Benefits and Performance of Sustainable Drainage Systems

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Summary

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This project was conceived to address the lack of quantitative data available to the drainage industry on the observed performance of sustainable drainage systems (SUDS), with respect to both hydraulics and water quality. The study was developed to improve knowledge of the level of performance that can be expected of different SUDS components and, where possible, to determine the likely influence of design criteria and maintenance activities on performance levels.

The specific objectives of the project were:

- To determine the technical performance and environmental benefits of using SUDS, through field monitoring at three operational sites that included a range of SUDS components;
- To identify any observable degradation in performance and contributory influences (including the effectiveness of maintenance);
- To improve guidance on the selection, design and maintenance of SUDS with a focus on maximising performance.

In order to be able to set the results of this study in context and to optimise the value of existing monitoring datasets, a comprehensive literature review of published SUDS performance information was also instigated. This review also provided valuable insight and guidance into the most appropriate methods for data interpretation and system performance assessment.

Data collation programmes are inherently extremely expensive and many published studies have therefore used numerical modelling to predict system performance, using established process and/or hydraulic equations, often verified with observed data. The value of both 'monitoring' and 'modelling' approaches is recognised and such studies have been reported on within the literature review. The approach of developing representative and validated system models with which to predict performance during more extreme conditions was also adopted within this piece of work for one of the monitoring sites.

This report describes the collection, analysis and interpretation of quantitative hydraulic and water quality performance data for a range of sustainable drainage systems. Conclusions and recommendations are drawn with respect to the monitoring, design and maintenance of future schemes.

Acknowledgements

This study has been undertaken under a DTI Partners in Innovation Contract 39/5/7134 cc2144. The objective of the project was to draw conclusions regarding the performance of SUDS systems and the likely influences of design criteria and maintenance activities, through collation and analysis of quantitative monitoring data. The project work was undertaken by HR Wallingford under project number MDS0427.

The Project Steering Group, many of whom made significant financial contribution to the work, guided the work undertaken by HR Wallingford. The names of the steering group members and their affiliations are listed below.

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Glossary

Attenuation	Slowing down the rate of flow, with a consequent increase in the duration of the flow.
Attenuation storage	Temporary storage required to reduce the peak discharge of a flood wave
Balancing pond	A pond designed to attenuate flows by storing runoff during the peak flow and releasing it at a controlled rate during and after the peak flow has passed. The pond always contains water.
Berm	A raised earthen bank or ledge used as a barrier.
Biodegradation	Decomposition of organic matter by micro-organisms and other living things.
Calibration	Adjusting model parameters in order that the model predictions matched observed measurements.
Catchment	The area contributing surface water flow to a point on a drainage or river system.
Check dam	Low weir used to slow flows and encourage infiltration.
Colloidal	A colloid or colloidal dispersion, is a form of matter intermediate between a true solution and a suspension. Microscopic particles of one substance, said to be in the dispersed or solute phase, are distributed throughout another, said to be in the dispersing, continuous, or solvent phase.
Concentration	The mass of a substance per unit volume of water in which it is dissolved.
Design storm	A storm with a specified profile, intensity and duration.
Detention basin	A basin, normally dry, constructed to store water temporarily to attenuate flows and provide some treatment.
Duration	The time period over which an event occurs.
Effluent	Fluid flowing out of a system
Extended detention basins	A detention basin designed to retain runoff for an extended period to provide a significant degree of treatment.
Filter drains/trenches	A linear drain consisting of a trench filled with a permeable material, often with a perforated pipe in the base of the trench to assist drainage.
Filter strip	Gently sloping vegetated areas designed to drain surface runoff as sheet flow from impermeable surfaces and remove sediment.
Geomembrane	A synthetic, low permeability membrane that is used to prevent the movement of fluids (often made of polyvinyl chloride (PVC) or high density polyethylene (HDPE)).
Geotextile	A fabric manufactured from synthetic fiber (in a woven or loose, non-woven state) that is designed to achieve specific engineering objectives, including seepage control, media separation (eg, between sand and soil), filtration, or the protection of other construction elements such as geomembranes

Greenfield	Land that has never been developed, other than for agricultural or recreational use.
Greenfield runoff	The runoff rate and volume from a site prior to development.
Impermeable	Will not allow water to pass through it.
Impermeable surface	An artificial non- porous surface that generates a surface water runoff after rainfall.
Infiltration	The passage of surface water into the ground.
Infiltration basin	A basin that is normally dry that is designed to store and infiltrate surface runoff into the ground.
Infiltration trench	A trench, designed to infiltrate surface water into the ground.
Influent	Fluid flowing into a system
Interceptor (pond/wetland)	A small pond/wetland that can be used to contain upstream pollution via the use of a penstock or other flow constraint.
Lag (time)	The time delay between the peak of the hydrograph (or other point on the hydrograph) at the inlet and the equivalent peak (or other point) at the outlet.
Loads/loadings	Total mass of a substance.
Mean	
Median	
Orifice	A small opening within a
Pavement	Technical name for the road or car park surface and underlying structure, usually asphalt, concrete or block paving. NB The path next to the road for pedestrians (colloquially called "pavement") is properly termed the footway.
Percentile	A value on a scale that indicates the percent of a distribution that is equal to it or below it. For example, a score at the 95th percentile is equal to or better than 95 percent of the scores.
Permeability	A measure of the ease with which a fluid can flow through a porous medium. It depends on the physical properties of the medium, for example grain size, porosity and pore shape.
Permeable surface	A surface formed of material that is itself impervious to water but, by virtue of voids formed through the surface, allows infiltration of water to the sub-base through the pattern of voids, for example concrete block paving.
Pervious surface	A surface that allows inflow of rainwater into the underlying construction or soil.
Pollution	A change in the physical, chemical, radiological or biological quality of a resource (air, water or land) caused by man or man's activities that is injurious to existing, intended or potential uses of the resource.
Rainfall time series	An observed or created series of rainfall depths in time.
Removal efficiency	The effectiveness of the removal of pollutants from the runoff, by the drainage component.
Residence time	The length of time that inflows from the upstream drainage system are likely to be retained within the permanent water body.

Retention pond	A pond where runoff is detained to allow settlement and biological treatment of some pollutants, as well as attenuate flows. The permanent pool volume provides treatment of the runoff.
Return period	This indicates the average period, in years, between events of the same intensity or of a greater intensity than a particular event.
Runoff	Water flow over the ground surface to the drainage system. This occurs if the ground is impermeable, is saturated or if rainfall is particularly intense.
Sedimentation	The process of subsidence and deposition of suspended matter from by gravity.
Short-circuit	Flow occurring rapidly from inlet to outlet without passing through appropriate treatment zones.
Siphon outlet	An arrangement whereby water is induced to flow naturally from an upper level to a lower level through a pipe or hose which spans an intermediate level that is higher than either.
Stochastic	Statistically generated
SUDS	Sustainable drainage systems or sustainable (urban) drainage systems: a sequence of management practices and control structures designed to drain surface water in a more sustainable fashion than some conventional techniques (may also be referred to as SuDS).
Suspended solids	Undissolved particles in a liquid.
Swale	A grass-lined channel designed to convey surface water, as well as controlling and treating the flow.
Treatment	Improving the quality of water by physical, chemical and/or biological means.
Treatment storage	Storage provided to enable poor water quality to be improved by sedimentation and other treatment processes.
Wetland basin	A pond that has a high proportion of emergent vegetation in relation to open water and usually has a requirement for a continuous base flow.
Wetland channel	A low flow conveyance channel that contains dense vegetation.

Abbrieviations

API_5	Antecedent Precipitation index (over the previous 5 days)
BOD	5-day Biological Oxygen Demand
Cd	Cadmium
CEH	Centre for Ecology and Hydrology
CFD	Computational Fluid Dynamics
CIRIA	Construction Industry Research and Information Service
COD	Chemical Oxygen Demand
Cr	Chromium
Cu	Copper
Defra	Department for Environment, Food and Rural Affairs
DOC	Dissolved Organic Carbon
FEH	Flood Estimation Handbook (Centre for Ecology and Hydrology (CEH), 1999)
FSR	Flood Studies Report (Institute of Hydrology, 1975)
FSSR	Flood Studies Supplementary Reports (Institute of Hydrology, 1976-1987)
Ni	Nickel
PAH	Polycyclic Aromatic Hydrocarbons
Pb	Lead
PR	Percentage Runoff
SUDS	Sustainable Drainage Systems
NH_4-N	Ammoniacal Nitrogen
NO_2-N	Nitrite
NO_3-N	Nitrate
NO_x-N	Oxidised Nitrogen
PO_4-P	Phosphate
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TP	Total Phosphorous
TSS	Total Suspended Solids
V_t	Water Quality Treatment Volume
Zn	Zinc

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Appendix

1. Introduction and Scope

1.1 INTRODUCTION

This project was conceived to address the lack of quantitative data available to the drainage industry on the observed performance of sustainable drainage systems (SUDS), with respect to both hydraulics and water quality. This study was developed to improve knowledge of the level of performance that can be expected of different SUDS components and, where possible, to determine the likely influence of design criteria and maintenance activities on performance levels.

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Data collation programmes are inherently extremely expensive and many published studies have therefore used numerical modelling to predict system performance, using established process and/or hydraulic equations, often verified with observed data. The value of both ‘monitoring’ and ‘modelling’ approaches is recognised and such studies have been reported on within the literature review. The approach of developing representative and validated system models with which to predict performance during more extreme conditions was also adopted within this piece of work for one of the monitoring sites.

1.2 SCOPE

This report begins by summarising the results of a review of published assessments of observed and modelled SUDS performance. The selection process for the SUDS monitoring sites adopted for this project is then described, and the data collation process and data interpretation and analysis is presented for each site. Site-specific and generic conclusions and recommendations have then been developed. The content of the report is briefly summarised below:

Chapter 2	Summary of literature review
Chapter 3	Monitoring site selection
Chapter 4	Hopwood Services Treatment Train
Chapter 5	Permeable Pavement, Kidderminster
Chapter 6	Retention pond, Thorpe Business Park, Leeds
Chapter 7	Permeable Pavement, Wokingham
Chapter 8	Conclusions

2. Summary of literature review

2.1 INTRODUCTION

The full literature review of published material documenting the hydraulic and water quality performance of SUDS is presented as Appendix 1. The review includes information from 63 international literature sources, many of which address different aspects of SUDS performance. A summary of the review is provided in this chapter, including key conclusions and recommendations, many of which contributed towards the planning of the project site monitoring programmes and the approach taken to the analyses of the collated datasets.

2.2 COLLATED PERFORMANCE DATASETS

A number of studies were identified as providing comparable quantitative detail of hydraulic and water quality performance. The number of each SUDS component for which quantitative data was collated is shown in the following table. A significant proportion of the data was extracted from the International Stormwater Best Management Practices (BMP) database (http://www.bmpdatabase.org/).

Table 2.1 Number of studies where quantitative monitoring data was available

SUDS component	Nr of studies where quantitative monitoring data was available	Nr of sites taken from ASCE BMP database
Permeable car park	5	
Grass swale	21	14
Retention pond	37	31
Detention basin	12	11
Wetland basin	9	9
Wetland channel	7	6

The type of system most intensively studied was found to be the retention pond, followed by the grass swale and the detention pond. Numerous additional studies were reviewed, but the format and content of the presented results (units, method of data collection, parameters etc) did not allow appropriate comparison of performance information.

Due to a lack of data, inconsistency in published details, and poor data comparability, it was not possible to undertake analysis based on hydraulic performance data. However, information from different sources was collected and is presented for each SUD type in the full literature review.

In terms of water quality, the chemical parameter most often presented was found to be Total Suspended Solids (TSS), followed by Lead (Pb) and Zinc (Zn). The total number of independent samples or sample sets analysed by the published studies for each component is shown in Figures 2.1 and 2.2.

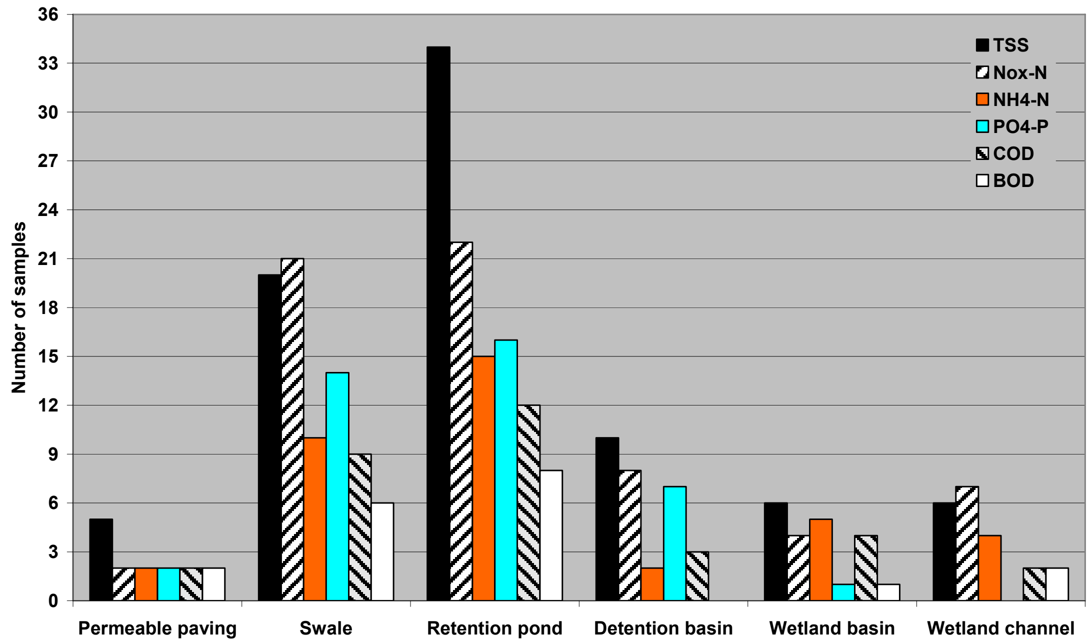


Figure 2.1 Total Number of Independent Study Samples or Sample Sets (Biological Parameters)

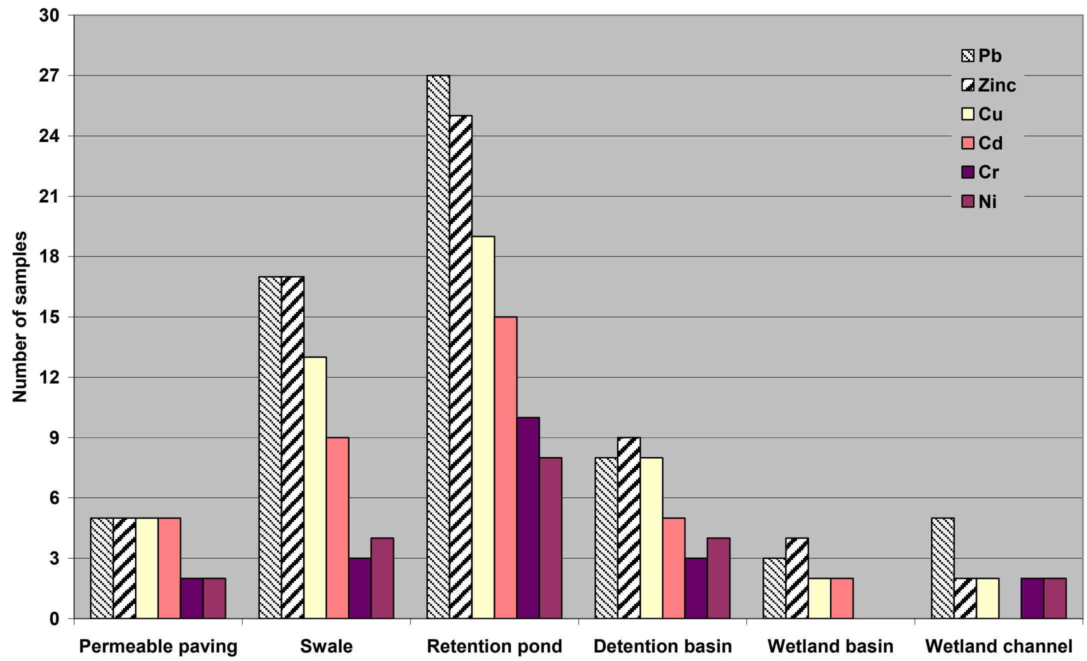


Figure 2.2 Total Number of Independent Study Samples or Sample Sets (Heavy Metals)

Although almost all of the studies investigated nutrients in the form of Nitrogen (TN, TKN, NH_4-N, NO_x-N, NO_3-N, NO_2-N) and/or Phosphate (TP, TP dissolved organic, TP dissolved, PO_4-P), the differences in the parameters measured made any interpretation complicated and subject to a high degree of uncertainty.

2.3 HYDRAULIC PERFORMANCE

Conclusions from the review of published detail on hydraulic performance of SUDS systems were that:

Ponds / Wetlands

- Siphon outlets can offer improved hydraulic conditions within a wetland over standard orifice outlets in terms of capturing runoff and providing a consistent water level regime;
- Where length:width ratios of pond/wetland systems are < 4:1, the efficiency with which incoming water is distributed within the system will be sacrificed unless a submerged berm at the inlet or flow distribution structure is employed;
- Vegetation within wetland systems is likely to contribute positively to the efficiency with which incoming water is distributed within the system;

Swales

- Hydraulic performance of swales is generally strongly influenced by soil infiltration characteristics and other factors that influence the infiltration capacity;
- Increased vegetation depths contribute to a higher retention of water in swales and lower peak velocities;
- For upstream point inflow scenarios, increasing swale slope tends to lead to increased mean velocities, increasing swale length to decreased mean velocities, and the presence of check dams reduces runoff peaks and volumes;

Permeable pavements

- The most unfavourable conditions with respect to the hydraulic performance of permeable pavements are at high API5 levels (wet antecedent conditions) and heavy storm rainfall (> 10 mm), at which point expected outfall volumes may be as much as 80 % of the event rain depth (i.e. approaching standard paved runoff volumes).

An additional finding was that although changes made by the drainage system to percentage run-off (PR) from the site is commonly used as an indicator of hydraulic performance, this parameter tends to be strongly correlated with rainfall and antecedent conditions and should not therefore be used to compare performance between systems or types of system. Flow duration curves derived from absolute values of peak flow (l/s/impermeable catchment area in ha) and flow attenuation (expressed as ratio of inflow and outflow characteristics) for a set range of inflow conditions are likely to provide more consistent tools for performance appraisal.

2.4 WATER QUALITY PERFORMANCE

Historically, water quality performance has generally been published in terms of percentage removal of the measured pollutant. Undoubtedly this parameter has been extremely useful in the water treatment industry, where the influent hydraulic and pollutant loads are relatively constant or vary within predictable ranges, and where the treatment process is ‘managed’ and thus relatively independent of seasonal and diurnal changes. However, one of the main characteristics of urban runoff is the highly variable character of the hydraulic and pollutant loads, and since SUDS systems tend to utilise natural treatment processes, their performance is strongly dependent on both loading and seasonal and diurnal changes. Work by other organisations corroborates the recommendation from this review that general stormwater management system efficiencies should not be specified in terms of percent removal, since some SUDS may be potentially mis-characterised as less effective because of cleaner influent concentrations.

The potential problems associated with specification of percentage removal efficiencies are demonstrated in the following figures. Figure 2.3 shows the median reported removal efficiencies for TSS for different types of SUDS for the period of monitoring i.e. averaged over the total number of monitored events.

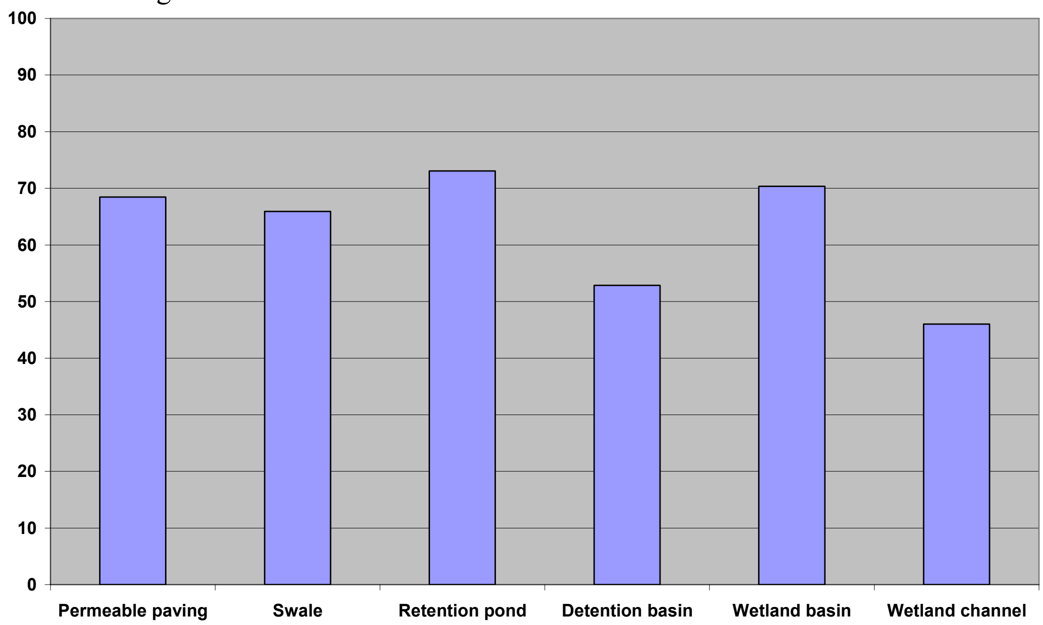


Figure 2.3 Median reported percentage removal of TSS for different types of SUDS

In the above figure the porous car park, the grass swale, the retention pond and the wetland basin appear to have similar median removal efficiencies. However the observed outlet concentrations shown in Figures 2.4 – 2.6 show significant differences.

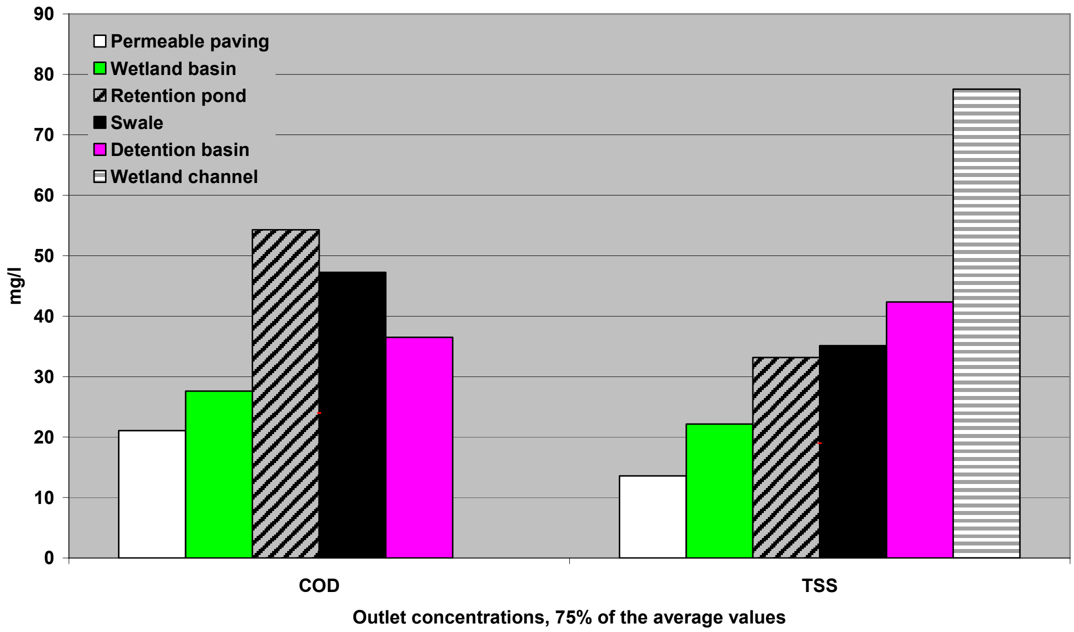


Figure 2.4 Outlet concentrations of COD and TSS (75th percentile of the average values i.e. 75 % of average outflow concentrations lie below this value)

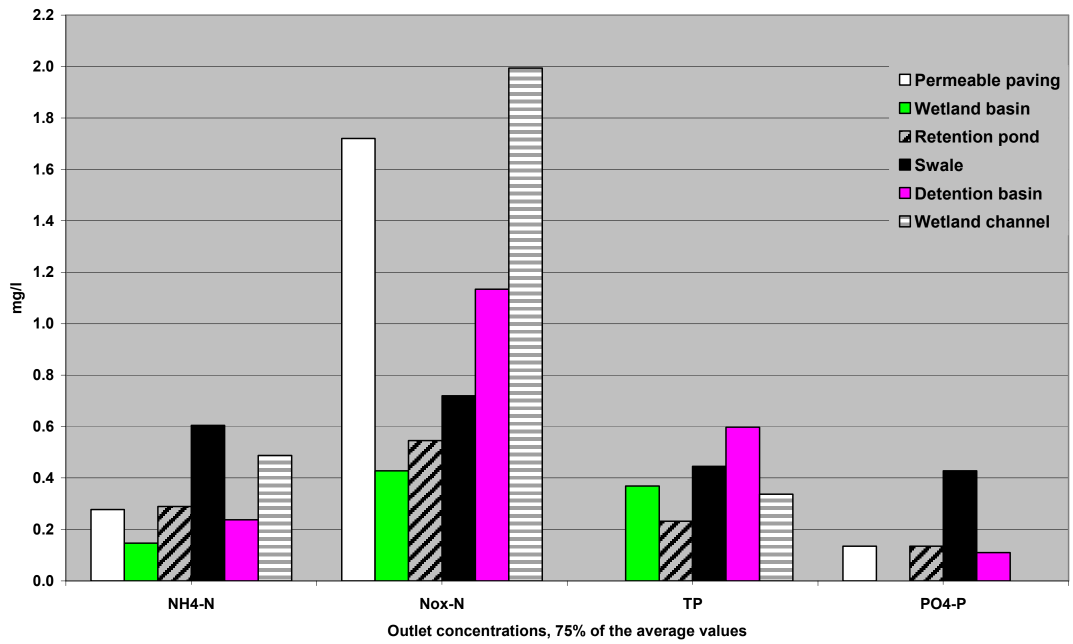


Figure 2.5 Outlet concentrations of NH4-N, NOx-N, TP, PO4-P (75th percentile of the average values)

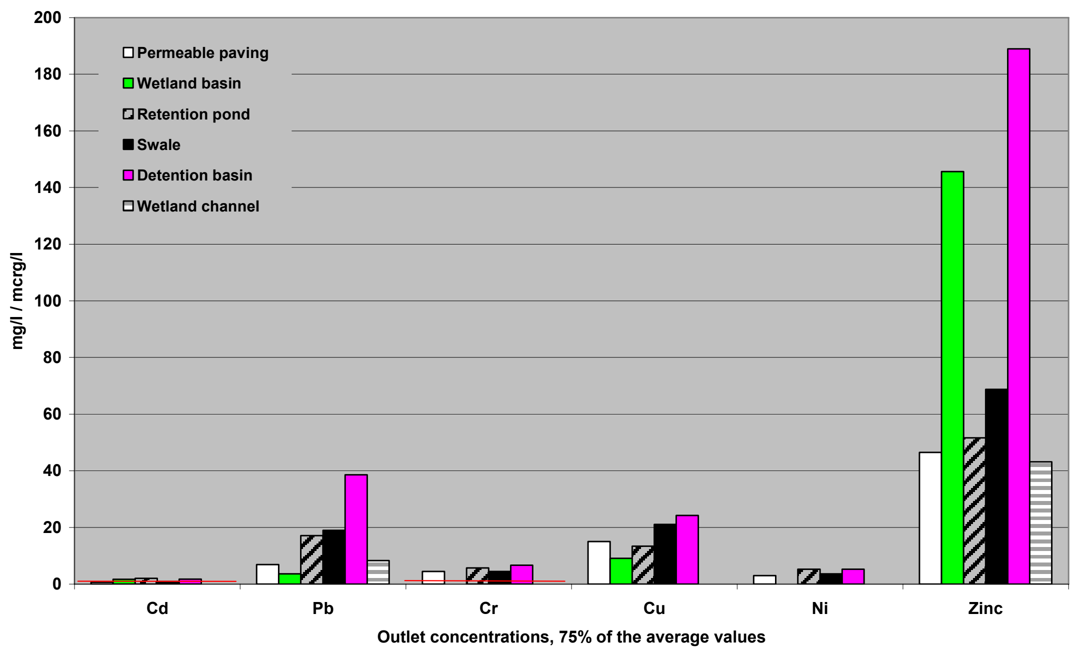


Figure 2.6 Outlet concentrations of Cd, Pb, Cr, Cu, Ni, Zn (75th percentile of the average values)

Therefore, although at an individual site is it still important to test whether the SUDS system has a statistically significant effect on water quality, effluent (outflow) quality is thought to be a better way to characterise efficiency of systems in general.

In order that the measured effluent quality data could be assessed against recognised standards, information was collated on the different water quality standards and/or categories in place in Europe and across the UK. Table 2.2 is taken from the HR Wallingford report SR 625 ‘Measuring the Ecological Benefits of Sustainable Drainage Schemes’ (HR Wallingford, 2003). It shows the biologically relevant levels of some commonly measured chemical determinands for both still and flowing waters, from various sources.

Table 2.2 Biologically relevant levels of some commonly measured chemical determinands

Variables selected for inclusion include those which are biologically relevant for which sufficient suitable data for the analysis were readily accessible.

Contaminant	The Drinking Water Standards. The Water Supply Regulations 1989	Level in minimally impaired ponds in the UK	Environment Agency River Ecosystem Class 1 limits (RE 1)	Environment Agency River Ecosystem Class 2 limits (RE2)	Environment Agency River Ecosystem Class 4 limits (RE4)	Swedish EPA limits	Urban Waste Water Treatment Directive 91/271/EEC	Minimum concentrations causing observable biological effect
Biochemical Oxygen Demand		No data	2.5 mg/l (90 percentile)	4 mg/l	8 mg/l	No data	25 mg/l	Not applicable
Soluble Reactive Phosphorus (PO_4-P)		Median: 5 µg/l Mean: 69 µg/l n = 162	No definition			No definition		Not applicable
Total Phosphorus	2200 µg/l	Median: 77 µg/l Mean: 190 µg/l n = 49 Note that this value should be treated with caution owing to the relatively small number of sites for which data area available.	No definition			100 µg/l (May – October) Data derived from lakes; concentrations above this value are regarded as hypertrophic. Note that some ponds may naturally be hypertrophic.	2000 µg/l	Not applicable
Total Oxidised Nitrogen		Median: 13 µg/l Mean: 496 µg/l n = 158	No definition			No definition		Not applicable
Total Nitrogen		Median: 1.5 mg/l Mean: 2.9 mg/l n = 45 Note that this value should be treated with caution owing to the small number of sites for which data are available.	No definition			1.25 - 5.00 mg/l (May - October)	15 mg/l	Not applicable

Table 2.2 Biologically relevant levels of some commonly measured chemical determinands (continued)

Contaminant	The Drinking Water Standards. The Water Supply Regulations 1989	Level in minimally impaired ponds in the UK	Environment Agency River Ecosystem Class 1 limits (RE 1)	Environment Agency River Ecosystem Class 2 limits (RE2)	Environment Agency River Ecosystem Class 4 limits (RE4)	Swedish EPA limits	Urban Waste Water Treatment Directive 91/271/EEC	Minimum concentrations causing observable biological effect
Suspended sediments		Median: 9.3 mg/l Mean: 19.1 mg/l n = 103	No data			No data	35 mg/l	25 mg/l[1]
Ammonia	500 µg/l	Median: 0.067 mg/l Mean: 0.27 mg/l n = 103	0.25 mg/l (90 percentile)	600 µg/l	2500 µg/l	No data		1.7 µg/l (US EPA Chronic Value)
Copper	3000 µg/l	11.48 µg/l	Hardness (mg/l CaCO3) – Copper (µg/l) ≤10 – 5 >10&≤50 – 22 >50&≤100 – 40 >100 – 112	5 – 112 µg/l	5 – 112 µg/l	Rivers: 1 µg/l Lakes: 0.3 µg/l		6.54 µg/l 1.1 µg/l (predicted NOEC) 5.3 µg/l (US EPA Chronic Value)
Cadmium	5 µg/l	No data	No data			Rivers: 0.003 µg/l Lakes: 0.005 µg/l		0.66 µg/l (US EPA Chronic Value)0.4 µg/l (Level below which 75% of European rivers fell[2])

[1]EIFAC (1964)
[2]Stanners and Bourdeau (1995)

Table 2.2 Biologically relevant levels of some commonly measured chemical determinands(continued)

Contaminant	The Drinking Water Standards. The Water Supply Regulations 1989	Level in minimally impaired ponds in the UK	Environment Agency River Ecosystem Class 1 limits (RE 1)	Environment Agency River Ecosystem Class 2 limits (RE2)	Environment Agency River Ecosystem Class 4 limits (RE4)	Swedish EPA limits	Urban Waste Water Treatment Directive 91/271/EEC	Minimum concentrations causing observable biological effect
Chromium	50 µg/l	No data	No data			Rivers: 0.2 µg/l Lakes: 0.05 µg/l		Cr (III): <44 µg/l (US EPA Chronic Value) Cr (VI): 2 µg/l (US EPA Chronic Value) 11.5 µg/l (Level below which 75% of European rivers fell[3])
Iron		Median: 221µg/l Mean: 836 µg/l n = 96	No data			No data		1000 µg/l
Lead		Median: 15.7µg/l Mean: 20.6 µg/l n = 96	No data			Rivers: 0.05 µg/l Lakes: 0.05 µg/l		12.26 µg/l
Mercury		No data	No data			Rivers: 0.0001 µg/l Lakes: 0.001 µg/l		0.012 µg/l
Nickel			Hardness (mg/l CaCO3) — Nickel (µg/l) annual average 0-50 — 50 >100-150 — 150 >150-250 — 200					5 µg/l
Zinc	5000 µg/l	Median: 80.1 µg/l Mean: 97.0 µg/l n = 109	Hardness (mg/l CaCO3) — Zinc (µg/l) ≤10 — 30 >10&≤50 — 200 >50&≤100 — 300 >100 — 500	30 – 500 µg/l	300 – 2000 µg/l	Rivers: 3 µg/l Lakes: 1 µg/l		30 µg/l
pH	It is impractical to provide a 'natural' baseline pH as pH varies naturally over a very wide range; thus pHs from 4.0-10.0 can be encountered in minimally impaired waters.							

[3]Stanners and Bourdeau (1995)

The Environment Agency River Ecosystem categories are described in the following table:

Table 2.3 Environment Agency River Ecosystem Categories

River Ecosystem Classification	General Quality Assessment	Likely uses and characteristics*
RE1	A - Very good	• All Abstractions • Very good salmonid fisheries • Cyprinid fisheries • Natural ecosystems
RE2	B - Good	• All abstractions • Salmonid fisheries • Cyprinid fisheries • Ecosystems at or close to natural
RE3	C – Fairly good	• Potable supply after advanced treatment • Other abstractions • Good cyprinid fisheries • Natural ecosystems, or those corresponding to good cyprinid fisheries
RE4	D – Fair	• Potable supply after advanced treatment • Other abstractions • Fair cyprinid fisheries • Impacted ecosystems
RE5	E – Poor	• Low grade abstractions for industry • Fish absent or sporadically present, vulnerable to pollution** • Impoverished ecosystems**
	F - Bad	• Very polluted rivers which may cause nuisance • Severely restricted ecosystems

* Provided other standards are met
** Where the grade is caused by discharges of organic pollution

Additional information was collated as part of this study, and a set of values against which SUDS performance could potentially be measured were then selected. This information is presented in Table 2.4.

Table 2.4 Water quality standards for a range of pollution components

Parameter	COD	BOD	TSS	NH_4-N	NOx-N	TP	PO_4-P	Cd	Pb	Cr	Cu	Ni	Zn
	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	µg/l	µg/l	µg/l	µg/l	µg/l	µg/l
Mean level in minimally impaired ponds (HR, 2003)			19.1	0.27	0.5	0.19	0.07		20.6		11.5		97
EA River Ecosystem, Class 2 limits		4		0.6							22		200
Swedish EPA limits						0.1		0.003	0.05	0.2	1		3
UK drinking water quality standards, 1989				0.5	11.3	2.2		5	50	50	3000	50	5000
Danish drinking water quality standards					11.3	0.15		5	50		100		100
Level below which 75% of European rivers fell								0.4		11.5			
Minimum concentration causing observable biological effects (HR, 2003)			25	0.0017			2200	0.15	12.3	2 - VI 44 - III	6.54		30
US, EPA, chronic values								0.66			5.3	5	
Adopted values for SUDS categorisation purposes for this study	**24**	**4**	**19.1**	**0.6**	**0.5**	**0.19**	**0.07**	**5**	**20.6**	**2(1)**	**22(2)**	**5**	**200**

Notes:

(1) For Chromium (Cr), a very low limit of 2 µg/l has been selected, which is the value for Cr (VI). However all of the studies have measured Total Cr which is likely to have other Cr components e.g. Cr (III), therefore the limit is likely to be conservative.

(2) The EA has a standard limit for dissolved copper (Cu), however the reported values are generally for Total Cu.

For this study, the values chosen to review SUDS performance data against are, wherever possible, the standards set by the UK Environment Agency (EA) for Class 2 rivers. If this standard was not available, then values that are characteristic for minimally impaired ponds in the UK were used. Where there were no UK sources, appropriate international levels were selected. As set out in the notes that accompany Table 2.4, in some cases standards may be low for the particular parameter observed, and comparisons therefore ‘conservative’.

On the basis of the effluent water quality data, an attempt was made to categorise different SUDS systems based on an overall water quality performance indicator. This indicator is expressed as a ratio of outflow pollutant concentrations to the adopted water quality standards for that parameter.

In the first instance, a parameter was only included in the overall categorisation if the SUD type had values from *at least three independent studies* for the same parameter. Thus the initial categorisation is based only on TSS, Pb and Zn. From this analysis, it was possible to highlight three potential categories of SUDS performance as follows:

Category	*SUDS component*	*Performance*
1	Permeable car park Wetland basin	Concentrations of pollutants in outflow lower than the standard limits by up to 30 %
2	Retention pond Grass swale	Concentrations of pollutants in outflow equal or close to the standard limits
3	Detention basin Wetland channel	Concentrations of pollutants in outflow higher than the standard limits by up to 30 %

In order that some biological parameters could be included in the categorisation, the limiting number of independent studies required for a given parameter was reduced to two. The parameters then included COD, NH_4-N, NO_x-N, TSS, Cd, Pb, Cu and Zn. Using this revised (and less robust) approach, it was possible to categorise the performance of systems as follows:

Category	*SUDS component*	*Performance*
1	Wetland basin	Concentrations of pollutants in outflow lower than the standard limits by up to 40 %
2	Permeable car park Retention pond Grass swale Wetland channel	Concentrations of pollutants in outflow within 20 % of standard limits
3	Detention basin	Concentrations of pollutants in outflow higher than the standard limits by up to 20 %

Typical outlet concentrations (based on 75th percentiles of observed data) are given in Table 2.5. These ranges are independent of inlet quality.

Table 2.5 Typical outlet pollutant concentrations for each performance category

Parameter	TSS mg/l	Pb mcrg/l	Zn mcrg/l
1 Category	13-22	3.6-6.9	46-146
2 Category	33-35	17-19	52-69
3 Category	42-77	8-39	43-51

The analysis shows that the permeable pavement and wetland appear to provide the most consistent water quality performance, and the detention basin shows the poorest performance levels. It is therefore likely that, where detention basins are not designed with additional water quality features, an additional downstream SUDS component should be included as part of a treatment train to minimise risks to receiving water bodies.

For most of the SUDS types, total effluent metal concentrations are strongly correlated with TSS concentrations. Therefore, if the TSS concentration is in compliance with the standard limits then the total metal concentrations also tend to comply. It is likely therefore that TSS could be adopted as a reliable parameter for review of likely loads of total metals within the outflow.

The analysis does show that more attention should be paid to the measurement of biological pollutants. They are of particular importance as, in addition to being present in stormwater effluent, they may also be generated within the sustainable urban drainage systems themselves. Reductions of nutrient loadings are also critical for a range of diffuse pollution management initiatives currently being driven by the Water Framework Directive.

The water quality performance of SUDS systems that include permanent water volumes, during dry weather has not been discussed in this literature review as quantitative data on this issue is limited. Several literature sources do however report increasing concentrations of biological parameters during dry periods. Depending on the size of the permanent volume and the characteristics of the influent pollutant concentrations, these may or may not have a significant impact on the receiving water.

2.5 DESIGN CHARACTERISTICS

Wherever it was possible, the influence of design parameters on water quality performance was investigated. This was extremely difficult due to poor data and a lack of reporting of design parameters for many of the study sites. However, some analysis has been undertaken by some authors on the influence of swale length, retention pond area/watershed area and design volume on water quality performance. The following conclusions reflect their findings:

- There is a tendency for higher outlet concentrations of pollutants to be observed for systems where the size of retention pond area is small when compared to the contributing impervious watershed area. Above 1.5%, there is no discernible trend.
- Due to the paucity of data, it was not possible to draw definitive conclusions from the analysis of the influence of permanent pond volume on effluent concentrations. However, the graphs may indicate that:
    1. where storms are smaller than the permanent pond volume, the effluent concentrations tend to lower values (this trend being most expressive for TSS);
    2. where the permanent pond volume is larger than the size of the storm, there appears to be little significant influence;
    3. for larger ponds, there are always a few data points with high pollutant concentrations. This may indicate internal pond processes causing poor water quality after long retention periods (possibly during dry weather). This issue was considered important, and some of the analysis of data monitored during subsequent studies was focused on this issue. In order to investigate the

likely influence of climate on retention times in systems with permanent water bodies, a separate hydrological study was undertaken. This is reported on in Chapter 3.

- Parameters accepted as likely to influence the water quality performance of swales are: length, flow velocities, water depth, longitudinal slope, hydraulic retention time and season. The literature review found that length appeared to positively influence the removal rates of TSS and heavy metals (with the exception of Pb), while concentrations of soluble pollutants (i.e. nutrients) tended to increase with length. The removal efficiency of TSS, organics and most metals decreased with increasing water depth, and were optimised at retention times of at least 9 minutes.

2.6 CONCLUSIONS

Conclusions drawn from the literature review allowed a more targeted approach for the monitoring campaigns and data analysis work implemented as part of the project. The review also provided a baseline statement of existing performance for this new work to be compared against. In particular, the review developed an understanding of the benefits of a water quality monitoring campaign based on a programme of grab samples and comparison with environmental water quality standards, rather than automatic sampling of inlet and outlet concentrations during a few events in order to determine individual event removal efficiencies.

3. Hydrologic investigation of the influence of climate on hydraulic residence time

3.1 INTRODUCTION

The potential impact (either positive or negative) of the length of time that inflows from the upstream drainage system are likely to be retained within the permanent water body (residence time) was considered to be a key issue, and one that some investigation within this study should be focussed on.

Therefore, to supplement measurements taken during the monitoring studies, preparatory hydrological analyses were undertaken to review the potential variability of likely residence times of water in retention ponds depending on both the size of the permanent pool and the local rainfall characteristics.

3.2 APPROACH

In order to try to assess the influence of climatic factors on the residence time of the ponds, rainfall data was collected from three sites across the UK: one in central Scotland (Anfield), one in southern England (Poole), and one in central England (Hopwood).

The proportion of exchange of the permanent pond volume per day (F_p,i) was defined as:

$$\frac{Ri}{Vp} = Fp, i$$

where:

R_i is the rainfall during the day, mm
V_p is the permanent volume of the pond, expressed as mm rainfall over the served area.

Most of the rainfall events were significantly smaller than V_p, therefore *Fp,i* was usually less than 1. If one makes the simplified assumption of plug flow then, depending on the frequency and the depth of the rainfall together with the size of the permanent pool, the water will be exchanged gradually until the total permanent volume has been displaced by fresh water. This interval (in days) is defined as the *Residence Time*.

A simple assumption of 100 % rainfall-runoff was adopted, with an 0.8 mm rainfall loss included at the start of a storm (this reflected the depth of rainfall prior to runoff observed at Hopwood). In practice, a lower percentage runoff is likely, so actual residence times may be higher.

The 5-day Antecedent Precipitation Index (API_5) was used as an indicator of the climate-influenced environmental conditions of the ponds prior to an 'effluent discharge' event from the pond. It is based on the level of precipitation falling on the catchment during the preceding 5 days and is calculated according to the following equation:

$$\mathrm{API}_{5,i} = \sqrt{0.5} * (\mathrm{R}_{i-1} + 0.5 * \mathrm{R}_{i-2} + 0.5^2 * \mathrm{R}_{i-3} + 0.5^3 * \mathrm{R}_{i-4} + 0.5^4 * \mathrm{R}_{i-5})$$

where:

$API_{5,i}$ is the API_5 at the start of day i
R_{i-1}, R_{i-2}, R_{i-3}, R_{i-4} and R_{i-5} are the rainfall depths during days i-1, i-2, i-3, i-4 and i-5 respectively

A high API_5 would therefore indicate a high level of rainfall prior to the event i.e. a wet pre-event catchment. A low API_5 would indicate a period of dry weather prior to the event.

3.3 RESULTS

Table 3.1 summarises some of the key characteristics of the precipitation at the three study sites.

Table 3.1 Statistical characteristics of annual rainfall for 3 sites for the period 1999-2003

Site	Average Annual rainfall	Average storm event	90^{th} %-ile storm event	Average rainy days per year
	mm	mm	mm	nr
Anfield	866	3.43	9.0	253
Hopwood	793	4.5	11.5	176
Poole	941	5.19	13.9	181

Figure 3.1 presents the distribution of API_5 at each case study site for the driest 3-month period and the wettest 3-month period during the year. Anfield in Scotland is characterised by relatively small but regular rain events, which is reflected in the smaller values of the average storm depth and 90^{th} percentile rainfall and a very small change in seasonal distribution of API_5. Poole in southern England has significant seasonal variations with long periods of dry weather and short heavy storms. Consequently, the size of the average and 90^{th} percentile storm event depths are larger than for the other sites and the API_5 also shows significant seasonal change. The rainfall at Hopwood also demonstrates seasonal variability, although not as strongly as at Poole.

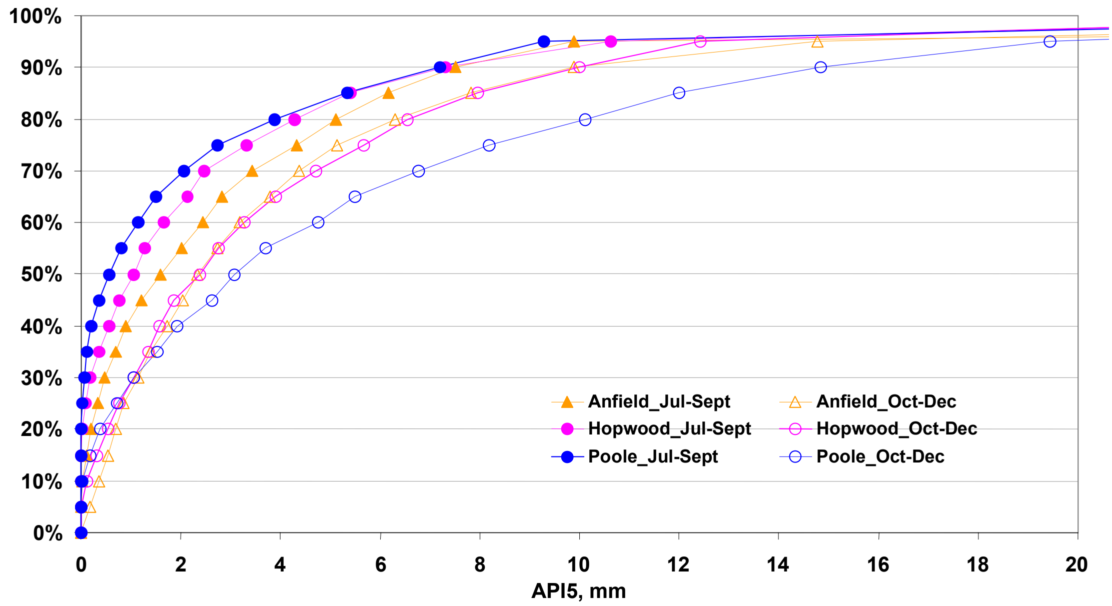


Figure 3.1 Comparison of the seasonal distribution of API_5 for the three case study sites

Figure 3.2 presents how the seasonal changes are reflected in the residence time of water in the ponds. For consistency, the permanent pond volume at all sites was assumed to be equal to four times the 90^{th} %-ile daily rainfall depth ($Vp=4xR_{90\%}$).

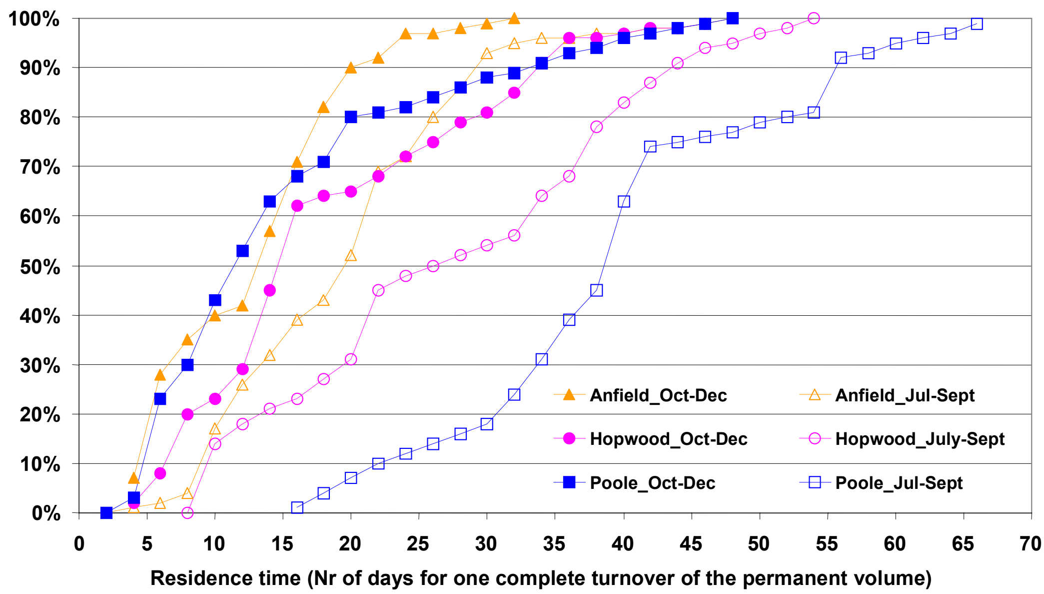


Figure 3.2 Seasonal distribution of the residence time at the three case study sites

As expected, the residence time at Anfield shows the smallest variation through the year, while the residence time at Poole show the biggest variability, with periods for complete permanent volume exchange reaching up to 66 days during summer months. Since the residence time is also a function of the permanent volume, Figure 3.3 indicates how the size of the permanent volume would influence the residence times at Hopwood. For this sensitivity test, the permanent volume has been calculated in three ways:

1. $Vp=R_{av}$ (equal to the average annual storm event, mm)
2. $Vp = R_{90\%}$ (equal to the 90th percentile storm event, mm)
3. $Vp = 4 \times R_{90\%}$ (equal to 4 times the 90th percentile storm event, mm)

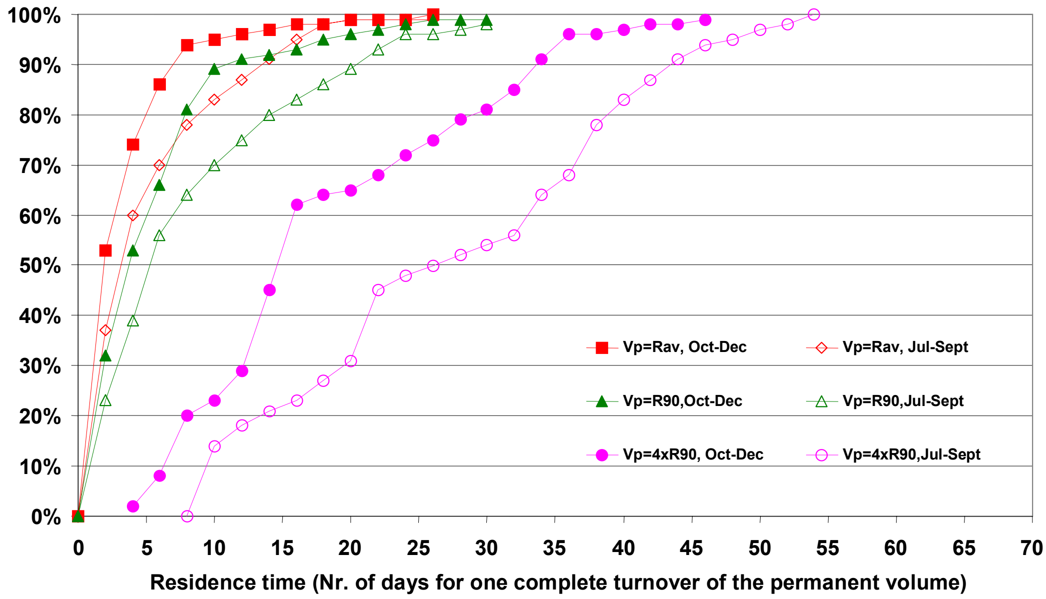


Figure 3.3 Seasonal distribution of residence time for different sizes of Vp at Hopwood

3.4 CONCLUSIONS

This hydrological analysis shows that the climate in some parts of the country (e.g. Anfield, Scotland) is characterised by relatively small but regular rain events, reflected in small average storm depths and 90th percentile rainfall, a large number of rainy days, and a relatively constant measure of catchment antecedent wetness. These characteristics tend to imply much shorter residence times for drainage inflows in the permanent pond than for locations in the south of England where long periods of dry weather and short, heavy storms are more common and residence times for large ponds could reach over 50 days. This could imply significantly different pond performance for certain parameters within the effluent.

