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Reservoir pavements for drainage attenuation

Client: **Highways Agency, Network Services - Technical Services Division (Environment)**

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Executive summary

Recent floods have served as a reminder of the dangers of climate change that is predicted to cause a 20 to 30% increase in rainfall. Unfortunately, sealed paved areas prevent the natural dissipation of rainwater. Consequently, increases in rainfall and infrastructure developments, will cause both the rate and volume of water runoff from built–up areas to increase unless mitigating actions are implemented. One solution to this problem is the wider use of Sustainable Drainage Systems (SuDS), which deal with water runoff at source or at places of its discharge by mimicking the natural processes of rainwater distribution to the air and the ground.

SuDS can be built into the road infrastructure in a variety of ways including the creation of an innovative reservoir road pavement. These pavement structures include porous materials, which delay and reduce the rate of water run-off to outfall drains. Also, when water is allowed to infiltrate into the soil, reservoir pavements reduce the total amount of water flowing to drains. A review of reservoir pavement technology summarised in this report concluded that these roads have historically been constructed for lightly trafficked applications. To permit their more extensive application on the trunk road network, the Highways Agency (HA) sponsored research to develop guidelines for their use on more heavily trafficked roads and to outline their potential application on the English strategic road network. The principle contributors to the project comprised research partners TRL (Lead Organisation) and Halcrow, aggregate producer and contractor Aggregate Industries as well as organisations ADAS and CEH. Guidance was also received from the Environment Agency.

To develop the technology and aid its implementation, Aggregate Industries built a trial incorporating their products at their Hulland Ward factory site. Four adjacent pavements were constructed with porous concrete as the main structural layer to cope with the design traffic. Several pavements were built with reservoirs enclosed by a tank, which protected the underlying soil from infiltrated water, and one pavement was constructed on a permeable soil that was considered capable of draining infiltrated water without significant weakening. Three surfacing options were provided. These variants included a porous asphalt surface, which reduces spray and simplified drainage design, and a traditional "impermeable" asphalt surface that is more robust than porous asphalt but requires the pavement to have an edge drain. Also, a permeable concrete block paving surface, which is more resistant to damage by fuel spillage than asphalt, was constructed to represent service areas where vehicles park.

This report describes the construction and assessment of the structural and hydraulic behaviours of the pilot-scale pavements.

The pavements were trafficked by heavy commercial vehicles and no evidence of deterioration by cracking or deformation was found from structural testing and visual condition surveys.

The hydraulic performance of each trial pavement under natural and simulated rainfall events has been investigated and the expected behaviours of full-scale reservoir pavements demonstrated. The effects of water infiltrating through a permeable pavement and draining into a sandy gravel subgrade were observed. The natural attenuations of peak rainfall intensities by the various pavement materials and structures were determined along with the associated delays to the peak water flows from the reservoirs. Greater control of water flows in outfall drains was obtained by the use of restrictor orifices that can be chosen to adjust flow rates to permitted values.

The pervious surfaces of block paving and porous asphalt became significantly clogged with detritus during the eighteen month study. The adoption of reservoir pavements with pervious surfaces for a specific site should be dependent on their likelihood of clogging. Equipment to clean pervious surfaces was assessed and one device was shown to produce a marked improvement in water infiltration capacity. Although cheaper to construct, reservoir pavements with pervious surfaces have a maintenance requirement

that will increase their whole life cost. An alternative approach is to adopt the version of reservoir pavement that collects water runoff from conventional, impermeable surfaces at edge drains and injects the runoff into the reservoir layer via sediment traps.

The findings of the research informed an Interim Advice Note for the Highways Agency that was produced within the project as a draft document for further consideration and development.

This trial is envisaged as the first stage of a programme of research to develop reservoir pavement technology and guide its application within the English strategic road network. A future research stage is intended to include the construction and monitoring of full-scale trials on the road network in, initially, low risk locations where subgrade conditions are unlikely to cause problems. As experience is developed, the application of reservoir pavements could be widened. In this way, the HA's guidance for the design, construction and maintenance of reservoir pavements could be confirmed and adapted.

Abstract

Infrastructure developments with hard paved areas prevent the natural dissipation of rainwater. Their adverse effects are cumulative and can lead to long-term problems of disposing of water that can result in flooding. The Highways Agency aims to maintain rainwater runoff rates from their roads at current levels, despite road widening and the increase in rainfall predicted by climate change. Sustainable Drainage Systems (SuDS), which deal with runoff at source by mimicking the natural processes of redistributing rainwater to the air and the ground, reduce the severity of these problems. One such system is a reservoir pavement that can either eliminate or reduce runoff, or just temporarily store water and reduce run-off flows. There are several configurations of these pavements to cope with different site specific issues. Reservoir pavements, however, have been used mainly for lightly trafficked applications. This report describes a pilot-scale trial of flexible pavements with porous concrete bases that have the potential to extend the technology to more demanding traffic levels. An assessment is made of the hydraulic and structural behaviours of a variety of reservoir pavement types that indicates the potential of these pavements. This research, sponsored by the Highways Agency, was undertaken by TRL with industry collaboration.

1 Introduction

Impermeable surfaces such as paved roads and parking areas prevent the natural dissipation of rainwater. Water from rainstorms quickly runs off these surfaces into drains and waterways. Infrastructure developments of a site with hard paved areas increase both rate of runoff and total volume of runoff. The adverse effects of this type of development are cumulative and can lead to significant long-term problems of the disposal of water and can result in flooding.

Planning directives require the implementation of Sustainable Drainage Systems (SuDS), which deal with runoff at the source by mimicking the natural processes of redistributing rainwater to the air and the ground (DCLG, 2006; Pratt et al, 2002). The Highways Agency (HA) is committed to cooperating with the strategy document issued by the Department for Environment, Food and Rural Affairs (DEFRA, 2004) entitled "Making Space for Water". This approach involves maintaining current discharge rates from roads, despite road widening and a predicted 20% to 30% increase in rainfall as a result of climate change, and updating advice to minimise flood risk.

Balancing ponds are often used to collect and attenuate the rate of run-off from these paved surfaces. Whilst balancing ponds are effective, they tie up expensive land. This problem is overcome by reservoir pavements, which is a special type of pavement that allows rain to pass through it, or be diverted into it by drains. Water is temporarily stored within the pavement and may be allowed to infiltrate into the soil. The reservoir pavement thereby reduces the rate and, for some pavement types, the amount of runoff from the site and surrounding areas. In certain circumstances, discharges from rainfall can be eliminated. Reservoir pavements can therefore be used as part of a storm water management system. In addition, porous materials within these pavements filter some pollutants from the runoff to improve water quality and those pavements with porous surfaces reduce traffic noise. The reservoir pavement is therefore an innovative pavement structure that has environmental benefits.

The original form of reservoir pavements was first proposed over forty years ago. With proper design and installation, reservoir pavements can provide cost effective solutions, some with proven life spans of 20 years or more. In recent years these pavements have been receiving more attention as a result of increased urbanisation and concerns about more intense periods of heavy rainfall. Traditionally these pavements have been used for lightly trafficked areas. However, the challenge is now to extend the use of these pavements to more heavily trafficked situations.

The HA therefore sponsored research to investigate the use of reservoir pavements for drainage attenuation with Mr Santi Santhalingam of the Environmental Team acting as Project Sponsor. The overall objective was to develop guidance on the use of reservoir pavements within the English strategic road network. This study involved a review of existing practices and the construction of short trial sections of reservoir pavements of different design configurations, including pavements with sealed reservoirs and a pavement that drained into the underlying ground. The hydraulic behaviour and structural performance of these test pavements under traffic were studied to assess their potential to attenuate run-off and to be included, where appropriate, within the English strategic road network.

This report briefly reviews the current state of the technology and then describes a pilot-scale trial of reservoir pavements. This trial was designed to extend the technology to more heavily trafficked pavements by demonstrating adequate structural design, material durability and hydraulic behaviour.

A successful demonstration of these pilot-scale pavements will be a precursor to carrying out full-scale trials on the HA's road network. To reduce risk, these trials will initially be built where the subgrade conditions are unlikely to be a source of problems. As experience is developed, the application of reservoir pavements could be widened.

2   Background

2.1   Review

In the late 1960s the original concept of reservoir pavements was proposed in the USA primarily to reduce storm water loading and risk of flooding and to replenish aquifers. In recent years these pavements have been receiving more attention as a result of increased urbanisation and concerns about more intense periods of heavy rainfall that are predicted to result from climate change. In the early 1980s the concept was introduced into France (Raimbault et al, 1982) and in 2003 the American National Asphalt Pavement Association (NAPA, 2003) produced a design guide. Pervious pavements are now widely used throughout the world and research has been carried out in many other countries including Australia, Brazil, Canada, Denmark, Japan, Singapore, Spain and Sweden.

Pervious pavements can be constructed with porous asphalt, no-fines concrete and permeable concrete block paving systems. The main function of these pavements is the temporary storage of water using porous materials. Pervious pavements can be located in roads, parking areas, etc. The main advantages of pervious pavements can be summarised as follows:

- Reduction or elimination of runoff from storm water.
- Reduction in flood risk
- Reduction in the need for kerbing, sewers and balancing ponds.
- Use as soakaways to deal with runoff from other structures.
- Removal of pollutants.
- Recharge of ground water.
- Improvement in safety resulting from a reduction in spray, glare and ice formation from surface water.

The majority of countries restrict the use of pervious pavements to lightly trafficked areas. The concept is most advanced in France where the French design guide deals with moderately heavily trafficked roads of up to six million 13 tonne equivalent standard axle loads. In the USA there are a few examples of pervious pavements with a successful history on State highways and there are moves currently underway to develop a structural approach to pervious concrete pavement design for heavily trafficked pavements.

Additional infrastructure developments with impermeable paved surfaces, such as road widening, and higher rainfall due to climate change, increase run-off that may overload the existing surface water drainage system when alternative drainage is not provided. Typical summer rain storms can be of high intensity for a short duration, whereas the intensity in winter is generally lower but for a longer duration. These storms create a large volume of runoff requiring costly drainage systems, which can be in the region of 10 % of the total road development cost. Recent experience in the UK has shown that in exceptional wet periods, this runoff can contribute to severe flooding in low lying urban areas. The reservoir pavement offers a cost effective alternative to conventional construction with many advantages.

2.2   The concept and pavement options

When rainwater lands on a dry impermeable surface, water first wets the surface with some water being absorbed. Thereafter, water may collect as puddles in depressions. After a short time, water will then begin to flow over the surface towards the drainage outlets. Depending on the intensity of rainfall and the gradient of the surface, rainwater

may take from 2 to 15 minutes to reach the outlets from all parts of an impermeable pavement. The amount of rain required to land on an impermeable surface before runoff begins is typically equivalent to a water depth of less than 1 mm.

The basic concept of a pervious pavement is shown schematically in Figure 2.1 for a lightly trafficked pavement. The rain that falls on the surface infiltrates across the entire surface into a porous granular subbase material (reservoir layer). Here, water can accumulate before it is dissipated more slowly into the soil subgrade or it is removed through a system of ancillary drains in the subbase into the main surface water, drainage system. This process reduces the rate of runoff (or outflow) that thereby mitigates the adverse effects of storm water surges and the risk of flooding as a result of the drainage system becoming overloaded.
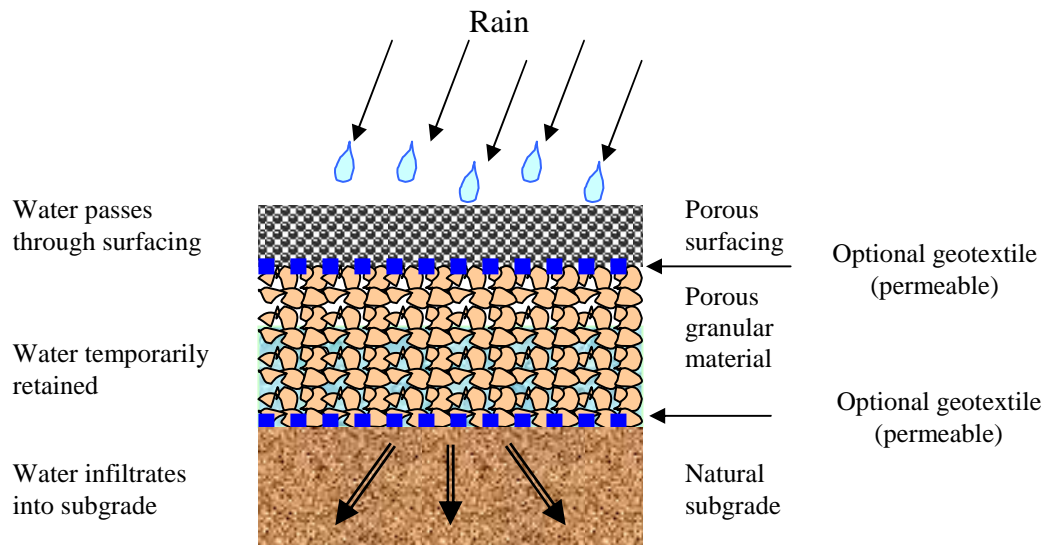

Figure 2.1 Concept of a pervious pavement

There are several possible design configurations for pervious pavements based on the main features illustrated in Figure 2.1. The CIRIA SUDS manual, Report C697 (2007) identifies three basic configurations, each with a pervious surface. These basic configurations may also be reproduced using conventional surfacing (asphalt or concrete) with edge drains collecting runoff for injection into the underlying reservoir layers. The six basic design configurations form variants of a group of pavements classified as reservoir pavements in this report. This description was chosen because the function of temporary storage of water within the pavement is possible for all six types.

The design configurations featuring pervious surfaces are shown schematically in Figure 2.2 and are as follows:

   I.   Reservoir Pavement Type I (equivalent to CIRIA Type A) in which rainfall passes through a permeable or porous surface into a porous subbase that provides the reservoir layer. The stored water is then discharged by infiltration into the subgrade.

  II.   Reservoir Pavement Type II (equivalent to CIRIA Type B) in which the flow path is as above, but where the ground (subgrade) is insufficiently permeable to allow dissipation of all design storm events. A network of perforated pipes is placed at the base of the reservoir layer to convey discharge to a receiving drainage system. In low permeability soils, this design prevents water levels in the reservoir layer rising and causing potential stability problems in the overlying structure.

 III.   Reservoir Pavement Type III (equivalent to CIRIA Type C) in which there is an impermeable flexible membrane placed at the base and around the sides of the reservoir layer. Water that has percolated into, and through, this reservoir layer is

discharged to a receiving drainage system by perforated pipes (or similar) at the base providing attenuation for both flow and pollution. The discharge can continue for hours and even days after the rainfall stops. Type III designs are intended for situations where:

- The underlying groundwater is sensitive and requires protection.

- Underlying moisture susceptible soils have low strength and could be weakened, or otherwise damaged, by the introduction of percolating water and building a thicker pavement would be impractical.

- The water table is within 1m of the subbase/reservoir layer.

- The site is contaminated and the risk of mobilising contaminants needs to be minimised.

Such Type III designs, possibly with the impermeable flexible membrane replaced by a geotextile, could also be used where the subgrade is insufficiently permeable to allow any infiltration, but where the designer wishes to take advantage of the benefits of reservoir pavements; that are, attenuation of flows and pollutants, minimal land take, etc.

Pavements of Type I, II and III that allow water to infiltrate through their surface have the simplest and cheapest design but their surfaces are subject to blockage by detritus and require regular interventions to maintain an adequate infiltration rate. This maintenance liability increases the whole life costs of the pavement. Also, to avoid clogging of pores, standard de-icing techniques using rock salt and grit cannot be used.

With both the subgrade infiltration and subbase under-drained options, it is also possible to use conventional impermeable surfacing materials with runoff injected into the reservoir layer from edge drains. These designs have advantages in that highly durable and low maintenance surfacings can be used in situations, for example, where there is high traffic on strategically important routes or where high shear forces are generated in the surfacing by manoeuvring vehicles. Also, where water contamination by oil is a risk, the oil can be separated from the water in a decantation chamber attached to the edge drains before the water is injected into the subbase. Design configurations are shown schematically in Figures 2.3 and are as follows:

IV. Reservoir Pavement Type IV in which road runoff enters the edge drain and is injected into the underlying reservoir layer (usually via pipes set at appropriate intervals) and then, as in Type I above, is discharged by infiltration into the ground beneath.

V. Reservoir Pavement Type V in which road runoff enters the edge drain, is injected into the underlying reservoir layer and as a result of low subgrade permeability, as in Type II above, requires a network of perforated pipes at its base to convey discharge to a receiving drainage system.

VI. Reservoir Pavement Type VI - equivalent to Type III above but with injection via edge drains as with Types IV and V.

The choice of system adopted will therefore be largely influenced by the nature of a specific site, its vulnerability to clogging by detritus and the clients view on maintenance and risk of inadequate hydraulic performance.

In many situations, a reservoir pavement is designed to simply capture runoff falling directly on the pavement surface. In other cases, the reservoir pavement may be designed to capture, in addition, runoff from other sources such as that from adjacent impermeable surfaces. These two applications may be termed "direct" and "extended" runoff mitigation, respectively.

It is necessary to design the structure, not only to temporarily store storm water, but also to carry the traffic. Hydraulic and structural design aspects are both considered in Section 3.
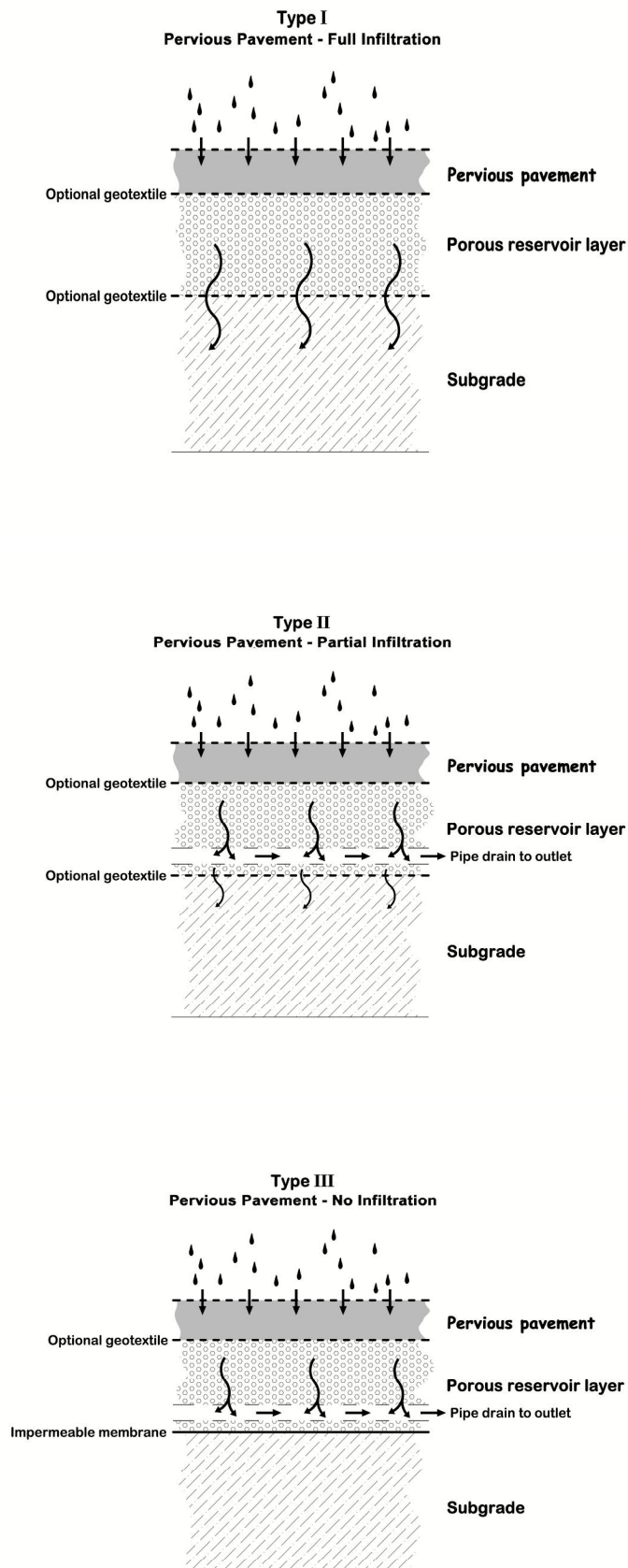
Figure 2.2   Reservoir Pavement Types I to III

Type IV
Impermeable Pavement, Injection via Edge Drains - Full Infiltration

Conventional impermeable pavement

Optional geotextile
Discharge to reservoir layer

Porous reservoir layer

Optional geotextile

Subgrade

Type V
Impermeable Pavement, Injection via Edge Drains - Partial Infiltration

Conventional impermeable pavement

Optional geotextile
Discharge to reservoir layer

Porous reservoir layer

Pipe drain to outlet

Optional geotextile

Subgrade

Type VI
Impermeable Pavement, Injection via Edge Drains - No Infiltration

Conventional impermeable pavement

Optional geotextile
Discharge to reservoir layer

Porous reservoir layer

Pipe drain to outlet

Impermeable membrane

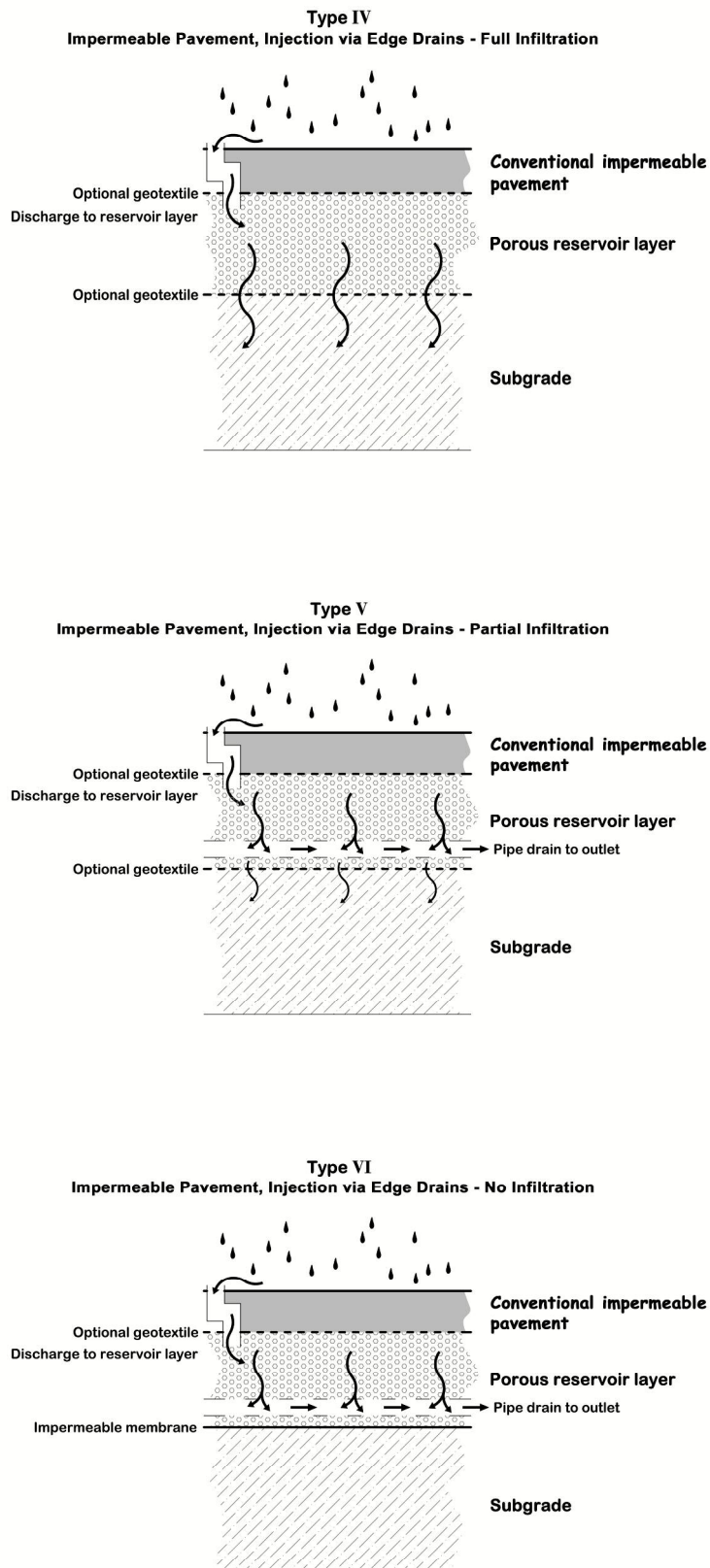Subgrade

Figure 2.3   Reservoir Pavement Types IV to VI

3   Design of reservoir pavements

3.1   Structural design

Worldwide developments of reservoir pavements were reviewed with the following conclusions.

