6 STORMWATER DRAINAGE DESIGN

One of the most important factors in designing sustainable stormwater drainage systems is the physical storage volume that needs to be provided to achieve flood control and minimise the pollution impact of urban stormwater runoff.

This section on stormwater drainage begins by examining the performance of current drainage systems and the conditions that lead to both flooding and poor water quality. Further information on the management of inflow and infiltration can be found in the Regional Drainage Policy on Inflow, Infiltration and Exfiltration.

Design and assessment criteria for sewers, rivers and SuDS measures are proposed together with design principles and procedures for estimating volumes of individual SuDS facilities. Appendix E provides an illustration of the approach for assessing stormwater storage requirements.

It is important to realise that all drainage systems are designed to a set of criteria that are subject to economic, social and environmental constraints. It is not feasible to design for all circumstances and there will always be instances when extreme events will exceed the design criteria. The design process therefore should be one of risk management, whereby the consequences of larger events than the design event are assessed for their cost and environmental impacts.

6.1 The Impact of Urban Stormwater Runoff

Rainfall runoff in an urban environment effectively takes place instantly for areas served by traditional drainage systems and nearly all the rain that falls on impermeable surfaces runs off. The rate of runoff and the volume of runoff are both important components in analysing the performance of a network. For storms above a certain magnitude the performance of the network downstream may be exceeded. Rainfall-related flooding of the drainage network, simply defined, is the concentration of stormwater to a point from which it cannot escape quickly enough to avoid ponding or passing on as overland flow.

In addition to the hydraulic behaviour of traditional drainage systems, their water quality management characteristics are poor and this problem is now recognised as a major issue in terms of polluting receiving waters. The quality of receiving waters and the types of main pollutants are covered in detail in the Regional Policy for Environmental Management.

The impact of rainfall in an urban environment is summarised below.

Foul Sewers – Inflow

Foul sewers, designed to be completely dedicated to wastewater, usually have a small proportion of impermeable surface (incorrectly) connected to them. If this is more than a small percentage of the total area, then the network becomes rapidly overloaded by even relatively small events, causing backing up and flooding either directly into houses or externally. Basements that are connected to the foul system are particularly susceptible to this form of flooding, and the social impact can be very high.

Normally, foul water is conveyed directly to WwTW after which it is discharged to a river or the sea. Flows passing to treatment works that are diluted by rainfall, result in reduced treatment efficiency at the works as well as discharging excess flows into storm tanks and, if these fill and spill, untreated effluent passes into the receiving waters.

Occasionally flood relief is provided to these sewers, due to the degree of impermeable area connected to them, by providing CSOs. The impact on the environment of spills to the river is significant and CSOs on separate foul systems should only be provided as an emergency measure.

Foul Sewers – Infiltration

Due to defects in the fabric of piped drainage systems, considerable volumes of groundwater often enter the foul system. This infiltration can be caused by a number of conditions. Infiltration can be due to temporary ground saturation due to recent rainfall, elevated groundwater levels caused by extended rainfall, or tidal influence in coastal low level systems. Due to their relatively small drainage capacity, it is possible for badly affected networks to become surcharged from relatively minor rainfall events.

For systems that are badly affected, infiltration can be more of a problem to treatment works than misconnected impermeable areas in that dilute flows will occur for extensive periods.

Combined Sewers

Water pollution and large discharges take place to receiving water bodies when combined sewers spill during wet weather. Pollution can be particularly acute during times of low river flow, particularly after prolonged dry periods when sediments, that have built-up in the pipe network, are scoured out in the first flush. For extreme rainfall, overflows of dilute sewage can be accommodated more easily in receiving waters, but they can be equally damaging due to the scouring effects of the very high discharge rates that can occur.

Stormwater Sewers

Stormwater sewers are designed to collect all run-off from paved areas and exclude foul sewage. When storm sewers are over-loaded, flooding can occur and this is particularly serious when internal flooding of properties takes place. The level of service provided by stormwater sewers is often much less than the initial design intended due to additional developments taking place either by in-filling existing urban areas or being extended upstream.

The polluting effects of stormwater runoff in streams or flooding in houses is not significantly different to flooding from foul sewers. The contaminated silts and other detritus from urban areas and the occasional illicit foul connection makes the impact of internal flooding equally unpleasant and damaging.

The high runoff rates which can occur, if unchecked, can cause erosion problems in receiving streams and also re-entrain polluted sediment from the riverbed. It is now recognised that surface water systems are a major cause of river pollution.

Open Channel Watercourses

While open channel watercourses, such as rivers and streams, normally have a greater hydraulic capacity than piped systems, the consequences of flooding are usually greater due to the scale of the event. This concern usually results in more conservative design criteria being used. The consequences of flooding from a culverted watercourse are usually far more dramatic than with river flooding. This is because the capacity of the river greatly increases as water levels rise, while the capacity of a culvert by comparison, once surcharged, only marginally increases with the increase in hydraulic head.

Culverting rivers also causes significant ecological loss, as well as producing negative aesthetic impact and other negative environmental effects. The water quality in open channel watercourses can be directly related to the catchment land use, either urban or rural. The base flows in watercourses in urban areas are reduced, peak flows during rainfall are higher and generally all measures of water quality show deterioration. This varies with land use type (residential, industrial and commercial areas), and depends on stormwater management techniques used. Spillages of toxic material in industrial estates can be particularly destructive.

6.2 Principles of Stormwater Design

The three principles behind the selection of design criteria are:

- Sustainability;
- level of service;
- cost-effectiveness.

Each of these three principles is expanded upon below. The drainage engineer should have a number of questions that are addressed by the proposed design. A non-exhaustive list includes:

- What are the normal operating and maintenance requirements of the design?
- What are the risks of failure of the proposed design and the consequences in terms of impact?
- What are the implications of failure for the rehabilitation of the system that will be needed?
- How effective will the system be in treating the stormwater?
- What are the social / aesthetic benefits of the proposed design?
- What are the environmental benefits / protection of the proposed design?

If consideration is given to all these questions it will generally ensure that a sustainable drainage system is designed.

Sustainability

Sustainability can be defined in a number of ways, but in terms of drainage it can be interpreted as:

- Drainage systems should utilise natural resources which can be reused and are energy efficient in terms of constituent products and construction process;
- Drainage systems should aim to replicate the natural characteristics of rainfall runoff for any site;
- The environmental impact of man should be minimised.

The concept of sustainability is now well accepted. This is resulting in a move away from traditional drainage methods, and the recommended use of SuDS systems to provide hydraulic, water quality and environmental benefits. In addition more attention is now being paid to the consumption of natural resources and the ability to recycle these materials. The issue of climate change is now of major importance and this draws attention to the energy aspects of construction. This includes not only the energy requirements to build the drainage system, but also the energy requirements for its maintenance and the energy needed to manufacture the components used in the system.

The design of the drainage system should try and replicate, in a general way, the same rainfall-runoff characteristics for the pre-development condition of the site. The runoff is much slower, less polluted and has virtually no runoff from ordinary rainfall events. The use of SuDS, particularly components which encourage infiltration, will enable this principle to be achieved.

The design of drainage systems needs to minimise water pollution and maximise environmental benefits. SuDS units are designed to address stormwater water quality as well as providing hydraulic conveyance. Consideration should also be given to what might happen if the drainage system “fails” as well as its performance during normal operation. Due to the nature of SuDS units, the consequences of failure tend to be less of a problem than failure of traditional drainage systems.

This is because failures of SuDS units tend to be incremental and not catastrophic as in the case of a pipe blockage or collapse.

Level of Service

- Flood protection should be provided to a minimum level of service;
- No negative aesthetic effects;
- Social benefits;
- Safety.

The principal objective of drainage is to provide protection from flooding due to rainfall on an area. The level of service provided is a function of society's expectations as well as the cost-benefit of the system based on the damage consequences due to flooding. Current design criteria normally require that no flooding occurs up to the 30 year return period, and properties are protected against flooding for the 100 year return period. The level of service for existing systems is usually a lower standard, with 5 years being considered as a minimum requirement.

Although aesthetics are rarely considered as an issue of level of service provision, considerable expenditure in the UK has been incurred in addressing aesthetic pollution from CSOs. As SuDS systems become more common, it is important to ensure that these are aesthetically acceptable as well as acting as efficient drainage systems.

Certain SuDS provide the opportunity for dual land use. Attenuation structures such as ponds have to have the ability to deal with events up to a 100 year return period. This requires large areas adjacent to these structures which are normally dry and can be used for other purposes.

Safety is not really a primary level of service issue, but it is clearly an essential aspiration in providing an appropriate design of any system.

Cost-effectiveness

- Principles of whole life costing (WLC) should be applied.

Drainage design should aim to provide the most cost-effective solution, particularly in terms of maintenance requirements. This requires consideration of whole-life costing of alternative options. Evaluation of the most appropriate system should include hydraulic, water quality and environmental benefits.

There is a limited, but growing data set of experience of the capital and operational costs of SuDS. In general, the cost of SuDS systems are believed to be comparable to traditional drainage systems. Long-term performance of SuDS units is still being investigated, particularly with regard to the extent of the maintenance needed.

“Failure” mechanisms (flooding and pollution) are more robust for SuDS than traditional systems. It should be recognised that any drainage system can fail, whether it is a traditional system or SuDS.

Attention to design detail is important to ensure easy and effective maintenance of all drainage systems.

6.3 Design Criteria

Drainage design criteria needs to consider the above principles in order to produce the most appropriate system for any location. Individual criterion can be developed to meet the various requirements of sustainability and levels of service.

Consideration of whole life costing does not result in specific criteria for design. Appropriate whole life costing requires appropriate weighting of maintenance against capital costs by applying a Net Present Value method. Sensitivity analysis should theoretically be carried out on various possible solutions to arrive at the most cost beneficial scheme rather than rigidly sticking to a specific design standard.

6.3.1 Sustainability

6.3.1.1 Energy and Use of Natural Resources

There are no design criteria that address the selection of appropriate drainage products and achieve the best design which meets energy and natural resource objectives. However certain features of drainage systems such as the use of pumping stations and large underground structures require considerable energy consumption in their construction and operation. There is less information available with regards to making the most sustainable choice when deciding between the use of one product over another. This is a complex area requiring a balance between costs, structural properties of drainage units, site specific aspects, maintenance and, in the long-term, the dismantling and disposal of the system.

Although there are no design criteria specifically addressing the minimisation of energy consumption and the use of natural resources, it is important for engineers to be aware that this is an issue which will become more important in the future.

6.3.1.2 Environmental Impact

Environmental impact of urban stormwater run-off is characterised by the high levels of sediment and other pollutants, both particulate and dissolved, together with the volume and rate of flow of the run-off causing flooding and erosion in the receiving water. Design criteria can be developed to address these various effects, but these are more easily considered by breaking down the various environmental impacts into their individual components and by comparing with the natural rainfall run-off processes which take place in the greenfield environment.

6.3.1.2.1 River Water Quality Protection

Run-off from natural greenfield areas (which are not farmed) contributes a nominal amount of pollutant and sediment in run-off to rivers. For most rainfall events, rainfall depths and intensities are relatively low and direct run-off to rivers does not take place with rainfall percolating into the ground. This water eventually supports the base flow in the river days and weeks after the event has taken place.

By contrast urban run-off, when drained by pipe systems, results in run-off from virtually every rainfall event with high levels of pollution, particularly in the first part of the run-off, with little of the rainfall actually percolating into the ground. This results in virtually no support for the base flows in rivers.

Table 6.1 summarises the differences in urban and greenfield run-off processes and provides an indication of the design criteria that need to be developed to enable urban run-off to more closely replicate the greenfield condition in protecting river water quality.

