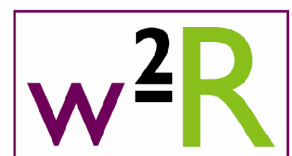


# Appendix 12.5

SuDS



**Staffordshire**  
County Council

A REPORT BY ENVIROS CONSULTING LIMITED: MARCH 2008

# **STAFFORDSHIRE COUNTY COUNCIL**

**FOUR ASHES ENERGY FROM WASTE PLANT  
SUSTAINABLE DRAINAGE STRATEGY STATEMENT**



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Drainage Strategy Statement

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## EXECUTIVE SUMMARY

Enviros Consulting was commissioned by Staffordshire County Council to undertake a storm water management assessment for the proposed Energy from Waste (EfW) plant site at Four Ashes in November 2007.

### *Drainage Strategy*

1. The site will be drained as two individual areas, one being roof drainage and the other road drainage, each with its own separate drainage system. These areas are shown on Figure 2.
2. Runoff from the site will be controlled so that peak runoff from the site will not exceed the runoff that would be expected from an equivalent greenfield site.
3. Flows from the roads area will be reduced to this greenfield runoff rate through the use of shallow ditches and a pond with floodable surrounds.
4. Flows from the roof area will be reduced through the use of green roof techniques and a downstream pond with floodable surrounds.
5. Discharge from the ponds will be into the Saredon Brook to the south of the site via the unnamed drain which flows along the site's eastern and southern boundaries.



## 1. INTRODUCTION

In February 2008 Enviro Consulting was commissioned by Staffordshire County Council (SCC) to prepare an outline sustainable drainage strategy for the proposed Energy from Waste (EfW) plant at the Four Ashes Industrial Estate in Wolverhampton, West Midlands. This work is based on initial work undertaken by Enviro as part of a flood risk assessment for the site. The purpose of this work is to prepare and demonstrate the feasibility of a drainage strategy for the site compliant with or exceeding current standards and policy and also following sustainability guidance provided by CIRIA.

This document sets out the surface water drainage strategy. A more detailed assessment with numerical justification and precise sizing of the design elements will be undertaken at the appropriate time.

## 2. SUSTAINABLE DRAINAGE

Flood risk in any area is controlled by a whole range of factors one of which is land use. Historically development within a catchment has resulted in water flowing into watercourses more quickly with rapid surface flow increased and infiltration to the ground reduced. As a direct consequence of this watercourses that flow from or through urban areas tend to respond more quickly to rainfall than rural watercourses. In addition peak rates of flow are correspondingly higher.

Flooding from these watercourses is therefore both more likely and potentially more severe. The social impact of this is high in terms of both damage to properties and disruption. The scale of these impacts is rising due to climate change and continuing inappropriate development.

Planning policy (Reference 1) therefore requires that the potential impact of a proposed development on flood risk elsewhere is considered as part of any planning application. Notably the management of surface water within a development is a material planning consideration. Typically this assessment is undertaken either within a flood risk assessment, or as part of a stand alone drainage strategy assessment.

Planning policy requires that volumes and peak flow rates of site runoff following development do not exceed the volumes and peak flow rates prior to development. An exception to this can be made if specific off-site measures are implemented to achieve the same net effect.

Current guidance provided by the Environment Agency goes further than this, suggesting that where re-development of brownfield sites is occurring, the opportunity should be taken to improve the local situation and reduce volumes and peak flow rates from a site down to the volumes and flow rates of an equivalent greenfield site. However challenging, particularly on confined urban sites, this guidance is increasingly being enforced.

The methods for achieving the necessary reductions in volumes and peak flow rates are collectively referred to as **Sustainable Drainage Systems<sup>1</sup>** (SUDS).

### 2.1 Sustainable Drainage Systems

SUDS aim to mimic natural systems on greenfield sites. In such situations water typically infiltrates to the ground and then flows through either shallow geology or deeper strata towards local watercourses. Alternatively where impermeable surface conditions exist water may at times flow overland through vegetation collecting in local hollows and depressions before eventually discharging to a surface water feature. The important thing to note is that natural systems generally provide mechanisms by which storm runoff is significantly attenuated and these processes are generally bypassed in urban developments, unless management techniques such as SUDS have been implemented.

Correctly designed SUDS enable water to be managed close to the location where precipitation falls. This is then released slowly over time into the local surface water system. As such the flow of water is decelerated and peak rates of discharge from a site are reduced. A range of potential benefits can occur as a result.

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<sup>1</sup> Also called **Sustainable Urban Drainage Systems** (SUDS) .

These potential benefits may impact on both the development site and the surrounding area and include the following:

- ◆ Alleviation of historic flooding problems at and downstream of a site;
- ◆ Development can be allowed in constrained areas where it would otherwise not be permitted;
- ◆ Removal of sediment from site discharge ;
- ◆ Biological treatment of oils and metals present in site discharge;
- ◆ Creation of pleasant amenity features within a site;
- ◆ Increase in groundwater recharge; and
- ◆ Improvement of base flow into local watercourses.

## 2.2 SUDS techniques

Available SUDS techniques can be broadly split into two categories as described below. It should however be noted that the division between these categories is not distinct and that many techniques could be placed in either category.