In France, fully porous flexible pavements are used on moderately trafficked roads that are designed to carry cumulative traffic of just over 2 msa_{130}. Also, porous concrete pavements can be designed for higher cumulative traffic loads up to 6 msa_{130}. France uses a 130 kN axle load as the reference for pavement design purposes, whereas in the UK an 80 kN axle load is used. The fourth power damage equation is generally used to relate the damaging power of different wheel loads, and if this equation is applied, in terms of an 80 kN equivalent standard axle load, these designs could be expected to carry approximately 14 and 40 msa_{80}, respectively.

In the United States, the use of porous pavements for heavily trafficked pavements is recognised as presenting a challenge. It is considered that porous concrete pavements may have more potential than porous asphalt pavements in heavier trafficked situations. The Federal Highways Administration (FHWA) has therefore produced a research statement for the development of porous concrete pavements. Delatte (2007) believed in 2007 that expansion of porous pavements into these heavy duty applications was hampered by the fact that then there was no rational method for structural design of porous concrete pavements. He proposed a method that could provide the basis for the design of these pavements for major arterial roads. Composite pavement construction incorporating both asphalt and cemented materials is rarely used in the USA and as a result this form of construction is underdeveloped in that country.

In the UK, there are established design methodologies for flexible pavements with either an asphalt base or a hydraulic bound base. These methodologies can be used to design porous pavements for heavily trafficked roads.

Porous paving materials, however, have lower strength and stiffness compared with similar materials that are densely graded. Also, an unprotected subgrade of a porous pavement may be weakened by water. Consequently a porous pavement often requires a thicker construction to compensate for the reduced structural properties of these layers.

In the following sections, the implications of incorporating porous materials in UK pavement design methodology are considered separately for the two pavement types. As there are many uncertainties, trials at pilot-scale and full-scale are required to demonstrate the potential of reservoir pavements incorporating porous materials and to help develop the technology.

3.1.1   Porous flexible pavement with asphalt base

Flexible pavements with asphalt bases, previously known as fully flexible pavements, can be designed as reservoir pavements by the method of Nunn (2004). As HD26 (HA et al, 2006) is based on Nunn (2004), then the derived reservoir pavement designs would also be consistent with the designs of other Highways Agency (HA) pavements. For consistency, material terminology and properties used by Nunn (2004) have been adopted in this report despite recent changes in the specification of asphalt materials.

Porous asphalt is considered in the UK to have a stiffness modulus of about 2 GPa compared with the value of 6.2 GPa for heavy duty macadam that was used by Nunn (2004). Structural design theory indicates that the thickness of the asphalt

layer would need to be about 40 % thicker to accommodate this reduction in asphalt stiffness and a potentially lower support to the pavement by the foundation caused by infiltrated water. For example, using the method of equivalent thicknesses (Odemark, 1949), a road designed to carry 30 msa requires a layer of porous asphalt incorporating 50 pen bitumen to be over 400 mm thick if it is laid on a Class 1 foundation. By comparison, a layer of dense graded, heavy duty macadam (HDM) would be only 280 mm thick when laid on a stiffer foundation of a conventional pavement that would be expected to be classified as a Class 2 foundation. This analysis indicates that the construction costs of a porous asphalt pavement could be much more expensive than a conventional pavement, although there could be savings elsewhere; for example, in the provision of drainage.

3.1.2 Porous flexible pavement with a cement bound base

Flexible pavements with a cement bound material base, previously known as a flexible composite pavement, can also be designed as reservoir pavements by Nunn (2004). As before, the material terminology and values used by this author are adopted in the following analysis despite changes in the specification of asphalt and cement bound materials. Traditional densely graded, cement bound material could be replaced by thicker porous versions of these materials. Porous concrete, however, can be designed to have structural properties equivalent to those of cement bound materials used in traditional flexible composite pavements. Consequently, the porous concrete base of a reservoir pavement can be built to the same thickness as the base of a traditional pavement built with dense cement bound material. It was therefore suggested that the combination of the smooth running surface of asphalt and the good load spreading ability of concrete may be the best form of porous pavement construction for heavier traffic applications.

Cement bound base

The National Ready Mix Concrete Association[1] of the USA suggests that porous concrete has a 28 day flexural strength that is in the range of 1.0 to 3.8 MPa, whereas pavement quality concrete has a 28 day flexural strength in the range 5 to 7 MPa. Although porous concrete is weaker than traditional pavement quality concrete, its strength can be seen in Table 3.1 to be comparable to the strengths of cement bound material (CBM) grades historically used in the UK. In this table, the properties of grades of cement bound materials (CBMs) with gravel aggregate (G) are given. Properties of porous and conventional asphalts are also recorded in Table 3.1.

Footnote: [1] http://www.perviouspavement.org/engineering%20properties.htm

Table 3.1 Mechanical properties of porous and dense materials

Material [1]	Compressive Strength[2] (MPa)	Flexural Strength[2] (MPa)	Dynamic Modulus (GPa)
CBM3G	12.5	1.38	30.3
CBM4G	18.75	2.06	36.1
CBM5G	25	2.75	40.3
Porous concrete (range)	3.5 to 28[3]	1.0 – 3.8[3]	25 to 45[4]
Porous concrete (typical)	17.0[3]	2.5[3]	38[4]
Dense bitumen macadam (DBM)	N/A	N/A	3.1
Heavy duty macadam (HDM)	N/A	N/A	6.2
Thin surface course	N/A	N/A	2.0
Porous asphalt	N/A	N/A	2.0

Key:　[1]　Pre HD26/06 terminology
　　　　[2]　28 day values
　　　　[3]　American National Ready Mix Concrete Association.
　　　　[4]　Determined using equation 3.1.
　　　　All other values were standard values used in UK pavement design by Nunn (2004)

As references for typical values of the dynamic modulus of porous concrete could not be found in the literature, it was assumed that the dynamic modulus of this material could be calculated from its flexural strength. The relationships between elastic stiffness and flexural strength for cemented granular materials containing various aggregates are shown graphically by Croney (1977). Nunn (2004) represented these relationships by equations of the form:

$$E = \frac{\text{Log}(f_f) + a}{b}$$
　　　　3.1

, where E is the dynamic modulus (GPa), f_f is the flexural strength (MPa) and 'a' and 'b' are constants that are recorded in Table 3.2.

Table 3.2:　Constants for use in equation 3.1

Aggregate	Values of constants:	
	a	b
Gravel	0.773	0.0301
Crushed rock	0.636	0.0295

For porous concrete, the relationship between dynamic modulus and flexural strength may differ from that for cemented granular materials. Notwithstanding this uncertainty, the values of dynamic stiffness estimated for porous concrete and given in Table 3.1 were, for consistency, based on the constants given in Table 3.2 that were derived for gravel aggregates.

It is concluded from the values given in Table 3.1 that porous concrete can easily achieve the properties of CBM3G and, moreover, may achieve comparability with the higher strength CBM grades.

The cement bound base layer of a traditional flexible composite pavement develops a regular pattern of transverse cracks soon after laying as a result of shrinkage during curing and thermal contraction during the colder nights. These cracks can appear at the surface of the asphalt a number of years later as reflection cracks. The function of the depth of asphalt surfacing in a flexible pavement with a cement bound base is primarily to delay the onset of this reflection cracking.

There are many uncertainties about the use of porous materials in flexible pavements with cement bound bases, but there may be a number of factors favouring this type of construction. These factors are:

- The current designs of flexible pavements with cement bound bases would require little modification to accommodate porous concrete.

- There is less shrinkage with porous concrete. Delatte (2007) points out that, because of this behaviour, it is not necessary to construct porous concrete pavements with construction joints. Consequently transverse cracks may not develop so readily in flexible pavements with porous concrete bases.

- There is anecdotal evidence that porous asphalt surfacing delays the onset of reflection cracking.

The current design methodology for flexible pavements with hydraulic bound bases in the UK was developed by Nunn (2004). This analytical design method uses the structural properties of hydraulically bound mixtures at 360 days rather than 28 days, which was adopted in the previous design approach. For conventional CBMs, the 360 day compressive strength of CBM is about 25 % higher than the 28 day strength. For faster curing porous concrete, this relationship may not apply and a conservative stance would be to assume that no further curing occurs after 28 days.

The combination of stiffness and strength is crucial for design of a hydraulic bound base. Two different hydraulically bound mixtures can have the same base thicknesses for a given level of traffic, provided their flexural strengths compensate for any differences in their levels of stiffness. If stiffness is increased, the traffic induced tensile stress in the base that influences performance will also increase; therefore the strength of the base would need to be higher to achieve the same performance. With any hydraulic bound base, it is therefore desirable to have a high flexural strength and a relatively low modulus. Relationships between flexural strength and dynamic elastic modulus have been developed for equivalent performance and grouped into nine zones of hydraulic bound base (H1 to H9). These zones are shown in Figure 3.1.

The values for the flexural strength and dynamic stiffness of porous concrete given in Table 3.1 vary over wide ranges and suggest that porous concrete could be characterised as a Zone H4 material, for the weakest material, up to Zone H8 material, for the strongest material. The mean values suggest, possibly, a Zone H7 material for porous concrete used in the USA. But, with no measurements available for the dynamic modulus of porous concrete, it is suggested that porous concrete should initially be classified as a Zone H5 material for design purposes.
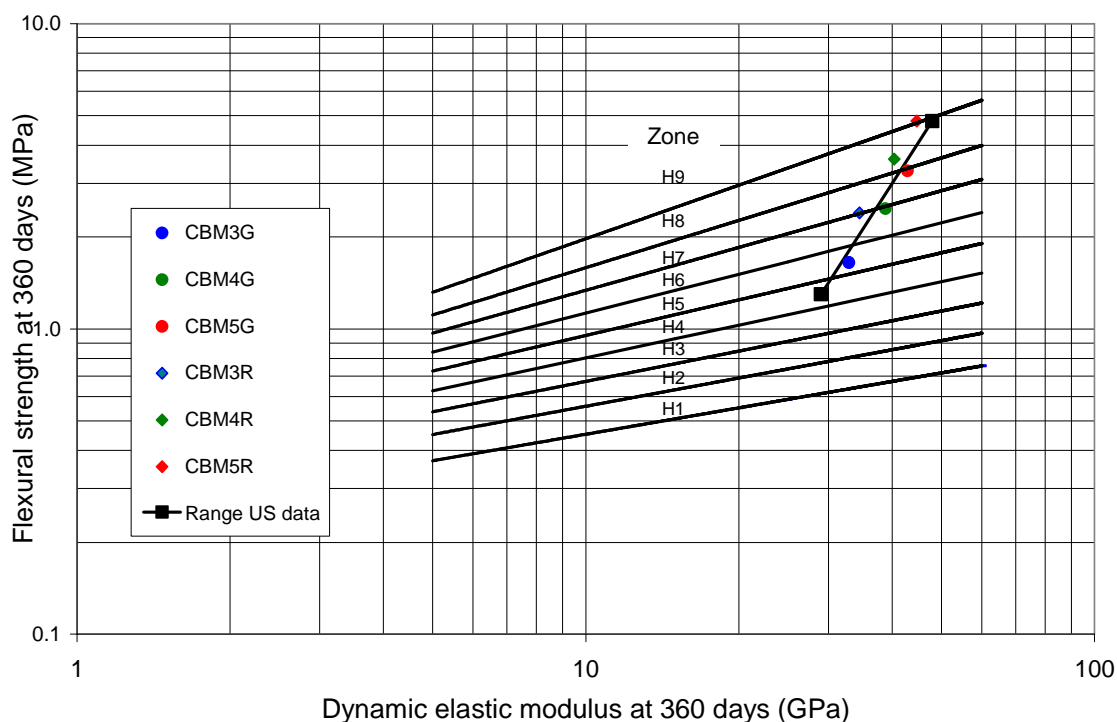
Figure 3.1 Relationship between strength and stiffness

Foundation

In the design method developed by Nunn (2004), the road foundation is categorised in terms of foundation stiffness classes. These classes are defined in terms of the long-term, equivalent half-space stiffness of the composite foundation. The four divisions are as follows:

- Class 1 ≥   50 MPa
- Class 2 ≥   100 MPa
- Class 3 ≥   200 MPa
- Class 4 ≥   400 MPa

The standard UK foundation (equivalent to 225 mm of Type 1, unbound granular subbase on a subgrade of CBR 5%) corresponds to Class 2. The Class 1 foundation applies to pavement construction on a capping layer and Class 3 and 4 foundations involve bound subbases. IAN 73 (HA, 2009) and Chaddock and Roberts (2006) describe the design of these foundation classes in more detail.

In designing foundations to a particular class, care must be taken in the selection of the design Californian Bearing Ratio (CBR) for the soil subgrade. The value used in design is the lower of the construction CBR and the long-term CBR. Whereas the conventional approach to pavement design is to protect the subgrade from ingress of water to the maximum extent possible, a drainage pavement can deliberately permit water to come directly into contact with the subgrade. The only exceptions are those situations when the subgrades are protected by impermeable membranes. The long-term CBR of a particular soil subgrade during the reservoir pavement's service life is therefore dependent on whether, or not, the soil is regularly wetted by the infiltration of water from the subbase reservoir. The mechanical properties of the subgrade required for design of reservoir pavements Types I, II, IV and V are those relating to a wetted condition. Experience from evaluation of conventional pavements shows that, in

cases where water has gained access to the subgrade, CBRs are typically much lower than elsewhere, although highly permeable, non-moisture susceptible subgrades are encountered. Because no absolute rule exists as to the degree of reduction in subgrade CBR by infiltrated water, soaked CBR tests should be carried out on soil representative of the *in situ* compacted condition of the subgrade to provide guidance. Further guidance on determination of the design CBR is given in IAN 73 (HA, 2009) and HA44 (HA et al, 1991) with modifications, when necessary, as a result of the wetting of the subgrade by infiltrated water.

The subbase reservoir for infiltrated water is constructed on the soil subgrade. The subbase is normally a structurally significant layer that also provides a working platform on which materials can be transported, laid and compacted. In a conventional pavement, unbound granular Type 1 subbase of MCHW 1 (HA et al, 2004) is often used. This material is well graded (poorly sorted) and contains an assortment of particles covering a wide range of grain sizes that minimise the volume of pore spaces and obstruct their interconnectivity when the subbase is well compacted. In a reservoir pavement, however, the subbase is used to temporarily store heavy rainfall. This function requires the granular subbase to have high voids content so that the quantity of water anticipated from a heavy storm can travel freely through this layer and also be stored without saturating the subbase. This behaviour can be achieved, for example, by removing the fine fraction from granular subbase to produce a material with air voids content in the 30 to 40% range and by building the subbase sufficiently thick. The subbase should be well compacted so that the high voids content is a result of the design grading and not of under-compaction.

To ensure the stability of a reservoir layer of a well sorted (poorly graded), unbound granular subbase, the French guide by CERTU (1998) specifies that the ratio of the maximum to minimum stone size should be greater than 3. When large sized granular materials are used in the granular subbase, the surface may not be stable or smooth enough to permit trouble-free construction of the bound base layer. In these instances, a blinding layer of smaller sized aggregates can be compacted into the subbase surface. The thickness of the main reservoir layer can be adjusted to accommodate this blinding layer.

Given the subgrade design CBR, the thickness of subbase for the required foundation class can then be determined using IAN 73 (2009). For example, a foundation that consists of about 400 mm of unbound granular material on a subgrade with a CBR of 2.5 % could produce a Class 1 or Class 2 foundation dependent on the quality of the granular material (Chaddock and Roberts, 2006). It is recommended that a minimum thickness of 350 mm be used to ensure that there is sufficient reserve water storage capacity. Should the thickness resulting from the structural design exceed the storage thickness requirement, then the structural design thickness should be used in preference.

Pavement design

The variations in design thicknesses of porous concrete base and asphalt surfacing with traffic are given in Figures 3.2 and 3.3 for foundations of Class 1 and 2 respectively. In Figure 3.2, for a Class 1 foundation, the thickness designs of a pervious composite pavement constructed with a porous asphalt surfacing on a porous concrete base are compared with those of an impermeable composite pavement built with a dense graded asphalt surfacing on a porous concrete. A similar comparison of designs is given in Figure 3.3 for the higher Class 2 foundation. In each case, the surfacings of porous asphalt and dense graded asphalt are of equal thickness for any selected design traffic. Therefore, because the same zone 5 porous concrete material (Structurally equivalent to CBM3G) is

used, the difference in thicknesses of porous concrete is to compensate for porous asphalt having a lower stiffness than that of dense graded asphalt.
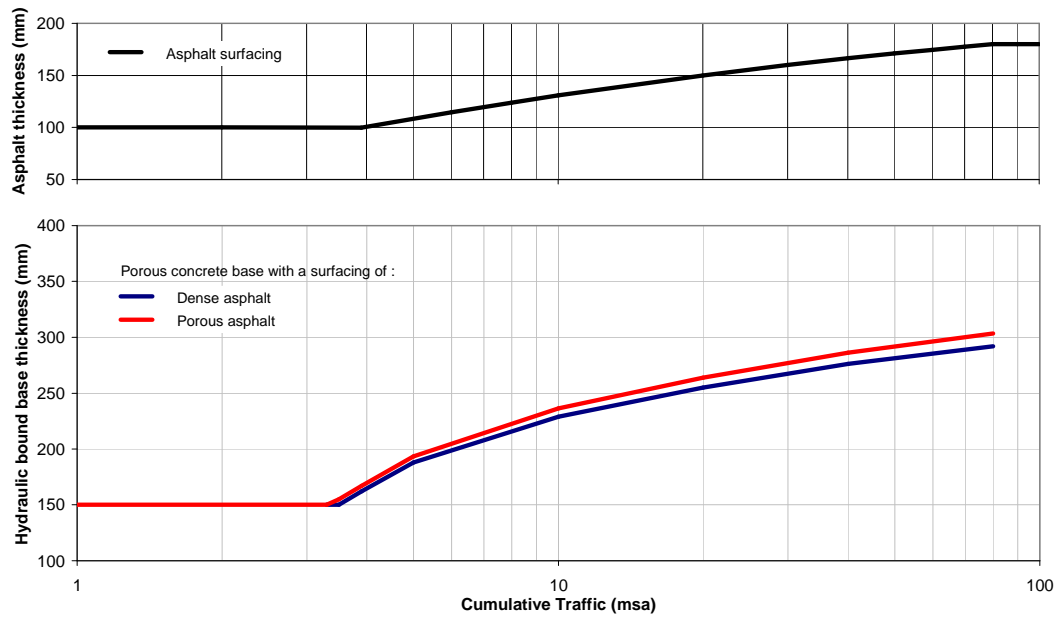

Figure 3.2 Thicknesses[1] of flexible pavements with a Zone 5, hydraulic bound base of porous concrete on a Class 1 foundation
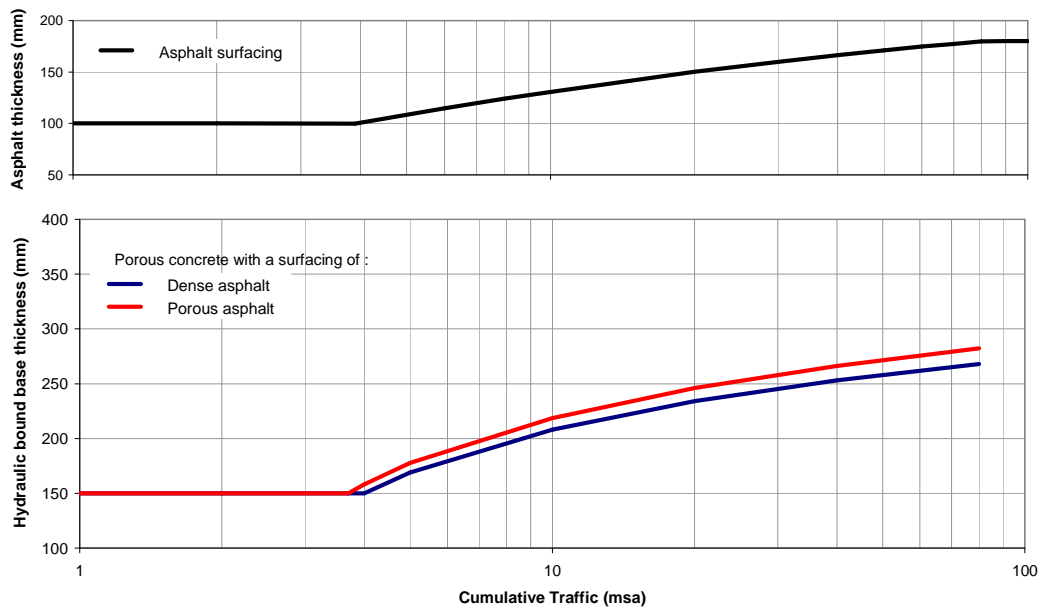

Figure 3.3 Thicknesses[1] of flexible pavements with a Zone 5, hydraulic bound base of porous concrete on a Class 2 foundation

Footnote: [1] These designs assume that the porous concrete material is structurally equivalent to CBM3G and were derived following the procedure of Section 6 Nunn (2004) for Non-Standard material, where the porous concrete was conservatively assigned to the lower boundary of Zone 5. The resulting designs are thicker than those given in Table 8 Nunn (2004) under CBM3G in which the porous concrete was effectively assigned to about the middle of Zone 5 (not the lower boundary) as it adopted the Standard material properties of the equivalent material CBM3G.

From a knowledge of the cumulative traffic that it is envisaged the road will carry throughout its life and the support the foundation is expected to offer the pavement as expressed through foundation class, the thicknesses of the Zone H5 porous concrete base when surfaced with porous and dense asphalt can be read from the design charts of Figures 3.2 and 3.3. The common thickness of asphalt can also be established. These figures show that the thickness of the porous concrete base needs to be increased by between 5 and 10 mm, depending on the design life, when porous asphalt instead of dense graded asphalt is used in the surfacing.

Potential developments

It is possible that more economic, porous composite design configurations could be developed along the following lines:

- Higher foundation classes could be constructed to better support the pavement by using a lower grade porous concrete subbase as reservoir and/or stabilising the subgrade. This approach would make the foundation less moisture susceptible but, unfortunately, it may reduce the subgrade's permeability and thereby its ability to drain infiltrated water. A balance may need to be sought between the hydraulic and structural requirements of the foundation.