Greenfield response	Urban response
No direct runoff	Direct runoff
Baseflow support	Limited infiltration
No pollutants	Highly polluted

Table 6.1 The Contrast Between Urban and Greenfield Stormwater Response for Small Rainfall Events

Appropriate design criteria to address these differences are therefore:

- No run-off to pass directly to the river for rainfall depths of 5mm and up to 10mm if possible (interception);
- Use of infiltration drainage techniques;
- Use of stormwater treatment techniques.

In practice, there are a number of practical constraints in applying these criteria. 10mm of rainfall run-off from an urban area, especially with a high-density development, provides a considerable volume of runoff. Infiltration may be a problem for several reasons; the first being that the soil may be fairly impervious (clay), secondly groundwater levels may be high at certain times in the year and thirdly washoff from certain surfaces, particularly roads, often contains high levels of polluted sediment and, depending upon the maintenance regime, will usually result in blockage of infiltration units over a period of time.

The fact that it might be difficult to comply with these design criteria in all circumstances does not mean that these criteria are not valid. They should be applied wherever a reliable solution is possible. Where it is not possible to store and dispose of 10 mm of rainfall, it might be possible to intercept runoff from 5 mm, which will still provide considerable benefits. It should be noted that the issue of river pollution is particularly a problem in the summer when river flows are low and dilution is minimal. However this is the period in which infiltration units are most likely to be effective as the soil moisture deficit and evaporation rate is high.

Achieving zero runoff from the first 5mm or 10mm of rainfall is often not practicable, and therefore emphasis is also needed on achieving some treatment of the stormwater run-off. This ensures that any runoff discharged to the river is of significantly better quality than direct runoff from a pipe network.

The various advantages of the different SuDS units are described in some detail in chapter 4 of this document and also, more fully, in the Environmental Management policy document.

SuDS units that treat stormwater include filter trenches, swales, wetlands, retention ponds and detention basins. One of the most commonly used SuDS unit is the retention pond. The design of the wet pond in terms of depth shape and volume is covered in SuDS design manuals and also in the Environmental Management policy document. In terms of design criteria to provide treatment, the concept of the “treatment volume” (Vt) has been defined for the permanent pool volume of a retention pond. In the SuDS design manuals CIRIA C521 and C522 it is recommended that up to 4 times Vt is required to provide good treatment of the stormwater volume.

The formula for Vt is:

$$Vt\ (m^3/ha) = 9 \times D(SOIL/2 + (1 - SOIL/2) \times I)$$

Where:

I = fraction of the area which is impervious (30% impermeable area = 0.3)

D = M5-60 rainfall depth (5 Year Return, 60 minute duration)

SOIL = Soil Classification (Flood Studies or the Wallingford Procedure WRAP map)

Vt is thus a function of local hydrological characteristics, soil type and the level of impermeability of the catchment. The presumption is that the treatment volume (permanent pool) is sized to capture runoff from 90% of storms that occur each year. The value for the Dublin region would be an equivalent rainfall depth of around 20mm. Thus 4 times Vt would imply a storage volume of 80mm. This is a very large volume of water and recent research suggests that the advantages of large volumes of storage, particularly deep ponds, have limited benefits. It is becoming clear that long residence times can result in the production of high levels of ammonia, due to anearobic conditions in the sediments, which is very poisonous to river crustaceans and fish.

Shallow ponds, although providing less opportunity for these conditions, have a number of limitations. These are:

- For any given volume, the shallower the pond the larger is the area needed;
- Plants will grow in depths of up to around 1m. This depth of open water cannot be guaranteed for shallow ponds. This implies increased maintenance and also reduced aesthetic value of the pond.

It is therefore recommended that a figure of 15 mm of rainfall is used for the Dublin region to determine the permanent pool volume, until research provides clear evidence as to what constitutes best practice.

It should be noted that the Standard Percentage Runoff (SPR) indices for SOIL are different in the Flood Studies Report (FSR) from the values in the Winter Rainfall Acceptance Potential (WRAP) map (from the Wallingford Procedure). This formula is generally applied using the WRAP values, but the difference is not worth debating.

Flows from large storms should be diverted around treatment facilities, with only runoff from ordinary events being treated. However as retention ponds are used to provide both treatment and also hydraulic attenuation for extreme events, careful design is needed to prevent resuspension of sediments.

The design of pond water levels should take account of winter levels of groundwater. Lining a pond (if needed to protect the a sensitive groundwater area from pollution) where ground water levels are high in winter, should ensure that the design pond water level is higher than the groundwater or else the pond liner will “float” up. If the pond is not to be lined, the groundwater level in summer needs to be known to determine the likely minimum water level in dry summer periods. The range of the pond water level in the seasons should be taken into account in its design, particularly its impact on barrier planting vegetation. The fact that the water level may drop below the outlet control is not necessarily a problem as no direct runoff to the watercourse reflects the normal greenfield response in dry summer periods.

Although water quality and hydraulic design features are the principle focus when designing the drainage system, it is important to maximise the environmental benefits of any design. Thus appropriate use of vegetation borders to ponds using native plants which support local fauna is to be considered whenever designing a system. The gradient of the ground at the edge of the pond should be designed to be fairly flat even though this may not be the most efficient hydraulic solution and require some additional land.

Guidance on best practice design of retention ponds is available from SuDS design manuals. It is inappropriate to use environmental criteria as primary design criteria. However environmental benefits need to be considered when developing the design proposals.

6.3.1.2.2 River Regime Protection

Rural runoff to rivers, when it occurs, is slow. To try and replicate this, urban runoff must be heavily constrained. Unrestrained runoff causes high velocities and erosion, affecting the morphology of the channel and the flora and fauna in the river.

Relevant design criterion to address this issue is to:

- Restrain the rate of discharge to the receiving water to that of greenfield runoff for the site.

A range of formulae exist for predicting greenfield runoff. The simplest and the one considered most appropriate for applying to this criterion was developed by the Institute of Hydrology in their report 124 “Flood estimation for small catchments”, 1994.

The work was based on 71 small rural catchments. A regression equation was produced to calculate $QBAR_{rural}$ the mean annual flood.

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

where:

$QBAR_{rural}$ is the mean annual flood flow from a rural catchment in m^3/s.

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 that accompany the Flood Studies Report.

QBAR can be factored using the Flood Studies Report regional growth curve for Ireland to produce peak flood flows for a number of return periods. Information on growth curves for UK and Ireland is available in Flood Studies Supplementary Report (FSSR) 14, 1987 produced by the Institute of Hydrology.

There is some indication that the Irish growth curve is not applicable for some Dublin rivers. Preliminary work carried out on the Carrickmines, Shanganagh and Tolka rivers has resulted in an alternative growth curve being proposed. The FSSR 14 growth curves together with the proposed regional curve for Dublin are shown in appendix C. These will be updated in due course when more research is made into this issue.

The formula for determining the peak greenfield runoff rate should not be applied to areas less than 50 hectares. As many developments are smaller than this size this constraint is avoided by calculating QBAR for 50 hectares and linearly interpolating flow rates for smaller areas.

6.3.1.3 River Flooding Protection

River flooding has serious consequences for affected properties and therefore return periods of 100 years are usually applied to determine the extent of floodplains and the risk to properties adjacent to watercourses. A return period of 200 years is normally recommended where flooding risk from the sea is possible. Flooding in rivers is exacerbated by urban runoff, particularly in catchments with a high degree of urbanisation. The floodplains provide a finite volume of storage, so not only is the rate of runoff from urban areas needing to be controlled, but also limiting the increase in volume of runoff compared to greenfield conditions should also be considered.

Relevant design criteria to address river flooding are to:

- Restrain the excess volume of runoff from urban developments to that of greenfield runoff;
- Avoid development on the floodplain.

6.3.1.4 Excess Urban Runoff Volume

It is important to realise that many river flood events are the result of multiple rainfall events and therefore it is unwise to try and design for the discharge to take place before or after the flood wave passing down the river. If all catchments are developed on the basis of reflecting the rural behaviour prior to development, both in terms of rate of runoff and volume of runoff, it is likely that the river will be protected effectively.

This additional volume of stormwater runoff is not a flooding issue in “normal” (frequent) rainfall events as long as runoff rates from sites are constrained. However in extreme events, where flooding is likely to occur in the river, it is important to limit this runoff volume. This can be achieved in the design of the drainage system by spilling from the attenuation storage system to an area which will drain very slowly, preferably by infiltration. As this is a rare event, by definition, this might be to a park or football field with appropriate land drainage provision at low points. This storage might be termed “long term” storage for river flood protection.

The river floodplain should generally be used as open space for ecological reasons as well as being a river flood corridor for extreme events. Planned development or even storage in the floodplain should generally be avoided. This is partly due to the fact that the storage attenuation system is bypassed by being flooded, and also creates a problem in terms of maintenance (depending on how frequently it is flooded). The likely consequence is that large volumes of sediment will be deposited in the storage systems by the floodwater when this occurs.

To achieve the necessary volumes of long term stored runoff, the return period at which runoff will start to pass to such an area will need to take place for events less than the 100 year event. However if flooding of the area occurs more often than say once in 10 years then the level of service for that public open space may be considered to be inadequate.

In some situations it might not be possible to achieve this approach. Also it requires detailed technical analysis to enable this to be designed accurately. The alternative is to provide for this volume in the form of infiltration which comes into effect for all storm events. This not only has the advantage of simplicity of design, but also provides good environmental benefits in terms of base flow support for rivers, and reduced runoff for small events (which replicates greenfield runoff). It should be noted that infiltration in extreme wet periods will be less effective than at other times, so infiltration storage should only be used where groundwater levels are known not to rise to the levels of the proposed infiltration units. Although detailed calculations can be carried out to establish the infiltration volumes needed by taking account of infiltration rates of the soil at the site, the soil moisture state during particularly wet periods will tend to be saturated and antecedent conditions may reduce the available storage volume. It is therefore suggested that the volume of storage normally provided as infiltration to meet this criterion is equal to the calculated value of the additional runoff volume. Detailed analysis, if carried out, can reduce this volume by taking into account the infiltration that will take place during the critical duration event.

It is possible that “long term” storage cannot be provided at certain sites. In these situations it is recommended that QBAR is used as the attenuation storage control requirement to ensure sufficient runoff is retained on site for extreme events. This will tend to be a less economic solution, but is the only way to ensure that urban runoff does not exacerbate flooding in a river. Where QBAR is a value which is less than 2 l/s/ha it is recommended that this figure is used to prevent excessive cost. Studies by HR Wallingford “Storage requirements for rainfall runoff from greenfield development sites” SR580 / SR591, 2002 showed that attenuation throttle rates needed to be less than 3 l/s/ha to be effective in limiting discharges to rivers during flooding.

In summary protection against river flooding by the provision of “long term” storage can be catered for in 3 ways:

1. Temporary flood storage spilling excess stormwater runoff to an infiltration area – probably public open space;

2. Provision of infiltration for excess stormwater runoff to come into effect for most or all events;

3. Attenuation storage designed with a limiting discharge throttle rate of QBAR for all extreme events (up to 100 years).

Assessment of the “long term” storage volume is detailed in section 6.7

Appendix E provides a worked example for illustration.

6.3.1.5 Development in the Flood Plain

Development in the floodplain creates a number of problems which is why developments within the floodplain are normally viewed as being unacceptable. This section therefore presupposes that all alternatives to development outside of the floodplain have been considered and rejected and that the local authority has allowed the development to be considered.

The risk of flooding the new development must be addressed and this is commonly avoided by raising ground levels. This creates potential problems both upstream and downstream by reducing river flood storage and raising water levels for a short distance upstream. For each individual development, this impact is virtually imperceptible, but development in this manner across the catchment will significantly modify the flow regime of the river.