4. Monitoring site selection

4.1 INVESTIGATION OF POTENTIAL MONITORING SITES

In agreement with the Project Steering Group (PSG), the selection of the systems to be monitored was to be based on the following criteria:

1. Ability to monitor at points in the system that will provide useful performance datasets;
2. Systems with commonly used components in order that the performance data is useful to a wide range of designers and regulators;
3. Systems where operators carry out maintenance operations; and
4. Systems where stakeholders are willing to participate in the research.

Potential sites were therefore identified in collaboration with members of the PSG. Baseline data for eight systems was obtained and site visits were undertaken to assess their suitability for monitoring. The decision-making process is summarised in Table 4.1. Additional considerations were:

- Bradford University were considered important project collaborators in terms of providing resources to monitor at one of the sites, and the location of this site was therefore constrained by distance from the university and ongoing project work being undertaken for Leeds City Council.

- The Environment Agency were already engaged in a water quality monitoring programme at Hopwood Services. This programme was due to end, but the results of which would be useful to enhance and inform any potential monitoring programme at the site.

- Formpave were a major project funder, and it was felt appropriate that a Formpave permeable block pavement should be monitored. Considerable effort was made to secure Formpave sites that fulfilled the following criteria:
  a. Lined system, in order to facilitate outlet monitoring;
  b. Systems that were likely to receive regular maintenance;
  c. Systems within reasonable proximity to HR Wallingford.

  Investigations and negotiations were initiated at over ten permeable pavement sites. However, the only site where site owners and operators could facilitate and fully cooperate with a monitoring programme was at the Tesco supermarket store site in Kidderminster.

Table 4.1 Site selection details

Ref	Site Name	Designers	Developers	Adopting Authority	Construction Date	Scheme Description	Photos		Benefits / Positive Issues	Risks / Negative Issues	Decision
1	Newhall Valley Water Management Strategy	Babtie Group	Consortium (Bovis, Bryant, Lovell, Rubery Owen)	Birmingham City Council	1999/2000	Development runoff passes through oil separator, via hydrobrake (with upstream, off-line containment storage of modular crate construction) into long swale that follows hillside contours to descend to receiving watercourse	  	Overflow from Hydrobrake chamber into off-line crate storage View of swale flowing down hillside View of swale outfall into receiving watercourse	• Performance data for swales (currently limited) • Knowledge of project partner with respect to design and operation • Support of project partner (Babties) in data collection	• Security of monitoring equipment • Runoff pre-treated by oil separator • Flow management largely achieved using conventional drainage (sub-surface storage) solution	✖
2	Hopwood Services: Main Car Park Treatment Train	Robert Bray Associates / Glanvilles	Welcome Break	Welcome Break	1998	Car park runoff is collected via slotted kerbs into sub-surface, gravel-filled collector trench. This passes through a balancing pond prior to discharge to the receiving watercourse.	 	Collector slotted kerbs and balancing pond	• Performance data for filter trench and retention pond	• Difficulties in estimating runoff to each trench • Depth of trench creates monitoring risks • Bypass channel masks water quality performance	✖
3	Hopwood Services: HGV Park Treatment Train	Robert Bray Associates / Glanvilles				Lateral runoff from the lorry park discharges across a grass filter strip into a filter trench. The outfall is collected and treated in a series of wetlands/ponds.	   	Filter strip and filter drain Wetland / pond treatment train	• Performance data for filter strip, filter trench and wetland/ pond systems	• Difficulties with estimating system inflows (quantity and quality) • Bypassing of filter drain outfall	✖

Table 4.1 Site selection details (continued)

Ref	Site Name	Designers	Developers	Adopting Authority	Construction Date	Scheme Description	Photos	Benefits / Positive Issues	Risks / Negative Issues	Decision
4	Hopwood Services: Coach Park Treatment Train	Robert Bray Associates / Glanvilles				Runoff is collected via a conventional gully and pipe system and discharged into a wetland / pond / wet swale treatment train. A decelerator was included to provide silt removal upstream of the SUDS train.	  Upper and lower sections of the treatment train	• Performance data for vegetated ponds and wetland swale • Support of project partner (Welcome Break)	• Distance from HR Wallingford • Upstream silt separator provides conventional pre-treatment • Rubbish collection area discharges into mid-train	✓
5	Tescos, Kidderminster Permeable Pavement (Formpave) Site	White, Young, Green	Tesco	Tesco	2002	Standard block pavement system draining car park for retail outlet. Asphalt road network which drains to permeable pavement. Lined system to prevent mobilisation of sub-surface contamination. Single outfall point to receiving watercourse.	 Block pavement	• Tanked system facilitates monitoring of outfall quality and quantity • High vehicle turnover	• Distance from HR Wallingford • Manhole depth requires confined spaces qualification and equipment	✓
6	Leeds County Council Refuse Site	W S Atkins	Leeds County Council	Leeds County Council	Not known	Swale discharging to balancing pond		• Swale and pond performance	• Risks of vandalism • Oil separator • Poor design	✖
7	Thorpe Business Park, Leeds: Retention Pond	WSP/HJT consultants	GMI Group	Severn Trent Property / GMI Rovinian Ltd	1999	Balancing pond draining a new commercial development and upstream semi-urban area. 2 inlets and a single outlet.	 Balancing pond outfall	• Large-scale pond performance data (to contrast to Hopwood) • Project partner (Bradford University) to provide staff to undertake monitoring	• Pond is on-line (one of the inlets has a stream baseflow component) • Proximity to Bradford	✓
8	Newbury Bypass Soakaways	Mott MacDonald	Highways Agency	Highways Agency	1998	A series of linked soakaways draining runoff from the A34 Newbury Bypass.	None available	• Performance data on operational infiltration systems • Support of project partner (Mott Macdonald)	• Limited value of data at one site • Performance dependent on soil characteristics, construction methods, location, age • Seepage water quality difficult to monitor	✖
9	Tescos, Wokingham Permeable Pavement site		Tesco	Tesco		Pervious asphalt surface above underground gravel-filled reservoir structure, draining car park for retail store. Asphalt road network which drains to permeable pavement. Lined system to prevent mobilisation of sub-surface contamination. Single outfall point to small wetland area.		• Monitoring programme already ongoing – no data collation cost • Support of Environment Agency	• Outfall monitoring point includes some direct road runoff	✓

4.2 SITE SELECTION

The selected sites were as follows:

Site 1: SUDS Treatment Train at Hopwood Park Services

Hopwood Park Services was agreed by the PSG to be a good site at which to monitor due to the wide range of potential SUDS components available, and the positive support provided by the site owners and operators. However, the gravel treatment trench originally identified as suitable for monitoring had to be dropped due to the extensive bypassing of water observed during events resulting in inaccuracies in the monitoring. The carpark treatment train comprised deep manholes, many of which were found to contain high volumes of silt and therefore these also had to be discounted. The coach park pond sequence was selected which allowed monitoring of the train of ponds, and a wetland swale.

Site 2: Permeable Pavement Formpave Carpark, Tesco store, Kidderminster

This is a lined, permeable pavement site which was under construction during the site appraisal discussions. This allowed monitoring equipment to be installed at the site during the handover period from contractor to owner/operator.

Site 3: Retention Pond at Thorpe Park Business Park, Leeds

This pond site was of interest to both the Environment Agency and Bradford University (responsible for monitoring one site for this project), and had a high profile in the local region. HR Wallingford assisted with provision and installation of equipment.

Site 4: Permeable Pavement Carpark, Tesco store, Wokingham

The Environment Agency had been monitoring the porous asphalt pavement site at the Tesco site in Wokingham from January 2000, however no analytical work or interpretation of the data had been undertaken. The Environment Agency therefore provided HR Wallingford with both rainfall and flow data from that time until June 2004, when the monitoring was discontinued. This provided an important dataset on which analysis work could be undertaken.

5. SUDS Treatment Train at Hopwood Park Services

5.1 SITE DESCRIPTION

The Hopwood Park Motorway Service Area is located off the M42 at Junction 2 near Bromsgrove in Worcestershire. It was designed by Bob Bray of Robert Bray Associates and uses the treatment train philosophy to minimise the risk of pollution to the receiving environment and to 'attempt to replicate natural drainage processes', with minimal impact on the pre-development hydrology. The site comprises 34 hectares, of which 9 ha are developed as Motorway Service Area and 25 hectares is maintained as Wildlife Reserve.

The SUDS treatment train that was investigated (Figure 5.1), treats the run-off from the main access road, the fuel filling area and the coach park area (1.4 hectares in total). Silt is initially removed within a proprietary vortex sediment control unit. Interceptor ponds, with wetland treatment zones and outlet valves are included to facilitate the isolation of any spillage events and to trap fine silt. The constructed wetland allows final settlement and bacterial breakdown. The long ditch / swale, with emergent vegetation, is designed to intercept minor organic pollution, continuing the natural breakdown of pollutants and delivering the runoff to the balancing pond. The balancing pond again has a wetland bench to 'polish' any remaining organic residues and is the final stage in the treatment train. Photos of the treatment train are given in Figure 5.2.

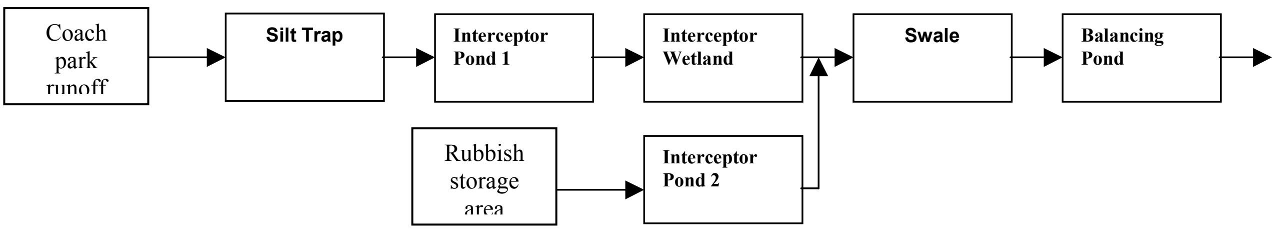


Figure 5.1 Scheme of the Coach Park treatment train at Hopwood

Figure 5.2 Photo of the Hopwood Coach Park Treatment Train (looking upstream from the inlet to the swale)

In winter, the vegetation coverage of the pond surface is of the order of 10 %, however in summer this increases to 70-80 %. The ponds are vegetated with dense emergent vegetation including typha angustifolia (cattail), and Iris pseudacorus (yellow iris). The wetland swale has a wide variety of grasses and a predominance of water mint.

Table 5.1 gives the storage capacities of each of the sustainable urban drainage units in the studied treatment train, calculated from measurements and design drawings. They are only approximate, and indicative for the purposes of this study, and may not be completely consistent with design calculations.

Table 5.1 Calculated storage capacities of each treatment train unit

SUDS Unit	Surface Area of the Permanent Pool	Volume		Volume (in mm rainfall over the drained area)	
		Attenuation	Permanent	Attenuation	Permanent
	m^2	m^3	m^3	mm	mm
Interceptor Ponds 1 & 2	208	61	11.5	4.8	1.5
Interceptor Wetland	333	80	35	10.5	4.1
Swale	160	24	11.2	12.6	5.0
Balancing pond	500	252	66.1	30.5	10.0

The capacities have been calculated for a drained area of 14,000 m^2 and assume that 0.8 mm depth of rainfall is lost upstream of the silt trap. Evaporation and evapotranspiration have not been taken into account.

5.2 MONITORING STRATEGY

An extensive programme of monitoring was implemented at Hopwood between March 2002 and July 2004. The monitoring objectives for the site were:

1. To assess hydraulic and water quality performance of the treatment train;
2. To determine influence of season, time and maintenance on performance.

These objectives were to be fulfilled through the following activities:

- Monitoring rainfall;
- Monitoring flow rates;
- Monitoring water quality;
- Monitoring sediment quality.

The hydraulic performance of the treatment train was evaluated through continuous monitoring of on-site rainfall, together with flows at three points in the treatment train:

1. The outlet from the Silt Trap (ST);
2. The inlet to the Wetland Swale (IP);
3. The outlet of the Wetland Swale (WS).

The locations of the monitoring points are shown on a site plan in Figure 5.3.

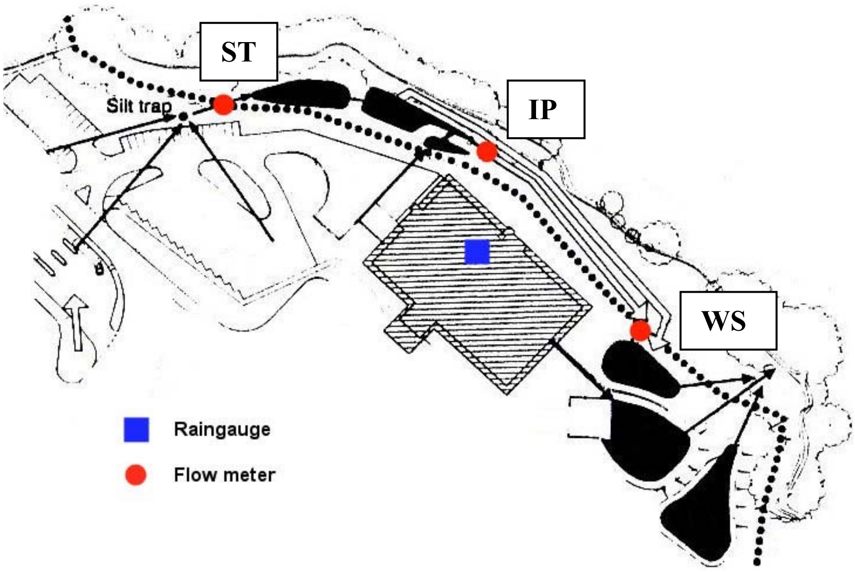


Figure 5.3 Hopwood Monitoring Site Plan

One of the original project aims was to collect 'event mean concentrations' for water quality, requiring automatic sampling through an event. An automatic sampler was installed and tested, however low system flows and high flow variability meant that this approach had to be abandoned.

The literature review (Chapter 2 and Appendix 1) and initial hydrologic analysis (Chapter 3) highlighted the potential benefits of a water quality monitoring campaign based on a programme of grab samples, and this approach was therefore taken forward for the remainder of the project period.

The Environment Agency also co-ordinated sediment sampling, which was undertaken when the ponds were de-silted during October 2003. This data was analysed by a student from Swansea University. Unfortunately, however, the results of this work have not been made available for inclusion within this study.

5.3 MONITORING EQUIPMENT

As there was no suitable existing raingauge close to the site, a 0.2 mm tipping bucket rain gauge was installed to monitor rainfall. It was located on the roof of the Welcome Break retail outlet, within the service area. Figure 5.4 shows the raingauge in its installed position and Figure 5.5 shows the data downloading process.

Figure 5.4 Raingauge on the roof at Hopwood Park Services

Figure 5.5 Data download from Hopwood raingauge

Buehler Montec Xytec 7050 flow loggers were used for measuring flows through the system. These were programmed to take measurements every 2 minutes. These loggers allow measurement of pipe flow using pressure transducers to measure head and ultrasonic sensors to measure velocity. Low metal weirs were installed behind the transducer heads to maintain a minimum water depth of 40 mm, which is recommended by the manufacturer for optimum operational conditions.

Figure 5.6 shows an installed flow logger and Figure 5.7 shows the associated data collation equipment housing.

Figure 5.6 Installed Flow Logger at Coach Park Treatment Train

Figure 5.7 Flow Measurement Data Collation Equipment and Housing

Water quality samples were collected in bottles and analysed by Severn Trent Chemical Laboratories.

5.4 MONITORING PROGRAMME

Flow depth and flow velocity were plotted for each flow data set and compared with the calibration plot provided by the manufacturer. This comparison was necessary to detect potential problems that may have occurred during the period of monitoring, and to gauge the reliability of the dataset.

The most common problems were:

(1) Accumulation of silt at the bottom of the pipes, especially the silt trap pipe, causing clogging of the transducer head and unreliable data or lack of data for flow velocity;
(2) Accumulation of dead foliage in front of the pipe grille, obstructing free run-off from the ponds;
(3) Failure in memory capacity of the flow loggers.

The first two issues demanded regular cleaning of the transducer head. Figure 5.8 shows the state of the pipes after a period of sediment accumulation. This can be compared to Figure 5.9 where the transducer is visible.

Figure 5.8 State of flow loggers following a period of sediment build-up

Figure 5.9 Flow loggers following sediment clearance

Figures 5.10 and 5.11 show the depth velocity plots for the monitoring equipment, before and after cleaning. The scatter in the depth-velocity relationship in Figure 5.10 demonstrates the detrimental impact of sediment on the quality of the processed data.

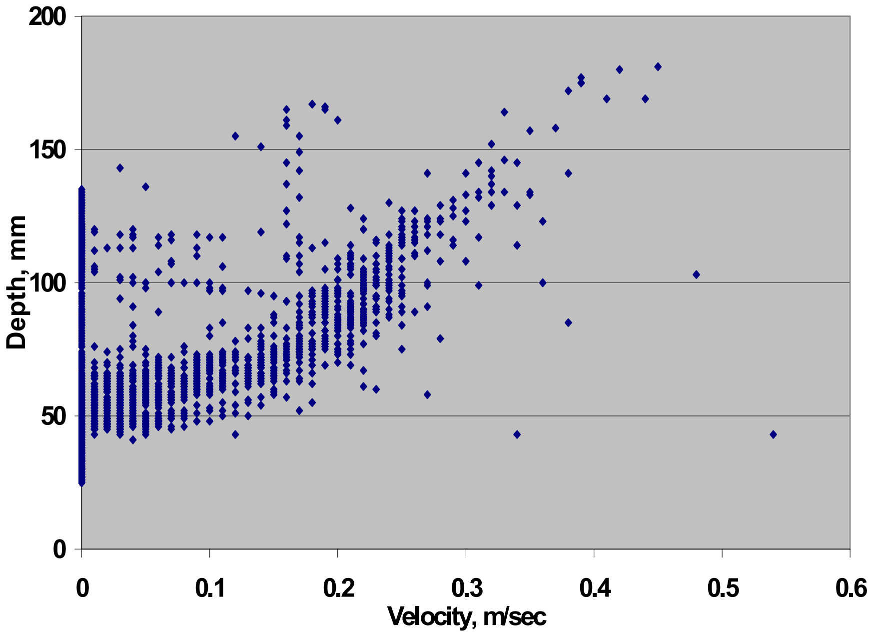


Figure 5.10 Depth / Velocity plot for flow monitor prior to cleaning

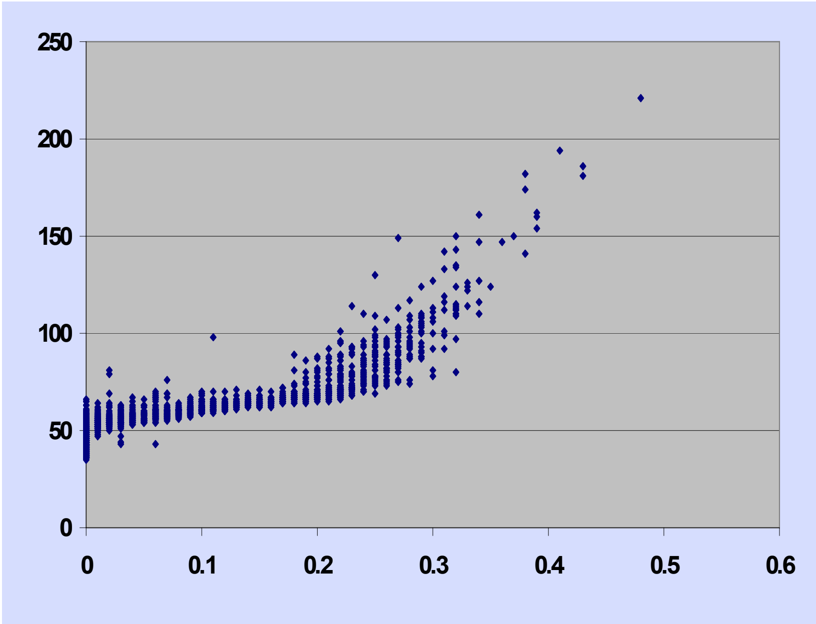


Figure 5.11 Depth / Velocity plot for flow monitor after cleaning

Some other technical failures required re-calibration of the equipment in laboratory conditions, requiring removal of the equipment from site.

Table 5.2 summarises all the collected data and gives a short description of data availability and the technical failures experienced.

Table 5.2 Hydrologic Monitoring Programme Data Availability: Hopwood Services

Period	Rain gauge	Silt Trap (ST)	Interceptor Pond (IP)	Wetland Swale (WS)
21.05.02-28.06.02	Reliable data	Reliable data	Reliable data	Reliable data
29.06.02-6.09.02	Reliable data	Not reliable data *(sediment in the pipe)*	Reliable data	Reliable data
7.09.02-11.09.02	No data *(memory capacity)*	No data *(memory capacity)*	No data *(memory capacity)*	No data *(memory capacity)*
12.09.02-12.10.02	Reliable Data	Reliable Data	Reliable Data	Reliable Data
19.10.02-4.01.02	Reliable Data	Reliable Data	Not all data are reliable *(sediment in the pipe)*	Reliable Data
05.01.02-10.01.02	Reliable Data	No data *(memory capacity)*	No data *(memory capacity)*	No data *(memory capacity)*
10.01.02-06.03.02	No data	No data	No data	No data
07.03.03-08.05.03	Reliable Data	Reliable Data	No data *(technical failure)*	Reliable Data
09.05.03-28.05.03	Reliable data	Reliable Data	Reliable Data	Reliable Data
29.05.03-03.06.03	No data *(memory capacity)*	No data *(memory capacity)*	No data *(memory capacity)*	No data *(memory capacity)*
04.06.03-19.06.03	No data	Reliable data	Reliable data	Reliable data
20.06.03-19.08.03	Reliable data	Reliable data	Reliable data	Reliable data
20.08.03-15.09.03	Reliable data	Not reliable data *(sediment in the pipe)*	Reliable data	Reliable data

Table 5.2 Hydrologic Monitoring Programme Data Availability: Hopwood Services (continued)

Period	**Rain gauge**	**Silt Trap (ST)**	**Interceptor Pond (IP)**	**Wetland Swale (WS)**
16.09.03-7.10.03	Reliable data	No data *(sediment in the pipe)*	No data	Reliable data
8.10.03–27.10.03	No data	Reliable data	No data	Reliable data
28.10.03-19.11.03	Not all data are reliable (funnel clogged by algae)	Not all data are reliable *(sediment in the pipe)*	Reliable data	Reliable data
20.11.03-15.12.03	Not all data are reliable (funnel clogged by algae)	Reliable data	Reliable data	Reliable data
16.12.03-05.01.04	Not all data are reliable *(funnel clogged by algae)*	No data (sediment in the pipe)	Reliable data	Reliable data
7.01.04-8.01.04	Not all data are reliable *(funnel clogged by algae)*	No data (memory capacity)	No data (memory capacity)	No data (memory capacity)
9.01.04-26.01.04	Reliable data	Reliable data	Reliable data	Reliable data
27.01.04-16.02.04	Reliable data	Not reliable data (*sediment in the pipe)*	Reliable data	Reliable data
17.02.04-9.03.04	Reliable data	Reliable data	Reliable data	Reliable data
10.03.04-15.03.04	Reliable data	No data *(memory capacity)*	No data *(memory capacity)*	No data *(memory capacity)*
16.03.04-06.04.04	Reliable data	Not all data are reliable (*sediment in the pipe)*	Reliable data	Not all data are reliable
7.04.04-27.04.04	Reliable data	Reliable data	Reliable data	Reliable data
28.04.04-11.05.04	Reliable data	Reliable data	Reliable data	Not all data are reliable
12.05.04-02.06.04	Reliable data	Reliable data	Reliable data	Reliable data
03.06.04-23.06.04	Reliable data	Reliable data	Reliable data	Reliable data

This information is summarised in Figure 5.12, below.

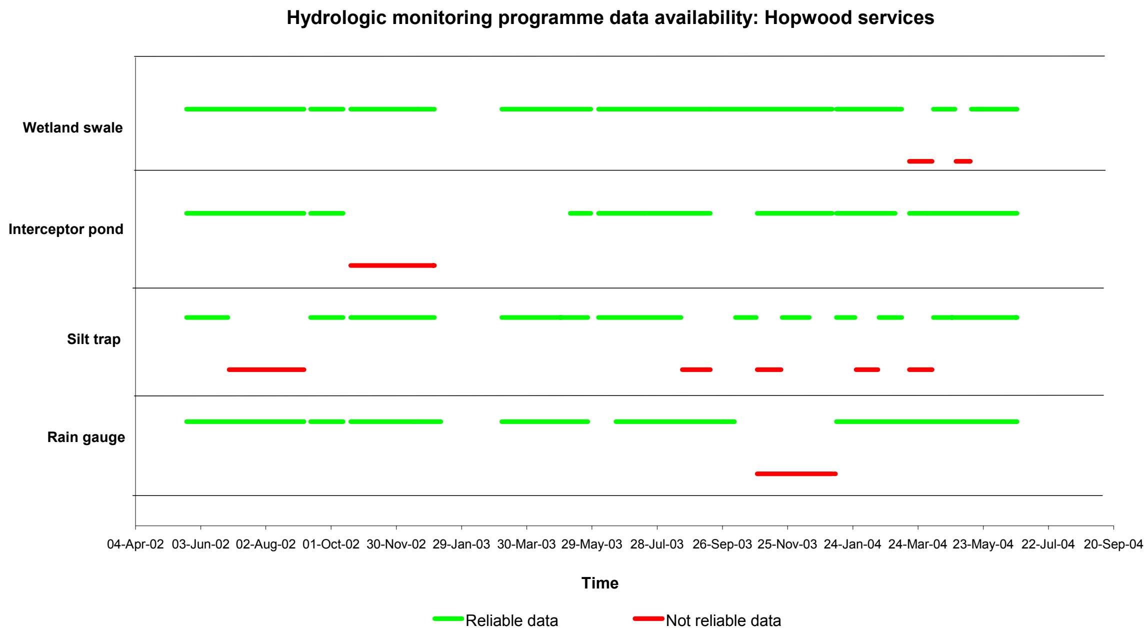


Figure 5.12 Hopwood Data Availability

The HR Wallingford water quality programme began in May 2003. Grab samples were taken from the outlets of all the units of the treatment trains at intervals of about 20 days. The choice of the monitoring sites and selection of the chemical parameters were co-ordinated with the data already available from the Environment Agency. The water quality performance of the Hopwood SUDS system had been monitored by the Environment Agency for more than two years, starting in February 2000. Grab samples (13 in total) were taken when flow through the system was taking place (i.e. either during a rainfall event or shortly afterwards). The data from the EA were analysed together with the data from HR Wallingford's own monitoring programme.

Both the EA and HR Wallingford monitored the pollutants: NH_4-N (ammoniacal nitrogen), NOx-N (oxidised nitrogen), BOD_5 (5 day biological oxygen demand), TSS (total suspended solids), Cu (copper), Zn (zinc), Pb (lead) and Ni (nickel) and TPH (total petroleum hydrocarbon). Additionally, the EA also monitored DOC (dissolved organic carbon), Cd (cadmium), Cr (chromium), Fe (iron) and Cl (chloride). From 30/07/03 HR Wallingford also monitored NO_x-N (oxidised nitrogen) and PO_4-P to try and get an improved picture of nutrient processes. During dry weather, when the water depth at the outlet was too low for effective sampling, the samples were taken from the downstream end of the permanent pool.

The sampling dates and the monitored parameters are shown in Table 5.3.

Table 5.3 Water Quality Sampling Programme: Hopwood Services

Date	SS	BOD5	NH4-N	NOx-N	PO4-P	Cu, total	Cu, dis	Zn, total	Zn, diss	Pb, total	Pb, diss	Ni, total	Ni, diss	TPH	Weather
08/02/00	√	√	√			√		√		√		√	√		
17/08/00	√	√	√			√	√	√	√	√	√				
13/11/00	√		√			√	√	√	√	√	√	√	√		
30/01/01	√	√	√			√	√	√	√	√	√	√	√		
24/04/01	√	√	√	√		√	√	√	√	√	√	√	√		
12/11/01	√	√	√	√		√	√	√	√	√	√	√	√		
04/12/01	√	√	√	√		√	√	√	√	√	√	√	√		
20/02/02	√	√	√	√		√	√	√	√	√	√	√	√		
20/03/02	√	√	√	√		√	√	√	√	√	√	√	√		
01/05/02	√	√	√	√		√	√	√	√	√	√	√	√		
15/10/02	√	√	√			√		√		√				√	wet
23/10/02	√	√	√	√		√	√	√	√	√	√	√	√		
26/11/02	√	√	√	√		√	√	√	√	√	√	√	√		
10/03/03	√	√	√	√		√	√	√	√	√	√	√	√		
08/05/03	√	√	√			√		√		√				√	dry
21/05/03	√	√	√	√		√	√	√	√	√	√	√	√		
19/06/03	√	√	√			√		√		√		√	√	√	dry
30/07/03	√	√	√	√	√	√		√	√			√	√	√	dry
27/08/03	√	√	√	√	√	√	√	√	√	√	√	√	√	√	dry
15/09/03	√	√	√	√	√	√	√	√	√	√	√	√	√	√	dry
27/10/03	√	√	√	√	√	√	√	√	√	√	√	√	√	√	dry
17/11/03	√	√	√	√	√	√	√	√	√	√	√	√	√	√	wet
15/12/03	√	√	√	√	√	√	√	√	√	√	√	√	√	√	dry
08/01/04	√	√	√	√	√	√	√	√	√	√	√	√	√	√	wet
26/01/04	√	√	√	√	√	√	√	√	√	√	√			√	snowy
16/02/04	√	√	√	√	√	√	√	√	√	√	√	√	√	√	dry
15/03/04	√	√	√	√	√	√	√	√	√	√	√	√	√	√	dry
06/04/04	√	√	√	√	√	√		√		√					wet
08/06/04	√	√	√	√	√	√		√		√					dry

5.5 DATA ANALYSIS AND INTERPRETATION: HYDRAULICS

5.5.1 Analysis objectives

The hydraulic analyses at Hopwood were aimed at gaining more practical knowledge of the following issues:

1. Actual hydraulic performance of the SUDS compared to design criteria in terms of:
   - The decrease in peak flows;
   - The water velocities in the swale at peak flows;
   - The retention time in the SUDS at peak flows.

2. The design parameters that influence hydraulic performance, including:
   - The influence of vegetation on peak flows and lag times;
   - The influence of rainfall quantity and antecedent conditions on the retention capacity of the ponds;

3. Recommendations for the design and operation of SUDS.

5.5.2 Approach

As discussed in Section 5.4, the flow data was not always reliable and some portions of the dataset had to be discarded. The series was split into representative 'flow events' for analysis. A representative 'event' passing through the treatment train is shown in Figure 5.13. Cumulative rainfall and runoff volumes for the same event are presented in Figure 5.14.

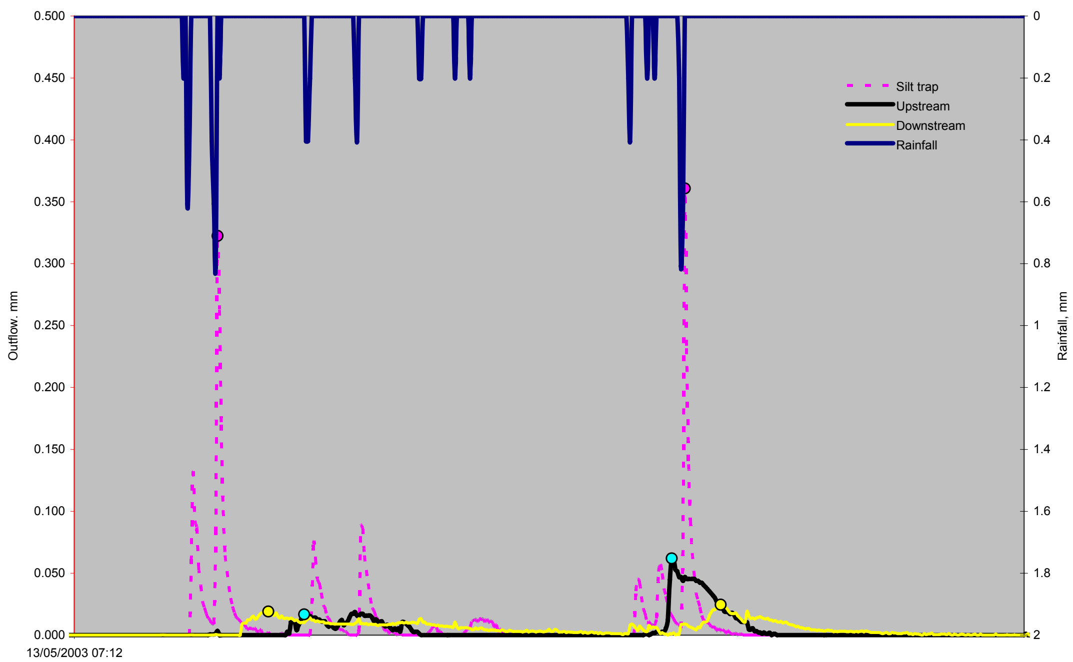


Figure 5.13 Event of 13th May 2003 through Coach Park Treatment Train

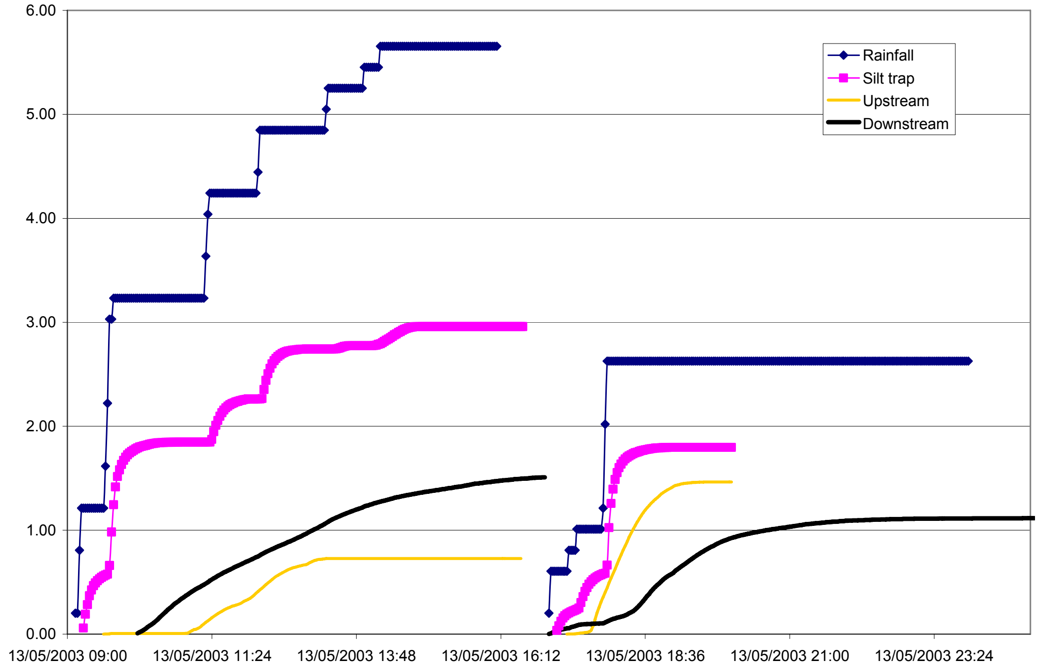


Figure 5.14 Cumulative rainfall / runoff depths for consecutive events on 13th May 2003

The recommendations of the literature review (Chapter 2) directed this study at an analysis of absolute peak flows and flow attenuation, rather than concentrating on percentage runoff, which was felt to be too dependent on antecedent conditions and rainfall intensity.

5.5.3 Reduction in peak flows

A key aim in the design of sustainable drainage systems is to mitigate for the effects of the development by providing sufficient attenuation that the site discharge should not be greater than the equivalent greenfield run-off with the same probability of occurrence.

Figure 5.15 presents the distribution of peak flows at the inlet of the silt trap, the outlet of the ponds, and outlet of the swale. This shows that although 70 % of flows at the outfall of the conventional drainage network are greater than the 2 year greenfield threshold, this is exceeded by only 30 % of events downstream of the two ponds, and only 5 % of events at the outfall of the swale (equivalent to 2 to 3 exceedances per year). It should be noted that this is upstream of the balancing pond where further attenuation will be achieved.

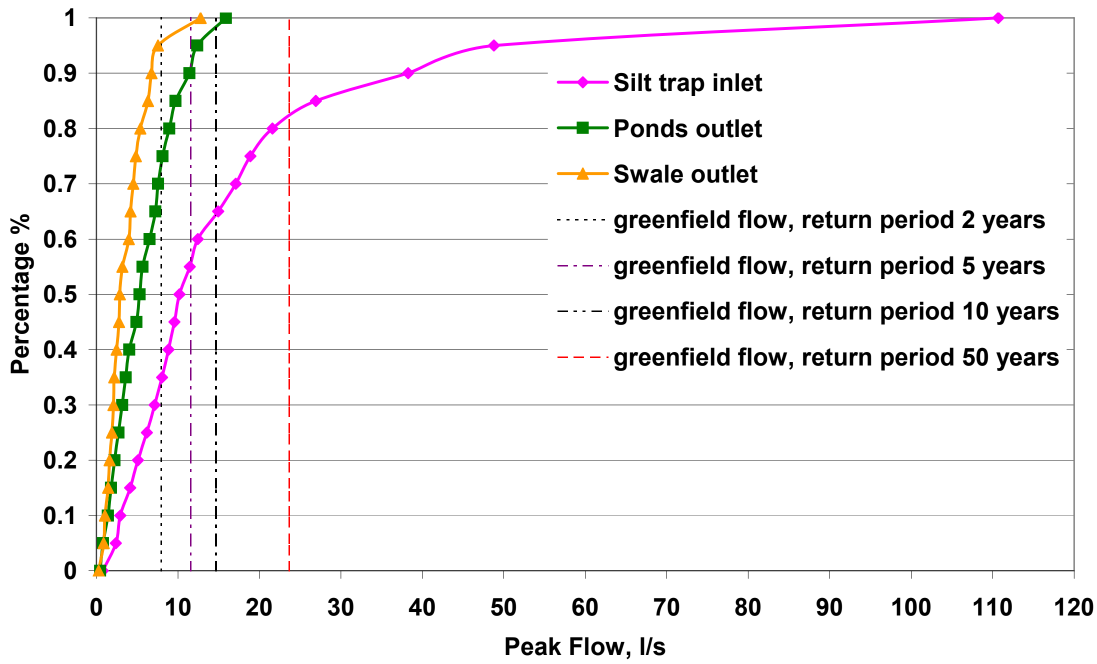


Figure 5.15 Distribution of peak flows

5.5.4 Water velocities in the swale at peak flows

Figure 5.16 presents the distribution of flow velocities in the swale at peak flows, assuming that the water depth is of the order of 0.10 m. It appears that all the velocities are less than the design limit of 0.4 m/s for flow conveyance and 0.3 m/s for water treatment purposes. This means that the hydraulic conditions are likely to favour the removal of the particulate matter from the water through sedimentation and trapping within the vegetation. The design slope of the swale is 1 in 200 m/m.

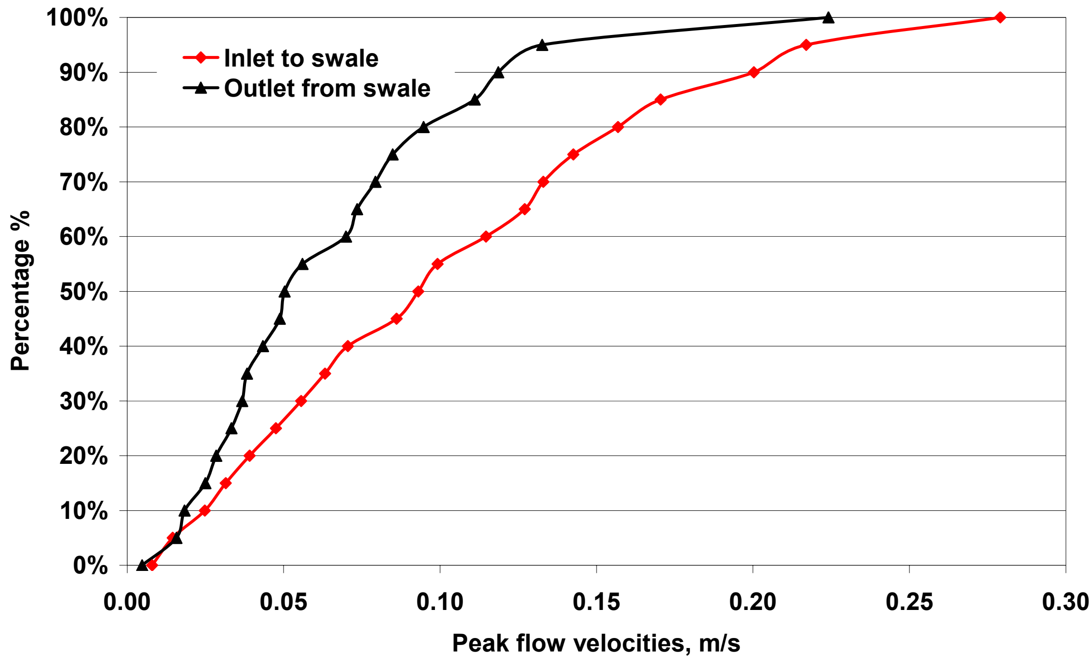


Figure 5.16 Distribution of Peak Velocities Upstream and Downstream of the Swale

5.5.5 Detention time in the system

The flow attenuation times of the two treatment ponds in series and the wetland swale are presented in Figure 5.17. The *lag time* has been calculated as the period of time between the flow peak at the inlet and the corresponding peak at the outlet. The data shows the tendency for a reduction in lag time with increasing peak flow. At the highest peak flows (> 30 l/s), the detention time between the inlet of the silt trap and the outlet of the wetland pond is between 5 and 40 minutes, with an average of 15 minutes. Between the inlet of the silt trap and the outlet of the wetland swale, the total lag is between 30 and 60 minutes, with an average of 40 minutes.

The significant scatter in the data is due to influence of factors other than the peak flow rate, such as rain event duration and intensity, vegetation, antecedent rainfall etc. In addition, many of the inflow events have more than one peak, so the outlet peak does not correspond to a single peak at the inlet.

Lines have been drawn by eye through the data points: increasing lag times at high peak flows may indicate different flood event processes when systems are overtopped and overland conveyance is initiated.

Although many performance studies to date have reported ‘lag time’, it is questionable as to whether this in itself is indicative of performance. It will be a function of the attenuation provided by the system, but will be unique to each system being studied.

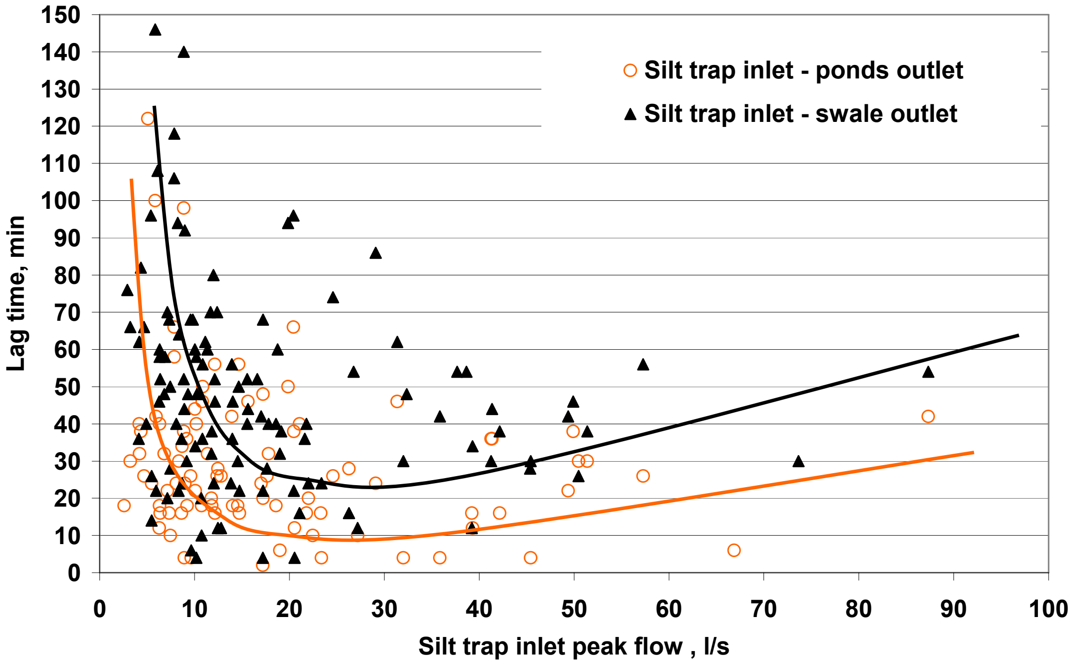


Figure 5.17 Relationship between the inlet peak flow and lag time through the treatment train

The swale is reviewed independently in Figure 5.18, which shows similar results although the trend is less distinct. The detention time in the swale at higher peak flows (> 8 l/s) is between 10 and 20 minutes. There are a number of negative values i.e. the peak at the outlet occurs before the peak at the inlet. This is likely to be due to two factors:

1. At a threshold water depth in the swale when the water depth in the swale reaches the invert level of the inlet pipe, free discharge from the upstream ponds is impeded, and velocities decrease. This produces a hysterisis effect in the depth : velocity relationship (see Figure 5.19);

2. Groundwater seepage and additional surface runoff into the system along the length of the swale during winter months when groundwater levels are raised and ground surfaces are saturated.

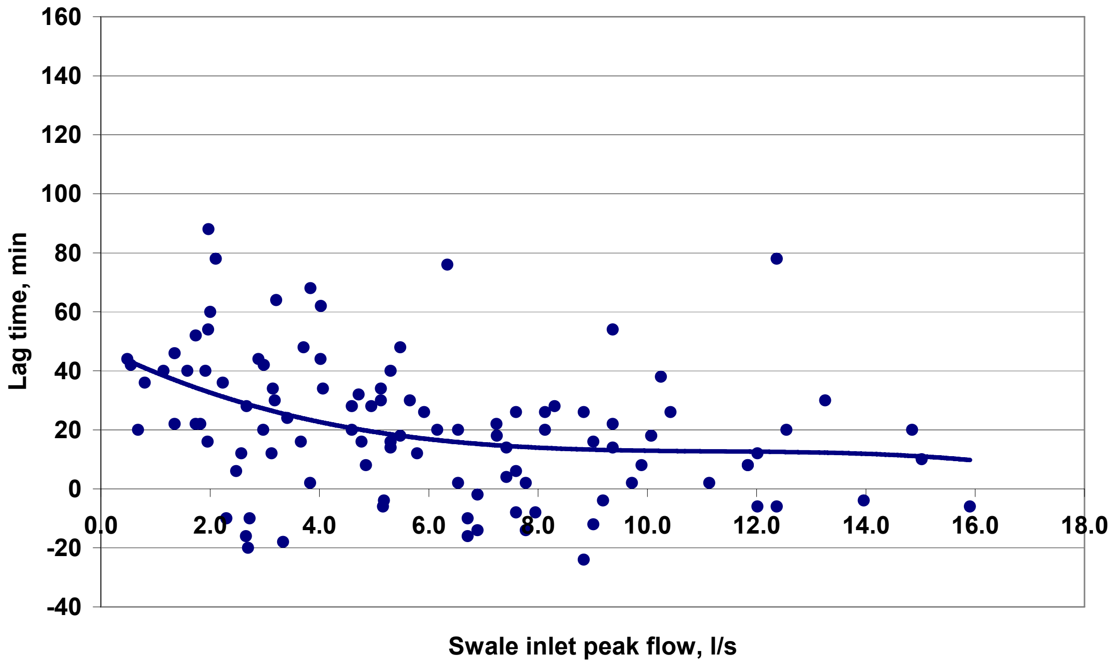


Figure 5.18 Relationship between the swale inlet flow peak and the lag time through the swale

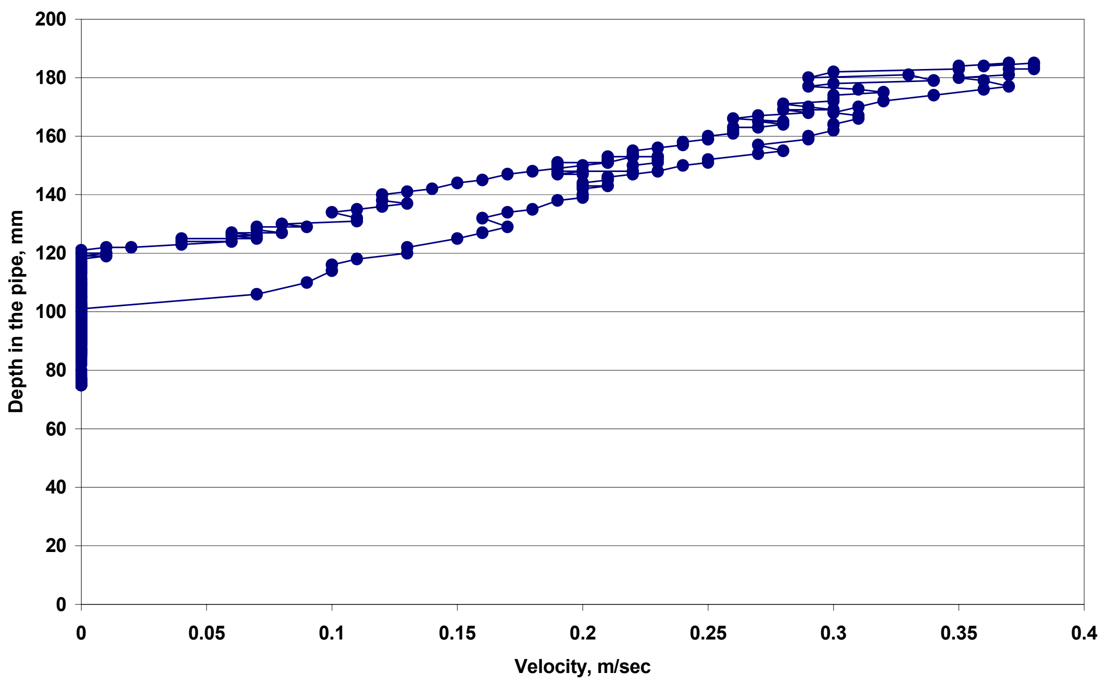


Figure 5.19 Hysterisis Relationship between velocity and depth in the inlet pipe of the swale (18/04/02 - 18/04/04)

5.5.6 *Influence of vegetation on the hydraulic performance*

The likely influence of vegetation on the hydraulic performance has been investigated through a comparison of peak flows and lag times during summer and winter. The results for the double pond system and the swale are presented in Figures 5.20 and 5.21, and 5.22 and 5.23 respectively.

