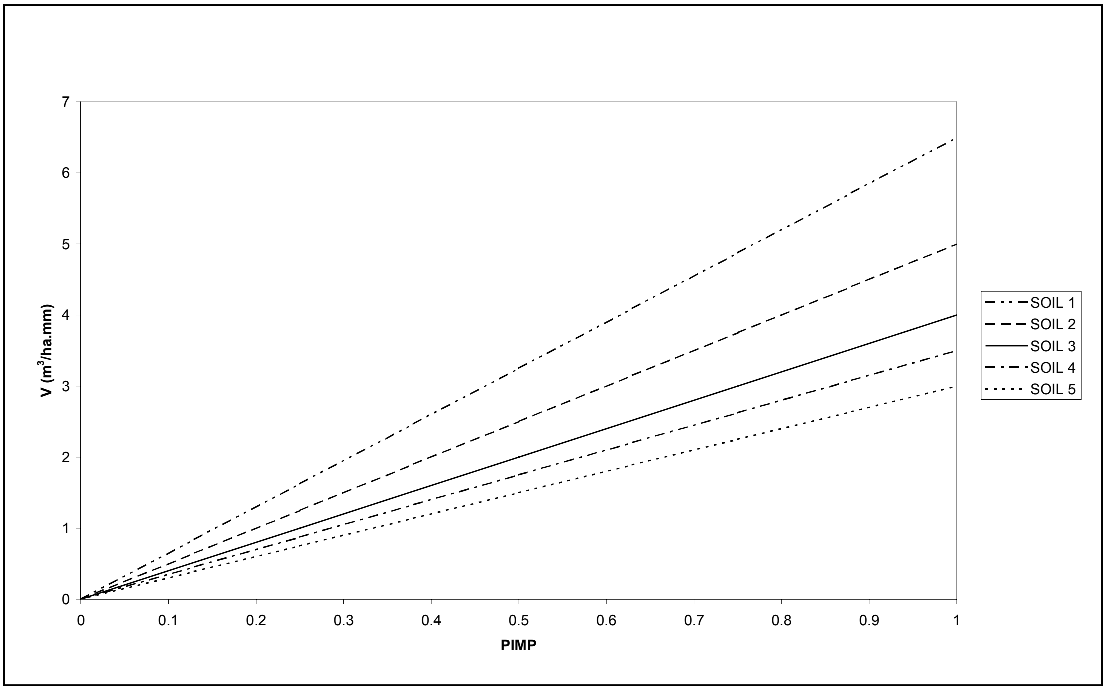



**Figure 10.4 Additional runoff caused by development with all pervious areas assumed to be positively drained**

This equation is conservative in not allowing for depression storage and making the assumption that all pervious areas can contribute, but it does provide a rapid method of assessing the maximum additional volume of runoff generated by a development.

The current requirements and also the proposed procedure are illustrated using two examples in Appendix F. The examples do not use the alternative approach for estimating excess runoff volume, but use the standard FSSR and Wallingford Procedure equations.

### 10.4.3 The calculation of $T_p$ for river flood protection analysis

The proposed methodology currently suggests that rainfall duration and therefore the event depth that should be applied to the analysis should be equal to $T_p$. $T_p$ is a value used in both FSR and FEH as a measure of the time to peak of a unit hydrograph for river catchment analysis although the correlation parameters are different. It could be loosely translated as time of concentration for urban drainage engineers.

From FEH

$$T_p(o) = 4.270\,\mathrm{DPSBAR}^{-0.35}\mathrm{PROPWET}^{0.80}\mathrm{DPLBAR}^{0.54}(1+\mathrm{URBEXT})^{-0.77}$$

where

| | |
|---|---|
| DPSBAR | Mean drainage path slope |
| DPLBAR | Mean drainage path length |
| PROP WET | Index of proportion of time that soils are saturated, based on estimates of soil moisture deficit |
| URBEXT | FEH index of fractional urban extent, judged from digital maps of land cover at 50 m intervals. |

In general Tp(o) will range from around 4 hours for very small catchments at the head of rivers through to 24 hours or more for large catchments. Most developments will be on small watercourses and streams due to the nature of topography. However on rivers such as the Thames, Tp can be much longer than 24 hours. As the storage volume is a direct function of rainfall depth which increases with duration, this means that two sites with the same level of development on the same soil type would require very different river flood protection storage if their catchment sizes are very different. The logic of using Tp cannot really be contested as river flooding is associated with the duration of the rainfall. However this would result in all developments adjacent to large rivers being far more expensive in terms of storm drainage provision than those on minor rivers. For this reason, and also for simplicity and pragmatism, it is therefore suggested that a value of Tp of 12 hours is used. This would generally be longer than the calculated value of Tp for small catchments (and therefore conservative) and also provide a reasonable basis for a realistic requirement for storage assessment to be effective for any catchment.

### 10.4.4 Effects of storage design on tank size

Throttle Limits

There are practical difficulties in meeting hydraulic criteria as adopting authorities rarely accept orifice controls or pipe sizes with diameters less than 150mm. Although there are vortex devices which can reduce the flow through a throttle unit, but still provide a free bore of 150mm, developments below a certain size will not be able to throttle the flow sufficiently to meet the stated criteria. Figure 10.5 illustrates this limitation based upon the hydraulic capacities of different pipe sizes. Although hydraulic vortex units are often used, these tend to approximate to equivalent pipe diameters of a smaller pipe and therefore equivalent curves can be developed.

Where permanent flood storage is required, the use of a throttle is unlikely to be praticable as the design rainfall intensities are so low that a throttle in the order of 1 or 2 l/s/ha may be necessary to mobilise it. This therefore indicates that the volumetric element of river flood protection is likely to be achieved by other methods (SUDS) such as ensuring that part of the paved runoff does not pass directly to the drainage system.

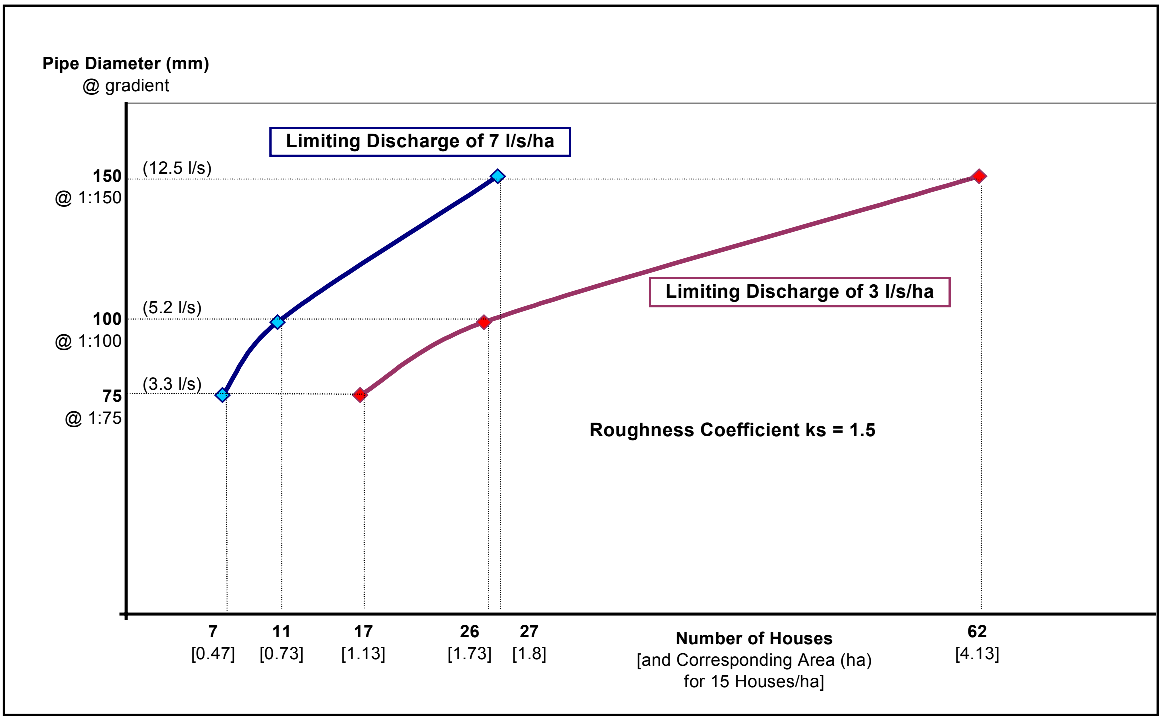



**Figure 10.5 Throttle size limitations related to development area**

Accuracy of tank sizing
Sizing of tanks is usually carried out assuming a fixed discharge limit. This discharge limit may be more than one value to meet various return period criteria. In practice this could result in a plethora of orifices at various levels in a tank, or variable weir profile controlling pond outflows. It is important to make sure the final design is practical as well as effective.

In addition it is important to realise that the head-discharge relationship of pipes (and other structures) need to be modelled accurately. In practice the difference between an assumed fixed discharge rate and a variable one can result in the tank size being increased by up to 30 percent. It is therefore important to consider tank configuration and filling methods in order to optimise the tank / pond design. Designing off-line tanks have obvious advantages in this respect of minimising the volume requirements, although they have other limitations in terms of operational and maintenance aspects.

## 10.5 Summary of methods for calculating storage requirements

The following statement provides the requirements of the current and proposed methodology for the design of stormwater storage.

Current Methodology

| | | |
|---|---|---|
| • Determine limiting discharge | → | Calculate storage requirement |

Proposed Methodology

| | | |
|---|---|---|
| • Determine range of limiting discharge rates | → | Calculate range of storage volumes |
| • Determine runoff volumes for greenfield and development sites | | Provision of "permanent"/long term storage for excess runoff |

### 10.5.1 Current methodology

The following flow charts summarise the approach currently used in calculating storage.
It should be noted that the Mean Annual Flood is quite a low rate of discharge (2.3 years) especially for sites with sandy soil characteristics and this results in very large storage volumes.

The following flow chart presumes that calculations are carried out using FSR or ADAS methods. It does not refer to COPAS, the Rational based methods or any of the other empirical approaches which are occasionally used by some engineers still.

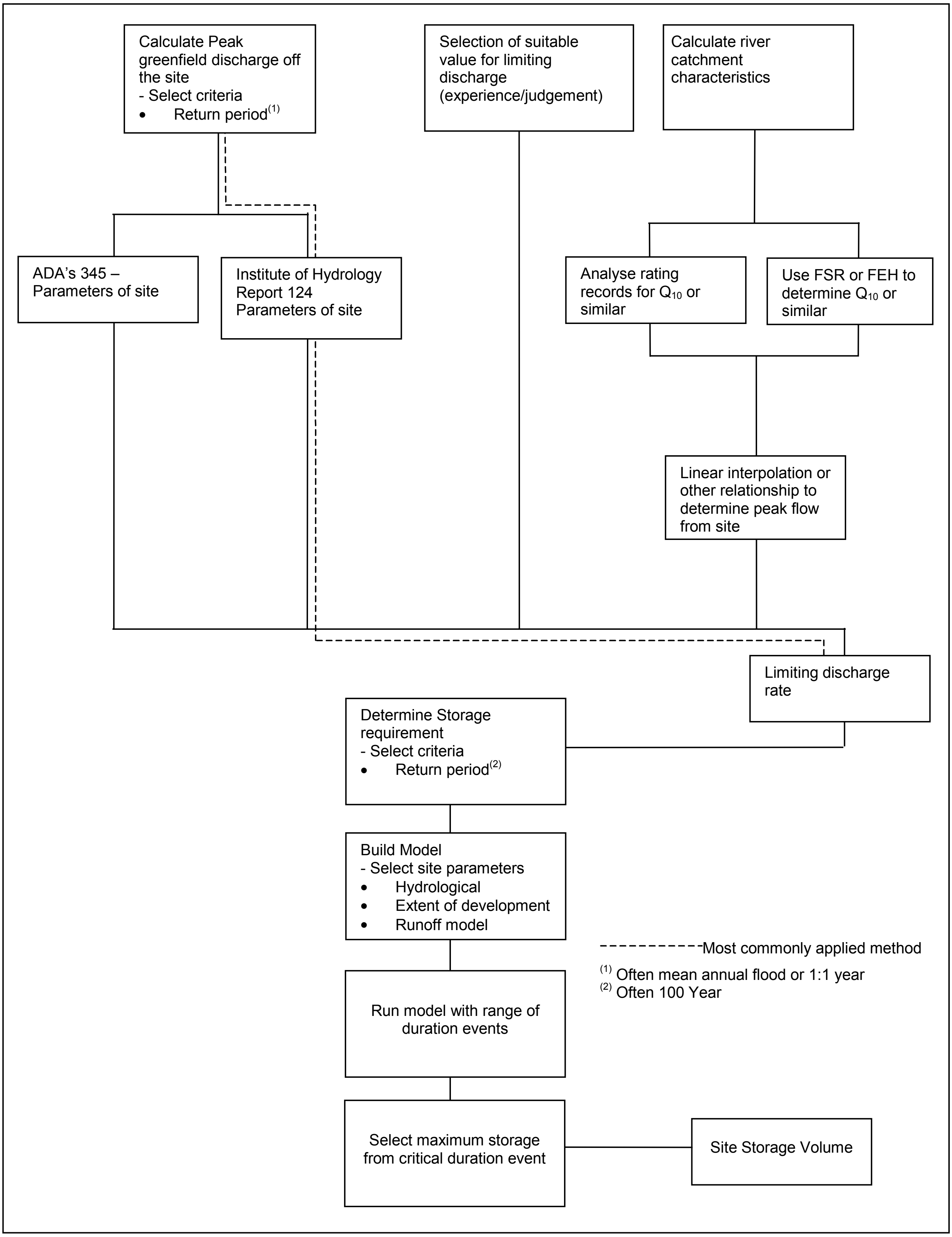



**Figure 10.6 Current methods for determining storage volume**

### 10.5.2 Proposed methodology

The following flow charts illustrate the requirements for the proposed method. Because there are up to 5 levels of storage, the process is more extended but use the same principles and a similar approach to the current procedure.

The following figure illustrates the process which is followed to determine the storage volumes rather than the one step used by the existing process. It should be stressed that the Regulating Authority may not require all five of the stages to be carried out.