To address this, where development is allowed to take place, it is important to provide compensation storage. Thus ground levels need to be modified such that the depth-area (and conveyance) relationship at any point on the river is the same before and after development.

Modification of the channel or development close to the main channel should be avoided. The morphology of any channel is a complex balance of erosion and deposition and changes will normally result in destabilising the channel. As development takes place upstream, even where stormwater controls are rigorously put in place, changes will occur in the channel which take into account the change in the river hydrological characteristics.

Another reason for avoiding development close to the river is that it provides a natural feature for both its social use as green space and a corridor for wildlife.

A third, and very important reason, in terms of drainage design, is that attenuation storage is normally needed to serve any development. This requires ponds or tanks to be provided, which, by definition, must be located at the lowest point in the site. This means that these structures often cannot avoid being built in the flood plain. This not only leads to difficulties in terms of operation (inflow, outflow and water level design), but also creates an increased risk of sedimentation problems if the river inundates the units. If this occurs early in the flood event, the drainage control of runoff from the site will have “failed” for that event, as unrestrained discharge will take place after the river floods the system. The lower the relative level of the drainage control system with respect to the top water level of the river in flood, the more difficult it will be to design the attenuation storage for the drainage system serving the development.

6.3.1.6 Extensive Catchment Development

Where development of the catchment is likely to be significant, application of these stormwater management principles will still result in some change in the normal behaviour of the river. In some catchments, particularly where compliance may be difficult to achieve for what ever reason, the local authority might chose to carry out a catchment study to check on the change in performance of the river to enable strategic decisions on drainage strategy to be made.

It should be recognised though that the effect of urban development across the catchment, particularly when SuDS is being used, is not well defined in terms of modelling representation. In addition, due to the need to consider the Water Framework Directive, the performance in the river should really be considered for both its “normal” state as well for extreme conditions.

Measuring the change in performance in the river on a site by site basis results in minimal change in the river state, especially if stormwater controls are being used. The exception to this rule is when the floodplain is being significantly modified by the development proposal, in which case a detailed model of the development proposals is appropriate. This should extend as far up and downstream as is necessary and should be compared to a model of the status quo.

6.3.2 Level of Service

There are four elements to consider for provision of adequate levels of service.

- Flood protection;
- Aesthetic effects;
- Safety;
- Climate change.

6.3.2.1 Flood Protection

Three criteria need to be applied to ensure against flooding. These are:

- Protection against river flooding;
- Protection against flooding from storage systems;
- Protection against flooding from overland flows.

It is recommended that the 100 year return period is applied to all these criteria for protection of flooding within properties. In addition a minimum level of flood nuisance to the community requires the selection of the 30 year return period, or similar, for the occurrence of any significant unplanned flooding anywhere on site. Figure 6.1 illustrates the level of service for the various components of the drainage system.

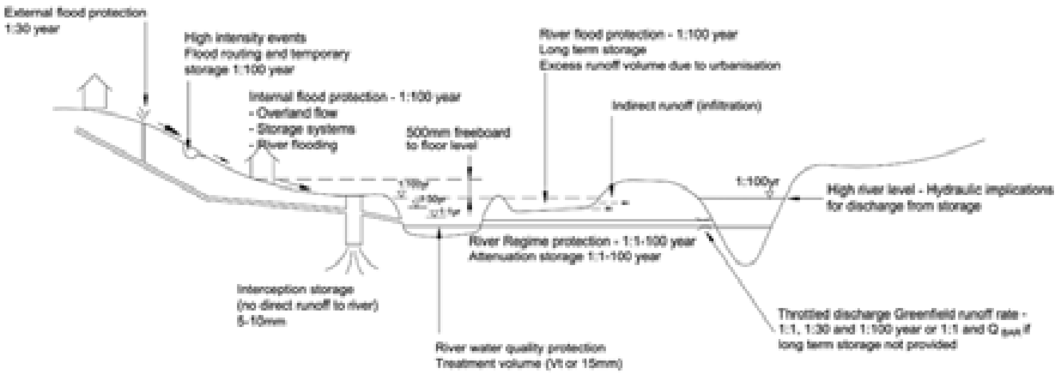

Figure 6.1 Level of Service and Flood Protection Principles

<u>*Protection Against River Flooding*</u>

The effect of river flooding is often severe and there is usually some degree of uncertainty with regard to the maximum flood level at any location for a particular return period. It is recommended that floor levels of all houses are at least 500mm above the predicted maximum 100 year flood level. This freeboard should be increased where there is a significant level of uncertainty and where predicted water levels are sensitive to the assumptions and analysis parameters being used. Flood maps are being produced for rivers in Ireland, but these may not exist for all locations or may be very approximate in their estimation. It is therefore important to investigate the likely accuracy of this information or assess it specifically as part of the planning and design process.

In addition to floor levels of dwellings, other aspects such as access and the location of sensitive and important buildings (hospitals) should also be designed taking into account flood risk.

In considering the maximum water level, appropriate precautions must be taken in assuming the scenarios which might affect the level of flooding. These include:

- Current and proposed urban development upstream in the catchment;
- Throttles and other attenuation features upstream due to bridges and dams need to be specifically considered. These may be removed over the course of time or may fail suddenly in flooding conditions;
- Floodplain storage upstream might be reduced.

A position needs to be taken on all these issues in determining maximum river water levels at any site. These should be defined by the local authority for the area based upon the local area structure plan or agreed with them in the early stages of considering a planning application. The general position that should be taken is that man-made obstacles are likely to alter in time, but that natural watercourse characteristics will be preserved by all future development. The level of future development upstream and the runoff characteristics will depend on the local authority's views regarding future development and the level of enforcement of SuDS techniques in that catchment with its particular soil and topographic characteristics. This should not be limited to the structure plan horizon which is often 20 years, though a longer term view of future development in the catchment may be difficult to arrive at.

Occasionally river and sea defences result in embankments. The location of houses behind these defences is a risk, which is a function of the water levels being restrained, the quality of the defence structure and the distance of the dwellings from the structure. There is limited guidance available for this situation, but it is important to carry out a risk analysis where this circumstance arises.

Storage Pond Flooding

Storage pond water levels are designed specifically, and therefore there is less uncertainty than for river flood water levels. However property floor levels must be provided with a safety freeboard and it is recommended that this is 500mm.

There are a number of less obvious aspects to consider related to storage ponds. These are:

- Hydraulic constraints to the pond outlet;
- Overflow provision and risk of failure;
- Hydraulic backwater effects at the pond inlet.

High water levels downstream of the storage unit may affect the top water level in the pond. This is a complex issue of joint probability (the river being high when the storage unit is full) and the relative levels of the water surfaces. A precautionary approach to the analysis should be assumed (possibly total dependency) to establish maximum storage water levels.

Similarly, flows into a storage unit which is full, may have a backwater issue for the inlet pipework serving the development which might result in local flooding upstream. This is only relevant for quite flat catchments, but generally this should not be too great a problem as the rainfall intensities in events where the storage is fairly full are generally not those which cause a pipe capacity problem.

The failure of a storage unit, particularly if it is embanked, can be dramatic, even if it is a relatively small reservoir. Reservoir design standards may be appropriate to consider in certain circumstances. Failure of the structure is not the only thing to consider. Very extreme events, much larger than 100 years, can occur. The design of overflow structures should be for a 200 year event and still providing a freeboard of at least 200mm.

Flooding from Overland Flow

Unlike the last two categories, which generally relate to long heavy rainfall periods, consideration needs to be given to short very high intensity thunderstorm type events. These events, often lasting for only 20 or 30 minutes, involve so much rainfall in this short period that the drainage system cannot cope with the runoff. In this situation water runs off down roads and overland through properties unless it is specifically taken into account. The impact of such events will generally be much less for SuDS based systems which tend to be based on provision of volume (swales, infiltration units etc).

Analysis of these situations requires careful examination of the topography of the proposed development and the layout of the roads system. A model is best used in this situation, particularly where piped networks are involved, as flooding can occur at a low point due to the drainage system and its hydraulic characteristics as much as due to flooding down roads due to gully incapacity.

Sites should take into account topography to maximise the benefits of low points for storage and avoid placing vulnerable structures and/or properties in these areas.

Basements are particularly at risk in these situations and they should be protected by the ground being suitably profiled to prevent entry of overland flows.

An issue that should not be overlooked is that of responsibility for flood flows and the rehabilitation related to it.

In general it is advised that the drainage system should be designed to cater for the 30 year event without causing any significant unplanned flooding, but that this should always be open to variation depending on the type of development being served and the drainage system proposed.

6.3.2.2 Aesthetics

It may not be immediately obvious why aesthetics is a consideration of design. Although it does not lead to a primary design criterion, the use of SuDS will require specific consideration of their visual impact. It will also draw attention to the maintenance requirements and thereby the costs for operating the units. A negative view of SuDS units will create problems in trying to get them generally accepted and used.

6.3.2.3 Safety

As with aesthetics, safety does not result in primary design criteria relating to the size of any unit. However the potential for accidents and the measures needed to limit such incidences requires drainage engineers to specifically consider this issue.

Underground storage volume facilities, if used, must be designed for safe access for maintenance. This will influence the minimum height within the structure, benching slopes, venting and other features. Drawdown facilities should be provided for all water retaining structures.

6.3.2.4 Climate Change

Climate change is acknowledged as taking place the world over. The GDSDS Climate Change policy document advises that rainfall event depths should be factored by 10% and that sea levels will rise by 400mm or more over the coming century. There is no specific advice for river flow rates, but the Defra advice in UK suggests a 20% increase in flood flows. The climate change policy also provides advice on the use of Time Series Rainfall.

If these criteria were not applied, and these predictions were found to be correct, then the level of service provided by the drainage system would be less than it was designed to achieve. It is therefore advised that climate change criteria are applied for the design of drainage systems for new developments.

Climate Change Category	Characteristics
River flows	20% increase in flows for all return periods up to 100 years
Sea level	400+mm rise (see Climate Change policy document for sea levels as a function of return period)
Rainfall	10% increase in depth (factor all intensities by 1.1)
	Modify time series rainfall in accordance with the GDSDS climate change policy document

Table 6.2 Climate Change Factors to be Applied to Drainage Design

As a precaution it is advised that the same uplift is not applied to the calculated flow rates from greenfield runoff. This provides a safety factor to the methodology. It can also be argued that the level of accuracy of the greenfield runoff formula and prediction of river behaviour warrants the addition of a safety factor anyway.

6.3.3 Other Design Issues

There are a few other issues which influence the generic design criteria discussed above. These are:

- Size of development;
- Environmental issues influencing storage design;
- Density of development;
- Location of development;
- Extending urban catchments.

6.3.3.1 Size of Development

To limit discharge to greenfield runoff rates usually requires a pipe or other form of throttle. These throttle sizes theoretically need to be quite small to achieve the required maximum rate of flow, especially for small developments. For operational purposes, it is recommended that the minimum throttle size for a pipe should be 150mm minimum diameter and any other orifice unit other than a pipe should be a minimum of 200mm diameter. This means that flows much below 10 l/s are rarely achievable. Thus small sites, by default, are often allowed a more generous discharge limit than larger developments. This can be partially re-dressed in three ways.

The first is to ensure the development area is planned on a catchment basis so that any development fits within a drainage strategy for a catchment.

Secondly building storage tanks and ponds in series can help in minimising peak flow rates.