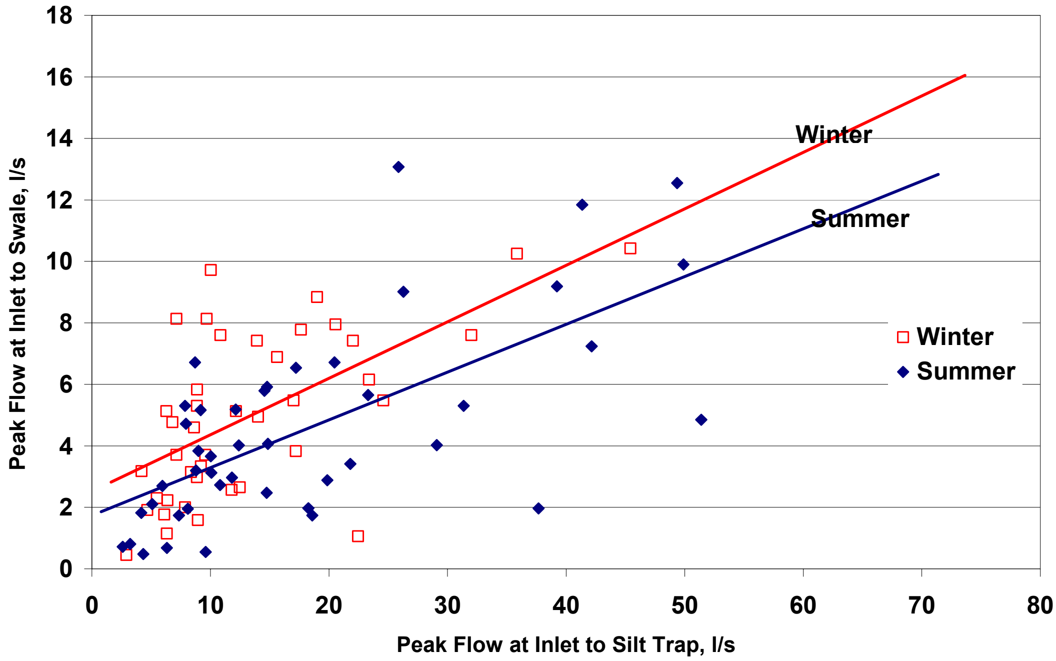


Figure 5.20 Seasonal Influence on Pond Peak Flows

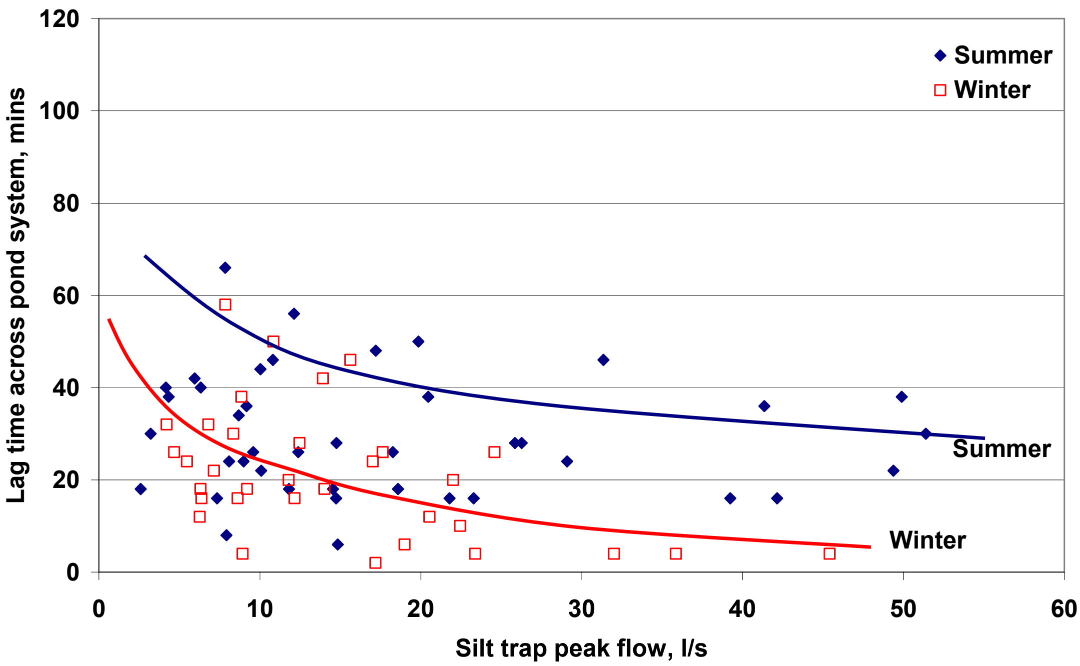


Figure 5.21 Seasonal Influence on Pond Lag Times

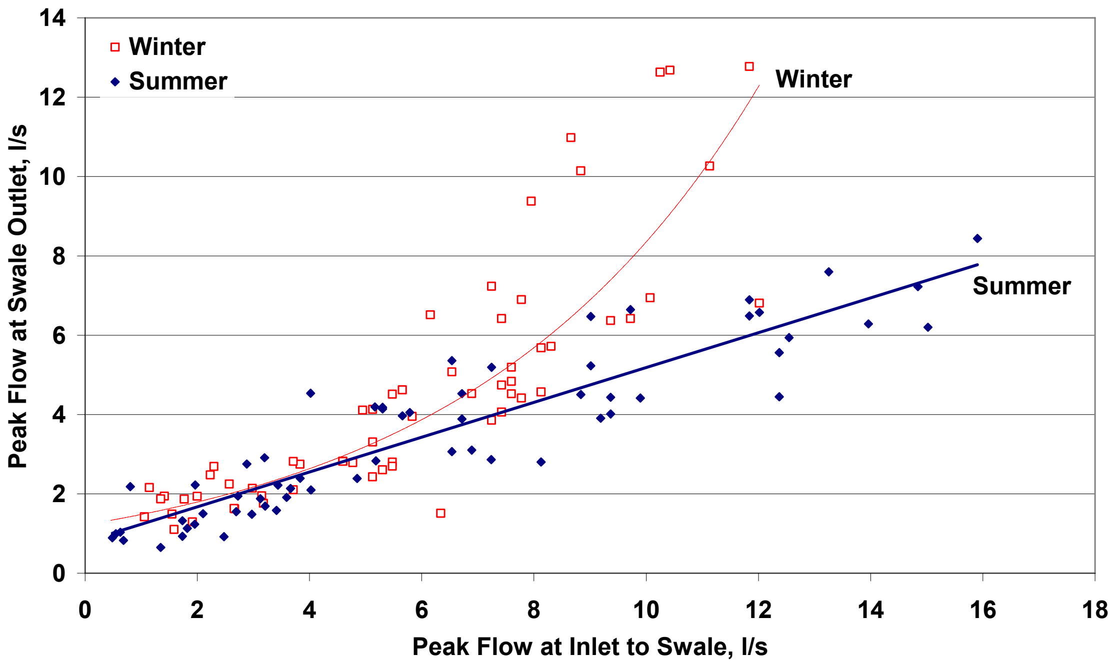


Figure 5.22 Seasonal Influence on Swale Peak Flows

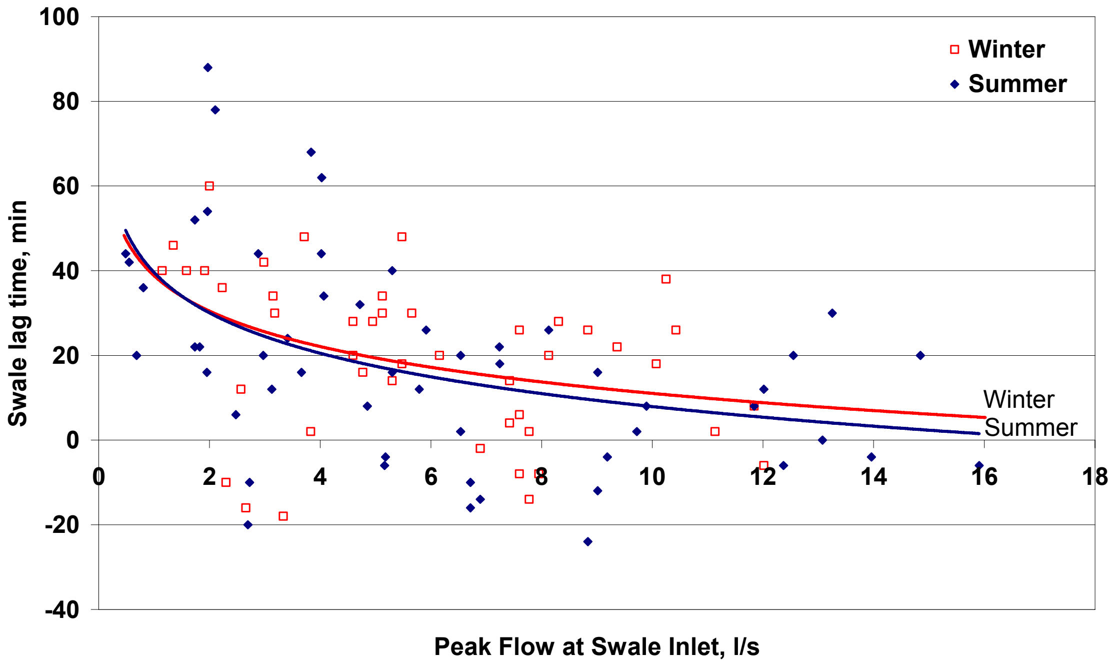


Figure 5.23 Seasonal Influence on Swale Lag Times

The variability in attenuation across the ponds reflects the influence of the pond surface area. When the inflow to the ponds increases, the water level at the outfall from the pond rises until the outflow matches the inflow. In winter, as the cross-sectional area of the pond is large, the water level in the pond is virtually horizontal. The amount of attenuation depends on the volume of storage that needs to be filled to achieve a given increase in the outfall rate. During the summer there is significant vegetation in the pond. This leads to a marginal reduction in the available storage but significantly increases the hydraulic roughness of the pond resulting in a large hydraulic gradient. This results in an increased amount of attenuation during the summer.

The swale is comparatively narrow and the flow is shallow. This means that flow in the swale is dominated by the hydraulic roughness of the swale and that, for a given inflow discharge, the water level in the swale varies from upstream to downstream. When the inflow to the swale increases then the water level at the outfall rises and the hydraulic gradient along the swale increases. As the width of the swale is limited the overall cross-sectional area is small. This means that the storage in the swale is small and depends upon the hydraulic gradient. The results indicate that during the winter there is very little attenuation of the peak discharge. During the summer the hydraulic roughness is significantly larger leading to larger hydraulic gradients for the same discharge. This leads to increased storage within the swale and hence to increased attenuation of the peak discharge.

When considering the lag times for the ponds and the swale it is important to remember that the two systems are in series. This means that the hydrograph entering the swale has already been attenuated within the ponds. The attenuation in the ponds means that the hydrograph entering the swale has been attenuated and has a lower peak than the hydrograph that entered the pond system. The lag times for summer and winter appear to be very similar despite the differences in peak attenuation that are observed. This suggests that the length of the swale is a significant factor in lag time measurements.

5.5.7 Retention capacity of the ponds

The ‘retention capacity’ (i.e. the capacity of the ponds to ‘retain’ the storm runoff) of the ponds has been investigated in terms of the depth of rainfall and the antecedent precipitation index API_5 for each event. API_5 is a parameter that evaluates the wetness of the catchment area prior to an event, based on the precipitation during the preceding 5 days. API_5 is calculated according to the following equation:

$$API_{5,i} = \sqrt{0.5} * (R_{i-1} + 0.5 * R_{i-2} + 0.5^2 * R_{i-3} + 0.5^3 * R_{i-4} + 0.5^4 * R_{i-5})$$

where:

$API_{5,i}$ is the API5 at the start of day i
R_{i-1}, R_{i-2}, R_{i-3}, R_{i-4} and R_{i-5} are the rainfall depths during days i-1, i-2, i-3, i-4 and i-5 respectively

A high API5, therefore, would indicate a high level of rainfall prior to the event i.e. a wet pre-event catchment. A low API5 would indicate a period of dry weather prior to the event. The design permanent volume of the ponds was used in the calculations. The evaporation and evapotranspiration were not taken into account.

The results are presented in Figure 5.24 for different sizes of rainfall events, as follows:

- < 2.2 mm;
- 2.2 mm - 4 mm;
- 4 mm – 6 mm;
- 6 – 14 mm;
- > 14 mm.

It appears that the proportion of the runoff event retained in the pond system decreases with increasing API5. For rainfall events > 14 mm, the limit of the retention capacity is reached and then part of the flow is "lost" via the overflow swale, which result in unrealistic data for the retention capacity especially at higher API5. For large rainfall events and high API5 conditions, the proportion of the event retained by the system i.e. event volume reduction through the system is very low (10%), so the actual design practice of not to consider any volume reduction within pond design is supported by this analysis.

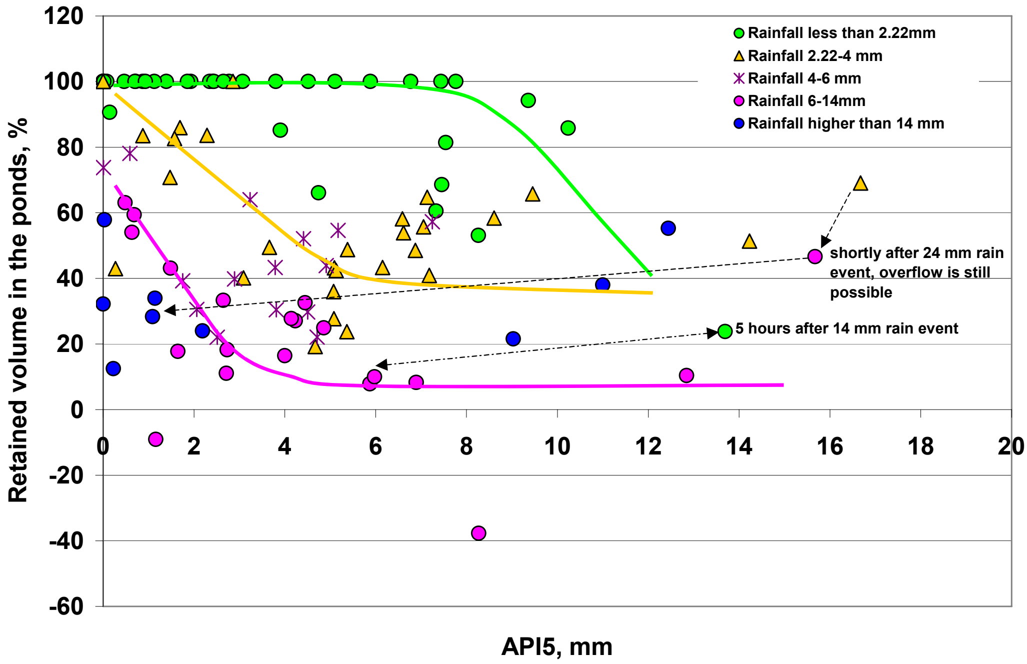


Figure 5.24 Retained volume in the ponds system as percentage of the inflow

Figure 5.25 presents the same relationship, however the retained water volumes are given in m^3/ha (mm) instead of as a percentage of the inflow.

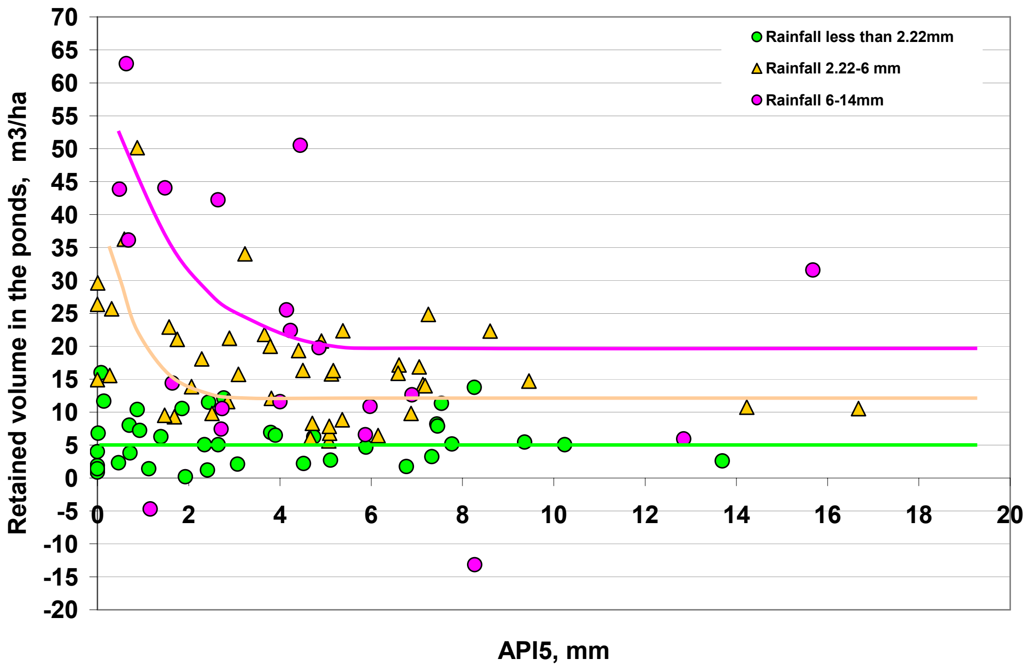


Figure 5.25 Retained volume in the ponds system expressed as m^3/ha

5.6 DATA ANALYSIS AND INTERPRETATION: WATER QUALITY

5.6.1 Analysis objectives

The water quality performance, as well as the aesthetic value of the drainage system, are key parameters in SUDS evaluation in addition to the hydraulics. Particular confidence in water quality improvement performance is attributed to systems with a permanent volume, such as retention ponds. The original SUDS design manuals used in the UK (CIRIA 2000 (a), (b), (c)), suggested that the permanent volume should be as much as 4 times the Treatment Volume (4xVt) and should “provide a residual pond retention time of approximately 14-21 days during the wettest months”. This requirement dictates the necessary space for the pond and the use of such criteria has been of concern in areas where available land is scarce.

The water quality analyses at Hopwood were therefore aimed at:

1. Understanding the water quality performance of the permanent volume of the pond systems;
2. Investigating the role of hydrological conditions, in particular the influence of rainfall quantity and occurrence on the performance of the ponds through the year;
3. Developing reliable criteria and methods for the design and sustainable operation of the ponds.

5.6.2 Approach

The frequency of occurrence and quantity of rainfall events are random in nature, so varying, non-predictable hydraulic and pollutant input loadings are inevitable for all SUDS systems. However, in spite of the probabilistic nature of the rainfall, the atmospheric and anthropogenic pollutant deposition, as well as the diurnal and annual changes in the systems themselves, SUDS are expected to provide an improvement in water quality.

The pollutant characteristics associated with the influent water quality will influence the quality of the permanent volume of the system. The influent pollutants are site specific and will depend on catchment characteristics such as percentage impermeability, traffic loadings, catchment activities, etc. The first flush of run-off after long periods of dry weather is likely to be more polluted than concentrations of runoff during frequent rainfall events. Thus wet conditions may be characterised by lower pollutant concentrations due to the cleaning effect of the impermeable surfaces by the preceding rainfall.

Traditionally the performance of constructed treatment facilities such as SUDS systems has been evaluated by investigating “removal efficiency”. This requires event mean concentrations upstream and downstream of the system, obtained via regular and frequent monitoring using event-based automatic flow-weighted sampling. Such an approach is likely to give reliable results for treatment facilities operating at constant loads and controllable processes of removal, however its application for SUDS is very difficult, due to the unpredictable variability of inlet loads, hydrological regime and the complex interaction among the natural biological, chemical and physical processes occurring within the SUDS system. At Hopwood, therefore, system outlet water quality samples were compared with appropriate environmental standards, in order that the level of residual environmental risk from system discharges could be assessed.

Frequent rainfall events provide a baseflow through the pond systems, and consequently decrease residence times. This may decrease the removal performance efficiency of fine silt particles and the heavy metals that are associated with them. On the other hand, prolonged dry, warm periods provide conditions for stagnant water in the ponds and may result in anaerobic processes of biodegradation occurring in the pond itself. Retention ponds are living ecosystems and are capable of both taking up and releasing nutrients via the vegetation and associated micro-organisms. There is therefore a risk that these processes could potentially increase the concentration of the nutrients in the water, which can in itself also cause release of sediments from the base of the pond. As stagnant water favours algae and vector population growth, the US EPA recommends maximum residence times of 7 days for water pools to prevent the proliferation of algae or mosquitoes.

This raises two questions that the following analysis sought to address:

1. To what extent do the environmental conditions and resulting biological processes in the ponds influence the quality of the permanent pool?
2. How consistent is the water quality treatment performance of the pond system and what are the influences on this performance?

5.6.3 Preliminary hydrological analyses

Catchment rainfall depth plays a key role in SUDS pond design for water quality performance. According to CIRIA, 2000, there are several methods for determining the Treatment Volume (Vt) for a pond:

a. 12-15 mm run-off distributed over each contributing sub-catchment area;
b. 12-15 mm run-off distributed over the contributing impervious area;

c. The volume of run-off generated from the mean annual storm over the total catchment area, or the contributing impervious area;
d. The volume that would capture the runoff from 90% of storms occurring in a year (this could be reduced to 75% if the catchment is small, uniform and has a limited population).

In order to clarify the likely influence of the quantity and distribution of the rainfall on SUDS design and performance, rainfall data from Hopwood was analysed for the period 1999-2003. A summary of some of the key rainfall characteristics is given in Table 5.4.

Table 5.4 Key rainfall characteristics at Hopwood for the period 1999-2003

Year	Annual rainfall	Annual average storm	90th percentile storm *	Total number of rainy days		Number of rainy days ≥1mm	
	mm	mm	mm	nr	%	nr	%
1999	860	4.37	11.7	197	54	165	45
2000	897	4.85	12.5	185	51	156	43
2001	788	4.45	10.9	177	48	138	38
2002	830	4.37	10.3	193	53	139	38
2003	616	4.50	11.3	173	47	114	31
Average	798	4.51	11.5	185	51	142	39

* assumed to be equal to 90th percentile of rainy days

It can be seen that the rainy days constitute about half of the days in a year, and that the number of rainy days with rainfall greater than 1 mm (that are actually likely to produce run-off) is less than 40% of the year.

The seasonal distribution of the rainfall is given on Figure 5.26. The vertical, dotted lines show the rainfall depths that correspond to the point at which the capacity (permanent volume) of different parts of the treatment train is reached.

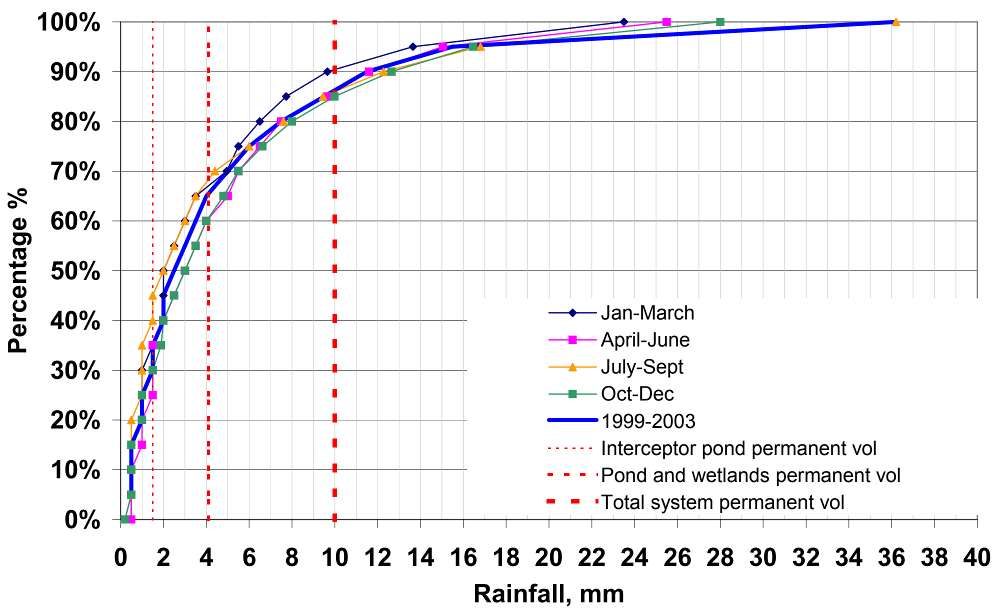


Figure 5.26 Seasonal distribution of rainfall as a percentage of rainy days (1999-2003)

The 10mm permanent (treatment) volume represents the 85th percentile of daily rainfalls over the study period. 35% of the rainy days produce less run-off than the permanent volume of Interceptor Pond l, so it will not be completely replaced by fresh run-off during these events. The corresponding values for the treatment train up to the inlet of the swale and for the whole system are 65% and 85% respectively. It can therefore be concluded that, in any one year, for 15% of the storm events, water quality at the outlet will be influenced, to some degree, by water quality at the inlet. These events, however, are large and the average concentrations of pollutants will be lower. For 85% of storm events the effluent water quality will be predetermined by the quality of the permanent volume in the system.

In Figure 5.27, the distribution of rainy day frequency is plotted against antecedent precipitation index (API_5). As would be expected, during the summer (July-September) 65% of the days have an API_5 less than 2mm and during October-December this reduces to 45%.

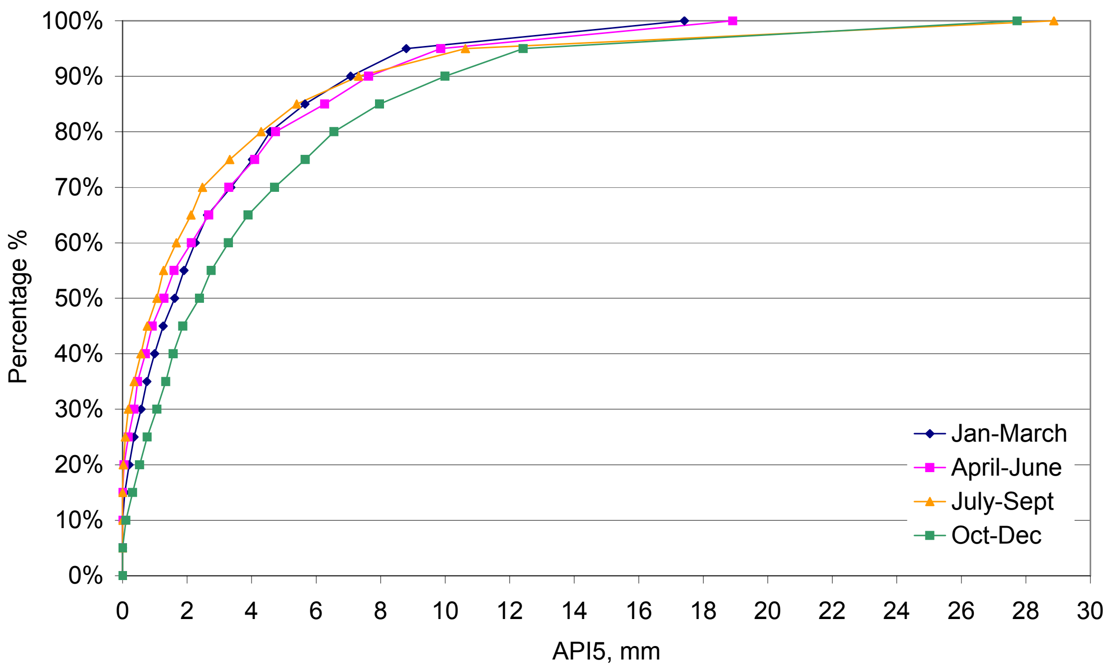
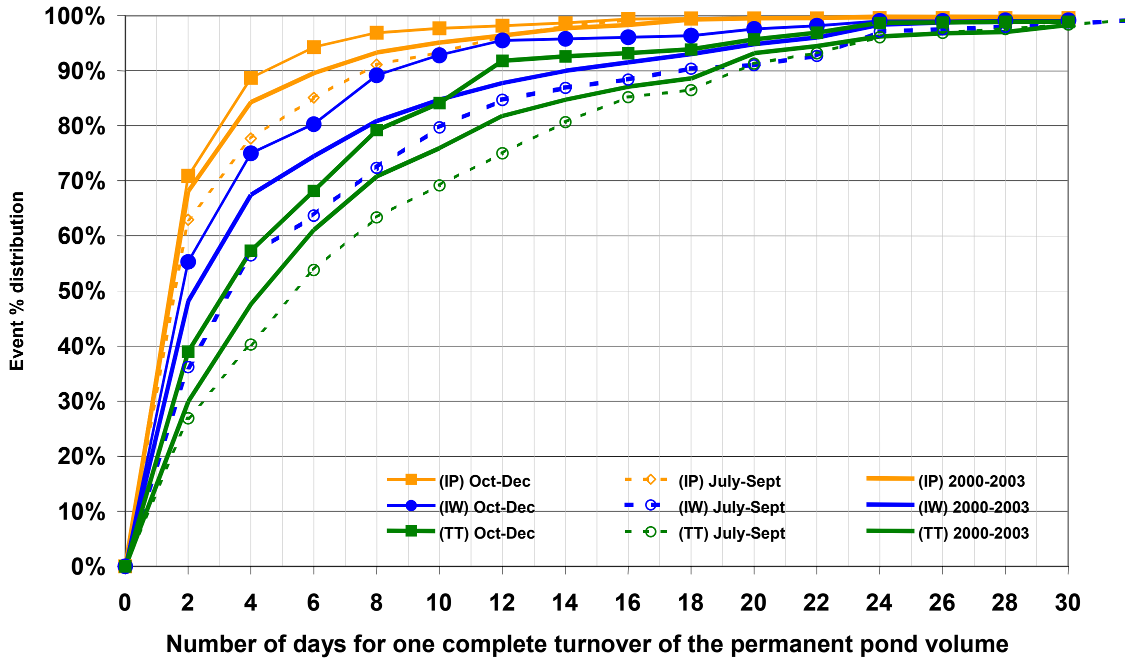


Figure 5.27 Seasonal distribution of rainfall against API5

The concept of 'residence time' was addressed in Chapter 3. Figure 5.28 presents the theoretically-derived residence time (number of days between a complete replacement of the permanent pond volume) in the different parts of the treatment train. These are given for the wettest period of the year (October – December) and the driest period of the year (July-September). Average annual values have also been given for comparative purposes.



Legend: IP – Interceptor Pond 1, IW – Interceptor Pond 1 + Interceptor Wetland + Interceptor Pond 2, TT – Interceptor Pond 1 + Interceptor Wetland + Interceptor Pond 2 + Wetland Swale + Balancing Pond

Figure 5.28 Residence time within the different units of the treatment train

Table 5.5 gives a summary of the residence times in the different parts of the system.

Table 5.5 Residence times through the treatment train

Components of the treatment train	25th percentile		50th percentile		75th percentile	
	Winter	Summer	Winter	Summer	Winter	Summer
Part 1: Interceptor Pond	1	1	1.5	1.5	2.5	4
Part 2: Interceptor Wetland	1	1	2	3	4	9
Part 3: Treatment Train	1	2	3	5.5	7.5	12

The upper components of the treatment train have a relatively frequent exchange of water due to the small permanent water volumes. The whole treatment train, however, has prolonged residence times. During the winter, the 90th percentile is 11.5 days, which can be compared with the recommended detention time given by CIRIA of 14-21 days.

The relation between the water quality of the permanent volume and the hydraulic conditions was further investigated through the water quality analyses, as discussed in the following sections.

5.6.4 Water quality performance against environmental standards

As discussed in Section 2.4, appropriate environmental standards were identified as part of the literature review to provide parameters against which performance of the components of the treatment train could be judged.

The water quality samples were appraised against 3 sets of criteria:

(A) The water quality standards adopted by the Environment Agency for Rivers Class 2, supplemented by additional standards for TSS, Pb and Cr as these parameters are not included in the EA categorisation (summarised in Table 5.6);

(B) Mean levels of pollutants observed in minimally impaired ponds in the UK, supplemented by additional criteria, where required (summarised in Table 5.7);

(C) The standards applied to the discharge of urban wastewater, according to EC directive 91/271/EEC, supplemented by additional criteria where required (summarised in Table 5.8).

Table 5.6 Criteria A: based on water quality standards for Environment Agency Rivers Class 2

Parameter	Limiting Values	Source
NH_4-N	0.6 mg/l	EA, rivers class 2
TSS	20 mg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
BOD_5	4.0 mg/l	EA, rivers class 2
Copper, dissolved	22 mcrg/l	10<CaCO3<50, EA, rivers class 2
Zinc, total	200 mcrg/l	10<CaCO3<50, EA, rivers class 2
Lead, total	20 mcrg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
Chrome IV	2 mcrg/l	US EPA Chronic Value
Chrome III	44 mcrg/l	US EPA Chronic Value

Table 5.7 Criteria B: based on water quality of minimally impaired ponds in the UK

Parameter	Limiting Values	Source
NH_4-N	0.3 mg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
TSS	19 mg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
BOD_5	2.5 mg/l	EA, rivers class 1
Copper, dissolved	11.5 mcrg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
Zinc, total	97 mcrg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
Lead, total	21 mcrg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
Chrome IV	2 mcrg/l	US EPA Chronic Value
Chrome III	44 mcrg/l	US EPA Chronic Value

Table 5.8 Criteria C: based on water quality standards from EU Directive 91/271/EEC for the discharge of urban waste water

Parameter	Limiting Values	Source
NH_4-N	0.6 mg/l	EA, river class 2
TSS	35 mg/l	Directive 91/271/EEC
BOD_5	25 mg/l	Directive 91/271/EEC
Copper, dissolved	22 mcrg/l	10<CaCO3<50, EA, river class 2
Zinc, total	200 mcrg/l	10<CaCO3<50, EA, river class 2
Lead, total	20 mcrg/l	Mean value in minimally impaired ponds in UK, (HR Wallingford, 2003)
Chrome IV	2 mcrg/l	US EPA Chronic Value
Chrome III	44 mcrg/l	US EPA Chronic Value

Figures 5.29 to 5.31 present the level of compliance of the samples with each of the set of criteria. The metals are expressed as concentration of total metals. Since there are no standard limits for total Copper, the recommendations for dissolved Copper limiting concentrations have been used. The more strict limit for Cr IV – 2 mcrg/l has also been used for further analyses, since the available data for Cr are expressed as Total Cr, without making reference to Cr IV or Cr III forms. The sustainable urban drainage units are ordered according to their place in the treatment train.

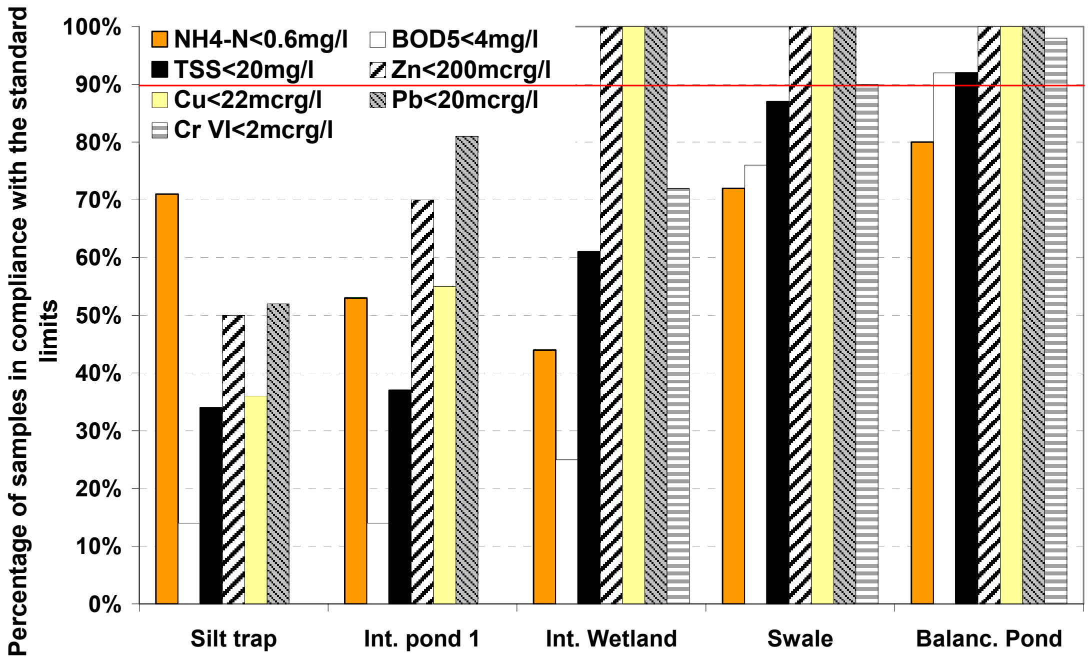


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Figure 5.29 Percentage of samples compliant with Criteria A

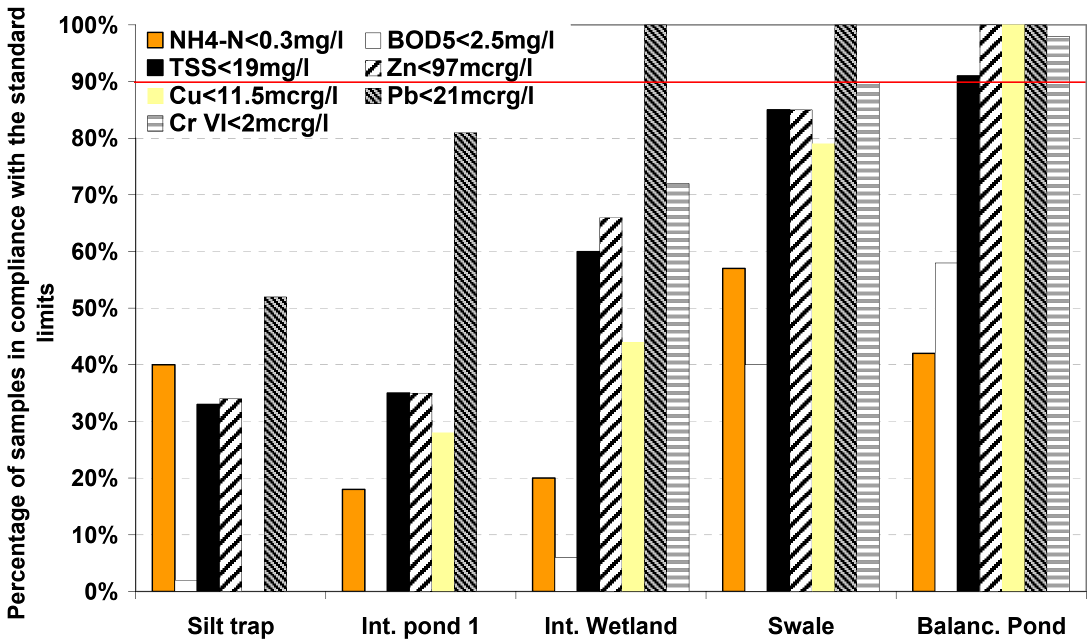


Figure 5.30 Percentage of samples compliant with Criteria B

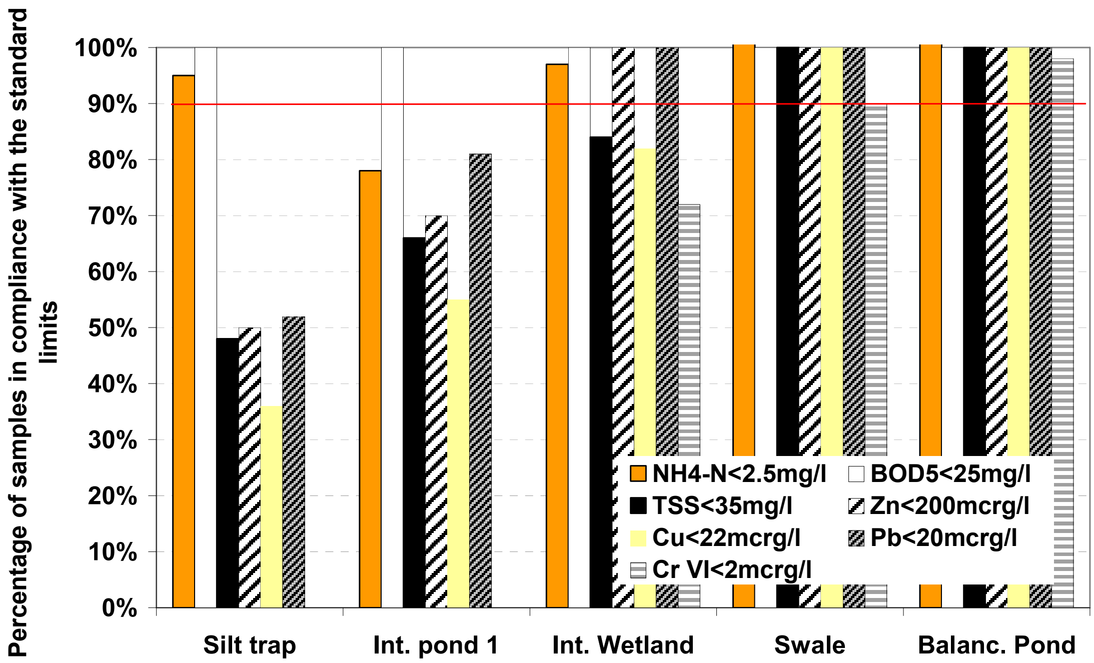


Figure 5.31 Percentage of samples compliant with Criteria C

For NH_4-N, BOD_5 and TSS, the percentage of compliance is low against Criteria A and B, due to the very strict standards for fresh water quality and natural ponds. However if the water in the ponds is regarded as urban wastewater, then NH_4-N and BOD_5 are not polluters of concern even at the inflow to the SUDS system. It is also observed that the percentage of compliance for the metals is high whichever criteria is applied, especially across the last three units of the treatment train.

In general, the water quality improves along the treatment train and the balancing pond has the highest percentage of compliance for all the concerned pollutants. All this emphasises the advantage of treatment trains over independent SUDS units.

5.6.5 Influence of residence time on water quality performance

A study was then undertaken to review the dependence of water quality of the permanent pools on the preceding residence time of that volume. Figures 5.32, 5.33 and 5.34 present the relationship between the concentrations of NH_4-N, TSS and BOD_5 in the permanent volume and the residence time. The trend lines are drawn by eye rather than being fitted to the observations. The horizontal dotted line shows the EA standard limits for Rivers Class 2 for NH_4-N and BOD_5, and the mean value of minimally impaired ponds in the UK for TSS respectively.

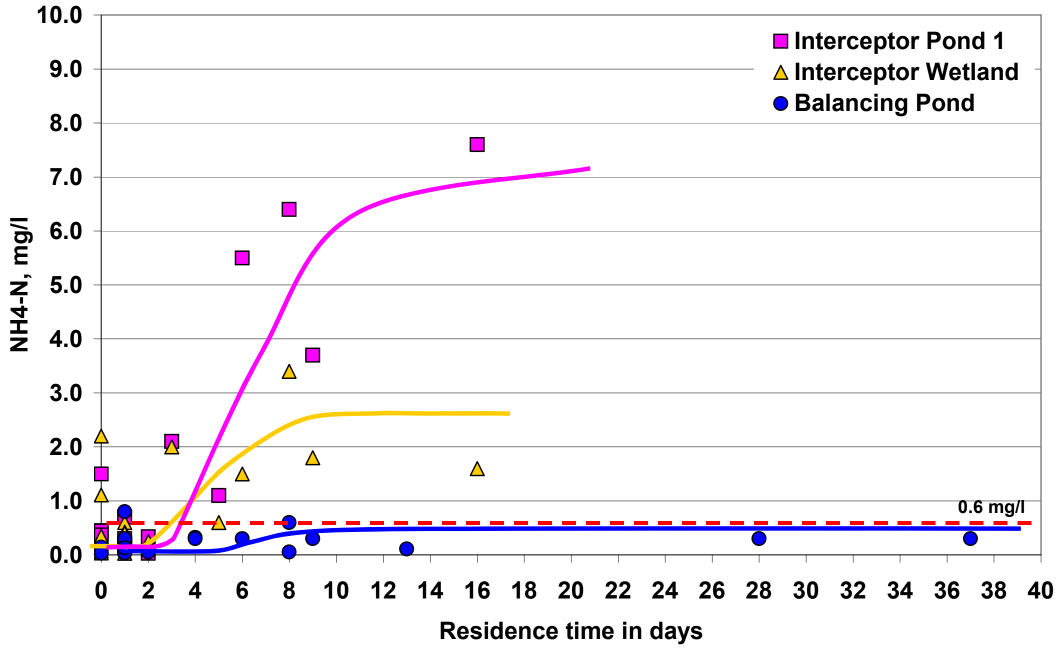


Figure 5.32 Relationship between NH_4-N concentrations and residence time

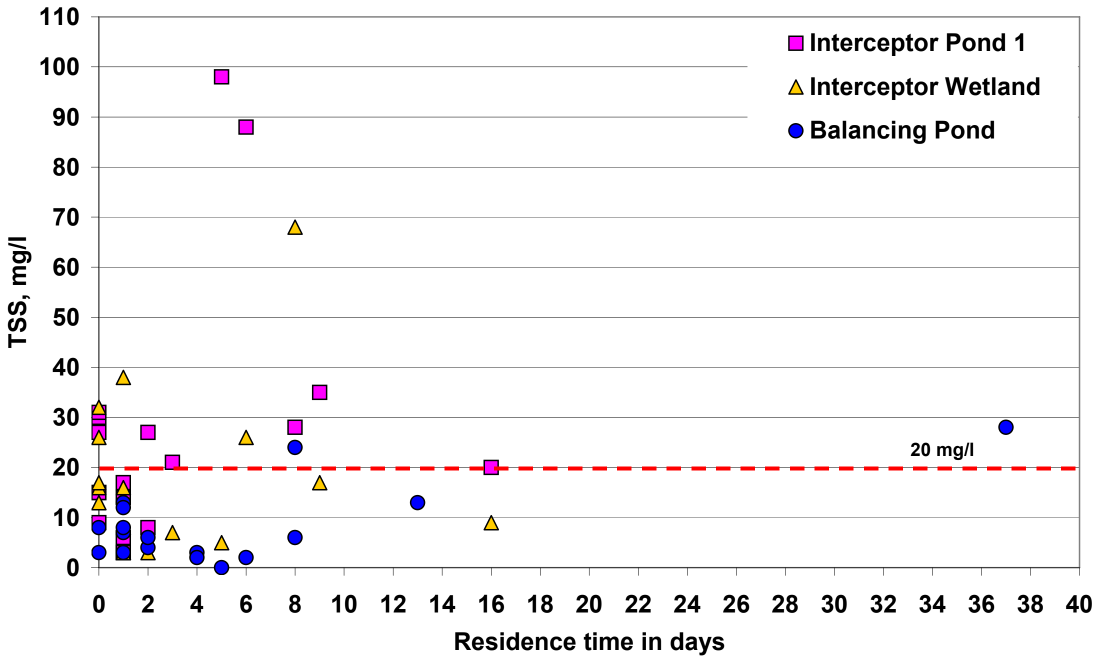


Figure 5.33 Relationship between TSS concentrations and residence time

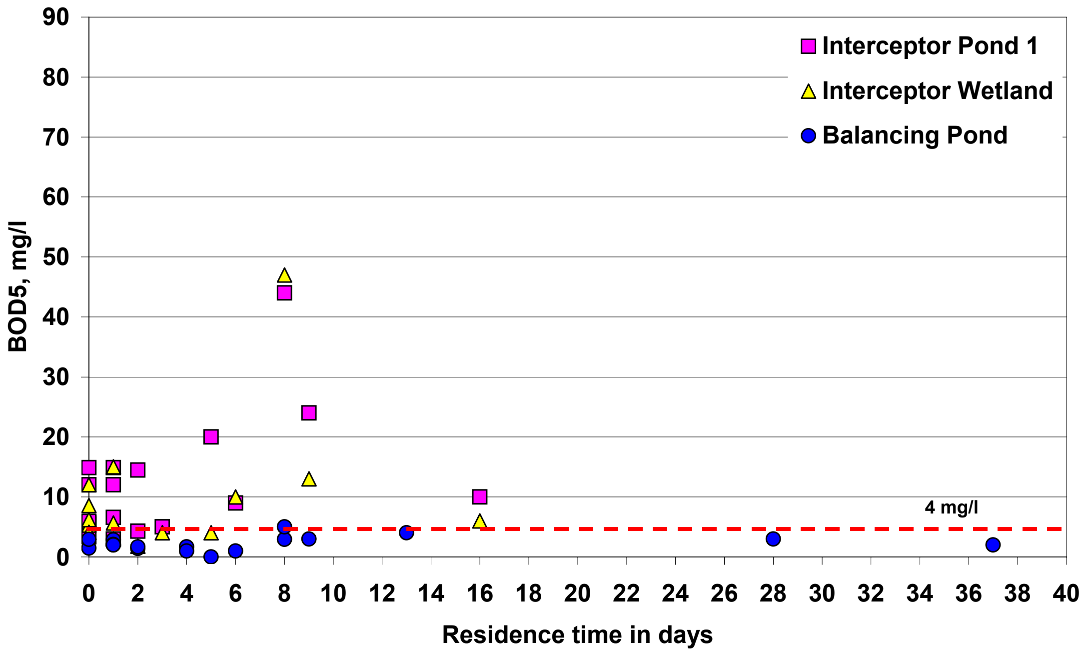


Figure 5.34 Relationship between BOD_5 concentrations and residence time

The water quality of the balancing pond appears to be independent from residence time and is characterised by pollutant concentrations well below the standard limits. However the water quality in the Interceptor Pond and Interceptor Wetland are generally above the limits and pollutant levels show some dependence on residence time. NH_4-N shows the greatest dependence, and the relationship shows a gradual increase in concentrations, for residence time higher than 2 days. The values for TSS are scattered which leads to the conclusion that are other factors that influence this parameter.

5.6.6 Influence of antecedent conditions on water quality performance for NH_4-N, TSS and BOD_5

Figures 5.35 to 5.37 present the relationship between the water quality of the permanent volume and the antecedent wetness, represented by API5. Where the data points are infilled with a darker spot, this indicates that the sample was taken immediately after a rainfall event. Horizontal, dotted lines are the EA standard limits for Class 2 Rivers.