It can be seen in Figure 10.7 that there is a check carried out after the calculation of Stage 4 (river flood protection). Stage 4 is the determination of the long term storage requirement. If the site system of storage proposes to mobilise this storage using a range of tanks, water levels and overflows it is quite possible that the long term storage will not be properly utilised by first designing for the "level of service" storage for the site. If this is the case, and an alternative SUDS solution is not proposed to meet this requirement, it would be necessary to mobilise the long term storage at an earlier stage i.e. after the river regime storage is utilised and before the level of service to storage comes fully into effect. This situation particularly occurs for SOIL types 1 and 2.

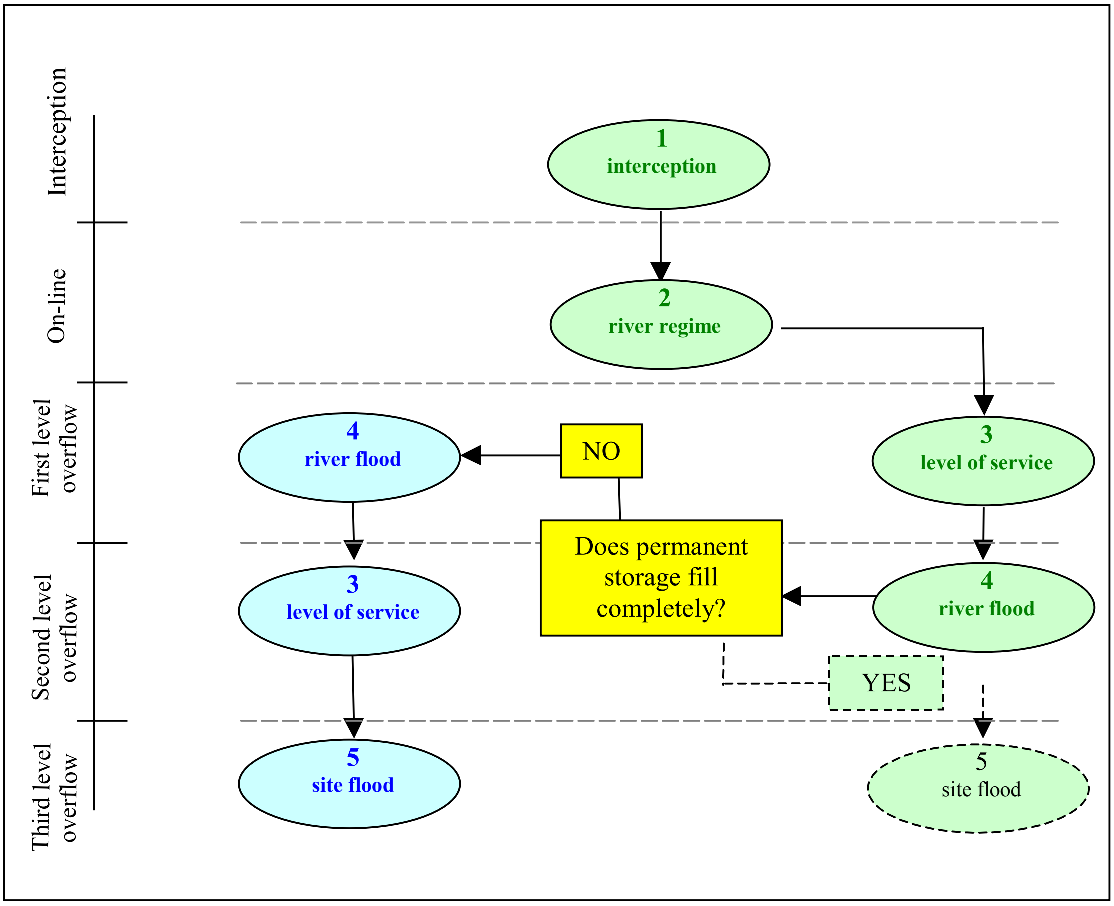



**Figure 10.7 Flow path of proposed methodology for storage volume design**

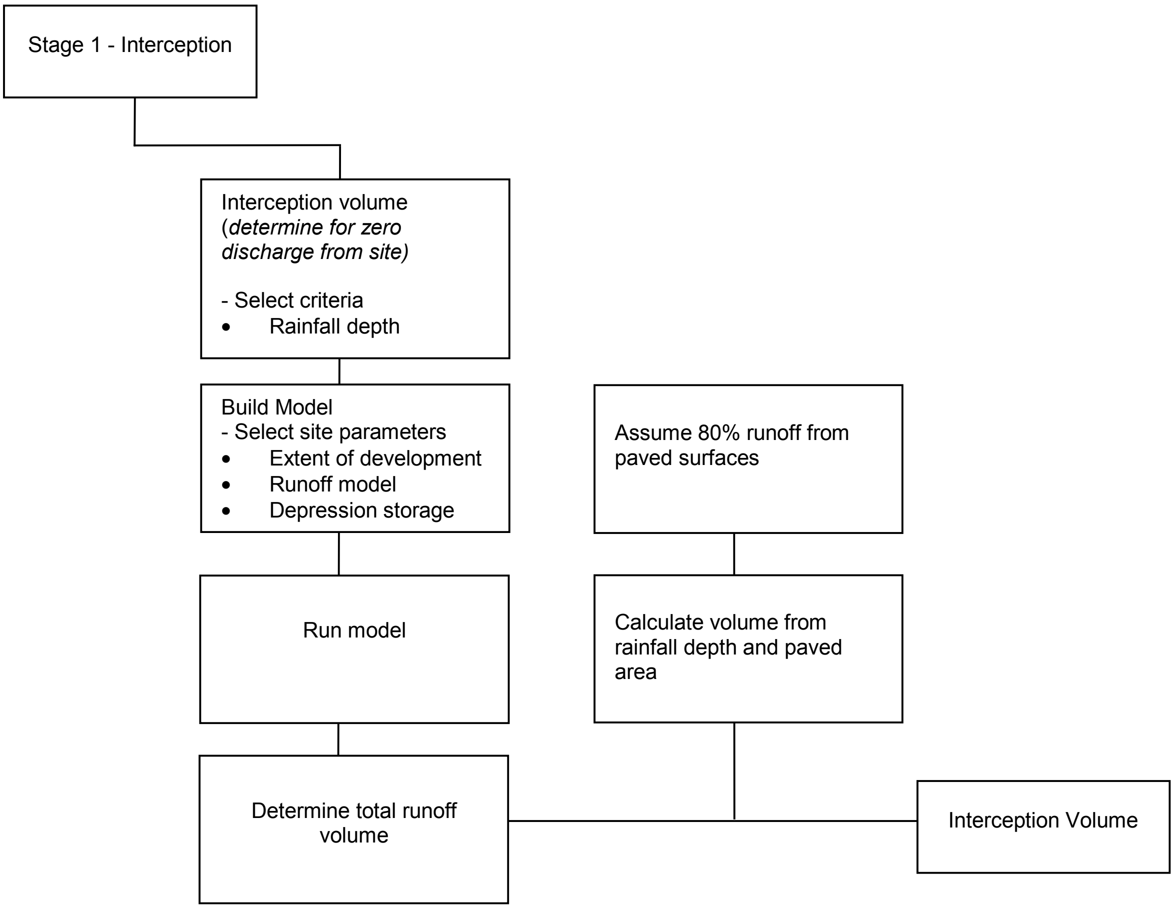



**Figure 10.8 Proposed storage design methodology – Stage 1, Interception**

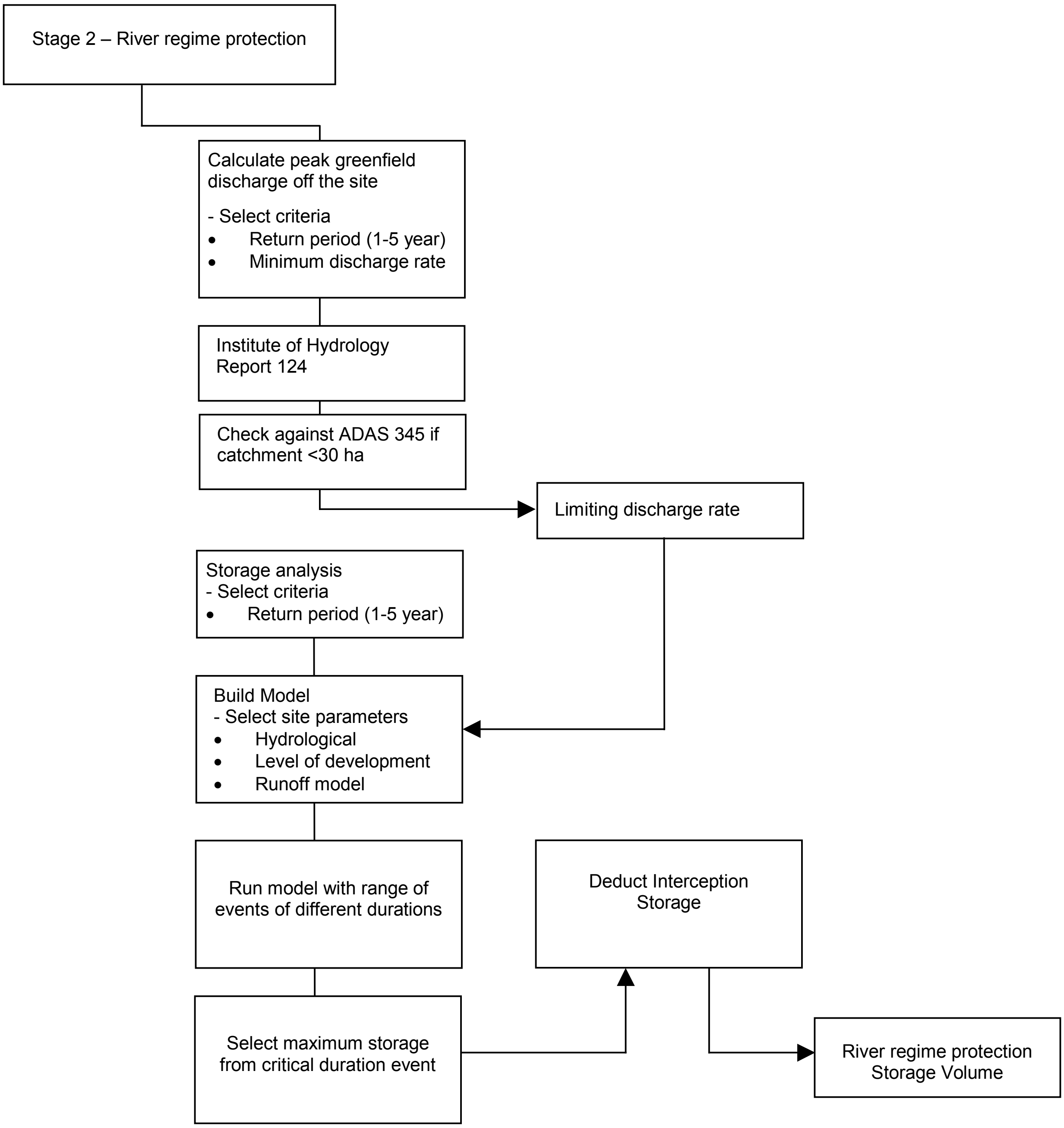



**Figure 10.9 Proposed storage design methodology – Stage 2, River Regime protection**

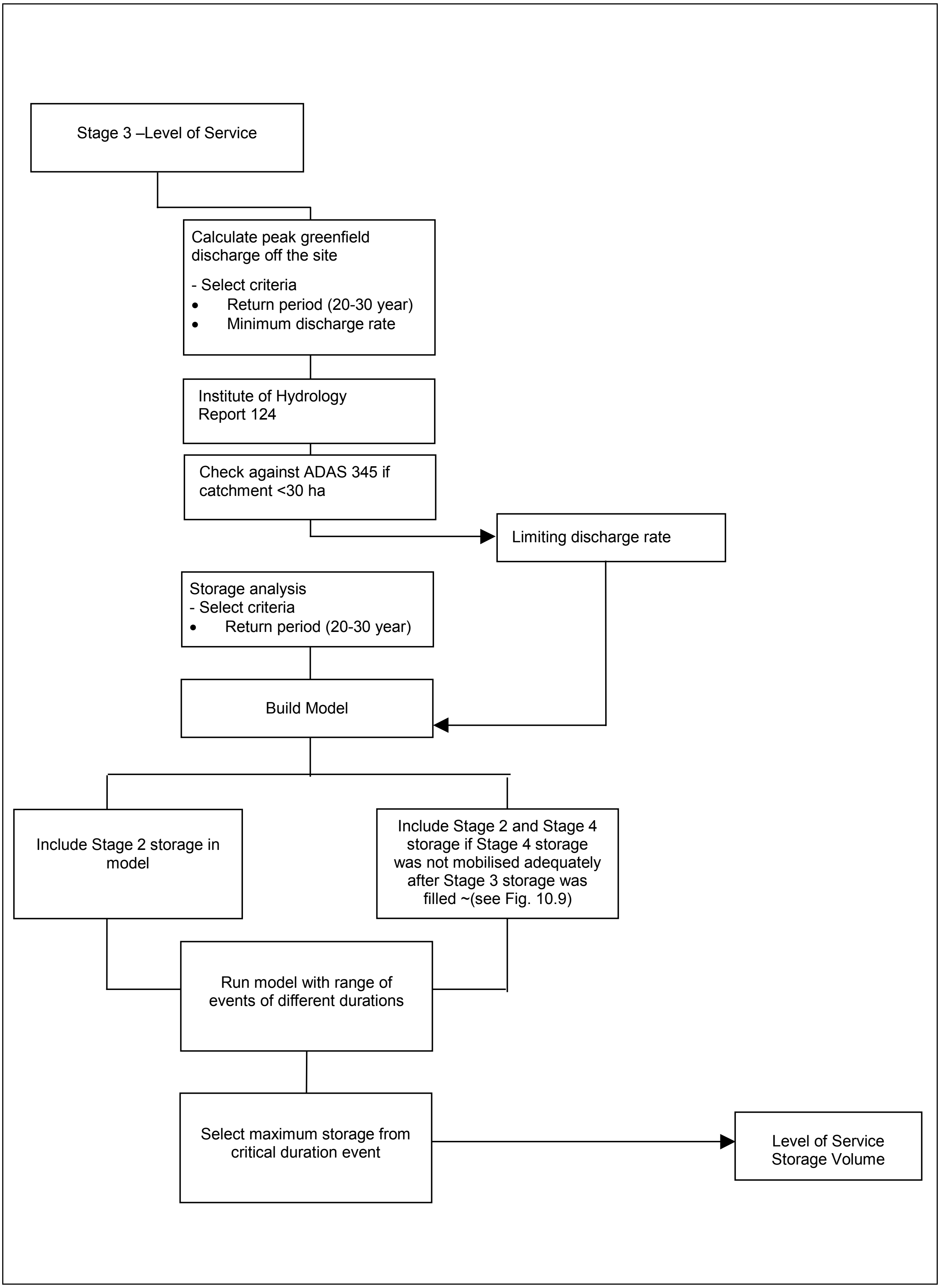
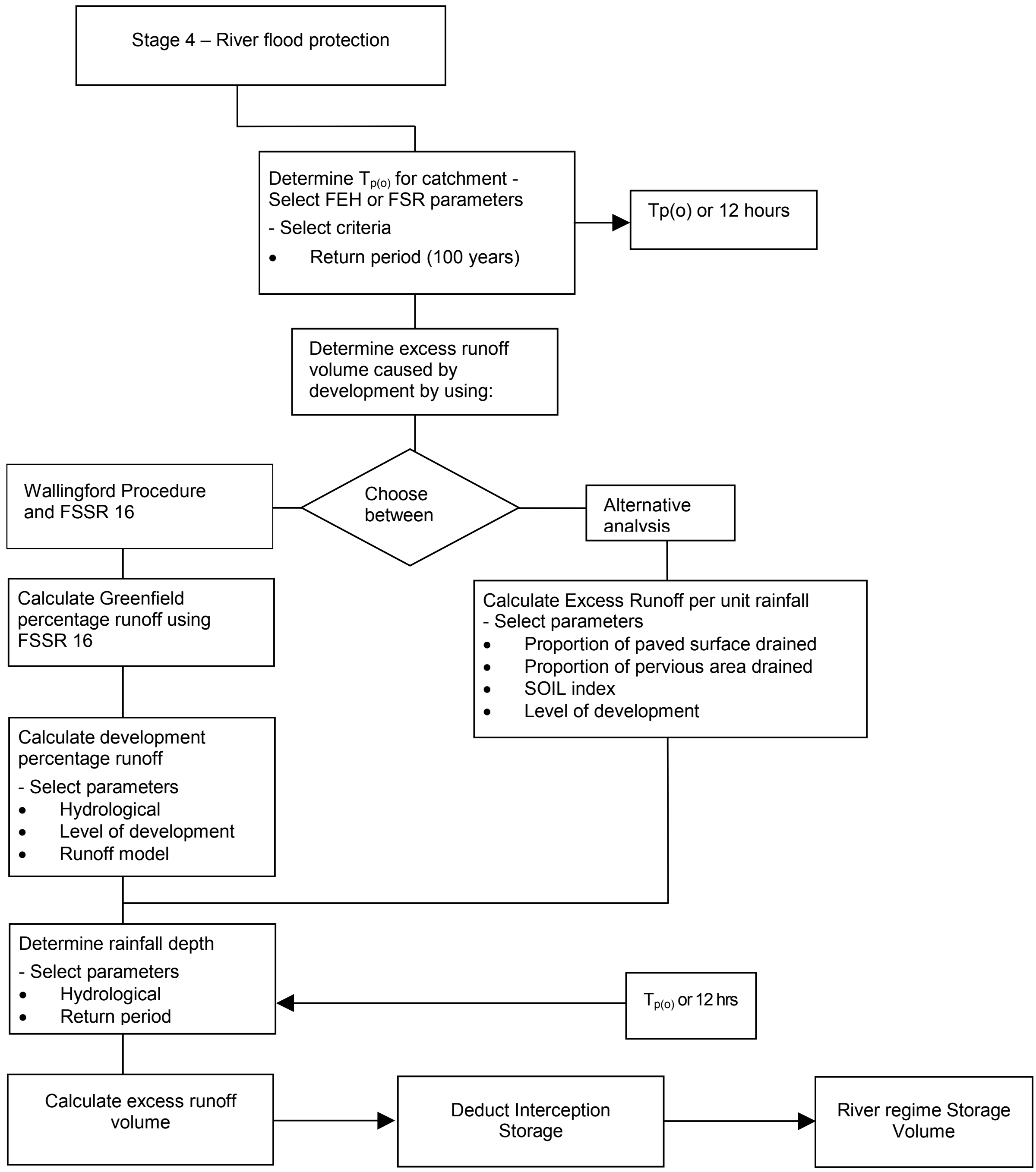



**Figure 10.10 Proposed storage design methodology – Stage 3, Level of Service**




*NOTE: This flow chart is different from all the other flow charts as the calculation is based on volumes of runoff and rates of runoff.*

**Figure 10.11 Proposed storage design methodology – Stage 4, River flood protection**

# *Appendices*

# Appendix A

Raingauge reliability analysis

# Appendix B

Storage volume return period analysis

# Appendix C

Utilisation of storage during periods of river flooding

# Appendix D

Flow in the river comparing greenfield and development runoff during flood events

# Appendix E

Rainfall events frequency and depth analysis

# Appendix F

Examples of stormwater storage design

## Appendix F Examples of stormwater storage design

The following two examples illustrate the approach for designing stormwater storage where it is needed to satisfy a limiting discharge consent.

The examples use standard computer models using fixed discharge values for the limiting discharge. It should be noted that the approximate methods normally employed for a first pass using models are often not conservative (in terms of storage volume) as the maximum flow rate for a limiting discharge is usually assumed to be effective at all water levels and therefore does not take into account head discharge relationships of outflow structures.

Figures in the examples illustrate how the model has been built and used.

### F1 Structure of appendix A

This section is in two parts. The two examples are used to illustrate both the present methodology that is generally used and the proposed methodology produced by recent research carried out at HR Wallingford. The first example is catchment "Oundle" has SOIL type 1 and the second, Hinkley has SOIL type 4.

On the basis that the 1 year greenfield runoff is used as determining the throttle limit (for the current method of analysis) this results in 0.3l/s/ha and 3 l/s/ha respectively for the two catchments. In the case of Oundle the throttle rate is increased to 1 l/s/ha as the site is assumed to be 10 ha in size. This is needed for practical drainage reasons as throttles below 10 l/s are generally impractical.

However, many regions would regard these rates as draconian so 4 other throttle rates have also been run to illustrate typical storage requirements for limiting the 100 year event.

### F2 Comparison of results from the existing and proposed procedures

The following table (Table A1) compares the storage volumes determined by the current and proposed methods using the 1 year greenfield runoff rate for making the comparison. A second table (Table A2) is given showing the other storage volumes for alternative throttle limits for the current procedure.

Table A1 shows a comparison of the two methods. The proposed method effectively splits the storage into several parts. It should be noted that all of the storage is not expected to be in the form of ponds or tanks.

In both cases, the river flood protection storage is based on the Wallingford Procedure runoff model and the FSSR 16 runoff equation. The value of $T_p$ has been calculated for the catchments (20 and 16 hours respectively).

In the case of Oundle, the traditional method results in a storage volume of 2396m$^3$. The proposed methodology suggests that 2420m is needed, but 1540m$^3$ of this would be "permanent" or indirect storage and is therefore unlikely to be provided in the form of a tank or storage pond. The volume required to protect the site using a tank is only 980m$^3$. This is likely to result in significant cost savings, depending on the mechanism used for storing the river flood protection storage.

The reason for no storage being needed for the "level of service" criterion is due to the need to mobilise as much "river flood protection" storage as possible. This flood protection is shown to be more than sufficient to cater for both the "level of service" storage and "site flood protection" storage. However the location of flooding that take place more frequently than 30 years will become an important issue.