Thirdly certain SuDS systems can result in significantly greater attenuation than just using a tank and orifice arrangement. Thus small sites should place particular emphasis on the use of unlined pervious pavements and infiltration units. Where the permeability of a soil is low and the use of infiltration is marginal, it should still be used, but systems should be designed with overflows to ensure against a level of service failure.

6.3.3.2 Environment Issues Influencing Storage Design

The previous section suggested that ponds should be built in series. There are strong environmental and other benefits for doing this. Although land take may be marginally greater, the following advantages are provided:

- maintenance is generally easier;
- desilting of the upstream units has minimal impact on the receiving water;
- greater flexibility in locating ponds;
- ecology gains with a range of different quality and physical characteristics of ponds.

It is therefore advisable to have a train of at least three ponds; the first providing a focus on sedimentation, the second on hydraulic attenuation and the third as a polishing pond, often a small wetland.

6.3.3.3 Density of Development

The drainage of any development, whatever size or location, should consider the opportunity to use appropriate SuDS techniques. However situations will exist where there will be limited opportunity to use SuDS or infiltration methods. Very high-density developments, usually planned for areas adjacent to primary traffic corridors of a city, may have very limited opportunity to use SuDS techniques. Specific consideration of using SuDS units should always be carried out, (and there are few circumstances where pervious pavements cannot be used), but it is possible that the use of a traditional pipe drainage system, with storage tanks (concrete or high cellular voids systems), may be the most appropriate drainage method to use.

6.3.3.4 Location of Development

There is a situation which is not really applicable to the generic approach described earlier. Developments that are proposed at the downstream end of a catchment, by definition, do not have to be concerned with worsening the river state downstream. In this situation, it may not be necessary to provide either “long term” storage or attenuation storage. Similarly issues such as river erosion might also not be applicable. Water quality may therefore be the only principle that needs to be considered in terms of the receiving water.

Applying the same principles detailed in section 6.2, it is there recommended that:

- Where there is little downstream to be concerned about with respect to flooding (discharging to the estuary or sea), criteria on flow rates and volumes of discharge are of little relevance. Water quality is the only issue needing to be addressed (primarily sedimentation);

- Where a river's morphological characteristics are important, but there are no developments downstream, water quality criteria should be applied, together with some flow rate control. However runoff for extreme events is of little concern. Therefore criteria would focus on both water quality elements and discharges from the site up to, say, the 10-year event.

In all cases levels of service for the development still apply.

6.3.3.5 Greenfield Developments and Infill Developments

New developments can take place in greenfield or brownfield locations. In theory design criteria need not be any different between these two situations. However, in practice, the precedent of existing high runoff rates from a previously developed site and the political and environmental value of re-using urban areas, often results in more liberal criteria being applied to these sites.

The contrary argument to this is that in locations where the urban drainage systems are particularly taxed (as would be demonstrated by frequent flooding or high spill frequencies from CSO's on combined systems), then onerous criteria will need to be applied to prevent existing levels of service reducing further. The choice of appropriate design criteria is a matter for the local authority to consider in the light of the current situation and flood risk downstream.

6.3.3.6 Extending Urban Areas

In some instances, particularly infill development on drainage systems, there may be downstream flooding already on the surface water system to which the development is to be connected. In these circumstances an alternative discharge location although desirable, may not be available. On the basis of the principle that there should be no detriment to the existing level of service to those downstream, it is likely that runoff constraints will need to be very strict with emphasis on the use of infiltration where possible. In addition there may be a need to provide flood alleviation solutions to locations downstream to minimise reductions in levels of service.

Assuming all options of runoff reduction have been considered and used, the attenuation discharge limit needs to be as onerous as reasonably can be applied to minimise the downstream impact, if the level of service downstream is less than a return period of 30 years. In this situation, subject to minimum throttle size constraints, 2l/s/ha should be considered as the throttle criterion. As the urban flooding criterion is 30 years, this would be applicable for determining the attenuation storage requirement, subject to meeting the requirements for the site and downstream flood protection for extreme events as discussed earlier. Extreme events must be addressed to prevent flooding of adjacent urban areas.

6.3.4 Summary of Design Criteria

Table 6.3 summarises the design criteria for the design of drainage systems. In principle these criteria should be applied to all sites, but certain practical limitations (throttle sizes for achieving low flow rates) and minimal consequences of non-compliance (draining to the estuary or coast) mean that an intelligent approach should be taken in applying these criteria. These criteria are explained further in Appendix E.

Climate change needs to be applied to all relevant elements of the design parameters used.

Figure 6.1 shown earlier schematically summarises all the criteria for drainage design.

Criteria	Sub-criterion	Return Period (Years)	Design Objective
Criterion 1 River water quality protection	1.1	<1	Interception storage of at least 5mm, and preferably 10mm, of rainfall where runoff to the receiving water can be prevented.
	1.2	<1	Where initial runoff from at least 5mm of rainfall cannot be intercepted, treatment of runoff (treatment volume) is required. Retention pond (if used) to have minimum pool volume equivalent to 15mm rainfall.
Criterion 2 River regime protection	2.1	1	Discharge rate equal to 1 year greenfield site peak runoff rate or 2l/s/ha, whichever is the greater. Site critical duration storm to be used to assess attenuation storage volume.
	2.2	100	Discharge rate equal to 1 in 100 year greenfield site peak runoff rate. Site critical duration storm to be used to assess attenuation storage volume.
Criterion 3 Level of service (flooding) for the site	3.1	30	No flooding on site except where specifically planned flooding is approved. Summer design storm of 15 or 30 minutes are normally critical.
	3.2	100	No internal property flooding. Planned flood routing and temporary flood storage accommodated on site for short high intensity storms. Site critical duration events.
	3.3	100	No internal property flooding. Floor levels at least 500mm above maximum river level and adjacent on-site storage retention.
	3.4	100	No flooding of adjacent urban areas. Overland flooding managed within the development.

Criteria	Sub-criterion	Return Period (Years)	Design Objective
Criterion 4 River flood protection (criterion 4.1, or 4.2 or 4.3 to be applied)	4.1	100	"Long-term" floodwater accommodated on site for development runoff volume which is in excess of the greenfield runoff volume. Temporary flood storage drained by infiltration on a designated flooding area brought into operation by extreme events only. 100 year, 6 hour duration storm to be used for assessment of the additional volume of runoff.
	4.2	100	Infiltration storage provided equal in volume to "long term" storage. Usually designed to operate for all events. 100year, 6 hour duration storm to be used for assessment of the additional volume of runoff.
	4.3	100	Maximum discharge rate of QBAR or 2 l/s/ha, whichever is the greater, for all attenuation storage where separate "long term" storage cannot be provided.

Table 6.3 Criteria for New Development Drainage

This process should be an integral part of design.

6.4 Hydraulic Design of Drainage Components - General

The design of a storm sewer network and determining its performance requires the use of network modelling tools, rainfall information based on the Flood Studies Report (FSR) and detailed network and ground level information. As climate change is now accepted as taking place, a precautionary position has been taken to cater for its effects. Details of these allowances are contained in the Regional Policy on Climate Change.

The design of a stormwater drainage system is expected to involve the use of SuDS. However in nearly all situations, pipes will also be involved to provide much of the conveyance of the runoff. The attenuation aspects of SuDS, together with the perception of possible premature failure of SuDS, need to be taken into consideration in the design of the supporting pipe system. Risk of sewer system failure can be due to:

- Structural failure;
- Pipe sedimentation / blockage;
- Inadequate capacity.

Design of sewers must therefore consider design for:

- Construction details;
- Velocity (to avoid sedimentation);
- Hydraulic capacity;
- Predicted performance using a simulation model.

Issues relating to pipe materials and the structural and construction requirements of drainage systems are addressed in Chapter 7. The following sections address the hydraulic issues of pipe design.

Design aspects relating to both SuDS design and quantifying site discharge constraints are also provided.

6.4.1 General Issues Related to Drainage Design

6.4.1.1 Site Constraints

Pipework routing and levels are often determined by site constraints, including topography (and the need to maintain cover to pipes whilst not entailing excessive excavation), other services and contaminated ground. All site constraints should be identified as soon as possible in the design process including areas of site not available due to temporary works.

By definition storage tends to be located towards the lower parts of the site. Flooding and overland flood routing can occur at any point of the site, but will then migrate towards low points. Due consideration of the space requirements for such issues needs to be given in the initial stages of the site layout and design.

6.4.1.2 Services Conflict

Foul and stormwater drainage are just two of the many services laid in the roads and footpaths in a new development. Care should be taken to ensure that services do not conflict. Most of the other services are placed above the sewers and therefore there is generally little risk of conflict.

Problems of level between the two sewer systems are also normally avoided as stormwater pipes are generally laid at flatter gradients than foul sewers. In general foul systems are therefore deeper than stormwater networks. Where steep catchments dictate pipe gradients, it is again preferable to put foul sewers below storm pipes; this both minimises the risk of foul pollution in the stormwater system and minimises trench depths for the larger pipes, making it cheaper to construct.

In general it is considered good practice not to have the foul and surface sewer in the same trench above each other. This causes obvious problems at manholes. The use of a single wide trench with the surface sewer off-set, benched at a different level, tends to cause more problems to build than digging a separate trench.

6.4.1.3 Temporary Drainage

The need for temporary site drainage during the construction phase should be considered. This can take the form of temporary ditches or the use of the designed storage system as long as re-instatement of these units is carried out at the end of the contract.

Settlement tanks or ponds will be required to prevent pollution of watercourses and siltation of existing drainage systems. Poor control of silt during construction can cause premature failure of infiltration systems. Consideration should be given to the use of a temporary drainage system until the development is complete and vegetation established.

6.4.1.4 Safety and Design

Design has a direct bearing on the Health and Safety risk for the construction, operation and dismantling of any structure. A significant category of Health and Safety concern is the collapse of trenches. However there are a whole range of possible hazards and thought should be given to considering what hazards exist. Once a hazard has been established and the level of risk estimated, efforts should be made to minimise or remove the hazard by posing appropriate questions.

- Can the design be changed to avoid the risk?
- Can the design be modified to reduce the risk – combat at source?
- What controls can be applied to reduce the risk to an acceptable level (minimal or low)?

This process should be an integral part of design.

6.4.1.5 Drainage Separators

Although rarely an issue for new residential development, separators will normally be required for certain commercial and industrial sites to address runoff polluted with light oils, heavy oils or grease.

Currently BS 8301 Building Drainage:1985 provides guidance on the use of these units and this will be replaced by the European Standards when they are issued. The Environment Agency UK also provides guidance. They have produced a series of documents entitled Pollution Prevention Guidelines. PPG3 is “The use and design of oil separators in surface water drainage systems”. This document itemises sites that normally do and do not require oil separators, gives general design criteria and a method for calculating separator size based on 6 minutes retention and catchment area.

6.5 Stormwater Pipe Design

6.5.1 Pipe Sizing for Standard Stormwater Networks

Design of surface water pipes, particularly small systems of up to 450mm diameter pipes, is often carried out using the Rational Method or the Modified Rational Method. They are normally subsequently analysed using a hydrograph method to check for flooding performance. Alternatively approximate pipe sizes can be quickly determined for small sites by using a rule of thumb approach of assuming a constant rainfall intensity of 50mm/hr.

Whatever approach is used to size pipes, this should only be done to provide an initial assessment of the network, and more detailed analysis should be carried out to justify/modify the pipe sizes and gradients to ensure an adequate level of service. This normally requires simulation modelling to enable an assessment of the flood risk for extreme events. Table 6.4 summarises the criteria which apply to the Dublin region.