### 2.2.1 Infiltration drainage

Site runoff is directed to areas or features in direct contact with the ground and water is allowed to percolate down into the natural or artificial strata that underlay the site. This results in both groundwater recharge and increased base flow to local watercourses.

Infiltration of storm water, whilst preferable to all other solutions, is not always possible. Physical factors that may prevent the use of techniques that rely on infiltration include;

- ◆ Presence of leachable contamination beneath the site

Reference should be made to the Phase 1 and Phase 2 land quality assessments undertaken for the site (References 2 and 3).

- ◆ Site located in groundwater protection zone

A review of groundwater protection zones as shown on the Environment Agency website is undertaken. When a site is in the inner protection zone typically only roof drainage can be infiltrated. Reference should be made to the Flood Risk Assessment undertaken for the site (Reference 4).

- ◆ Shallow groundwater table beneath the site

This includes seasonally shallow groundwater. Reference should be made to the infiltration test results for the site (Reference 5).

- ◆ Impermeable / low permeability soils and geology beneath a site

Whilst outline feasibility can be assessed based on a broad understanding of the geology and soils beneath a site, site specific infiltration testing always needs to be

undertaken in accordance with the methodology outlined within BRE 365 (Reference 6) or CIRIA 156 (Reference 7). As before, reference should be made to the infiltration test results for the site (Reference 5).

### **2.2.2 Attenuation storage**

Runoff from the site is directed to a feature that can store the water. Discharge from this feature is then set at an agreed low rate (i.e. typically greenfield runoff rate). This throttling of flows can be achieved using a variety of possible outlet structures, including perforated plates or risers, small pipes, V-notch weirs or hydro-breaks. The volume of storage required in any given situation is set according to the difference between the volume of water generated from the site during the design event and that which can be discharged.

Water can be stored on a site in a number of ways, the choice of which is made based on the local context, the nature of the development, the space available, other amenity and ecological objectives and cost.

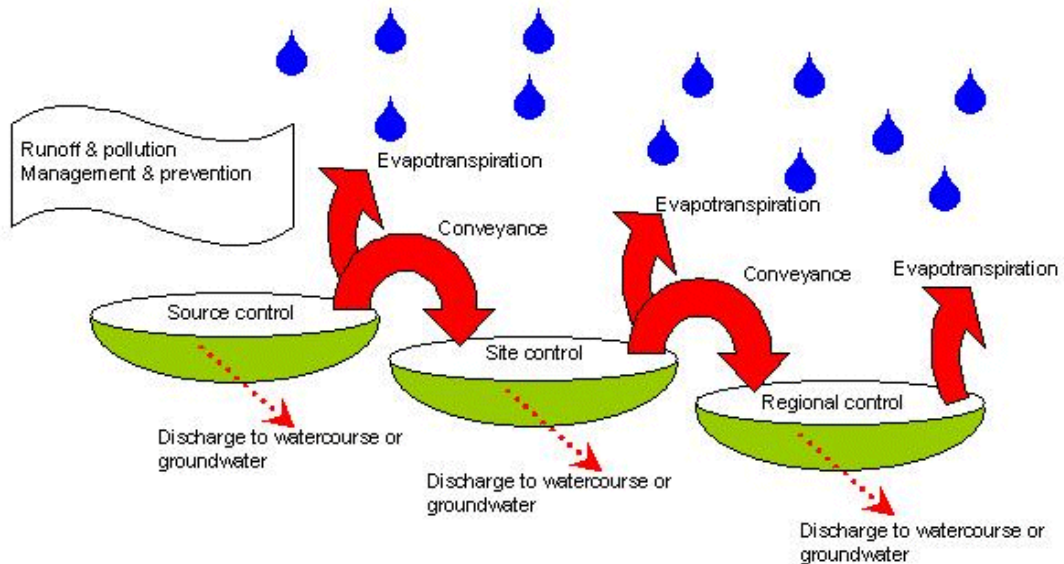
Using attenuation techniques is almost always possible although on constrained urban sites where space is very limited it can often be too difficult to find sufficient space. This is particularly problematic where these issues are only considered after the development design has been broadly set.

When relying on attenuation techniques as part of a SUDS scheme it is essential that the issue of the site outfall is considered. In some situations where development is occurring on a greenfield site it is possible that no obvious pre-existing site outfall exists. In these situations negotiation with third parties such as the local sewerage undertaker is often crucial before plans proceed too far.

## **2.3 SUDS Management Train**

When considering the choice of SUDS techniques within a given development the SUDS management train is a useful guide that sets out the principles of how and where control of storm water should occur. This concept is illustrated pictorially in Figure 1 below.

Figure 1 SUDS Management Train



This demonstrates that, ideally, generation of runoff and pollution is minimised by appropriate design. This would typically include maximising permeable surfacing, through permeable paving, green roofs and collecting and using the rainwater as a resource.

Where this is not possible, or cannot in itself reduce flows sufficiently then the next level of treatment and control should occur locally at an individual building level, returning water to natural systems as close to the source as possible. This typically occurs through the use of localised infiltration features distributed across the site. Local storage and discharge to surface water features is sometimes also possible.