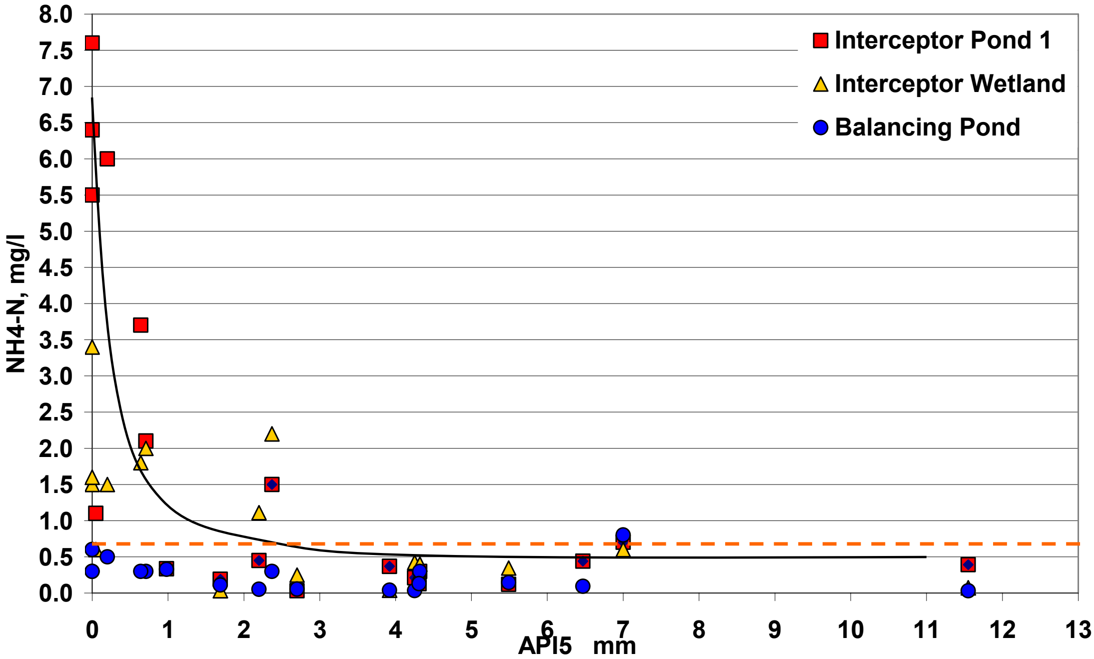


Figure 5.35 Relationship between NH_4-N concentrations and API5

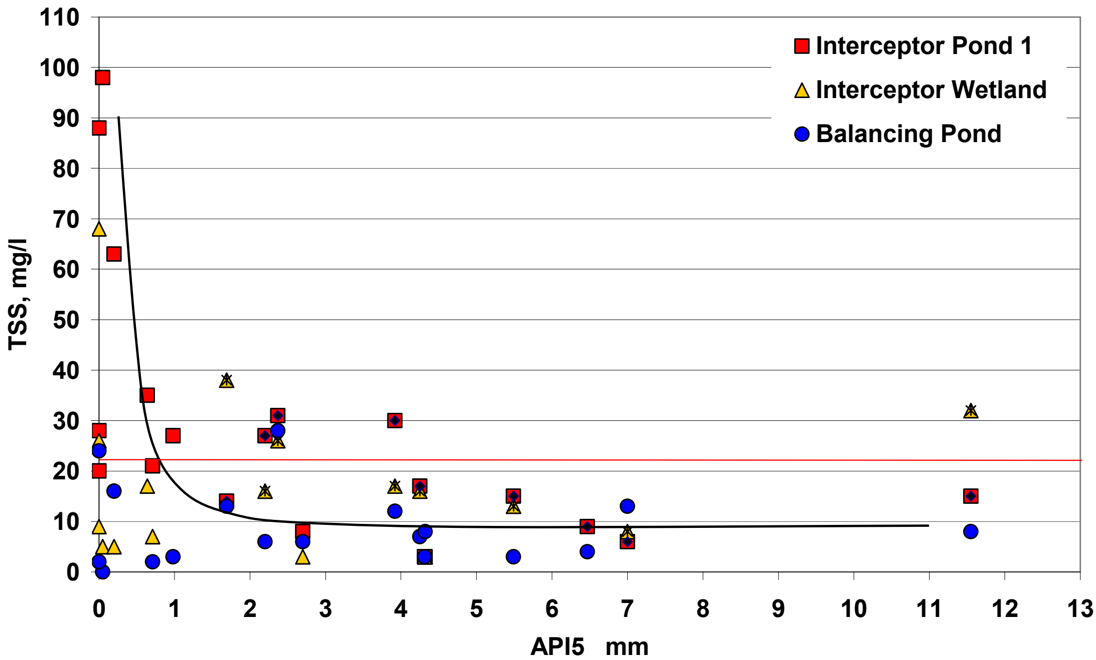


Figure 5.36 Relationship between TSS concentrations and API5

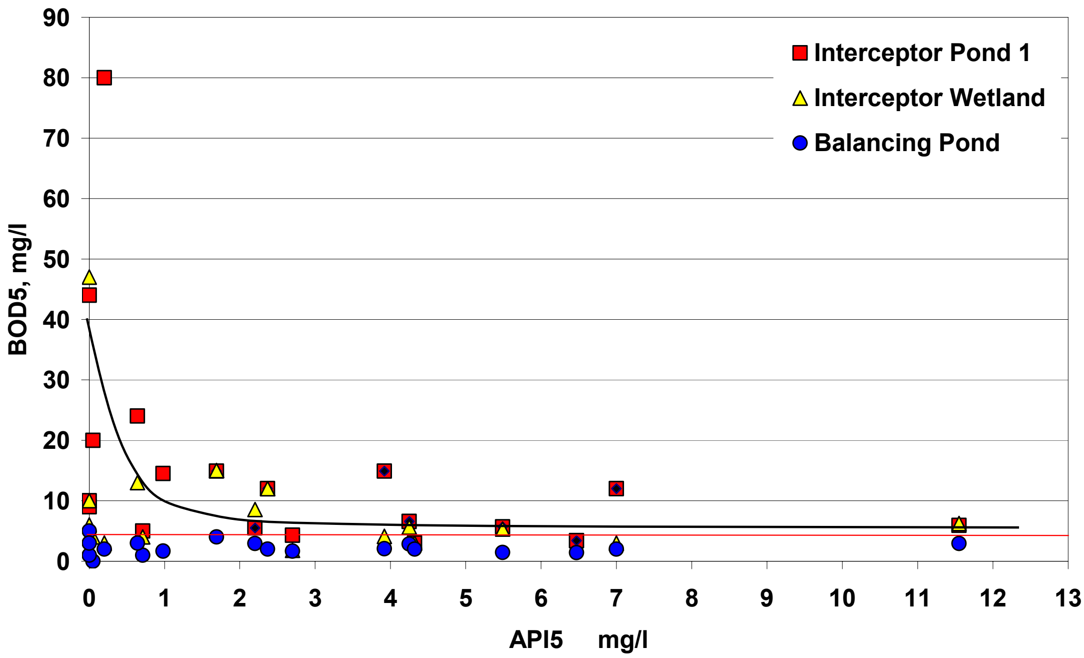


Figure 5.37 Relationship between BOD_5 concentrations and API5

These plots show that, for this site, API5 is a useful indicator of the water quality of the interceptor pond with respect to NH_4-N, TSS and BOD_5, and an API5 of less than 3 mm appears to correlate with an increase in concentrations of these parameters.

5.6.7 Relationships between sediment and heavy metal concentrations

Heavy metals can exist in surface waters in three forms:

a. Particulate forms : adsorbed to clay, silica and organic matter;
b. Colloidal forms: as hydroxides, oxides, silicates or sulphides;
c. Dissolved forms: as ions.

Metals tend to be concentrated onto smaller particles due to their increased specific surface. The finer the silt particles, the greater their absorption surface and the greater the potential concentration of heavy metals. The size of the particles making up the sediment composition at Hopwood was not measured as part of this study.

Metal concentrations in the ponds tend to show a strong relationship with the level of suspended solids. Figure 5.38 presents the results from Hopwood for different heavy metals. Although some of the results are quite dispersed, all of them show reliable correlations.

'Metals in SS' is equal to 'Total Metals – Dissolved Metals', and thus represents the portion of metal pollutant that is sorbed onto the suspended sediments. This is therefore the particulate or colloidal form of the metal.

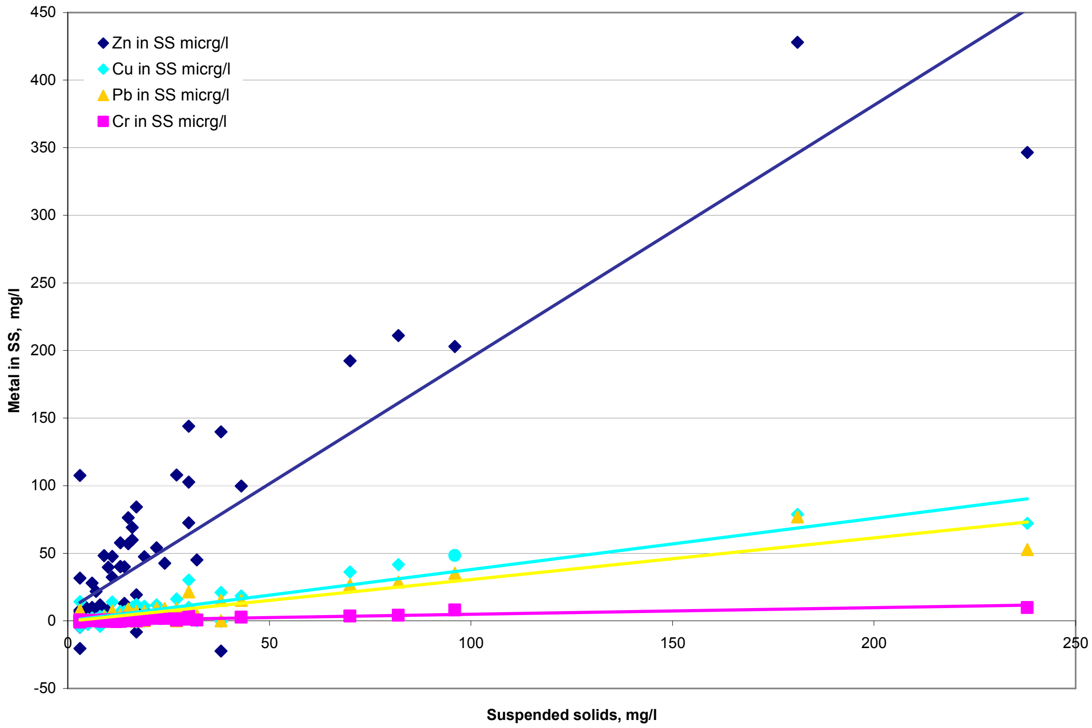


Figure 5.38 Relationship between suspended solids and particulate metal concentrations

In the following graphs, the metal pollutant levels are expressed as microgram of metal in suspended solids per gram of suspended solid, rather than per litre of water as in Figure 5.38.

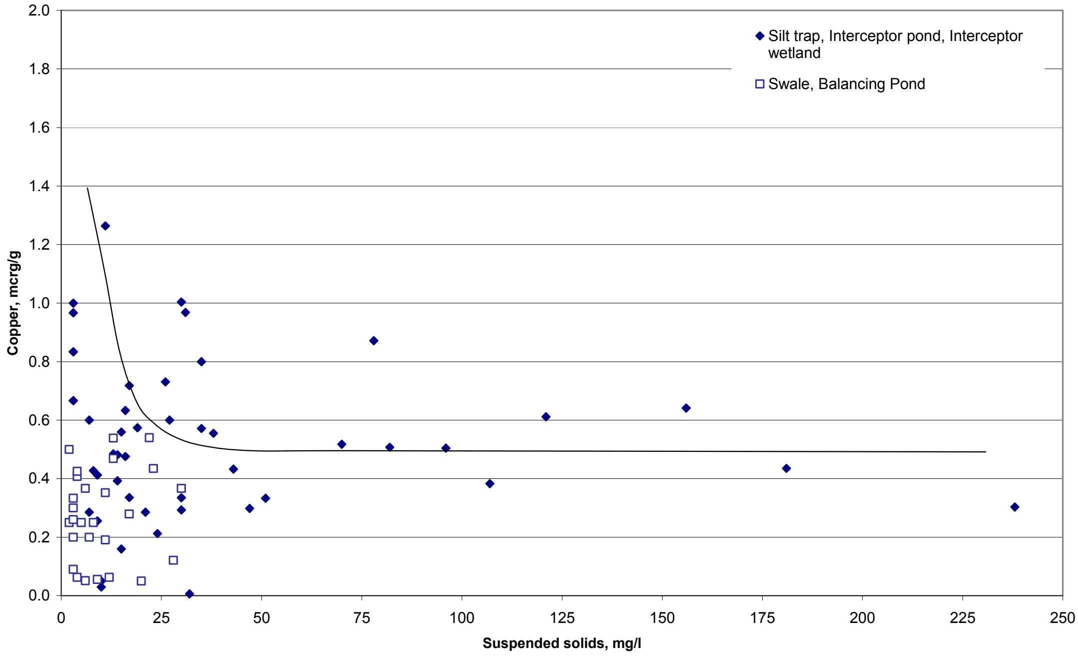


Figure 5.39 Relationship between suspended solids and copper proportion

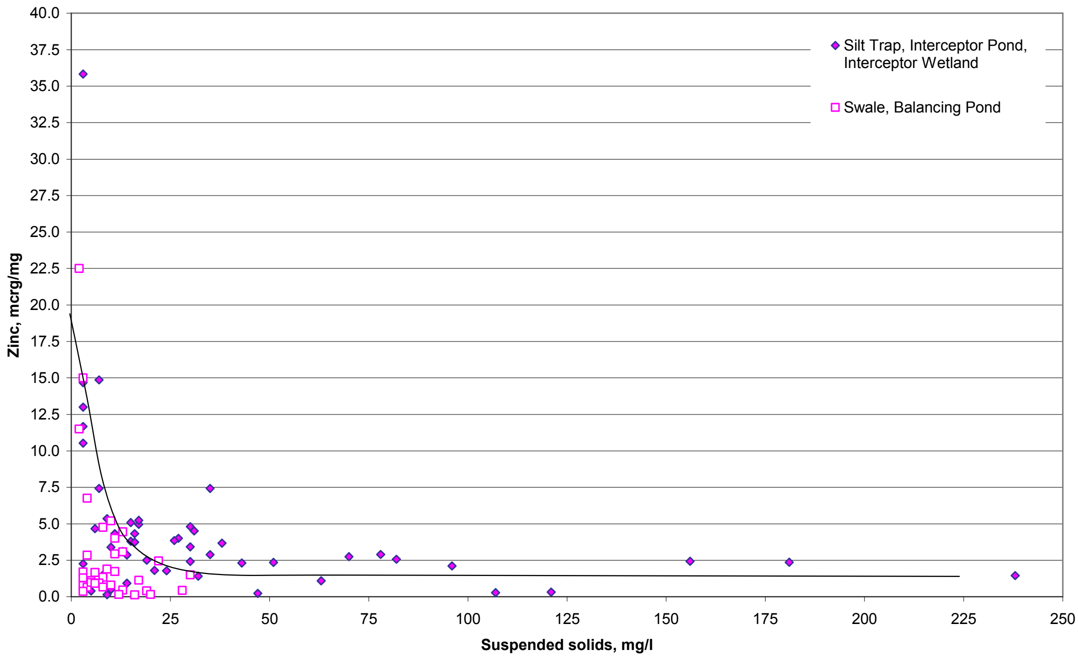


Figure 5.40 Relationship between suspended solids and zinc proportion

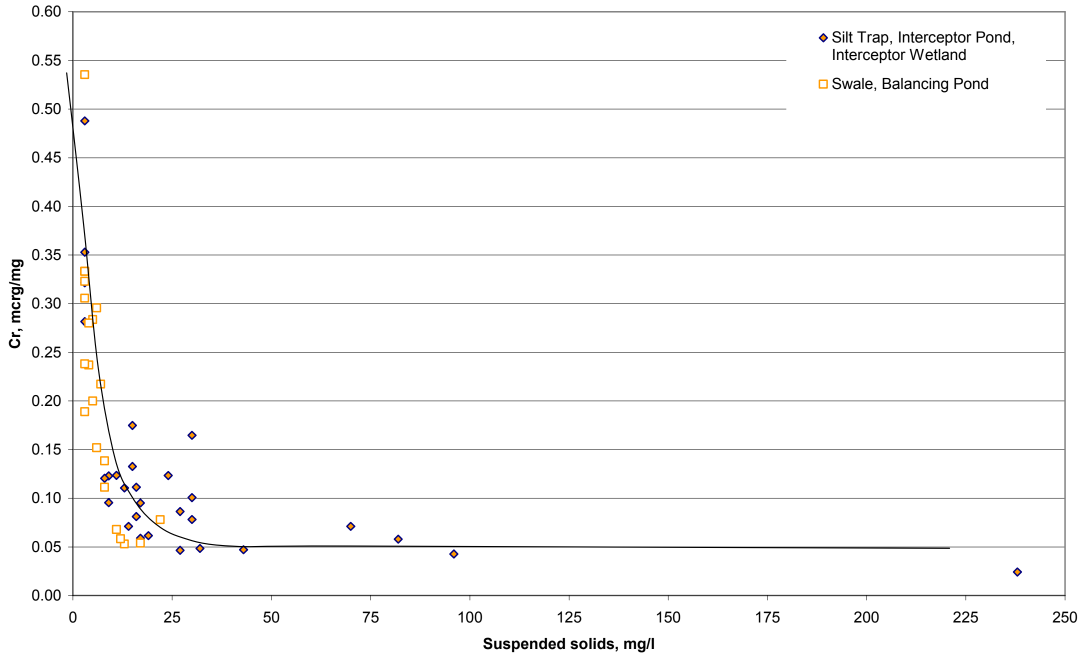


Figure 5.41 Relationship between suspended solids and chromium proportion

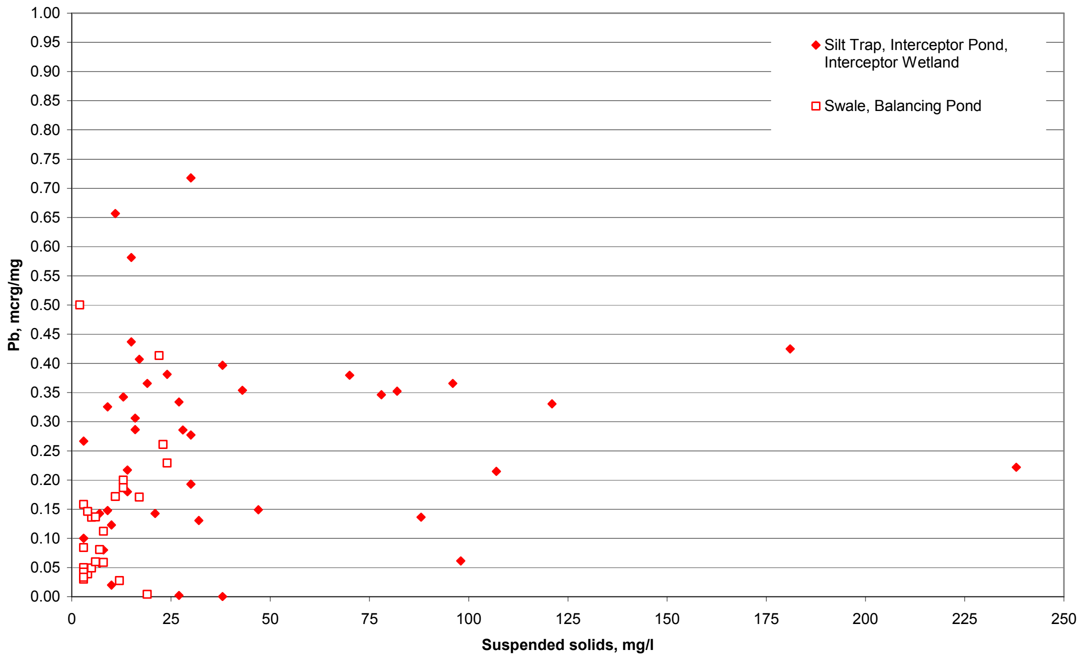


Figure 5.42 Relationship between suspended solids and lead proportion

During / post heavy rainfall events, at high suspended solids concentrations, there will be a greater proportion of coarse particles in suspension which have a lower active surface for metal absorption and therefore the metal content will be reduced.

All the investigated metals except for lead (Pb) show a trend of limiting minimum values at higher suspended solids concentrations for the first SUDS units in the train (the Silt Trap, Interceptor Pond and Interceptor Wetland). Knowing the event mean concentrations (EMCs) of SS, the loads of heavy metals that enter the system can be predicted. It appears that the content of heavy metals in the suspended solids in the last two units of the treatment trend is lower and does not tend to show any relation with the suspended solid concentrations. This phenomenon requires more detailed investigation.

Metals can exist in water in dissolved and particulate forms. While the particulate form can be removed through sedimentation of the silt, the dissolved form exists in ions and cannot be removed through physical processes. A part of the dissolved metals may be removed through biochemical processes (e.g. uptake by microorganisms or plants). However there is a risk that these portions may be released back in the water through decay of the organic matter.

The figures below present the observed relationships between dissolved metals and total metal concentrations. All the trend lines are drawn by eye, rather than fitted to the observed data.

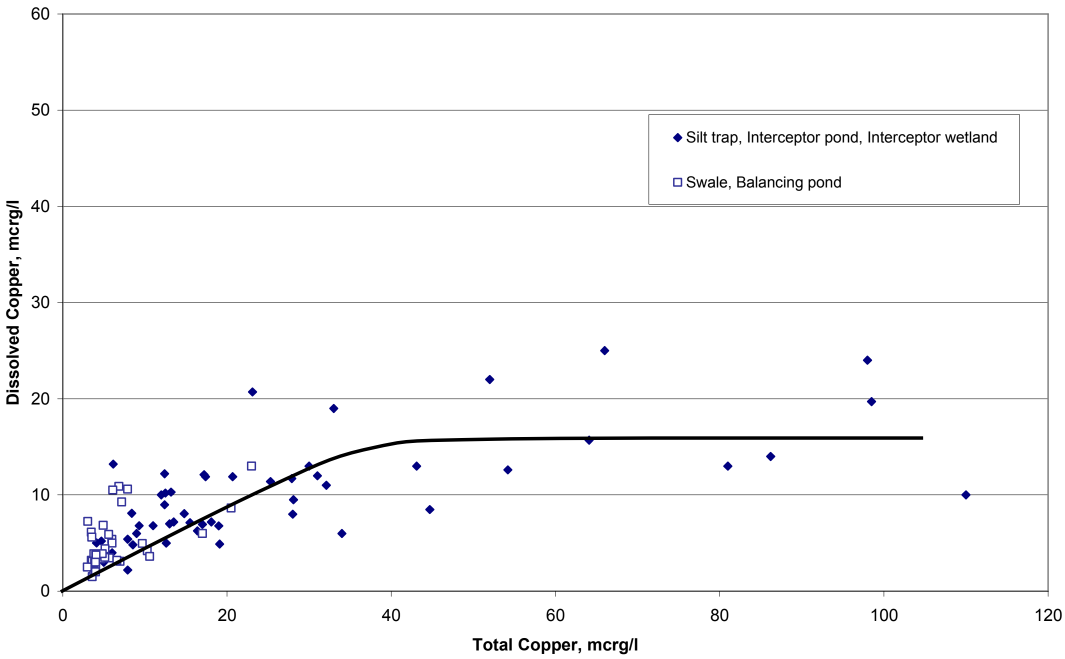


Figure 5.43 Relationship between dissolved and total copper

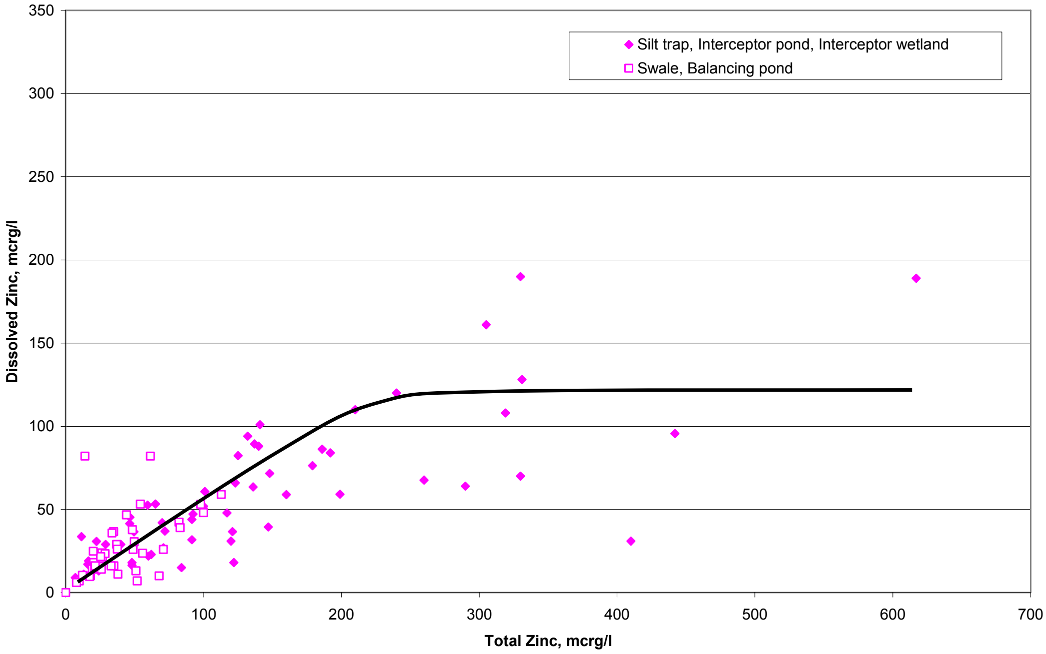


Figure 5.44 Relationship between dissolved and total zinc

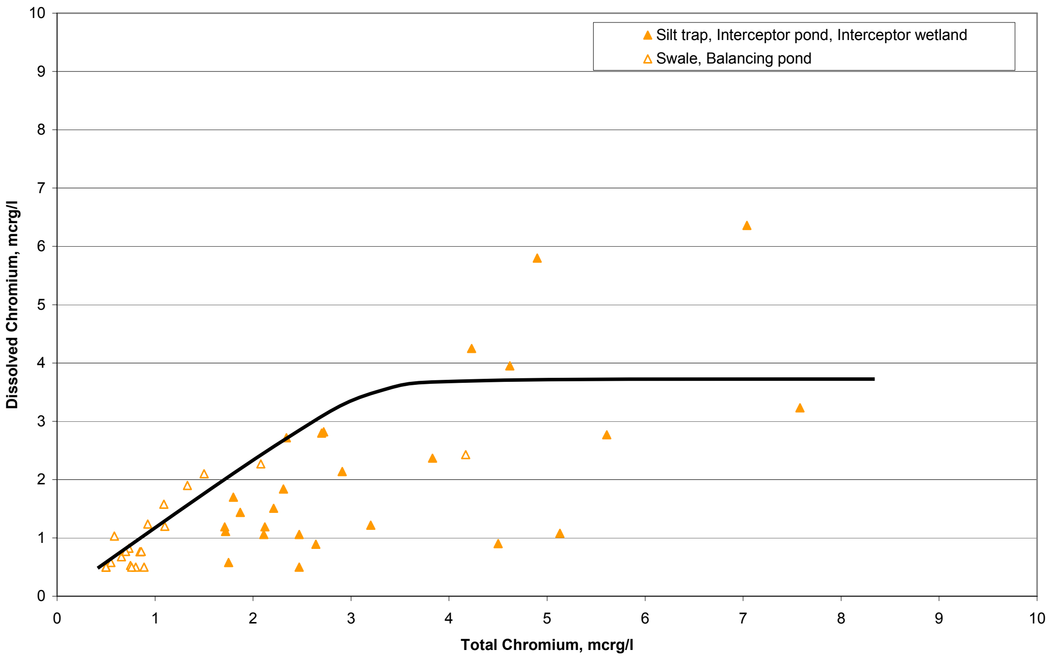


Figure 5.45 Relationship between dissolved and total chromium

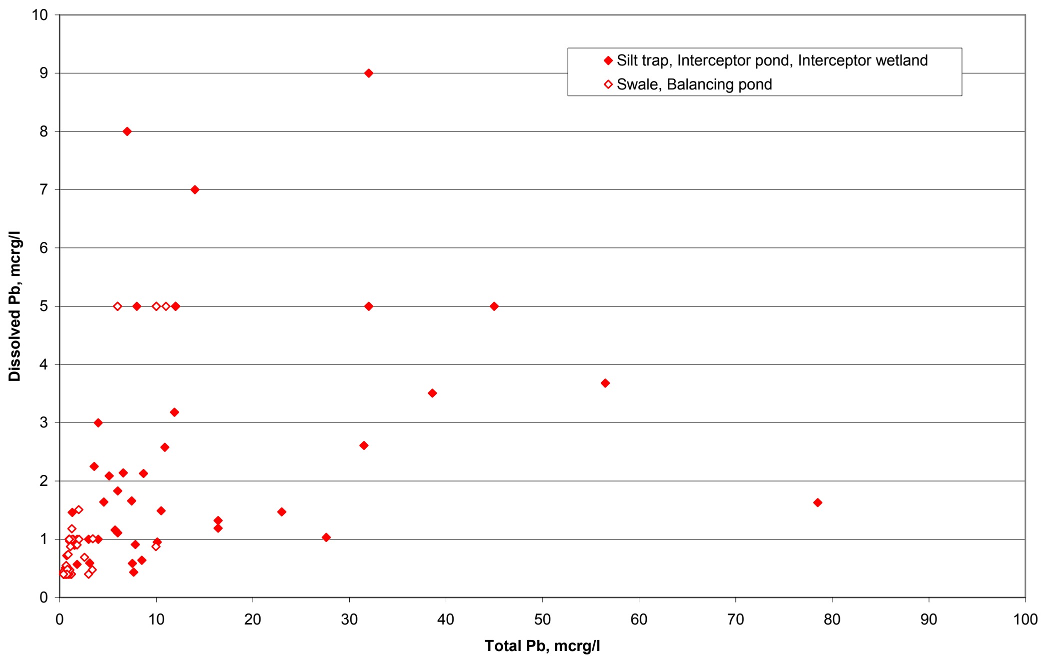


Figure 5.46 Relationship between dissolved and total lead

For Cu and Zn at lower concentrations the dissolved form comprises about 25 % and for Cr about 50% of the total form and shows a trend of limiting values at higher concentrations of the total form. For Cu, Zn and Pb it appears that the prevailing form of the metal is the particulate one, so the main mechanism of removal should be via sedimentation.

5.6.8 Influence of antecedent conditions on water quality performance for heavy metals

During dry weather (lower API5), due to the quiescent conditions in the water body, it is assumed that the majority of the gravitational sedimentation has taken place and that the fine colloidal silt particles or organic matter (algae) will form the greater part of the suspended matter. During wet weather (higher API5) or shortly after rainfall events, the gravity sedimentation will not be complete and the SS concentration as well as the size of the particles in the water pool will be greater. Where the data points are infilled with a darker spot, this indicates that the sample was taken shortly after a rainfall event.

Figures 5.47 to 5.49 show the observed relationships between total metal concentrations and antecedent conditions as API5. An initial look at the graphs confirms that that the total concentration of metals does not appear to be related to the antecedent conditions represented by API5. However, if the points where sampling was undertaken < 24 hours after a rainfall event (and therefore the settlement process is unlikely to be complete) are removed, a relationship may exist.

Horizontal, dotted lines are the EA standard limits for Class 2 Rivers.

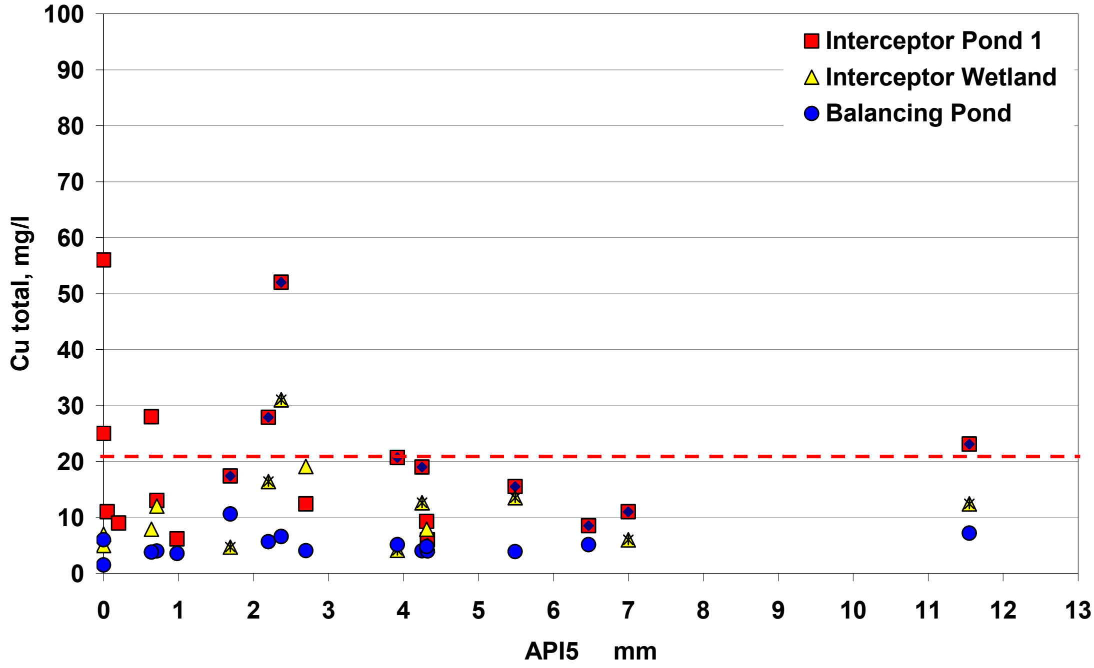


Figure 5.47 Relationship between Cu concentrations and API5

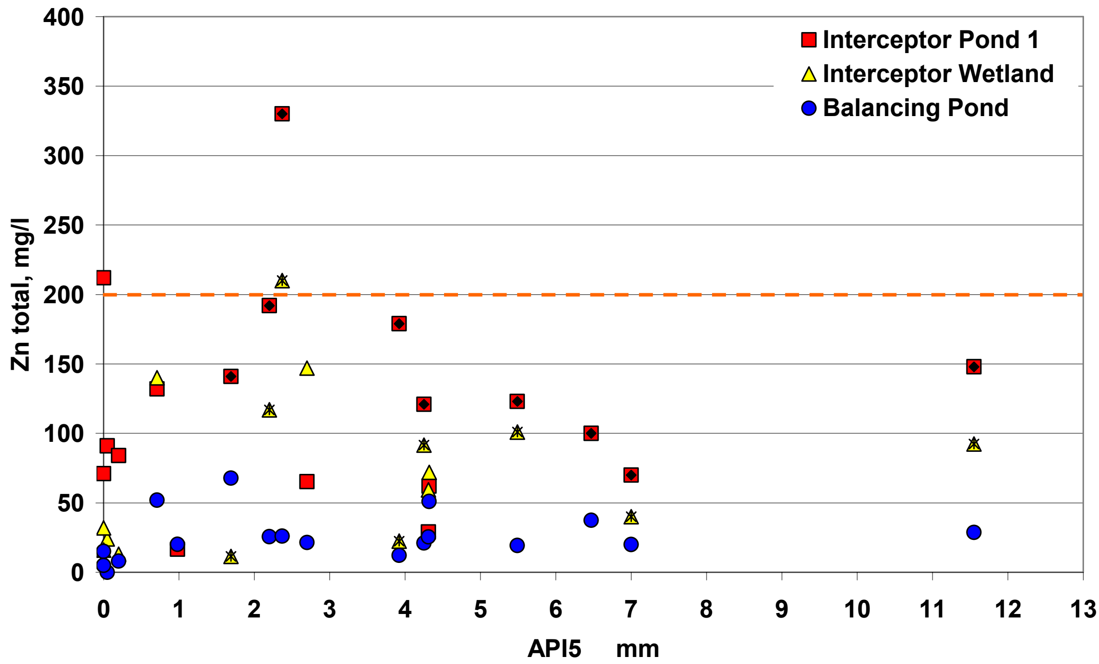


Figure 5.48 Relationship between total Zn concentrations and API5

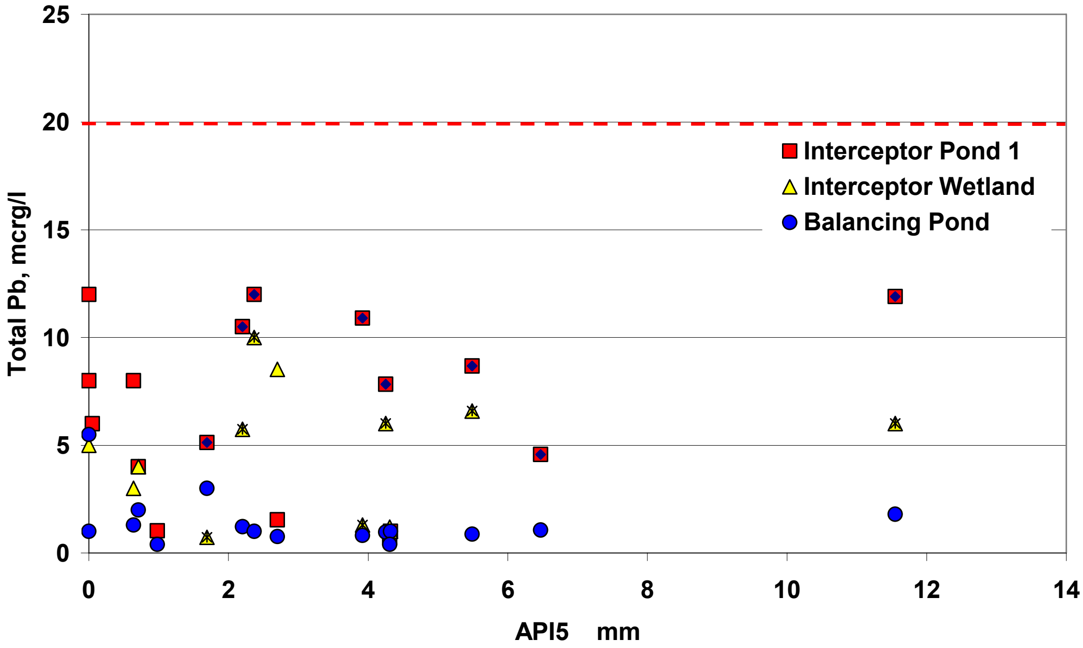


Figure 5.49 Relationship between total Pb concentrations and API5

5.7 CONCLUSIONS

Hydraulic performance

- Significant reductions were observed in peak flows through the treatment train. For small events, and following dry periods, these reductions were greater than anticipated i.e. flow was 'lost' from the system, which is a benefit in terms of protection of the receiving waters from pollutant loads.

- For design events and high API5 (i.e. wet pre-event conditions), the percentage of the rainfall event retained by the system is low. It is therefore appropriate that, for design events, no account should be taken of the system losses that are observed for small events and following dry periods.

- Although 70% of flows at the outfall of the upstream conventional drainage network were greater than the predicted 2 year greenfield flow threshold, only 5% of flows from the outfall of the swale reached this level. Additional balancing was then achieved in the downstream balancing pond. This shows that SUDS systems can provide the means by which the objective of achieving greenfield site runoff conditions can be met.

- Flow velocities in the wetland swale were all found to be less than 0.3m/s, meeting conveyance and water quality design criteria.

- The systems perform better (in terms of attenuation) in summer, when water levels drop and vegetation levels increase.

Water quality performance

- For 15% of the time (during large storm events), water quality at the outlet of the treatment ponds is likely to be influenced by water quality of the inflow to some degree;

- For 85% of the time, water quality at the outlet of the treatment ponds will effectively be pre-determined by the quality of the permanent pool volume.

- In summer, 50% of inflow events are retained for more than 6 days, with 25% being retained for 12 days. This compares to recommended detention times of 14 to 21 days (CIRIA, 2000).

- The residence time in the ponds at Hopwood was shown to influence the concentrations of some nutrients and TSS loads. Residence times of greater than 3-5 days are seen to significantly increase the concentration of NH_4-N in the SUDS units in the upper parts of the treatment train. Residence times are dependent on the characteristics of the rainfall time series (which will vary with geographical location), and on the size of the permanent volume. Due to the seasonal characteristics of the precipitation, residence times will show seasonal variability and smaller permanent pond volumes will allow a more rapid exchange of water.

- At Hopwood, antecedent precipitation index (API_5) was shown to be a useful indicator for certain pollutant parameters. It is likely that it could also be used as an indicator for prediction of water quality performance more generally, however research at other sites would be needed to confirm this. For Hopwood, an API_5 of less than 3 mm correlates with an increase in NH_4-N, BOD_5 and TSS concentrations. Consideration of seasonal rainfall characteristics is therefore important.

- Observed concentrations of metals (when compared to current water quality standards such as the Environment Agency Class 2 Rivers) are low, especially across the last units of the SUDS treatment train. The metal concentrations have a direct relationship with suspended solids, which could permit prediction of the loads of heavy metals, knowing the concentration of suspended sediments. The predominant form of the heavy metals is the particulate one (via adsorption), so the main process of removal should be sedimentation rather than up-take of dissolved forms from living microorganisms.

- In general, the water quality improves along the treatment train. The results emphasise the advantage of using a treatment train (i.e. SUDS units in series) over the use of single SUDS units.

Monitoring

- Event-based auto-sampling requires regular staff attendance on site and accurate knowledge of likely site weather conditions so that samples can be collected for analysis at short notice. Auto-sampling requires an appropriate trigger (either flow depth or rainfall) and becomes increasingly difficult where monitored flows are both low and unpredictable. Management costs of such an approach are high.

6. Permeable Pavement, Kidderminster

6.1 SITE DESCRIPTION

This site comprises a carpark surfaced with Formpave block pavers over the parking spaces and asphalt roadways, draining to the surface of the pavement. Infiltrating water is collected within the granular sub-base which filters and attenuates the runoff, providing water quality treatment and flow management prior to discharge, via an oil interceptor, to the receiving River Stour. The site is approximately 1 hectare in size.

A photo of the permeable pavement system is given in the figure below (Figure 6.1) and a plan of the site is presented as Figure 6.2.

Figure 6.1 Photo showing the permeable pavement system at Tesco's, Kidderminster (courtesy of Formpave Ltd.)

6.2 MONITORING STRATEGY

The monitoring objectives for the site were:

1. To assess hydraulic and water quality performance of a Formpave permeable pavement;
2. To determine influence of season, time and maintenance on performance.

These objectives were to be fulfilled through the following activities:

- Monitoring rainfall;
- Monitoring flow rates;
- Modelling hydrologic / hydraulic response;
- Monitoring water quality;
- Monitoring surface infiltration rates.

A raingauge was installed on the roof of the retail store, and an electromagnetic flow monitoring device was installed in the outlet pipe, just upstream of the oil separator. The location of the device is shown on Figure 5.2.

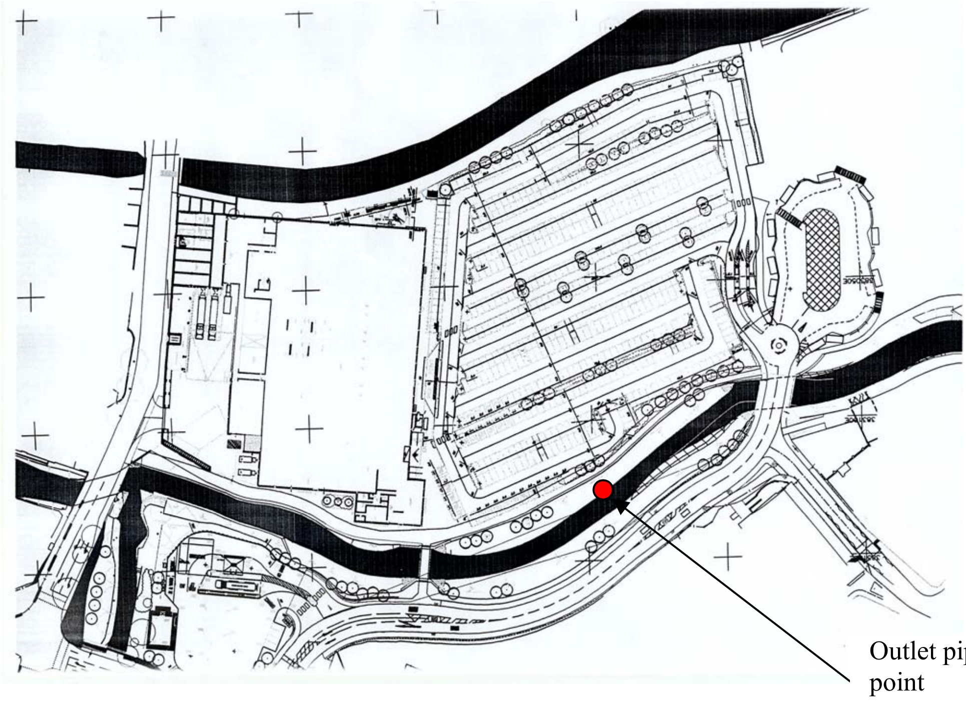


Figure 6.2 Plan of Tesco Formpave Site, Kidderminster showing flow monitoring site

6.3 MONITORING EQUIPMENT AND PROGRAMME

The rainfall and flow measurement devices were the same as described in Section 4.2, for Hopwood Services. Initially the flow discharge was monitored using the Buehler Montec Xytec 7050 flow logger. However extremely low outflows from the car park were recorded during the initial period (Autumn/Winter 2002) and a water balance analysis indicated significant losses. It was felt that this was likely to be due either to an inappropriate range of conditions for the flow logger (low depth, low velocity of discharge), leakage around the flow monitor, or to the fact that the car park was not properly lined so that a proportion of the runoff was infiltrating into the ground. The monitoring equipment was removed and re-calibrated, re-configured, and installed in a new position to reduce the risk of leakage – with no improvement in the results.

By mid summer 2003, the monitoring was abandoned and concerted attempts were made, through liason with Formpave, to secure an alternative monitoring site for the remainder of the study period, but to no avail. A decision was made to return to Kidderminster and make revisions to the monitoring device.

In January 2004, a V-notch metal plate weir was installed in the pipe discharging to the river. The V-notch was calibrated in the hydraulic laboratories at HR Wallingford. The transducer head of the flow logger was used for measuring the water depth in front of the V-notch, and flows were calculated according to the calibration stage-discharge curve. The recorded discharges from the site were still very low. In addition to the very low recorded flows, there was evidence that shortly after heavy rain events, the discharge from the car park was impeded by backflow from the river. Having established that the low outflow from the car park was not an error of monitoring, it was concluded that the liner was not providing an effective seal and that the majority of the rainfall runoff was passing to ground. This led to the monitoring being abandoned and monitoring equipment being removed at the end of July 2004.

6.4 CONCLUSIONS

- Careful selection is needed when securing an appropriate SUDS site for monitoring purposes, particularly if sites are not owned and/or operated by the funding body for the monitoring campaign. Particular difficulties that may need to be overcome include:
  a) reluctance of site owners / operators to sanction monitoring due to concerns over potential failures that may be observed with respect to system performance. SUDS are generally perceived as new and uncertain technology and owners may be concerned over potential future problems should the performance be found not to match design expectations;
  b) poor designs and/or construction of the SUDS system;
  c) complexities of collecting data from systems not designed with data collection in mind;
  d) complexities, interference and nuisance associated with installing monitoring equipment post construction; and
  e) risks from vandalism.

- Outfall discharge rates from SUDS systems (especially permeable pavements where filtration/storage volumes are high) can be zero or very low for long periods and appropriate flow measurement devices should be selected with this in mind. Increased equipment maintenance is required for these conditions.

7. Retention Pond, Thorpe Business Park, Leeds

7.1 SITE DESCRIPTION

A study of a balancing pond within a large new business park was undertaken by the Pennine Water Group (PWG) at Bradford University (partners in the project). A number of ponds were to be constructed on the development site, with two completed at the time that the monitoring campaign was initiated. The monitored pond (shown above) has been operational since 1999. It is designed to take stormwater from the developed area at the western edge of the site, to include surface runoff and roof water via on-site private drainage. The areas most recently developed, and for which the pond was built, comprise a total of 8.25ha impermeable and 1.43ha permeable, increasing to a total impermeable area of 13.35ha when the upstream drained area is connected. The developers agreed with the EA and Leeds City Council that the pond would limit pass forward flow to the value of pre-development flow to the watercourse and provide attenuation storage for the excess water generated by the development.

A photo of the pond is shown in Figure 7.1.

Figure 7.1 The Balancing Pond at Thorpe Park

7.2 MONITORING STRATEGY

The monitoring objectives for the site were:

1. To assess hydraulic and water quality performance of the balancing pond;
2. To determine influence of season, time and maintenance on performance.

These objectives were to be fulfilled through the following activities:

- Monitoring rainfall;
- Monitoring flow rates;
- Monitoring water quality.

7.3 MONITORING EQUIPMENT

Prior to initiating any quality performance studies, the hydrology and hydraulics of the pond were investigated by installing a recording rain gauge and flow monitors for the inlets and outlet. A 0.2mm tipping bucket raingauge was installed on the roof of an adjacent building and flow monitors were installed at each inlet and outlet. This was to check the inlet and outlet flow characteristics and to assess the flow attenuation through the pond. Weir plates were constructed for each of the inlet channels and pipes to cover the flow ranges expected and to provide sufficient upstream head to monitor the flows, as illustrated in Figure 7.2. Buhler-Montec units with Doppler velocity meters and pressure transducers for depth were installed upstream of the weir plates. These were installed by the University of Bradford in September 2001.

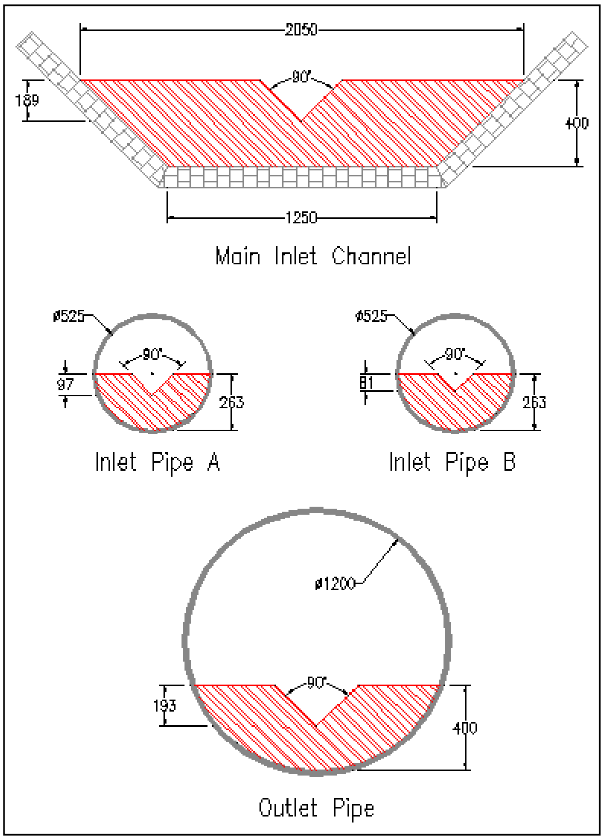


Figure 7.2 Weir plates designed for implementation of flow monitoring at Thorpe Park

7.4 MONITORING PROGRAMME

Despite considerable effort by PWG in trying to monitor flows from September 2001 for a year, problems related to the reliability of flow monitors (sediment obscuring sensors, sensor malfunction and vandalism) precluded the collection of any consistent data for any duration longer than a week. In addition, it was apparent that significant, and increasing, infiltration was entering the pond through the southern embankment (see Figure 7.3) and an additional inflow point, close to the pond outlet, draining runoff from a local highway was also identified. Hence the attempts to monitor the flows had to be abandoned and in December 2002 the monitoring equipment was removed and it was decided to seek alternative site options for monitoring. However, for various reasons none of the options pursued resulting in any information that was of benefit to the project.

Figure 7.3 Photo of Thorpe Park Balancing Pond showing location of infiltration zone

7.5 COMPUTATIONAL FLUID DYNAMIC MODELLING FOR BALANCING POND PERFORMANCE ANALYSIS

Dr Virginia Stovin of the PWG at Sheffield University undertook a preliminary desktop investigation into the potential hydraulic performance of the pond using computational fluid dynamic (CFD) modelling. The study made use of the Fluent 5 CFD software. The model was based on the WSP engineering drawings of the pond and used a 3D unstructured mesh comprising approximately 400,000 elements. A schematic of the pond showing the bathymetry is shown in Figure 7.4. Flow rates for the three inflows were set at 176.15 l/s (main upstream), 30.1 l/s (secondary upstream) and 18.75 l/s (minor downstream), and the water surface level was 78.4 m AOD. These values corresponded to the peak flow rates expected in response to a 1 in 30 year storm event.