Parameter	Surface Water Sewers
Minimum depth	1.2m cover under highways 0.9m elsewhere
Maximum depth	Normally 5m
Minimum sewer size	225mm
Runoff factors for pipe sizing	100% paved and roof surfaces 0% off pervious surfaces
Rainfall for initial pipe sizing	50mm/hr rainfall intensity
Minimum velocity (pipe full)	1.0m/s
Flooding	Checks made for adequate protection * No flooding for return period less than 30 years except where explicitly planned Simulation modelling is required for sites greater than 24ha**
Roughness – ks	0.6mm

Table 6.4 Surface Water Design Criteria

** It should be noted that a check for adequate protection against flooding cannot be made without simulation. Thus in practice nearly all systems are modelled to demonstrate that their performance is adequate for protection against flooding.*

*** The runoff model normally used for simulation is the New UK PR model, but the fixed runoff model can also be used.*

6.5.1.1 Private Sewers (those not vested)

Sewers serving individual properties can be 100mm. However pipe sizes and gradients are based on pipe capacity and velocity dictated by the criteria in Table 6.4.

Minimum cover to pipes can be reduced to 900mm when under lightly trafficked areas such as driveways, though this is very much a function of pipe material and vehicle loadings. In other circumstances cover should not be less than 600mm unless suitably protected.

6.5.1.2 Minimum Pipe Size and Gradient

The concept of pipe full design criterion is largely redundant in practice, as flooding is usually the controlling criteria. The use of pipe full criterion helps guide the designer in achieving pipe sizes which are likely ensure this condition. Although simulation modelling is not required for sites less than 24ha, flooding can only be predicted using computer simulation. The development of very small sites may not warrant the expense of assessing flooding.

6.5.1.3 Roughness

Guidance on the roughness of sewers for various materials is based on work carried out at HR Wallingford (1982). Guidance on pipe roughness is more fully advised in “Tables for Hydraulic design of pipes, sewers and channels” – volume 1, 6th edition from HR Wallingford.

6.5.1.4 Roughness and Velocity

CIRIA report 141 “Design of sewers to control sediment problems” has been produced to provide an alternative design approach based upon the solids carrying capacity of flows in sewers. This is applicable to both foul and surface water sewer design. It allows sewer gradients to be designed specifically for sediment loads for any pipe size. It is important to note that self-cleansing velocities increase with increasing pipe size. Very large sewers require high self-cleansing velocities (in excess of 3 m/s). Large sewers (those in excess of 1m diameter) should therefore be designed to allow a small amount of sediment deposition and should be specifically analysed using CIRIA report 141.

6.5.1.5 Runoff

Although the Wallingford Procedure uses a different runoff model to the fixed rates given in Table 6.4, in practice this is not a particular issue as the runoff volumes tend to be fairly similar for normal urban environments. Figure 6.2 illustrates the differences between the two models and shows that, for fairly high development densities, the assumptions in Table 6.4 of 100% and 0% runoff for paved and pervious areas respectively are conservative in predicting volumes of runoff.

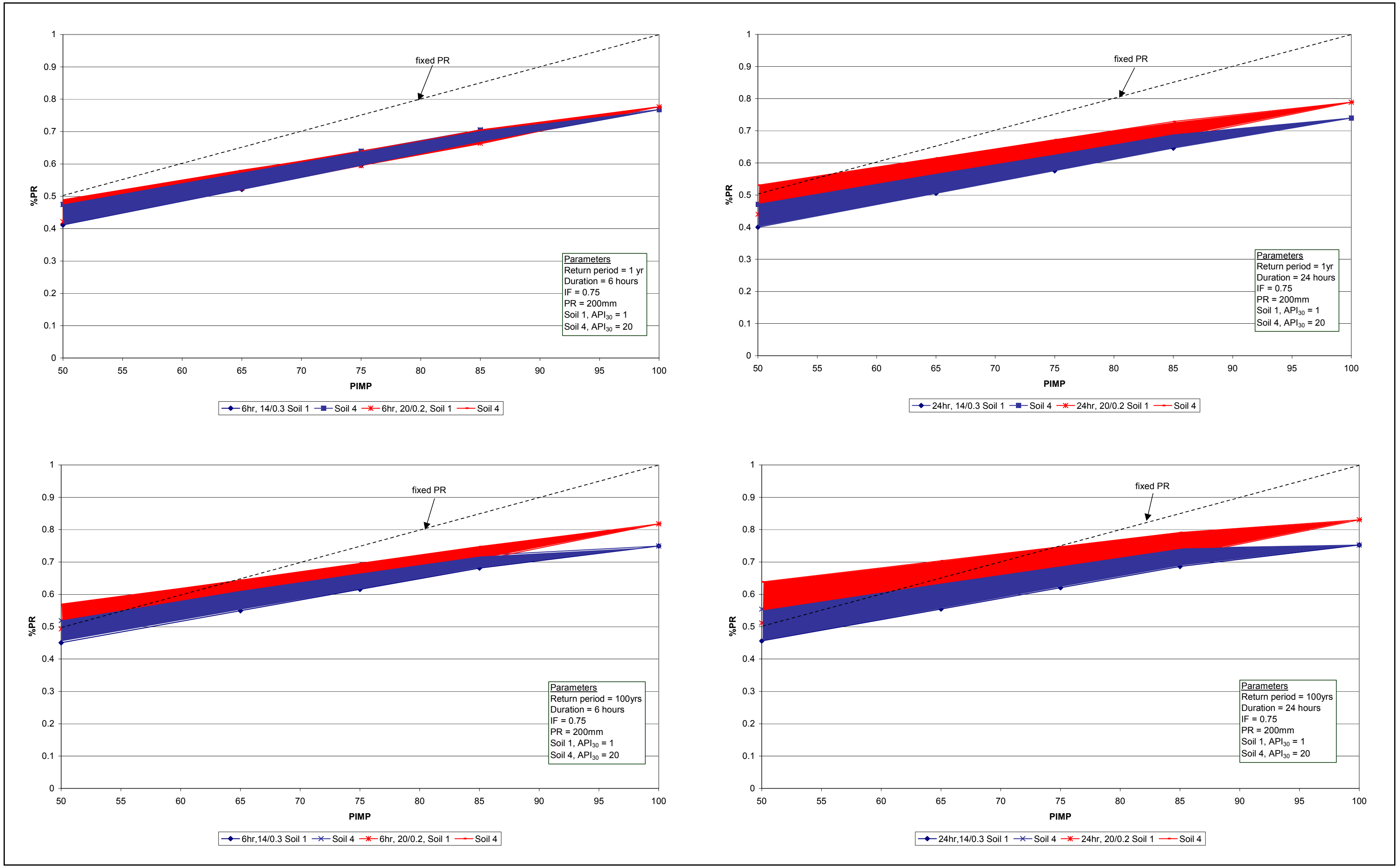

Figure 6.2 Comparison of PR between the Variable Wallingford Procedure Runoff Model and the Use of 100% and 0% Used for Initial Pipe Sizing

6.5.2 Pipe Sizing for Stormwater Systems Incorporating SuDS

There is concern in certain quarters that SuDS do not have an established track record in terms of performance. In addition as they are often landscape features, the opportunity to modify them, and hence adversely affect their operation, is much greater than comparable alteration of a traditional pipe system. For example driveways in private properties might be designed to be unlined pervious pavements, and these might subsequently be sealed.

Discharge rates and volumes from these surfaces could therefore dramatically change. Where pipe systems are used to pick up and convey these flows, the subsequent level of service provided will be much reduced.

For these reasons it is proposed that pipes should be sized assuming the SuDS units continue to work correctly, but to assess the risk of change and add appropriate safety factors accordingly. The risk assessment should also evaluate the consequences of these changes occurring.

Runoff from some SuDS units such as swales and pervious pavements will normally be significantly attenuated and reduced in volume. This means that flow depths and velocities will be low for pipes laid at traditional gradients. Fortunately these discharges will normally have very little sediment and therefore self-cleansing flows should not be an issue. In this situation a velocity of only 0.3m/s is likely to be sufficient. However any pipework which receives runoff, where the sediment loading is not addressed effectively by the SuDS unit, should apply traditional velocity criteria.

6.5.3 Discharge to Watercourses from Attenuation Storage in Floodplains

As discussed in 6.3.1.5, storage systems that cannot avoid being built in the floodplain have to take account of the relative water levels in the storage system and the receiving water. It is important to design the outlet characteristics to meet the discharge constraints which are defined by the drainage criteria. Simple assumptions regarding water levels in this situation can rarely be made. For instance if one assumes the river level is normally low then if it is actually high when discharge is taking place from a tank, (which is highly likely in very wet periods) then run-off from a tank storage structure will be less than the design flow rate. This means that the tank will completely fill for a smaller rainfall event than designed and will have insufficient capacity for the design storm. However if the opposite is assumed (that the river level is high for the operation of the tank), then discharge from the tank when the river is low will be too high. The closer the difference in the water levels between the storage unit and the river becomes, the more difficult this problem is to solve.

There are two ways in which this problem can be addressed. The first is to assume a low water level in the river and design the storage system appropriately so that the maximum discharge from the tank will never be exceeded. Then evaluate the performance of the tank assuming the river level is at its maximum height. This will show the additional volume needed to meet this extreme assumption. Where this additional volume is not particularly great, then the design can be considered to be appropriate. In fact it has the significant advantage in that the worst-case scenarios are all catered for.

The second approach is to make some assumption with regards to the dependency between the river water level and the water level in the storage system. This method requires the existence of models for both the site drainage system and the river. These water levels are varying continuously and are relatively different for different rainfall events, due to the catchment response characteristics compared to the development site run-off characteristics. In practice the important event is the design extreme event which requires the largest volume of storage on site. The hundred year return period is the only event needing to be considered. As the critical duration event for the storage system on the site is likely to be in the region of 24 hours, this is likely to be fairly similar to the critical duration of the river unless it is a major national river. It is therefore suggested that the precautionary position of total dependency is assumed in terms of rainfall events. However if the site discharge constraint is fairly generous in terms of throttle rate, the critical duration event may well be between 2 and 6 hours, in which case the assumption of total dependency can be relaxed.

A range of duration events for the 100 year return period would then be run on both models with the predicted water level of the river at the location of the storage outfall being used as the downstream water level. This methodology will result in a less conservative estimate of the additional storage volume needed to cater for periods of high river water levels.

It should always be borne in mind that the use of design storms does not necessarily reflect what happens in reality. These are current best practice methods used to arrive at a solution giving appropriate provision for situations that might take place. The consequence of failure should always be a part of an engineer's design approach to ensure a full appreciation of the proposed system design.

6.6 Attenuation Storage Design

This section details the methods and equations to be used to enable the determination of storage volumes for development sites.

The approach should always involve the use of a hydrograph method. Time series rainfall (TSR) is theoretically better than design events, particularly for frequent event criteria, as a complex mix of SuDS units will have runoff characteristics which will not be accurately reflected by the use of design storms. However, the computational effort, and the lack of an extended series suitable for any particular region, makes this approach relatively impractical. So although design events will be the normal approach for system design and analysis, where a suitable time series exists, a check on the system performance is recommended. Empirical design rules also exist for sizing certain SuDS items.

The method for finding the stormwater attenuation volume is:

Step 1	Find the greenfield peak runoff rate for the site;
Step 2	Apply this rate as a throttle to the model of the development and run it with a range of duration events for design return periods in accordance with the design criteria.

Assessment of the storage requirement using models is normally carried out by applying the maximum discharge flow rate as the discharge limit. This method provides a reasonable estimate of the volume needed. However depending on the configuration and design of the storage system, this will under-predict the volume by as much as 20 or 30% due to the variable head-discharge curve for any throttle if this is not represented in the model. Thus it will be important to be aware of the potential under-prediction and to prove the adequacy of the storage provision at the detailed design stage by building an accurate model which takes into account the depth-storage relationship together with the head-discharge relationship of the unit.