In some situations such a strategy is not possible due to site constraints (see Section 2.2.1). In these situations it is encouraged that alongside the more local scale control measures, management and control of flows and pollution is undertaken at the site level. This typically involves attenuation storage of some kind. It is also important that water quality and other intrinsic benefits are considered. As such surface features or filtrating systems would be strongly favoured over traditional hard engineered solutions.

Very occasionally it will not be possible to achieve the necessary criteria as described above. Typically however this only occurs in very constrained urban sites where no or very limited flexibility exists with regard to site and development design. Other situations may exist where economically achieving the necessary criteria is highly challenging and may therefore discourage regeneration. If there is a strategic imperative for development to proceed, then the Local Authority could undertake or facilitate / approve more regional storm water control measures. These may take the form of a large attenuation lake serving a number of developments, or alternatively engineering modifications can be made to the receiving watercourse.

It should however be noted that such measures are considered as the least favourable SUDS tier and are not considered relevant or appropriate for the Four Ashes site.

### 3. BASELINE SITUATION

#### 3.1 Hydrological Receptors

##### 3.1.1 Ground

An intrusive investigation was undertaken by Enviros staff in February 2008. This found that the ground conditions underlying the site were variable in terms of the thickness and composition of the Made Ground and Drift Cover. The underlying solid geology is classified as a Major Aquifer (Reference 3). The site is located within an outer groundwater source protection zone (SPZ 2), as defined by the EA, and also in a designated Nitrate Vulnerable Zone (NVZ) under the Nitrates Directive.

Moreover the land quality assessment of the site indicates that there are several contaminants of concern (including arsenic, ammonium, nickel, boron, hydrocarbons and PAHs) present in the groundwater below the site. Elevated concentrations of ammonium, benzo(a)pyrene, zinc and arsenic were also detected in the soil, which could leach into the groundwater and affect the Major Aquifer underlying the site (Reference 2). To avoid leaching and further spread of contamination, it is recommended that infiltration on the site is prevented by fitting drainage features with impermeable liners.

The infiltration testing conducted in the superficial deposits (up to 2mbgl) at the site indicates that the formation has negligible infiltration rates. It was observed that much of the subsurface layers were saturated. Furthermore, at depths of between 1m and 2m (the drainage zone of the infiltration trenches) the ground was shown to have a relatively high clay content and therefore typically low permeability. These two characteristics combined would account for the negligible infiltration rates recorded at the site (Reference 5).

##### 3.1.2 Surface Water

###### ***Saredon Brook***

The Saredon Brook flows in a westerly direction along the southernmost site boundary. It is a tributary of the River Penk, which it joins some 3km downstream. The brook has a catchment area of approximately 69km<sup>2</sup> upstream of the site (Reference 8).

###### ***Staffordshire and Worcestershire Canal***

The canal is located at its closest distance 150m north of the site. In this area it runs east to west. Approximately 1km east of the site it splits in two, where the main branch runs in a southerly direction and the now disused Hatherton Canal branch continues to the east.

The Staffordshire & Worcestershire Canal is 74km in length from the River Severn at Stourport Basins to the Trent and Mersey Canal at the Great Haywood Junction. It was constructed in 1772 and has 45 locks although there are none within 1km of the site.



### 3.2 Existing Site Drainage Network

There is currently no existing formal site drainage system on the site. However an unnamed drain is located along the eastern boundary of the site which turns in the south-eastern site corner and then flows along the southern boundary until it joins Saredon Brook.

## 4. PROPOSED STRATEGY FOR THE SITE

### 4.1 System Objectives

Given the nature of the development proposed and the environmental conditions, the sustainable drainage strategy at the Four Ashes site needs to meet the following criteria for the lifetime of the development taking possible changes due to climate change into account.

- ◆ Discharge from the site should not normally exceed the median annual runoff from an equivalent greenfield site and the system should be designed such that the annual probability of higher runoff rates is less than 1%.
- ◆ The annual probability of any part of the system being liable to any form of flooding should not be greater than 3.3% with the lowest levels of service being directed toward those aspects of the development which are least sensitive to flooding.
- ◆ The system should be designed to help ensure that the quality of discharge from the site is of a good standard.
- ◆ The SUDS management train should be employed meaning that control of storm water should be encouraged at a building, then site level.
- ◆ The SUDS features used should be appropriate to the land use with more naturalised features and ecological enhancement encouraged around the periphery of the development.
- ◆ The system should be designed such that both flow and pollution control measures are in place through the construction of the site.
- ◆ As required under Construction Design Management (CDM) regulations the health and safety implications of all design works, including those for the drainage strategy should be considered and undue risks avoided.
- ◆ Ongoing management and maintenance requirements of all systems should not be unduly arduous.

### 4.2 Climate Change

PPS25 states that flood risk should be considered and managed through the lifetime of the development. For industrial development the Environment Agency recommend that a development lifespan of 50 years is assumed. Within this period significant changes in climate are expected to occur due to global warming. These are expected to exacerbate the risk of flooding on local, regional and national scales.

With regard to site drainage strategies the major potential pathway for such impact will occur through changes in rainfall patterns. Current global circulation models are not able to precisely predict what these changes might be, but indications are that in England winters will become wetter, and summers drier, with an increased occurrence of short duration high intensity events.