In the case of Hinkley, the traditional approach results in a slightly smaller volume (than Oundle) needing 1873m$^3$ and the proposed method requiring even less at 1310m$^3$. Of this 1310m$^3$, "site flood protection" requires 470m$^3$ and this is unlikely to be provided in the form of a pond or tank. Thus only 840m$^3$ would be catered for specifically in the form of a pond or tank resulting in a saving of over two times that of the traditional approach. Finally it should be noted that all these numbers are dependent on parameters and equations being selected, many of which have proposed alternatives (for example 12 hours might be used instead of $T_p$) and these would affect the results. However the principles are clearly demonstrated by these two examples of the flexibility provided by the proposed method.

**Table F1 Comparison of currents and proposed methods of analysis**

| | Current method | | Proposed method | | | | | |
|---|---|---|---|---|---|---|---|---|
| Catchment | Greenfield Runoff Throttle Rate (l/s/ha) 1 | 3 | Interception | River regime | level of service | river flood protection | Site Flood protection | Total |
| Oundle | 2396 | - | 120 | 760 | - | 1540 | - | 2420 |
| Hinkley | - | 1873 | 120 | 420 | 300 | - | 470 | 1310 |

Table A2 clearly illustrates the reduction in storage volume needed for more relaxed throttle limits. However, although these throttles all protect the receiving water from flushing effects of "instantaneous" runoff, they generally provide little benefit in terms of river flood protection.

**Table F2 Current Method of analysis – Various throttle rates for 100 year critical duration events**

| | Throttle rates (l/s/ha) | | | |
|---|---|---|---|---|
| Catchment | 1 | 3 | 5 | 7 |
| Oundle ($m^3$) | 2396 | 1760 | 1476 | 1300 |
| Hinkley ($m^3$) | 2929 | 1873 | 1536 | 1343 |

## F3 General comments on the use of models

1. The New PR equation of the Wallingford Procedure has been used.
   The New PR equation is:

$$PR = IF \times PIMP + (100 - IF \times PIMP) \times \frac{NAPI}{PF}$$

   NAPI increases with rainfall depth during the event and therefore PR also increases. A design value for NAPI has been taken as zero. PF is the default value of 200mm.

2. Use of hydrodynamic models
   When modelling to determine the approximate storage required, the pipe system is often modelled with a limit of discharge throttle and an overflow. The volume passing over the overflow is the storage needed. A range of different storm durations are used to determine the maximum volume.

   When detailed design (final design) of storage is carried out, models should be built which explicitly represent all storage volumes and also the head-discharge relationship of all throttle units.

## F4 Comments relating to the proposed method of storage analysis

3. When analysing for the storage needed for a certain criterion (say “Level of Service” for the Site), the previously defined storage (say “River Regime Protection”) is modelled explicitly as a storage structure and an overflow weir is used to determine the additional storage needed.

4. Mobilising “River Flood Protection”
   The volumetric analysis for “River Flow Protection” is not affected by design event profile and is purely a comparison of pre and post development runoff volumes. This means that the PR equation is used to obtain the post development runoff volume. Although NAPI increases with rainfall during the event, this is not a linear process as there is decay function in the formula. However if an approximation for this mean value of NAPI is needed for a manual assessment of runoff volume, a conservative assumption would be to assume that the NAPI value at the end of the event was equal to the rainfall depth added to the value of NAPI at the start of the event and therefore the mean value of NAPI could be used in a calculation of PR.

   The permanent storage for “River Flood Protection” will not necessarily be adequately mobilised after storage has been utilised for the 20yr or 30yr Level of Service for the Site due to the lower intensities of longer duration rainfall. Therefore “permanent” storage may need to be designed to start coming into effect at a lower return period.

   It is advised that Time Series Rainfall is used to check that “permanent” storage is mobilised effectively. This is because volumetric aspects of storage analysis are not perfectly represented by standard design event profiles which are primarily aimed at flow rates for pipe design.

5. Definition of the Time to Peak ($T_p$)
   The estimation of the time to peak ($T_p$) has been carried out following the procedure described in the Flood Estimation Handbook (CEH 1999). The procedure uses the Unit hydrograph theory. The time to peak of the instantaneous unit hydrograph is $T_p(0)$ and can be evaluated from catchment descriptors.

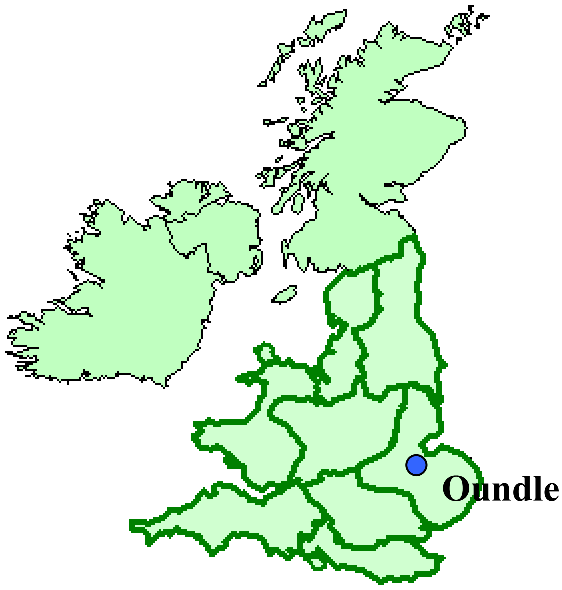
   $$T_p(0)=4.270\ \mathrm{DPSBAR}^{-0.35}\ \mathrm{PROPWET}^{-0.80}\ \mathrm{DPLBAR}^{0.54}\ (1+\mathrm{URBEXT})^{-0.77}$$

   $T_p$ has been selected as an approximation of the critical duration of the catchment.

# Appendix F  Examples part 1 – Current Method

The following two examples are illustrative of the current requirements for providing storage using a 100 year event.  A range of results is provided to illustrate the effect of using different throttle rates on the predicted storage volumes.