6.6.1 Assessment of Greenfield Site Rate of Runoff

There are numerous hydrological techniques currently in use to estimate green-field runoff rates. This section briefly reviews the techniques most commonly used by drainage engineers. Most of these methods have been developed in the UK, but they are considered applicable to Ireland. (The exception is the Rational Method, which in its original formulation is generally acknowledged to have been defined by an Irishman, Mulvaney). The methods include:

- Agricultural Development and Advisory Service, Report 345 (ADAS) (1982);
- Techniques based on the Rational Method;
- Prudhoe and Young, Transport and Road Research Laboratory, Report LR 565, (TRRL, 1973);
- Flood Studies Report (FSR) statistical and rainfall runoff – various methods (National Environment Research Council, 1975);

- Flood estimation for small catchments, Report no. 124 (Institute of Hydrology, 1994);
- Flood Estimation Handbook (FEH) (Institute of Hydrology, 1999).

The last method is mentioned due to its importance in the UK. However it has no applicability in Ireland as it is based on multiple regression characteristics for every location (1 km grid) across the UK, and therefore is not discussed further. Its only relevant feature is that the FEH rainfall analysis of data for UK has resulted in some significant changes in rainfall depths for various durations and return periods compared with FSR. This implies that the FSR values for Ireland might be equally in need of correction.

The proposed method used for determining peak flow rates for small greenfield catchments is *IH Report 124, Flood estimation for small catchments*. A direct comparison between this method and the ADAS 345 method, which is also commonly used in UK, is not easy, due to the different parameters used, but in general the differences have been found to be relatively small for most typical catchments. The other methods are thought to be less appropriate.

In theory FSR-based methods are limited to catchments greater than 50 ha while the ADAS method is only to be applied to catchments which are smaller than 30 ha. However for simplicity it is proposed that Report 124 is applied to all catchment sizes by applying it to a 50ha site and linearly interpolating the result for smaller areas.

6.6.1.1 Agricultural Development and Advisory Service (ADAS) Report 345 Technique

The Agricultural and Development Advisory (ADAS) Report 345, produced in 1982, details a technique which is primarily aimed at providing information to determine the size of pipes required for field drainage systems. This is an important distinction as the attenuation process of soil percolation will reduce the catchment response rate. The method is based on measurements taken on a number of small rural catchments.

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

$Q = S_T F A$

Where:

Q is the 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 average slope, maximum drainage length and 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.

Guidance on the values of the above variables is given in the ADAS report, together with a nomograph that can be used to estimate the flow. The predicted peak flow resulting from the ADAS equation for “grass” should be taken as being the one year return period flood and not the mean annual flood for the catchment. The other 2 curves represent the return periods of 5 and 10 years. Flow rates for higher return periods can then be calculated by using the appropriate regional growth curve.

6.6.1.2 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 on flood estimation for small catchments. The research was based on 71 small rural catchments (< 25 km^2). A new regression equation was produced to calculate $QBAR_{rural}$ the mean annual flood. $QBAR_{rural}$ is estimated from the equation:

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

Where:

$QBAR_{rural}$ is the mean annual flood flow from a rural catchment in m^3/s.

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 that accompany the Flood Studies Report.

QBAR can be factored by the regional growth curve to produce peak flood flows for other return periods.

Table 6.5 provides typical values of QBAR per hectare for a typical SAAR value for Dublin of 750mm for SOIL types 2, 3 and 4. QBAR growth curve factors from the proposed growth curves for the Dublin region are provided in Table 6.6.

	SOIL type 2	SOIL type 3	SOIL type 4
QBAR/ha (l/s/ha).	2.0	3.1	5.2

Table 6.5 Typical Values of QBAR for Dublin (based on 50ha)

Return period (years)	Growth curve factor
1	0.85
QBAR	1.0
10	1.7
30	2.1
100	2.6
200	2.9

Table 6.6 Proposed Growth Curve Values for QBAR for Dublin (interim)

Appendix E provides a worked example for the storage requirement related to this design criterion.

6.6.2 Assessment of Development Runoff Rate

Runoff from positively drained paved areas is effectively instantaneous by comparison with 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 high intensity short storms up to around 20 to 30 year return period events. Above this return period, short duration “summer” storms have intensities which are so great that temporary flooding takes place due to the inadequate capacity of the pipe system and gullies to cope with the volume of water. It is therefore unimportant to determine peak flow rates from the site for evaluating the volume site storage.

6.7 "Long term" Storage Design

The objective of providing "long term" storage is to protect the river from effects of the increased volume of runoff compared to greenfield conditions. Peak flow rates are addressed by the use of attenuation storage, but it is also important to minimise the additional volume of runoff created by the development.

As discussed earlier, the primary aim is to protect the river at times of flooding, and therefore this long-term storage volume need not be mobilised for small events. Although relatively difficult to design to operate in this way, (together with the additional uncertainty of design storms representing all real storms), it is possible to design long term flood storage areas which come into effect for larger events and only drain by infiltration. This volume can be quite large depending on the catchment soil type and density of development, so flooding of the "long term" storage unit(s) needs to start taking place at return periods significantly less than 100 years to ensure the volume retained at the 100 year event meets the criteria. Therefore consideration must also be given to meeting levels of service criteria.

Alternatively, and more simply, this volume can be provided in the form of infiltration volume around the site which comes into effect for all events. In this case it is particularly important that the soil characteristics and water table levels are suitable as there must be a reasonable expectation that much of this storage volume is available for an extreme event.

To determine a storage volume requires the selection of an event duration for the 100 year return period. Six hours has been selected as the duration of the design event to compute the additional storage. In theory, the river should be protected for its critical duration at the point of the development. However for simplicity and practicality it is proposed to use a duration which is appropriate for shorter rivers. This is not only relevant for those in the Dublin Region, but also it is these smaller rivers which are going to be more sensitive to the effects of development runoff.

The estimation of the "long term" storage volume is a simple calculation of finding the difference between the runoff volume generated by the development site and that for the greenfield site using the 100 year 6 hour event.

6.7.1 Assessment of Development Runoff Volumes

There are two Wallingford Procedure runoff models which are based on statistical correlation and the fixed percentage runoff model usually used when applying the Rational Method. The two Wallingford Procedure runoff models are described in Appendix D.

Although the Wallingford Procedure New UK PR equation is generally regarded as the most appropriate runoff model for simulation modelling, there are certain situations, particularly for developments in areas with SOIL type 4 (clay), where it will actually predict less runoff for extreme events than the formula used for predicting greenfield runoff volume (FSSR 16, *FSR Rainfall Runoff Model Parameters Estimate Equations Updated, December 1985*). Intuitively this seems unlikely, as paved surfaces have around 70 to 80 percent runoff, whereas the FSR analysis cannot give values much greater than around 55 percent. There are a number of arguments which can be made to support the Wallingford Procedure runoff model. The main one is that the land form in urban areas is heavily modified with many obstructions (garden walls, buildings, undrained low points) which prevent runoff from occurring from some paved and unpaved surfaces. However for simplicity and taking a precautionary position, it is proposed that a fixed runoff model for the paved surfaces and an allowance for runoff from the pervious surfaces, that is consistent with the analysis for the greenfield runoff volume approach, is used.

It is proposed that an 80% runoff is assumed for impervious surfaces and the SPR value of the soil is assumed for the pervious area based on the local soil type.

6.7.2 Assessment of Greenfield Runoff Volumes

The estimation of runoff volume from pervious areas using FSSR 16 is detailed in Appendix D. However this closely approximates to an assumption that runoff volume is equal to the SPR value for the soil type. Table 6.7 summarises the SPR value for the 5 soil types used in the FSR procedure.

SOIL	SPR value (% runoff)
1	0.1
2	0.3
3	0.37
4	0.47
5	0.53

Table 6.7 SPR Values for SOIL (pervious surface runoff factor)

6.7.3 Estimation of the Difference Between Greenfield and Development Runoff Volumes

Two extreme assumptions can be made with regard to greenfield runoff volumes after site development. The first is that there is no runoff from pervious areas due to urbanisation effects impeding runoff from these surfaces, and the second is that all the pervious area continues to contribute in the same way as it did prior to the site being developed. In practice the reality is somewhere between the two and is dependent on the design layout. For example a park area might be designed to positively not drain to the drainage system or river. In this case it would be reasonable to assume that no runoff allowance need be made for this area.

The following general formula provides an estimate of the "long term" storage volume. This calculation of excess urban runoff takes account of any surfaces, either impervious areas or pervious areas, which are not served by the drainage system.

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

Where:

Vol_{XS}	is the extra runoff volume (m^3) of development runoff over Greenfield runoff.
RD	is the rainfall depth for the 100 year, 6 hour event (mm).
PIMP	is the impermeable area as a percentage of the total area (values from 0 to100).
A	is the area of the site (ha).
SOIL	is the "SPR" index from FSR.
α	is the proportion of paved area draining to the network or directly to the river (values from 0 to1).
β	is the proportion of pervious area draining to the network or directly to the river (values from 0 to1).

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:

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

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

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

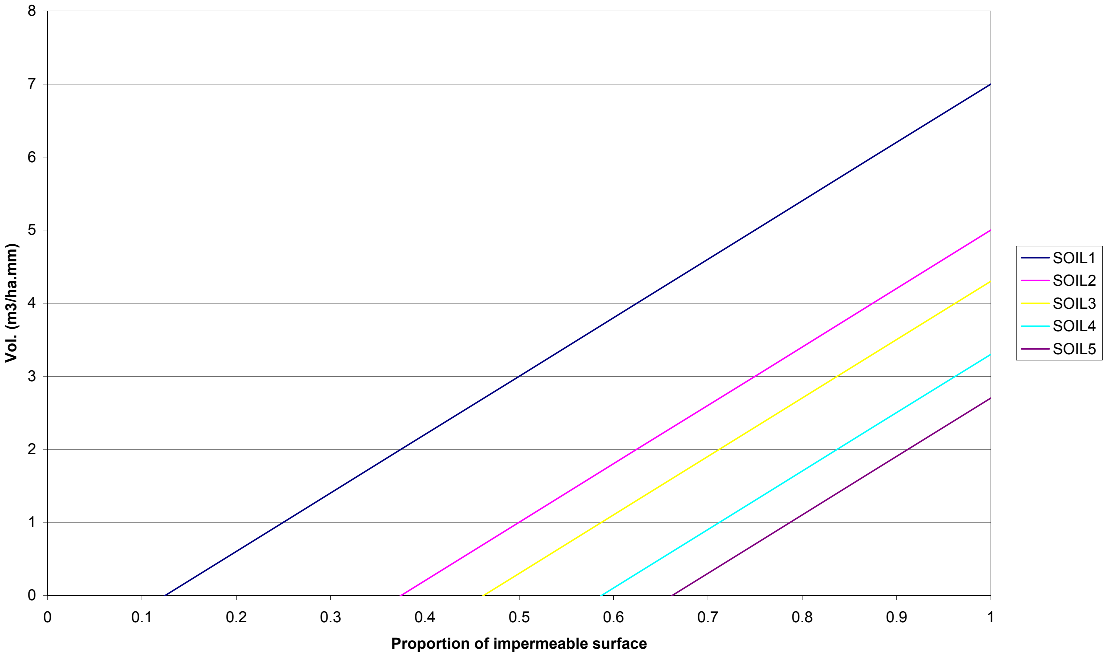

Figure 6.3 Long-term storage for developments where all pervious areas are assumed not to drain to the drainage network or river

Figures 6.3 and 6.4 illustrate the "long term" storage volumes that these two extremes (disconnected/connected pervious surfaces) represent for developments for different soil types for any development density. These figures demonstrate the importance of soil type, the use of infiltration to disconnect impermeable areas from the drainage network and the need to be efficient in designing the general landscape.