In the light of this PPS25 recommends that design rainfall depths and intensities may increase by 20% by 2085. All storm water systems within the site will therefore

be designed based on the recommended assumption that peak rainfall depths for all events could be up to 20% greater than would currently be predicted. All design calculations contained within this report have been based on this assumption.

## **4.3 Modelling**

### **4.3.1 Runoff Modelling**

#### ***Greenfield Runoff***

The greenfield peak runoff rate from the site has been determined using the methodology set out in Institute of Hydrology Report 124 (Reference 9). This technique returns an estimate of the maximum allowable discharge rate of 3.3l/s/ha when considering the site as a whole.

The assumptions made for the estimation of the greenfield runoff rate, from the existing site are outlined in Appendix 1.

#### ***Proposed Development Runoff***

The potential runoff generated from the proposed development and site layout has been estimated using the Modified Rational Method (Reference 10). The parameters and methodology used in this model are outlined in Appendix 2.

### **4.3.2 Attenuation Storage Modelling**

On sites where infiltration of storm water is not possible the rate of site discharge must be controlled through the use of storm water storage areas within the development. These areas receive storm water during periods of high flow and hold it, discharging it gradually over an extended period of time following the storm.

The volume of storage required at a given site is determined through modelling the difference in the volume of runoff created within a site and the volume of runoff that can be discharged safely. The difference between these two volumes must be stored if flooding on the site is to be avoided. The methodology used within this study for calculating the storage volume required is taken from standard industry guidance (Reference 11). Further details of these calculations are contained in Appendix 3.

## **4.4 Site Drainage Strategy**

Gravity drainage of surface water on any site is governed by the topography which we have assumed will, broadly, remain as shown on the topographic survey prepared for the site by Infomap Surveys (Appendix 4).

Based on topography and development plans the site has been divided into three drainage areas (Figure 4). The runoff from the roofs (drainage area 1) can be regarded as independent from the road drainage (drainage area 2) and drainage systems can therefore be managed accordingly.

## **4.5 Site SUDS Indicative Design**

It should be noted that this SUDS design for this site is indicative. The results will be used to demonstrate that the objectives set out above are achievable given the current site Masterplan. More detailed design work will be undertaken at the

appropriate time. Consequent designs will however adhere to the strategy set out here.

Initial design and calculations have been carried out according to guidelines set out in CIRIA's SUDS Manual (Reference 12).

#### 4.5.1 Drainage Area 1 – Roofs<sub>2</sub>

This area covers an estimated 1.3 hectares (ha), of which at least 1.1ha will be designed as a green roof. Details of the structural roof design were not known at the time of writing this report. It is recommended that the pitch of those parts of the roof proposed as green roof will be at least 1 in 80 and no greater than 1 in 3 (Reference 12).

For the purpose of these initial design calculations it has been assumed that a low-cost, low-maintenance extensive roof will be constructed. Green roofs are normally designed to store runoff from a 1 in 2 year storm (with climate change allowance) reduced to the 1 in 2 year greenfield runoff rate. This requires a storage volume of 228m<sup>2</sup>. Assuming a conservative value of 20% water retention by volume of growth medium, this would require a depth of growth medium of approximately 105mm. Extensive roofs are typically constructed with a depth of growth medium of between 25mm and 125mm.

It has been assumed that the green roof area will store at least 228m<sup>2</sup> of runoff during all storm events. However a total attenuation volume of 1063m<sup>3</sup> is required for the total roof area. As such, a second SUDS technique will be used to provide the remaining required attenuation volume of 835m<sup>3</sup>. It is proposed that a pond is constructed in the north-eastern corner of the site for this purpose. The green roof areas will be fitted with outlets / overflows, from where runoff will be routed into the pond. Runoff from any areas of conventional roofing (0.2ha has been assumed) will be piped directly into the pond.

Ponds are designed to hold a permanent pool volume for water quality treatment. The temporary storage required for attenuation of storm water is held above the permanent water level, in an extended floodable area around the permanent pool. In this case, a permanent pool volume of between 143m<sup>3</sup> and 195m<sup>3</sup> is required.

Outfall arrangements from the pond are described in Section 4.7 below.

#### 4.5.2 Drainage Areas 2 – On-site Roads

The contributing catchment of this area covers approximately 1.2ha in total. This includes areas of hard-standing (mainly roads and parking) but also contributing landscaped areas, as ground infiltration rates have been found to be negligible.

Drainage area 2 will be drained by shallow ditches which will be constructed on the outside of the proposed road and associated parking / pedestrian areas (Figure 2). Initial calculations show that shallow ditches with a total width of 2.6m would be required to provide sufficient capacity for conveyance of runoff. It has been assumed that the shallow ditches would have a base width of 0.6m, side slopes of 1 in 4 and a depth of 0.25m. Checks have been carried out to ensure that a first stage of water quality treatment is provided for. This requires a minimum residence time

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<sup>2</sup> Please note that for the purpose of calculating runoff from the roof area, it has been assumed initially that a 100% impermeable (i.e. conventional) roof structure would be constructed. The runoff value obtained (788l/s) has then been used to calculate the total attenuation storage required for the roof area. The storage capacity of the green roof has been calculated separately and this value then deducted from the attenuation volume required in the downstream pond.

in the shallow ditch of 10mins to ensure settlement and a flow depth of no more than 100mm for filtration. For effective filtration, the grass cover of the shallow ditches should be maintained at a length of approximately 100mm. A check has also been carried out to ensure a flow velocity of 1m/s is not exceeded for erosion control.