OUNDLE (Anglia Region)

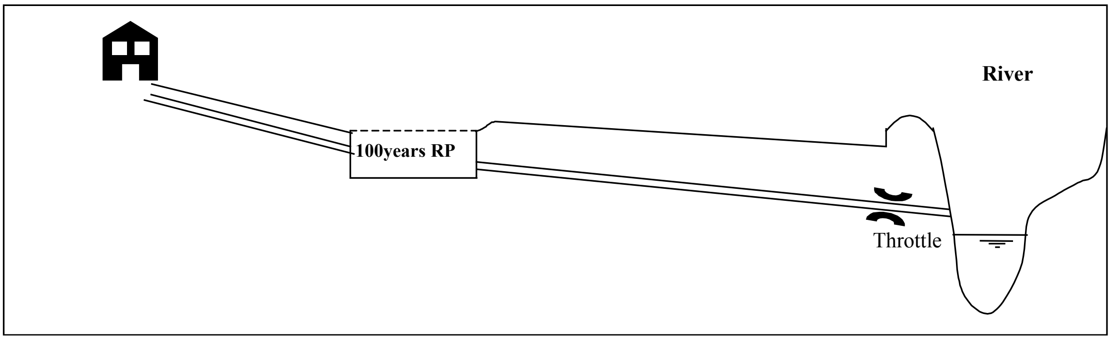



The following example illustrates the method of storage design currently practised. The theoretical catchment chosen is assumed to be at Oundle. Its characteristics are reported below together with a range of possible discharge limits to demonstrate the effect on storage volume.

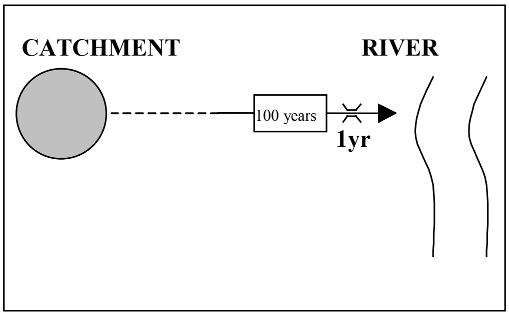
<u>CATCHMENT CHARACTERISTICS (Oundle – Anglia Region):</u>

| | |
|---|---|
| Site Area | = 10ha |
| SAAR | = 616mm |
| SOIL | = 1 |
| M5-60 | = 19mm |
| r | = 0.42 |
| PIMP | = 50% |
| IF | = 0.7 |
| PF | = 200mm |
| Throttle limits | = 1, 3, 5, 7 l/s/ha |

## CURRENT PROCEDURE (OUNDLE)




DRAINAGE MODEL




River Regime Protection requires:
Throttle limit discharge = **1**yr Greenfield Runoff
(minimum **10**l/s)

Using IOH Report Nr. 124:
$Q_{rural} = 0.00108\ AREA^{0.89}\ SAAR^{1.17}\ SOIL^{2.17}$
$\mathbf{Q} = Q_{rural} \times F$
Regional growth factor (F) = 0.70

**Oundle limit discharge**
$\mathbf{Q} = 0.003m^3/s < 0.010m^3/s$
therefore
$\mathbf{Q} = 0.010m^3/s$ (1 l/s/ha)

Model run with 1 in 100yr rainfall events.

$V_1$ l/s/ha = $2396m^3$
Critical Duration = 20 hrs

$V_3$ l/s/ha $1760m^3$
Critical Duration = 5 hrs

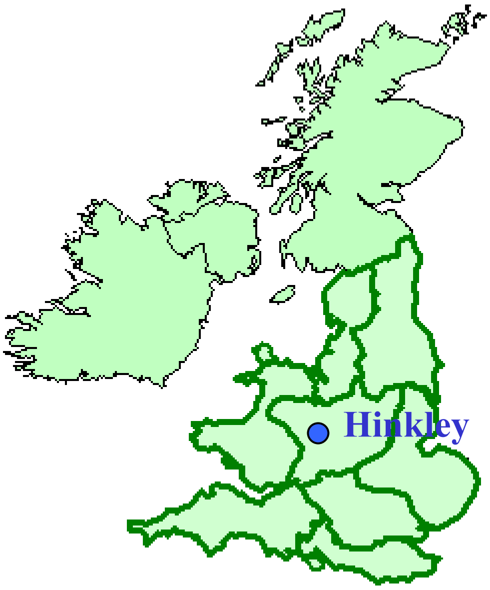
$V_5$ l/s/ha $1476m^3$
Critical Duration = 3 hrs

$V_7$ l/s/ha $1300m^3$
Critical Duration = 3 hrs

*Comments:*

Theoretically the 1 year greenfield runoff rate is only 3 l/s for this 10ha site. 10 l/s is assumed based on practical consideration of throttle sizes. This represents 1 l/s/ha which is generally considered as being onerous. Higher throttle rates have been provided for illustration as typical values are often in the range of 5 to 7 l/s/ha at present.

HINKLEY (Midlands Region)

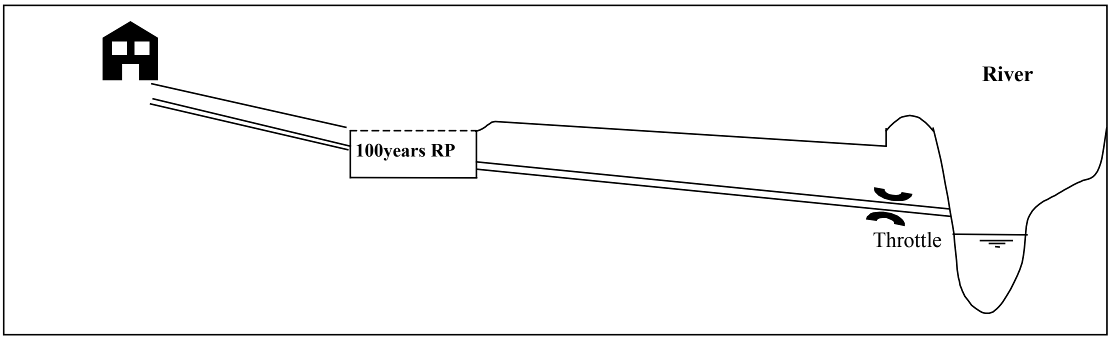



The following example illustrates the method of the storage design currently predicted. The theoretical catchment chosen is assumed to be at Hinkley. Its characteristics are reported below together with a range of possible discharge limits to demonstrate the effect on storage volume.

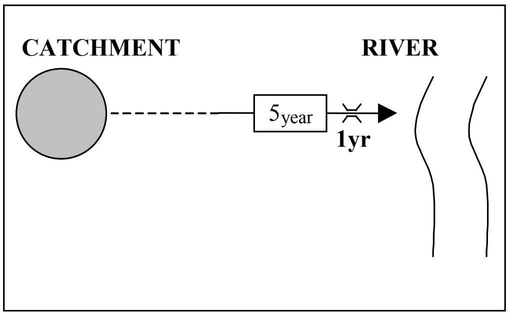
<u>CATCHMENT CHARACTERISTICS (Hinkley – Midland Region):</u>

| | |
|---|---|
| Site Area | = 10ha |
| SAAR | = 682mm |
| SOIL | = 4 |
| M5-60 | = 19mm |
| R | = 0.40 |
| PIMP | = 50% |
| IF | = 0.7 |
| PF | = 200mm |
| Throttle limits | = 1, 3, 5, 7 l/s/ha |

## CURRENT PROCEDURE (HINKLEY)




DRAINAGE MODEL




River Regime Protection requires:
Throttle limit discharge = **1**yr Greenfield Runoff
(minimum **10**l/s)

Using IOH Report Nr. 124:
$Q_{rural}$ = 0.00108 $AREA^{0.89}$ $SAAR^{1.17}$ $SOIL^{2.17}$
= 0.048$m^3$/s
**Q** = $Q_{rural}$ x F
Regional growth factor (F) = 0.65

**Hinkley limit discharge = Q**
**Q =** 0.030$m^3$/s > 0.010 $m^3$/s

Model run with 1 in 100yr rainfall event.

$V_1$ l/s/ha = 2829$m^3$
Critical Duration = 33 hrs

$V_3$ l/s/ha 1873$m^3$
Critical Duration = 7 hrs

$V_5$ l/s/ha 1536$m^3$
Critical Duration = 5 hrs

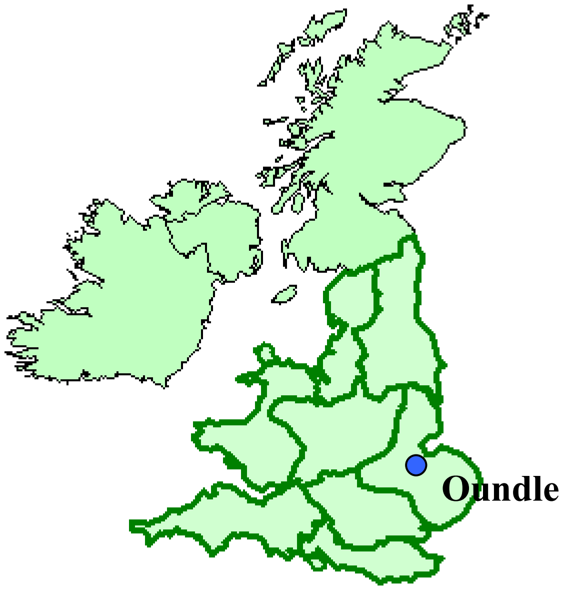
$V_7$ l/s/ha 1343$m^3$
Critical Duration = 3 hrs

*Comments:*

## Appendix F Examples part 2 – Proposed Methodology

The following examples are illustrations of the proposed storage methodology. This is more complex than the simple throttle and storage approach currently used and illustrated in Part 1, but in principle the modelling processes used are the same.

OUNDLE (Anglia Region)



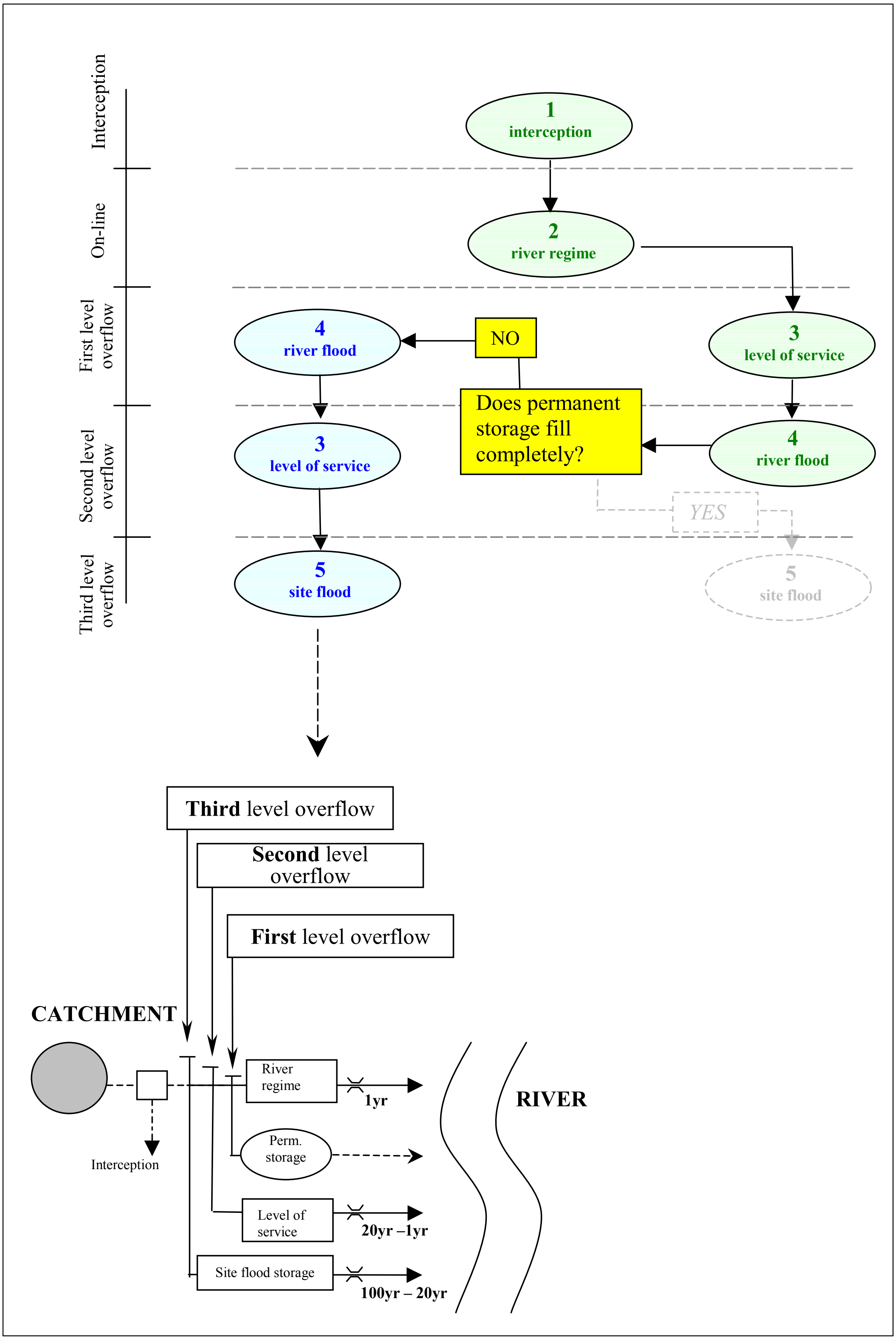

The following example illustrates the use of the proposed storage design procedure. The theoretical catchment chosen is assumed to be at Oundle. Its characteristics are reported below together with requirements for each of the criteria considered.