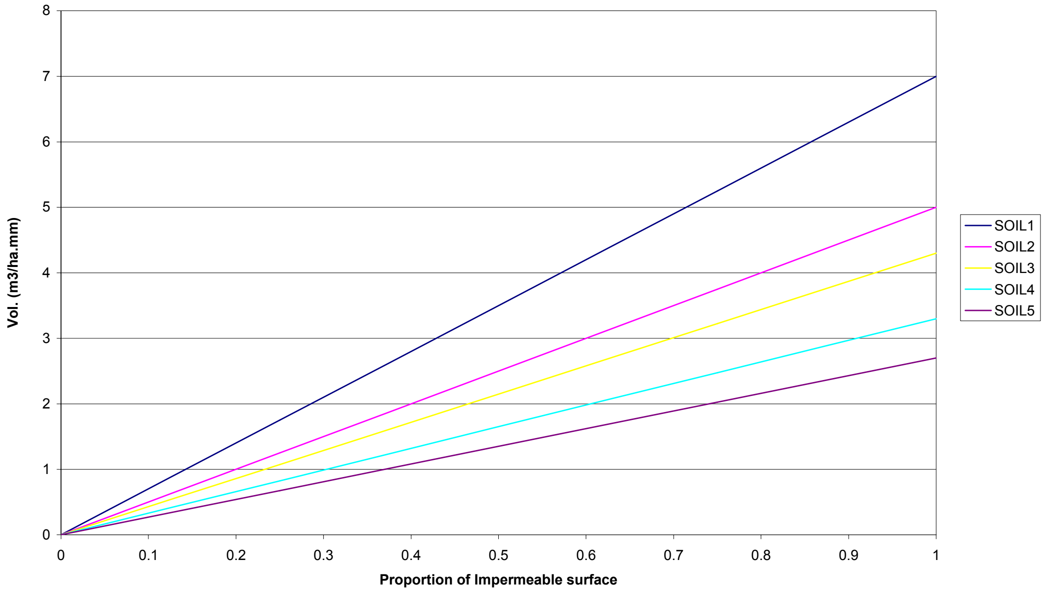

Figure 6.4 Long-term storage for developments where all pervious areas are assumed to be drained to the drainage network or river

There are elements of conservativism in these assumptions in not allowing for depression storage and evaporation or infiltration taking place during the event, but it does provide a rapid and robust method for assessing the maximum additional volume of runoff generated by a development.

For ease of reference the 100 year 6 hour event for the Dublin area is given in Table 6.8.

Location	M5-60 (mm)	Ratio "r"	Rainfall Depth (mm)
North Dublin and City centre	15	0.25	58.7
South Dublin	17	0.30	60.9

Table 6.8 Rainfall Depths for Dublin; 100 year 6 hour event

South Dublin is differentiated as the mountains make the rainfall characteristics for the region slightly different. However it can be seen that although the rainfall characteristics are different, the actual rainfall depth for the design event happens to be very similar. It is therefore recommended that a figure of 60mm is used throughout the Dublin region. It should be noted that soil type is a far more important variable in this regard and that although much of Dublin is categorised as SOIL type 2, in practice some areas might be closer to SOIL type 4. As this makes such a big difference to the "long term" storage requirements, it is important to carry out site tests on soil characteristics to choose an appropriate SOIL category.

Although slope is not a function of the procedure (and not included in the assessment of peak runoff rate in the greenfield peak discharge formula), the rate and amount of runoff from the greenfield site is going to be influenced to some degree by the slope. An intelligent and flexible approach in the application of these equations and criteria is therefore needed. Recognition and compliance to the key principles underpinning the drainage criteria is the important feature of this drainage design philosophy.

As this criterion is a relatively arbitrary one, it is considered inappropriate to modify it to take account of climate change.

6.8 Hydraulic Design of SuDS Systems

Sustainable Drainage Systems (SuDS) cover a range of methods used in the design of modern drainage systems. The objective of SuDS is to attenuate and reduce stormwater runoff volume and reduce pollution impact due to urban development. General criteria for selection of SuDS options are contained in the Regional Policy on Environmental Management. This section considers the hydraulic design aspects of SuDS. Their hydraulic characteristics are summarised in Table 6.9.

Drainage System	Rate of Discharge	Volume of Discharge
Direct Pipework	Very fast	No reduction
Swales (standard)	Fast – Medium	Limited reduction
Storage Tanks	Fast – slow	No reduction
Lined and unlined Ponds	Fast – slow	Potential for limited reduction
Detention Basins	Fast – slow	Limited reduction
Wetlands	Fast – slow	Limited reduction
Lined sub-pavement storage	Medium	Limited reduction
Unlined sub-pavement storage	Medium	Significant reduction
Filter drains	Medium	Limited reduction
Swales (under-drained)	Medium – slow	Significant reduction
Soakaways	Effectively none	Large reduction
Infiltration trenches	Effectively none	Large reduction
Infiltration from temporary storage*	Effectively none	Large reduction

Table 6.9 Hydraulic Characteristics of Drainage Systems (for large rainfall events)

**Land allocated for temporary flooding to meet “long term” storage requirements.*

The use of the term fast and slow here is only a qualitative statement relating the speed of runoff for piped systems (very fast) to greenfield behaviour (slow).

It must be recognised as a limitation of SuDS systems even though SuDS tend to have greater volumetric storage than pipe based systems, that extreme events, with large volumes of runoff, can overwhelm SuDS units as much as traditional pipe systems. In such circumstances, their performance in terms of attenuation and stormwater volume reduction is not necessarily any better than pipe based drainage systems. This should not be regarded as a failure, but as a normal consequence that should be explicitly catered for in the design process, with emergency overflow and flood routing arrangements included which direct flow away from properties. This is no different than the design process which should be followed for areas drained by pipe networks.

Hydraulic modelling assumptions for SuDS units are dependent on the site conditions (rate and proportion of runoff). Modelling of pervious pavements is still in its infancy and therefore a precautionary approach should be taken with its representation, but for most SuDS units their representation is relatively obvious in terms of the modelling approach and values to be used.

6.8.1 Hydraulic Categories of SuDS Components

The stormwater design procedure has defined the criteria to enable storage volumes and discharge limits to be calculated. However, these calculations need to be applied in reality using a range of drainage components. In each case there are practical issues that need to be considered to enable an effective hydraulic design of the drainage system to be built. This section deals with each component, providing guidance on the hydraulic aspects affecting their construction and proposed use.

It is important to be aware that design of these units for hydraulic performance needs to take a precautionary approach especially when selecting infiltration rates for soils which need to work during wet winter conditions.

Design of these units should comply with the Design Manuals for SuDS, CIRIA C521 and C522 and other relevant and subsequent publications, including the GDSDS Environmental Management policy document.

There are effectively three categories of SuDS units:

- Attenuation only - Retention and Detention Ponds, Tanks;
- Runoff reduction and attenuation - Swales, permeable pavements and filter trenches;
- No runoff –Infiltration systems.

6.8.2 Retention / Detention Ponds and Tanks

These types of stormwater units are aimed at providing storage and attenuation and some degree of treatment. The design of ponds and tanks uses the inflow/outflow hydrograph process. The actual volume required is defined by a matrix of parameters that are summarised as:

- Depth / area storage relationship;
- Head / discharge relationship;
- Throttle rate;
- Effective contributing area;
- Rainfall characteristics of the area;
- Level of service;
- Safety.

Some of these aspects have been addressed earlier and are therefore not discussed here. The hydraulic design requirements are considered below. Other aspects (safety for instance) are dealt with in the Environmental Management Policy document.

6.8.2.1 Depth / Area Storage Relationship

The depth / area storage function is largely dictated by topography and outfall levels. However other issues such as treatment processes in ponds will dictate depth requirements. Volumetric allowances for vegetation should also be provided for in ponds, which might be as much as 20 percent for heavily vegetated systems.

6.8.2.2 Head / Discharge Relationship

Structures are normally designed to a specified maximum discharge rate and this is usually achieved when the storage structure is full. This means that the outflow at lower water levels is passing less flow forwards making the effective volume needed larger, but theoretically approximately reflects the increasing greenfield peak flow rates for lesser events. This additional volume requirement can often be minimised by having the storage structure as an off-line pond, but this has disadvantages for water quality treatment. As this criteria involves 3 flow rates (1, 30 and 100 year return periods), analysis for attenuation storage needs to be carried out in 3 stages.

This analysis could result in the use of 3 orifices to achieve a good fit to the discharge requirements for the 3 return periods. In practice it should result in 1 low level orifice and a second outlet which might be an orifice or slot of some kind. It is important to make sure the final design is practical as well as effective. The flow rate control system should aim to approximate to the calculated system, but common sense must be exercised in finding the most practical solution.

Storage systems are often located near the river to which they discharge. River levels will provide backwater effects, which will modify the discharge at times of high water levels. This has been discussed in some detail earlier.

6.8.2.3 Throttle Sizes and Discharge Rate

There are practical difficulties in meeting hydraulic criteria for very low flow rates, as local authorities rarely take in charge 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. For example if 2l/s/ha is used for the 1 year throttle rate, and the minimum flow rate is considered to be 10l/s, then the minimum drainage area served is 5 ha.

This criterion for minimum orifice size, although a constraint for the authorities, need not necessarily apply to private owners. Systems such as pervious pavements, due to the very limited risk of obstruction, can be designed with orifice sizes of 75mm or even smaller.

This constraint draws attention to two aspects of good drainage practice. The first is that areas should be drained in an integrated manner, so that even if a single developer is only developing a 1ha site, it should fit into a larger drainage strategy. The second is that this element of storage may be better achieved by other drainage components, particularly those which have slow release characteristics for “small” events. SuDS units such as lined or unlined permeable pavement car parks or under-drained swales can provide low discharge rates.

6.8.3 Swales

Swales, Permeable Pavements and Filter Drains can be designed in various ways, and are therefore difficult to categorise hydraulically. In terms of their hydraulic behaviour, they generally fall into a composite group that provide both attenuation and runoff reduction. Their relative merits in each of these two categories is a function of their design, topography, the soil type and size of rainfall event. It is therefore important not to be too prescriptive about their generic attributes, but to consider for each their characteristics depending on the site situation and the type of rainfall event.

For clarity each of these units is considered separately, although the same hydraulic issues affect their design and performance. Aspects such as maintenance and operation and water quality are not dealt with here, but are covered in the Environmental Management policy document. These issues are:

- Runoff attenuation and reduction;
- Hydraulic & physical constraints;
- Level of service.

In terms of hydraulic and physical constraints, all infiltration structures should be built at least 1.0m above the maximum groundwater level. Infiltration structures must only be built where groundwater classification in that area allows. Sites located close to drinking water borehole abstraction points should carefully be considered in terms of the pollution risks related to the use of infiltration related units.

A major concern is knowing the design soil condition. Wet periods in winter are common and are predicted to become more frequent. The soil characteristics and its capacity to infiltrate in these conditions is not easily determined. Caution is required in selecting design infiltration rates for units relying on the ability to infiltrate which may cause a problem, if surcharged. CIRIA report 156 “Infiltration drainage – Manual of good practice”, 1996, provides guidance on how and when to use infiltration. The normal cut-off point for use of infiltration is 0.001 mm/s. With the emphasis now on maximising infiltration, its use should still be encouraged for percolation rates found to be around this value as well as at locations where better soil conditions occur. In locations where “failure” of a unit would cause a problem, overflow facilities should be provided.