For the purpose of initial design calculations it has been assumed that there are no flow controls in the shallow ditches and that the shallow ditches will not provide any attenuation storage. Importantly, the shallow ditches will provide a first stage of treatment for the road runoff, which will be contaminated by oils, hydrocarbons and possibly more.

Attenuation and the second stage of treatment will be provided by a pond in the south-eastern corner of the site. The (temporary) attenuation volume required here is 457m<sup>3</sup>. The treatment volume (= permanent pool volume) required is between 134m<sup>3</sup> and 184m<sup>3</sup> in size.

Outfall arrangements from the pond are described in Section 4.7 below.

#### **4.6 Additional Water Quality Treatment**

The flood risk assessment for the site (Reference 4) recommends the use of oil interceptors for the runoff from roads and parking areas. It is possible to incorporate these into the design of the shallow ditches. However, both capital and maintenance costs are high for these devices. In addition, the use of oil interceptors would result in the implementation of flow controls on the shallow ditches.

As such an alternative third treatment stage, as required for runoff from industrial sites (Reference 12), is proposed here in the form of a wetland area (most likely a reed bed). Constructed wetlands are now commonly used to treat effluent or 'dirty' runoff. The mechanisms by which treatment is provided includes sedimentation, filtration, adsorption, biodegradation and volatilisation. It should be noted that wetlands need a continuous base flow and their land-take is generally high. However, it may be possible to construct a wetland in the high flood risk area to the southwest of the site. Discussion with the Environment Agency is required to confirm if this is appropriate.

#### **4.7 Outfall Arrangements**

The ponds will discharge into the existing unnamed drain which flows along the site's eastern, then southern boundary. Discharge will be restricted to greenfield runoff rates, i.e. 4.3l/s for the north-eastern pond and 3.9l/s for the south-eastern pond). The existing on-site drain will continue to discharge into the Saredon Brook in the south-western corner of the site (i.e. in the high fluvial flood risk zone), as it currently does.

## 5. REFERENCES

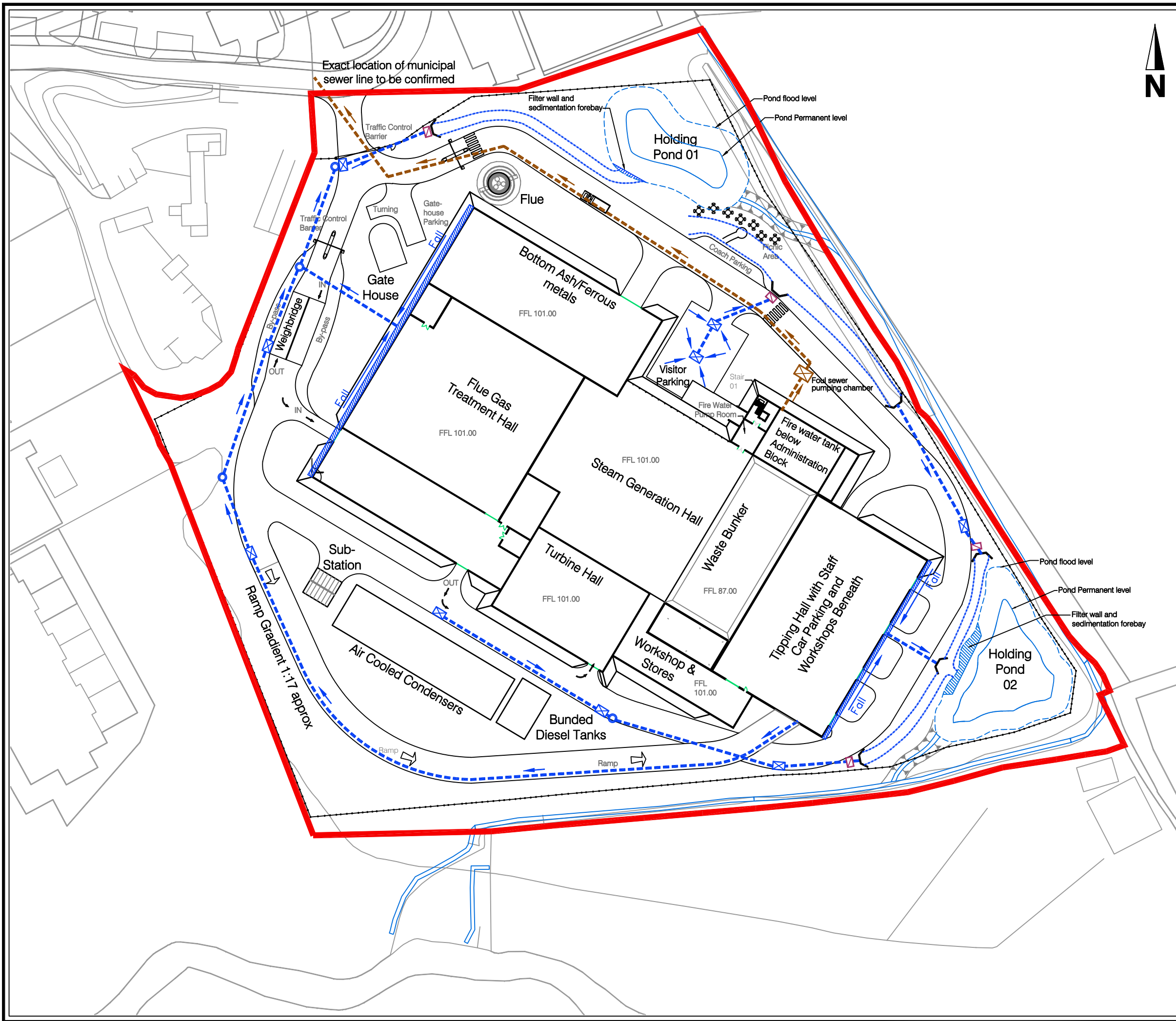
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- Reference 12. CIRIA C397 – The SUDS Manual, CIRIA, 2007