CATCHMENT CHARACTERISTICS (Oundle – Anglia Region):

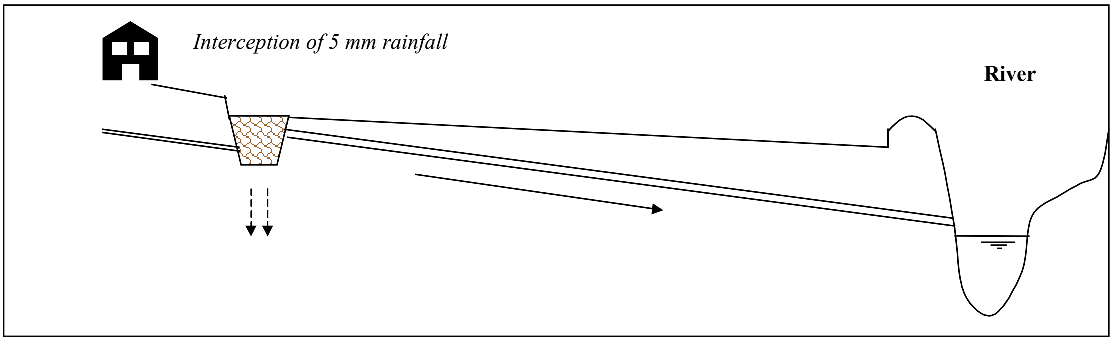
| | |
|---|---|
| Site Area | = 10ha |
| SAAR | = 616mm |
| SOIL | = 1 |
| M5-60 | = 19mm |
| r | = 0.42 |
| PIMP | = 50% |
| IF | = 0.7 |
| PF | = 200mm |
| Catchment Area | = 90km$^2$ |
| Receiving River | = River Willow Brook |
| Catchment Tp | = 20.1hr |

| **Criterion** | **Rainfall Interception** | **Runoff Limit Discharge** | **"Attenuation" Storage Event** | **"Permanent" Storage Event** | **"Flood" Storage Event** |
|---|---|---|---|---|---|
| 1. River Water Quality Protection | 5mm (min 20m$^3$) | - | - | - | - |
| 2. River Regime Protection | - | 1year RP site Greenfield runoff (min 10 l/s) | 5year RP (site critical duration) | - | - |
| 3. Level of Service for the Site | - | 20year RP site Greenfield runoff (min 10 l/s) | 20year RP (site critical duration) | - | - |
| 4. River Flood Protection | - | - | - | 100year RP (catchment critical duration) | - |
| 5. Site Flood Protection | | 100year RP site Greenfield runoff (min 10l/s) | - | - | 100year RP (site critical duration) |
| 6. Site Catastrophic Protection | - | - | - | - | - |

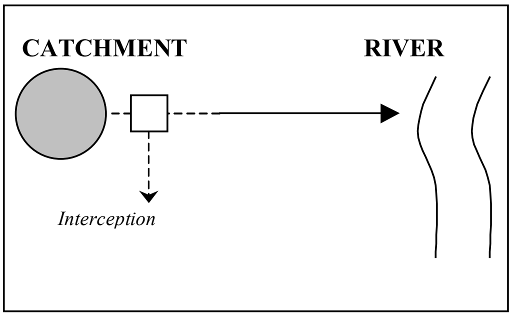
The figure below shows the analysis path followed by the example and the layout of the numerical model used for preliminary estimation of storage requirements.



# 1. PROPOSED PROCEDURE - WATER QUALITY PROTECTION (OUNDLE)




DRAINAGE MODEL




The volume needed to intercept 5mm of rainfall is:

$V_{interception}$ = 120m$^3$ [i]

Model run selected year from TSR (1993)[ii]

TSR characteristics:

Minimum rainfall depth = 2mm [iii]

Minimum antecedent dry period = 6hr

Events causing runoff: [iv]

| Interception | No. of discharges to river | |
|---|---|---|
| | Summer | All year |
| 0 mm | 20 | 56 |
| 5mm | 9 | 28 |

<u>*Comments:*</u>

The approximate volume needed to intercept 5mm of rainfall can be manually evaluated as follows:

$$\mathbf{V}_{\mathbf{interception}} = \text{Area} \times \text{PIMP} \times \text{IF} \times \text{Rainfall depth}$$
$$= 100{,}000 \text{ x } 0.5 \text{ x } 0.7 \text{ x } 0.005$$
$$= 175 \text{ m}^3$$

This assumes no runoff from permeable areas during the first 5mm of rainfall and also no depression storage on paved surfaces.

<u>*Notes:*</u>

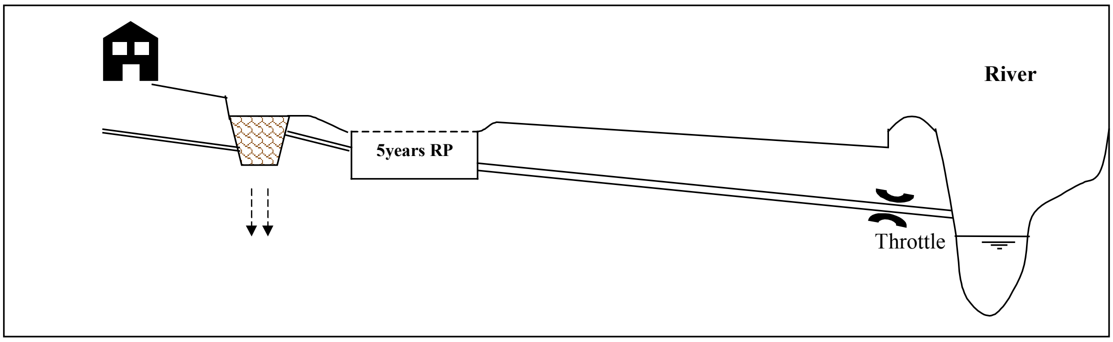
[i] Modelling method used:
- a) Create rainfall event of 5mm
- b) Standard values used for depression storage

[ii] Selection of a typical year could be improved upon by selecting 10 consecutive years. Real data (rather than stochastic) should be used if it exists.

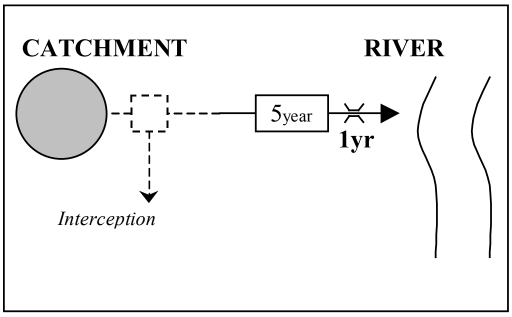
[iii] Time Series Analysis assumptions to assess number of events:
- a) Any rainfall with less than 2mm of rainfall results in minimal runoff.
- b) Any rainfall taking place within 6 hours of previous rainfall is considered to be the same event.
- c) Summer is the period May to September inclusive

[iv] Analysis of rainfall for number of events before and after subtracting 5mm depth of rain.

## 2. PROPOSED PROCEDURE - RIVER REGIME PROTECTION (OUNDLE)




DRAINAGE MODEL




River Regime Protection requires:
Throttle limit discharge = **1**yr Greenfield Runoff (minimum **10**l/s)

Using IOH Report Nr. 124:
$Q_{rural} = 0.00108\ AREA^{0.89}\ SAAR^{1.17}\ SOIL^{2.17}$
$\mathbf{Q} = Q_{rural} \times F$
Regional growth factor (F) = 0.70

**Oundle limit discharge**
$\mathbf{Q} = 0.003m^3/s < 0.010m^3/s$
therefore
$\mathbf{Q} = 0.010m^3/s$

Model run with 1 in 5yr rainfall events. The critical duration is found to be 12hr.

$\mathbf{V}_{\mathbf{interception + 5yr}} = 880m^3$ [v]

As:
$\mathbf{V}_{\mathbf{interception}} = 120m^3$
therefore
$\mathbf{V}_{\mathbf{5yr\ storage}} = 760m^3$

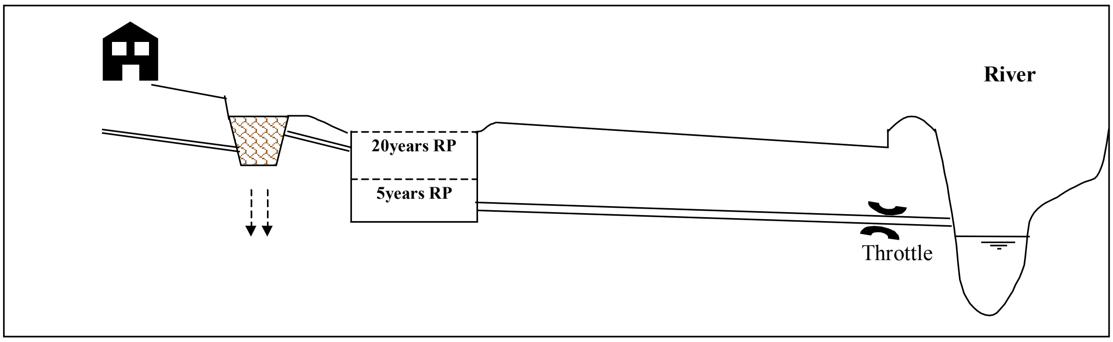
*Comments:*

The long critical duration is due to the very tight discharge limit of 1 l/s/ha. A shorter duration event would be relevant if either a smaller event was used (1 year, say) or a larger throttle limit.

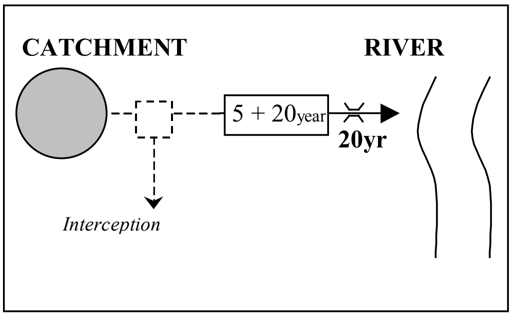
*Notes:*

[v] The model was run without the interception storage. Therefore River Regime Storage is $880m^3 - 120m^3$

## 3. PROPOSED ANALYSIS - LEVEL OF SERVICE FOR THE SITE (OUNDLE)




DRAINAGE MODEL




Level of Service for the site requires:
Limit discharge = **20**yr Greenfield Runoff (minimum **10**l/s)
Using IOH Report Nr. 124:

**Oundle limit discharge**
**Q** = $0.008 m^3/s < 0.010 m^3/s$
Therefore
**Q** = $0.010 m^3/s$

Model run with 1 in 20yr rainfall events. The site critical duration is found to be 15hr.

$V_{\text{interception}} = 120 m^3$
$V_{\text{5yr storage}} = 760 m^3$

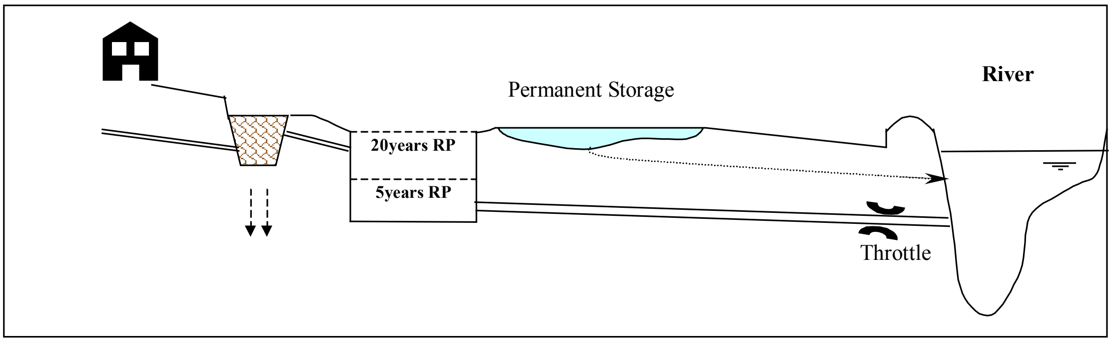
As:
$V_{\text{interception + 5yr+20yr storage}} = 1480 m^3$
Therefore:
$V_{\text{20yr storage}} = 600 m^3$ [vi]

*Comments:*

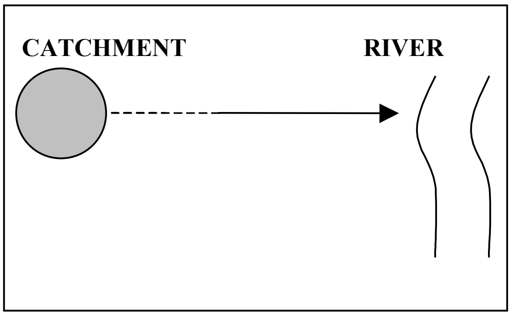
*Notes:*

[vi] The analysis of the 5yr and 20yr volumes is carried out as a single storage structure as the throttle rate is 10l/s for both cases. See Hinkley example where 5yr and 20yr criteria flow rates are different.

## 4. PROPOSED ANALYSIS - RIVER FLOOD PROTECTION – (OUNDLE)




DRAINAGE MODEL




$\mathbf{V_{100yr\ perm\ stor} = Runoff_{Develop} - Runoff_{Greenfield}}$

Using Flood Study Supplementary Report 16:

$$
\begin{aligned}
PR_{Greenfield} &= SPR + DPR_{CWI} + DPR_{rain} \\
&= 10 + 0 + 0.45\ (76.5 - 40)^{0.7} \\
&= 15.58 \\
Runoff_{Greenfield} &= PR_{Greenfield} \text{ x } A \text{ x } RD \\
&= 0.155 \text{ x } 100{,}000 \text{ x } 0.0765 \\
&= 1190m^3
\end{aligned}
$$ [vii]

Model run with 1 in 100yr rainfall event. No structures included. Using MicroFSR the catchment duration $T_p$ is equal to 20hr (see introduction note 5)

$Runoff_{Development} = 3176m^3$ (model output) [viii]

$\mathbf{V_{additional\ runoff}} = 3176 - 1190 = 1986m^3$ [ix]

Model run with 1 in 100yr winter rainfall event. Catchment duration $T_p$ = 20hr:

$V_{interception} = 120m^3$
$V_{5yr} = 760m^3$
$V_{20yr} = 600m^3$

Spill volume from model:
$\mathbf{V_{100yr\ permanent\ storage}} = 980m^3$ $<1986m^3$

*Inadequate river flood protection: analyse "river flood protection" after "river regime protection"*

*Comments:*

Mobilising "River Flood Protection"
The volumetric analysis for "River Flood Protection" is not affected by design event profile and is purely a comparison of pre and post development runoff volumes. This means that the PR equation is used to determine runoff volume after development

The permanent storage for "River Flood Protection" has not been adequately mobilised after storage has been utilised for the 20yr Level of Service for the Site. Therefore "permanent" storage will need to be designed to come into effect at a lower return period.

It is advised that Time Series Rainfall is used to check that permanent storage is mobilised effectively. This is because volumetric aspects of storage analysis are not perfectly represented by standard design event profiles which are primarily aimed at pipe size analysis and flow rates.