- Structural classes of porous concrete could be developed as suggested by Delatte (2007). This approach may require the reduction of their voids content, but, as noted by Delatte, the permeability of the current grade of porous concrete is higher than required.

As for Figures 3.2 and 3.3, the design charts of Nunn et al (2004) for foundations of higher Classes 3 and 4 and porous cemented bases superior to a Zone H5 category could be adapted, if required, for porous asphalt surfacing.

3.2 Hydraulic design

The same approach to hydraulic design can be applied to any type of reservoir pavement. The key design processes for reservoir pavements are as follows:

- Selection of design rainfall.

- Confirmation of adequate rate of infiltration through the pervious surface (Types I-III only).

- Determination of the storage capacity required to manage the design storm.

- Determination of the outlet capacity and approach, either by infiltration into the soil, or by provision of sub-surface drainage pipes or by a combination of these methods.

- Management of extreme events in excess of the design storm.

3.2.1 Rainfall

Details of rainfall can be obtained from the Meteorological (MET) Office for any specified period. Generally, western regions in the UK are wetter than eastern regions[2].

Footnote: [2] http://www.british-towns.net/weather/annual_precipitation.asp

The key parameter for design is the maximum rainfall total over a defined return period. It is recommended that a 1:10 year design storm, as suggested as a minimum requirement by CIRIA (2002, 2007), be adopted. A practical design approach for the UK could be to devise a regional classification system along the lines of very wet, wet, average and dry.

3.2.2   Inflow to reservoir

For pervious pavements, the rate of infiltration through both the pavement and the underlying foundation layers should be checked to ensure that they can accommodate the design storm rainfall. For design, it is normal practice to allow for a 90% loss in permeability of the pervious surfacing due to clogging (CIRIA, 2002). An infiltration rate in excess of a high 1 in 10 year storm intensity is required to avoid runoff.

Where test data suggest that spare drainage capacity is available, reservoir pavements may be used to drain a larger area than that of the immediately overlying pavement by, for example, taking drainage water from adjacent roads with impermeable surfaces. These pavements are referred to as "extended" reservoir pavements as opposed to "direct" systems, which drain only the overlying pavement. Interpave (2010) suggests that the ratio of impermeable to permeable surface should be no greater than 2:1. Extended systems may utilise the pervious surface of the reservoir pavement to drain run-off, although the additional sediment generated by this extended surface should be taken into account. It is important that this additional area does not include surfaces such as embankments or verges that could generate large amounts of sediment. Extended systems may be more suitable for injected reservoir pavement types (Types IV-VI), where separate silt/sediment control is available.

Spare storage capacity within the reservoir pavement may also be used as an attenuating facility within a larger main surface water drainage system by diverting runoff from elsewhere into the reservoir layer, which then acts, for example, as either an in-line balancing facility or as a supplementary soakaway. This configuration can work well with very porous, non-moisture susceptible subgrades. Alternatively, a sub-surface reservoir may be used in verges or in other off-line locations to provide flow and pollutant attenuation. Hydraulic design would in this case be determined by the inflow from piped systems and outflow either by infiltration into the soil or from discharge pipes. In either approach, appropriate measures to control and contain the movement of sediment to the reservoir will be required.

3.2.3   Sub-surface storage capacity

It is necessary to store rainwater temporarily in a reservoir to balance the rate of inflow of water into the subsurface reservoir with the lower rate of discharge out of the system. The discharge rate may be constrained by either the rate water infiltrates into the subgrade (as identified by site specific infiltration tests) or by limitations placed on the discharge rate into a receiving system, whether this flow is to an existing drainage system or a natural water course.

To determine the storage volume required for the temporary retention of water generated by the design storm and thus the thickness of the storage layer, which depends on its void ratio, the following steady state mass balance calculation is made of inflow and outflow from the pavement:

Reservoir storage volume = volume of rainfall during design storm – volume of outflow from reservoir during storm

For a reservoir pavement system that allows infiltration out of the bottom of the reservoir, the method of determining the required storage volume for plane infiltration systems given in CIRIA (1996) may be used. For systems with a piped discharge, however, the guidance in CIRIA (2002) is that there is insufficient information available to accurately model the internal flow and storage properties within the subbase. Therefore, it is reasonable to make no allowance for any outflow during the storm when calculating the storage volumes with piped outflows. This approach will lead to an overestimate of the required storage capacity. However, experience suggests that discharge pipe systems can easily accommodate the required flow rate and, in practice, need to be throttled to meet outflow discharge restrictions. Designers may therefore allow for outfall discharges in the calculation of required storage capacity at rates up to the agreed discharge rate.

The method of calculating volume given in CIRIA (2002) is as follows:

The required input parameters for infiltration systems are:

q = Infiltration coefficient of the subgrade from percolation tests (m/h) - determined following the procedure given in CIRIA (1996).

A_d = Total area to be drained including any adjacent impermeable areas (m^2).

n = Porosity of subbase material.

i = Rainfall intensity (m/h).

D = Rainfall duration (h)

A_b = Base area of infiltration system beneath the pervious pavement (m^2).

For internal storage, the maximum depth of water (h_{max}) that will occur in the subbase is given by the following equation:

$$h_{max} = (Ri - q)D/n$$

,where, R = ratio of the drained area to base area of pervious surface, A_d/A_b.

The calculation is carried out for a range of storm durations (15min, 30min, 60min etc.) and related rainfall intensities for the 1 in 10 year design storm return period and the maximum value of h_{max} determined. This value is then the required minimum thickness of the subbase for water storage. Subbase depths for structural requirements and for frost resistance will also need to be taken into account.

For piped outflows, the calculation can be simplified to:

$$h_{max} = RiD/n$$

The calculation is carried out for a range of storm durations and related rainfall intensities for the 1 in 10 year design storm return period.

In order to accept subsequent storms, it is recommended that the design should ensure that water held in storage under the design storm empties from full capacity to 50% or less within 24 hours, but without exceeding discharge limits.

Notwithstanding the need for overflow facilities when the design storm is exceeded (see Section 3.2.5), a safety factor should be introduced in cases where subgrade infiltration forms the sole discharge; that is, there is no supplementary drainage. This factor should be in the form of a reduced subgrade infiltration rate. A value of safety factor of 10 is recommended; that is, the measured subgrade infiltration rate used in the above calculations should be divided by 10, until greater experience and feedback is gained with the design of these systems.

Single sized unbound aggregates (larger than 2.5 mm) with high voids content will hold water internally. The storage volume should be increased by 30 % to

allow any ice formed in cold conditions to expand into the free space without disturbing the structure. The CERTU (1999) design guide recommends that a minimum thickness of 350 mm be used to ensure that there is sufficient reserve capacity, even for more arid areas. Should the thickness resulting from the structural design exceed the storage requirement, then the structural design thickness should be used in preference.

3.2.4   Outflow from reservoir

Infiltration of water into the subgrade depends on the properties of the soil. It can be deduced from Interpave (2010) that soil permeability of the order of 10^{-6} m/s is required for the subgrade to completely drain all water falling on the pavement. If the soil permeability is lower than this value, then the water discharge component from infiltration into the subgrade is likely to be insufficient and underdrained and piped systems will be required.

Infiltration into the subgrade is important for both direct and extended systems. Estimating the infiltration rate for design purposes is imprecise, and the actual process of soil infiltration is complex. A simple model is generally acceptable for these applications, and initial estimates for preliminary designs can be made with satisfactory accuracy using conservative estimates for infiltration rates. Where ground/subgrade conditions are particularly variable, there may be no option other than to undertake *in situ* infiltration tests using the procedure of CIRIA (1996) to provide adequate guidance for the preliminary designs. Once the final design process is underway, *in situ* measurements of infiltration rate should be undertaken at the proposed location.

Guidance on the selection of an appropriate infiltration rate to use in preliminary designs can be found in the literature. For example, Table 3.3 gives ranges of values for the permeability of various soils that have been derived from the Code of Practice for Foundations produced by the British Standards Institution (1986).

Table 3.3:   Permeability of soils

Soil type	Approximate range of permeability values, k, (m/s)
Clean gravels	$1 \times 10^{+1}$ to 5×10^{-2}
Clean sands and sand–gravel mixtures	5×10^{-2} to 5×10^{-5}
Very fine sands, silts and clay-silt laminate	5×10^{-5} to 5×10^{-7}
Unfissured clays and well mixed clay silts containing more than 20% clay	5×10^{-7} to 1×10^{-10}
Desiccated and fissured clays	5×10^{-2} to 5×10^{-7}

Subgrades with the recommended minimum value of permeability of about 1×10^{-6} m/s are primarily comprised of sands and/or gravels. Other subgrades comprised of a mixture of soil types may be suitable to drain water if they contain interconnected drainage paths.

Where the reservoir is discharged via pipes to an existing piped drainage system or piped to a natural water course, there may be constraints on the rate of discharge to avoid increasing flows in the drain and the potential for flooding. The actual outflow rate should be controlled as follows:

- To less than, or equal to, the original runoff rate for newly developed areas.
- To less than the capacity of the downstream network.

If the maximum discharge rate from the reservoir is higher than that required to protect the downstream network, then a suitable throttle, with associated bypass or overflow, should be provided. The use of robust and simple control devices is preferred; for example, throttle pipes. Where the discharge is to a natural watercourse, the outflow should be limited to greenfield runoff rates.

For design purposes, the total drawdown time (the time until 100% of the storage capacity has been recovered) should be as short as possible, and generally should not exceed five days (National Ready Mix Concrete Association website[1]).

3.2.5   Rainfall events exceeding design storms

Additional overflows and outlets will be required for extreme events, which are in excess of the design storm and may cause backing up of water within the reservoir pavement. Without these emergency outlets, excess water may possibly compromise the integrity of the surface of the pavement or, at worse, its entire structure. Any water from emergency overflows and outlets should be routed to avoid flooding the road and also should not impact upon third parties adjacent to the Highway.

Footnote: [1] http://www.perviouspavement.org/engineering%20properties.htm

4 Design and construction of pilot-scale trial

The pilot-scale trial was built on the approach to a weighbridge in Aggregate Industries' concrete manufacturing facility at Hulland Ward, Derbyshire. The location was chosen so that traffic loading of the trial pavements by heavy goods vehicles could be monitored.

4.1 Design of test pavements

4.1.1 Structural design

The subgrade at the trial site is reasonably stiff, sandy gravel. A foundation comprising this subgrade and 350 mm of a well sorted[1], granular material would be expected to be a Class 2 foundation (Chaddock and Roberts, 2006)

In Section 3.1, it was recommended that reservoir pavements be based on a flexible pavement with a porous concrete base for the more heavily trafficked situations. In the design method, which follows that by Nunn (2004), it is assumed that it is possible to develop a Zone H5 porous concrete. This choice is considered to be conservative as the literature suggests that porous concrete can be designed within the range of mid-Zone H4 to the upper level of Zone H8. Porous concrete will conform to this Zone H5 classification when the combination of dynamic stiffness and flexural strength is above the lower bound line for the H5 Zone given in Figure 3.1. With this assumption, the resultant designs for a long-life pavement (> 80 msa) were read from Figures 3.2 and 3.3 and are given in Table 4.1.

Table 4.1 Designs for long life (>80msa) option

Class 2 Foundation		
	Conventional asphalt surfacing	Porous asphalt surfacing
Thickness of asphalt surfacing (mm)	180	180
Thickness of porous concrete (mm)	270	280
Thickness of unbound granular subbase (mm)	350	350
Class 1 Foundation		
	Conventional asphalt surfacing	Porous asphalt surfacing
Thickness of asphalt surfacing (mm)	180	180
Thickness of porous concrete (mm)	290	310
Thickness of unbound granular subbase (mm)	350	350

Footnote: [1] A well sorted (poorly graded) soil has particles that are uniform in size.

The design thickness for the porous concrete base was selected as 290 mm so that, together with the 180 mm asphalt surfacing, the pavement should readily achieve a life of 80 msa when built on a foundation of Class 2.

However, in the event, for example, that moisture infiltration into the foundation over time results in its degradation to a Class 1 foundation, then the pavement will still be able to carry 80 msa when surfaced by conventional dense asphalt and would still be capable of carrying at least 30 msa when surfaced by porous asphalt. In all cases, the proposed designs will still be extremely conservative for the predicted traffic that the trial pavements are required to carry at the factory site.

4.1.2   Hydraulic design

For a subbase reservoir, which is built to the standard 2.5% crossfall, 71 mm of rainfall is needed to fill a Permavoid module of depth 150 mm at its low side and 82 mm is required to similarly fill up granular subbase of depth 350 mm and of porosity 30%. The structural design thickness of granular subbase given in Table 4.1 therefore provides ample temporary storage capacity for severe storms. By choice of orifice size, it can be arranged for this stored water to discharge from the reservoir over 1 to 3 days so that the pavement can be ready for the next rainfall event. In exceptionally wet conditions, the porous concrete base layers can also act as an additional reservoir layer.

4.2   Test pavements

4.2.1   Trial description

The trial pavements described in this report are shown schematically in plan and cross-section in Figures 4.1 and 4.2 respectively. The test pavements are separated from one another by concrete walls and comprise the following structures:

Bay 1: Permeable, block paver surfacing (80mm) on aggregate laying course (50mm) on porous concrete base (350mm) on a Permavoid geocellular subbase (150mm) on imported unbound granular subgrade. (Reservoir pavement Type I).

Bay 2: Traditional asphalt surfacing (180mm) on porous concrete base (290mm) on a tanked, Permavoid geocellular subbase (150mm) on imported unbound granular subgrade. (Reservoir pavement Type VI)

Bay 3: Traditional asphalt surfacing (180mm) on porous concrete base (290mm) on a tanked, well sorted, unbound granular subbase (350mm), on imported unbound granular subgrade. (Reservoir pavement Type VI)

Bay 4: Porous asphalt surfacing (180mm) on porous concrete base (290mm) on a tanked, well sorted, unbound granular subbase (350mm) on imported unbound granular subgrade. (Reservoir pavement Type III)

The stated thicknesses are design, not actual, values.

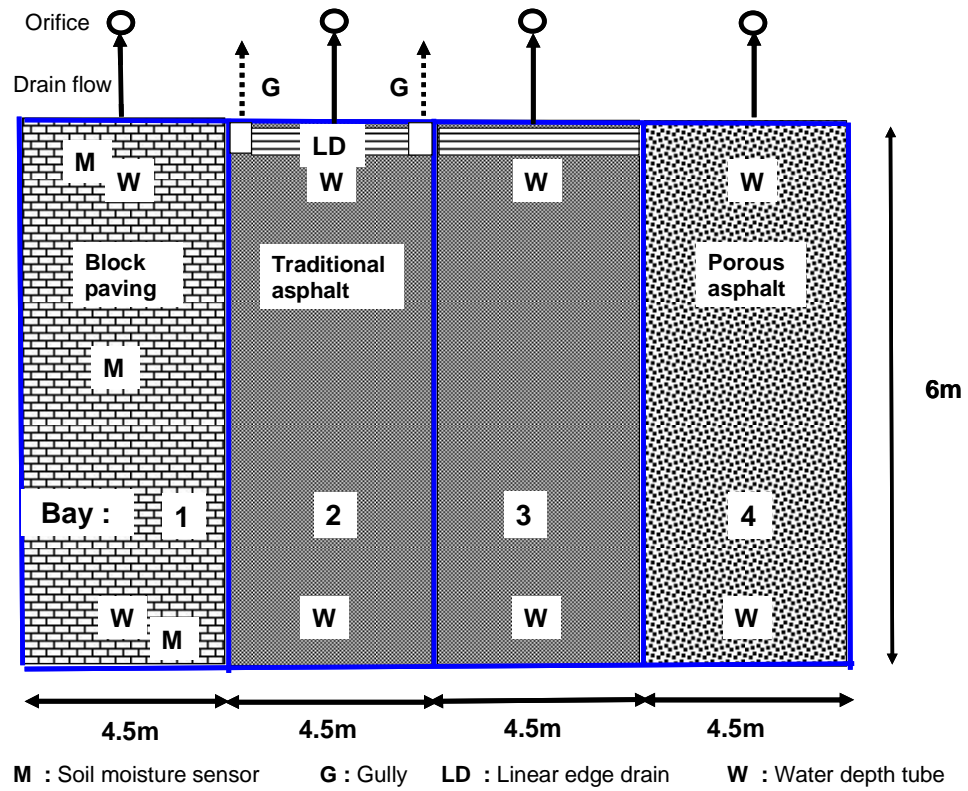The pavements were constructed to a crossfall of 2.5 per cent.

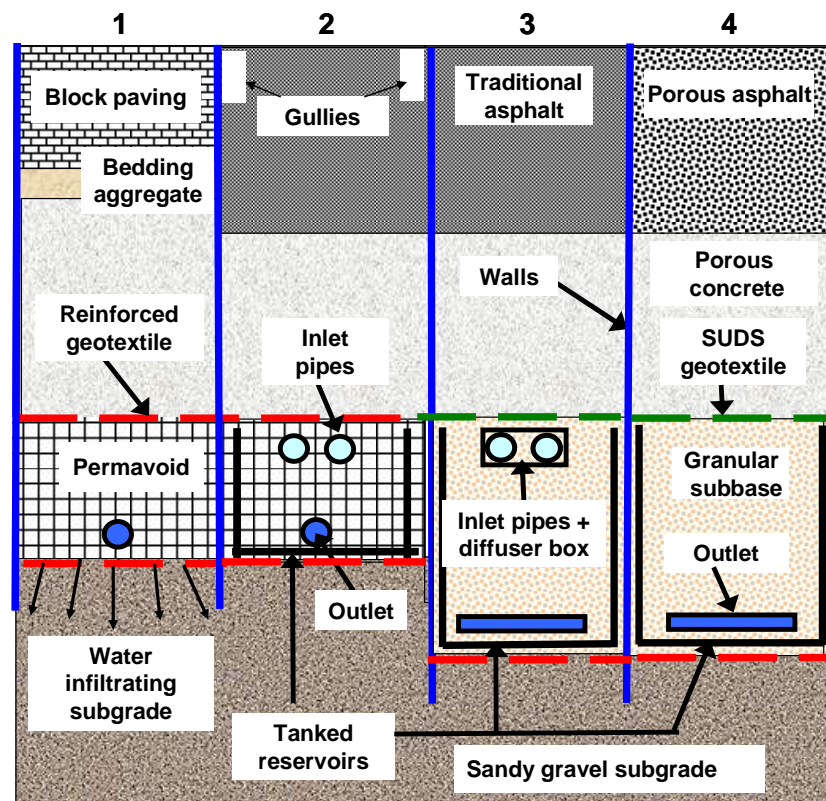**Figure 4.1 Schematic plan section of test pavements**

Figure 4.2 Schematic cross-section of test pavements

The structures can be seen to include permeable reinforcing and separation geotextiles, impermeable containment membranes as well as inlet and outlet pipes.

Bay 1 permits infiltration of water through its surfacing into a subbase storage reservoir comprised of linked geocellular boxes via a porous concrete base. The water within the subbase can discharge into the underlying subgrade with excess flowing out of the subbase through an outlet drain.

In Bay 2, the runoff of rainfall from a traditional "impermeable" asphalt surfacing is collected at an edge drain and diverted into a subbase storage reservoir comprised of linked geocellular boxes that is tanked to prevent discharge of water into the subgrade. The discharge of water held within the subbase is via an outlet drain, which can be restricted to limit the peak flows to permitted values.

For Bay 3, the runoff of rainfall from a traditional "impermeable" asphalt surfacing is collected at an edge drain and injected into an unbound granular, subbase storage reservoir that is tanked to prevent discharge of water into the subgrade. As with Bay 2, the discharge of water held within the subbase is via an outlet drain, which can be restricted to limit the peak flows to permitted values.

In Bay 4, the rainfall infiltrates into porous asphalt surfacing and passes via a porous concrete base into an unbound granular subbase storage reservoir that is tanked to prevent discharge of water into the subgrade. The discharge of water held within the subbase is via an outlet drain, which can be restricted to limit the peak flows to permitted values.

In all these pavements, there is reserve storage capacity for water in the porous concrete base overlying the subbase reservoirs.

Bay 2 is also fitted with edge gullies to simulate conventional drainage in which rainfall flows over the pavement surface and is discharged directly into an outlet drain without storage in the pavement.

Porous pavement materials generally have inferior structural properties compared with dense graded, impermeable materials. However, porous or no-fines concrete was selected for the main structural element in all the test pavements because this material can be designed to have a reasonable combination of stiffness and flexural strength. All the test pavements are flexible with a hydraulically bound mixture (HBM) base construction. The same nominal porous concrete design thickness of 290 mm was adopted for the pavements in Bays 2 to 4 regardless of whether they were surfaced with porous or conventional asphalt to facilitate direct comparison of their performance. The porous concrete design thickness in Bay 1 was increased to 350 mm to compensate for the reduced structural capacity of the pervious block paver surfacing compared to the asphalt surfacings of pavements in Bays 2 to 4. Each test pavement is 4.5 m long by 6.0 m wide.

In the pavement of Bay 1, water can infiltrate into the imported sandy gravel subgrade. A reinforced permeable, non-woven filter membrane with a permeability of 36 litres/square metre/sec was used to separate the linked modular subbase from the subgrade. This membrane needed to be tough enough to avoid tearing during construction and also to have a greater permeability than that of the underlying soil.

Pavements in Bays 2, 3 and 4 each have a tanked reservoir in which water can be stored prior to being removed at a controlled rate through an ancillary, restricted drainage system to attenuate rainwater flows. The watertight tank was formed using a heavy-duty, polypropylene membrane. The joints in the 1.0 mm thick

impermeable liner were welded and the welds pressure tested to ensure that the reservoirs did not leak water into the underlying subgrade.

The reservoir layer of pavements in Bays 3 and 4 were constructed using a gap graded granular subbase, which has an air voids content of between 30 and 40%. In Bays 1 and 2, the reservoir layer consisted of a single layer of linked Permavoid geocellular boxes. These units are 150 mm deep x 708 mm long and 354 mm wide and can be keyed together laterally and, when necessary, vertically to form a thicker layer. A Permavoid unit is shown in Figure 4.3. These units have an air voids content of 95 % and a vertical compressive strength of 715 kN/m^2.

Figure 4.3 Permavoid geocellular box

In this manner, a variety of reservoir pavement types and materials/components were studied. One of the options allowed rainfall to enter the reservoir pavement through a porous asphalt surfacing. This option simplifies the pavement structure because, in principle, auxiliary drains are not required. Porous asphalt, however, needs regular cleaning to maintain an adequate permeability to rainwater. Also, porous asphalt deteriorates faster than traditional "impermeable" asphalt and is currently not promoted by the Highways Agency (HA) for use on its road network. Consequently, an option is provided using dense "impermeable" asphalt with water collected at edge drains and diverted into the subbase reservoir. The block paver surfacing option is useful to differentiate the pavement from normal carriageway lanes. It is useful in parking areas such as safe havens and other service areas where fuel spillage may occur and damage asphalt. The option of a geocellular subbase as a replacement of unbound granular subbase is valuable when a greater quantity of water needs to be stored in a given thickness of pavement. There is also the option of discharging water into the soil when the soil permeability is sufficient and the soil will not weaken excessively when wetted. Otherwise, the option of enclosing the subbase reservoir in a tank is provided when it is necessary to protect the subgrade, or ground water or to avoid mobilising contaminants in the soil.