Swales are effectively shallow wide ditches in which grass is grown and regularly maintained. Inflow into swales is usually by continuous distributed runoff from road surfaces, although these can be made semi-continuous with numerous point inputs.

6.8.3.1 Hydraulic and Physical Constraints of Swales

Swales are very susceptible to erosion and low flow pathways developing. To avoid this, gradients need to be minimised wherever possible. Recommended gradients for standard swales are between 1:20 and 1:300, to avoid erosion and ponding respectively. Point inflows should be avoided. Kerbs are often put in for safety reasons or to preserve the verge. Entry points for runoff should be as frequent as possible to avoid local erosion.

Pipes connecting swales under driveways or roads provide a focal point for erosion and sediment deposition. Conduits should be as large as reasonably possible and not sized on the basis of pipe capacity. Alternatively pipes may be selected as throttles to try and restrain flows. However as 150mm pipes are normally the minimum size acceptable, and as swales tend not to be very long, to prevent high flows developing, the use of 150mm pipes as throttles will not normally be very effective.

Overland flooding from swales that are full, due to extreme conditions or steepness of the catchment, needs to be assessed. Overland flow routes must be considered to ensure that flooding of properties does not take place. Houses located below the road level are especially at risk from major events and particular care is needed (whether or not swales are used) to ensure these properties are not flooded by extreme events.

Considerable variation occurs with regard to outflow design. There are three main methods:

- Invert level outflow;
- High level outflow;
- Infiltration outflow (under-drainage).

Each method is discussed with regard to attenuation and reduction of runoff.

6.8.3.2 Swales with Invert Level Outflows

With outflows at the invert of the swale, its hydraulic behaviour is the same as a rough channel with water level being either a function of normal depth, or if the outfall pipe is a constraint, the level is a function of the throttle and storage relationship.

In this situation, some attenuation is achieved and this varies with the return period of the event and the degree of throttling of the outflow. Volume reduction may be slightly enhanced, but in long wet winter periods, ground saturation may result in virtually no reduction of runoff. These types of swales therefore cannot be relied upon to meet either volume reduction or attenuation targets for large events.

6.8.3.3 Swales with High Level Outflows

A swale designed to have a high level outflow is effectively a combination of a mini retention basin and conveyance channel. The “deep” water allows low values of conveyance velocity to be determined which will minimise scour.

An important parameter for this type of swale is the permeability of the soil. Application of this method in clay soils will result in die-off of the grass due to long periods of saturation. In terms of the hydrological benefits, the effective reduction of runoff volume might be less than the volume of storage theoretically available due to antecedent conditions, especially if the soil is not very pervious.

The use of this type of swale is therefore more appropriate where soil conditions are relatively permeable or where enhanced infiltration in the base of the swale is provided. In these situations the volume of storage within the swale (below the outfall) could be used to assess the reduction in runoff volume. If all roads (which were appropriate to drain using swales) were designed in this way, the volume of runoff would be significantly reduced as well as contributing to water quality improvements.

6.8.3.4 Swales with Piped Under-Drainage

The difference between this type of swale and the previous two types is that it is not meant to function as a conveyance channel. The objective is to use the swale as a retention basin and for runoff treatment, with flows passing to a perforated drainage pipe below the swale. This enables the swale to be designed as a balancing system with a controlled outflow based on the pipe size serving the system of swales. The great advantage of this system is that there is considerably less risk of erosion from flows passing along the swale as they will tend to be short individual lengths. The physical problems related to pipe connections, which are needed to pass under roads and driveways crossing the swale, are also avoided.

Inflow / outflow design should be based on infiltration techniques and the hydraulic constraint of the receiving pipe. In addition the under-drain is likely to have a continuous low flow during wet winter periods and some account of this should be made in checking on the possible range of the system performance. Design therefore requires careful application to make the most of this drainage system.

If no under-drainage is provided and natural or enhanced infiltration into the soil alone is being used, the volume reduction achieved is 100% (until the swale is full). The use of these swales needs to be constrained to locations where saturation of the soil is unlikely and winter groundwater levels remain well below the bottom of the swale. If there is doubt about drain-down of the swale between winter events, reduction in available storage volume needs to be made.

The limited experience in UK has shown under-drained swales to be very effective. Their use in Germany is extensive.

6.8.4 Pervious Pavements

Although pervious pavements are traditionally made using granular material for the sub-structure into which the water percolates, there are a range of high voids-ratio plastic media products also available. Voids ratios range from 30 to 95%.

The water quality outflow from these pavements is generally high. It is thought that the treatment is mainly achieved by the geo-textile membrane (preferably unwoven) placed immediately below the blockwork. This requires aerobic conditions for the bacteria to be effective. Therefore although geo-textile might usefully be placed at the bottom of the structure for other reasons, it is unlikely to contribute to treatment of the surface water at this location.

Several permeable pavements have been monitored in UK and elsewhere in the world. The volumetric reduction is largely a function of whether the pavement is lined or not, and seasonal effects. Short storms in summer often have only a nominal outflow, while long wet winter events do not achieve a significant volume reduction compared with standard drainage.

The performance of unlined pavements is a function of both the receiving soil type and construction technique, as it has been found that permeable surfaces can have their porosity significantly reduced by the construction process. It is reported that unlined pavements, even in clays, still achieve considerable reductions of runoff for ordinary events.

For systems designed to only drain by infiltration, it is important to provide a relief pipe to cope with excess runoff in case of reduced infiltration rates and / or very extended wet periods, where surcharge would be a problem. Reduction of runoff over a season of rainfall may be very great, but hydraulic design of these units should be based on their performance under extreme conditions.

Lined pavements are built where there is a concern to protect the groundwater from pollutants. For lined systems, runoff reductions are still significant although less than unlined systems. During long wet winter periods, runoff volumes might only be reduced by 30 percent in lined permeable pavements, though average annual figures have been found to be up to 55 percent.

Observed runoff rates from these units, even in the wettest periods, are low, usually below 2l/s/ha, for much of the storm runoff volume. The maximum flow rates recorded are in the order of 25l/s/ha, but these may have been constrained by the outlet pipe system. The figures suggest that these units are very effective in limiting the impact of runoff on receiving streams and urban drainage systems.

6.8.4.1 Hydraulic and Physical Constraints

Pervious pavements often cover very large areas, such as supermarket car parks. In this situation it is possible to design the outfall pipe to act as a throttle for extreme events.

The use of pervious pavements in private driveways cannot be relied upon not to be modified, particularly by sealing (to avoid weed growth), as householders may seek to minimise maintenance effort. The use of permeable pavements on common car parking areas for groups of houses is more likely to remain as designed as these will probably be managed by either the local authority or a management contractor.

Permeable blocks are susceptible to clogging due to oils (from cars) and sediments (from flowerbeds and construction techniques). The industry has therefore now moved away from these products and use a construction gap between solid blocks of around 3mm to ensure hydraulic performance in the long term.

Point input of inflows into the sub-base should be avoided, unless the flow is known to have minimal sediment load, since clogging may take place in due course at these locations. Additional inflows can be introduced into a pervious pavement area from adjacent roof runoff, subject to adequate sediment protection provision. The design of the additional area that can be served is a function of the effective storage volume and design criteria applied.

It is advised to keep to traditional car park gradients (fairly flat), where possible, to minimise excessive hydraulic loading at the lower edges of the permeable pavements. Also the hydraulic outflow behaviour relies upon the base of the unit being flat and so creating a minimal hydraulic gradient towards the outflow pipe, if there is one.

6.8.4.2 Level of Service

The depth of sub-base storage zone needs to relate to the design rainfall depth taking into account the voids ratio of around 30 percent (for gravel based fill units). If this is less than the critical 30-year event (probably 6 to 12 hour duration), the overland flow or flood depth across the car park should be specifically designed. In practice with a design depth normally of 350mm at 30% voids, the depth of rainfall that it can serve is in excess of 100mm; more than a 100 year event even with no outflow.

Outflow pipework should be hydraulically designed for both the collector system and the high level relief, although the acceptable minimum pipe diameter is likely to be the main constraint if the unit is to be vested.

6.8.5 Filter Drains

Filter Drains are trenches adjacent to roads with flows passing into the soil from a trench filled with a coarse stone mix. A perforated pipe usually passes along the length of the trench, to ensure water levels are kept well below the road subbase. The depth of the trench below the perforated pipe can be selected to meet storage design requirements.

In wet winter conditions if the soil is saturated, it is likely that Filter trenches will work in reverse acting as drainage systems, contributing to runoff. The assessment of their hydraulic design performance both in terms of attenuation and runoff reduction is therefore site specific.

6.8.6 Infiltration Units – Soakaways, Infiltration Trenches and Flooding of Public Open Spaces

These are the only systems that do not have some runoff contribution to a receiving water. Where the soil conditions exist and groundwater classification allows, soakaway systems can minimise the impact of development runoff and maximise water resource recharge.

6.8.6.1 Soakaways and Infiltration Trenches

Soakaways have been in existence for many years. They range from rudimentary, rubble filled pits to large tank structures serving large areas of runoff. Their design is well covered by manuals, two of which are generally applied across the UK. These are the CIRIA report 156 and BRE 365.

Although soakaways have been applied to Highway drainage, their use for anything other than roof water is generally not advised, as the high sediment loads from road runoff usually cause blockage problems within 20 years. These problems can be avoided by appropriate upkeep, which involves routine removal and replacement of sand layers on an annual basis, but this philosophy of high maintenance levels is not attractive to local authorities. The whole life cost evaluation of this approach would probably not make this drainage solution the most cost effective approach for most situations.

All soakaway structures should be evaluated for extreme event exceedence and provided with overflow pipework where a certain level of service cannot be assured and there is a risk of flooding as a result. Consideration of topography is important to ensure overland flows are directed away from properties.

Infiltration trenches are an alternative to soakaways. They tend to be more effective in many instances as they allow much greater efficiencies to be achieved, due to the units having greater surface area per unit volume. Also as the bottom of the trench tends to be nearer the surface than the base of a soakaway, this reduces the risk of direct interaction between the infiltration unit and the groundwater table.

The use of Infiltration trenches in private properties to serve roofs is at some risk due to landscaping and gardening activities. They should be located at sufficient depth to ensure that they are unlikely to be damaged. They should not be located on common boundaries as construction of fences and hedges will damage the drainage system.

The location of filter drains should theoretically be constrained in the same way as soakaways, and should be at least 5m from the property in compliance with Building Regulations. However as they are not deep, it is suggested that the minimum distance should be at least three times the depth of the trench, assuming adjacent buildings have appropriate foundations.

In the UK, where pervious pavements have been used as infiltration units, these have been located as close as 1m from the property where the soil is highly permeable.

A 10 year event is commonly used for design of property infiltration systems. However this might be increased significantly if they are seen as one of the mechanisms for meeting the requirement for “long term” storage.

These units should individually serve only one or very few properties. This is needed to avoid flow taking place along a trench to a low point and focussing all the potential flooding in one garden / location.

6.8.6.2 Flooding of Public Open Spaces

Infiltration in public open spaces is not a recognised SuDS system, but is included to represent those areas where flooding is planned to take place only in extreme events to deal with either overland flooding or “long term” storage.

Parks and other types of open spaces need to be carefully contoured and provided with suitable under-drainage to ensure they dry out effectively. However, to comply with the principles of long-term storage, it is important that this is not rapid and therefore not connected directly to the main drainage system.

Design Requirements

Developers should continue to provide particular design details and parameters for large residential, commercial, industrial and institutional developments

Design criteria for stormwater drainage for both runoff attenuation and reduction should be used for drainage design for New Developments