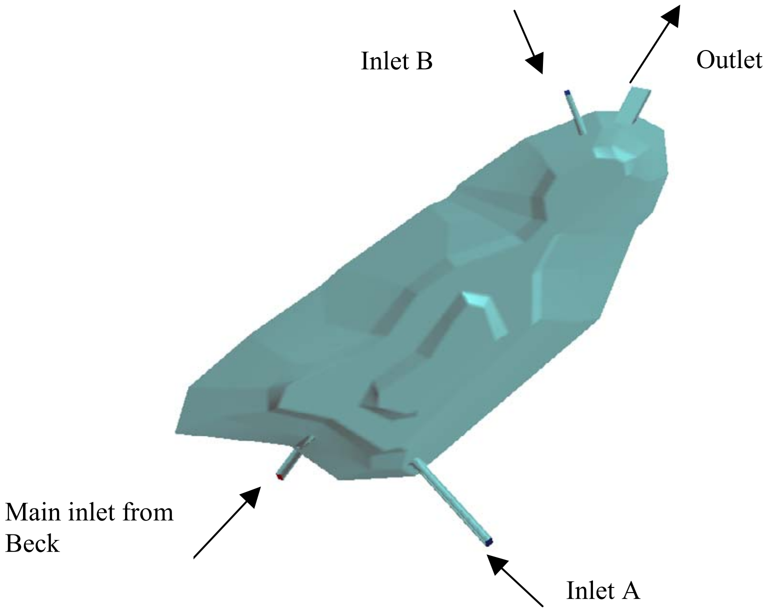
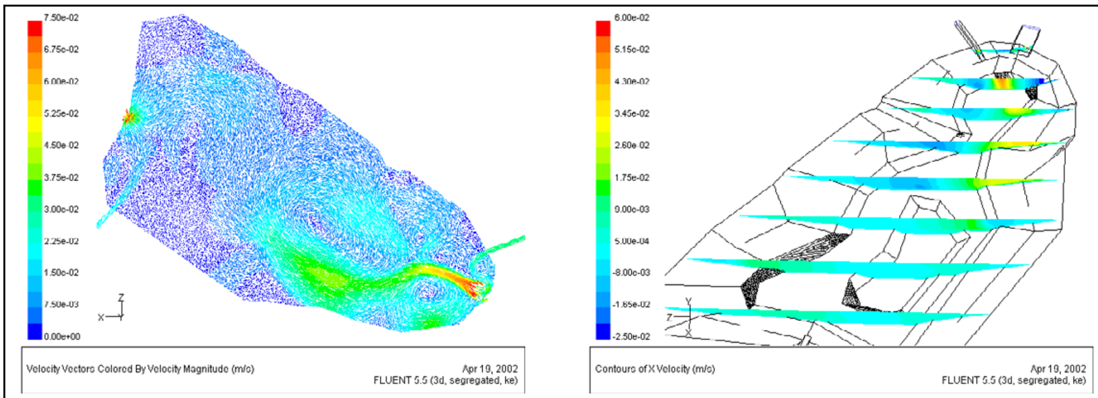


Figure 7.4 Schematic layout of pond showing inlets, outlet and bathymetry

Figures 7.5 and 7.6 show the predicted flow patterns. 3D effects, with circulations returning towards the main inlet, are evident. The hydraulic characteristics of the pond affect its ability to retain (treat) polluted inflows. Figure 5 shows particle tracks corresponding to the paths taken by fine sediments suspended in the two secondary inlet flows. The inevitable short-circuiting between the Inlet (B) and the pond outlet is clearly illustrated.



a) Surface velocity patterns b) Contours of X-Velocity

Figure 7.5 Simulated flow field in the Thorpe Park pond

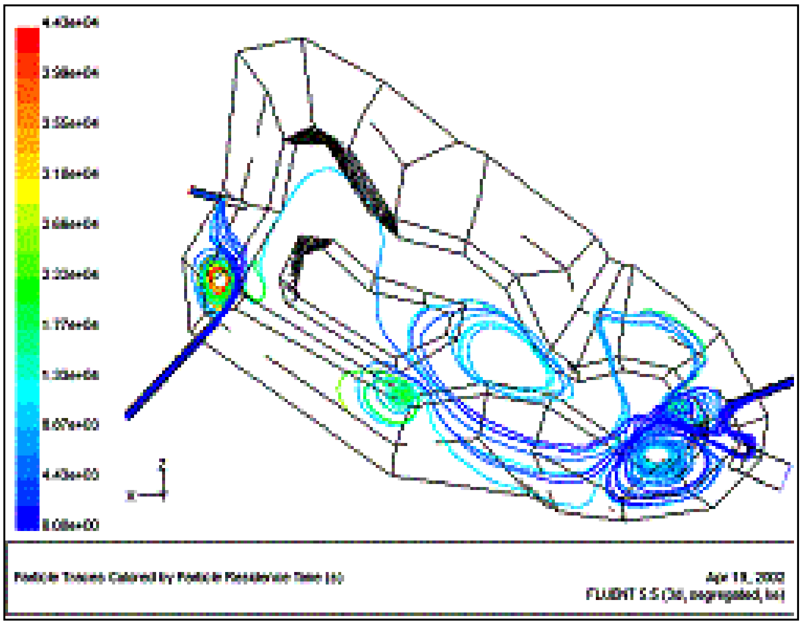


Figure 7.6 Particle tracks originating from the two secondary inlets

7.6 CONCLUSIONS

- Vandalism is a major potential problem for open sites such as ponds. Detailed pond monitoring may require prolonged measurements of in and outflows, and whether in a residential or business area, the equipment is difficult to protect from vandalism. Ideally, monitoring chambers should be constructed specifically to secure equipment. Sites remote from housing areas may offer lower risk solutions.

- Flow circulation patterns in ponds are amenable to computational fluid dynamic (CFD) modelling. Although at Thorpe Park a prototype study was carried out without on-site verification, the CFD modelling demonstrated the valuable role that CFD may have in understanding the behaviour of SUDS retention ponds.

8. *Permeable Pavement, Wokingham*

8.1 SITE DESCRIPTION

The Tesco superstore is situated on the outskirts of Wokingham (Finchampstead Road). The site of approximately 2.5 hectares includes a superstore, a permeable asphalt car park, a green area, impermeable roads and a pond which receives the outflow from the pavement system. The car park is approximately 1 hectare in size. The site is generally flat, with a slight fall towards the receiving watercourse that runs along the northern boundary. This watercourse is a tributary of the Emm Brook.

The site was initially constructed in 1997-1998. Remedial works to the pavement were undertaken during December 2001 to increase the infiltration of the water into the sub-base, as there had been problems with surface ponding in some areas.

The drainage network comprises:

- The roof of the superstore which drains directly to the watercourse via a siphonic system;
- A part of the road network which joins the outfall via a conventional drainage system;
- The road within the car park which is cambered to direct water onto the permeable surface;
- The permeable pavement.

The layout of the site drainage system is shown in Figure 8.1. A photograph of the pavement is given in Figure 8.2, and Figure 8.3 shows the downstream wetland.

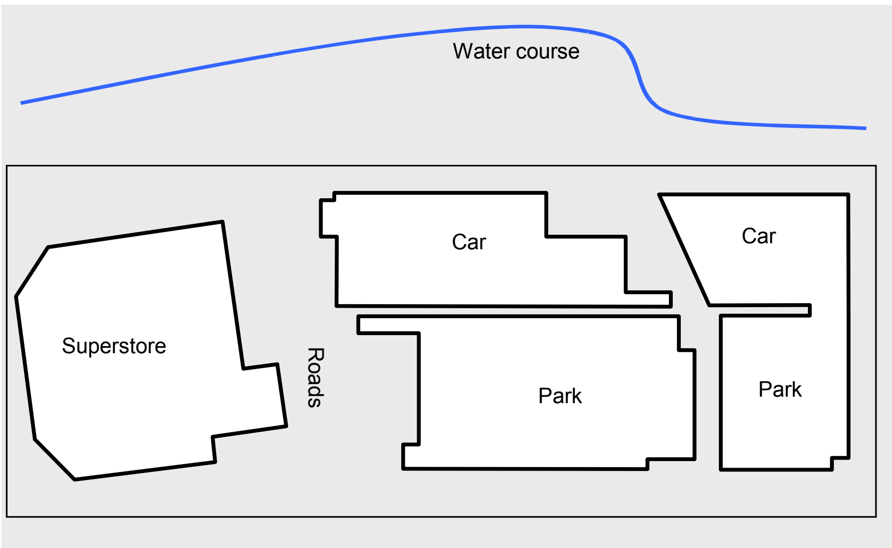


Figure 8.1 The Site Drainage System, Tesco's, Wokingham

Figure 8.2 Photograph of permeable pavement at Tesco, Wokingham

Figure 8.3 Photograph of balancing wetland at Tesco, Wokingham

The permeable car park is composed of three layers. The permeable asphalt drains the water into the granular sub-base. The sub-base material had been seeded with bacteria in order to accelerate the break-down of hydrocarbons. An impermeable geomembrane seals the car park to prevent infiltration. The underlying soil was historically part of an industrial site, so it was necessary to avoid mobilising the pollution.

A schematic of the three layers is given in Figure 8.4. The total pavement depth is approximately 0.5 m.

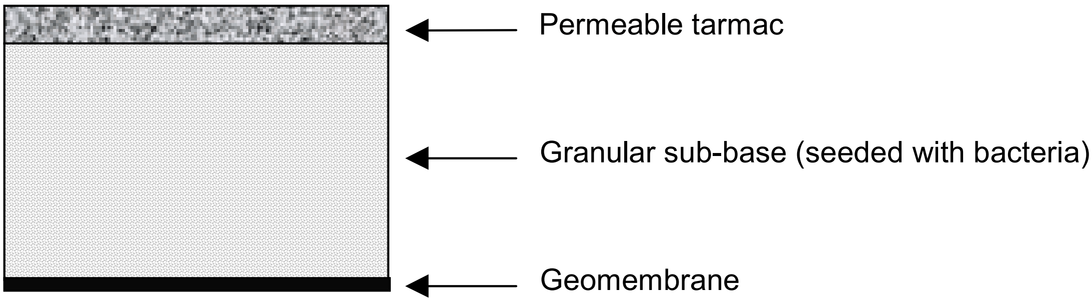


Figure 8.4 Permeable Car Park Sub Layers

To maximise the attenuation of water, the car park is subdivided into eight zones by internal dams located within the sub-base. A perforated pipe drains each zone and discharges the water in the sub-base of the zone located downstream. There are two such schemes in parallel, as shown on the following figure (Figure 7.5). The last zone discharges the water into a perforated pipe which outfalls to the wetland. This wetland provides final polishing and balancing of the flow prior to discharge into the adjacent stream.

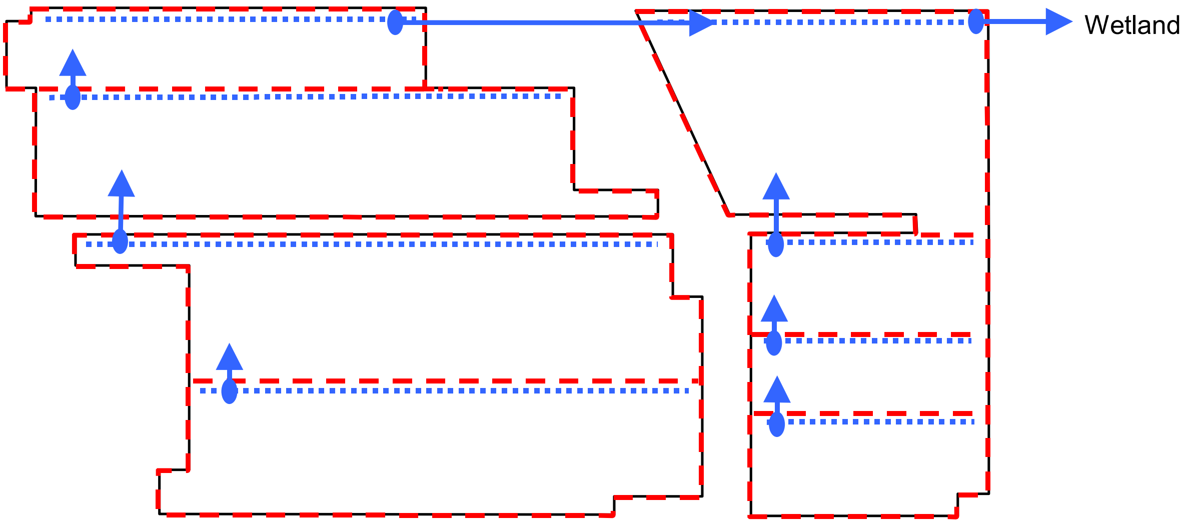


Figure 8.5 Drainage Flow Paths within the Permeable Car Park

8.2 MONITORING STRATEGY

The monitoring objective for the site was:

1. To assess the hydraulic performance of the permeable pavement.

This objective was to be fulfilled via:

- Monitoring rainfall data;
- Monitoring outfall discharge rates;
- Monitoring water levels in the pavement.

8.3 MONITORING EQUIPMENT AND PROGRAMME

Monitoring at the site was undertaken by the Environment Agency from January 2000 to early June 2004.

Pressure transducers, placed into two piezometers, recorded the level of the water in the sub-base at two points (called "near" and "far" from the store). There is data from 07/2000 to 01/2001 at a 15 minute frequency, and from 01/2000 to 12/2001 at a 24 hour frequency. Unfortunately, the piezometer furthest from store had operational problems and the data could not be relied upon.

The rainfall was monitored with a 0.2mm tipping bucket rain gauge located on a school roof near the site. The outflow from the pavement system was measured using a V-notch weir in a manhole located on the outfall pipe of the system to the wetland. There are some parts of the recorded dataset that are unreliable and others where data is missing. Table 8.1 summarises all the collected data and gives a short description of data availability.

Table 8.1 Hydrologic Monitoring Programme Data Availability: Tesco's, Wokingham

Period	Rain gauge	Outflow
01.01.00-31.05.01	Reliable data	Reliable data
01.06.01-15.06.01	Reliable data	Not reliable data
16.06.01-18.06.01	Reliable data	Reliable data
19.06.01-06.07.01	Reliable data	Not reliable data
07.07.01-21.07.01	Reliable data	Reliable data
22.07.01-31.07.01	Reliable data	Not reliable data
01.08.01-24.08.01	Reliable data	Reliable data
25.08.01-02.09.01	Reliable data	Not reliable data
03.09.01-08.09.01	Not reliable data	Reliable data
09.09.01-28.09.01	Not reliable data	Not reliable data
29.09.01-05.11.01	Not reliable data	Reliable data
06.11.01-07.12.01	Reliable data	Reliable data
08.12.01-14.01.02	Reliable data	Not reliable data
15.01.02-31.03.02	Reliable data	Reliable data
01.04.02-25.04.02	Reliable data	Not reliable data
26.04.02-17.07.02	Reliable data	Reliable data
18.07.02-31.07.02	Reliable data	Not reliable data
01.08.02-25.09.02	Reliable data	Reliable data
26.09.02-12.10.02	Reliable data	Not reliable data

Table 8.1 Hydrologic Monitoring Programme Data Availability: Tesco's, Wokingham (continued)

Period	Rain gauge	Outflow
13.10.02-30.11.02	Reliable data	Reliable data
01.12.02-14.01.03	No data	Reliable data
15.01.03-22.04.03	Reliable data	Reliable data
23.04.03-24.04.03	Reliable data	Not reliable data
25.04.03-08.07.03	Reliable data	Reliable data
09.07.03-16.07.03	Reliable data	Not reliable data
17.07.03-20.07.03	Reliable data	Reliable data
21.07.03-24.07.03	Reliable data	Not reliable data
25.07.03-01.08.03	Reliable data	Reliable data
02.08.03-27.08.03	Reliable data	Not reliable data
28.08.03-30.08.03	Reliable data	Reliable data
31.08.03-31.10.03	Reliable data	Not reliable data
01.11.03-08.11.03	Reliable data	Reliable data
09.11.03-11.11.03	Reliable data	Not reliable data
12.11.03-15.11.03	Reliable data	Reliable data
16.11.03-20.11.03	Reliable data	Not reliable data
21.11.03-08.01.04	Reliable data	Reliable data
09.01.04-05.04.04	Reliable data	Reliable data
06.04.04-26.05.04	No data	Reliable data
27.05.04-31.05.04	No data	Not reliable data
01.06.04-03.06.04	No data	Reliable data

The flow monitoring equipment is located in a manhole immediately upstream of the wetland, shown in Figure 8.6.

Figure 8.6 Flow Monitoring Equipment, Tesco's, Wokingham

8.4 DATA ANALYSIS AND INTERPRETATION: HYDRAULICS

8.4.1 Analysis objectives

- To review and analyse actual system performance;
- To assess the capability of an existing urban drainage software to represent SUDS response;
- Recommend software improvements to enable permeable pavements to be better represented;
- To use a calibrated model to predict likely system performance to design events.

8.4.2 Approach

The assessment of the performance of the system focused on the relationship between runoff volumes and rainfall events, and the rate of flow discharged.

The response of the system to the very significant period of rainfall that fell during November 2000 is shown in the following figure (Figure 8.7). This shows that outflows were limited to below 7 l/s/ha throughout this period.

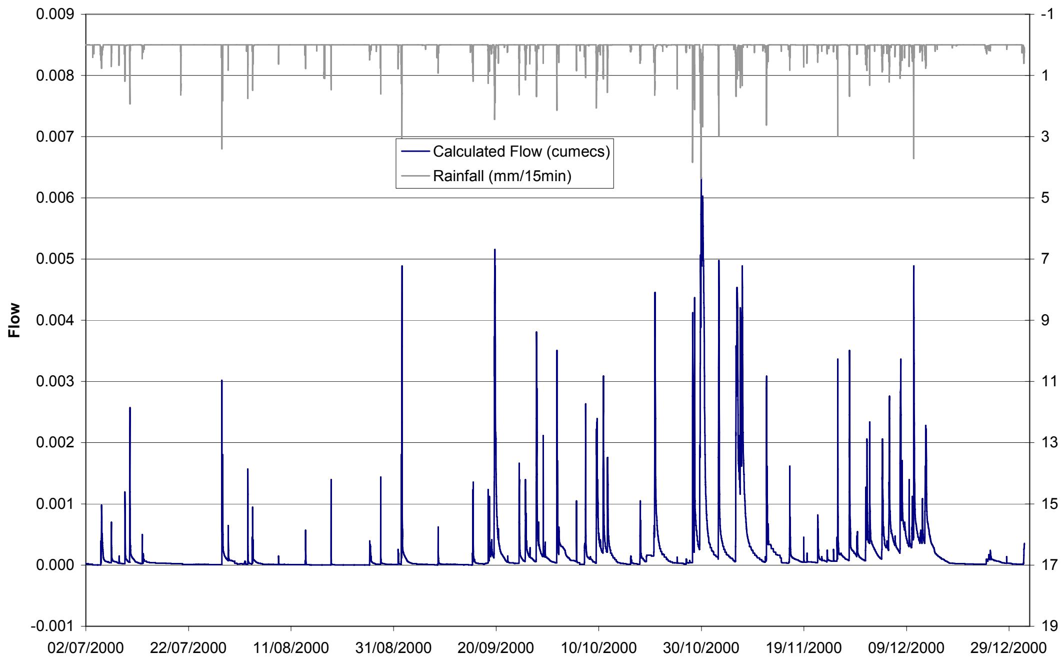


Figure 8.7 Hydrometric records at Tesco's, Wokingham during November 2000 event

8.4.3 Prevented events

330 rain events of more than 0.5 mm were recorded between January 2000 and April 2004. Events were defined as rainfall occurrences, when they were separated by an interval of at least 6 hours. From these 330 events, 47 did not give any measured outflow. The permeable system therefore prevented outflow from up to 25% of the rain events in summer and 15% in winter.

The graph below shows the distribution of the percentage of prevented events through the year:

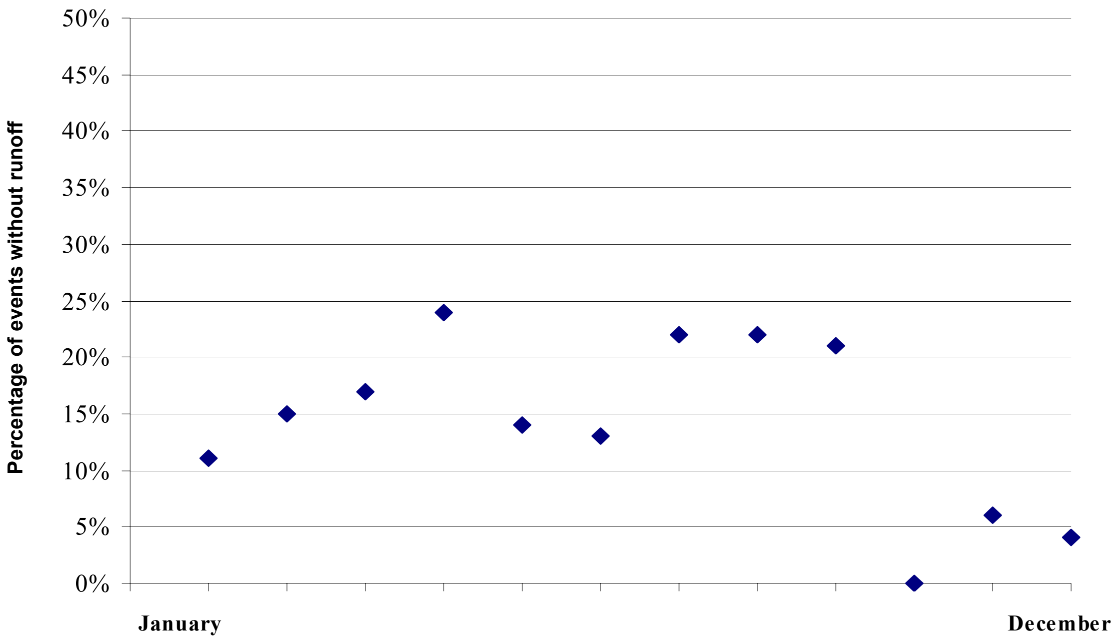


Figure 8.8 Percentage of prevented events through the year (average 2000 – 2004)

In a conventional system, all events are likely to cause runoff. In the SUDS system, 'suppressed' events are more likely to occur following periods of dry weather. It is these same events that in a conventional system are likely to have a high pollution risk due to the buildup of silts and other contaminants on the runoff surface. In addition, the receiving watercourse may be at greater risk during the summer when baseflows are low and the dilution potential for pollutants is reduced.

8.4.4 Outflow percentage (volume reduction)

17 rain periods (of durations between 2 days and 27 days) were selected for analysis. The percentage outflow of these events was found to range between 19% and 93% with an average of 61%.

Figure 8.9 shows how the percentage outflow tends to vary through the year:

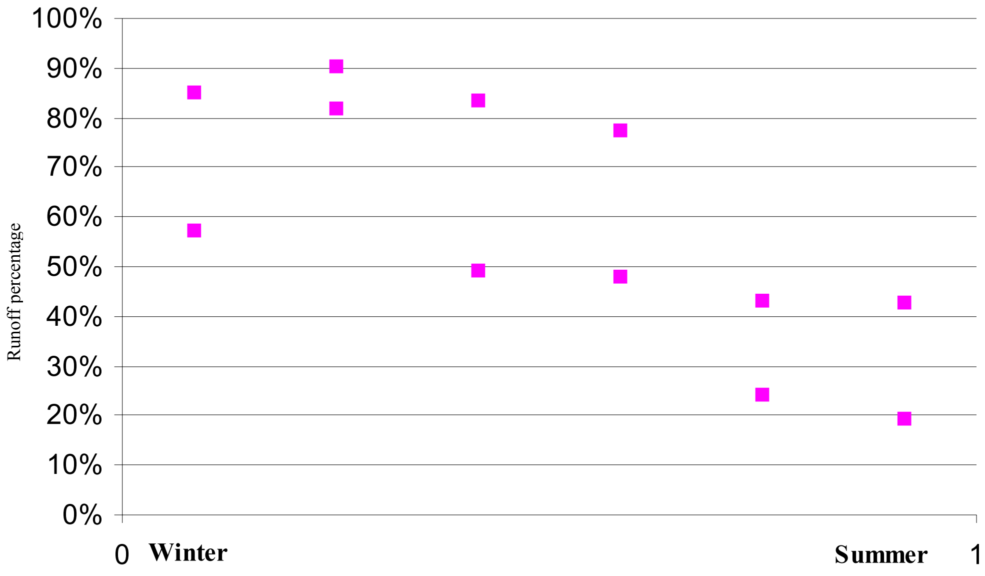


Figure 8.9 Outflow percentage through the year

The percentage of runoff is significantly dependent on the rainfall depth of the storm event as can be seen in Figure 8.10.

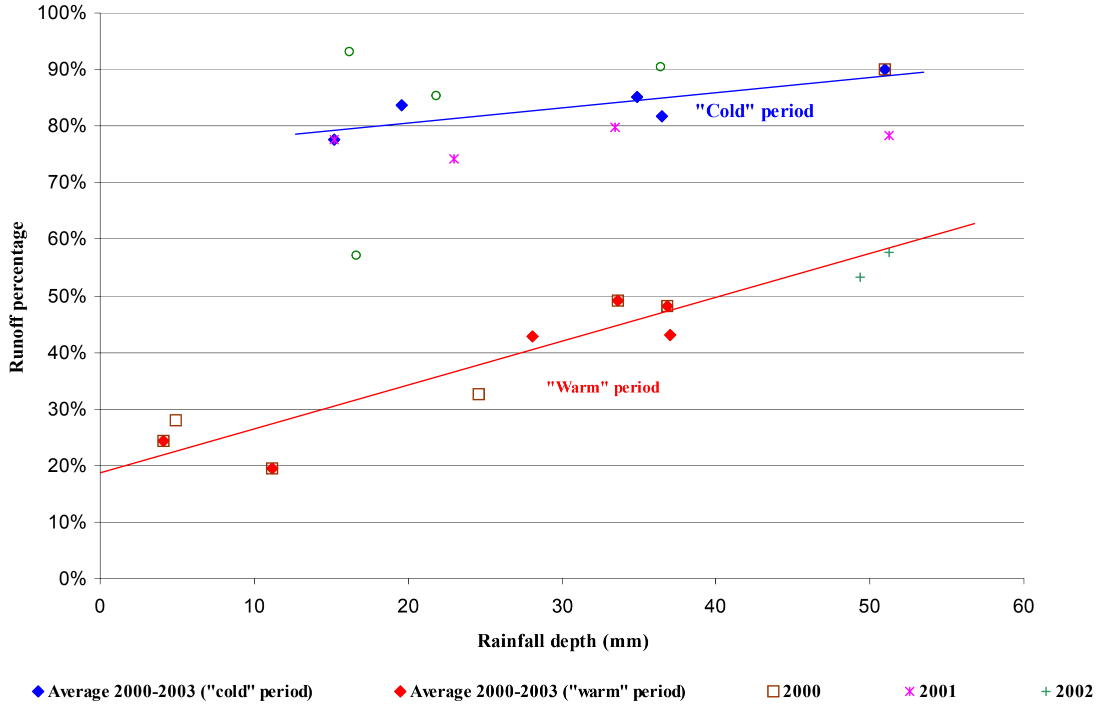


Figure 8.10 Outflow percentage as a function of event rain depth

For a conventional system, the percentage runoff is generally between 60 and 90%.

8.4.5 Lag times

The lag times have been calculated for the 17 selected rain periods. The time delay between the beginning of the rain and the rise in flow discharge varies between 15 and 120 minutes with an average of 44 minutes. Although the lag time tends to increase with increased antecedent dry periods, no clear relationship could be established.

8.4.6 Peak rates of outflow

The statistical distribution of peak outfall discharge rates recorded at the outfall from the site were assessed. A frequency curve is presented in Figure 8.11, which was derived using a peaks over threshold (PoT) analysis.

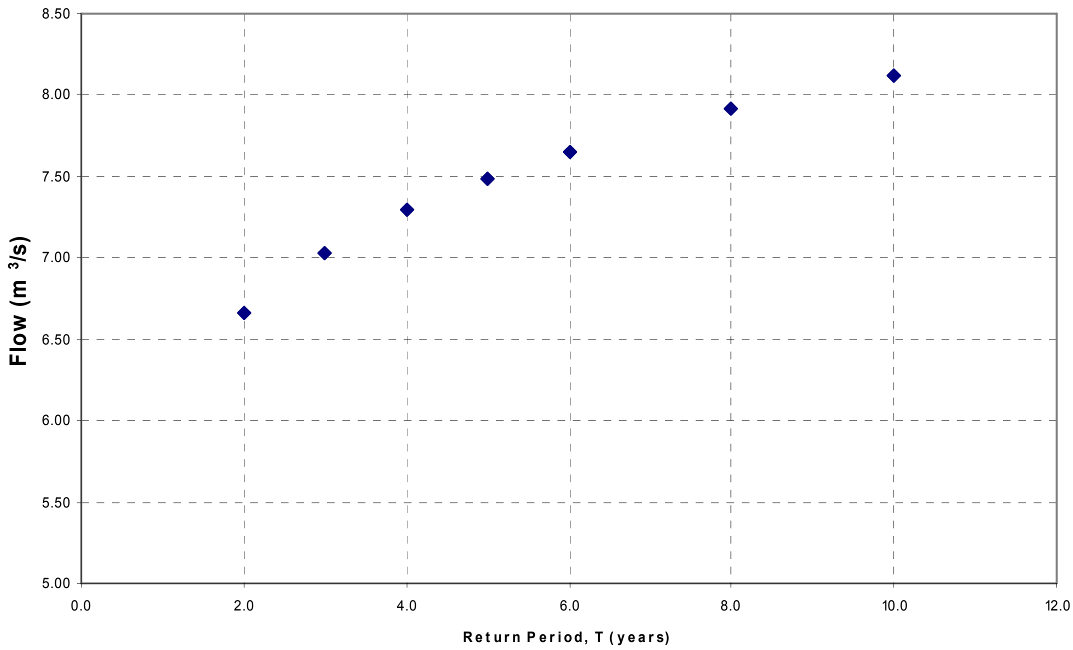


Figure 8.11 Frequency curve for the system outflow calculated with a PoT analysis

In order to assess the differences in peak discharge rate when compared to a conventional network, modelling was used, and this analysis is presented in Section 8.5.

8.4.7 Comparison with other permeable pavement monitoring datasets

A review of data from literature allowed the following performance comparison to be made.

Table 8.2 Performance of monitored permeable surface

Site	Type of permeable surface		% outflow	% of prevented events	Peak flow rate reduction	Lag time
NATS, Edinburgh	Block paving			61% (>0.8mm)	76.8%	Average : 181.5 min
Nottingham test site (lined)	Block paving with sub-base:	Furnace slag Limestone Gravel Granite	55% 61% 63% 75%			
Wheatley	Block paving		67%		87.6%	Less than 5 min up to 2 hours
Royal Bank of Scotland, Edinburgh (lined)	Block paving		46.5%			Between 42 and 143 min
Wokingham (lined)	Asphalt		50%	41% (>0.5mm)	81%	Between 15 min and 2 hours Average : 1 hour

The observed peak flow rate reductions and the lag times found at Wokingham are similar to those found in the literature review data.

8.5 MODELLING OF THE WOKINGHAM SYSTEM USING INFOWORKS CS

8.5.1 Representation of the network

Pervious pavements exhibit a complex hydraulic behaviour. This reflects the mechanisms associated with the rainwater passing through the blockwork or porous asphalt, the passage of water through the goetextile and underlying stone media, and the collection system being used (e.g. a grid of perforated pipes or a single collector pipe along one edge). In addition, there are lateral hydraulic gradients that are created within the subsurface reservoir, which are influenced by the pipe collection system, the pavement's geometrical characteristics, the pavement depth and the storage media. The carpark at the Wokingham Tesco site comprises a porous asphalt, and granular sub-base reservoir structure with a small proportion of the outflow resulting directly from impermeable road runoff which constitutes 5% of the contributing area. Figure 7.8 clearly shows the dual behaviour of system i.e. a proportion of fairly rapid response together with a significant period of slower, attenuated pavement discharge.

At present, the design of pervious pavement models tends to use a voids ratio for a structure the same shape and size as the pervious pavement, with a throttle (often a small orifice) to limit the outflow. However, modelled in this way, the simulated performance is unlikely to reflect the actual hydraulic behaviour of such units. InfoWorks CS was therefore applied by using observed data recorded in the field to calibrate the infiltration runoff module.

This module uses three separate storage functions, as described in the following table:

Table 8.3 Description of Infoworks CS Storage Components

Model Storage	Description
1. Depression storage	Incident rainfall is initially stored in surface depressions, which are subject to evaporative losses. When rainfall exceeds depression storage in a given time step, a proportion of the excess rainfall goes to runoff. The remaining rainfall is directed into the soil storage reservoir.
2. Soil reservoir	When the soil reaches a given saturation threshold (the percolation threshold), water starts to percolate downwards. A proportion of this percolation flow infiltrates directly into the sewer network, the remainder penetrates deeper to feed the groundwater storage reservoir.
3. Ground reservoir	When water in the groundwater storage reservoir reaches a particular threshold water, loss due to baseflow occurs. When the groundwater level reaches a further threshold, groundwater infiltration into the network occurs.

The series of storages is also shown in the following figure:

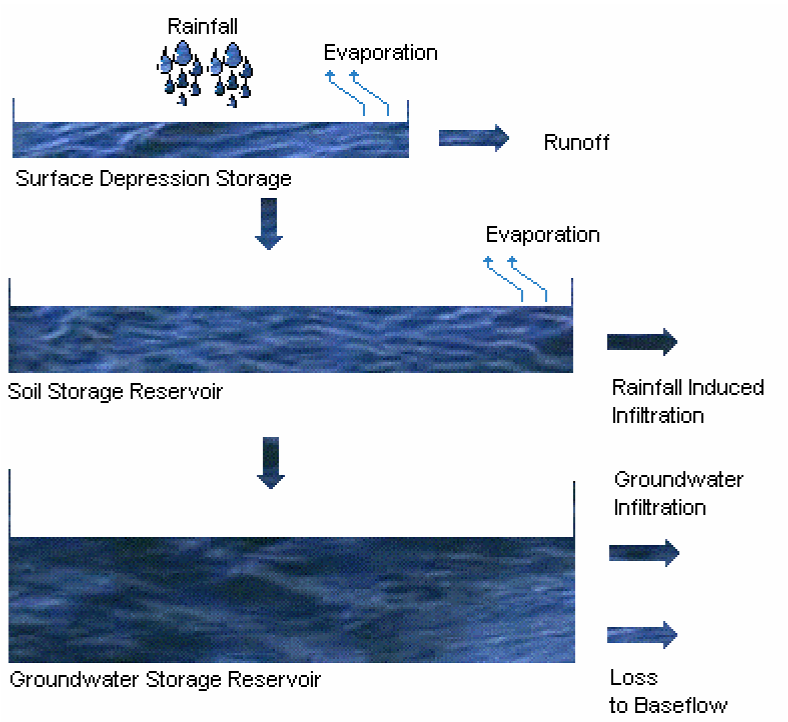


Figure 8.12 Infoworks CS Storage Models

This site could not easily be represented within current urban drainage software. It was difficult to represent the subdivision of the sub-base into zones, and the transfer of the water between adjacent zones. The selected model representation involved the creation of two permeable subcatchments with the condition that water must flow through the sub-base prior to discharge to the outflow pipe. Several configurations were modelled and gave very similar results. The network included three subcatchments in total (the third representing the impermeable road runoff), as shown in the following figure:

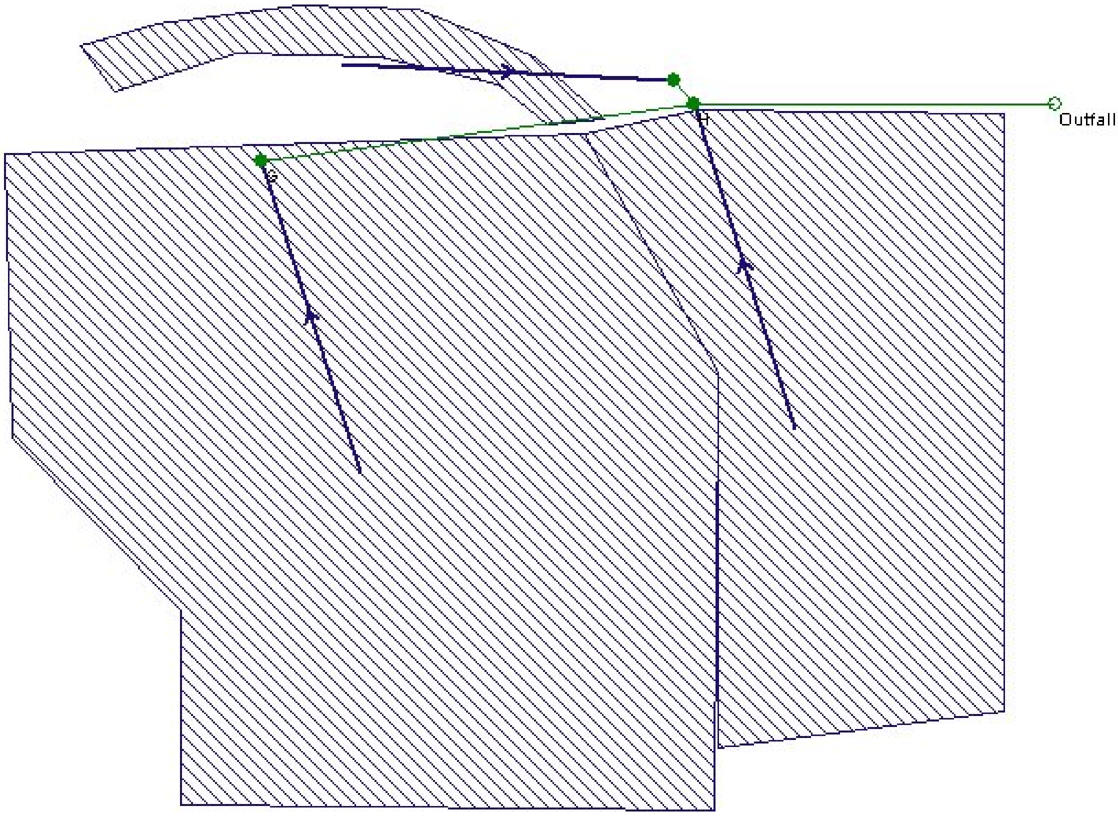


Figure 8.13 InfoWorks network

Each subcatchment was drained to a node (manhole). The nodes were joined by pipes of 150 mm diameter which convey water to the outfall. The cambered impermeable road which drains onto the permeable surface was represented by a runoff surface with 80% impervious and 20% pervious characteristics.

8.5.2 Calibration

Model calibration was undertaken using 8 selected events between January 2000 and August 2001. The events included all the high flow periods, and varied in length between 3 days and 7 weeks.

The calibration method involved:

- Modification of each model parameter to check its influence;
- Finding a set of values giving a reasonable fit to observed discharges;
- Evaluation of the model fit for each event (volume ratio, fit to peak, lag time, etc.),
- Review of the effect of antecedent conditions and season on these parameters,
- Application of a calibration parameter correction to offset the influence of the season and antecedent conditions.

The final set of parameters for the calibrated model and the 8 calibration plots are presented in Appendix 2. An example calibration plot is given in Figure 8.14 below.

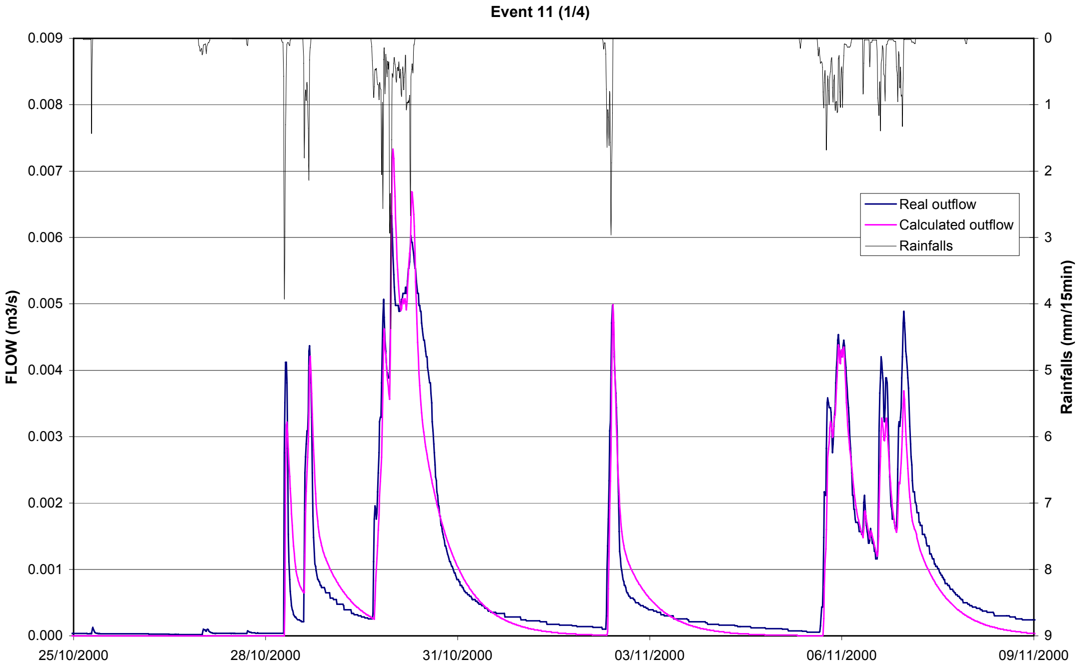


Figure 8.14 Model Performance for November 2000 Event

8.5.3 *Model performance: Flow volume*

On average, the absolute value of the difference between measured and modelled flow volumes is 13.9%. The distribution of these differences is shown in the following graph (Figure 8.15). The volume 'ratio' is calculated as follows:

$$\text{Volume ratio} = \frac{\text{Modelled Volume} - \text{Observed Volume}}{\text{Observed Volume}} \times 100$$

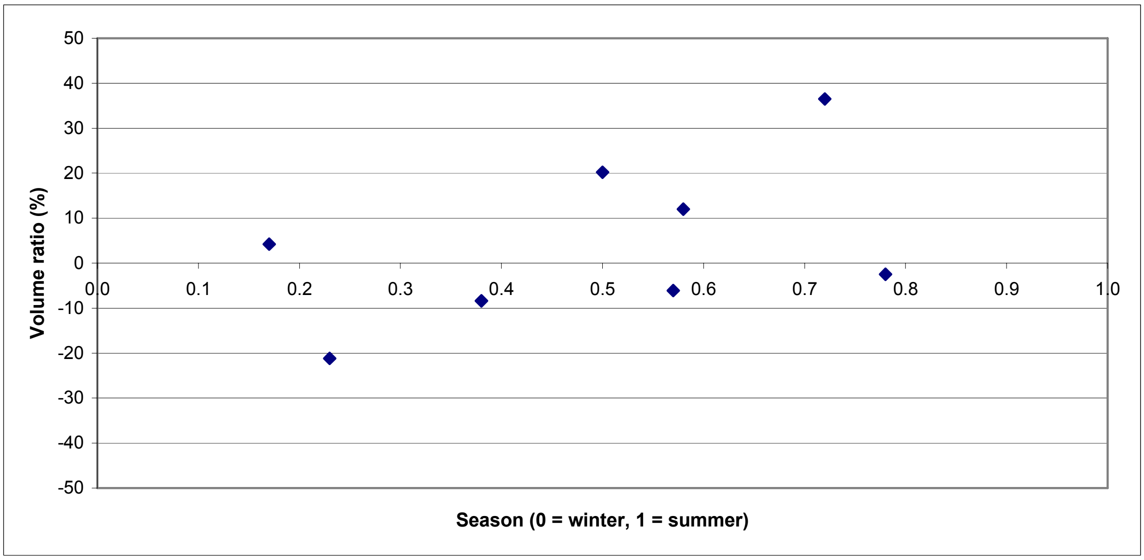


Figure 8.15 Calibration goodness of fit with respect to 'volume ratio'

8.5.4 Model performance: Peak flow

The differences between observed and modelled peak flows are presented in Table 8.4 below:

Table 8.4 Differences between observed and modelled peak flows as a function of flow rate

Peaks (in l/s)	0.5-1	1-1.5	1.5-2	2-2.5	2.5-3	3-3.5	3.5-4	4-4.5	4.5-5	5-5.5	6-6.5	7.5-8
Average error (l/s)	0.02	-0.11	-0.11	-0.45	-0.46	-0.47	-1.55	-0.62	-0.30	-0.73	0.81	-0.47
Absolute error (l/s)	0.26	0.46	0.12	0.50	0.46	0.60	1.55	0.62	0.66	0.73	0.81	0.47
Absolute error in %	34%	37%	7%	22%	17%	19%	41%	15%	14%	14%	13%	6%

Figure 8.16 shows the average difference for each interval of peak flow rate. It is likely that the calibration could be improved through the use of additional calibration events.

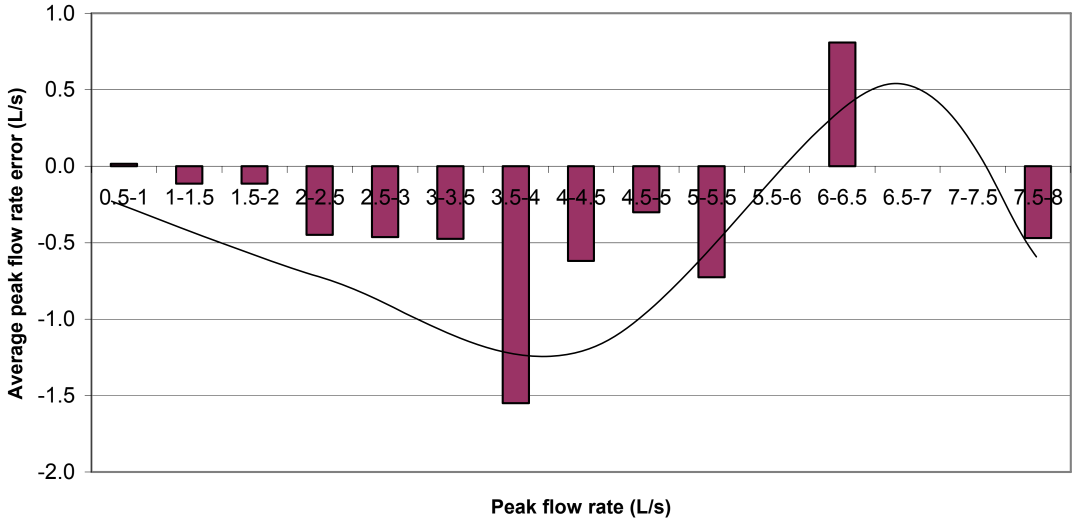


Figure 8.16 Average difference in peak flow as a function of the peak flow rate (l/s)

Figure 8.17 represents this difference as a percentage of the peak flow rate.

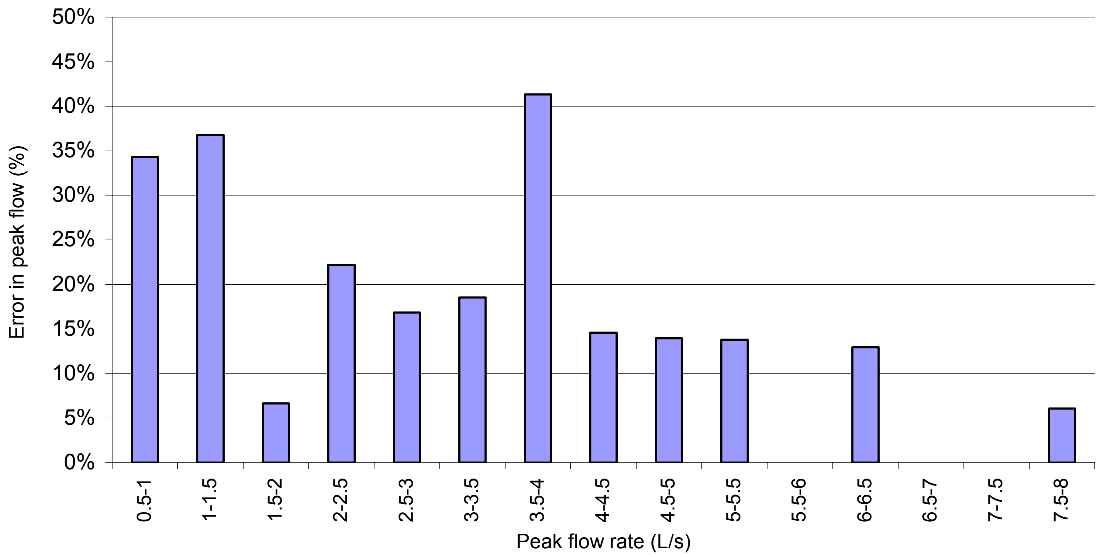


Figure 8.17 Peak flow difference as a function of the peak flow rate

It can be seen that the model performance is relatively poor at low flows. But above 4l/s, the difference is below 15%.

8.5.5 Extreme event performance

The performance of the modelled permeable pavement was tested for potential extreme event performance by assessing the system under two scenarios:

a. FEH design storms (including 30, 60 and 120 minute duration, 5, 10, 25, 50 and 100 year events)
b. A 100 year stochastically generated rainfall time series for the south east of England.

8.5.5.1 FEH design events

The response of the model to FEH design storms is given in Table 8.5, below.

Table 8.5 Response of the calibrated model to FEH design events

Return period (years)	5			10			25			50			100		
Storm duration (min)	30	60	120	30	60	120	30	60	120	30	60	120	30	60	120
Peak flow (l/s/ha)	7.3	8.1	8.2	8.4	8.5	8.6	8.7	8.8	8.8	8.9	9.0	9.0	9.0	9.1	9.2
Outflow percentage (%)	65.5	69.0	71.4	65.8	69.2	71.9	65.7	69.3	71.8	65.8	69.4	71.8	66.2	69.4	71.8

The following graph (Figure 8.18) shows the peaks flow as a function of return period for each of the different durations:

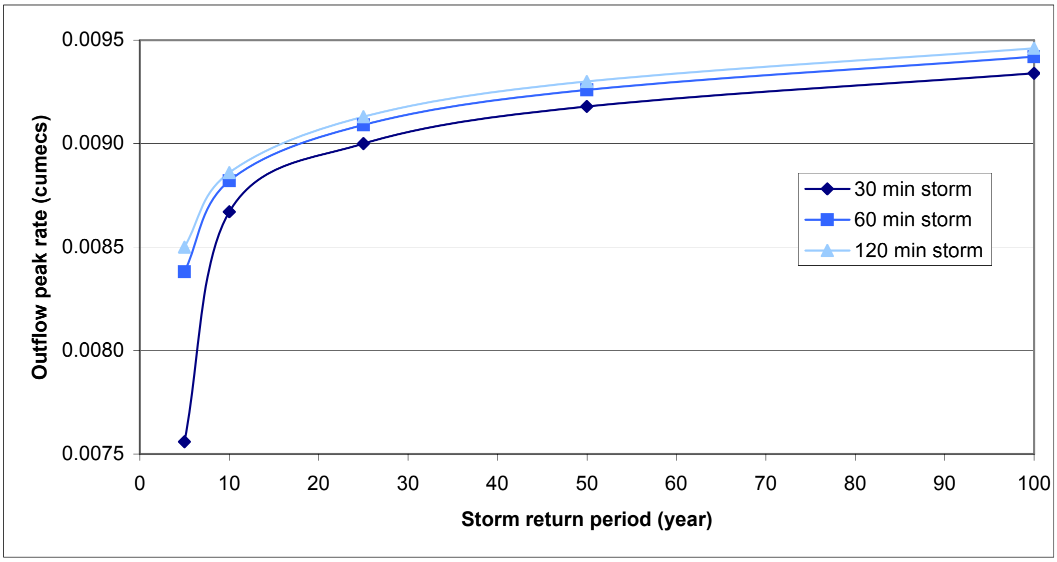


Figure 8.18 Outflow peak rate as a function of the storm return period

It is observed that the rate of increase of peak flow decreases with return period. This is due to the small outlet pipe (150 mm) that constrains the discharge. Thus, the increase in outflow is only due to a bigger head of water within the model nodes. To remedy this, a new model was constructed with a 300 mm outlet pipe. It was then possible to observe the unconstrained outflows from the car park. Figure 8.19 shows the difference between the discharges from both models, for a 60 minute storm. This assumes that the model is applicable for these much greater events.

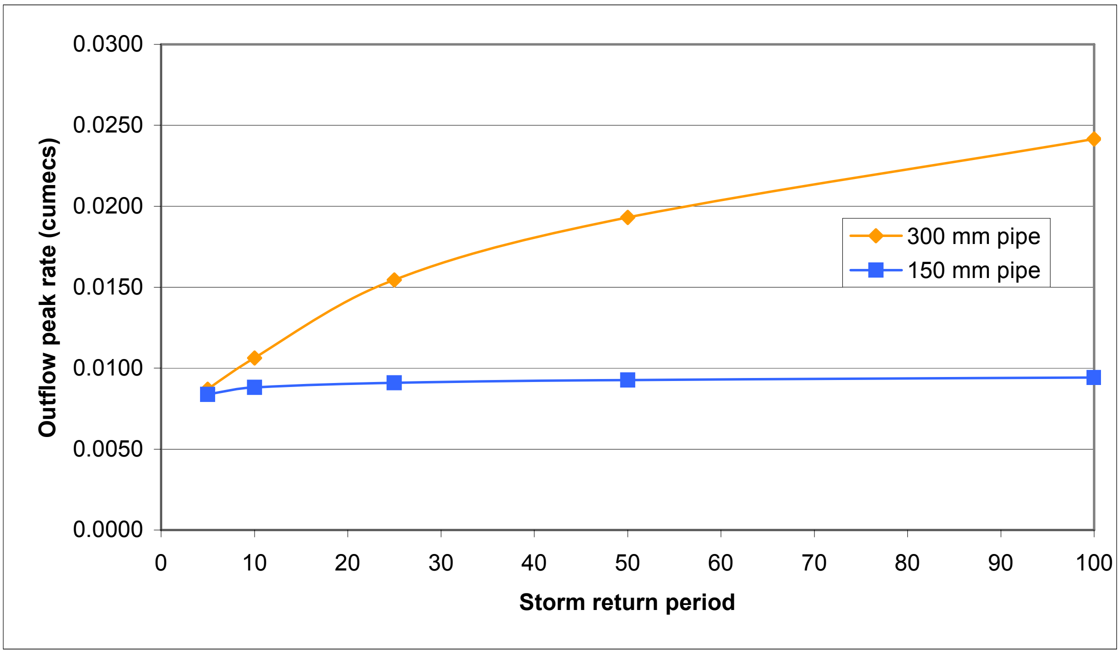


Figure 8.19 Outflow peak rate as a function of the storm return period for 300 and 150 mm outlet pipe model (60 minute storm)

8.5.5.2 100 year rainfall time series

The use of time series rainfall data is an alternative to the design event approach. It inputs a large number of rainfall events into the model, most of which will not be particularly relevant in terms of extreme performance, however some of which may represent worst case conditions with respect to system performance. It is clearly computationally quite inefficient, but it does represent realistic rainfall conditions over 100 years and includes events that occur after long periods of wet weather. Figure 8.20 shows the performance of the system for two rainfall time series – one for the south east of England (Greenwich), and one for the north west of England (Princetown). It can be seen that the performance of the model results in a significant difference to the predictions produced by the design events.