The water storage capacity of the trial pavements are given in Table 4.2 for two possible situations; namely, when the reservoir is full at its low side and also when the porous concrete base stores excess water in exceptionally wet conditions until it is also full at its down-slope side. These volumes are converted into equivalent depths of rainfall.

Table 4.2 Storage capacity of trial reservoir pavements

Water contained in:	Volume of water (m^3)/ Equivalent rainfall depth (mm)			
	Bay 1	Bay 2	Bay 3	Bay 4
Reservoir only	1.9 m^3/ 71 mm	1.9 m^3/ 71 mm	2.2 m^3/ 82 mm	2.2 m^3/ 82 mm
Reservoir and overlying porous concrete	5.0 m^3/184 mm	4.7 m^3/175 mm	3.7 m^3/137 mm	3.7 m^3/137 mm

Assumptions: each pavement of area 27 m^2.
Porosity: Permavoid 95%   Unbound granular material 30%   Porous concrete 15%

Without taking into account any outflow of water from the reservoir that is laid to a crossfall of 2.5%, 71mm or 82mm of rainfall will fill the reservoir at its low side depending on the reservoir type. When the porous concrete is used as an emergency reservoir, rainfall to equivalent depths of between 137mm to 184mm can be stored.

The storage capacity can be used not only for direct rainfall but also for rainfall from adjacent paved areas. Interpave (2010) recommends that the runoff area drained should be no greater than twice the area of the reservoir pavement. In that case, the maximum capacity of these pavements filled to the top of the porous concrete base layer at their down-slope side, is still a generous 45 to 60mm rainfall.

In this trial, overflow outlets were not provided to drain the pavements of stored water that exceeded the maximum design quantities. In practice, overflow outlets would be needed to avoid excess water compromising the integrity of the pavement.

4.2.2   Trial construction

The construction of the trial pavements is shown in Figures 4.4 to 4.10. The concrete walls of each bay of the pilot-scale trial during construction can be seen in Figure 4.4. The construction at the top of the imported sandy gravel soil is shown in Figure 4.5. The unbound granular material and the Permavoid modular subbases used as reservoirs are shown in Figures 4.6 and 4.7 respectively. The porous concrete and the porous asphalt surfacing are shown in Figures 4.8 and 4.9 respectively. The completed pilot-scale trial bounded by crash barriers is illustrated in Figure 4.10. The individual test pavements can be distinguished by the top of the concrete walls that formed the test bays.

Figure 4.4 Concrete bays constructed to contain the test pavements

Figure 4.5 Imported sandy gravel soil formation

Figure 4.6 Unbound granular reservoir subbase

Figure 4.7 Permavoid modular reservoir subbase

Figure 4.8 Porous concrete base of trial pavements

Figure 4.9 Porous asphalt surfacing of trial pavements

Figure 4.10 Completed pilot-scale trial

5 Pavement construction tests

At each stage in the construction of the trial, measurements were made and material was sampled for laboratory testing. The results determined are described below.

5.1 Pavement layer thicknesses

The average measured thicknesses of the pavement layers are given in Table 5.1.

Table 5.1 Constructed layer thicknesses of test pavements

Bay	Surfacing		Base		Subbase (Reservoir layer)	
	Material	Thickness	Material	Thickness	Material	Thickness
1	Permeable block paving	80 mm + 44 mm aggregate laying course	Porous concrete	351 mm	Permavoid modules	156 mm
2	Conventional asphalt	173 mm	Porous concrete	292 mm	Permavoid modules	156 mm
3	Conventional Asphalt	172 mm	Porous concrete	299 mm	Unbound granular	351 mm
4	Porous asphalt	173 mm	Porous concrete	298 mm	Unbound granular	351 mm

These thicknesses are in reasonable agreement with the nominal design thicknesses given in Section 4.1.

5.2 Foundation strength

Four dynamic cone penetrometer (DCP) tests were carried out on the subgrade in each pavement to depths of between about 800 and 900 mm prior to placing the reservoir layer. The CBR values deduced from these DCP measurements for regions of approximately constant strength are shown in Table 5.2.

Table 5.2 DCP measurements of subgrade strength

Test section	CBR (%) of subgrade layers:			Thickness (mm) of top layer	Test depth (mm) analysed
	1 (Top)	2	3		
1	7	24	21	155	867
	12	28	12	153	873
	9	23	19	159	866
	10	27	34	187	560
Average:	**10**			164	
2	9	20	44	160	905
	16	35	45	159	875
	13	27	43	176	894
	13	32	51	156	876
Average:	**13**			162	
3	10	31	64	168	612
	11	43	28	161	878
	12	42	31	173	837
	11	29	67	172	642
Average:	**11**			169	
4	11	39	27	203	869
	12	22		204	864
	10	21	39	156	385
	11	22	33	175	600
Average:	**11**			185	

These results indicated that the California bearing ratio (CBR) of the top 150 to 200 mm of imported material immediately below the subbase was on average 11 %. Individual readings were in the range 7 to 16 %. Below this layer and up to depths of about 900 mm, individual CBR values were generally in excess of 20 %.

The imported upper subgrade is a sandy gravel material. During pavement design, it was considered likely that this soil together with the overlying 350 mm granular subbase as reservoir layer would provide a Class 2 foundation within the completed pavement. This assertion was supported by tests with a Light Weight Deflectometer (LWD) at formation level that resulted in average surface modulus values of 143 MPa, 122 MPa and 114 MPa for Bays 2, 3 and 4 respectively and a lower value of 86 MPa for Bay 1. LWD tests on the surface of the unbound granular reservoir layer of Bays 3 and 4, however, were inconclusive. Their average surface modulus values were 34 MPa and 38 MPa for Bays 3 and 4 respectively. These values are low, as might be expected for tests directly on a well sorted (poorly graded), unbound granular material in an unconfined state. Higher values of foundation stiffness might have been measured during pavement construction if a blinding layer of smaller sized particles than the subbase aggregates had been compacted into the subbase surface as described in Section 3.1. It was expected, however, that the stiffnesses of the trial foundations would be higher when the subbase was confined by bound layers. This supposition was

supported by the results from later tests on the completed pavement that are reported in Section 7.

LWD tests on the Permavoid geocellular subbase were not performed during pavement construction because the subbase comprises an interlocking set of boxes and its behaviour under a bound pavement layer is unlikely to be predicted by a direct dynamic plate loading test.

5.3 Pavement material properties

The porous concrete material, Hydrain, used a crushed rock aggregate. Its strength properties were measured in the laboratory after various curing times using material sampled from the site and compacted to different levels of compaction in moulds. These results are shown in Table 5.3.

Table 5.3 Properties of Hydrain, porous concrete

Compaction level		Development of Compressive strength (MPa)			Flexural strength f_f (MPa)	Static modulus E_s (GPa)	Dynamic modulus E_D (GPa)
Description	(%v/v)				Age (Days)		
		7	28	180	28	28	28
Low	48.3	1.0	7.4	8.3	1.6	10.8	28.5
Medium	28.4	3.0	16.4	17.3	2.6	17.5	35.6
High	13.5	7.9	28.6	30.6	3.0	20.2	37.7

Indicative values of the static modulus (E_S) were estimated using the formula E_S = 6750 x flexural strength (f_f), suggested by Delatte (2007), and the dynamic modulus (E_D) was estimated using the following formula of Nunn (2004) for cement bound materials based on crushed rock aggregate:

$$\mathbf{E_D = (Log(f_f) + 0.636) / 0.0295} \qquad 5.1$$

Table 5.3 indicates that at the minimum target air voids content of the porous concrete of about 14 %v/v, the flexural strength, static modulus and dynamic modulus of the porous concrete are in the region of 3.0 MPa, 20 GPa and 37 GPa respectively at 28 days. According to Figure 3.1, this material could be classified as a Zone H7 hydraulic bound base despite the mechanical properties being applicable to an age of 28 not 360 days. The porous concrete therefore comfortably satisfied the pavement design requirement of a Zone H5 base.

The similarity of the 28 and 180 day compressive strengths in Table 5.3 indicates that porous concrete cures more rapidly than conventional materials as noted by Kolias & Williams (1978) and that its long term properties may be realised after about 180 days.

The mean air voids content of the porous asphalt was 27.4 %v/v and the mean indirect tensile stiffness modulus (ITSM), measured using laboratory produced specimens, was approximately 3.0 GPa. This value is comfortably above the design value of 2.0 GPa used for pavement design.

6 Tests on completed pavements

The construction of the trial was completed in early summer 2008 and the following studies were undertaken.

6.1 Pavement studies

The pavements were loaded by heavy vehicles and structural evaluations performed as follows:

- The effects of time, traffic and temperature on the structural behaviours of the test pavements were assessed using a falling weight deflectometer.

- Evidence of deterioration of the pavement was sought from visual condition surveys of surface defects and cracks and from monitoring pavement deformation in the wheel tracks.

6.2 Hydraulic studies

In addition to natural rainfall, various rainfall events were simulated and, where appropriate, infiltration tests carried out on the surfaces of the reservoir pavements. Hydraulic investigations were carried out as follows:

- An assessment was made of the ability of the subgrade of the pavement in Bay 1 to drain rainfall.

- Determinations were made of the natural attenuations to peak rainfall intensities that were caused by water flowing through the various materials and structures of the reservoir pavements. The associated delays to the peak water flows from the reservoirs were also measured.

- The effect of restrictor orifices on peak water flow in drains was quantitatively investigated.

- Reductions to the water infiltration capacity of the pervious surfaces of the pavements in Bays 1 and 4, which were caused by clogging with detritus, were monitored. The effectiveness of various methods of cleaning these surfaces was investigated.

7 Structural performance of pilot-scale trial pavements

The structural performance of the trial pavements under lorry traffic was assessed by structural tests and visual condition surveys.

7.1 Traffic loading

The test pavements were trafficked by heavy goods vehicles carrying aggregate and cement as they approached the weighbridge before entering the factory site. The gross vehicle weight (GVW) of each vehicle was recorded along with its type for nineteen weeks during the six month period following the opening of the trial to traffic. Based on these measurements, a reasonable projection of traffic for the next 15.5 months was then made. Traffic was deducted from weighbridge data for the periods when the trial was closed; for example, to carry out hydraulic tests or for cleaning the pervious surfaces. The distribution of the vehicle loads over 2, 3, 4 or 6 axles of the various lorry types was estimated using the work of Newton (2010). These data were converted into a number of equivalent 80 kN standard axle (sa) loads, assuming that the fourth power damage law applies. The fourth power law assumes that N_L passes of an axle load of L kN will cause as much damage as N_{80} passes of an axle load of 80 kN as given by the following equation:

$$N_{80} = N_L \left(L/80 \right)^4 \qquad \qquad \text{...... } 7.1$$

For the 21.5 months following the opening of the trial, it was estimated that the trial pavements carried about 70,000 standard axles.

7.2 Structural assessment: Transient deflection behaviour

The falling weight deflectometer (FWD) was used to test the trial pavements. The FWD is a non-destructive testing device that applies to the pavement a dynamic load generated by a falling weight. The resulting pavement deflections at different distances from the loaded area are measured using a series of geophones. These deflections were analysed to give the overall pavement stiffness and estimates of the stiffnesses of the component layers of the pavement.

Tests with the FWD on the completed trial pavements were conducted on the 27th May 2008, before opening the pavements to traffic, and also on the 15th June 2009. Tests were conducted in the wheelpaths and between these wheelpaths. For each trial pavement, twelve positions in wheelpaths were tested and up to four locations were tested between wheelpaths. The analysis of the deflection measurements by two procedures is described in the following sections.

7.2.1 Surface modulus analysis

The stiffness of the combined pavement and foundation, which is called in this document "surface modulus", was calculated from the load applied to the plate and the resulting deflection at the plate centre as given in the Highway Agency's Highways document HD29 (HA et al, 2008).

The values of surface modulus determined on the 15th June 2009 are plotted against those values determined on the 27th May 2008. A line of equality is also plotted to aid their interpretation. The temperature at a depth of 100 mm in the pavement was on average 13.4°C in May 2008, but much hotter at a mean value

of 32.9°C in June 2009. Tests in wheelpaths are distinguished from those between wheelpaths. Results from the four Bays labelled 1 to 4 in Figure 4.1 are shown in Figures 7.1 to 7.4 respectively.
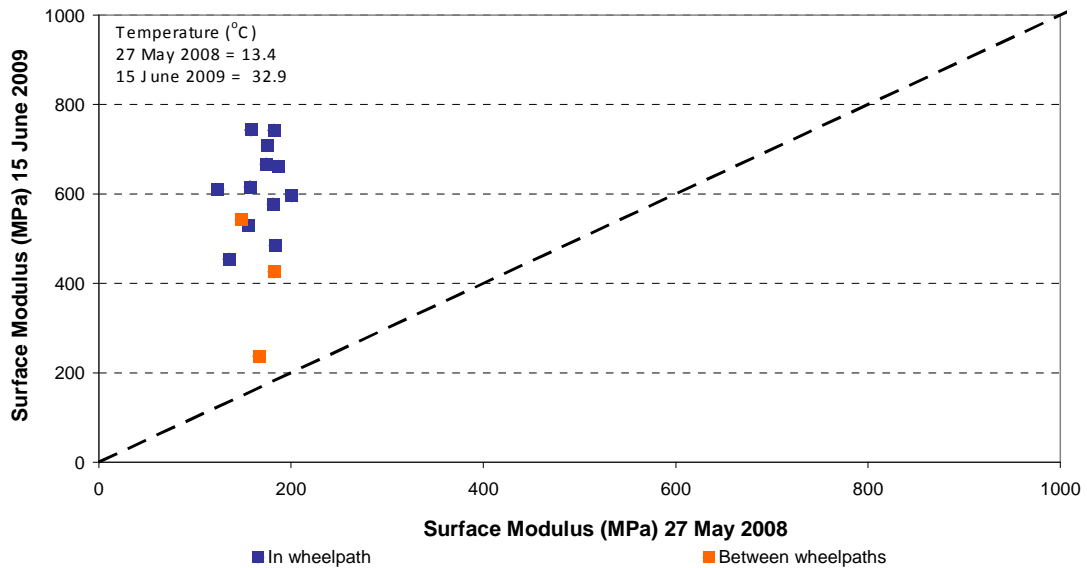

Figure 7.1: Combined effect of time, temperature and traffic on the surface modulus of Bay 1
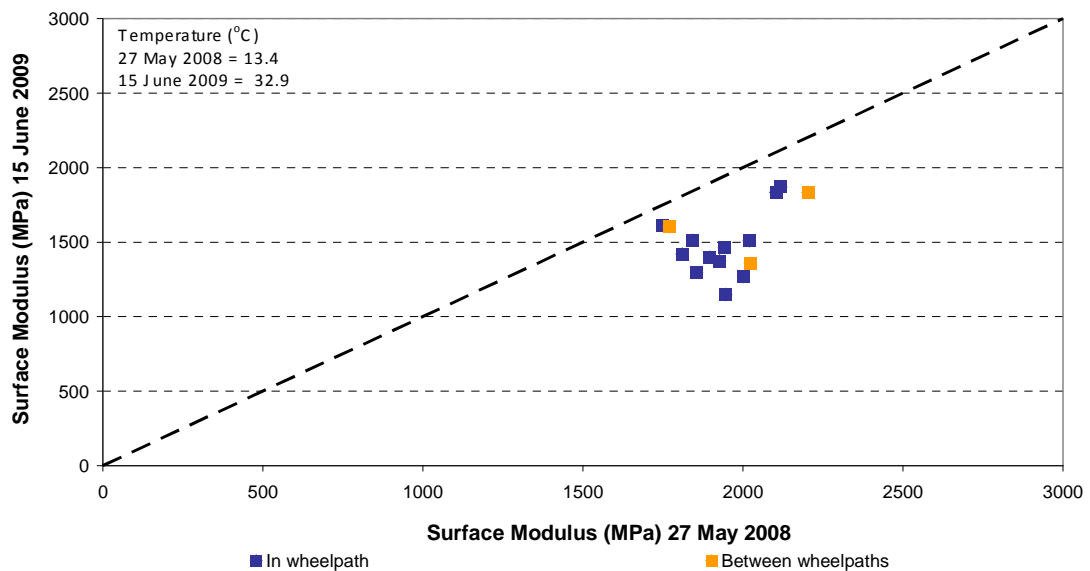

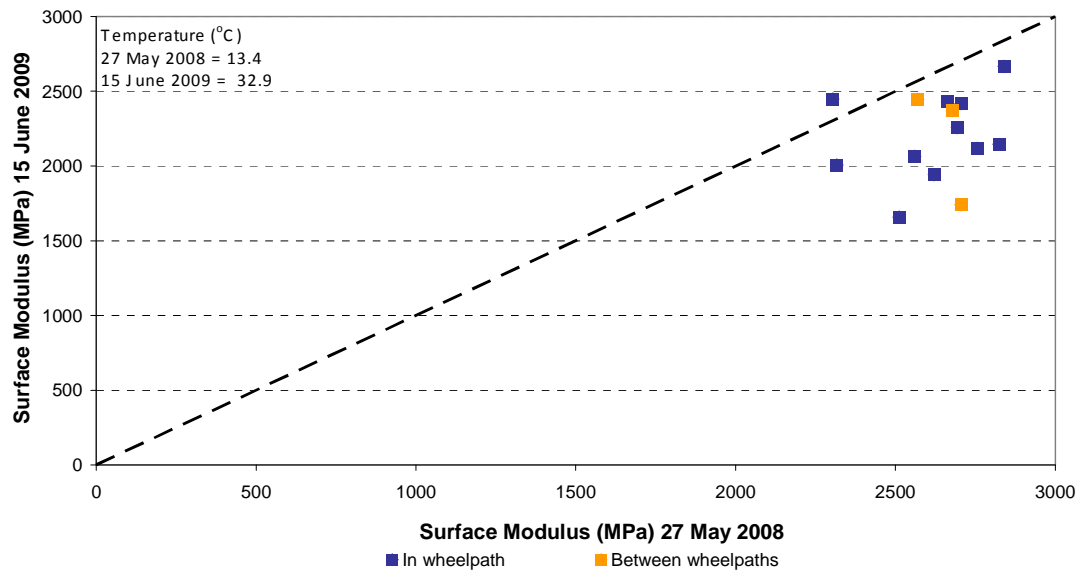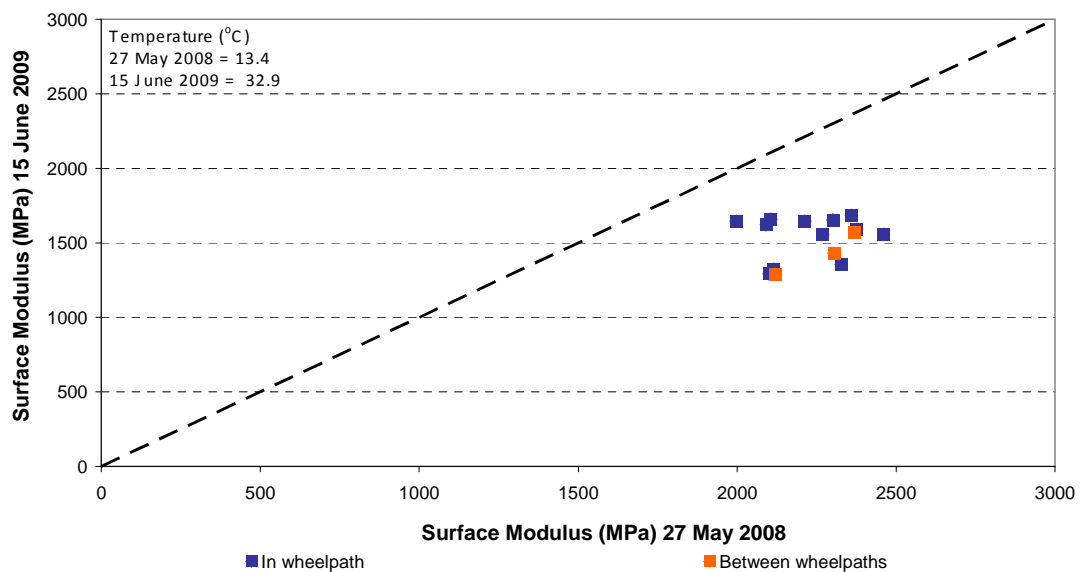**Figure 7.2: Combined effect of time, temperature and traffic on the surface modulus of Bay 2**

Figure 7.3: Combined effect of time, temperature and traffic on the surface modulus of Bay 3

Figure 7.4: Combined effect of time, temperature and traffic on the surface modulus of Bay 4

Figure 7.1 shows that for the structure in Bay 1, surfaced by block paving, the surface modulus in June 2009 is higher than its value in May 2008. More specifically, there is a greater increase in the structure's surface modulus in the wheelpaths.

The surface modulus values of the structures in Bays 2 to 4 that were determined in June 2009 are shown in Figures 7.2 to 7.4 respectively to be, for all but one

test, less than the equivalent values determined in May 2008. For these particular pavement structures, however, there are no significant differences between the surface modulus values determined in the wheelpaths and those values determined between the wheelpaths.

7.2.2   Back analysis

For each of the Bays 2 to 4, the layer stiffnesses of the composite bound pavement and the underlying foundation were estimated by back analysis of the deflection profiles of the pavement's surface caused by loads applied by the FWD. The composite bound pavement comprised asphalt on concrete while the foundation consisted of subbase on subgrade; that is, natural and the overlying imported soil. For Bay 2, the foundation included the linked modular subbase. The average values of pavement and foundation stiffness are given in Table 7.1 for tests in the wheelpaths and between the wheelpaths of each bay and for each test series.

Table 7.1: Layer stiffnesses from back-analysis of FWD delection profiles

Pavement construction measurements			
Measure:	Bay 2	Bay 3	Bay 4
Formation stiffness (MPa) :	143	122	114
Bound pavement thickness (mm) :	465	471	471
Layer stiffnesses - Tests on 27th May 2008 {Temperature at a depth of 100 mm = 13.4ºC}			
Measure:	Bay 2	Bay 3	Bay 4
Pavement layer stiffness (GPa):			
In wheelpaths ([1])	10.16 (4)	13.31 (3)	10.39 (4)
Between wheelpaths ([1])	12.45 (1)	13.58 (1)	10.83 (1)
Foundation half-space stiffness MPa):			
In wheelpaths([1])	192 (4)	280 (3)	259 (4)
Between wheelpaths ([1])	183 (1)	291 (1)	267 (1)
Layer stiffnesses - Tests on 15th June 2009 {Temperature at a depth of 100 mm = 32.9ºC}			
Measure	Bay 2	Bay 3	Bay 4
Pavement layer stiffness (GPa):			
In wheelpaths ([1])	5.24 (4)	8.17 (3)	4.84 (4)
Between wheelpaths ([1])	5.44 (1)	6.27 (1)	4.25 (1)
Foundation half-space stiffness MPa):			
In wheelpaths ([1])	168 (4)	251 (3)	268 (4)
Between wheelpaths ([1])	169 (1)	256 (1)	270 (1)
Percentage differences in stiffness (%): 15th June 2009 compared to 27th May 2008			
Measure	Bay 2	Bay 3	Bay 4
Pavement layer stiffness :			
In wheelpaths	48.4 ([2])	38.6 ([2])	53.4([2])
Between wheelpaths	56.3([2])	53.8([2])	60.8 ([2])
Foundation half-space stiffness :			
In wheelpaths	12.5 ([2])	10.4 ([2])	-3.5 ([2])
Between wheelpaths	7.7 ([2])	12.0 ([2])	-1.1 ([2])

([1]) Number of readings   ([2]) {(May stiffness-June stiffness)*100 / May stiffness)}

For Bay 1, the back analysis process resulted in goodness of fit values of the calculated bowl shape that were outside the guide values given in HD29. It was therefore concluded that this form of analysis was inappropriate for a pavement structure comprising a block paving on bedding material surfacing on a porous concrete base.