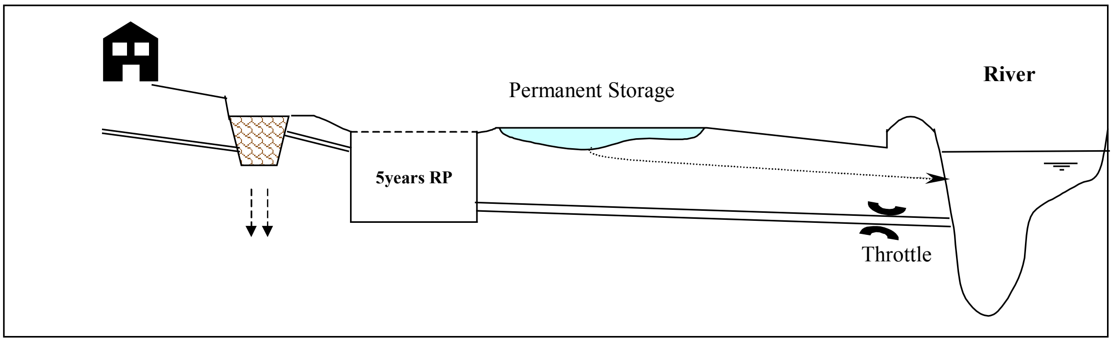
*Notes:*

vii A = Area ($m^2$) and
RD = Rainfall Depth (m)

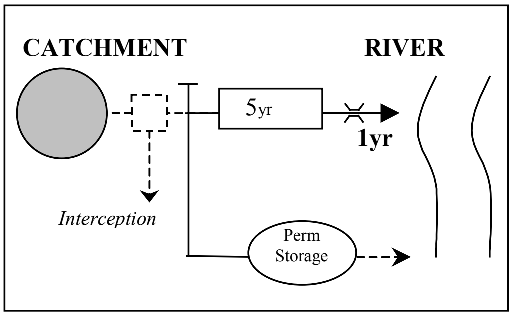
viii Model run without interception storage.

ix Storage required in order to reproduce Greenfield runoff.

## 4. PROPOSED ANALYSIS – RIVER FLOOD PROTECTION (OUNDLE)




DRAINAGE MODEL




Model run with 1 in 100yr rainfall event. Catchment duration $T_p$ = 20hr.

$V_{interception}$ = 120m$^3$
$V_{5yr\ storage}$ = 760m$^3$

$\mathbf{V_{100yr\ permanent\ storage}}$ = 1540m$^3$

Although 1540m$^3$ < 1986m$^3$ a permanent storage of 1540m$^3$ is accepted. x

*Comments:*

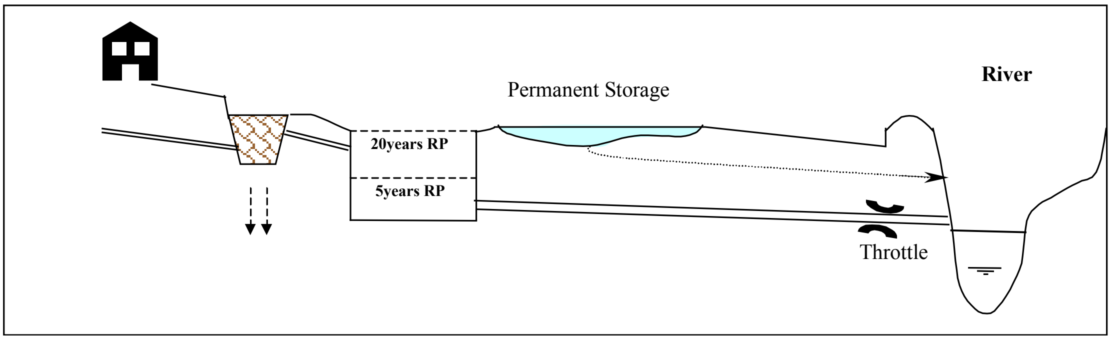
1) Note that Stage 4 is now being carried out before Stage 3 as shown in the flow chart at the start of this example.
2) This result illustrates the massive increase in runoff where development sites take place on areas of SOIL type 1.

.

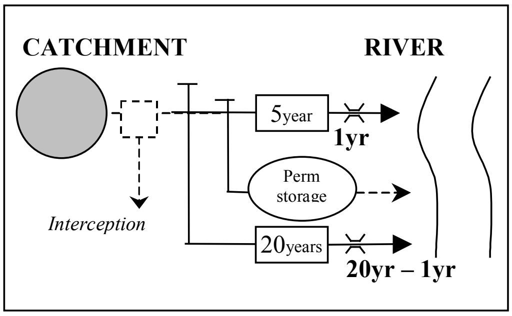
*Notes:*

x Permanent storage is not fully utilised due to the minimum throttle discharge rate of 10l/s and the rainfall profile for storm duration $T_p$. A greater volume could only be utilised if River Regime storage and Interception storage were reduced. A permanent storage of 1540m$^3$ is therefore accepted for River flood protection in this case. It is likely that for SOIL type 1 the criteria for permanent storage may be relaxed in some cases. The alternative is to have flooding of "permanent" storage taking place more frequently than every 5 years

# 3. PROPOSED ANALYSIS – LEVEL OF SERVICE (OUNDLE)




DRAINAGE MODEL




Model run with 1 in 20yr rainfall event. Site critical duration (15hr).

$V_{\text{on-line tank 5yr}}$ = 880m$^3$

$\mathbf{V}_{\text{on-line tank 20yr}}$ = 0m$^3$ [xi]

$\mathbf{V}_{\text{100yr permanent storage}}$ = 550m$^3$

$\mathbf{V}_{\text{interception + 5yr+20yr+perm.storage}}$ = 1430m$^3$

*Comments:*

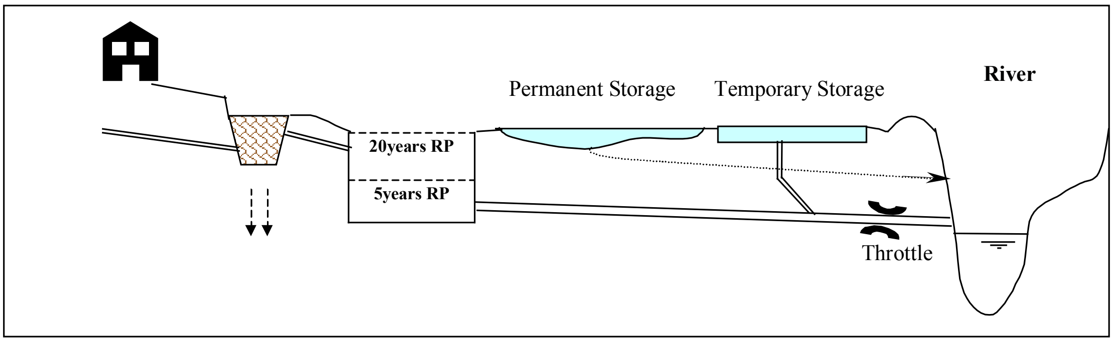
1) Note Stage 3 now being carried out after Stage 4 as shown in the flow chart at the start of this example

*Notes:*

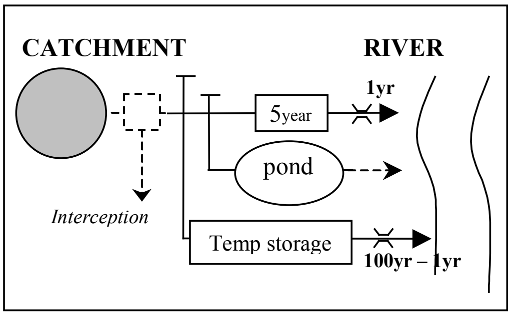
[xi] This shows that 20yr level of service storage is not required as the permanent storage of 1540m$^3$ is not filled for a return period of 20yr.

Note that throttle rate analysis is shown on Level of Service analysis previously

# 5. PROPOSED ANALYSIS - SITE FLOOD PROTECTION (OUNDLE)




DRAINAGE MODEL




Site protection requires:
Limit discharge = **100**yr Greenfield Runoff
(minimum **10**l/s)

Using IOH Report Nr. 124:

**Oundle limit discharge= 0. 015m$^3$/s**

Model run with 1 in 100yr rainfall event. The site critical duration is 20 hours [xii]

Summarising:

| | |
|---|---|
| $\mathbf{V}_{\text{interception}}$ | = 120m$^3$ |
| $\mathbf{V}_{\text{5yr}}$ | = 760m$^3$ |
| $\mathbf{V}_{\text{20yr}}$ | = 0m$^3$ |
| $\mathbf{V}_{\text{100yr permanent storage}}$ | = 1540m$^3$ |
| $\mathbf{V}_{\text{100yr flood storage}}$ | = 0m$^3$ |

As a consequence:
$\mathbf{V}_{\text{interception + 5yr+100yr perm.storage}}$ = 2420m$^3$

*Comments*

This analysis shows that the catchment requires interception and 5 year storage with all higher return period events up to 100 years all passing to “permanent” storage.

*Notes:*

[xii] Site critical duration and catchment $T_p$ are the same.

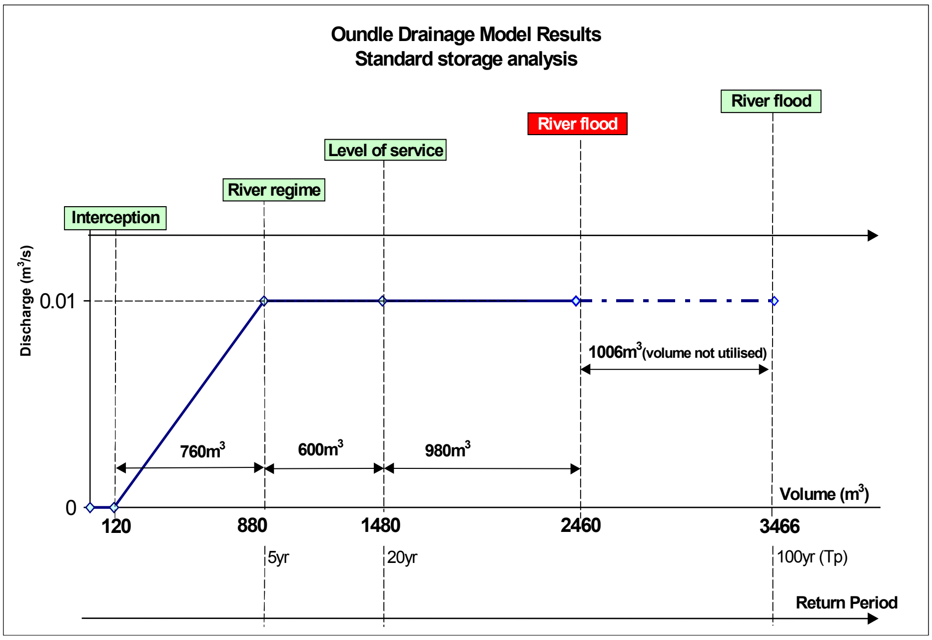
## CONCLUSIONS (OUNDLE)

***Drainage model:***

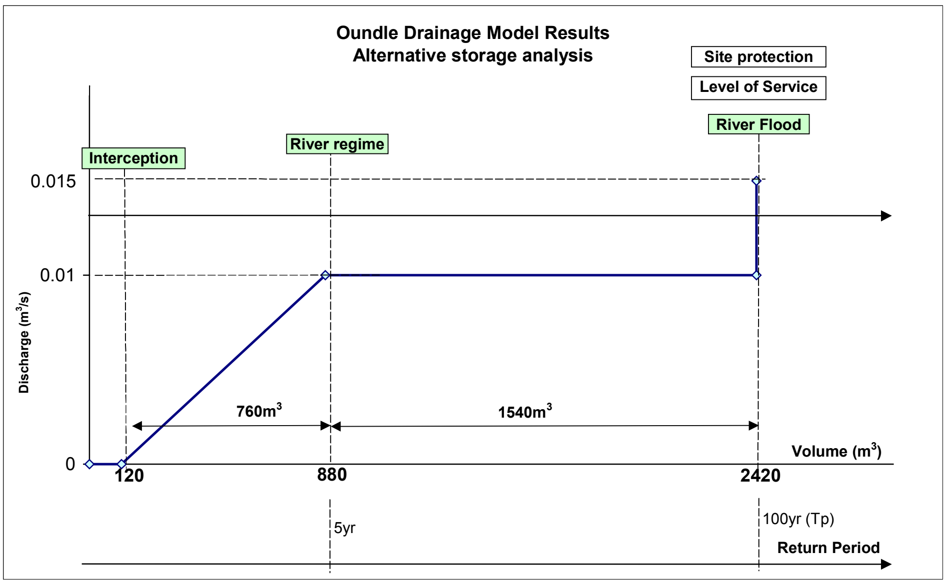
*According to the Greenfield runoff formula the 1 in 1year discharge for Oundle is 3l/s and the 1 in 20year is 8l/s. The discharge limit of 10l/s is set as a minimum throttle discharge limit and the rainfall profile for storm duration $T_p$ results in the permanent storage not being fully utilised.*

*A permanent storage of 1540m$^3$ is accepted for "River Flood Protection" because it is not possible to mobilise more volume without mobilising "permanent" storage more frequently than a 5year return period.*

*The results obtained following the standard procedure can be graphically illustrated as follows. When analysing for Level of Service before Permanent storage it showed that Permanent storage is not sufficiently utilised: river flood protection criterion is therefore not completely satisfied.*




*The following figure summarises the results obtained following the alternative storage analysis by running Stage 4 after Stage 2, before assessing Stages 3 and 5.*