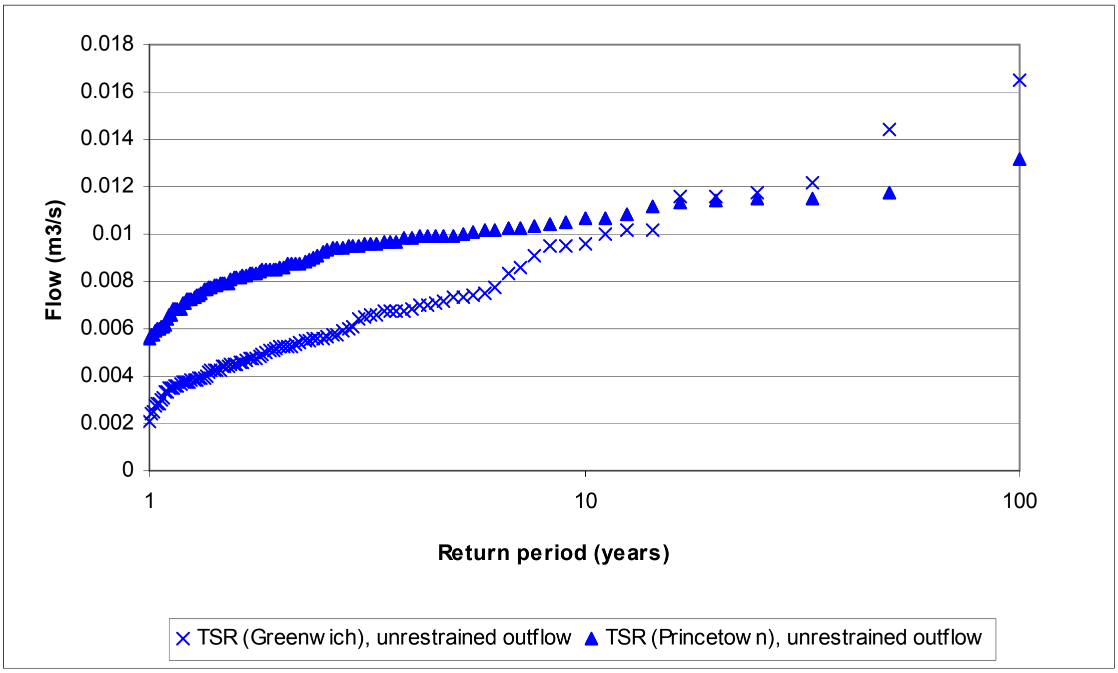


Figure 8.20 System performance under 100 year time series rainfall series

8.5.6 Comparison of performance with a conventional drainage system

Another model of the site was constructed using an appropriate equivalent conventional drainage system layout, comprising gullies linked by 300 mm diameter pipes.

The table below shows the results of the modelling for the same events as used for the permeable pavement model.

Table 8.6 Response of the conventional drainage model for design events

Return period (years)	5			10			25			50			100		
Storm duration (min)	30	60	120	30	60	120	30	60	120	30	60	120	30	60	120
Peak flow (l/s/ha)	90	96	88	111	110	101	134	136	121	152	150	139	169	163	151
Outflow percentage (%)	83.3	86.9	83.9	79.4	85.6	87.3	75.0	80.8	83.4	84.8	86.0	86.6	76.6	81.3	84.7

The following graph (Figure 8.21) shows the peak flows as function of the return period for different storms and compares the results with the permeable surface system.

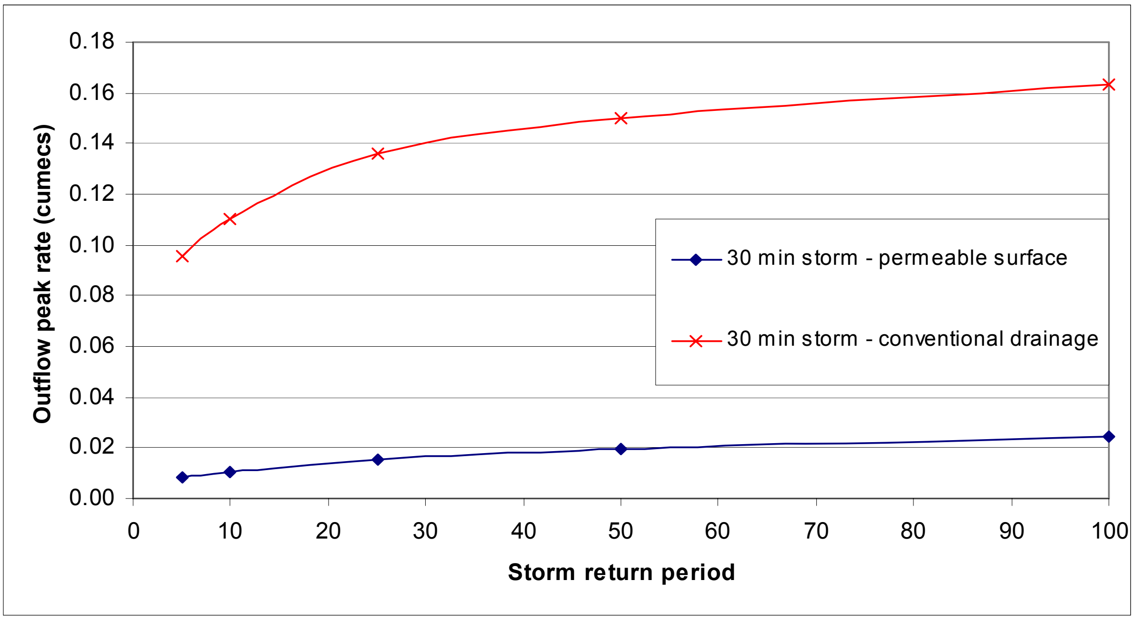


Figure 8.21 Outflow peak rate as a function of the storm return period

It is observed that the peak flow rates predicted for an equivalent conventional system are far higher than for the permeable surface. This is to be expected as the conventional drainage does not include any storage elements to attenuate the flow, and outflow rates are not constrained in any way. If outflow rates are constrained to 9 l/s/ha, it is calculated that the required sub-surface storage would be of the order of 450 m^3, however this would not provide the same delay, volume reduction or water quality performance characteristics if it was provided as a storage chamber.

The runoff surfaces are equivalent for both models and it is observed that the proportion of runoff (i.e.volumetric performance) is considerably less for the permeable pavement scenario.

8.5.7 Comparison of performance with Greenfield response

The "regional growth curves" of the Flood Studies Report provides a recommended procedure for estimating the T-year return period flood Q(T) from the mean annual greenfield flood, Q_{BAR}. The mean annual flood is calculated using the IH 124 equation. We can then estimate design greenfield peak flow rates as function of the rainfall return period.

The graph below (Figure 8.22) shows the frequency curve obtained using this procedure, together with the previous scenarios for comparative purposes.

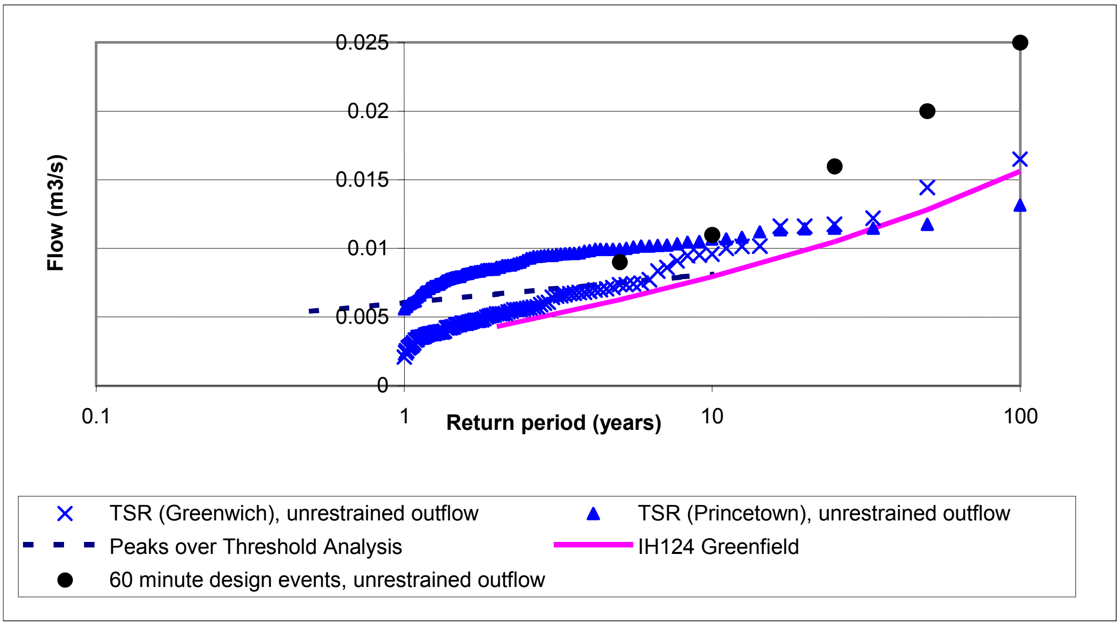


Figure 8.22 System performance for complete range of scenarios

This shows that, using a 100 year rainfall time series the system response appears to match estimated greenfield runoff flows well. Using 60 minute FEH design events, the system response is approximately 60 % greater than the estimated greenfield response curve. It therefore appears that for these conditions, FEH design events still present the most conservative condition for system design and that systems are unlikely to match greenfield flows unless an outlet constraint is imposed.

8.6 SUB-BASE SATURATION

Since the outflow is limited by the pipe size, the likely failure mode for the system is surface flooding. For an average rainfall intensity, calculations were carried out to determine what storm duration would be necessary to fill the sub-base storage, thus causing flooding (Figure 8.23).

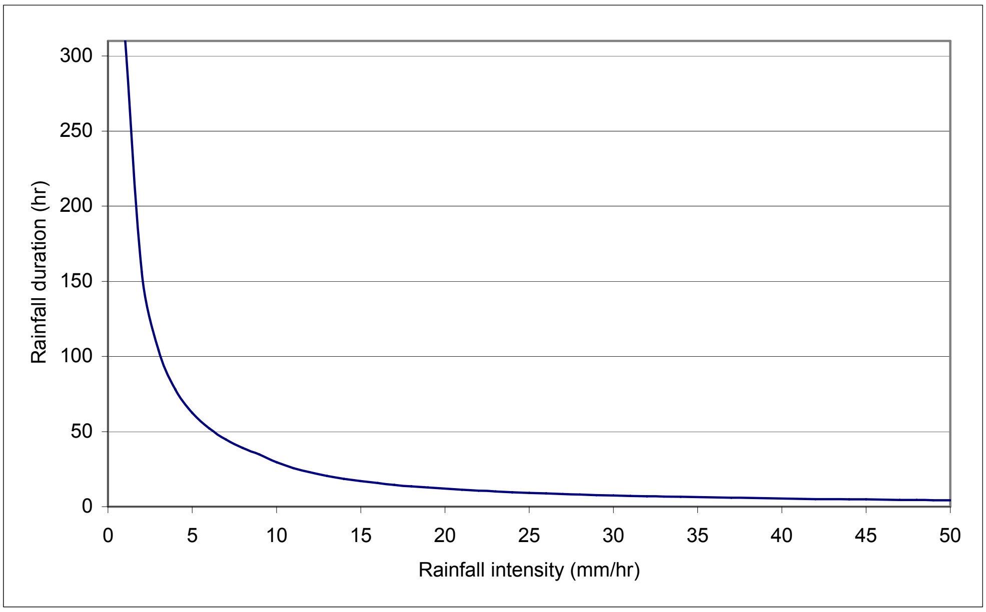


Figure 8.23 Storm duration necessary to fill the sub base storage as a function of the rainfall

This graph shows that the rainfall events that would be required would be extremely unlikely to occur.

8.7 CONCLUSIONS

- One of the key objectives for SUDS such as permeable pavement systems is to reduce the risk of flooding resulting from high runoff rates and volumes from development sites. The degree of hydraulic impact of the system was evaluated by assessing a number of key parameters. The main benefits were seen as a substantial reduction in storm peaks, attenuation of the storm duration and significant reduction in outflow volumes compared to rainfall volumes. Seasonal influences were also shown to be important with wet antecedent conditions contributing to less attenuation and percentage runoff reduction.

- The hydraulic response of the pavement system appears to compare favourably with greenfield runoff response, in that a large proportion of the runoff takes place at very low rates of discharge (below 2 l/s/ha). Recent research at HR Wallingford (Kellagher, 2002) has shown that outfall rates needed to be as low as 2 l/s/ha to effectively protect the river during periods of increased flood risk. A simple approach of tightening throttles to this level is not likely to be tenable in terms of the huge resulting attenuation volumes, so there are moves towards a new twin track approach whereby part of the runoff is constrained to very low rates of runoff, with the remainder being discharged at more relaxed flow limits. However, permeable pavements appear to achieve low discharge rates without the use of small orifice throttles.

- Pervious pavements exhibit a complex hydraulic behaviour. This reflects the mechanisms associated with the rainwater passing through the blockwork or porous asphalt, the passage of water through the goetextile and underlying stone media, and the collection system being used (e.g. a grid of perforated pipes or a single collector pipe along one edge). In addition, there are lateral hydraulic gradients that are created within the subsurface reservoir, which are influenced by the pavement's geometrical characteristics, the pavement depth and the storage media. It was possible to use the infiltration runoff module of a proprietary urban drainage modelling system (Infoworks CS) to provide reasonable representation of the performance of a pervious pavement and associated sub-surface storage structure, and to use the calibrated model to predict likely system performance for a range of extreme events. However, the calibration parameters are likely to be highly site specific and not suitable for generic application.

- The performance was tested using both FEH design events for return periods up to 100 years, and a 100 year rainfall time series. The modelled performance to time series rainfall compared well with estimated greenfield frequency curves for the site, however the system response to design events gave flows of up to 60 % greater. This shows that the use of FEH events for design is likely to be a precautionary and conservative approach, and that the use of time series rainfall should be explored further.

- There is a need for general drainage software to be able to represent pervious pavements more accurately than they do at present.

9. Conclusions

Site-specific conclusions are given in the individual report chapters, however generic conclusions are presented in the following sections.

9.1 HYDRAULIC PERFORMANCE OF SUDS

9.1.1 Treatment Trains

- The monitoring at Hopwood Park Services confirms that for design events and wet pre-event conditions, the percentage of the rainfall event retained by the SUDS system is low. It is therefore appropriate that, for design events, no account be taken of the system losses that are observed for small events and following dry periods.

- The reduction in peak flow rates and volumes observed through the treatment train at Hopwood Park Services demonstrates that:
  (1) SUDS systems are potentially useful tools for meeting objectives of achieving greenfield site runoff conditions;
  (2) A treatment train can promote good reduction in peak flow rates via incremental improvements across train components.

- Systems are likely to perform better (in terms of attenuation) during the summer, when water levels drop and vegetation levels increase.

9.1.2 Permeable Pavements

- Key benefits of permeable pavement systems were observed as substantial reduction in storm peaks, attenuation of the storm duration and significant reduction in outflow volumes compared to rainfall volumes. Reduced attenuation and percentage runoff reduction were observed following wet antecedent conditions.

- The observed hydraulic response of the permeable pavement system at Wokingham appears to compare favourably with greenfield runoff response, in that a large proportion of the runoff takes place at very low rates of discharge (below 2 l/s/ha). Recent research at HR Wallingford (Kellagher, 2002) has shown that outfall rates needed to be as low as 2 l/s/ha to effectively protect the river during periods of increased flood risk. A simple approach of tightening throttles to this level is not likely to be tenable in terms of the huge resulting attenuation volumes, so there are moves towards a new twin track approach whereby part of the runoff is constrained to very low rates of runoff, with the remainder being discharged at more relaxed flow limits. This study shows that permeable pavements appear to achieve low discharge rates without the use of small orifice throttles which is a significant bonus in terms of reducing the risk associated with future blockages.

9.2 WATER QUALITY PERFORMANCE OF SUDS

- The analysis of performance data undertaken as part of the literature review indicates that out of six SUDS components (permeable pavement, wetland, pond, swale, detention basin and wetland channel), the pavement and wetland appear to provide the most consistent water quality performance. The detention basin shows the poorest performance levels.

- Removal percentages are unlikely to be useful parameters for characterising performance, unless evaluated using a high resolution time-step to enable an accurate assessment of flow and pollutant loads at both the inlet and outlet of the system. Using event-based removal percentages, there is a risk that some systems may be mis-characterised as less effective because of cleaner influents. The recommended approach involves a comparison of effluent quality with appropriate environmental standards to evaluate the residual environmental risk of system discharges.

- During large storm events, water quality at the outlet of a pond is likely to be influenced by water quality of the inflow; however for the majority of the time, effluent quality will be pre-determined by the quality of the permanent pond volume.

- Residence times of inflow water within a permanent pond are dependent on the characteristics of the rainfall time series (which will vary with geographical location), and on the size of the permanent volume. Due to the seasonal characteristics of the precipitation, residence times will show significant seasonal variability.

- Data analysis presented as part of the literature review indicates that:
  - where storms are smaller than the permanent pond volume, the effluent concentrations tend to lower values (this trend being most obvious for TSS);
  - above a ratio of between 1 and 2 (i.e. where the permanent pond volume is between 1 and 2 times the size of the storm), there appears to be little significant influence;
  - for large ponds, there are always a few data points with high pollutant concentrations. These may indicate internal pond processes causing poor water quality after long retention periods (possibly during dry weather). As these are all results from smaller events, short-circuiting is not seen as a cause and it is considered more probable that this may be due to internal (anerobic) pond processes, releasing organic pollutants and causing poor water quality after long retention periods.

- For large rainfall events, the influence of the bulk volume on water quality is less significant and the main process that determines the effluent water quality during the storm event is whether effective sedimentation is allowed to take place. This emphasises the need for good pond layout and appropriate treatment train design.

- The residence time in the ponds at Hopwood was shown to influence the concentrations of some nutrients and TSS loads. Residence times of greater than 3-5 days were seen to increase the concentration of NH_4-N in the SUDS units in the upper parts of the treatment train.

- It is likely that loads of system heavy metals could, in many cases, be predicted knowing the concentrations of the sediments in the flows, as the metal concentrations measured at Hopwood Services showed a direct relationship with suspended solids.

- The studies at Hopwood confirmed that the predominant form of the heavy metals in surface water runoff is likely to be the particulate one. Therefore the main process of removal of heavy metals is likely to be via sedimentation rather than by up-take of dissolved forms from living microorganisms.

- The studies at Hopwood show that, in general, the water quality improves significantly along the treatment train and that the last component shows good compliance with environmental quality standards. This emphasises the advantage of using SUDS units in series.

9.3 MONITORING OF SUDS SYSTEMS

- Careful selection is needed when securing an appropriate SUDS site for monitoring purposes, particularly if sites are not owned and/or operated by the funding body for the monitoring campaign. Particular difficulties that may need to be overcome include:
  - f) reluctance of site owners / operators to sanction monitoring due to concerns over potential failures that may be observed with respect to system performance. SUDS are generally perceived as new and uncertain technology and owners may be concerned over potential future problems should the performance be found not to match design expectations;
  - g) poor designs and/or construction of the SUDS system;
  - h) complexities of collecting data from systems not designed with data collection in mind;
  - i) complexities, interference and nuisance associated with installing monitoring equipment post construction; and
  - j) risks from vandalism.

- Securing equipment against vandalism is often a major challenge for open sites such as ponds. Detailed pond monitoring may require prolonged measurements of in and outflows, and whether in a residential or business area, any equipment is potentially a target. Ideally, monitoring chambers should be constructed specifically to secure equipment. Sites remote from housing areas may offer lower risk solutions, although dependence on battery power is an obvious disadvantage.

- Runoff from SUDS systems (especially permeable pavements where filtration/storage volumes are high) can be zero or very low for long periods and appropriate flow measurement devices should be selected with this in mind. Regular equipment maintenance is also required for these conditions.

- Event-based auto-sampling requires regular staff attendance on site and accurate knowledge of likely site weather conditions so that samples can be collected for analysis at short notice. Auto-sampling requires an appropriate trigger (either flow depth or rainfall) and becomes increasingly difficult where monitored flows profiles are both variable and unpredictable. Management costs of such an approach are high.

9.4 MODELLING OF SUDS SYSTEMS

- Flow circulation patterns in ponds are amenable to computational fluid dynamic (CFD) modelling and this study has demonstrated the valuable role that CFD may have in understanding the behaviour of SUDS retention ponds. Pond hydraulic performance is a function of a complex 3D internal flow field. CFD may be used to optimise the geometric characteristics of the pond using particle characteristics corresponding to the full range of anticipated suspended sediments, as well as dissolved pollutants. Furthermore, time-dependent simulation makes it feasible to consider both the whole-storm assessment of performance and long-term performance giving likely annual outfall loadings and deposition rates, rather than restricting the assessment to steady flow scenarios. Ideally such studies would be complemented by field measurements of hydraulic and pollutant retention performance.

- Pervious pavements exhibit a complex hydraulic behaviour. This reflects the mechanisms associated with the rainwater passing through the blockwork or porous asphalt, the passage of water through the goetextile and underlying stone media, and the collection system being used (e.g. a grid of perforated pipes or a single collector pipe along one edge). In addition, there are lateral hydraulic gradients that are created within the subsurface reservoir, which are influenced by the pavement's geometrical characteristics, the pavement depth and the storage media. In this study, it was found to be possible to use the infiltration runoff module of a proprietary urban drainage modelling system (Infoworks CS) to provide reasonable representation of the performance of a pervious pavement and associated sub-surface storage structure, and to use the calibrated model to predict likely system performance for range of extreme events. Frequency curves of peak modelled flows using a 100 year rainfall time series compared well with estimated greenfield frequency curves for the site. Frequency curves of peak modelled flows using FEH design events gave a frequency curve up to 60 % greater than the estimated Greenfield flows. It could therefore be concluded that the use of FEH events for design is likely to be a precautionary and conservative approach, and that the use of time series rainfall should be explored further.

10. Recommendations

10.1 SUDS DESIGN AND MANAGEMENT

It is recommended that future design processes should include:

a) Consideration of site-specific climate conditions and their influence on likely residence times and pond depths. It is suggested that time series rainfall be used to ensure that the 80%-ile retention time for either annual or summer events is less than 7 days. This will be most important for sites without regular baseflow and where low water depths and high levels of vegetation increase the risk of high nutrient concentrations;

b) Consideration of the vulnerability of receiving waters to high nutrient loadings during dry summer periods when residence times within permanent water bodies are extended. The third quartile (July to September) is likely to represent the most critical period for most sites;

c) Consideration of appropriate levels of aquatic vegetation and pond depths: in some situations shallow waters and dense vegetation may increase the risk of high nutrient levels;

d) Consideration of amenity and insect control requirements i.e. community acceptability of algal blooms and insect breeding sites that may be more likely to occur within stagnant water bodies;

e) Use of SUDS in series to maximise treatment opportunities and minimise the risks of high nutrient concentrations in SUDS discharges.

f) Effective sediment removal upstream of pond and other open water systems.

Figure 10.1 summarises the likely processes / components that may potentially lead to good or poor water quality (with respect to nutrients), and indicates the design and management approaches that should be considered.

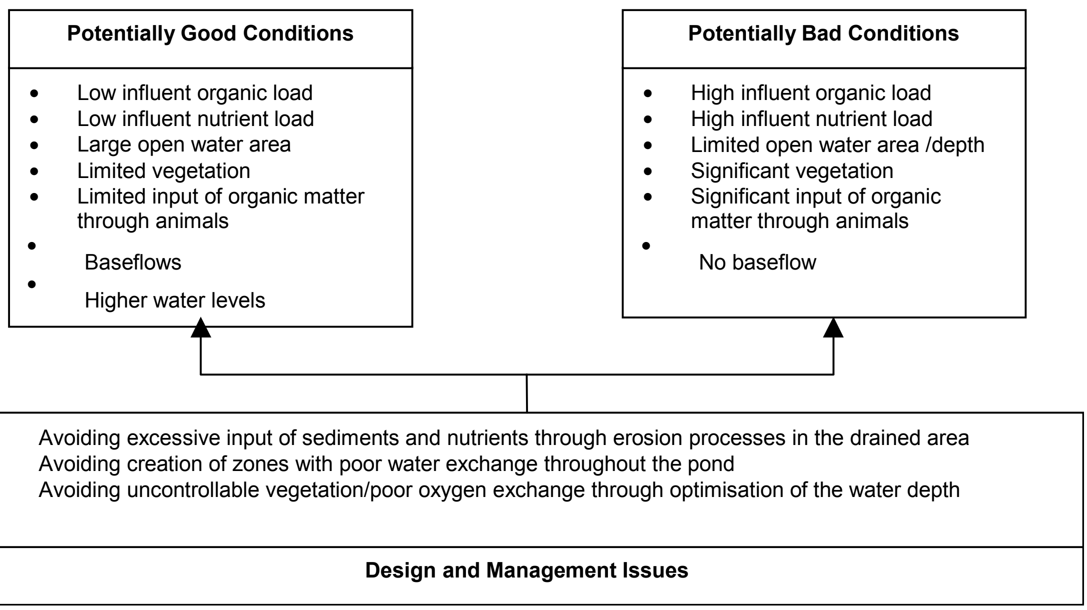


Figure 10.2 Potential management methods to reduce risks of poor water quality

- From the results of the literature review, the following recommendations can be made:
  - Where length:width ratios of pond/wetland systems are less than 4:1, a submerged berm (or other flow distribution device) should be incorporated close to the inlet to ensure effective distribution of the incoming water;
  - The ratio of retention pond area to impervious watershed area should be at least 2.5%, as below this level, there is a tendency for higher outlet pollutant concentrations to be observed.
  - To optimise the pollutant removal efficiencies for swales, they should be designed to promote retention times of at least 9 minutes. Where nutrients are of concern, long length of swales are not recommended, as there is evidence that concentrations of soluble pollutants such as nutrients tend to rise, with increasing swale length.

- An aspect of modelling which needs to be addressed is the rainfall input needed to evaluate the ‘design’ performance of a pervious pavement or, indeed, any SUDS network. The concept of using a critical duration storm for the design of pipe systems or for determining attenuation storage volumes is very useful, as it is computationally efficient. However, when a new development has a multiplicity of different SUDS components, each of which has very different critical duration characteristics, the composite response from the site or from parts of the site will vary for different types of storms. It is also questionable as to whether long duration, low intensity events might constitute more critical conditions than the relatively short duration, high intensity design events currently being tested. Allied to this is the water quality issue, which is only going to become more important in the future, where small events and dry periods will assume greater importance than they do now. It is likely that for the foreseeable future, design storm events still provide an adequate approach in demonstrating the hydraulic performance of runoff from the site and compliance to discharge requirements. However, the use of time series rainfall data is likely to become important in evaluating system sustainability.

10.2 SUDS MONITORING AND PERFORMANCE REPORTING

- The hydraulic and water quality performance monitoring of SUDS systems requires careful planning. Ideally, appropriate monitoring points that facilitate effective and robust flow measurement should be incorporated into the final designs and equipment installed during the construction period. V-notched weirs within manholes, or within open channels where risks of vandalism are very low, can allow accurate measurement over a wide range of flows. The performance of electromagnetic ring devices installed within pipes is often poor at very low flows, particularly where flows have a high silt content.

- In order that all published data on performance can be compared in the future, it is recommended that a protocol be developed for essential data quality attributes required within the published references. These should include:
  - contributing impermeable catchment area;
  - contributing catchment area description;
  - antecedent conditions for all analysed events;
  - flow hydrographs and rainfall hyetographs for all analysed events;
  - complete description of water quality sampling and analysis methods;

  - complete geometric characteristics of all SUDS components;
  - permanent water levels and hydraulic control characteristics;
  - qualitative description of all SUDS components, including photographs;
  - design exceedance processes and design flows;
  - maintenance programme ongoing during the monitoring.

- Although at an individual site it is important to test whether the SUDS system has a statistically significant effect on water quality, it is recommended that effluent quality, rather than 'removal efficiency' be used to characterise water quality treatment performance. Effluent quality for a range of events and seasons can then be compared to environmental standards appropriate for managing the risks to the receiving water body.

- More attention should be paid to the measurement of biological pollutants. They are particular importance as, in addition to being present in the stormwater effluent, they may also be generated within the SUDS themselves. Reductions of nutrient loads are also important for a range of diffuse pollution management initiatives currently being driven by the Water Framework Directive.

10.3 SUDS MODELLING

At this time, it is still uncertain as to whether a 1D modelling tool can provide a generic method which is sufficiently accurate for predicting the hydraulic behaviour of pervious pavement systems, or whether 2D or 3D finite element analyses are required. The majority of proprietary software packages used for general drainage analysis are 1D models and cannot therefore support the application of 2D or 3D finite element analyses to modelling flows through the pervious pavement storage system. However as this form of pavement design becomes commonplace, it will be essential to be able to predict their hydraulic behaviour with a reasonable degree of accuracy using simple tools. There may be a need to have a more advanced tool available to assist in the design of pervious pavements.

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Appendices

Appendix 1 Literature review of the hydraulic and water quality performance of SUDS

1. Hydraulic and water quality performance of wetlands and retention ponds

1.1 Hydraulic performance

The Scottish group (SNIFFER, 2001; SNIFFER, 2003) report peak flow reductions of between 40 and 52% for three different ponds.

An Australian team of scientists has investigated (through mathematical modelling) the *hydrologic effectiveness and hydraulic efficiency* of ponds and constructed wetlands as a function of the storage volume (percentage of mean annual run-off), detention time, pond layout and inlet/outlet structures (Wong and Breen, 2002; Persson J., 2000; Somes and Wong, 1997; Wong and Somes, 1995). Their key findings are presented below:

a) Influence of outlet structure on the hydrological effectiveness

Hydrologic effectivenss is defined as the long-term average percentage of stormwater runoff subjected to treatment in the wetland. Together with treatment efficiency, this gives the overall treatment performance of a wetland.

Table 1.1. Required storage volume as a percentage of the annual run-off volume for achieving hydrologic effectiveness greater than 90% for constructed stormwater wetlands (Wong & Breen, 2002; Somes and Wong, 1997)

Detention time (hrs)	Storage volume as a % of mean annual runoff volume	
	Orifice outlet	Siphon Outlet
24	2.5	3.75
48	3	5
72	3.5	5.5
120	4.5	6.5
240	6.4	7.5

Whilst an orifice outlet can achieve a high degree of hydrological effectiveness, storage volume and detention time, such systems undergo regular changes in the water depth. Similar conclusions were made for different types of riser outlet structures. Thus the wetland is likely to be providing poor treatment of a large proportion of the run-off that is captured. In addition to providing poor treatment, the short detention time of the upper part of the storage has the potential to produce zones of high velocity in the storage as the storm water flows from the inlet to the outlet. Such zones have potential to cause scour and re-suspend deposited particles, further reducing the wetland effectiveness. The siphon outlet was found to offer the best combination of capturing run-off and providing a reduced range of wetland water levels.

b) Influence of pond layout on hydraulic efficiency

Figure 1.1 is taken from Wong and Breen, 2002, and Persson, 1999 and 2000. It shows the influence of the pond layout on the hydraulic efficiency represented by 'λ', a measure of flow hydrodynamic conditions in constructed wetlands and ponds that describes how well the incoming water distributes within the system. 'λ' ranges from 0 to 1, with 1 representing the best hydrodynamic conditions for storm water treatment.

$$\lambda = e\left(1-\frac{1}{N}\right)=\left(\frac{t_{mean}}{t_n}\right)\left(1-\frac{t_{mean}-t_p}{t_{mean}}\right)=\frac{t_p}{t_n}$$

The effective volume ratio, *e*, is defined by (Thanckson, Shields, & Schoroeder, 1987):

$$e=\frac{t_{mean}}{t_n}=\frac{V_{effective}}{V_{total}}$$

where V_{total} is the total volume of the water system and $V_{effective}$ is the total volume minus the dead volume (i.e., volume of water that has no interaction with the water flowing through the system).

N is the number of CSTRs (Continuous Stirred Tank Reactors) in series; t_{mean} is the mean detention time; t_n is the nominal detention time, defined by the ratio between the volume and the flow; t_p is the time of the peak outflow concentration.

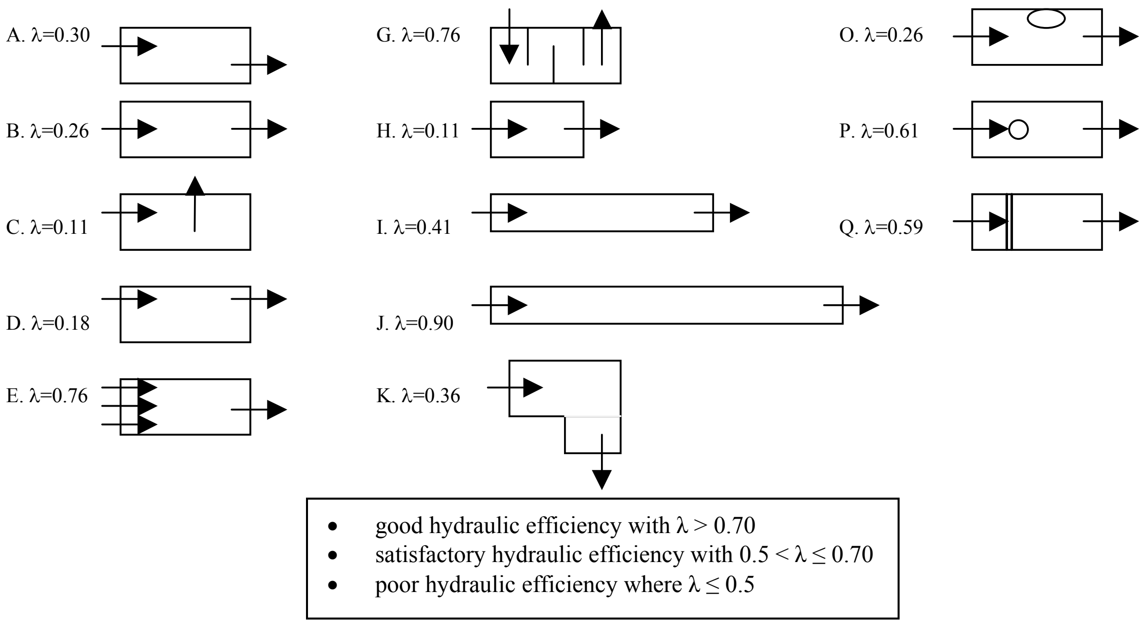

Figure 1.1 Influence of the pond layout on the hydraulic efficiency (Wong and Breen, 2002; Persson, 2000, Persson et al, 1999)

The simulations have shown that designs with a length to width ratio of 4:1 or less, and with point inflow and outflow (cases A, B, D, H and I) will not promote good hydraulic efficiency and high effective volumes unless steps are taken to evenly distribute the inflow across the width of the detention storage. The inclusion of a small island in front of the inlet (case P) was found to more than double the hydraulic efficiency (compared to case B) of the system. The use of a submerged berm at the inlet, or a flow distribution inlet structure was found to have a similar, with the distributed inlet providing a clear advantage. While no simulations were carried out for the case of a distributed outlet, it was suggested that a single outlet point would not affect the hydraulic efficiency significantly.

c) Influence of vegetation on hydraulic efficiency

Table 1.2 shows the influence of the wetland vegetation on the hydraulic efficiency of a wetland.

Table 1.2 Influence of the wetland vegetation on the hydraulic efficiency (Persson et al., 1999)

Case	λ
Base case	0.32
Full vegetation	0.64
Banded Bathymetry and Fringing vegetation	0.64
Labyrinth Bathymetry and Full vegetation	0.52
Banded Bathymetry and Full vegetation	0.76
Uniform Depth and Full vegetation	0.74

This shows that fully vegetated systems contribute positively to the hydraulic efficiency of the systems, however maintaining a sustainable botanical structure is not a simple matter and requires particular attention to be given to the hydrologic regime control of the system and the matching of vegetation type to the wetness gradient.

1.2 Water quality performance

Reports on the water quality performance of 40 retention ponds during storm events have been reviewed. 33 of these were taken from the American BMP database (BMP database, 2003). The others were taken from the following sources: Petterson T., (1998), Petterson T., et al, (1999), Semadeni-Davies A., (2002), Adolf Spitzer, (personal contacts).

The 75th percentile of the average outflow concentrations was calculated and compared with the corresponding values from other systems. The results are summarised in Table 1.3. Figures 1.2 and 1.3 present the effluent water quality levels compared to standard limits.

All the metals except for Chromium (Cr) are in compliance with the standard limits. Some of the biological parameter concentrations are up to twice the adopted standards. The most problematic parameters seem to be COD and PO4-P, followed by TSS. A comparison between the median removal efficiencies (see Figure 1.4) shows that the removal of biological pollutants appears to be less efficient than metal removal. So if the influent has high concentrations of Nitrogen and Phosphate containing compounds, then the effluent water quality might be compromised. For some systems, the removal efficiency for some metals (e.g. Ni, Cr and Cd) are also poor. This probably depends on the form in which the metals are presented in the water. In general the particulate matters have higher removal efficiencies (due to sedimentation) than the soluble forms.

Table 1.3 Summary of the water quality performance for retention ponds (based on 36 case studies)

Outlet concentrations	COD				BOD			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	48	32	143	39	5.64	3.79	14.85	52
2 Quartile/min removal	31	22	40	-19	2.94	2.05	4.18	3
3 Quartile /max removal	54	36	163	82	5.44	3.62	16.45	90
No Samples	12	11	10	11	8	8	8	8
	NH4-N				NOx-N			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.18	0.10	1.77	20	0.44	0.29	22.54	30
2 Quartile/min removal	0.05	0.03	0.17	-125	0.09	0.04	0.26	-153
3 Quartile /max removal	0.29	0.12	0.93	100	0.55	0.39	1.10	76
No Samples	15	15	15	14	22	22	21	21
	TKN				TP			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.96	0.62	1.36	17	0.42	0.29	1.16	48
2 Quartile/min removal	0.48	0.36	0.60	-125	0.09	0.05	0.15	-60
3 Quartile /max removal	1.35	0.81	1.41	51	0.23	0.18	0.55	88
No Samples	19	19	19	15	31	30	30	29
	PO4-P				TSS			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.11	0.07	0.19	25	47	35	130	73
2 Quartile/min removal	0.03	0.01	0.05	-315	12	7	26	-428
3 Quartile /max removal	0.14	0.11	0.19	74	33	21	92	99
No Samples	16	16	15	15	34	34	34	31
	Cd				Pb			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	1.63	1.15	25.42	40	13	12	358	63
2 Quartile/min removal	0.15	0.15	0.55	-131	3.4	2.1	9.0	-21
3 Quartile /max removal	2.05	1.21	27.36	88	17	11	55	98
No Samples	15	12	12	15	27	22	22	27
	Cr				Cu			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	*Removal	Average	min	max	*Removal
Average /median removal	6.70	6.34	74.59	59	11.54	8.88	80.46	39
2 Quartile/min removal	1.53	0.97	7.59	-164	5.01	4.27	7.04	-40
3 Quartile /max removal	5.69	5.87	73.68	92	13.36	7.28	58.15	94
No Samples	10	8	8	10	19	16	16	19
	Ni				Zn			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	*Removal	Average	min	max	*Removal
Average /median removal	7.52	2.49	6.60	21	95.55	31.22	87.42	70
2 Quartile/min removal	1.59	1.18	3.21	-2	21.00	10.00	48.33	-54
3 Quartile /max removal	5.25	3.36	4.74	78	51.60	32.76	92.33	90
No Samples	8	7	5	7	25	24	23	24

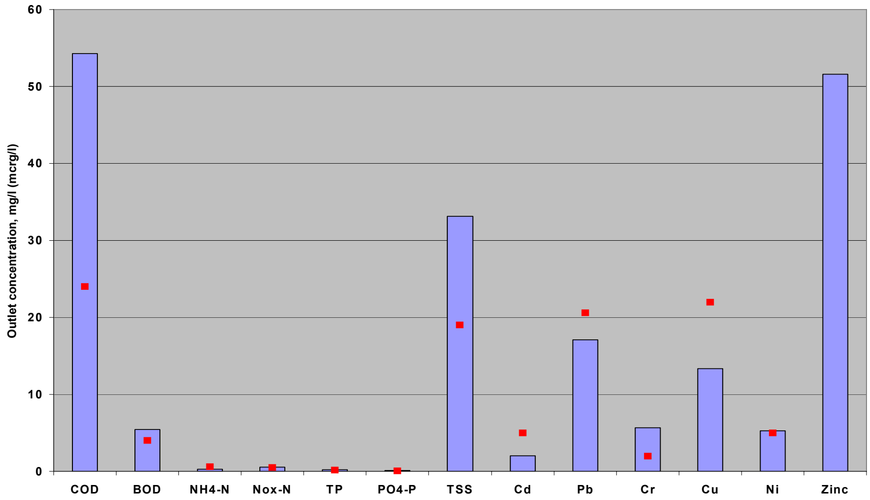

Figure 1.2 Retention pond effluent water quality (75 percentile outlet concentrations), based on the referenced studies (Note: the standard limit for Zn is 200 mcrg/l)

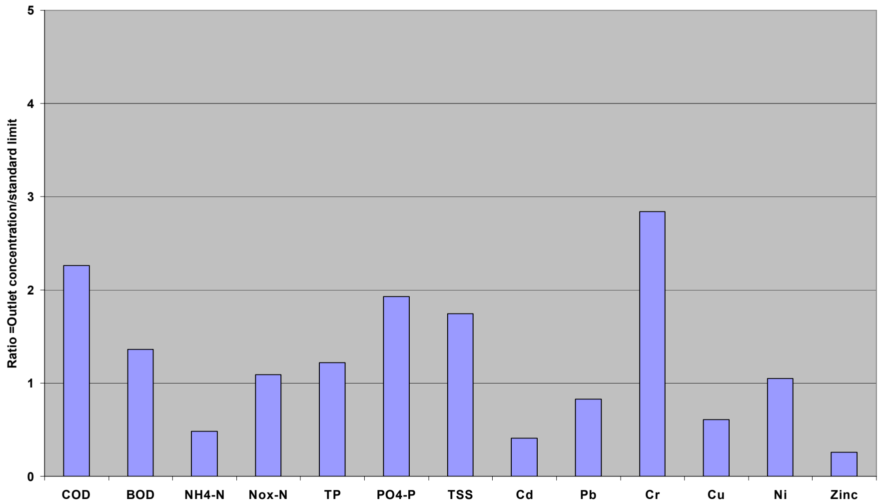

Figure 1.3 Ratio between the outlet concentration and the standard limit value for different parameters

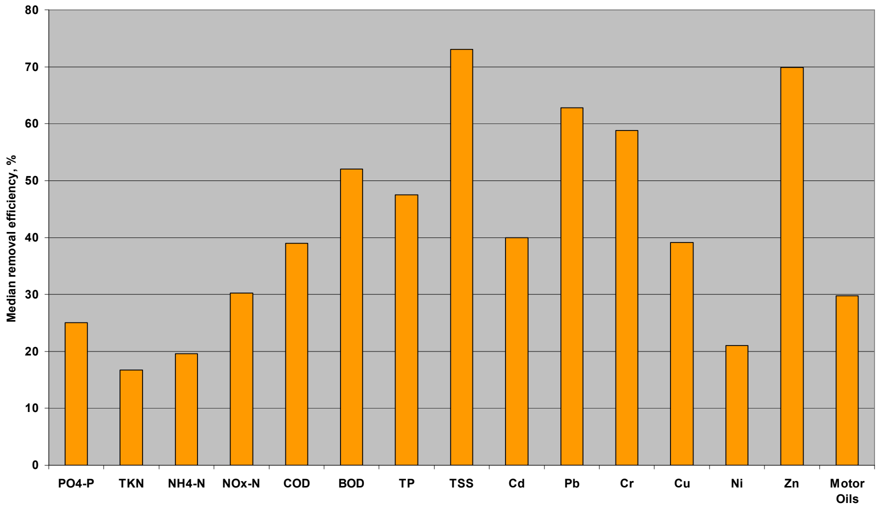

Figure 1.4 Median removal efficiencies of retention ponds

Ponds in general provide longer detention times than wetlands, and are therefore expected to be more effective in promoting sedimentation as the fastest mechanism for water treatment. However systems with a larger permanent pool volume also have a higher propensity for poor flow patterns due to the presence of short-circuiting and zones of stagnation. The lack of emergent vegetation due to higher water depths may also negatively influence the removal of fine colloidal particles and soluble pollutants. Systems with large permanent pools can also lead to poor dissolved oxygen and redox potential conditions that may lead to re-mobilisation of the contaminants in the sediment (Wong and Breen, 2002).

For design purposes, it is essential to be able to link the design characteristics with the required water quality performance. The most common relationships that have been investigated are:

1. *Specific areal ratio* (ratio of the surface of the permanent water volume surface to the contributing catchment area)

A comparison between the removal efficiency of different parameters for five retention ponds (Petterson T., et al., 1999) shows that for specific pond areas up *to 250 m^2/ha (0.025),* the removal efficiency tends to increase proportionally to the increase in the specific area, for most of the parameters. The removal efficiency reaches a plateau at further increases in specific ratio. This assertion is confirmed by a theoretical investigation, where this relation was examined and the results showed the same behaviour (Petterson and Svenson, 1998).

Figures 1.5 and 1.6 present the influence of the specific area ratio on the effluent water quality (not all parameters are included). Figure 1.7 presents a similar relation, but the ratio is calculated with the impervious proportion of the catchment, rather than the total catchment area.

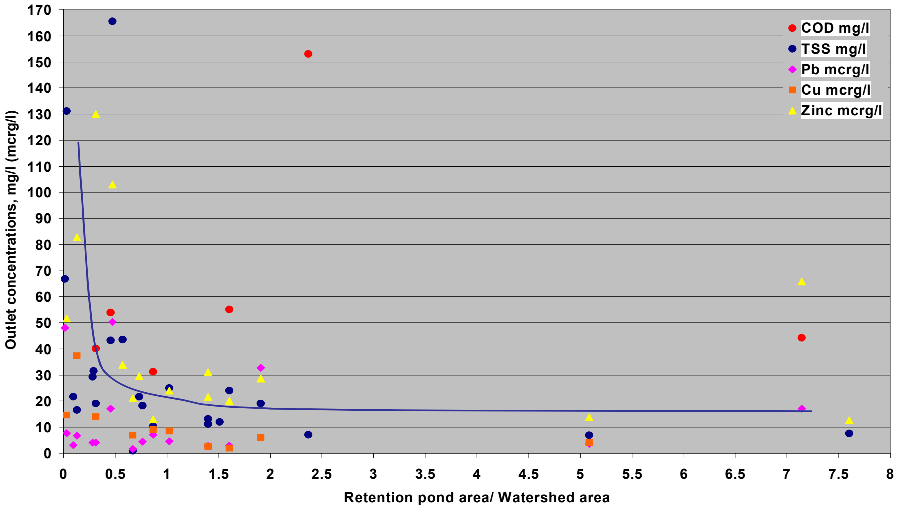

Figure 1.5 Effluent water quality as a function of specific areal ratio

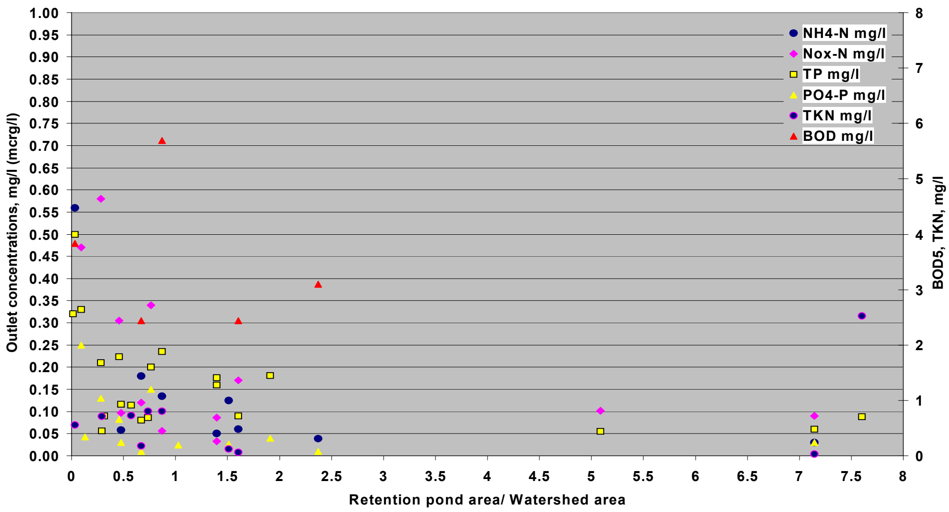

Figure 1.6 Effluent water quality as a function of specific areal ratio

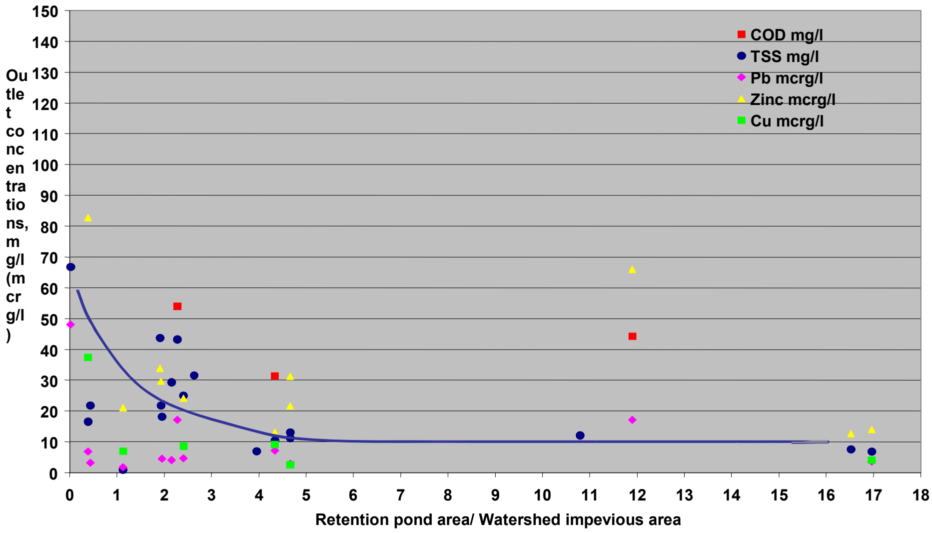

Figure 1.7 Effluent water quality as a function of specific areal ratio, calculated with the impervious watershed area

All the parameters follow a similar tendency, with higher values at lower areal ratios. The effluent tends to reach a stable water quality at an areal ratio of 1.5 % (or 3 % if plotted against impervious area), where most of the outlet concentrations are in compliance with the standard limits. Greater ratios don't tend to contribute to improved water quality. It should be noted that if the TSS are in compliance with the standard limits, then metals concentrations, as well as most of the biological parameters are also within the standard limits. The most problematic parameters seem to be COD and BOD5 which do not show direct relationships with areal ratio.

2. Treatment volume

The majority of the following graphical analyses are based on data from the ASCE BMP database analysed by GeoSyntec Consultants and the Urban Water Resources Research Council of the ASCE in collaboration with the US EPA (ASCE/EPA, 2000) . Results from other studies have been superimposed on these original analyses (larger dots). Due to the paucity of data, it is difficult to make any definitive conclusions.