Only several positions in the wheelpaths and between wheelpaths near the centre of each pavement slab were back analysed to limit the undesirable effects of the pavement edge.

In May 2008, the equivalent stiffness of the combined asphalt and concrete layer varied from approximately 10 GPa to about 13.5 GPa with the foundation half-space stiffness varying from just over 180 MPa to about 290 MPa. Later in June 2009, the equivalent stiffness of the bound pavement layer ranged from just over 4 GPa to about 8 GPa with the foundation stiffness varying from just under 170 MPa to 270 MPa. The proportional differences in these stiffnesses when June 2009 values are compared to May 2008 determinations are also given in Table 7.1.

7.2.3   Discussion

It was considered that the geometry of the site would cause lorries to drive primarily down the centre line of the trial, although transverse deviations were expected. For Bay 1, it is postulated from the evidence shown in Figure 7.1 that the surface modulus of the complete structure in the wheelpaths increased over that between the wheelpaths because the trafficking by loaded lorries compacted the surfacing of block paving and aggregate bedding material.

The Highways Agency's document HD 29 (HA *et al*, 2008) states that a back-analysed foundation stiffness of greater than 100 MPa is generally associated with good performance. All of the foundations of the pavements in Bays 2 to 4 have stiffnesses that exceed this indicative value. For Bays 3 and 4, the estimates of foundation stiffness, when compared to the values measured directly in Section 5.2, also imply that the stiffness of the well sorted (poorly graded), granular subbase is enhanced when in a confined state.

The estimated foundation stiffness in Bay 2 is shown in Table 7.1 to be less than the foundation stiffnesses in Bays 3 and 4. As the formation stiffness at the top of the imported soil, also given in Table 7.1, was greater in Bay 2 than in Bays 3 and 4, it can therefore be concluded that this lower foundation stiffness estimate for Bay 2 is because of the use of a linked modular subbase system of thickness 150mm instead of a thicker 350mm layer of unbound granular subbase.

The composite bound layers of test pavement 4 generally has a lower back-analysed stiffness than the same layers of test pavements 2 and 3 and this behaviour may be due to the former pavement being surfaced by porous asphalt, whereas the latter pavements have surfaces of conventional asphalt that are structurally more competent.

Figures 7.2 to 7.4 show a similarity in the surface modulus values of the complete structure in the wheelpaths and in between wheelpaths for the pavements in Bays 2 to 4 respectively. This behaviour implies that structural deterioration such as the cracking of the porous concrete by trafficking had not occurred. This deduction is supported by the data in Table 7.1 that shows that the percentage difference in stiffness between the May 2008 tests and the June 2009 tests for the bound pavement layer is not greater in the wheelpaths than between the wheelpaths for each Bay. Measurements show the opposite to have occurred.

The surface modulus of the pavements in Bays 2 to 4 can also be seen in Figures 7.2 to 7.4 to be lower in June 2009 compared to the equivalent values in May 2008. As no deterioration by traffic in the pavement layers is suspected and foundation stiffnesses are similar on these two occasions; that is, differing only by between an increase of about 4 per cent and a decrease of approximately 13 per cent, it is concluded that the reduction in the surface modulus of the complete pavements is due to the marked reduction in the layer stiffness of the asphalt.

This behaviour is considered to be a result of the higher temperatures in June 2009 of about 33°C compared to those temperatures in May 2008 of about 13°C.

7.3   Structural assessment: Permanent deformation behaviour

Deformation of the pavement in the wheelpaths by traffic was measured by straight edge and wedge tests. The rut depth in the wheelpaths of Bay 1 that is surfaced by block paving ranged from 3 mm to 11 mm and was on average 6 mm. This deformation is attributed to the compaction of the aggregate bedding material and its migration into joints between the blocks. Rut depths in Bays 2 and 3 and Bay 4, which were surfaced by dense and porous asphalt respectively, were less than those in Bay 1 and were so small compared to the accuracy of measurement that numerical values cannot be assigned.

7.4   Structural assessment: Cracking behaviour

The pavement trial sections of length 4.5 m long by 6 m wide were not pre-cracked due to their short length. Pavements in normal construction, however, would be pre-cracked when the concrete has a compressive strength at 7 days of 10 MPa and higher. The cracks, which according to HD 26 (HA *et al*, 2006) are required to be spaced 3 m apart, would normally lower the layer stiffness of the bound material predicted by back analysis by an extent depending on the load transfer from one concrete slab to the next.

As explained earlier in Section 7.2.3, there is no evidence of cracking of the pavement under traffic from analysis of the FWD tests. Also, no cracks were found in the surfacings of the pavement by visual condition survey.

7.5   Summary and conclusions of the structural investigation

- The pavement surfaced by block paving became stiffer in the wheelpaths of loaded lorries.

- The flexible pavements with porous concrete bases showed, to date, no evidence of deterioration under traffic by cracking within the porous concrete base.

- The stiffnesses of the complete pavements, as quantified by surface modulus determinations, were lower in June 2009 than in May 2008, presumably due to the effect on the asphalt of the markedly higher test temperature, which was 20°C greater during the tests in June 2009 than those in May 2008.

- The contribution of a linked modular subbase system to foundation stiffness, which describes the foundation's support of the overlying pavement, should be explored along with how this type of subbase layer can be incorporated in traditional pavement design methods.

- Further FWD tests are recommended to continue the establishment of changes in the structural performance of all the trial pavements with traffic and environmental factors to establish their deterioration mechanisms. Specific attention is required to pavements that incorporate a linked modular subbase.

- Rutting of the block paving of Bay 1 in the wheelpaths of loaded lorries was, on average, about 6 mm. No significant wheelpath rutting, however, was detected in the asphalt surfacing of the other pavements in Bays 2 to 4 regardless of whether the asphalt was comprised of dense or porous materials.

8 Hydraulic performance of pilot-scale trial pavements

The hydraulic performance of the trial pavements was, in the main, assessed by simulating rainfall and measuring water flow in outfall drains. Three series of hydraulic tests were performed by ADAS. Initially tests were carried out to commission the rain simulation equipment and to confirm the operation of the reservoir pavements. A series of quantitative tests were then made on the pavements with the drains fully open and, separately, restricted by orifice plates to delay the flow of water from the reservoir. Unfortunately, leaks were found in the pipes outside the reservoir pavements that connected the pavements to the measurement chamber. Following remedial work to seal these leaks, a final series of measurements was conducted with, as before, the drains with, and without, orifice restriction. More details of the experimental programme are given in Table 8.1.

8.1 Test methods and results

8.1.1 Initial commissioning tests

In the initial tests, about 850 litres of water were applied to the surfaces of the trial pavements by a specially developed sprinkler system to simulate rainfall. The equipment was a simplified variant on the standard set up shown in Figures 8.1 and 8.2 operating on a block paved and asphalt surfaced pavement respectively. In the standard arrangement, four straight pipes were each fitted with five rotating sprinklers at 500 mm spacings. The simplified system was based on three pipes that were operated continuously over approximately 50 minutes. The simulated rainfall equated to an average intensity of about 37 mm/h. The water application is recorded in Table 8.2 along with qualitative descriptions of the hydraulic behaviour of each pavement.

Table 8.2 Initial simulated rainfall tests

Bay	Water applied (litres)	Duration (minutes)	Pavement area (m^2)	"Rainfall" rate (mm/h)	Dipwells High	Low	Drain flow observed
1	>760	45					
	Corrected ~ 840	50	27	37.5	Dry	Dry	No
2	846	50	27	37.6	Dry	Wet	Yes
3	847	50	27	37.6	Dry	Wet	Yes
4	823	50	27	36.6	Dry	Wet	Yes

Table 8.1 Programme of hydraulic tests

Date (dd-mm-yy)	Rainfall simulation tests: Simplified	Rainfall simulation tests: Standard storm	Tests without orifice restrictors in Bays: 1	2 G	2 LD	3	4	Tests with orifice restrictors in Bays: 1 [c]	2 [a] G	2 [a] LD	3 [b]	4 [b]	Comments
6-8-08	Yes (Y) [1]	-	Y	Y	Y	Y	Y	-	-	-	-	-	Commissioning tests: No drain flow measurements
24-9-08	-	Y [2]	Y	Y	-	Y	Y	-	-	-	-	-	
25-9-08	-	Y [2]	-	-	Y	Y	Y	-	-	-	-	-	Series 2 tests: Without orifice
8-10-08	-	Y [2]	-	-	-	-	-	-	-	-	Y	Y	
10-10-08	-	Y [2]	-	-	-	-	-	-	-	Y	-	-	With orifice
10-10-08	-	Y ([2] + edge water)	-	-	-	-	-	Y	-	-	-	-	
19-1-09 to 21-1-09	-	-	-	-	-	-	-	-	-	-	-	-	Tests in Bays 2, 3 and 4 to investigate water leakage
16-6-09	-	-	-	-	-	-	-	-	-	-	-	-	Relative hydraulic conductivity tests on Bays 1 and 4

Key: G: Rainfall runoff drained at edge of pavement by gullies.   LD: Rainfall runoff drained at edge of pavement by linear drain.
[1] Constant rainfall of 840 litres over 50 minutes.   [2] Variable rainfall to simulate the growth and decline in intensity of a 60 minute storm.
[a] Orifice size 5 mm   [b] Orifice size 4 mm   [c] Orifice size: 8 mm

Table 8.1 Programme of hydraulic tests (continued)

Date (dd-mm-yy)	Rainfall simulation tests:		Tests without orifice restrictors in Bays:					Tests with orifice restrictors in Bays:					Comments
	Simplified	Standard storm	1	2 G	2 LD	3	4	1	2ª G	2ª LD	3ᵇ	4ᵇ	
01-09-09 to 09-09-09			Remedial work to seal leaks in connecting pipes to Bays 2, 3 and 4										
18-11-09, 19-11-09, 02-12-09			Cleaning and relative hydraulic conductivity tests on Bays 1 and 4										
16-12-09	-	Y [2]	A [4]	Y	-	-	A [4]	-	-	-	-	-	Series 3 tests: Without orifice
17-12-09	-	Y [2]	-	-	-	Y	-	-	-	-	-	-	
02-02-10	-	Y [2]	-	-	Y	-	-	-	-	-	-	-	
04-02-10	-	Y [2]	-	-	-	-	-	-	-	Y	Y	-	With orifice
17-02-10			Cleaning of Bays 1 and 4 and relative hydraulic conductivity tests on Bays 4										
10-03-10	Y [3]	-	Y	-	-	-	-	-	-	-	-	-	
10-03-10	-	Y [2]	-	-	-	-	Y	-	-	-	-	-	Series 3 tests: Without orifice
11-03-10	-	Y [2]	-	-	-	-	-	-	-	-	-	Y	With orifice

Key: G: Rainfall runoff drained at edge of pavement by gullies.   LD: Rainfall runoff drained at edge of pavement by linear drain.
[2] Variable rainfall to simulate the growth and decline in intensity of a 60 minute storm.   [3] Constant rainfall of 432 litres over 60 minutes
[4] A: Abandoned   ᵃ Orifice size 5 mm   ᵇ Orifice size 4 mm

8.1.2 Second test series

In the second series of tests, a closer simulation of a storm was sought by applying rainfall that was of a lower intensity at the start and end of the storm than in the middle of the storm. A standard storm was selected and delivered by equipment that comprised four parallel pipes that were placed in the middle of each bay transverse to the line of traffic as shown in Figures 8.1 and 8.2. Because cumulative water flow from the outfall drain is quantitatively compared to the amount of the applied rainfall, the edges of the pavement were enclosed by polythene sheeting to confine all water to the pavement surface.

Figure 8.1: Simulated rainfall on block paving (Bay 1)

Figure 8.2: Simulated rainfall on asphalt (Bay 2)

The standard storm was about 1 hour duration. To approximately simulate the typical variation in the rainfall intensity, the standard storm ran the two inner pipes in the first and last quarter of the hour and all four pipes for the middle half hour. For tests without orifice restrictors, about 1250 litres of water were applied over 1 hour such that about 275, 360, 350, 265 litres were applied in successive quarter of an hour periods. For the pavement dimensions of 6m by 4.5m, or 27 m^2, the average rainfall intensity was about 44 mm/h with approximate minimum and maximum intensities of 40mm/h and 53mm/h. By comparison, the simulated storm exceeds a 1 in 5 year, one hour summer storm in the London area that has an average intensity of 21mm/h. The simulated storm variation is shown in, for example, Figures 8.3 to 8.5.

Depending on the pavement structure, the rainfall either permeates through the pavement surface into a subbase reservoir or it flows over the pavement into an edge drain that is connected to a subbase reservoir. The water can be temporarily stored in the reservoir before flowing out of the pavement and through an outfall drain to a measurement chamber. For this test series, the water flow was measured by a calibrated weir device. Fitting an orifice restrictor to the drain delays water flow and reduces its rate of flow. Tests were conducted with, and without, restrictor orifices.

Unfortunately, there was leakage in the connecting pipes between the pavements and the measurement chamber. The only Series 2 tests reported are those in which more than 66 % of the water applied was collected. These particular tests are those without orifices on Bays 1, 2 and 4. For tests on Bays 2 and 4, variations with time of the measured water flow and of the simulated storm intensities are compared in Figures 8.3 and 8.4 and Figure 8.5 respectively.
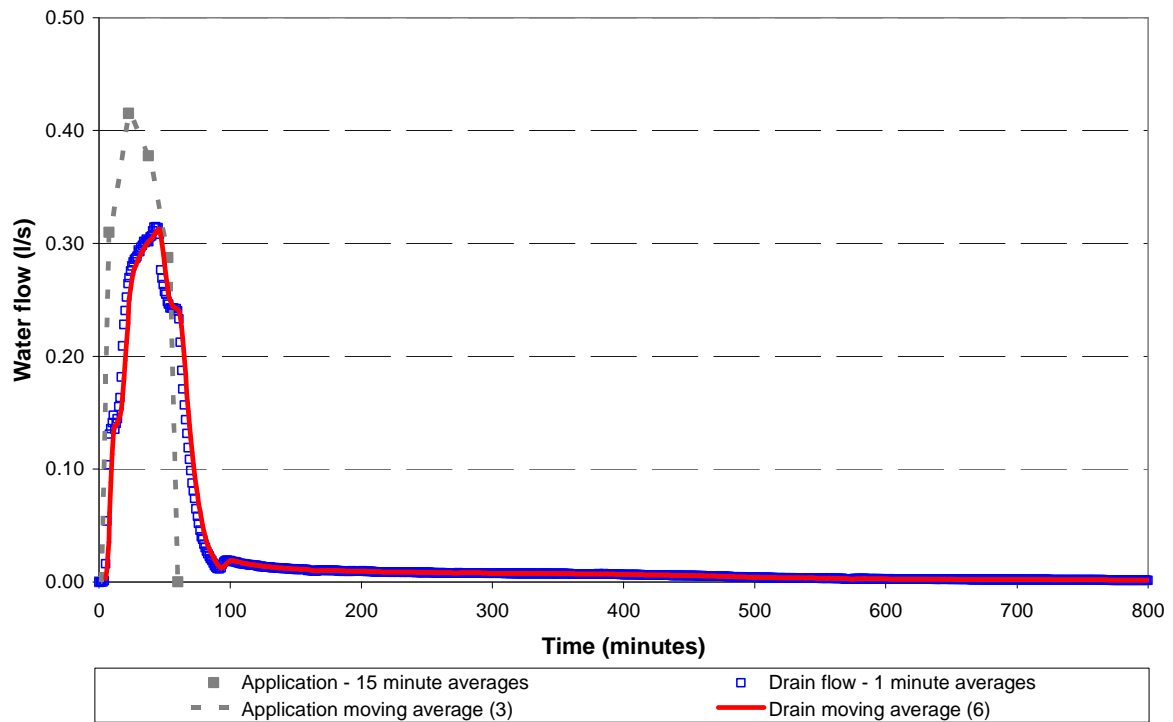
Figure 8.3 Variation in water flow with time for Bay 2 edge gullies: Unrestricted flow
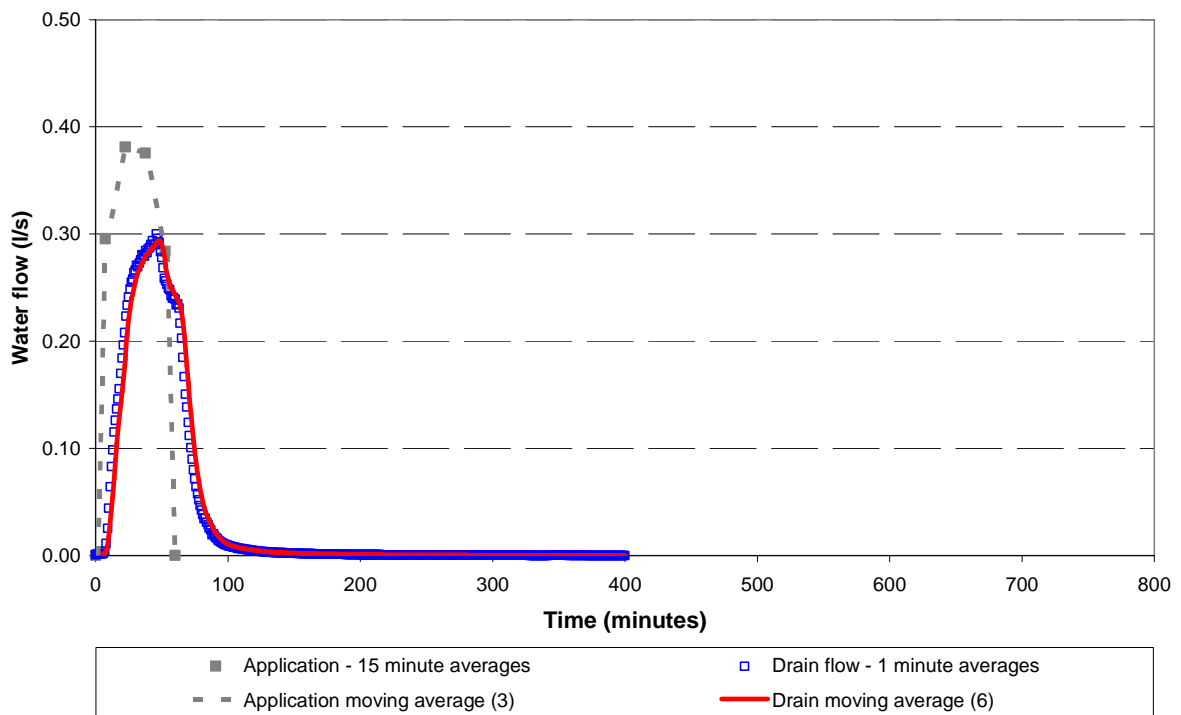
Figure 8.4 Variation in water flow with time for Bay 2 linear edge drain: Unrestricted flow
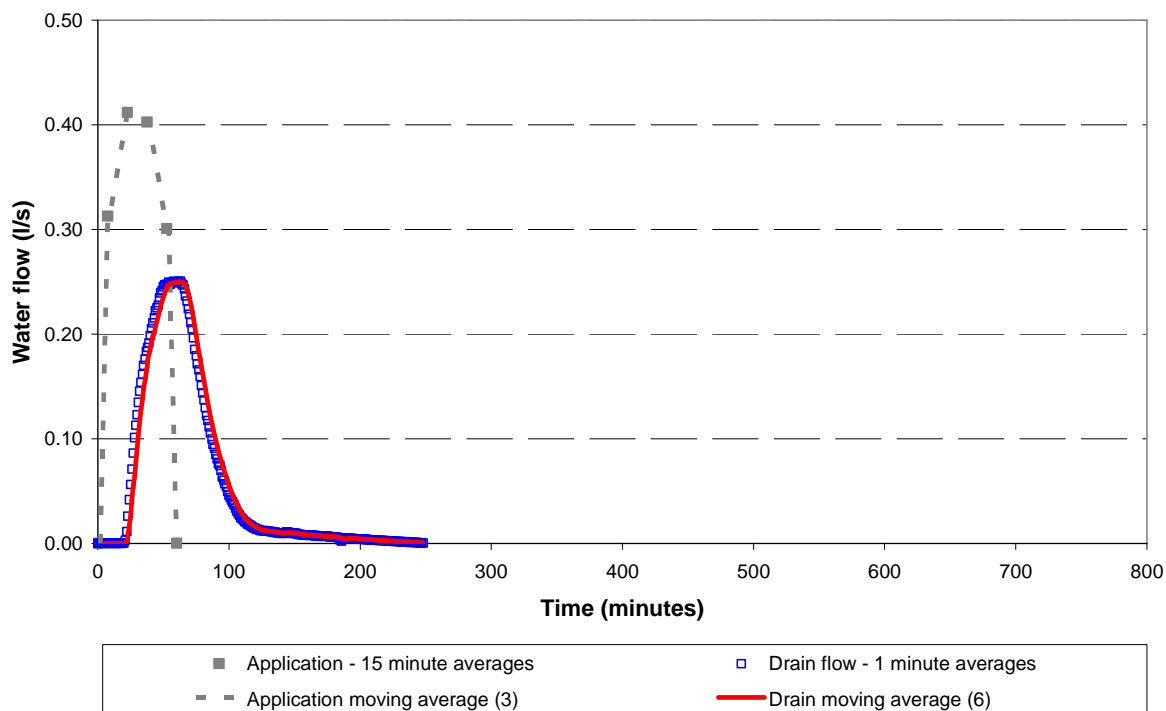
Figure 8.5 Variation in water flow with time for Bay 4 Porous asphalt: Unrestricted flow

8.1.3 Third test series

An investigation identified leaks in the pipes connecting the pavements and the measurement chamber for Bays 2 to 4 inclusive with Bay 3 identified as loosing most water. These leaks were sealed and a third series of tests was then carried out with similar experimental arrangements to the previous series of tests. In this case, however, a tipping bucket device was used for measuring water flow from the pavements instead of the weir apparatus for added precision over the range of flows anticipated from open drains and those fitted with restrictors.

A similar rainfall event was adopted and, for tests without orifice restrictors, about 1225 litres of water were applied over 1 hour such that about 225, 395, 400 and 205 litres were used in successive quarter of an hour periods.

For the Bays 2 and 3 with impermeable surfaces and edge drainage, the pavements successfully drained the substantial standard rainfall event.

For Bays 1 and 4, however, the permeable block paved and porous asphalt surfaces at this industrial complex had become substantially clogged during their use and water from the standard rainfall event ran off the low edge of these pavements instead of infiltrating into these surfaces. Various devices were used to clean the block paved and porous asphalt surfaces with relative hydraulic conductivity tests carried out after each cleaning method to judge their effectiveness as described in section 8.2.3. The hydraulic conductivity of Bay 4 was sufficiently improved to permit the porous asphalt surfacing to drain the standard storm of average rainfall intensity of about 46 mm/h. The hydraulic performance of Bay 4 is illustrated below in this section together with those of Bays 2 and 3. Although improved, further cleaning of the block paved surface of Bay 1 was required. Its hydraulic behaviour is discussed separately in Section 8.2.1.2.