**FIGURES**



**Figure 2 SUDS Plan**



**KEY:**

- Site Boundary
- ST/W Swales
- ST/W Pipework
- ST/W Manhole
- ST/W Grid Channel
- ST/W Road Catchpit
- ST/W Pipe Outflow
- Foul Sewer Manhole
- Foul Sewer Pipework
- Class 1 : By-Pass Oil Separator

REV.	DESCRIPTION	DATE



**PROPOSED ENERGY FROM WASTE FACILITY AT FOUR ASHES**

**FIGURE 2**  
SITE DRAINAGE LAYOUT

SCALE	1:1,000	CAN	ST0070013X
CONTENT	PLI	DRAWN	AJR
CHECKED		DATE	APRIL 2008





**APPENDICES**



## 1. GREENFIELD RUNOFF CALCULATIONS

**Calculation Sheet (MAFF 1982, IH 1994)**

Catchment Area (A) =	2.5	ha
Maximum Length of Catchment (L) =	250	m
Average Slope of Catchment (S) = $\left( S = \frac{\text{height}}{L} \right)$	0.01	
Annual Average Rainfall (AAR) =	679	mm

**ADAS 345 (not advised for catchments > 30ha)**

Catchment Characteristic (C) = $\left( C = 0.0001 \frac{L}{S} \right)$	2.08	
Dominant Crop Type =	grass	
Soil Type Factor (ST) =	0.5	
F Number = (Using graph, C, crop type and AAR)	4.26	
1 in 2 year Peak Flow = $Q = S_T \times F$	2.1	l/s/ha
1 in 2 year Peak Site Flow = $Q_s = Q \times A$	5.3	l/s
1 in 100 year Storm Event Peak Site Flow = $Q_s \times 2.57 =$	13.7	l/s

**Institute of Hydrology Report 124 (not advised for catchments < 50ha)**

Soil Index (SOIL, from FSR) =	0.4	
1 in 2 year Peak Flow =	3.3	l/s/ha
1 in 2 year Peak Site Flow = $Q_s = 0.0108 \text{AREA}^{0.89} \text{SAAR}^{1.17} \text{SOIL}^{2.17}$	8.2	l/s

Where the catchment area is less than 50ha, the area (AREA) is set at 50ha then the resultant runoff is scaled by the ratio of the actual site area and 50ha.

1 in 100 year Storm Event Peak Site Flow = $Q_s \times 2.57 =$	21.1	l/s
--	------	-----



## 2. POST DEVELOPMENT RUNOFF CALCULATIONS

## Post-Development Modified Rational Method Calculation Sheet Drainage Area 1 - Roofs

Site Area (A) = 1.3 ha  
 Maximum Altitude = 103 m aOD  
 Minimum Altitude = 100 m aOD  
 Maximum flow path (L) = 200 m

These site descriptors can be used to calculate the time taken for water to travel from the furthest most point to the site outfall. After this period of time, known as the time of concentration, the entire site is contributing to site runoff. Maximum flow off the site should coincide with this.

This length of time can be calculated using the Bransby Williams formula shown below

$$T_c = \frac{58.5 L}{A^{0.1} S^{0.2}}$$

Time of Concentration ( $T_c$ ) = 10.5 minutes

The depth of rainfall that will fall within this, and any other, period of time can be estimated using Depth Duration Frequency (DDF) modelling as outlined in the Flood Estimation Handbook (IH, 1999). A series of descriptors are available on a one kilometre grid across the UK on the FEH CD ROM. These are used to obtain a rainfall depth, which can then be easily converted to intensity.