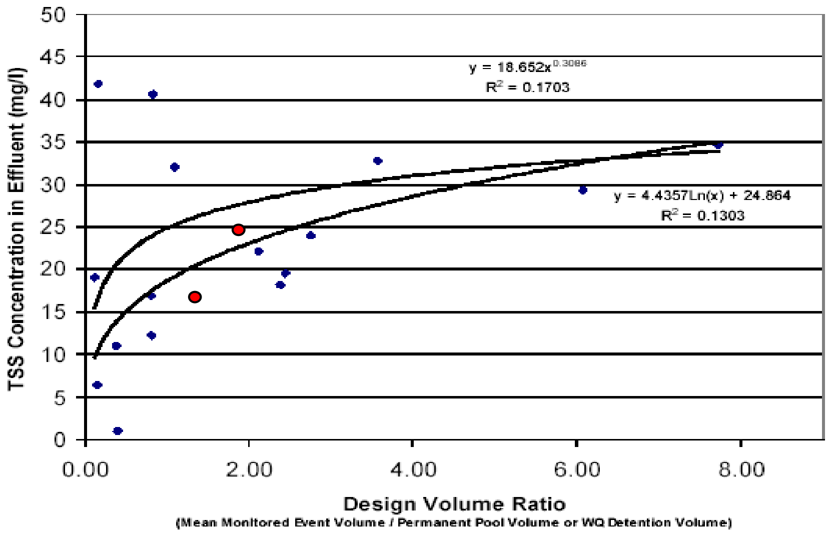

Figure 1.8 Plot of effluent TSS concentration as a function of the design volume ratio for wet ponds

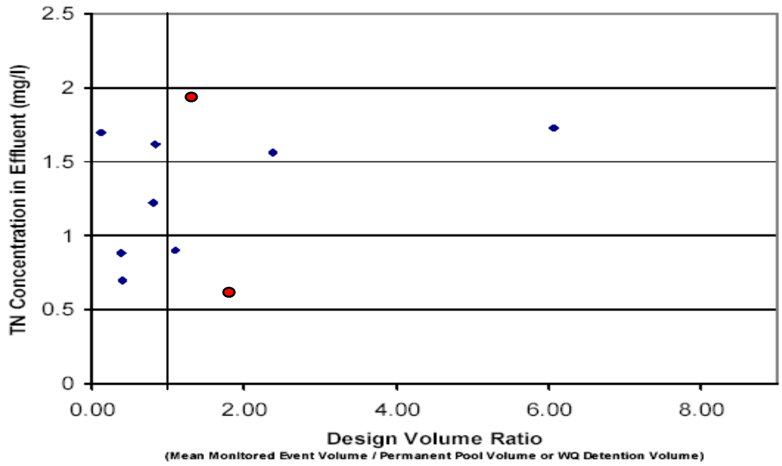

Figure 1.9 Plot of effluent total nitrogen concentration as a function of the design volume ratio for wet ponds

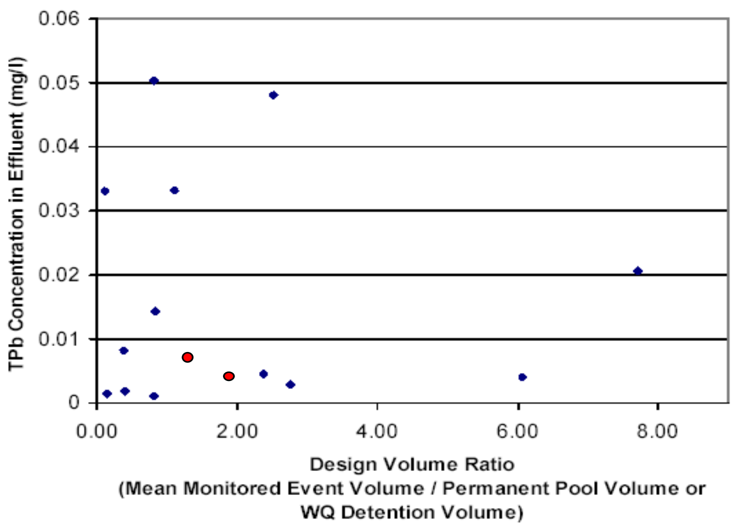

Figure 1.10 Plot of effluent total lead concentration as a function of the design volume ratio for wet ponds

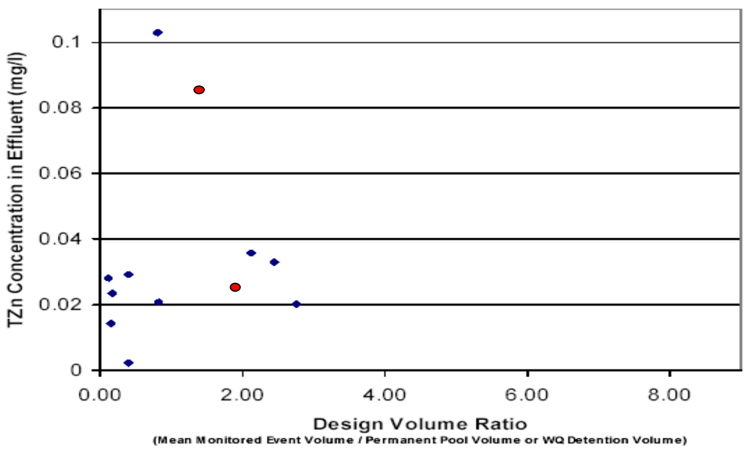

Figure 1.11 Plot of effluent total zinc concentration as a function of the design volume ratio for wet ponds

However, the figures may indicate that:

1. where storms are smaller than the permanent pond volume, the effluent concentrations tend to lower values (this trend being most expressive for TSS);
2. above a ratio of between 1 and 2 (i.e. where the permanent pond volume is between 1 and 2 times the size of the storm), there appears to be little significant influence;
3. for larger ponds, there are always a few data points with high pollutant concentrations. This may indicate internal pond processes causing poor water quality after long retention periods (possibly during dry weather). As these are all smaller events, short-circuiting is not seen as a likely reason and it is considered more probable that this may be due to internal (anerobic) pond processes, releasing organic pollutants and causing poor water quality after long retention periods (possibly during dry weather).

2. Hydraulic and water quality performance of wetlands

2.1 Hydraulic Performance

The hydraulic performance of wetlands is partially described together with the hydraulic performance of retention ponds in Section 1.1. Studies reporting results of flow attenuation or reductions in peak flows specifically for wetlands have not been found. However Wong and Breen, (2002) report that the two-dimensional flow pattern during the filling phase of the wetland helps ensures effective utilisation of the available storage.

2.2 Water Quality Performance

With respect to water quality performance, Wong and Breen, (2002) report the advantages of wetlands over retention ponds as follows:

1. A diversity of aquatic macrophytes within the detention system, which promote uniform flow conditions;
2. The presence of macrophytes facilitate the effective removal of fine particulates and soluble pollutants;
3. More rapid rate of degradation of the deposited organic material.;
4. Progressively less reversible sediment fixation of contaminants in the substratum.

Studies reporting water quality performance of wetlands have been reviewed, and the results are presented in the following table, Table 2.1. All the data for the performance of wetlands have been taken from the BMP database. Nine studies are specifically referenced (BMP database, 2003). Figures 2.1 and 2.2 present graphs of effluent water quality in comparison to standard limits. Wetlands demonstrate excellent performance for most of the parameters. However the dataset is still incomplete, especially with regard to biological parameters. For some parameters, such as BOD, PO4-P, Cr and Ni there was either no data or just one value. The median removal efficiencies for TKN and NH4-N have negative values which implies generation of nutrients within the system.

Table 2.1 Summary of the water quality performance for wetland basin (based on 9 studies)

	COD				BOD			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average	25	17	162	45	5.29	4.24	7.30	48
2 Quartile/min removal	17	10	47	27	5.29	4.24	7.30	48
3 Quartile /max removal	28	21	172	66	5.29	4.24	7.30	48
No Samples	4	4	4	4	1	1	1	1
	NH4-N				**NOx-N**			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average	0.16	0.10	0.60	-4	0.36	0.26	1.46	34
2 Quartile/min removal	0.12	0.07	0.20	-54	0.12	0.09	0.18	-72
3 Quartile /max removal	0.15	0.12	1.24	48	0.43	0.33	1.64	73
No Samples	5	5	5	5	4	4	4	4
	TKN				**TP**			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average	1.45	0.99	4.50	-25	0.30	0.22	0.80	37
2 Quartile/min removal	1.11	0.96	1.41	-91	0.09	0.06	0.20	-50
3 Quartile /max removal	1.61	1.14	4.95	28	0.37	0.30	1.17	66
No Samples	4	4	4	4	8	8	8	8
	PO4-P				**TSS**			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average	0.15	0.11	0.33	78	29	18	76	70
2 Quartile/min removal	0.15	0.11	0.33	78	6.7	4.3	10.0	31
3 Quartile /max removal	0.15	0.11	0.33	78	22	16	98	75
No Samples	1	1	1	1	6	6	6	6
	Cd				**Pb**			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average	1.26	0.64	3.30	26	2.81	1.74	4.18	77
2 Quartile/min removal	0.83	0.43	2.59	22	2.07	0.97	2.89	27
3 Quartile /max removal	1.68	0.84	4.01	29	3.58	2.19	5.47	82
No Samples	2	2	2	2	3	3	2	3
	Cr				**Cu**			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average					7.16	6.20	8.81	44
2 Quartile/min removal					5.20	4.57	6.44	30
3 Quartile /max removal					9.11	7.83	11.18	58
No Samples	0	0	0	0	2	2	2	2
	Ni				**Zn**			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average					121	62	686	12
2 Quartile/min removal					26	22	33	-48
3 Quartile /max removal					146	77	766	76
No Samples	0	0	0	0	4	4	4	4

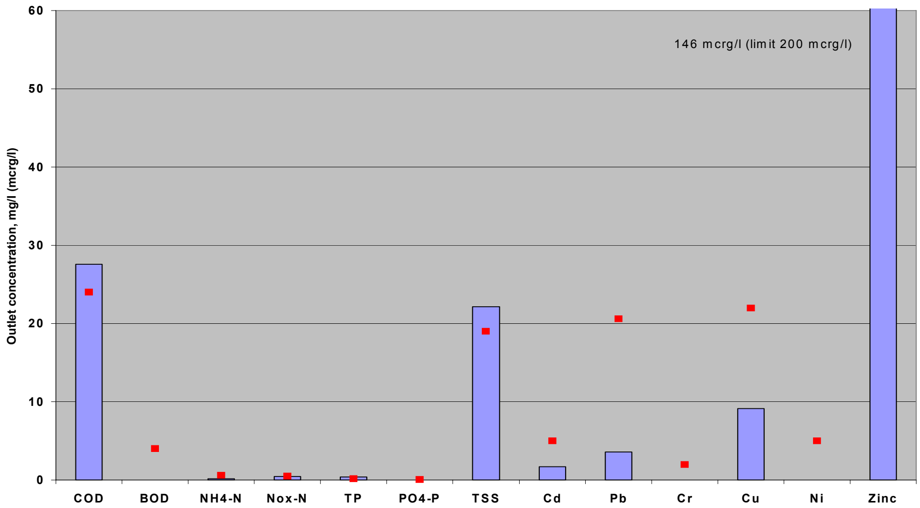

Figure 2.1 Wetland basin effluent water quality (75th percentile of the outlet concentrations), based on the referenced studies (Note: the standard limit for Zn is 200 mcrg/l)

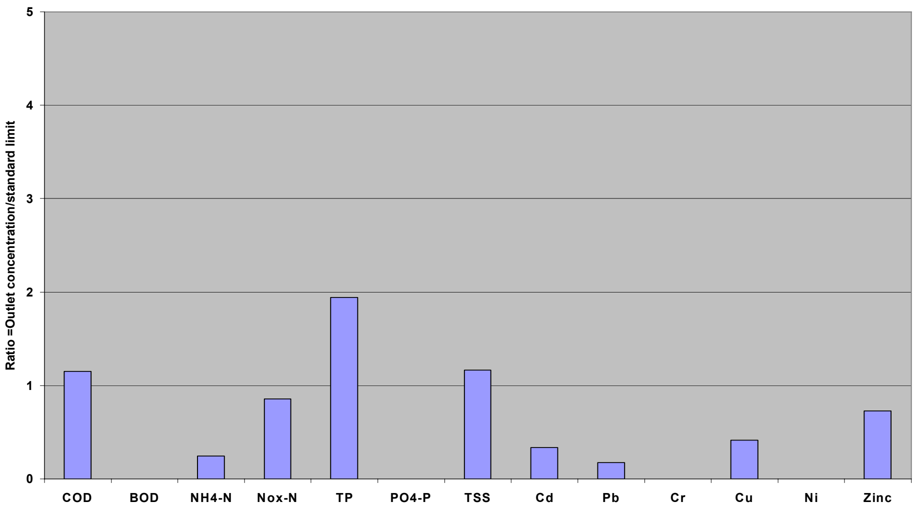

Figure 2.2 Ratio between the outlet concentration and the standard limit value for different parameters

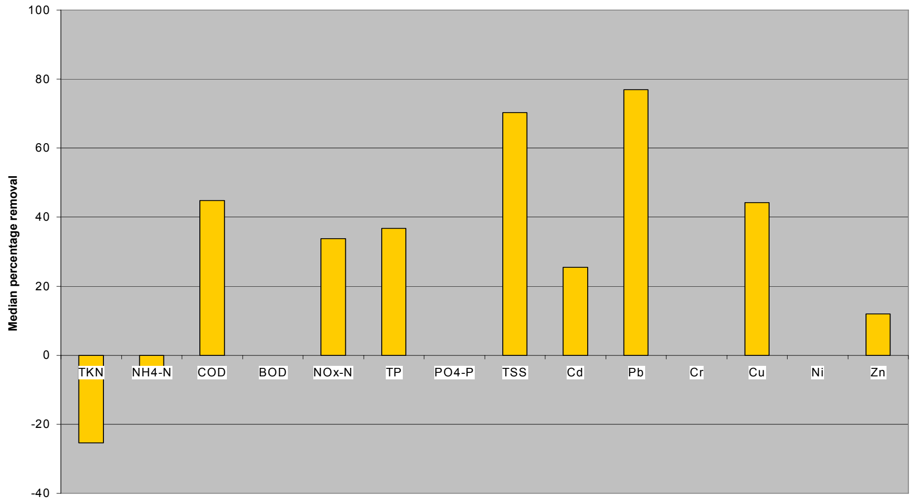

Figure 2.3 Median removal efficiencies for the referred studies of the retention ponds

Despite the paucity of data, an attempt was made to link the effluent water quality with the specific areal ratio, calculated using the total watershed area. The results are presented in Figure 3.3. As with the results for retention ponds, the outlet concentrations tend to decrease with increasing specific areal ratio up to approximately 3%. However this figure should be treated with caution, due to the lack of data for many parameters.

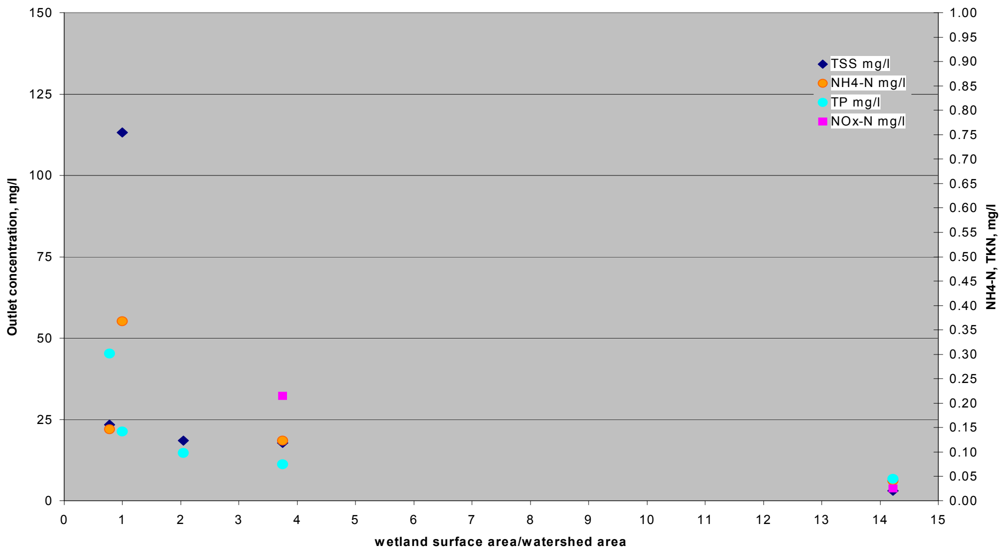

Figure 2.4 Wetland effluent water quality as a function of specific areal ratio

Another study (Carleton J.N et. al, 2001) has compared the removal efficiency of 49 separate wetland systems (located in the USA), receiving urban or agricultural storm water run-off. The authors investigated the applicability of a "*k-C**" model for stormwater treatment wetlands for certain parameters: OP, TP, NH_3, NO_3^{2-}, TN, TSS, Cd, Cu, Pb and Zn.

According to the *k-C** approach, concentration at the outlet of the system is described by the following equation:

$$Ln\ \frac{C\ -\ C^{*}}{C_i - C^{*}} = -\frac{k_a}{q}$$

where q is the hydraulic loading rate (e.g. in m/yr), C_i is the inlet concentration, C^* is the irreducible background concentration of the constituent, and k_a is the "areal" rate constant, expressed in units of length over time.

Their conclusions were that the long-term pollutant removals in stormwater wetlands can be adequately described in terms of mean detention time and hydraulic loading rate using the same kinds of first order steady flow design equations currently employed for waste water treatment wetlands. The higher the wetland area/watershed area ratio is the bigger the removal efficiency is. The investigated parameters reach maximum removal efficiencies (for naturally impounded wetlands) of TSS 90%, TN 40%, Pb 90%, Zn 80%, Cu 90%, for an areal ratio of 20-30%.

The results presented above consider wetlands that are constructed as wetland basins. There is another type of wetland, which are constructed as channels with a wetland bottom. Surprisingly, their water quality appears to differ quite substantially from the wetland basins, so they will be discussed separately.

Table 2.2 presents a summary of the effluent parameters (BMP data base, 2003; Oberts and Osgood, 1991). Figures 2.5 and 2.6 present the effluent water quality compared to standard limits. The wetland channel shows a poor performance for TSS and NOx-N. Though the median removal for TSS is the highest, the 75th percentile of the average values for TSS and Nox-N is still about four times higher than the standard limits. This might be due either to the incomplete data set or to poor hydraulic regime during the storms, which allows higher velocities and possible re-suspension of the already accumulated sediments. More data is necessary for a more robust conclusion.

Table 2.2 Summary of the water quality performance for wetland channel (7 studies)

	COD				BOD			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	45	31	109	45	5.99	5.25	6.93	-56
1 Quartile/min removal	42	26	88	45	5.88	5.15	6.82	-62
3 Quartile /max removal	49	37	131	45	6.09	5.34	7.03	-51
No Samples	2	2	2	1	2	2	2	2
	NH4-N				NOx-N			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	0.32	0.25	0.87	3	1.18	0.81	2.06	14
1 Quartile/min removal	0.12	0.06	0.42	-157	0.10	0.07	0.20	-9
3 Quartile /max removal	0.49	0.40	1.13	50	1.99	1.35	4.25	58
No Samples	4	4	4	4	7	7	6	6
	TKN				TP			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	1.27	0.89	2.97	20	0.25	0.21	0.33	36
1 Quartile/min removal	1.19	0.78	2.14	16	0.15	0.13	0.21	22
3 Quartile /max removal	1.36	1.04	3.69	23	0.34	0.28	0.45	75
No Samples	3	3	3	2	6	6	6	3
	PO4-P				TSS			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal					56	44	82	46
1 Quartile/min removal					20	14	43	39
3 Quartile /max removal					78	64	96	85
No Samples	0	0	0	0	6	6	6	3
	Cd				Pb			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal					8.66	5.96	135	15
1 Quartile/min removal					6.40	4.23	32	12
3 Quartile /max removal					8.29	7.29	195	17
No Samples	0	0	0	0	5	3	3	2
	Cr				Cu			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	24	20	30	30	12.9	10.0	15.0	12
1 Quartile/min removal	23	20	30	29	12.2	10.0	12.5	3
3 Quartile /max removal	24	20	30	31	13.5	10.0	17.5	21
No Samples	2	2	2	2	2	2	2	2
	Ni				Zn			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	36	30	40	10	40	27	64	9
1 Quartile/min removal	36	30	40	9	36	25	61	9
3 Quartile /max removal	36	30	40	11	43	28	67	9
No Samples	2	2	2	2	2	2	2	1

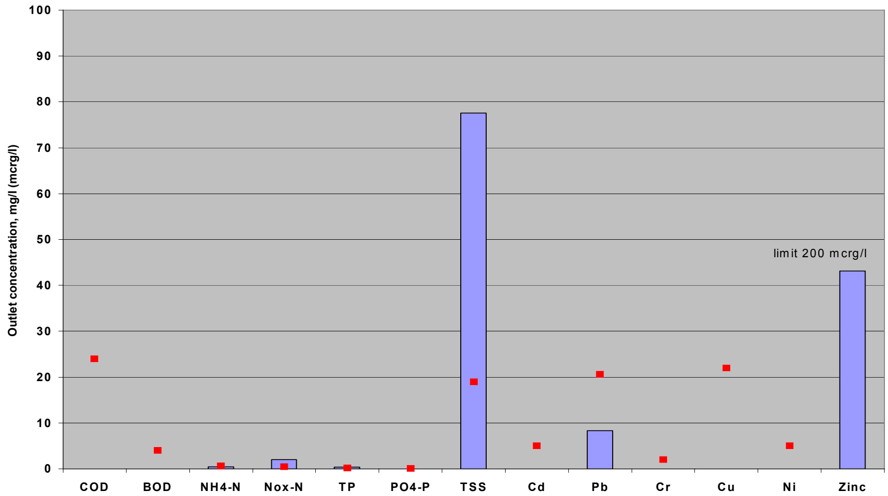

Figure 2.5 Wetland channel effluent water quality (75 percentile of the outlet concentrations), based on the referenced studies

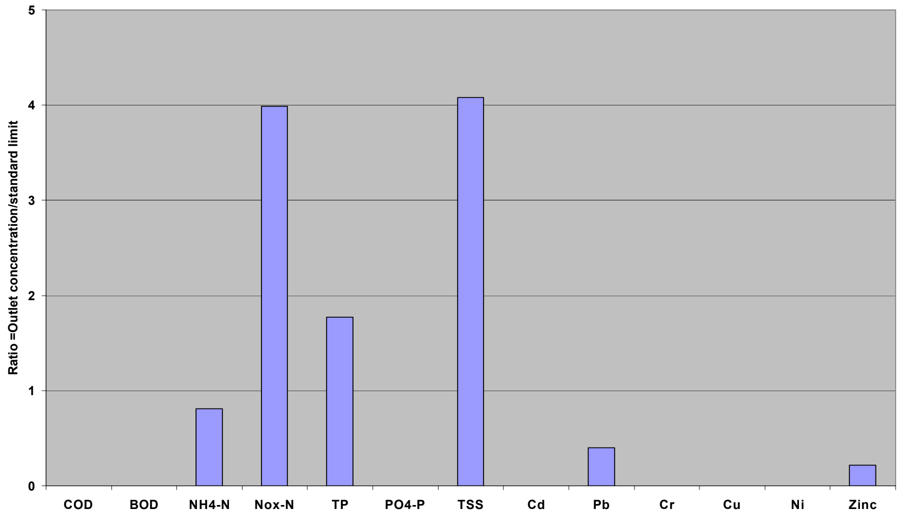

Figure 2.6 Relation between the outlet concentration and the standard limits for different parameters

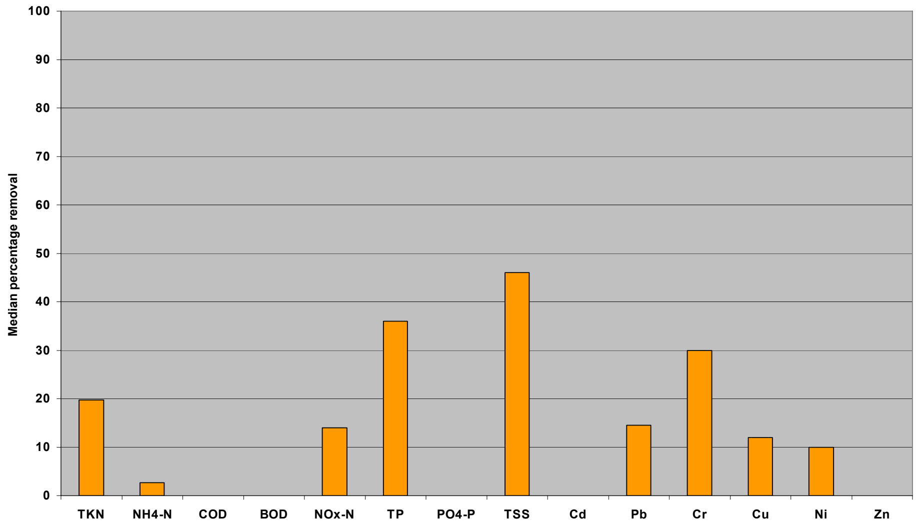

Figure 2.7 Median removal efficiencies for the referred studies of the wetland channel

3. Hydraulic and water quality performance of detention basins

3.1 Hydraulic performance

No studies have been found reporting on the hydraulic performance of detention basins.

3.2 Water quality performance

The American BMP database (2003) reports 12 case studies for detention basins with grassy bottoms. One more study has been found from a French team (Bardin J.P., et all, 2001) for a two chamber detention basin comprising a settlement chamber and infiltration basin. A summary of the results is presented in Table 3.1.

Figures 3.1 and 3.2 present effluent water quality in comparison to the standard values. Probably as a result of the shorter detention times, most of the parameters are up to three times above the standard limits. The concentrations of most of the metals are still below the standard values, except for Lead and Chromium.

Table 3.1 Summary of the water quality performance for detention basin (13 case studies)

	COD				BOD			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	35	21	73	35				
1 Quartile/min removal	34	17	58	7				
3 Quartile /max removal	37	25	82	42				
No Samples	3	3	3	3	0	0	0	0
	NH4-N				NOx-N			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	0.20	0.11	0.52	5	0.72	0.55	1.15	7
1 Quartile/min removal	0.15	0.08	0.52	-33	0.34	0.25	0.87	-6
3 Quartile /max removal	0.24	0.15	0.52	44	1.13	0.88	1.61	82
No Samples	2	2	1	2	8	8	8	8
	TKN				TP			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	1.67	1.29	2.68	6	0.25	0.27	0.43	33
1 Quartile/min removal	1.29	1.03	1.81	-8	0.19	0.24	0.25	-56
3 Quartile /max removal	2.20	1.70	2.91	23	0.30	0.37	0.60	58
No Samples	5	5	5	5	9	9	9	9
	PO4-P				TSS			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	0.09	0.07	0.19	9	32	21	61	53
1 Quartile/min removal	0.04	0.03	0.08	-45	19	12	45	-81
3 Quartile /max removal	0.11	0.08	0.28	27	42	28	72	88
No Samples	7	7	7	6	10	10	9	10
	Cd				Pb			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	1.20	0.51	4.15	35	37	15	78	56
1 Quartile/min removal	0.50	0.31	0.76	-29	14	9	34	-35
3 Quartile /max removal	1.75	0.70	4.62	50	39	21	97	81
No Samples	5	4	4	4	8	8	7	8
	Cr				Cu			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	5.20	2.15	15.98	12	21	15	39	36
1 Quartile/min removal	4.26	1.26	7.78	-8	18	11	30	-2
3 Quartile /max removal	6.67	2.73	21.20	32	24	20	43	61
No Samples	3	3	3	2	8	8	7	8
	Ni				Zn			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average/median removal	4.77	3.23	12	42	130	90	245	44
1 Quartile/min removal	4.14	2.53	10	13	55	45	72	-54
3 Quartile /max removal	5.21	3.90	14	52	189	124	278	81
No Samples	4	3	3	3	9	9	9	8

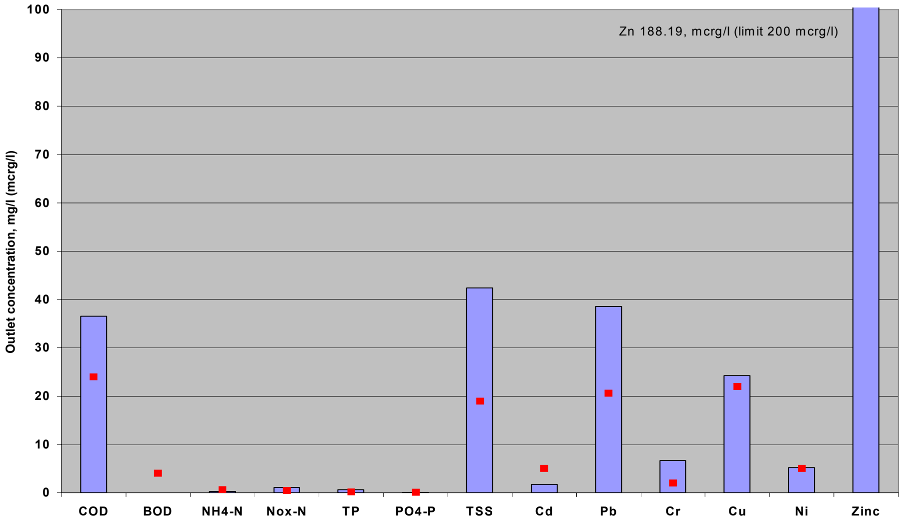

Figure 3.1 Detention basin effluent water quality (75 percentile of the outlet concentrations), based on the referenced studies

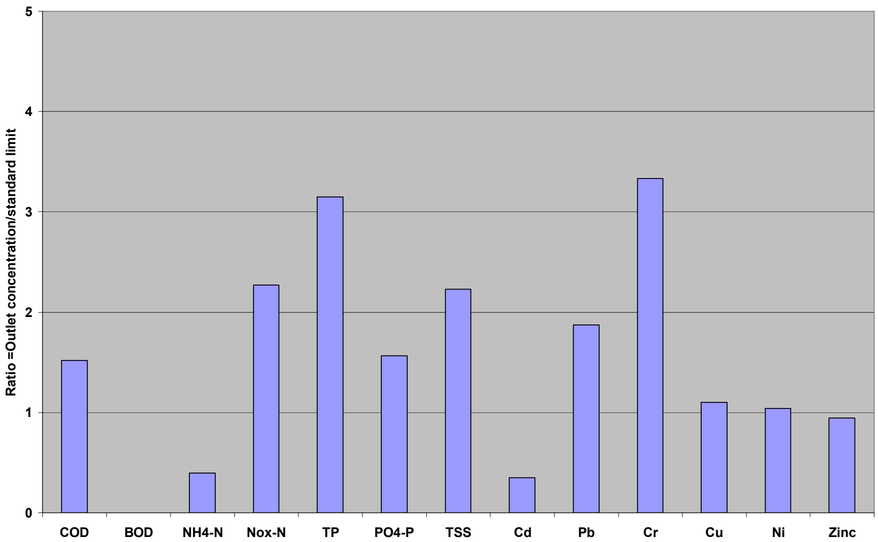

Figure 3.2 Ratio between the outlet concentration and the standard limit value for different parameters

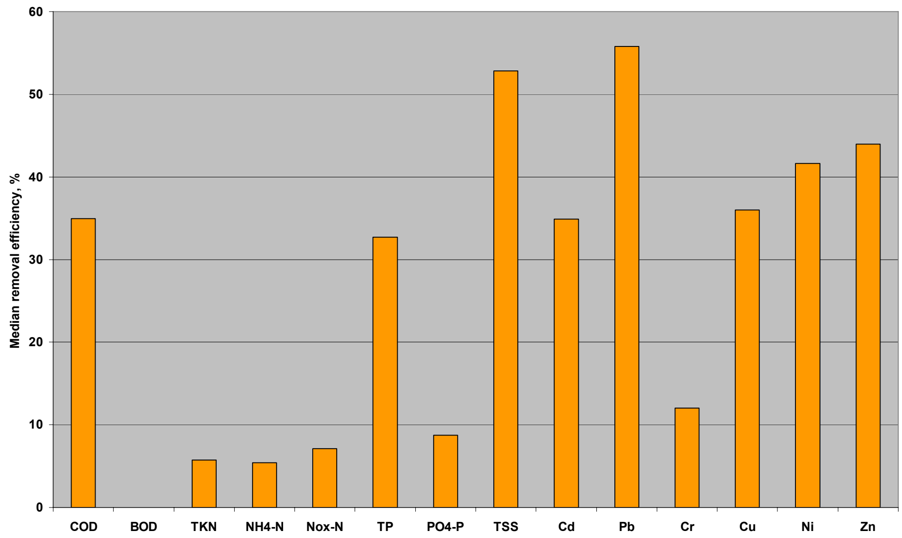

Figure 3.3 Median removal efficiencies for the referred studies of the detention basin

4. Hydraulic and water quality performance of swales

4.1 Hydraulic performance

Three studies have been found that investigate the hydraulic performance of swales during storm events. The key findings are presented in Table 4.1.

Table 4.1 Summary of Swale

Reference	Location	Lag time	Initial Run-off loss	Percentage outflow volume reduction	Peak reduction
		min	**mm**	**%**	**%**
Macdonald K&Jefferies C,2003	Emmock Wood, SC	11.6	5	6.53	52.2
Macdonald K&Jefferies C,2003	West Grange – modified outlet, SC	14.3	1.2	36.7	1.2
Macdonald K&Jefferies C,2003	West Grange, original outlet, SC		3.18	6.3	65
Yousef Y.A, et all, 1987	Maitland, FL, USA			52.40	
Yousef Y.A, et all, 1987	EPCOT , FL, USA, 20% grass coverage			62.05	
Yousef Y.A, et all, 1987	EPCOT , FL, USA, 80% grass coverage			49.68	
Barret M, 1998 *	MoPac Expressway, TX, USA			40	

*During saturated ground conditions, the swale percentage outflow reaches 90%

The percentage outflow volume reduction has been calculated in all the studies. However the values range from 6.3% to 62.05%. It should be noted that the Emmock Swale has a gravel layer underneath, which is likely to contribute to additional flow attenuation and enhanced infiltration and produce correspondingly low percentage outflows. Other studies (Kuo Jan-Tai, et al) have recorded 0% outflow for storms with less than 12.7mm total precipitation. Complete infiltration of run-off was reported for two 30m swales from storms with less than 5-7 mm total precipitation.

Such a simple comparison of small datasets is not enough for making a reliable conclusion regarding the hydraulic performance of swales. Since the "loss" of outflow is due mainly to soil characteristics, the hydraulic performance should be directly related to the soil infiltration characteristics and other factors that influence the infiltration capacity. However no studies have been found that investigate this issue specifically.

Another study (Mazer et al., 2000) characterises the hydraulic performance of swales in terms of peak water depth (cm), velocity (m/s), discharge (m^3/sec), hydraulic residence time (HRT, the time required for an aliquot of water to travel from inlet to outlet, in minutes), and hydraulic loading rate (the ratio of inflow discharge at the 10m gauge to swale area, in m/d).

Eight swales were investigated. The results are presented in the following table:

Table 4.2 Mean peak depth and mean peak flow during rain events, Mazer, G., et al. 2001

Swale	Slope	Width	Length	Spring		Summer		Check dams	Vegetation
				Mean peak depth	Mean peak flow	Mean peak depth	Mean peak flow		
	%	m	M	mm	m/sec	mm	m/s		
SAY7	1.9	1.8	65	44	0.15	2	0.02	No	high
SAY8	1.9	1.8	50	70	0.20	39	0.14	No	low
SAY9	0.2	1.9	50	59	0.10	39	0.08	No	low
DISC	1.8	2	84	88	0.13	104	0.14	No	high
CUH	1.6	1.7	76	15	0.03	14	0.04	No	high
PLP	0.7	3.7	60	147	0.03	21	0.01	Yes	high
PLEa	0.4	1.3	29	76	0.02	57	0.01	Yes	both
PLEb	0.5	2.2	66	102	0.03	13	0.003	Yes	low
average	**1.13**	**2.1**	**60**	**75**	**0.09**	**36**	**0.06**		
max	**1.90**	**3.7**	**84**	**147**	**0.20**	**104**	**0.14**		
min	**0.20**	**1.3**	**29**	**15**	**0.02**	**2**	**0.003**		

The vegetation contributes to a higher retention of water in the swale resulting in slightly lower peak flow velocities. However the study reported that the vegetation did not appear to perform its intended function of significant flow retardance. The authors report that the maximum discharge and water velocity are in compliance with the design values (0.14 m^3/sec and 0.3 m/sec). However the maximum peak flow events for both the spring and summer sampling periods occurred during storm events of much lower intensities than what may be regarded as the maximum treatable (2 year, 24 h) for biofiltration according to their design standards.

The hydraulic retention time varied between 2.5 - 31.5 minutes (mean 13.2 minutes) during the spring events and 4.8 - 89.3 minutes (mean 26.5 minutes) during the summer events. Below are presented some relationships between the mean peak velocity and swale design, derived from the published data, Mazer G., et. al (2001). The swales are situated in different areas, with different soil conditions. However in general the increase in slope leads to increases in mean peak velocity, while the increase in length leads to decreasing mean peak velocity. The presence of check dams structures favourably influences the hydraulic performance.

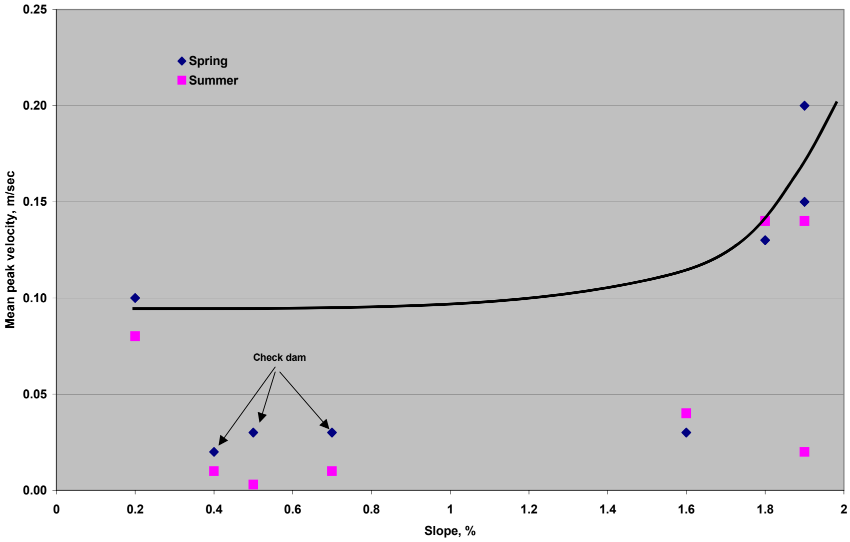

Figure 4.1 Relation between the slope and mean peak velocity, Mazer G., et.al. 2001
(a trend line was drawn for data with comparable design parameters)

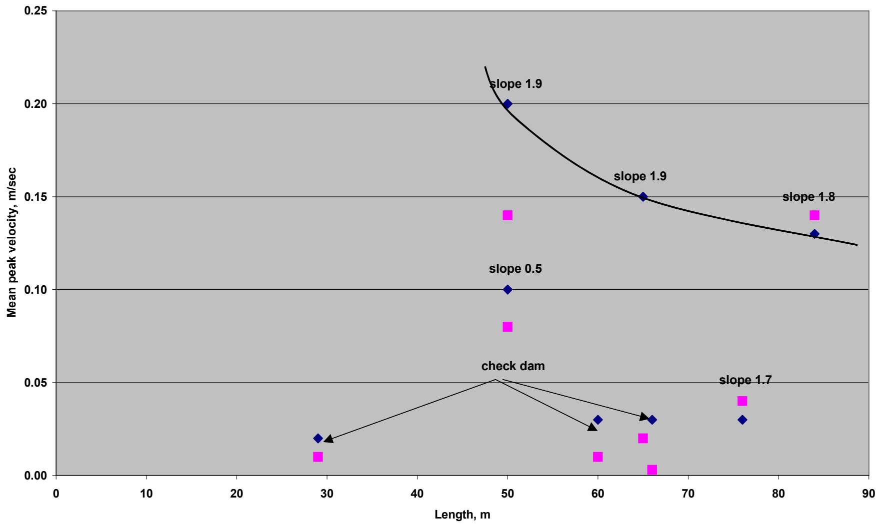

Figure 4.2 Relation between the swale length and mean peak velocity, Mazer G., et.al. 2001 (a trend line was drawn for data with comparable design parameters)

4.2 Water quality performance

Traditionally, most of the studies report water quality in terms of removal efficiencies. As with the other SUDS types, the percentage removal varies significantly, often from negative to close to 100 % values. Table 4.3 presents the average, minimum and maximum removal efficiencies from 21 grass swales: Barret M, et al, (1998a), Barret M, et al, (1998b), Yousef Y.A, et al, (1987), Fletcher, et al, (2002), BMP database, (2003), Macdonalds K& Jefferies C, (2003), Macdonalds K& Jefferies C, (2003b).

Table 4.3 Summary of the water quality performance for swale (21 case studies)

	COD				BOD			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	44.6	39.3	81.0	35	4.72	4.01	7.35	2.0
1 Quartile/min removal	33.6	37.7	54.2	-13	4.43	3.44	7.34	-7.3
3 Quartile /max removal	47.3	44.7	111.1	63	5.76	5.14	7.38	14.3
No Samples	9	6	6	6	6	3	3	6
	NH4-N				**NOx-N**			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.41	0.04	0.29	6	0.47	0.37	2.27	2
1 Quartile/min removal	0.15	0.02	0.07	-10	0.19	0.20	0.67	-42
3 Quartile /max removal	0.60	0.04	0.41	74	0.72	0.51	1.28	74
No Samples	10	4	4	10	21	12	12	21
	TKN				**TP**			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	1.5	1.0	3.8	31.1	0.34	0.29	0.68	-1
1 Quartile/min removal	1.4	0.7	2.5	-9.0	0.19	0.17	0.35	-193
3 Quartile /max removal	1.5	1.2	5.4	51.8	0.45	0.31	0.88	100
No Samples	8	6	6	9	24	14	14	24
	PO4-P				**TSS**			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.28	0.14	0.86	-22	43	15	72	66
1 Quartile/min removal	0.16	0.09	0.33	-448	15	9	24	3
3 Quartile /max removal	0.43	0.18	0.53	100	35	21	45	87
No Samples	14	8	8	14	20	14	14	20
	Cd				**Pb**			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	1.08	0.23	2.34	56	22	13	35	58
1 Quartile/min removal	0.25	0.19	0.34	14	3	2	5	16
3 Quartile /max removal	0.89	0.26	2.32	81	19	9	30	83
No Samples	9	7	7	7	17	13	13	17
	Cr				**Cu**			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	3.68	2.84	13.80	48	17.21	8.88	18.55	46
1 Quartile/min removal	2.50	2.30	8.35	9	5.50	4.50	9.41	-85
3 Quartile /max removal	4.44	3.38	19.24	53	21.10	13.85	26.36	75
No Samples	3	2	2	3	13	10	10	13
	Ni				**Zn**			
	µg/l	µg/l	µg/l	%	µg/l	µg/l	µg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	3.39	2.60	14.92	62	48	33	2884	43
1 Quartile/min removal	2.88	2.29	8.71	50	30	23	34	-14
3 Quartile /max removal	3.61	2.91	21.12	76	69	40	100	91
No Samples	4	2	2	4	17	12	11	17

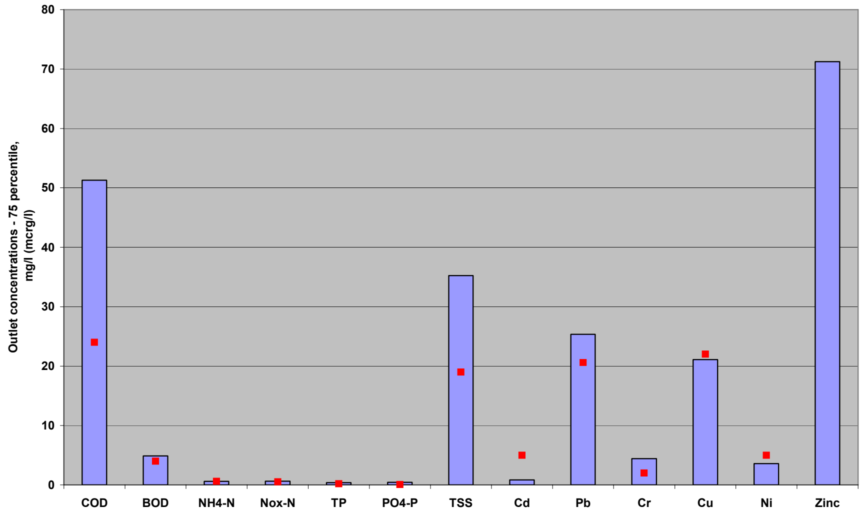

Figure 4.3 Swale effluent water quality (75th percentile outlet concentrations), based on the referenced studies (Note: the standard limit for Zn is 200 mcrg/l)

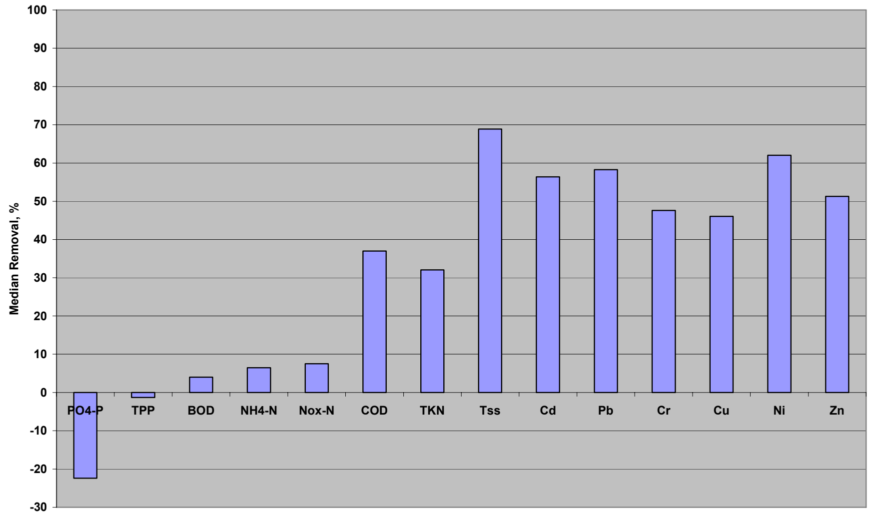

Figure 4.4 Ratio between the outlet concentration and the standard limit value for different parameters

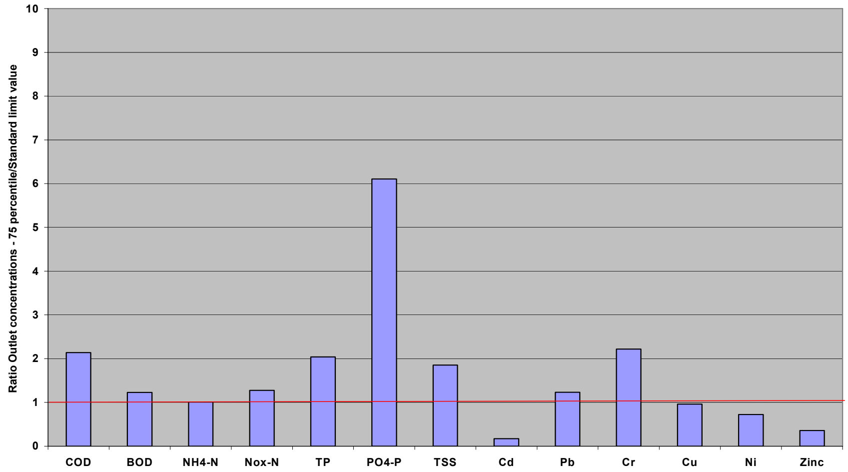

Figure 4.5 Swale median removal efficiencies for the referenced studies

The dots show the value of the standard limit for the corresponding parameter. The biological parameters are the parameters of concern as opposed to the heavy metals. A ratio of 1 means that the outlet concentration is equal to the standard limit. The poorest performance is for PO4-P, followed by Cr, TP, COD and TSS.

The removal efficiencies show that all biological parameters have poor median removal efficiency (i.e. less than 10%), with Phosphorus efficiencies having negative values. This is probably due to the ionic form in which these parameters exist, which make them hard to remove via sedimentation. Many authors report improved load-based removal efficiencies than concentration-based efficiencies, thus emphasising the positive influence of infiltration upon the water quality. In this respect, the hydraulic, as well as water quality performance is strongly dependent on the infiltration characteristics of the soil.

Relationship between water quality and swale design

Several studies have attempted to find relationships between the water quality, hydraulic regime and swale geometry. The parameters likely to influence the water quality performance are accepted to be: length, water flow velocity, water depth, longitudinal slope, hydraulic retention time and season. A comprehensive investigation is not however available, which makes comparison of the different studies difficult. Below are key findings of the different studies.

1. ***Length:*** The longer swales have better water quality performance. However different studies report different optimum lengths**:** 20m, Walsch Patrick, (1997); 75m, Yu Shaw L. et. al, (2001); 32.5m, Fletcher D., (2002). Most of the authors mention that the removal efficiency depends not only on the length but also on the form of pollutants. The fine particles and pollutants in ionic form may not be removed by the swale. In such cases some authors suggest a combined system i.e. a swale followed by retention pond or wetland. Figures 4.6 and 4.7 summarise the results of all studies reviewed here. The length of the swale positively influences the removal rates of TSS and heavy metals (except for Pb), while the soluble pollutants in the outflow tends to increase with increasing length.

   Thus, according to this analysis, swale lengths greater than 50m ensure good effluent concentrations for TSS and most of the heavy metals, however the increasing length of swale seems to influence negatively the removal of nutrients.

2. ***Water depth*****:** The removal efficiency for TSS, organic material and most metals decreased with increasing water depth. Small water depths are also favourable for the development of vegetation in the swale. Walsh Patric et al., 1997 reports higher removal efficiencies at 3-4cm, than at 7.5 –10cm. Fletcher, (2002) reports best removal efficiencies for TSS at 5.4cm depth, compared to efficiencies at depths of 6.5 – 9.3cm. Yosef at al, (1987) finds 3.5-4cm is likely to be optimum. However lower water depths has zero or negative effect upon the removal of fine suspended particles and pollutants in ionic form.

3. ***Slope:*** All the studies recommend low longitudinal slopes. Yu Shaw et all, (2001) recommends slopes up to 3%. Mazer G, et al (2001) recommends slopes less than 1.5 % to prevent erosion.

4. ***Hydraulic Retention Time (HRT)***: Walsh Patric, (1997) reports good pollutant removal for retention times of around 9 minutes. However Mazer G., 2001 reports recommended minimum retention times of 9 minutes and required minimum times of 5 minutes.