The variations with time of the intensities of the rainfall event and of the flows in the outfall drains, which were unrestricted by orifice restrictors, are illustrated below in Figures 8.6 and 8.7, 8.8 and 8.9 for Bay 2 drained by gullies or linear edge drain, Bay 3 and Bay 4 respectively.
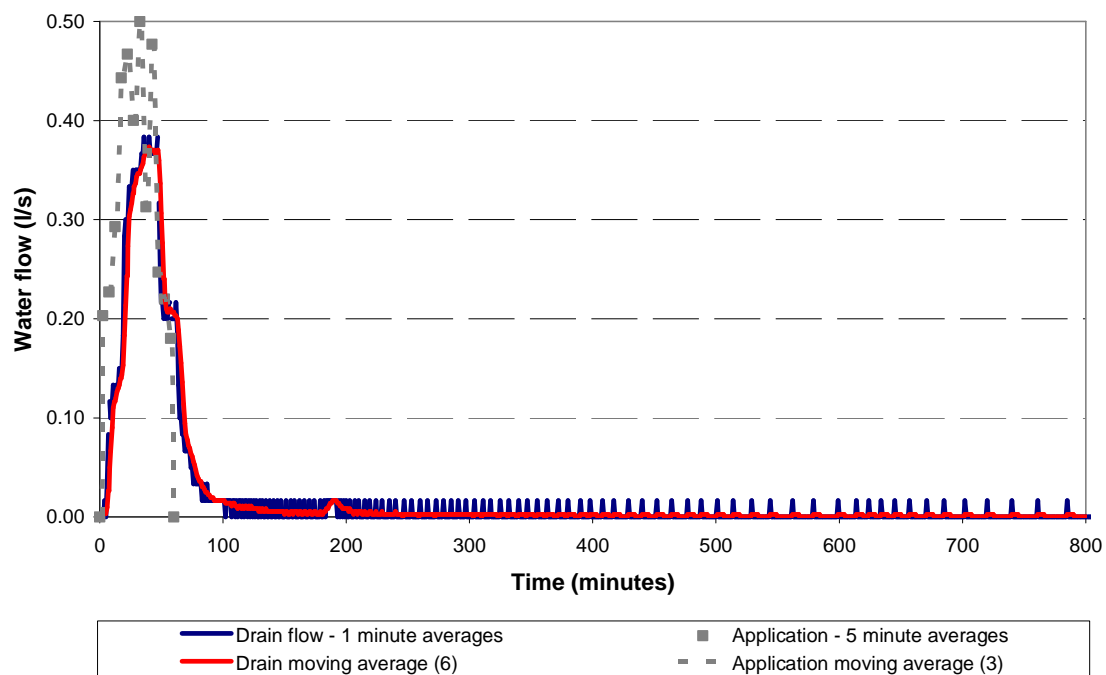
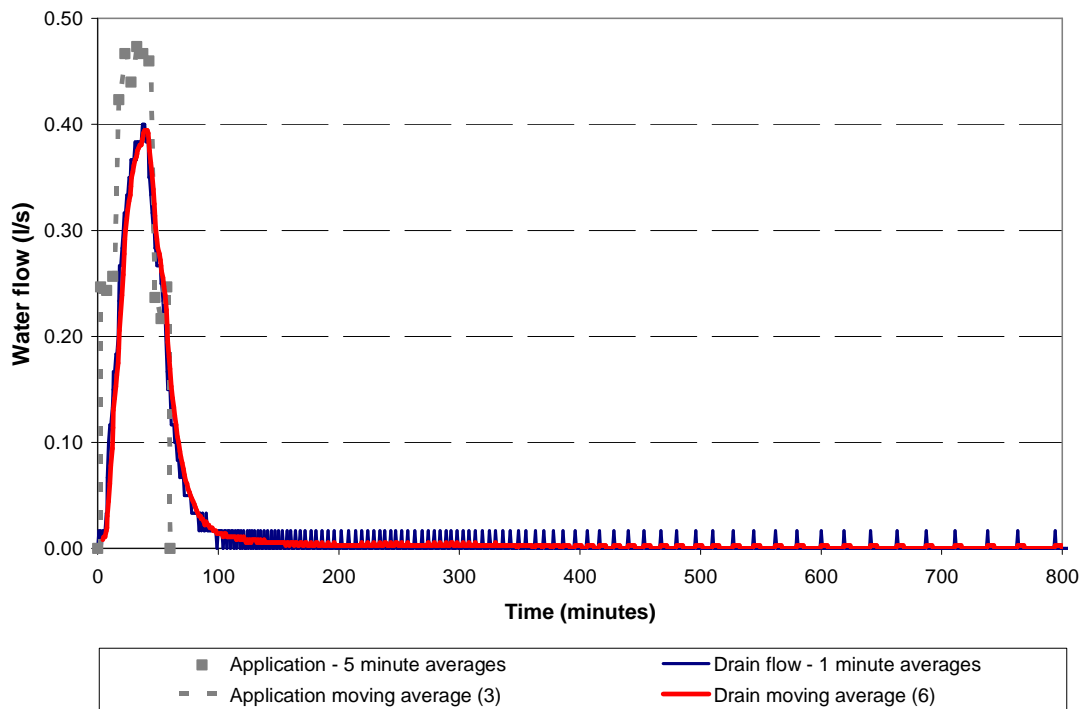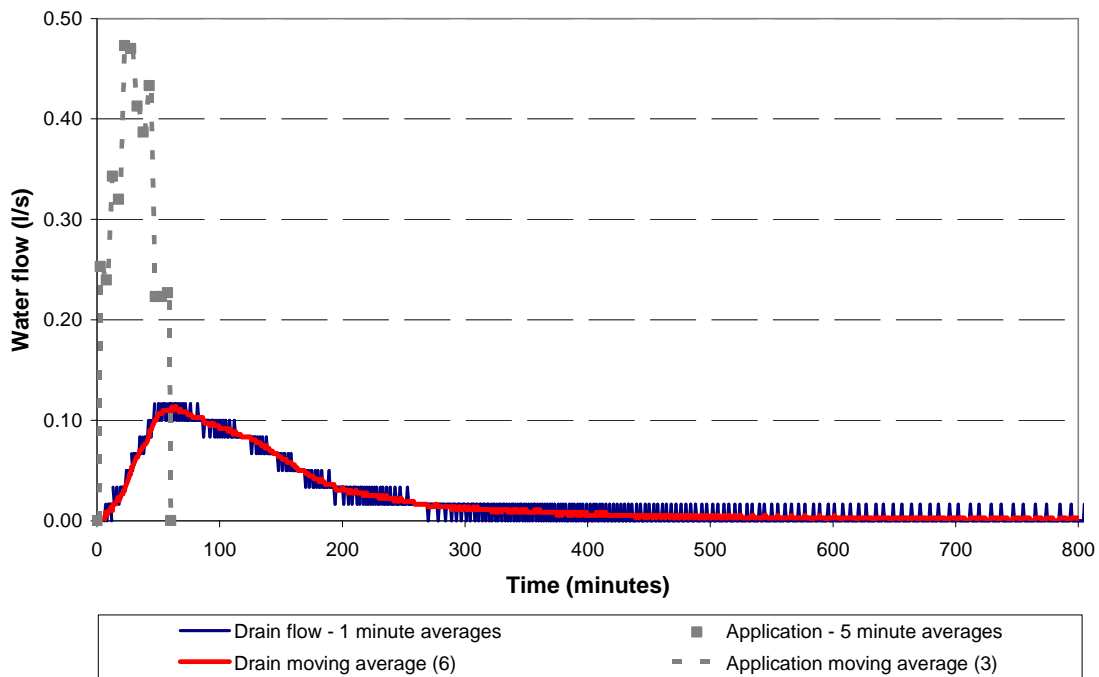**Figure 8.6 Variation in water flow with time for Bay 2 edge gullies: Unrestricted flow**

Figure 8.7 Variation in water flow with time for Bay 2 linear edge drain: Unrestricted flow

Figure 8.8 Variation in water flow with time for Bay 3 linear edge drain: Unrestricted flow
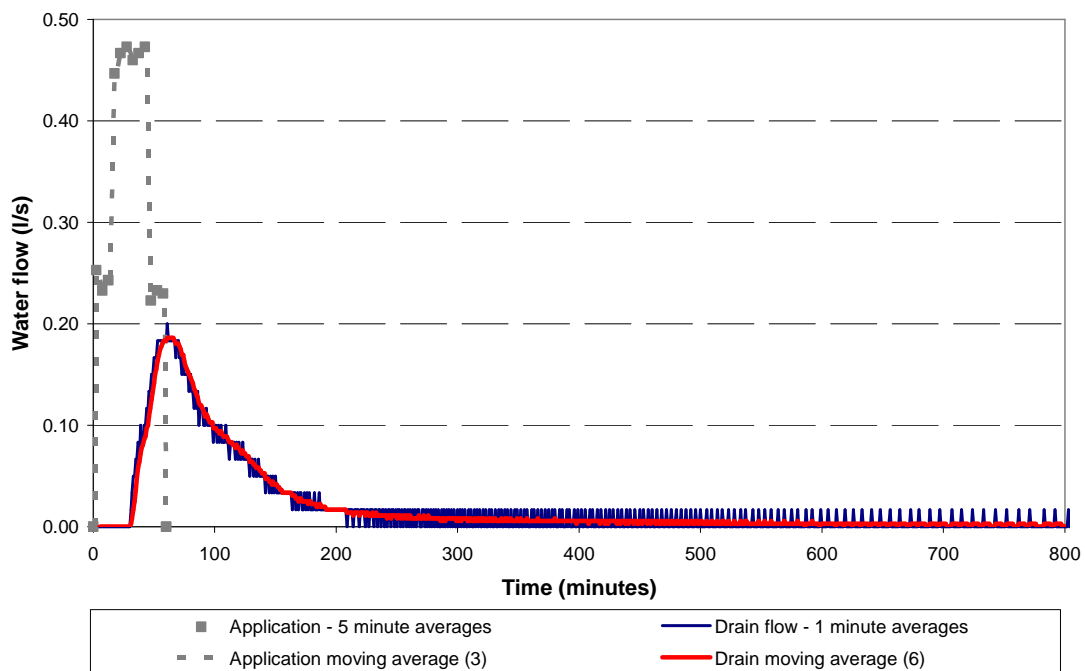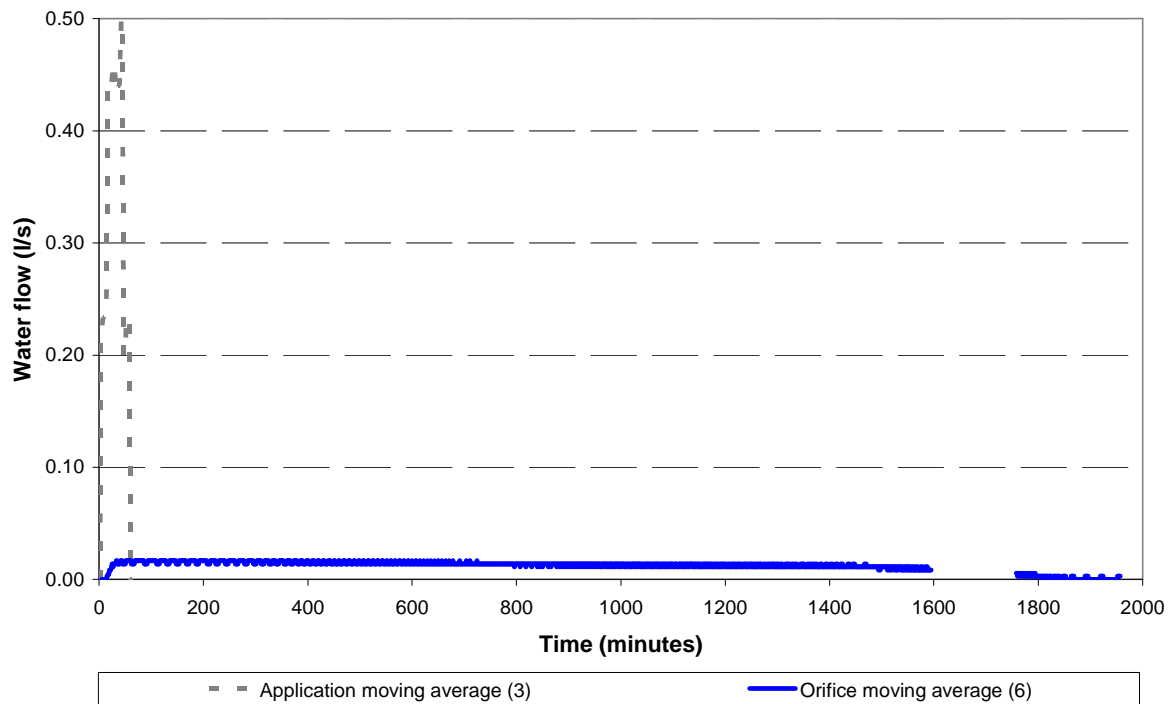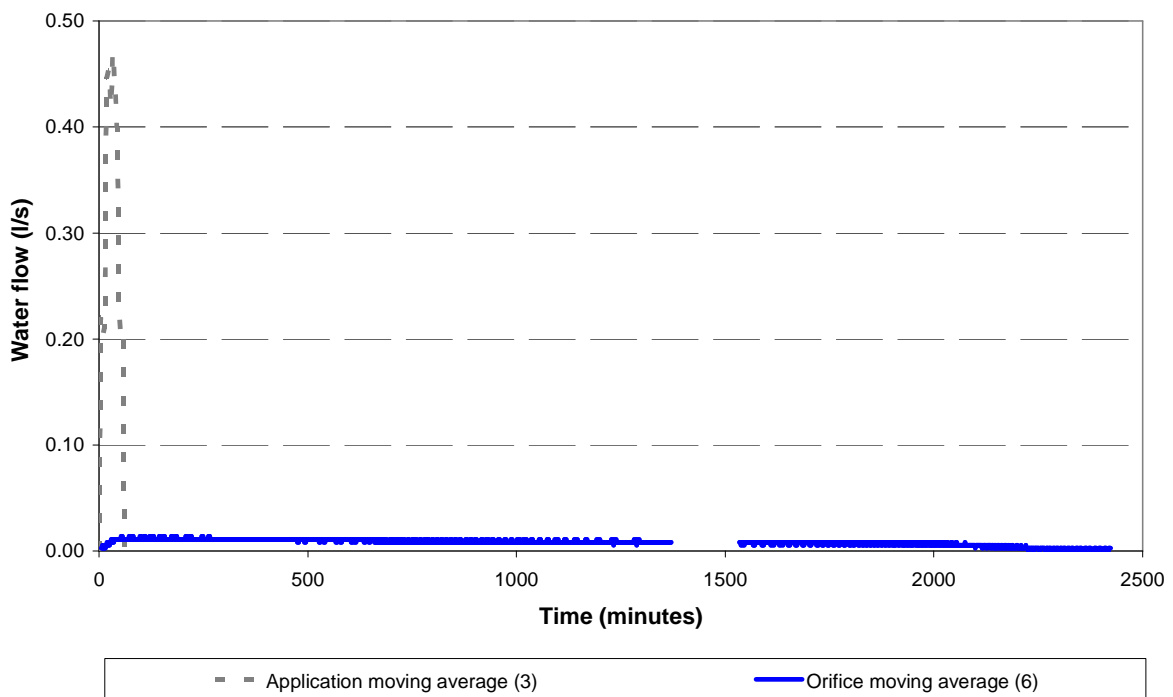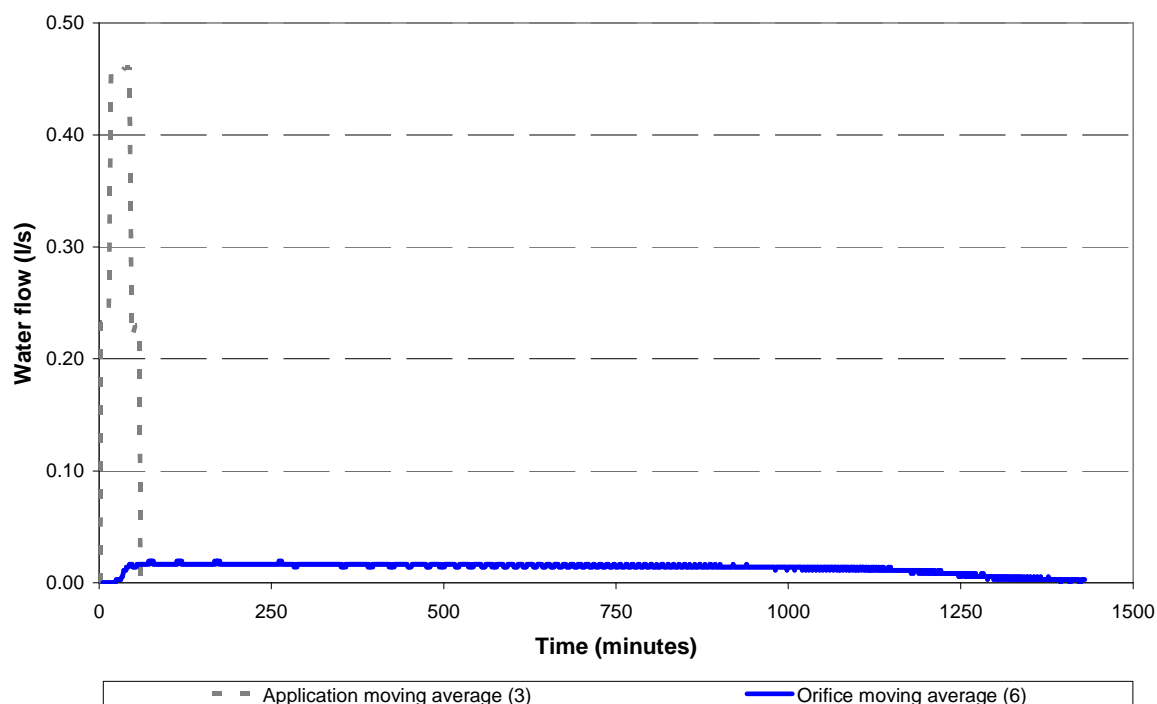

Figure 8.9 Variation in water flow with time for Bay 4 porous asphalt: Unrestricted flow

For the tests with the drain flows restricted by orifices, about 1210 litres of water were applied over 1 hour such that about 205, 405, 405 and 195 litres were used in successive quarter of an hour periods. The variations with time of the water flows from the subbase reservoirs and of the simulated storm intensities are compared in Figures

8.10, 8.11 and 8.12 for Bays 2, 3 and 4 respectively when orifice restrictors in the outfall drains are used. For clarity, the graphs shown are moving averages.

Figure 8.10 Variation in water flow with time for Bay 2 linear edge drain: Flow restricted by orifice

Figure 8.11 Variation in water flow with time for Bay 3 linear edge drain: Flow restricted by orifice

Figure 8.12 Variation in water flow with time for Bay 4 porous asphalt: Flow restricted by orifice

8.2 Analysis

The results of the hydraulic study were analysed to provide information on the following:

- Use of the subgrade as soakaway.
- Drainage attenuation of pavement runoff.
- Siltration of pervious surfaces.

8.2.1 Use of subgrade as soakaway

8.2.1.1 Behaviour of subgrade in trials

The permeable pavement of Bay 1 comprises a block paving surfacing on porous concrete on a subbase of linked Permavoid modules over the soil subgrade, with bedding aggregate for the block paving and reinforced filter geotextiles at selected locations. The purpose of the design is to permit the infiltration of rainfall into the pavement through the permeable surfacing and its flow through the pavement into the reservoir subbase, where it is then allowed to infiltrate into the subgrade with excess water passing through an outfall drain.

The initial commissioning tests, in which an estimated rainfall rate of 37.5 mm/h was applied for 50 minutes to the permeable surfacing of block paving, resulted in no flow of water from the pavement drain. All water is considered to have infiltrated into the subgrade without temporary storage in the Permavoid reservoir because no water was detected in the dipwells.

In the Series 2 tests, the standard storm was applied and the rates of water application were between 40 and 53 mm/h. There was no flow in the pavement drain with all water

considered to be absorbed by the soil. Very little water was recorded as being contained in the subbase of Permavoid modules as the maximum depth of water recorded at the low side water table depth tube was 3 mm and occurred at the end of the simulated rainfall event. These results are consistent with the initial tests. Tests with orifice restrictors were not performed because, with this standard storm, all water infiltrated into the soil and there was insufficient water to be held back by the restrictor orifice.

In the Series 3 tests, following partial cleaning of the block paving surfacing of Bay 1, a rainfall intensity of only 16 mm/h for one hour infiltrated through the pavement surface.

8.2.1.2   Infiltration tests on subgrade

The ability of water to infiltrate the subgrade was studied by the infiltration tests shown in Figures 8.13 and 8.14 for the as-found soil and the imported sandy gravel soil respectively.

Figure 8.13 Infiltration tests in as-found soil.

Figure 8.14 Infiltration tests in the imported soil.

The results of the infiltration tests for Bay 1 are given in Table 8.3

Table 8.3 Infiltration tests in soil of Bay 1

Soil:	Infiltration rate (mm/h):			
	Individual			Average
	NSWP LHS[1]	Between wheelpaths[1]	OSWP RHS[1]	
As-found	77.5	56.7	25.7	53.3
Imported	143.1	179.1	145.2	155.8

[1] Transverse centre line with orientation facing weighbridge

The average infiltration rate for the imported soil is higher than the as-found soil and therefore infiltrated water will drain away less easily in the as-found soil than in the imported soil. The average rates of infiltration of both soil types, however, exceed the average rainfall rate of 44mm/h from the simulated rainfall event. Lower intensities of water are expected to be presented to the soil as attenuation of the rainfall intensity as water infiltrates through the pavement is expected. Therefore no build-up of water is expected in the reservoir in this trial as demonstrated by the water depth tubes staying dry throughout.

The imported soil came from Uttoxeter quarry. Its nominal particle size distribution (psd) is given in Table 8.4 and the material is described as sandy gravel.

Table 8.4 Nominal particle size distribution of imported soil

	Percentage passing (%) various sieve sizes:															
Sieve size (mm)	0.063	0.25	0.5	1	2	2.8	4	5	6.3	8	10	12.5	14	16	20	25
Percentage passing	2	10	26	34	40	43	45	47	50	54	62	72	79	87	96	100

According to British Standards Institution (1986), the typical permeability of clean sand-gravel mixtures can be as low as about $5x10^{-2}$ m/s or approximately 180 mm/h. This estimate is broadly similar to the measured average infiltration rate of about 160 mm/h.

Although the rough guides of British Standards Institution (1986) and other references such as Interpave (2010) are useful in indicating the likely permeability of a particular soil, direct measurements of water infiltration rates are advised to reduce uncertainty in design.

8.2.1.3 *Variations of moisture content of soil subgrade with rainfall events*

The effect of water infiltrating into the soil was assessed from measurements by moisture content gauges buried in the top of the imported soil. Theta probes (Miller and Gaskin[1]) were used to measure the soil moisture content. The probes were interrogated by a Campbell CR1000 data logger which sampled their voltage output every minute and then recorded an average for each 30 minute interval. Formulae were then used to convert the probe voltage output to moisture content [2].

The changes in soil moisture content with time due to water infiltrating in Bay 1 are shown in Figures 8.15 and 8.16 for periods of time covering the initial commissioning tests and the second series of hydraulic tests. Natural rainfall is also recorded in these figures as daily rainfall depths. Numerous significant water infiltration events caused by percolating water from natural rainfall and simulated rainfall on the pavement surface were observed. Infiltration events caused by natural rainfall can be identified by observation of peaks in the daily rainfall, whereas events due to simulated rainfall are identified by the trial dates recorded on these figures.