Rainfall Intensity (I) = 191.6 mm/hr during the 10.5 minute, 100 year storm

This rainfall intensity is converted to a flow off the site using the Wallingford procedure (1981). A number of further descriptors that can be defined on either a regional, local or site level, are required. These are:

UCWI ( <b>U</b> rban <b>C</b> atchment <b>W</b> etness Index)	66	(non dimensional)
SOIL (Related to site permeability)	0.4	(non dimensional)
PIMP ( <b>P</b> ercentage of <b>IMP</b> ervious Area)	100	%

These descriptors are used to calculate the percentage runoff (PR) from the site as follows.

$$PR = 0.829 PIMP + 25 SOIL + 0.078 UCWI - 20.7$$

Therefore;

Percentage Runoff (PR) = 77.3 %

Given this site runoff can be estimated using the following formula

$$Q = 2.78 C_v C_t I A$$

Where:

$C_v$  (a routing coefficient) = 1.3, and

$C_t$  = PR/100

Given the above peak flow from the site is estimated at **0.768 m<sup>3</sup>/s** = **768 l/s**



## Post-Development Modified Rational Method Calculation Sheet Drainage Area 2 - Roads

Site Area (A) = 1.2 ha  
Maximum Altitude = 103 m aOD  
Minimum Altitude = 100 m aOD  
Maximum flow path (L) = 200 m

These site descriptors can be used to calculate the time taken for water to travel from the furthest most point to the site outfall. After this period of time, known as the time of concentration, the entire site is contributing to site runoff. Maximum flow off the site should coincide with this.

This length of time can be calculated using the Bransby Williams formula shown below

$$T_c = \frac{58.5 L}{A^{0.1} S^{0.2}}$$

Time of Concentration ( $T_c$ ) = 10.6 minutes

The depth of rainfall that will fall within this, and any other, period of time can be estimated using Depth Duration Frequency (DDF) modelling as outlined in the Flood Estimation Handbook (IH, 1999). A series of descriptors are available on a one kilometre grid across the UK on the FEH CD ROM. These are used to obtain a rainfall depth, which can then be easily converted to intensity.

Rainfall Intensity (I) = 190.4 mm/hr during the 10.6 minute, 100 year storm

This rainfall intensity is converted to a flow off the site using the Wallingford procedure (1981). A number of further descriptors that can be defined on either a regional, local or site level, are required. These are:

UCWI (**U**rban **C**atchment **W**etness **I**ndex) 66 (non dimensional)  
SOIL (Related to site permeability) 0.45 (non dimensional)  
PIMP (**P**ercentage of **I**MPervious Area) 55.5 %

These descriptors are used to calculate the percentage runoff (PR) from the site as follows.

$$PR = 0.829 PIMP + 25 SOIL + 0.078 UCWI - 20.7$$

Therefore;

Percentage Runoff (PR) = 41.7 %

Given this site runoff can be estimated using the following formula

$$Q = 2.78 C_v C_t I A$$

Where:

$C_v$  (a routing coefficient) = 1.3, and

$C_t$  = PR/100

Given the above peak flow from the site is estimated at **0.381 m<sup>3</sup>/s** = **381 l/s**

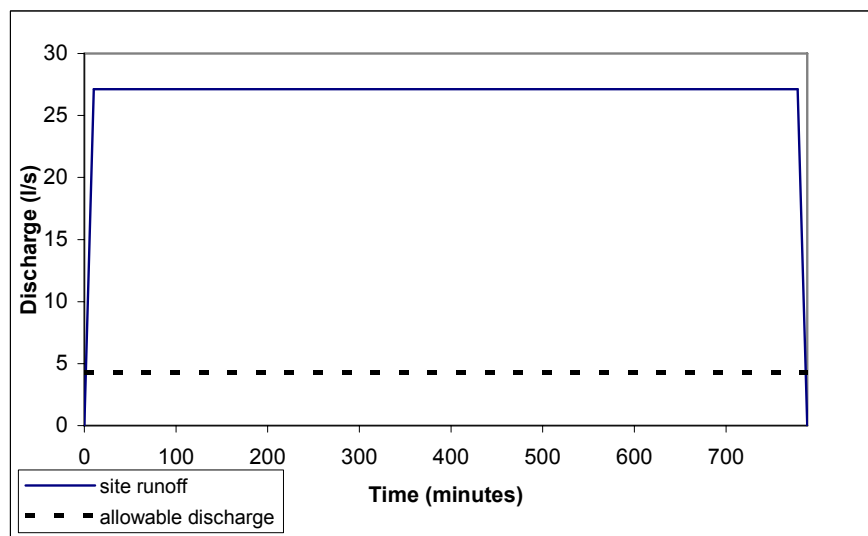


### 3. STORAGE VOLUMES CALCULATIONS

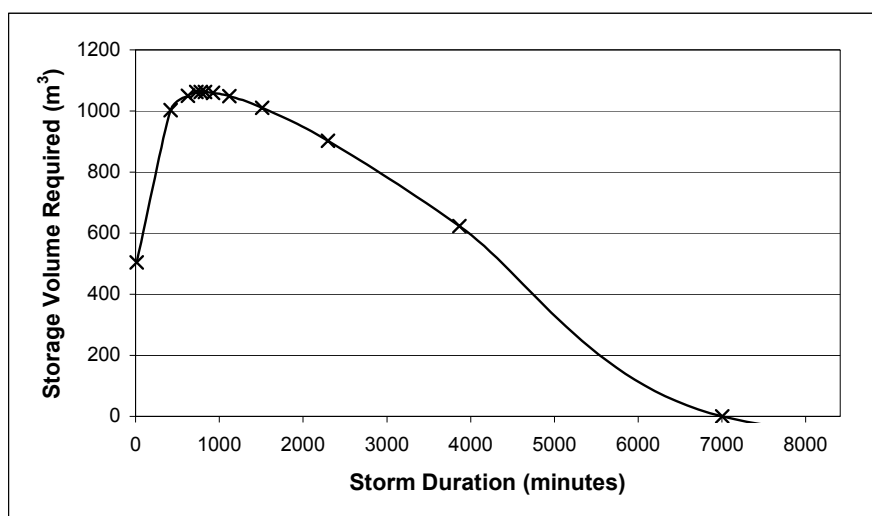
## Storage Requirement Estimation, Calculation Sheet