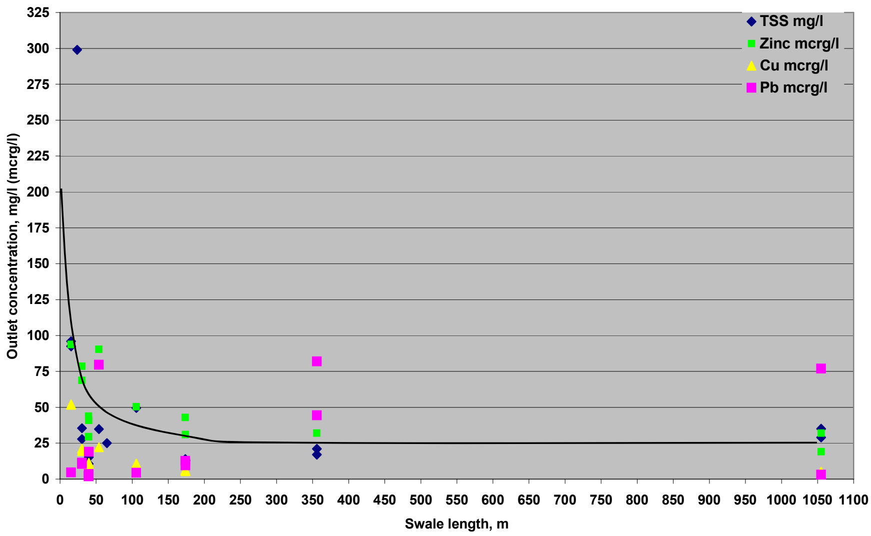

Figure 4.6 Relation between the length of the swale and outlet concentrations of Zn, Cu and TSS

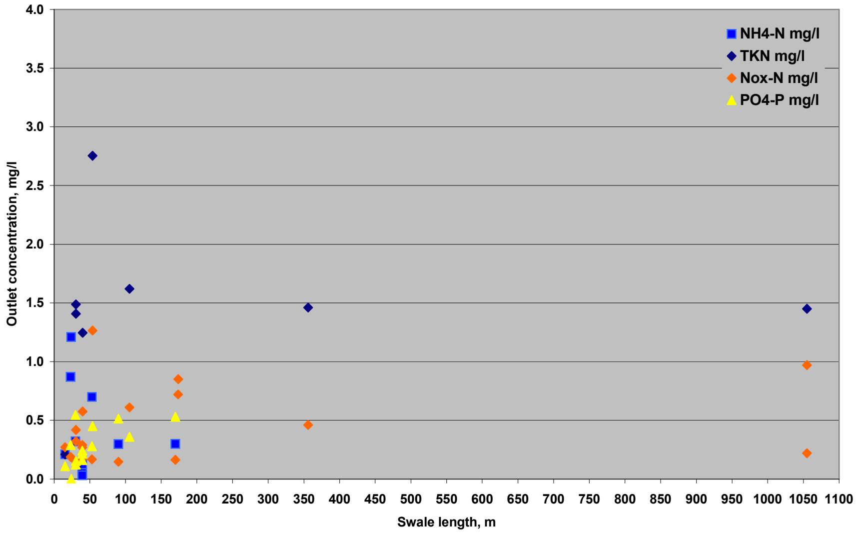

Figure 4.7 Relation between the length of the swale and outlet concentrations of NOx-N, NH4-N and PO4-P

5. Hydraulic and water quality performance of pervious pavement car parks

5.1 Hydraulic performance

Four UK studies have been reviewed: two in Scotland and three in England. Common parameters that have been measured are: the lag time, percentage outflow and peak flow reduction. The Scottish team has also introduced the parameter *benefit factor*, which is a percentage relation of the total pervious outflow water to the total tarmac runoff. However this parameter can be calculated only when there is data about the runoff from an equivalent tarmac surface in the vicinity. Abbot et al (2000) have introduced the 'storm attenuation factor', which is the "ratio of the duration of the outflow to the duration of the storm event".

In Table 5.1 these parameters are compared. Usually the minimum, maximum and average values are reported. In addition, a compound average value and 75th %-ile have been derived from all the reported values. The results show that the average value is quite similar to the 75th percentile. Data from other published sources give comparable results.

All the studies report that many factors influence the hydraulic performance of pervious pavements. Some authors (Mantle, 1993) have developed mathematical models for predicting the outflow. The input data are the total rainfall, the duration and the antecedent conditions.

Table 5.1 Hydraulic Parameters of Studied Lined Permeable Pavements

	Reference	Location	Lag time, min			Initial run off loss, mm			Percentage outflow			Peak flow reduction, %		
			min	max	avr	min	Max	avr	min	max	avr	Min	max	avr
1	Pratt C.J., et al, 1995	*Trent Polytechnic, Nottingham						3.0			37			
2	Pratt C.J. et al, 1995	*Trent Polytechnic, Nottingham						3.2			34			
3	Pratt C.J. et al, 1995	*Trent Polytechnic, Nottingham						2.4			47			
4	Pratt C.J. et al, 1995	*Trent Polytechnic, Nottingham						2.8			45			
5	Macdonald & Jefferies, 2001	NATS, Edinburgh	29	600	181	3.6	18.6	7.4	2.5	66	21.7	23.7	98.4	76.8
6	Schluter and Jefferies , 2001	RBS, Edinburgh	127	143	90			1.65	14.2	79.5	46.5			
7	Abbot CL at al, 2003	TESCO Wokingham	5	540	106				30	120	67	67	99	88
8	Abbot CL at al, 2003	Welcome Break Wheatley	15	150	52				35	77	48.5	67	91	81
	Average		**23**	**358**	**107**			**3.41**	**20**	**86**	**43**	**53**	**96**	**82**
	75 percentile		**32**	**555**	**125**			**3.15**	**31**	**90**	**47**	**67**	**99**	**85**

*Different materials for sub base layer have been used

Figure 5.1 gives the most common comparison presented in almost all the studies: the relationship between percentage runoff and total rainfall. Though the different studies report different absolute values, percentage runoff tends to increase with increasing total rainfall up to a maximum threshold value.

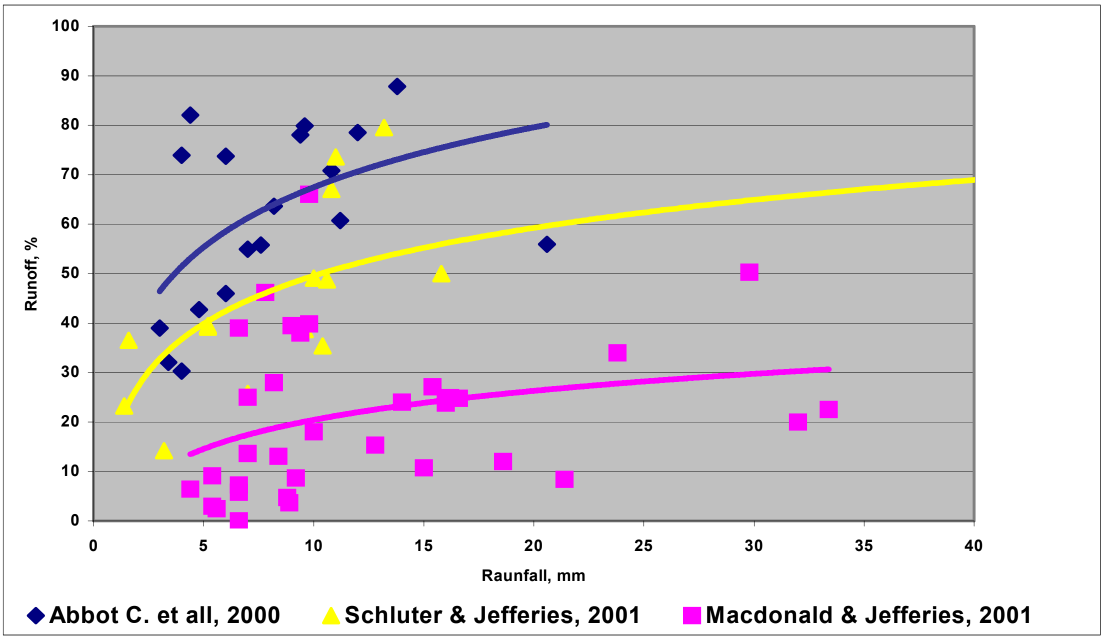

Figure 5.1 Relation between total rainfall and percentage runoff

Figures 5.2 and 5.3 present the change in mean lag time and percentage runoff with API5, according to the data presented in two studies Kirsteen M & Jefferies C, (2001) and Schulter W & Jefferies C, (2001).

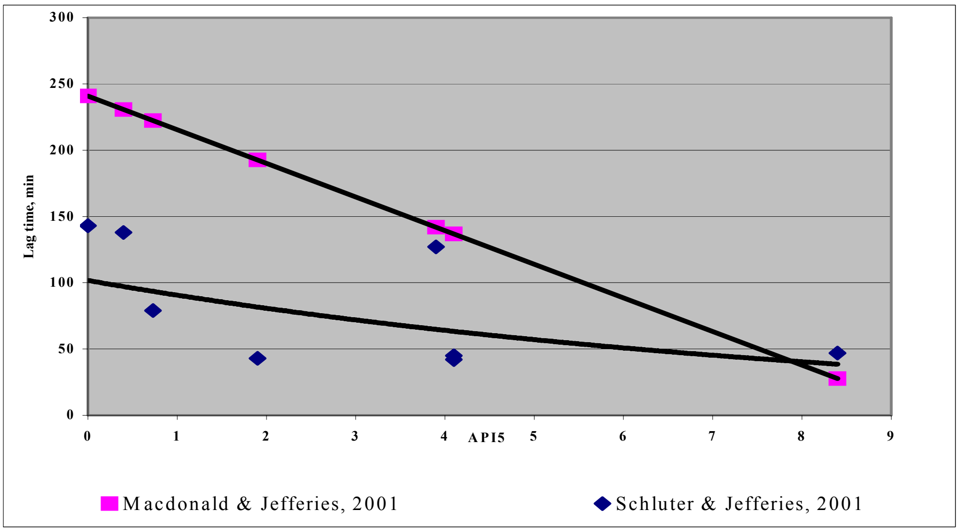

Figure 5.2 Relation between the lag time and API5

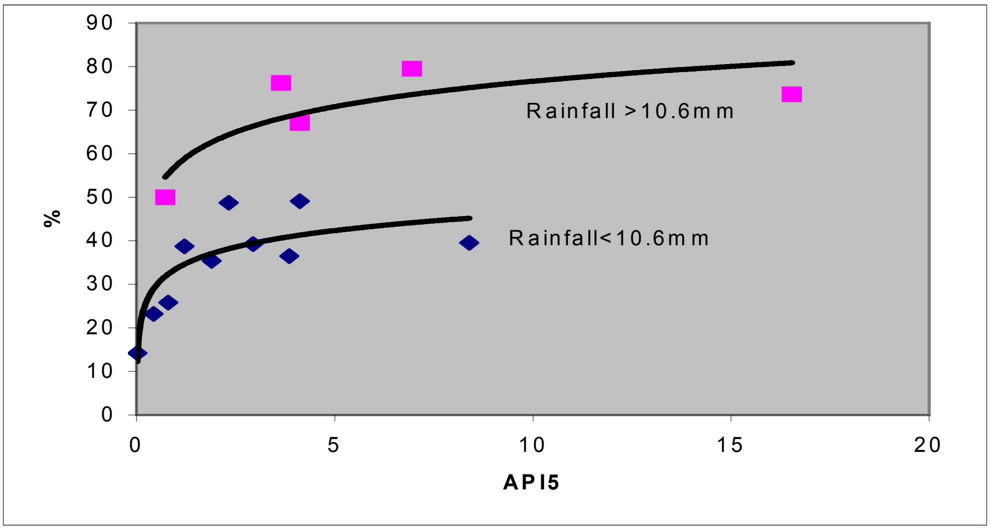

Figure 5.3 Relation between Percentage run-off and API5. Data taken from Schulter W and Jefferies C, 2001

It is obvious that the most unfavourable conditions are at higher API and heavy rain (>10.6 mm). At this point the expected runoff is around 80% of the rainfall, which is a value similar to the run-off from impervious areas.

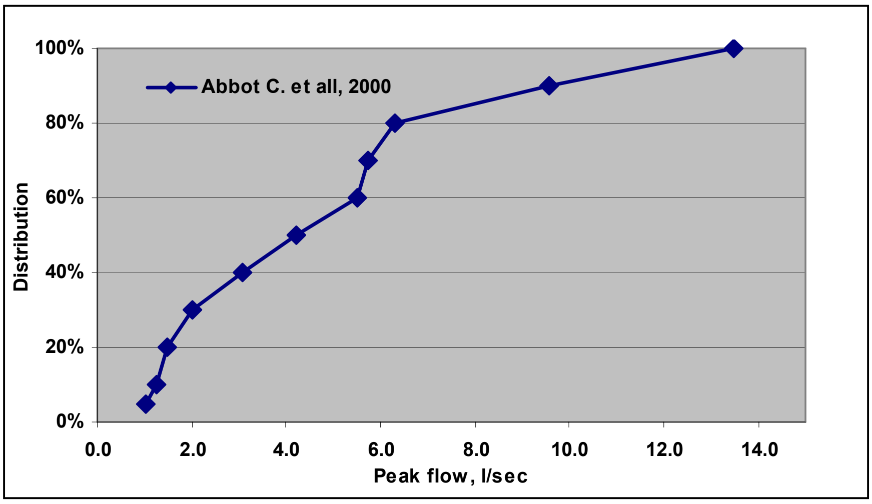

Figure 5.4 Distribution of peak flow, Abbot C., (2000)

Other studies report on pervious pavement car parks with unlined bases which allow the rainfall to infiltrate into the soil. The authors report values of water loss into the subsoil in the range of 87% (Raimbault, 1994) and 96.7%, Colandini, 1997.

5.2 Water Quality Performance

11 studies have been reviewed. Some of these present results from laboratory studies, others discuss permeability and water quality of thin pervious layers (up to 40 mm depth) like porous asphalt or porous concrete blocks Stotz & Krauth, (1994), Brattebo & Booth, (2003),Booth and Leavitt, (1999).

A difficulty arises from the problem that different authors have investigated different parameters or different, incomparable forms of the same parameter. 6 studies have therefore been selected that investigate the performance of comparable systems. Three studies refer to the same car park, but the investigations have been carried at different times. A summary of the results is presented in Table 5.2.

Figures 5.5 and 5.6 present the effluent concentrations compared to standard limits. Most of the parameter concentrations are lower than the standard limits except for NOx-N, PO4-P and Cr. However most of the parameters have less than two values for comparison, so the results should only be considered as being indicative of likely performance.

Table 5.2 Summary of the water quality performance for pervious pavements (11 case studies)

Outlet concentrations	COD				BOD			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	20.2	12.9	35.9	9.0	2.34	1.70	3.80	49
2 Quartile/min removal	19.2	9.3	35.8	6.7	2.12	1.45	3.20	49
3 Quartile /max removal	21.1	16.4	35.9	11.3	2.56	1.95	4.40	49
No Samples	2	2	2	2	2	2	2	1
	NH4-N				NOx-N			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.21	0.03	0.70	32	1.45	1.02	1.73	-165
2 Quartile/min removal	0.15	0.03	0.49	32	1.17	0.69	1.54	-165
3 Quartile /max removal	0.28	0.03	0.92	32	1.72	1.35	1.91	-165
No Samples	2	2	2	1	2	2	2	1
	TKN				TP			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal								
2 Quartile/min removal								
3 Quartile /max removal								
No Samples	0	0	0	0	0	0	0	0
	PO4-P				TSS			
	mg/l	mg/l	mg/l	%	mg/l	mg/l	mg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.12	0.04	0.36	-159	12	3.51	57	68
2 Quartile/min removal	0.11	0.04	0.28	-159	8	1.60	39	56
3 Quartile /max removal	0.14	0.05	0.45	-159	14	3.43	43	75
No Samples	2	2	2	1	5	5	5	4
	Cd				Pb			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	0.57	0.04	2.61	65	5.91	0.71	38	77
2 Quartile/min removal	0.25	0.02	0.70	48	2.43	0.50	7	73
3 Quartile /max removal	0.59	0.03	3.49	70	6.90	0.90	78	82
No Samples	5	5	5	4	5	5	5	4
	Cr				Cu			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	3.69	2.73	7	-26	11.69	3.78	61	-36
2 Quartile/min removal	2.94	2.22	6	-26	8.26	3.50	20	-36
3 Quartile /max removal	4.43	3.25	8	-26	15.00	4.30	127	-5
No Samples	2	2	2	1	5	5	5	3
	Ni				Zn			
	μg/l	μg/l	μg/l	%	μg/l	μg/l	μg/l	%
	Average	min	max	Removal	Average	min	max	Removal
Average /median removal	2.59	0.88	6	63	44	11.06	192	70
2 Quartile/min removal	2.14	0.85	5	63	46	8.00	67	41
3 Quartile /max removal	3.03	0.92	8	63	47	13.30	340	71
No Samples	2	2	2	1	5	5	5	4

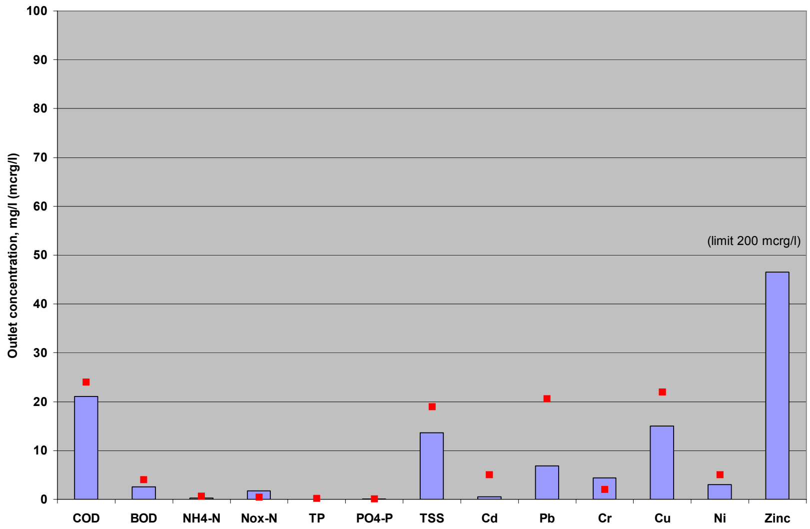

Figure 5.5 Pervious pavement car park effluent water quality (75 percentile outlet concentrations), based on the referred studies

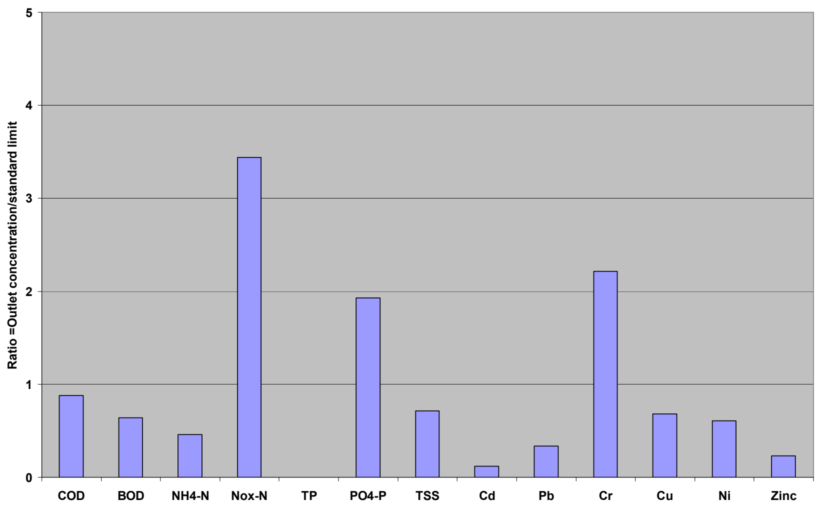

Figure 5.6 Ratio between the outlet concentration and the standard limit value for different parameters

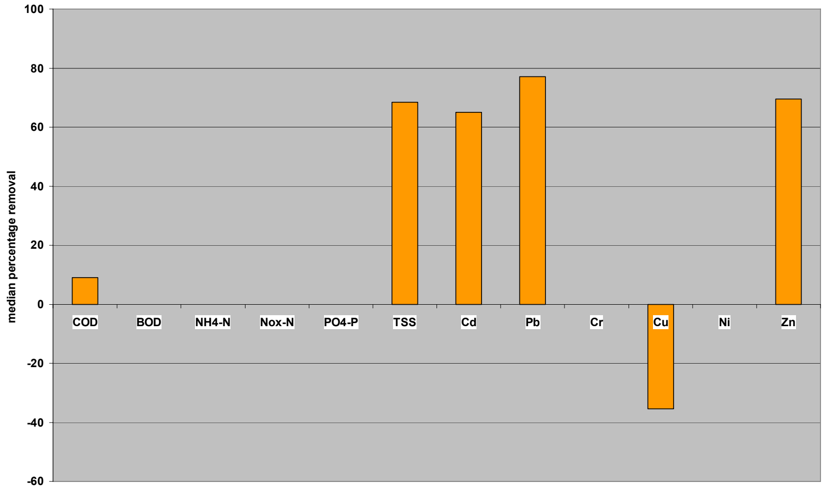

Figure 5.7 Median removal efficiencies for various studies of pervious pavement car parks

Appendix 2 Detail for Modelling of Pervious Pavement at Tesco's, Wokingham

2.1 InfoWorks final calibration parameters

Surface runoff:

	Permeable tarmac	Impermeable tarmac
Surface type	Pervious	Impervious
Initial loss value	2mm (fixed)	2mm (fixed)
Runoff routing value	20 (Rel)	10 (Rel)
Runoff coefficient	0.2 (fixed)	0.95 (fixed)

Ground infiltration:

Soil depth	0.5 m
Percolation coefficient	0.5
Percolation threshold	20 %
Porosity of the soil	60 %

Seasonal parameters:

Season	Mid Winter	Late Autumn, Early Spring	Early Autumn, Late Spring	Early Summer, Late Summer	Mid Summer
Evaporation (mm/day)	1	2	3	4	5
Percolation (% infiltrating)	85	80	75	70	65

Initial soil saturation : 17 %.

A value of 10 for the runoff routing parameter for the impermeable tarmac (road) is high, as such a parameter value has a default of 1. This is to account for the effects of the perforated outlet pipe. An additional 'infiltration' loss was required to account for the 'slow-release' base flow, which was difficult to represent in the model.

2.2 Event Calibration Plots

The following figures show the representation of observed flow events by the calibrated model:

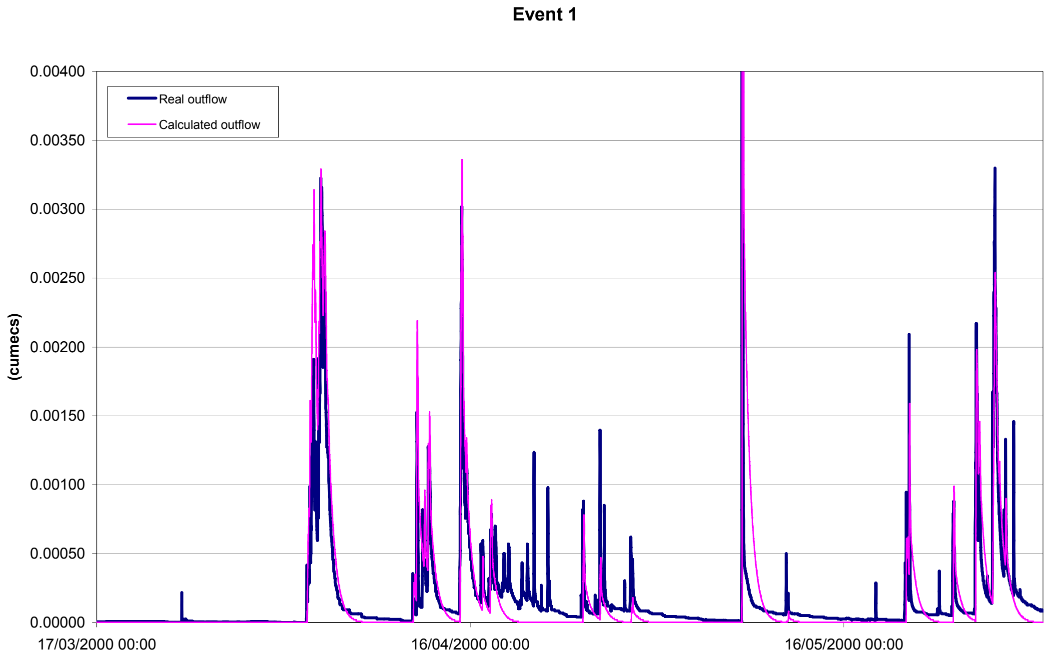

Figure 2.1

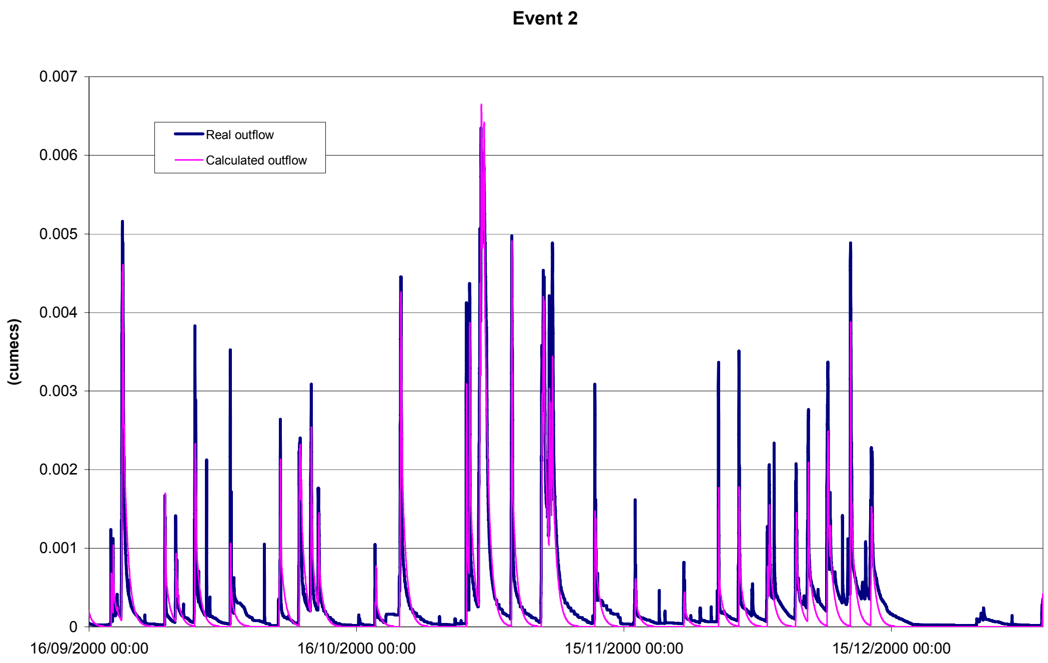

Figure 2.2

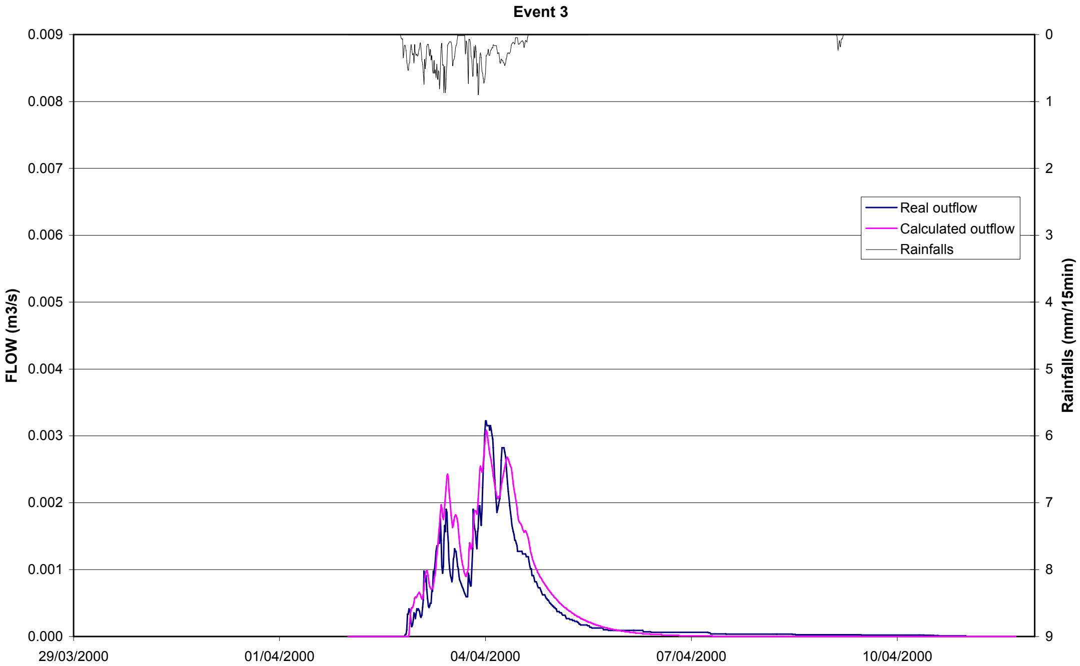

Figure 2.3

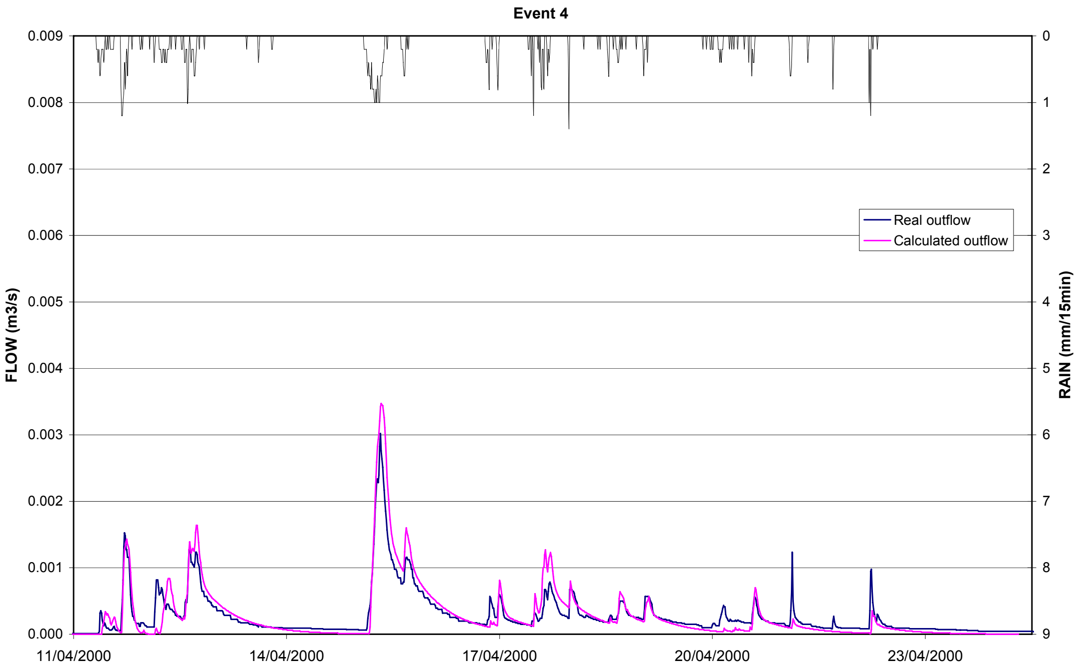

Figure 2.4

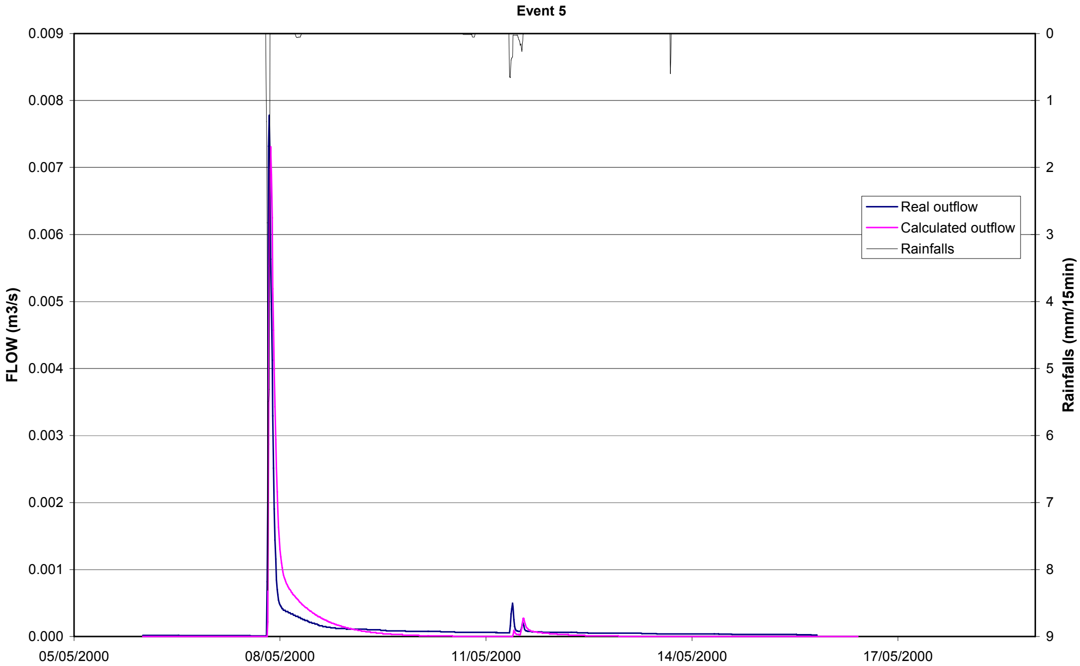

Figure 2.5

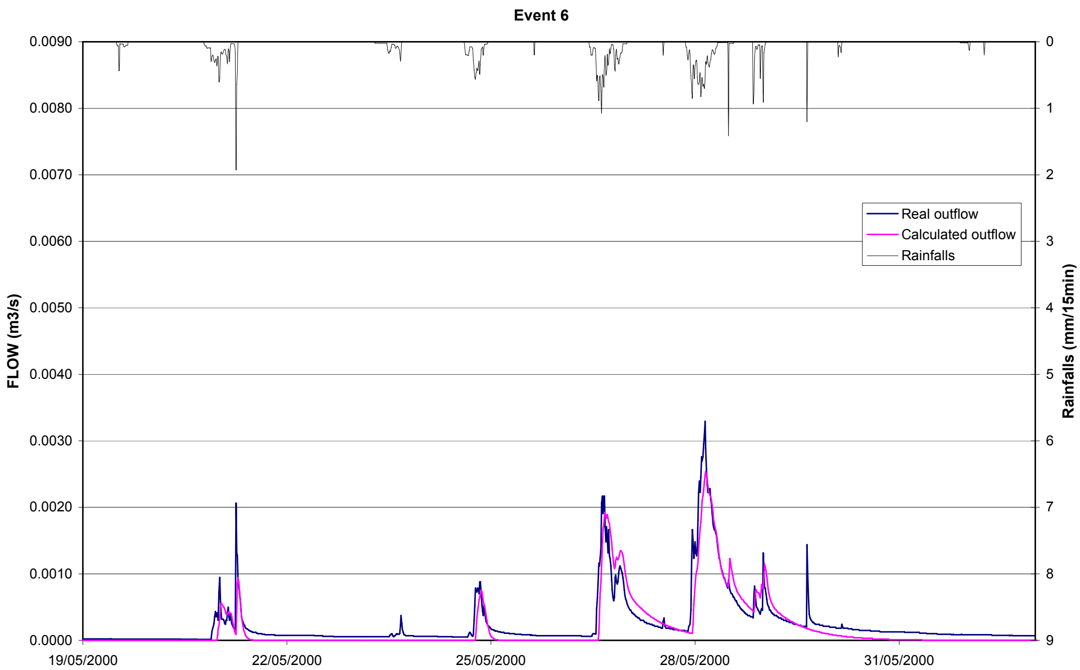

Figure 2.6

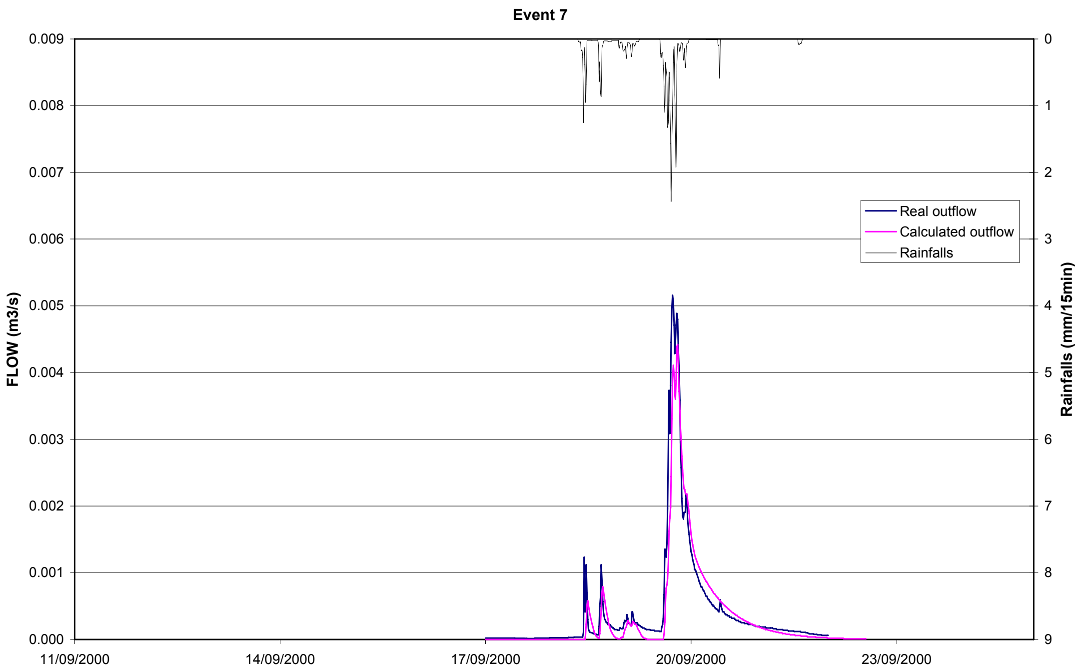

Figure 2.7

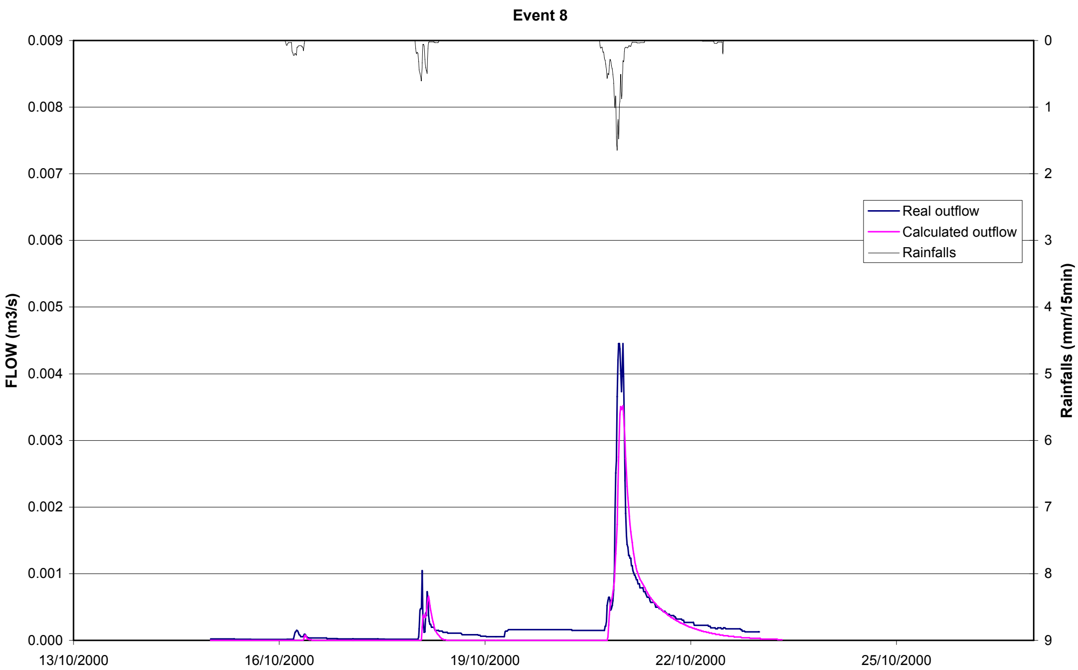

Figure 2.8

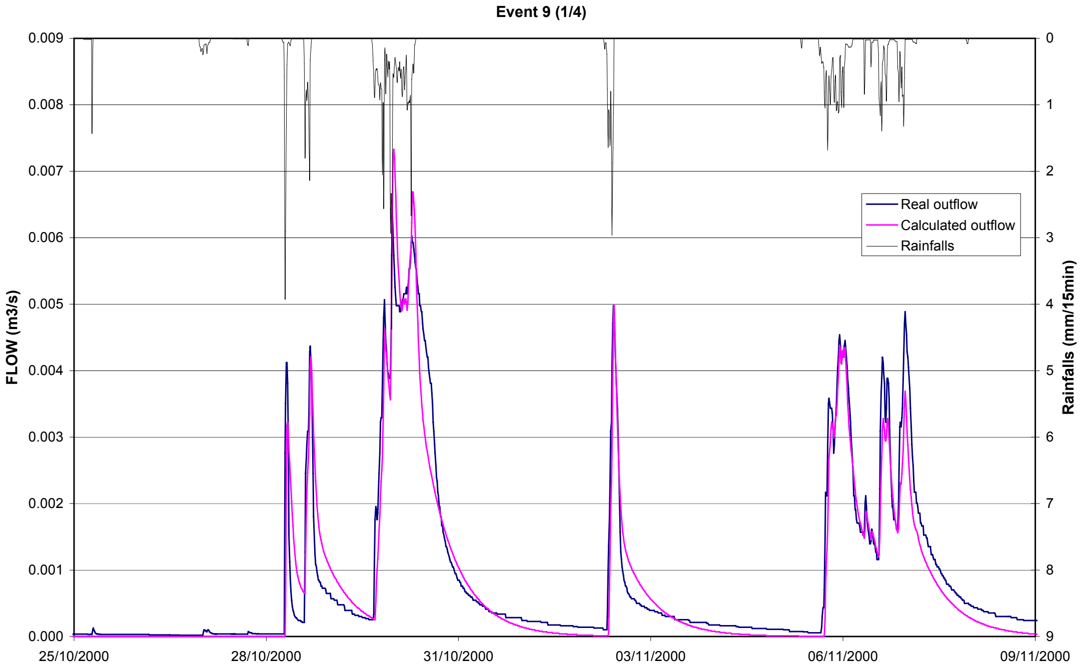

Figure 2.9

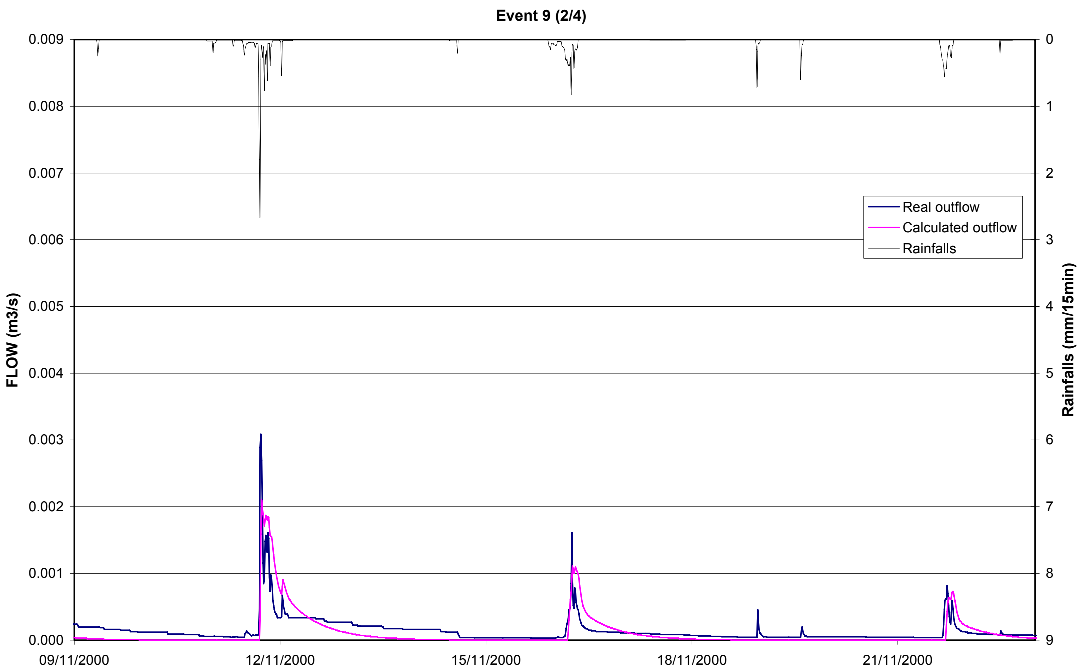

Figure 2.10

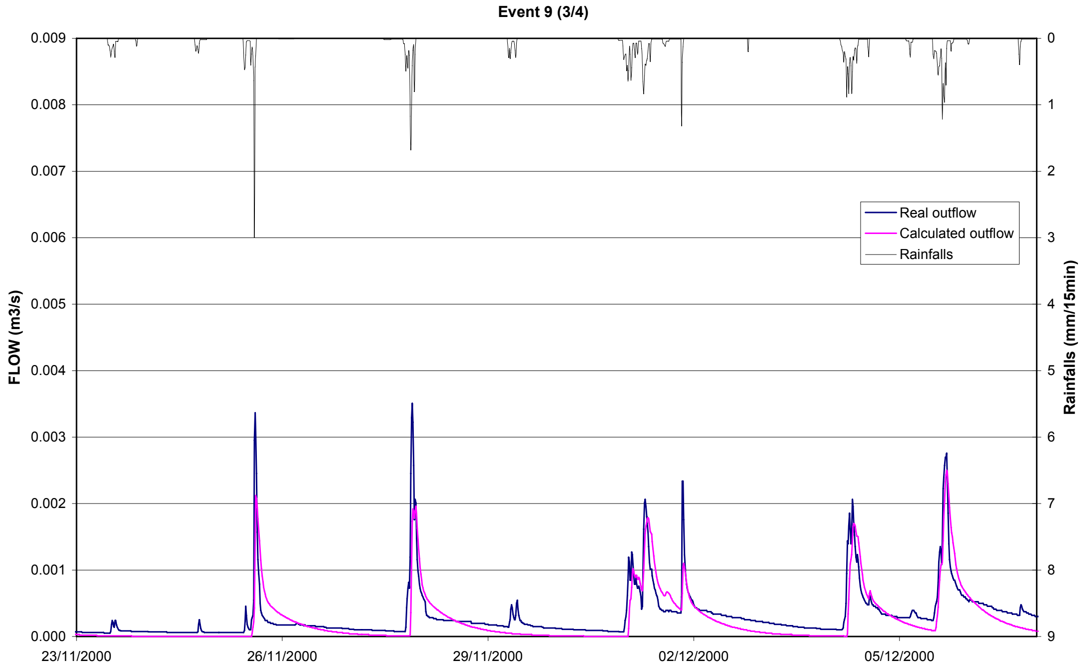

Figure 2.11

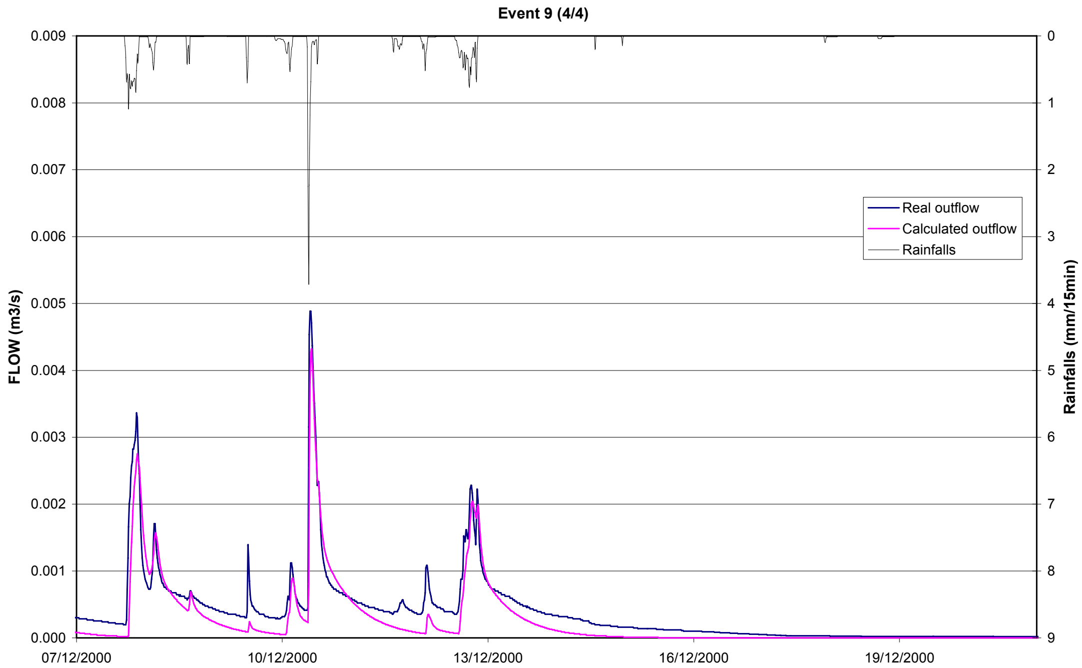

Figure 2.12

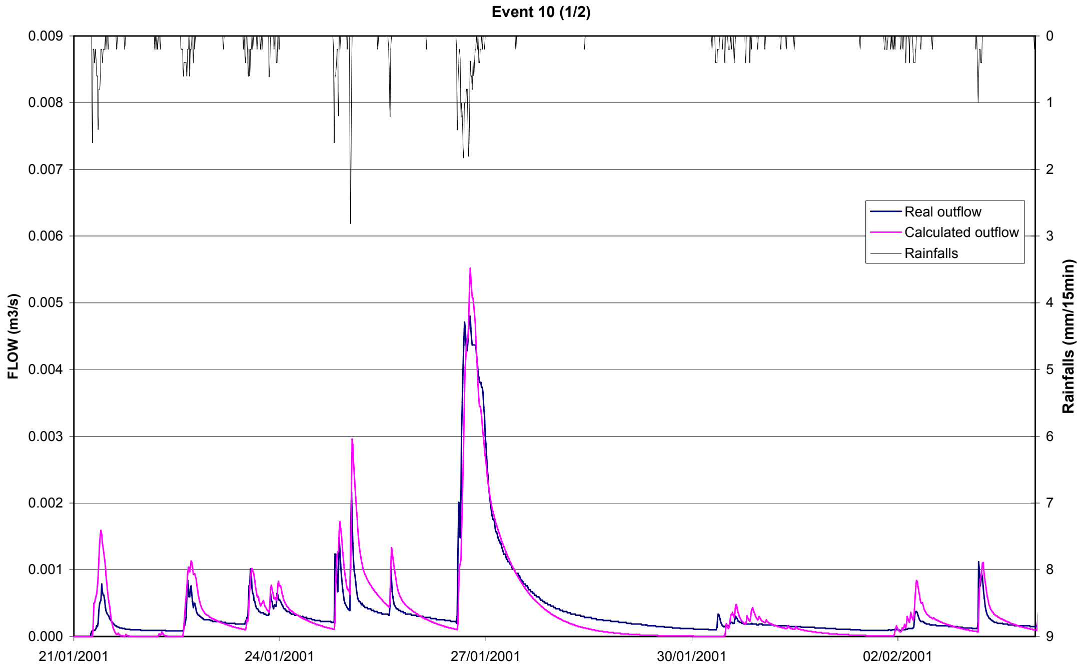

Figure 2.13

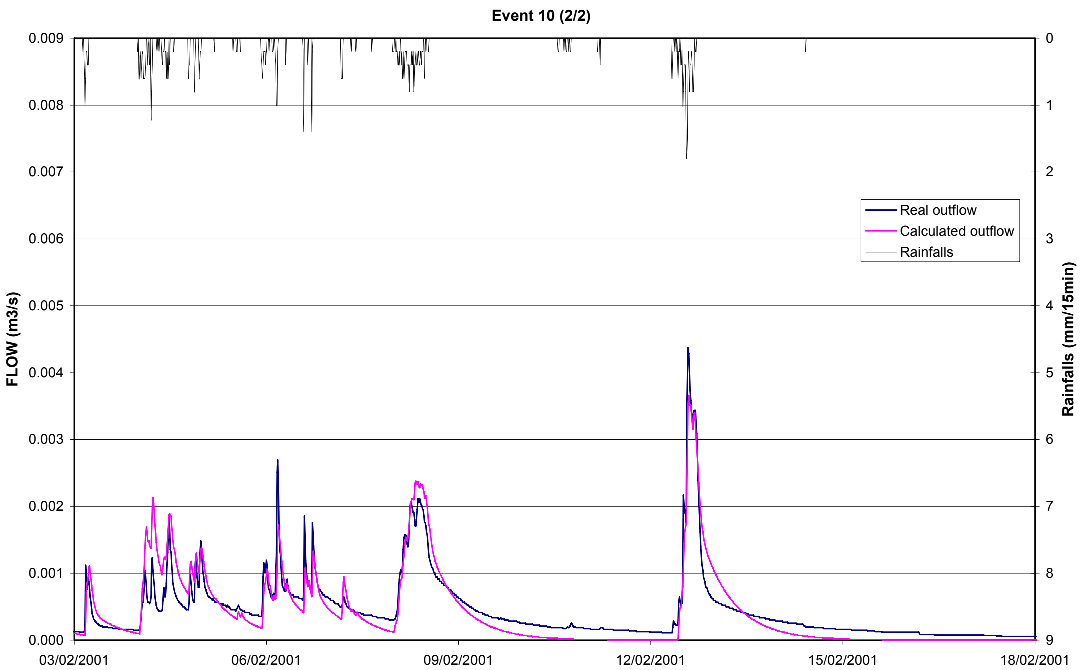

Figure 2.14