Footnotes:
[1] **J D Miller and G J Gaskin, ThetaProbe ML2x Principles of operation and applications**
MLURI Technical Note (2nd ed) http://www.macaulay.ac.uk/MRCS/pdf/tprobe.pdf

[2] The formulae used to convert the millivolt (mV) output (X) from the probe into percentage water content (Y) were derived from the lookup table in the Thetaprobe manual (ML2x-UM-1.21 page 13[3]). Three equations are implemented over the full range of the probe as the output from the probe becomes progressively more non-linear at both extremes of the probe range. Due to the range of data measured in this trial, however, only the first equation was actually used. The equations are:

$Y = 5.020x10^{-8}X^3 - 6.546x10^{-5}X^2 + 7.056x10^{-2}X - 5.683$
($R^2 = 0.9998$) for the range 0-40%.

$Y = 1.488x10^{-5}X^3 - 4.196x10^{-2}X^2 + 3.948x10^{+1}X - 1.234x10^{+4}$
($R^2=1$) for the range 40-55%.

$Y = -8.257x10^{-5}X^3 + 2.671x10^{-1}X^2 - 2.874x10^{+2}X + 1.029x10^{+5}$
($R^2=0.997$) for the range 55-100%.

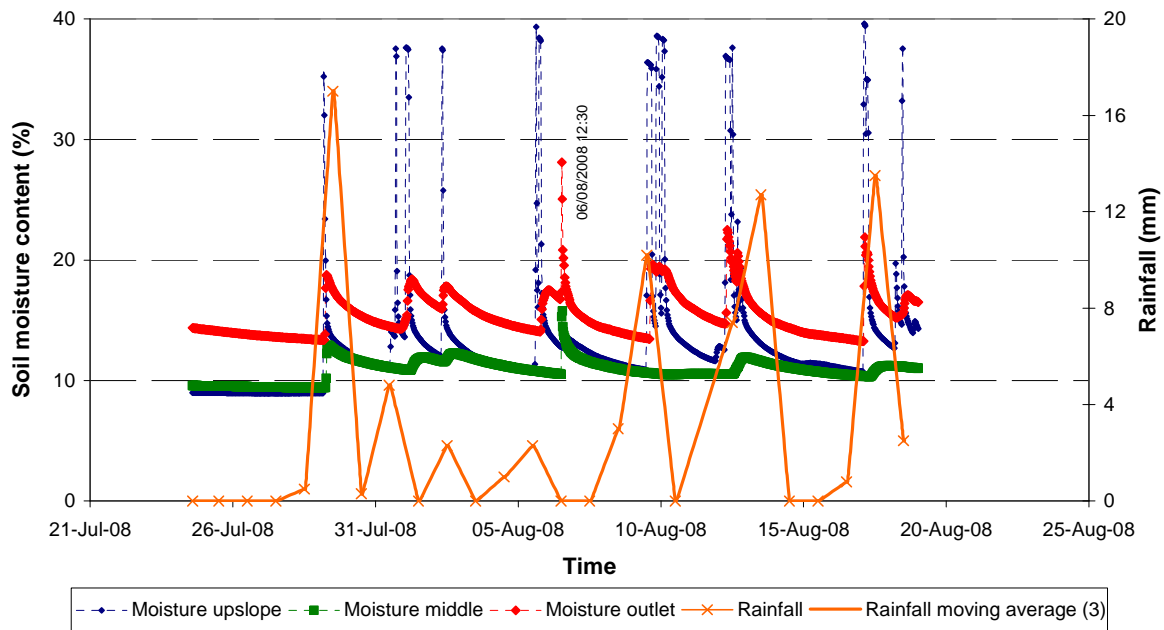[3] http://www.delta-t.co.uk/cgi-bin/attach.cgi?item=faq2005100703502.2

Figure 8.15 Variation with time of moisture content of soil in Bay 1: 24/7/2008 to 19/8/2008
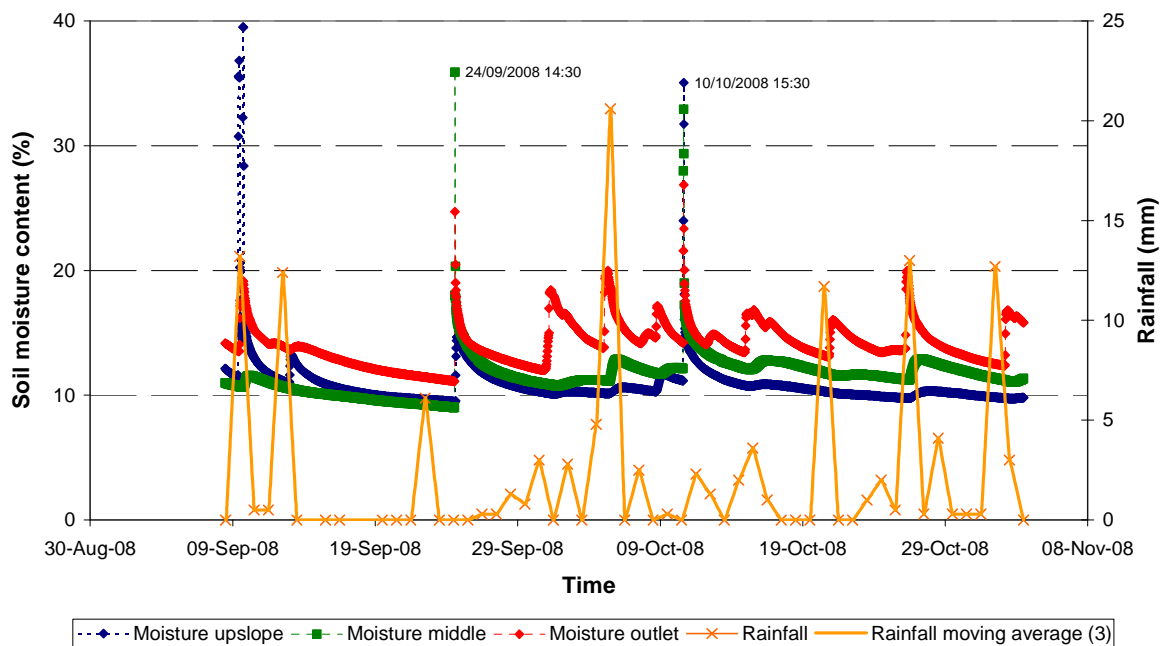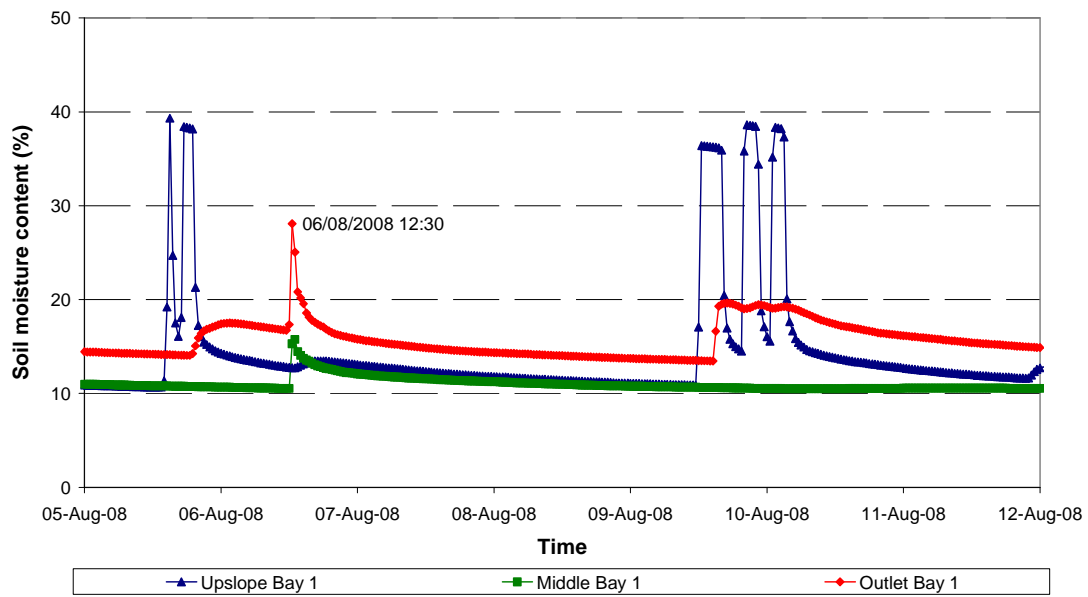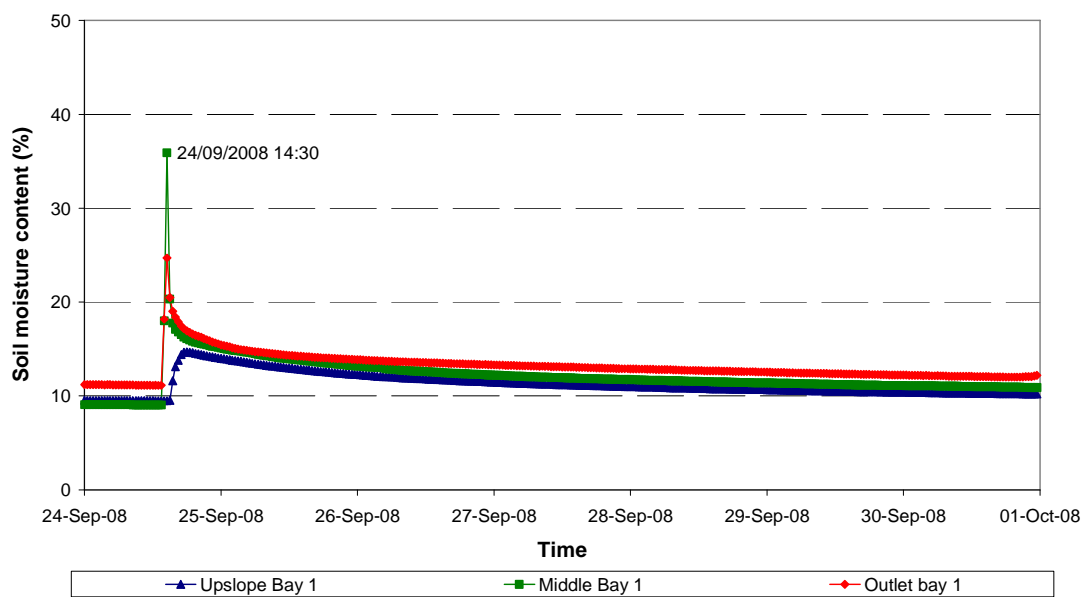

Figure 8.16 Variation with time of moisture content of soil in Bay 1: 8/9/2008 to 3/11/2008
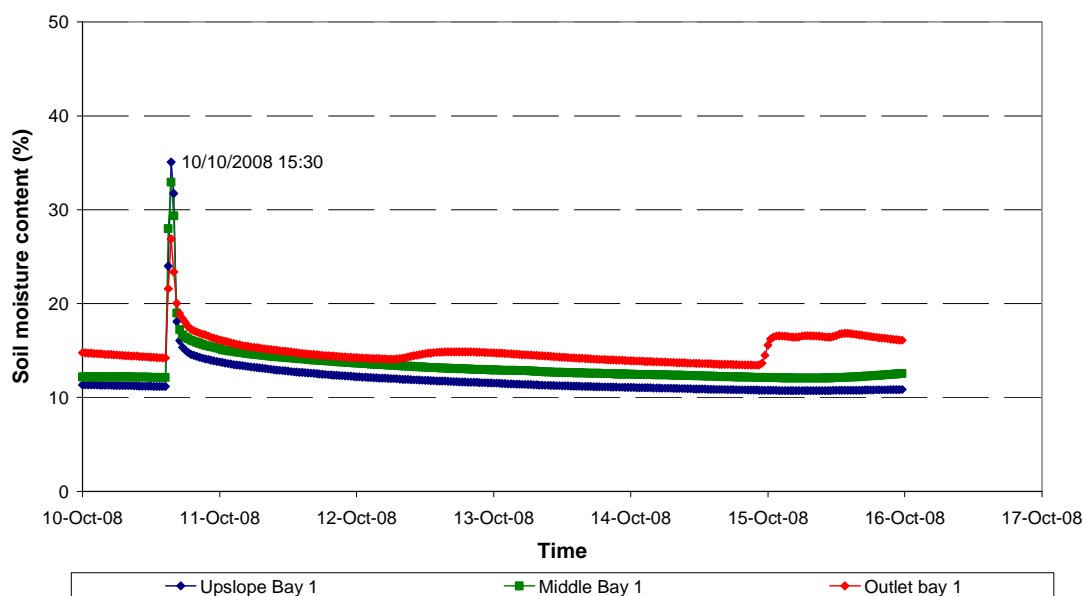
The change in soil moisture content with time for three simulated rainfall events are shown in more detail in Figures 8.17, 8.18 and 8.19, where a rapid increase in moisture content was followed by a gradual drying back of the soil over 3 or more days towards those values that were measured just prior to the soil wetting.

Figure 8.17 Variation with time of moisture content in soil in Bay 1: 5/8/2008 to 11/8/2008

Figure 8.18 Variation with time of moisture content of soil in Bay 1: 24/9/2008 to 30/9/2008

Figure 8.19 Variation with time of moisture content of soil in Bay 1: 10/10/2008 to 15/10/2008

The changes in the extent of the wetted soil in Bay 1 are shown by comparing the Figures 8.17, 8.18 and 8.19. During the tests shown in these figures, about 840 litres, 1270 litres and 1810 litres were applied during tests on 6/8/2008, 24/9/2008 and 10/10/2008 respectively. These tests comprised vertical rainfall only in the first two tests with both vertical rainfall and linear water flow from the top edge of the pavement in the last test to simulate water flow from adjacent impermeable pavements. For this last test, a total vertical rainfall of 1215 litres over one hour was augmented by an edge flow of about 595 litres in this time to give a total application of 1810 litres in an hour. In all cases, the response of the soil moisture content gauge near the outlet drain from the pavement was significant and similar in each case. For the middle gauge part way up the sloped subgrade, a modest response to the lowest volume of water applied in the first test changed to a marked response in the latter two cases. For the gauge at the top of the sloped subgrade, moisture content changes were minor and then modest for the first two tests and only became marked in the final test when water was applied to the top edge of the pavement in addition to vertical rainfall. The extent of the subgrade wetted by infiltrated water therefore increased throughout this sequence of tests. The measured behaviour is consistent with changes in the amount of water and its application method over this group of tests. The observations also imply that the water was, at the time of the tests, able to infiltrate into the block paving over the whole of its surface. Less obvious, but feasible, is the possibility of water migrating within the pavement to its low side.

Figures 8.20, 8.21 and 8.22 show three time periods within about 17 months in which soil moisture in Bay 1 was measured. Daily rainfall is also recorded. It can be seen that similar intensity rainfall events occurred in these three periods. But the soil moisture response of all transducers reduced with time such that after 18 months from opening of the road, there was very little increase in soil moisture content with rainfall on the pavement surface. This behaviour implies that the surface water was not infiltrating through the pavement, presumably due to clogging of the joints in the block paved surface. Surface siltration of pervious surfaces is discussed in more detail in Section 8.2.3.
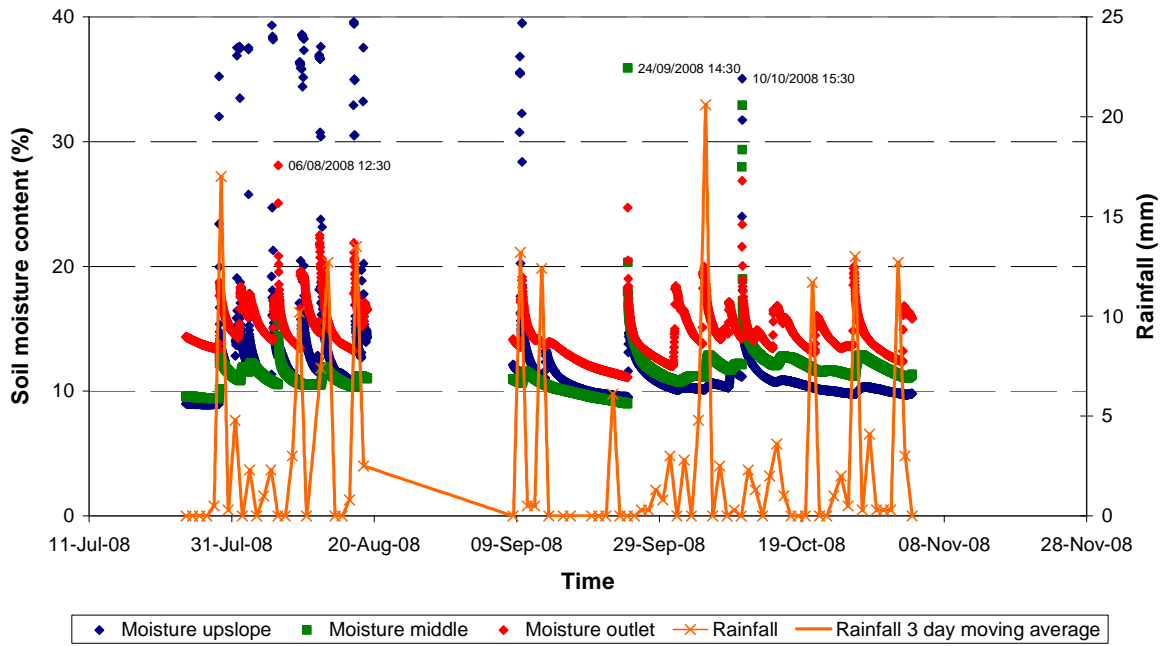
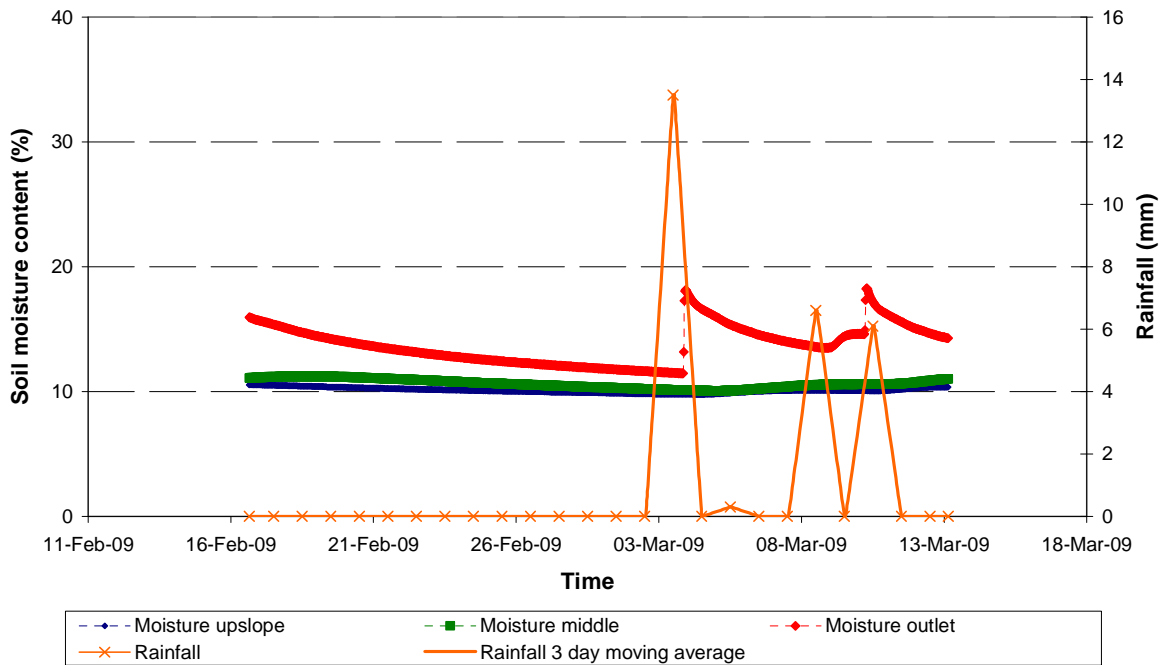**Figure 8.20 Soil moisture content response to rainfall after about 4 months use of Bay 1**

Figure 8.21 Soil moisture content response to rainfall after about 9 months use of Bay 1
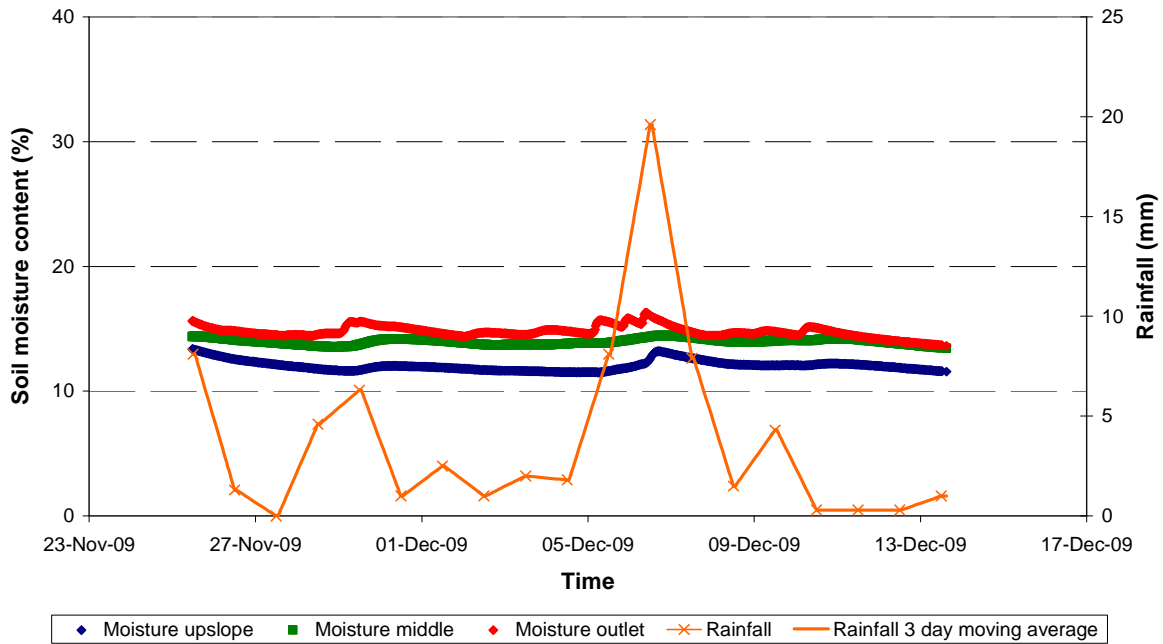
Figure 8.22 Soil moisture content response to rainfall after about 18 months use of Bay 1

The block paved surfacing of Bay 1 was cleaned to a limited extent by the Osprey 3000 equipment shown in Figure 8.27. As stated in Section 8.2.1.1, an average rainfall intensity of 16 mm/h for one hour infiltrated through the surfacing. Figure 8.23 shows that the soil moisture content measured by the outlet gauge increased slightly, which demonstrated that the infiltrated water once again wetted the soil subgrade. The middle and upslope gauges showed no response to this reduced rainfall event.
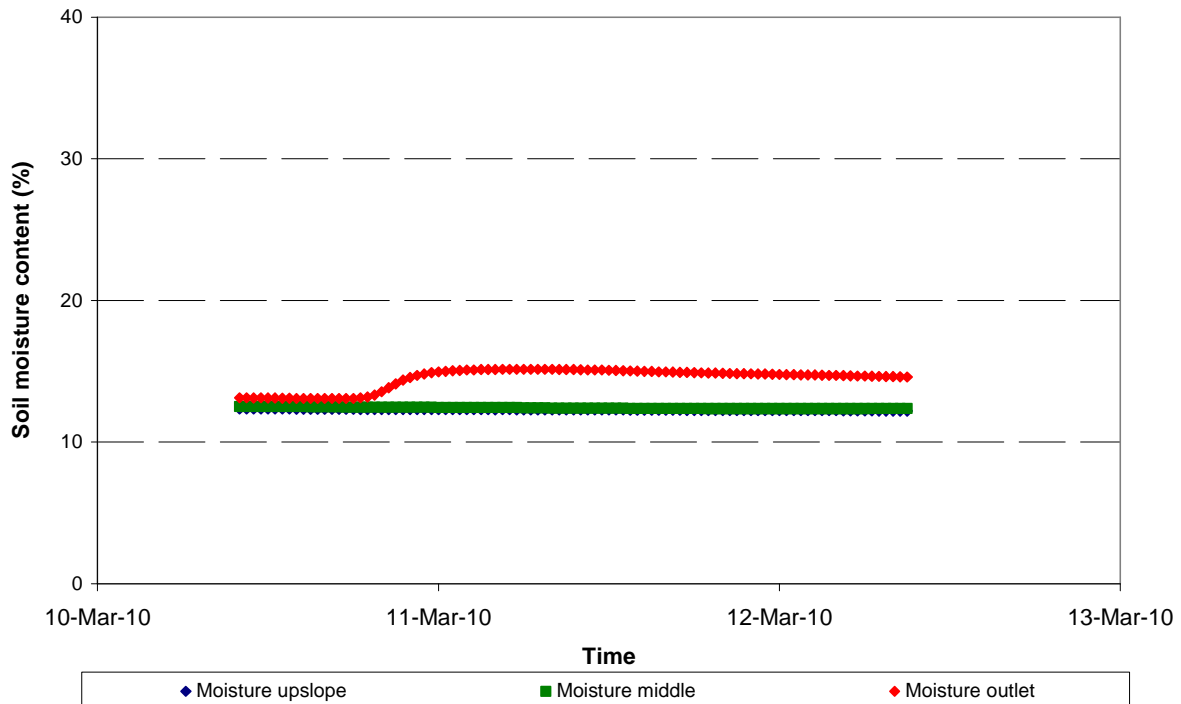

Figure 8.23 Soil moisture content response to rainfall after cleaning Bay 1

8.2.2 Drainage attenuation of pavement runoff

8.2.2.1 Natural attenuation

Reservoir pavements can be designed to reduce the flow of water to drains and also to delay the peak flows. The natural attenuation caused by pavement materials, associated connecting pipes, etc. of various designs of reservoir pavements was investigated by hydraulic testing of the pilot-scale trial pavements. Also, the use of orifice restrictors in the pavement drains was demonstrated.