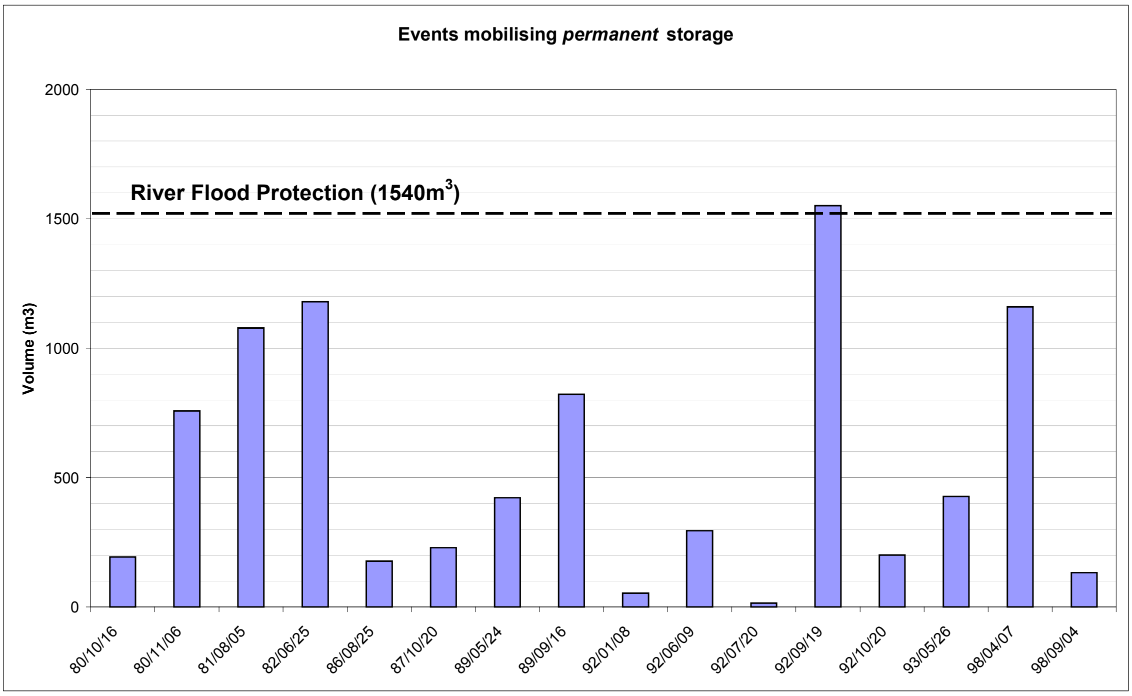
*Once the permanent storage required by River Flood Protection (1540m$^3$) is in place, it also provides the storage required by Level of Service and Site flood protection.*

***TSR Analysis:***

*The model was run with the Oundle 21 years Time Series Rainfall. The results show that the river flood protection storage is fully utilised once.*

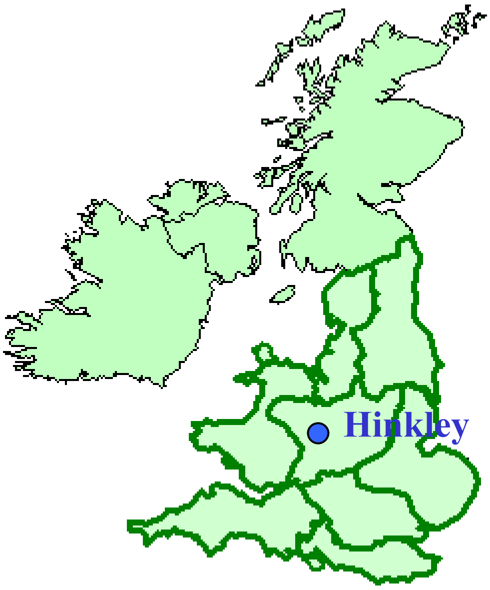
*The100yr permanent storage is mobilised 15 times. This is more frequent than expected (once every 5 years) because in the alternative storage analysis the storage volume providing River Flood Protection is mobilised immediately after the storage providing River Regime protection which is designed for a return period of 5 years. The TSR analysis indicates that it occurs nearly every year.*

*11 out of the 15 events mobilising the 100yr permanent storage are "flooding events" which means they occurred when the flow in the river was above the Q10 flow rate. This demonstrates the value of this procedure in trying to minimise river flooding. The following figure shows the 100yr permanent storage volumes mobilised during each of these events.*




*In addition to the permanent storage being mobilised, the design volume of 1540 was exceeded once by the TSR events and would have mobilised the Site Level of Service Flood Protection, if any had been provided. The 20yr Level of Service protection storage is mobilised once with 30m$^3$ on 19/9/92. The event mobilising this storage is a "river flooding event".*

HINKLEY (Midlands Region)



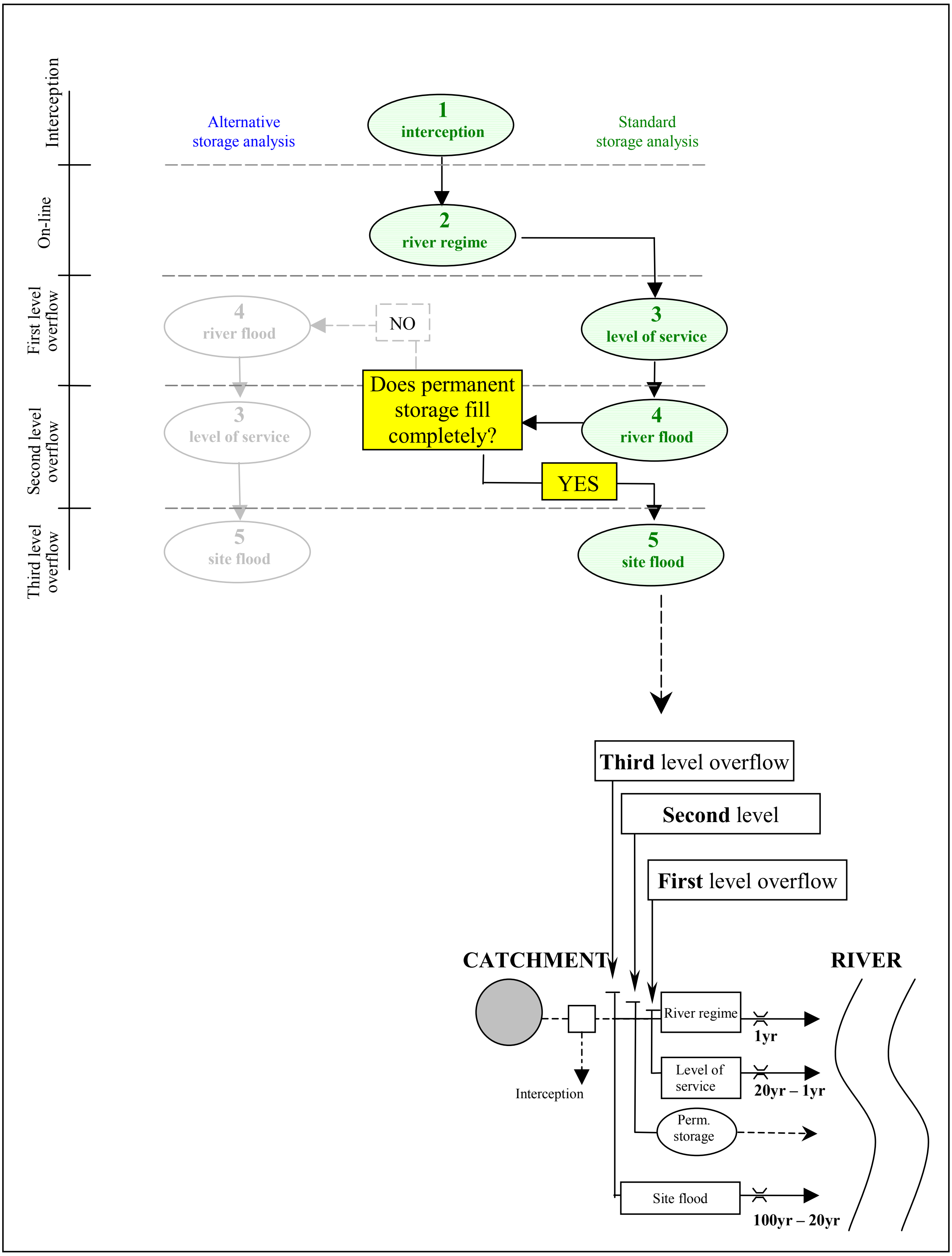

The following example illustrates the use of the storage design procedure. The theoretical catchment chosen is assumed to be at Hinkley. Its characteristics are reported below together with requirements for each of the criteria considered.

CATCHMENT CHARACTERISTICS (Hinkley – Midland Region):

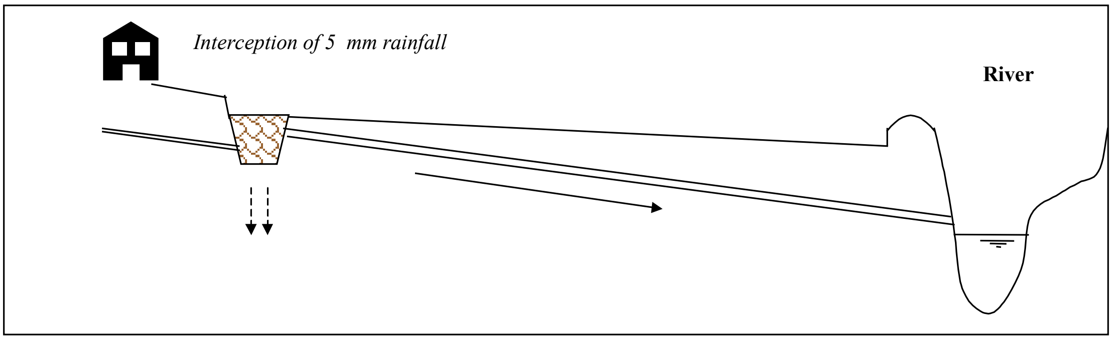
| | |
|---|---|
| Site Area | = 10ha |
| SAAR | = 682mm |
| SOIL | = 4 |
| M5-60 | = 19mm |
| R | = 0.40 |
| PIMP | = 50% |
| IF | = 0.7 |
| PF | = 200mm |
| Catchment Area | = 262km$^2$ |
| Receiving River | = River Sowe |
| Catchment Tp | = 15.6hr |

| **Criterion** | **Interception of rainfall** | **Runoff Limit Discharge** | **"Attenuation" Storage Event** | **"Permanent" Storage Event** | **"Flood" Storage Event** |
|---|---|---|---|---|---|
| 1. River Water Quality Protection | 5mm | - | - | - | - |
| 2. River Regime Protection | - | 1year RP site Greenfield runoff (min 10l/s) | 5year RP (site critical duration) | - | - |
| 3. Level of Service for the Site | - | 20year RP site Greenfield runoff (min 10l/s) | 20year RP (site critical duration) | - | - |
| 4. River Flood Protection | - | - | - | 100year RP (catchment critical duration) | - |
| 5. Site Flood Protection | | 100year RP site Greenfield runoff (min 10l/s) | - | - | 100year RP (site critical duration) |
| 6. Site Catastrophic Protection | - | - | - | - | - |

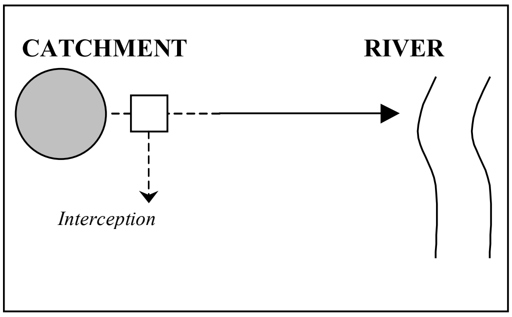
The figure below shows the path followed during the examples and the layout of the numerical model use for preliminary estimation of storage requirements.



# 1. PROPOSED ANALYSIS - RIVER WATER QUALITY PROTECTION (HINKLEY)




DRAINAGE MODEL




The volume needed to intercept 5mm of rainfall is:

$V_{\text{interception}} = 120\text{m}^3$ [xiii]

Model run selected year from TSR (1993) [xiv]
TSR characteristics:
Minimum rainfall depth = 2mm [xv]
Minimum antecedent dry period = 6hr

Events causing runoff [xvi]

| Interception | No of discharges to river | |
|---|---|---|
| | Summer | All year |
| 0mm | 25 | 68 |
| 5mm | 12 | 36 |

*Comments:*

The volume needed to intercept 5mm of rainfall can be manually evaluated as follows:

$$\begin{aligned} \mathbf{V}_{\textbf{interception}} &= \text{Area} \times \text{PIMP} \times \text{IF} \times \text{Rainfall depth} \\ &= 100{,}000 \text{ x } 0.5 \text{ x } 0.7 \text{ x } 0.005 \\ &= 175 \text{ m}^3 \end{aligned}$$

*Notes:*

[xiii] Modelling method used:
Standard rainfall event of 5mm
Standard values used for depression storage

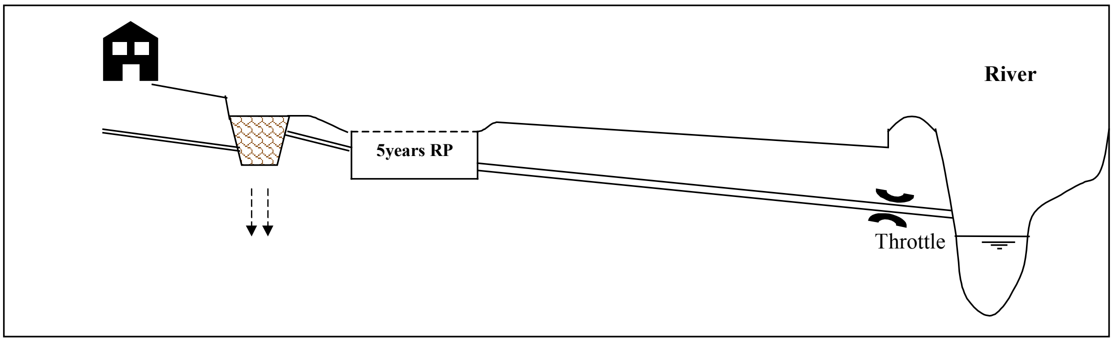
[xiv] Selection of a typical year could be improved upon by selecting 10 consecutive years. Real data (rather than statistic) should be used.

[xv] Time Series Analysis assumptions:
Any rainfall with less than 2mm of rainfall results in minimal runoff.
Any rainfall taking place within 6 hours of previous rainfall is considered to be the same event.

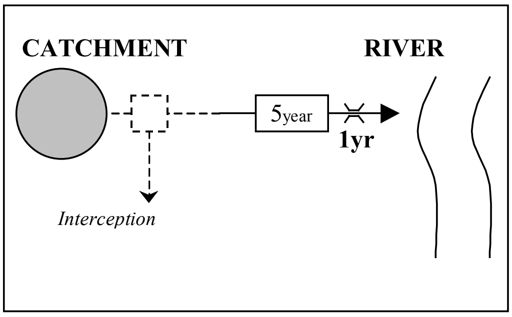
[xvi] Analysis of rainfall for number of events before and after subtracting 5mm depth of rain.