### Drainage Area 1 - Roofs

During intense rainfall events the volume of water arriving at the site outfall will exceed the rate at which it is either possible or allowable to discharge it. In this situation on site storage is required to prevent flooding. For very short events, whilst peak flows are often very high, total volumes are relatively low due to the short time period. For very long events the outflow will be similar to or exceed the flow generated by the site, so again required storage volumes will not be large. Between these two extremes are a series of events where significant storage is required. A range of events must be considered to ensure that the design event is captured.



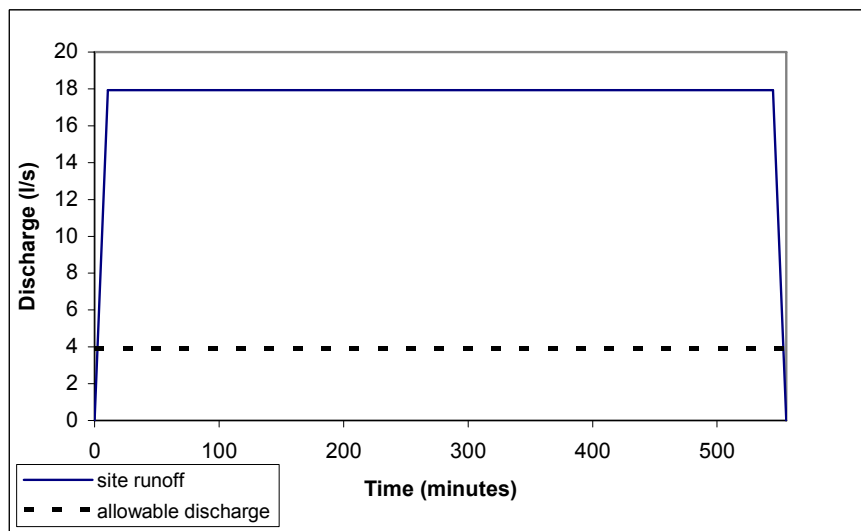
The graph above plots both the water arriving at the site outflow and the maximum allowable discharge. The area above this second line (dashed) corresponds to the required storage. This area has been calculated for a range of storm durations and the required storages obtained are plotted against storm duration below.



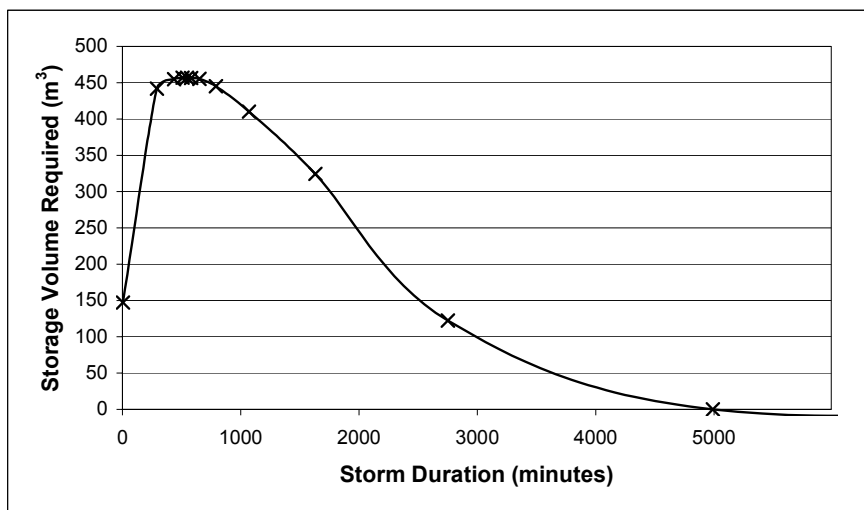
This indicates that the design storm for storage is the 778 minute 100 year storm. To prevent flooding during this event 1063 m<sup>3</sup> of storage would be required.

## Storage Requirement Estimation, Calculation Sheet Drainage Area 2 - Roads

During intense rainfall events the volume of water arriving at the site outfall will exceed the rate at which it is either possible or allowable to discharge it. In this situation on site storage is required to prevent flooding. For very short events, whilst peak flows are often very high, total volumes are relatively low due to the short time period. For very long events the outflow will be similar to or exceed the flow generated by the site, so again required storage volumes will not be large. Between these two extremes are a series of events where significant storage is required. A range of events must be considered to ensure that the design event is captured.



The graph above plots both the water arriving at the site outflow and the maximum allowable discharge. The area above this second line (dashed) corresponds to the required storage. This area has been calculated for a range of storm durations and the required storages obtained are plotted against storm duration below.



This indicates that the design storm for storage is the 545 minute 100 year storm. To prevent flooding during this event 457 m<sup>3</sup> of storage would be required.