Summer is May to September inclusive.

## 2. PROPOSED PROCEDURE - RIVER REGIME PROTECTION (HINKLEY)




DRAINAGE MODEL




River Regime Protection requires:
Throttle limit discharge = **1**yr Greenfield Runoff
(minimum **10**l/s)

Using IOH Report Nr. 124:
$Q_{rural}$ = 0.00108 $AREA^{0.89}$ $SAAR^{1.17}$ $SOIL^{2.17}$
= $0.048m^3/s$
**Q** = $Q_{rural}$ x F
Regional growth factor (F) = 0.65

**Hinkley limit discharge = Q**
**Q =** $0.030m^3/s > 0.010\ m^3/s$

Model run with 1 in 5yr rainfall event. The site critical duration is 4 hours

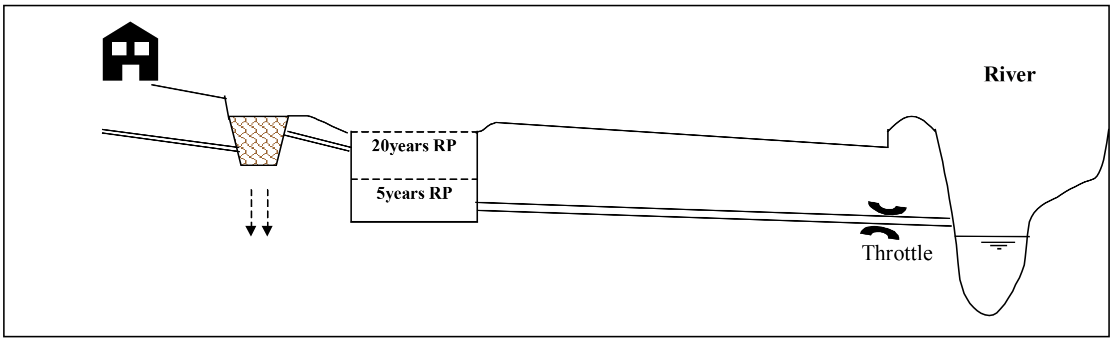
$\mathbf{V}_{\text{interception + 5years}}$ = $540m^3$

As:
$\mathbf{V}_{\text{interception}}$ = $120m^3$
Therefore
$\mathbf{V}_{\text{5yr storage}}$ = $420m^3$

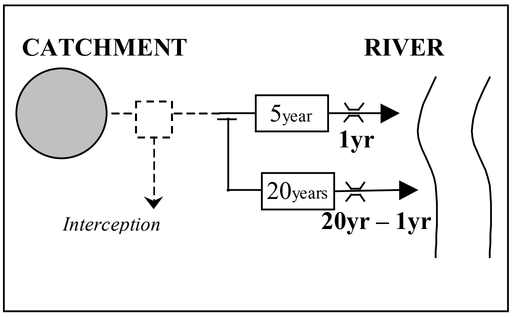
*Comments:*

*Notes:*

## 3. PROPOSED ANALYSIS - LEVEL OF SERVICE FOR THE SITE (HINKLEY)




DRAINAGE MODEL




Level of Service for the site requires:
Limit discharge = **20**yr Greenfield Runoff
(minimum **10**l/s)

Using IOH Report Nr. 124:

**Hinkley limit discharge**
**Q** = $0.084 m^3/s > 0.010 m^3/s$
therefore
**Q** = $0.084 m^3/s$

Model run with 1 in 20yr rainfall events.
The site critical duration is 4 hours

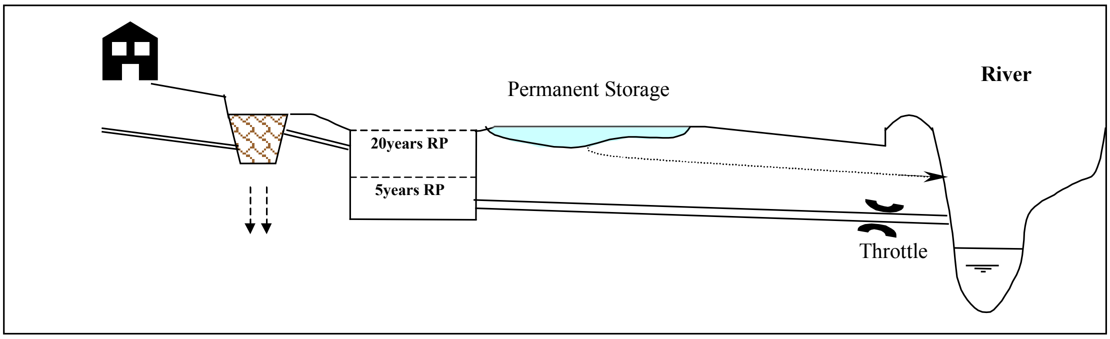
$V_{\text{interception}} = 120 m^3$
$V_{\text{5yr storage}} = 420 m^3$

As:
$V_{\text{interception + 5yr+20yr storage}} = 840 m^3$
therefore:
$V_{\text{20yr storage}} = 300 m^3$ [xvii]

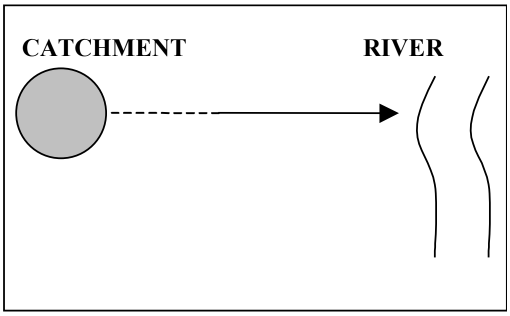
*Comments:*

*Notes:*
[xvii] The analysis of the 5yr and 20yr volumes is carried out considering two storage structures connected by a weir. When the 5yr storage is full ($540 m^3$) the water spills into the 20yr storage ($300 m^3$). The total discharge rate to River is 84l/s: because 30l/s is the discharge limit for River Regime Protection (see Step 2) the remaining 54l/s represent the limit discharge from the 20yr storage structure.

# 4. PROPOSED ANALYSIS - RIVER FLOOD PROTECTION – (HINKLEY)




DRAINAGE MODEL




$\mathbf{V_{100yr\ perm\ stor} = Runoff_{Develop} - Runoff_{Greenf}}$

Using Flood Study Supplementary Report Nr.16:

$$PR_{Grenfield} = SPR + DPR_{CWI} + PR_{rain}$$
$$= 47 + 0 + 0.45\,(75.15-40)^{0.7}$$
$$= 52.43$$

$$Runoff_{Greenfield} = PR_{Greenfield} \times A \times RD$$
$$= 3940 m^3$$

Model run with 1 in 100yr rainfall event.
Catchment duration $T_p$ = 16hr

$Runoff_{Development}$= 3260$m^3$ (model output) [xviii]

$\mathbf{V}_{\textbf{additional runoff}}$ = 3260 – 3940
= -680$m^3$ [xix]

Because Greenfield runoff is (theoretically) more than the runoff from the development, the system does not require any Permanent Storage. [xx]

*Comments:*

1. Although the mathematical basis for the analysis is correct, judgement is needed as to whether the rate of runoff is such that there should be some Permanent Storage utilised (see Chapter 10 for alternative analysis options). In addition it must be recognised that the PR equations (urban and rural) have their limitations in terms of accuracy, even though they represent current best practice.

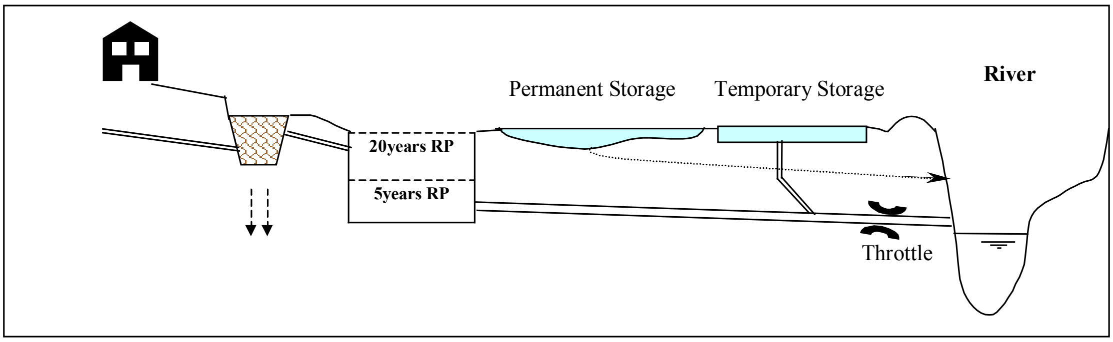
*Notes:*

[xviii] Model run without storage.

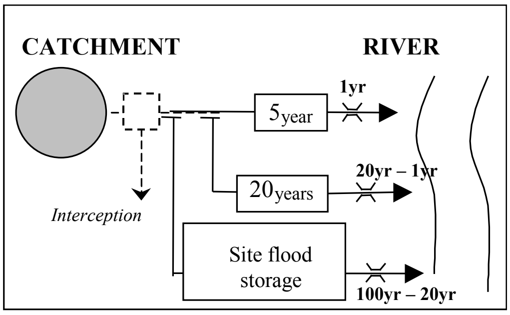
[xix] Storage required in order to reproduce Greenfield runoff.

[xx] This reflects the possible limited accuracy of the two equations used for determining runoff volumes.

## 5. PROPOSED ANALYSIS - SITE FLOOD PROTECTION (HINKLEY)




DRAINAGE MODEL




Site protection requires:
Limit discharge = **100**yr Greenfield Runoff (minimum **10**l/s)

Using IOH Report Nr. 124 (F = 2.45):

**Hinkley limit discharge = 0.122m$^3$/s**

Model run with 1 in 100yr rainfall event. The site critical duration is 4 hours

Summarising:

$V_{\text{interception}}$ = 120m$^3$
$V_{\text{5yr storage}}$ = 420m$^3$
$V_{\text{20yr storage}}$ = 300m$^3$
$V_{\text{100yr permanent storage}}$ = 0m$^3$

$V_{\text{100yr flood storage}}$ = 470m$^3$

As a consequence:
$V_{\text{interception + 5yr+20yr+ 100yr temp.storage}}$ = 1310m$^3$

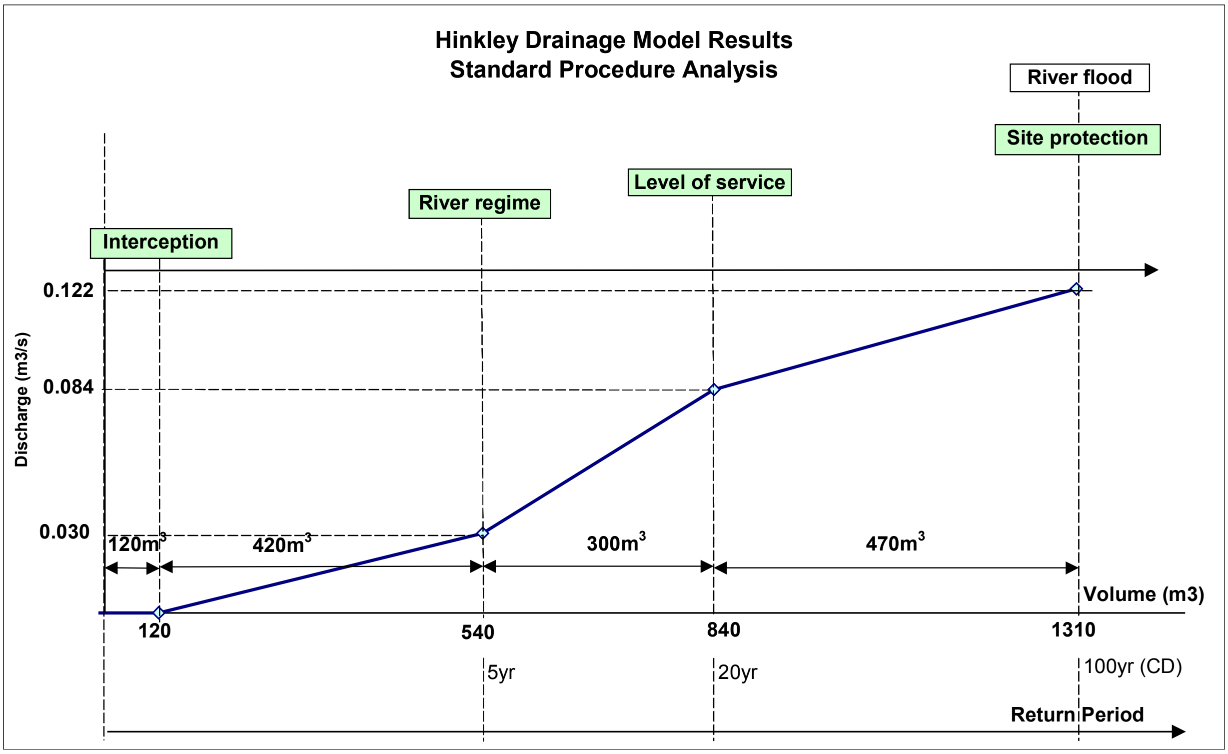
*Comments:*

*Notes:*

## CONCLUSION - HINKLEY

***Drainage model***

*The volume of runoff for green-field and the developed site is virtually the same and therefore (theoretically) there is no permanent storage requirement needed. The results obtained are summarised in the graph below.*




*Discussion on an alternative and more conservative assumption on runoff volumes is given in Chapter 10. The Environment Agency may wish to accept either method of runoff volume analysis for "Permanent" Storage assessment depending on other considerations, principally the receiving river and its behaviour.*

***TSR Analysis***

*The model was run with the Hinkley 36 years Time Series Rainfall. The results showed that the 20yr storage structure was filled once. On this occasion (rainfall event dated 10/07/68) 725m$^3$ spilled into and passed through the 20yr storage structure. There was no predicted use of Site Flood Storage.*

*Unfortunately no flow records are available for the River Sowe before 1978 and therefore it was not possible to check whether the event mobilising the 20yr storage volume occurred during a period of high flow in the river.*

*These TSR results indicate that the design storage provided is approximately correct.*