The reductions in peak flow and the delay of the peak flow for these various pavements are given in Table 8.5 together with the proportion of the water applied that was recovered. Assessments of the reduction in peak flow were not recorded in this table when significant drained water leaked away before measurement.

Table 8.5 Natural attenuation of rainfall by various pavement designs

Bay	Test series	Pavement type	Water flow	Water recovered	Peak flow[1] reduction factor	Delay in peak flow[2] (mins)
2G	2	"Impermeable" asphalt on porous concrete on tanked, Permavoid modular subbase	Pavement run-off to gullies (G) at edge of pavement to outfall drain	97% at 936 mins.	x0.76	13.0
2G	3			92% at 1440 mins.	x0.77	11.5
				Mean:	**x0.76**	**12**
2LD	2	"Impermeable" asphalt on porous concrete on tanked, Permavoid modular subbase	Pavement run-off to linear edge drain (LD) to subbase reservoir to outfall drain	76% at 637 mins.	See text [3]	16.0
2LD	3			103% at 1440 mins.	x0.85	15.5
				Mean:	**x0.85**	**16**
3LD	3	"Impermeable" asphalt on porous concrete on tanked, porous granular subbase	Pavement run-off to linear edge drain (LD) to subbase reservoir to outfall drain	92% at 1440 mins.	**x0.25**	**35**
4	2	Porous asphalt on porous concrete on tanked, porous granular subbase	Water infiltration through pavement to subbase reservoir to outfall drain	66% at 248 mins.	See text[4]	31
4	3			83% at 1414 mins.	x0.42	31
					x0.42	**31**

Key: [1] $\frac{\text{(Peak flow in drain (l/s))}}{\text{(Peak rainfall (l/s))}} *100$ [2] (Time for peak drain flow – 30) minutes

The reference conditions were set by the traditional drainage by edge of pavement gullies of Bay 2. Peak flow was only delayed in this pilot-scale facility by about 12 minutes with peak flow in the drain as 76% of the peak intensity of the rainfall event.

When water drained to the edge of pavement linear drain of Bay 2 and then flowed via the tanked Permavoid subbase reservoir to the outfall drain, the peak flow was delayed 16 minutes; that is, only a further 4 minutes over direct flow to the outfall drain via the gullies. Peak drain flow[3] was 85% of the peak intensity of the rainfall event. Hence, the two drainage variations of Bay 2 behaved in a very similar manner.

Bay 3 differed from Bay 2 in the use of a tanked unbound granular subbase instead of tanked Permavoid modular subbase. The peak flow was delayed by 35 minutes and peak flow in the drain was 25% of the peak intensity of the rainfall event. The differences between the water flows recorded in Bay 3 compared to Bay 2 are consistent with the replacement of the Permavoid modules of 95% porosity by unbound granular material of porosity about 30% and a more tortuous flow path with variations in the size of the flow channels.

Flow of water through a porous pavement into a tanked porous unbound granular subbase reservoir and then to the outfall measurement chamber was investigated in Bay 4. The peak flow was delayed by 31 minutes and peak flow in the drain[4] was 42% of the peak intensity of the rainfall event.

8.2.2.2   *Attenuation by restrictor orifices*

The maximum natural attenuation of the trial pavements occurred in Bay 3 and reduced the peak drain flow to 25% of the peak intensity of the rainfall event. Also, this peak flow was delayed by approximately a further 20 minutes over and above the delay that occurred with direct drainage by gullies in Bay 2. Although this modification of water flow is advantageous, restrictors in drains could further reduce peak water flow and lengthen drainage time to more effectively mitigate against flooding incidents. The effect of orifice restrictors in the outfall drain is shown in Figures 8.10, 8.11 and 8.12, which record the hydraulic behaviour of Bays 2, 3 and 4 respectively under the standard rainfall event. The orifices were circular and of diameter 4 mm for Bay 2 and 5 mm for Bays 3 and 4. Larger orifice sizes would be used in typical installations, where the catchments are larger than the small pavement areas investigated in these pilot-scale trials.

In these trials, the 1 hour rainfall event is transformed after its peak intensity is reached at 30 minutes by an orifice in the outfall drain to a low, steadily declining flow. The reductions in peak flow and the delays to the peak flows for the three reservoir types in Bays 2, 3 and 4 are recorded in Table 8.6, together with the proportion of the applied water applied that was recovered.

Footnotes:

[3] For the earlier Series two test, peak flow in the drain was at least 79% of the peak intensity of the rainfall event but was not recorded in Table 8.5 as this value may underestimate the actual value due to the leakage in connecting pipes to the measurement chamber of about one quarter of all water applied to the pavement surface.

[4] For the earlier Series two test, peak flow in the drain was at least 61% of the peak intensity of the rainfall event but was not recorded in Table 8.5 for the same reason as Note 3 because of the leakage of about one third of all water applied to the pavement surface.

Table 8.6: Attenuation of rainfall by various reservoir pavement designs fitted with orifice restrictors

Bay	Series	Pavement type	Water flow	Water recovered[3]	Peak flow[1] reduction factor	Delay in "peak flow" [2] (minutes)
2LD	3	"Impermeable" asphalt on porous concrete on tanked, Permavoid modular subbase	Pavement run-off to linear edge drain (LD) to subbase reservoir to outfall drain	**106%** at 1957 minutes	**x0.033**	**658**
3LD	3	"Impermeable" asphalt on porous concrete on tanked, porous granular subbase	Pavement run-off to linear edge drain (LD) to subbase reservoir to outfall drain	**96%** at 2423 minutes	**x0.035**	**904**
4	3	Porous asphalt on porous concrete on tanked, porous granular subbase	Water infiltration through pavement to subbase reservoir to outfall drain	**97%** at 4651 minutes	**x0.072**	**638**

Key: [1] (Peak flow in drain (l/s) *100
      (Peak rainfall (l/s)

[2] (Time for 50 % recovery of applied rainfall – 30 minutes)

[3] (Total flow in drain (l/s) *100 - corrected for natural rainfall for the 24 hours after rainfall simulation.
    (Total rainfall (l/s)

The peak flow was reduced substantially to 3.3%, 3.5% and 7.2% of its applied value for the Bays 2, 3 and 4 respectively.

Since the water flow after restriction by the orifice has a steadily declining flow over a long period, it is appropriate to redefine the time to peak flow to be the 50 percentile time or the time at which 50% of the applied rainfall event has flowed to the outfall. For Bays 2, 3 and 4, 50% of the rainfall drained in 688, 934 and 668 minutes respectively. The delays to the peak of the applied 60 minute rainfall event were then calculated as these 50 percentile times less 30 minutes and are given in Table 8.6. The delays were long at 658, 904 and 638 minutes for Bays 2, 3 and 4 respectively; that is, at least 10 hours.

The degree of recovery of water from the simulated rainfall events was over 100% for Bays 2 and 3 due to natural rainfall in the measurement period. After correction for natural rainfall, albeit only for rain that fell in the 24 hours following the rainfall simulation, the recovery of water from the simulated rainfall event fell to 106% and 96% for Bays 2 and 3 respectively. For Bay 4, a high value of 97% was recovered after rainfall correction during the recording time of 4651 minutes. These high values

demonstrated that the pavements and connecting drainage pipes to the measurement chamber are now very well sealed against leakage of water.

The tests on Bays 2, 3 and 4 are summarised in Figures 8.24, 8.25 and 8.26 respectively where the simulated rainfall events are compared with the water flows drained when the outfall pipes were either fully open or fitted with orifice restrictors.
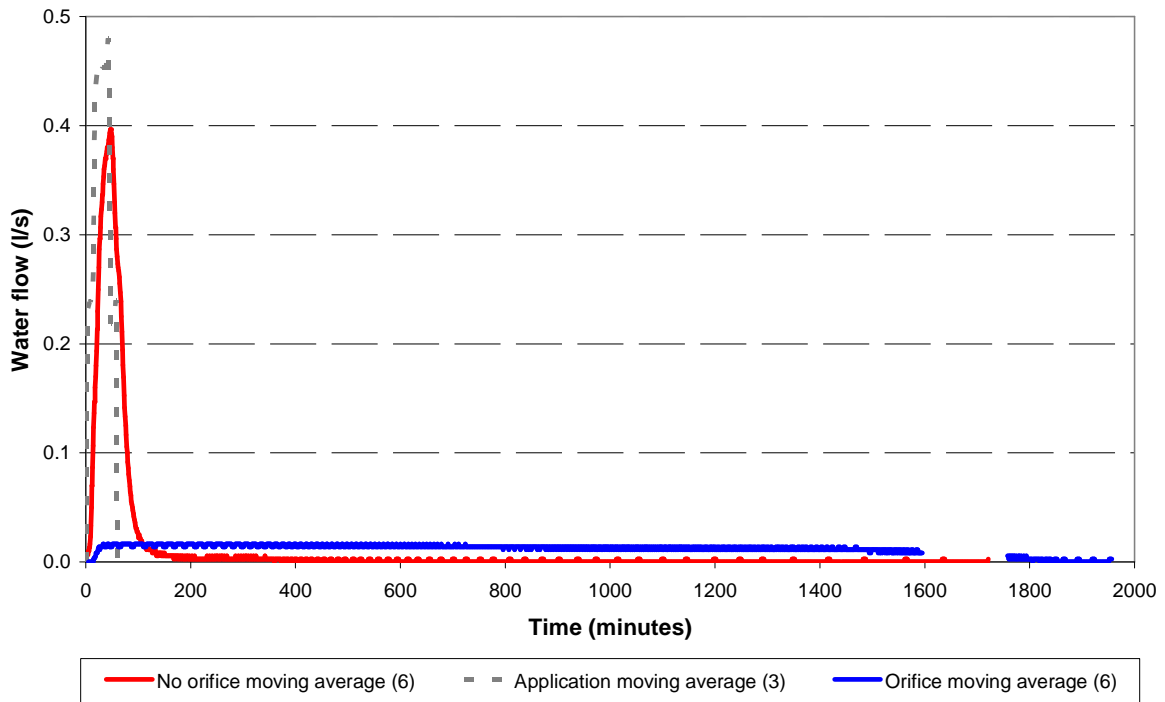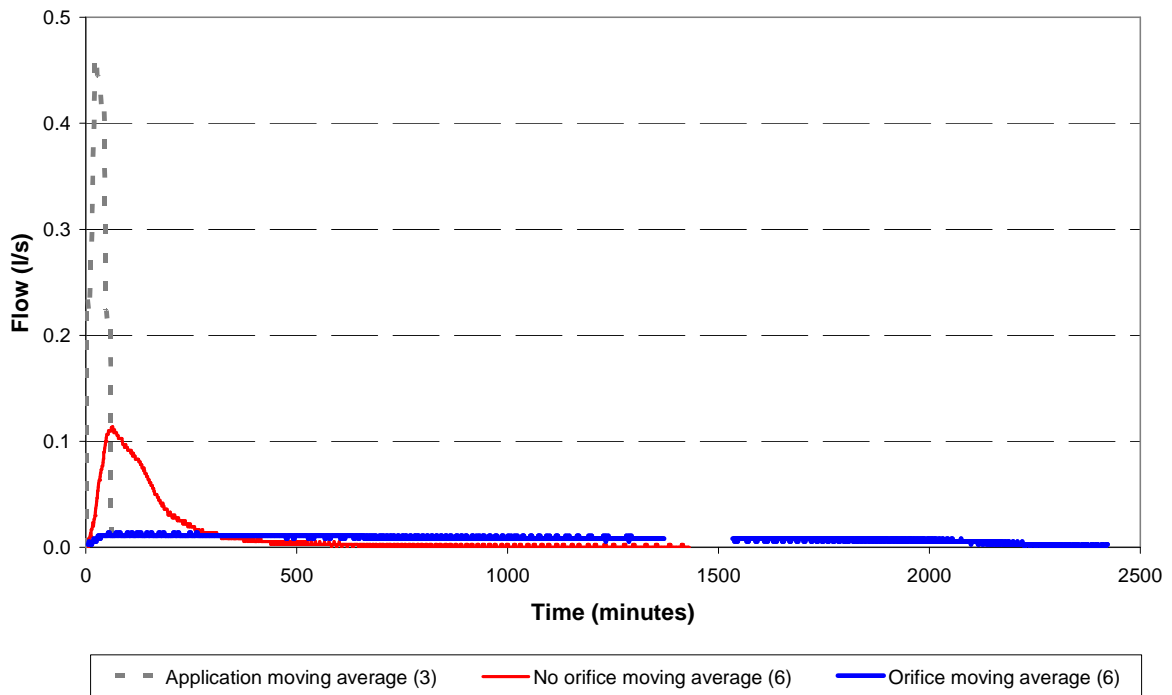

Figure 8.24 Variation in water flow with time for Bay 2

Figure 8.25 Variation in water flow with time for Bay 3
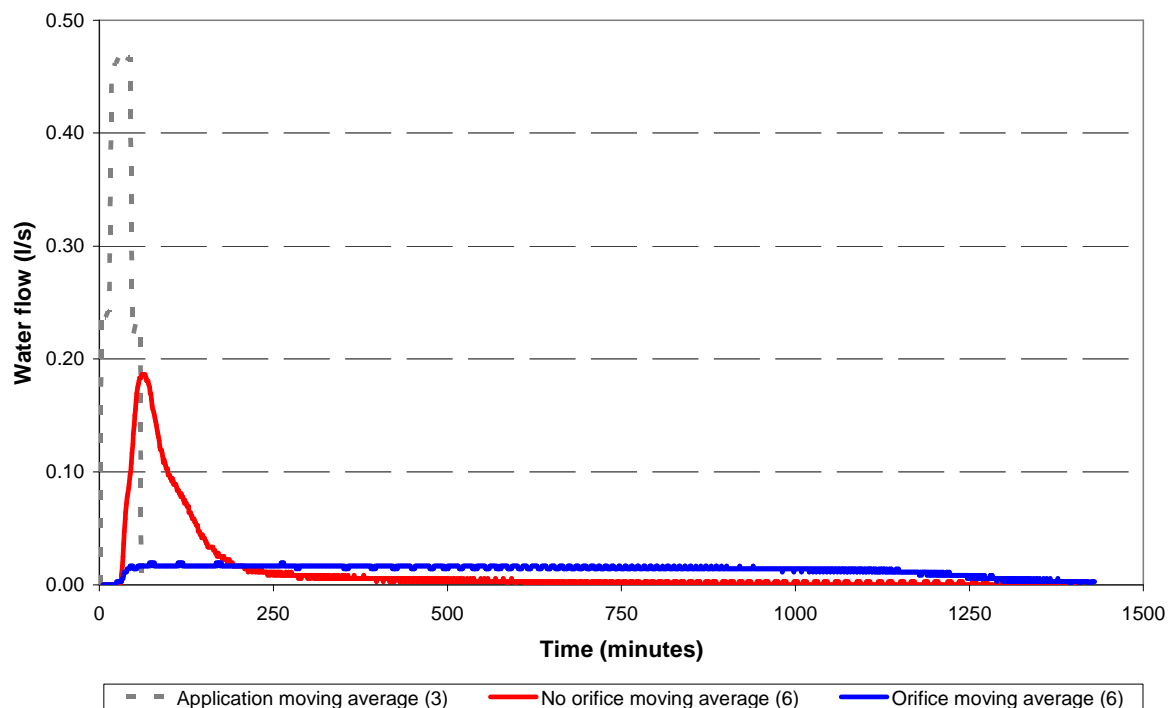
Figure 8.26 Variation in water flow with time for Bay 4

The orifice size can be adjusted to regulate flow and, in a specific development, would be chosen to limit drainage to permitted values. The behaviours of these reservoir pavements fitted with orifice restrictors demonstrate the ability of reservoir pavements to markedly reduce flow rates and delay flow to drains.

8.2.3   Siltration of pervious surfaces

The trial was opened to traffic on the 16th June 2008. Soon after construction of the trials, the initial hydraulic tests showed that both the block paving and porous asphalt surfaces of Bays 1 and 4 had a minimum infiltration capacity of 37 mm/h because no run-off was observed. Several months later, during the second series of tests, a minimum infiltration capacity of 53 mm/h was similarly deduced. The infiltration capacity of these pavements at these times may have been higher but, in both of these cases, higher intensity rainfall events were not simulated to find the maximum rainfall intensity to cause runoff.

Pervious surfacings comprising block paving or porous asphalt, however, can become clogged over time by detritus that reduces their infiltration capacity. Almost a year after opening of the road, the block paving surface and the porous asphalt surface (PAS-A) of Bay 4 and an adjacent porous asphalt surface (PAS-B), which had an identical pavement structure, appeared silted. The pavements were built on a factory site and are subject to the movements of lorries loaded with aggregates and travelling at low speed. Consequently there was a high probability of the deposition of fine material that can clog the porous surfaces without an element of self cleaning that occurs with high speed traffic. Under the conditions of the trials, it was expected that clogging would have been accelerated.

Changes in the permeability to water of the porous asphalt surfaces PAS-A and PAS-B were investigated by carrying out relative hydraulic conductivity (RHC) tests according to

a Draft for Development of the British Standards Institution (1996). This test wasn't considered suitable for the block paving surface.

RHC tests were initially carried out on the porous asphalt surfacing almost a year after opening of the trial. After a further 5 months of use, various methods of cleaning these surfaces were assessed. The bays were cleaned with a road sweeper that only sprayed water and applied suction to remove the debris and did not use the sweeping function. Later a hand held lance of a pressure washer was used to dislodge sediment. Finally, after several more months of use of the trial by site traffic, specialist equipment shown in Figure 8.27 was used to clean the porous surfaces. This machine is an Osprey 3000 and it is often used to remove rubber deposits from airfield runways. It uses water jets of pressure up to 3000 bar that are delivered to the contaminated pavement surface by a cleaning head that both rotates and oscillates to and fro. The jetted water and recovered deposits are conveyed by a pneumatic system to an onboard debris storage tank prior to disposal at a licensed land fill site. RHC tests were conducted before and after each cleaning attempt.

The block paving surface was also cleaned in a similar manner although its RHC is not reported.

Figure 8.27: Specialist cleaning of trial bays

To investigate the effectiveness of cleaning of these particular porous asphalt surfaces by the Osprey 3000 equipment, an initial equipment set-up was adopted and several passes of the device were carried out with intermediate measurements of RHC. A comparison in the appearance of the cleaned and uncleaned asphalt surfacing after one cleaning pass is shown in Figure 8.28.

Figure 8.28: Specialist cleaning of porous asphalt

The changes in RHC with number of passes are shown in Figures 8.29 and 8.30 for the asphalt surfaces PAS-A and PAS-B respectively. Tests were conducted in the nearside wheelpath (NSWP) and offside wheelpath (OSWP) and also outside wheelpaths (WP).
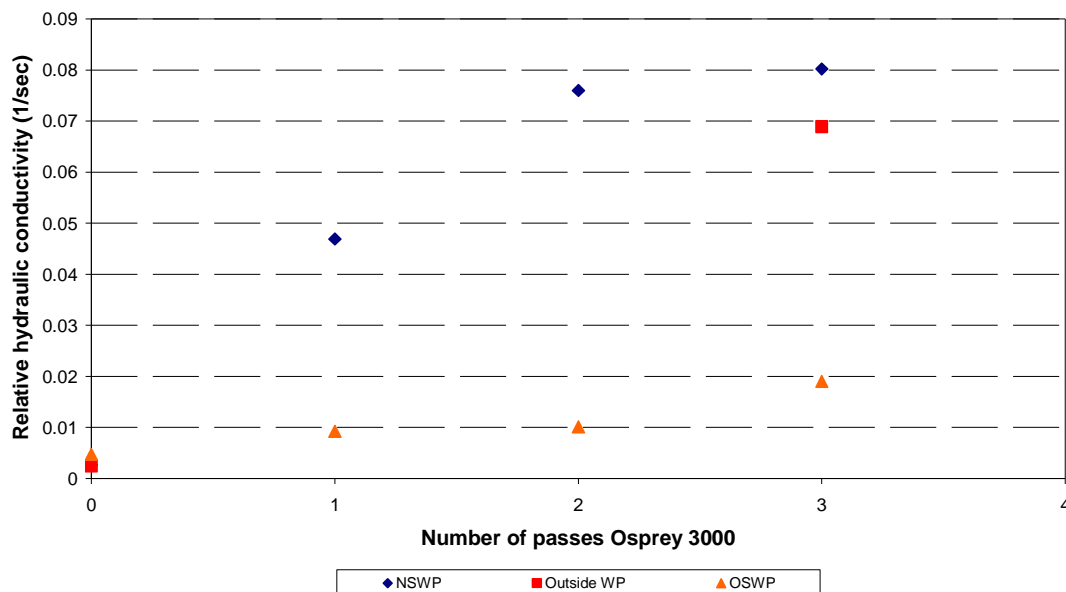

Figure 8.29: Variation of RHC with number of passes of Osprey 3000 for porous asphalt surface PAS-A
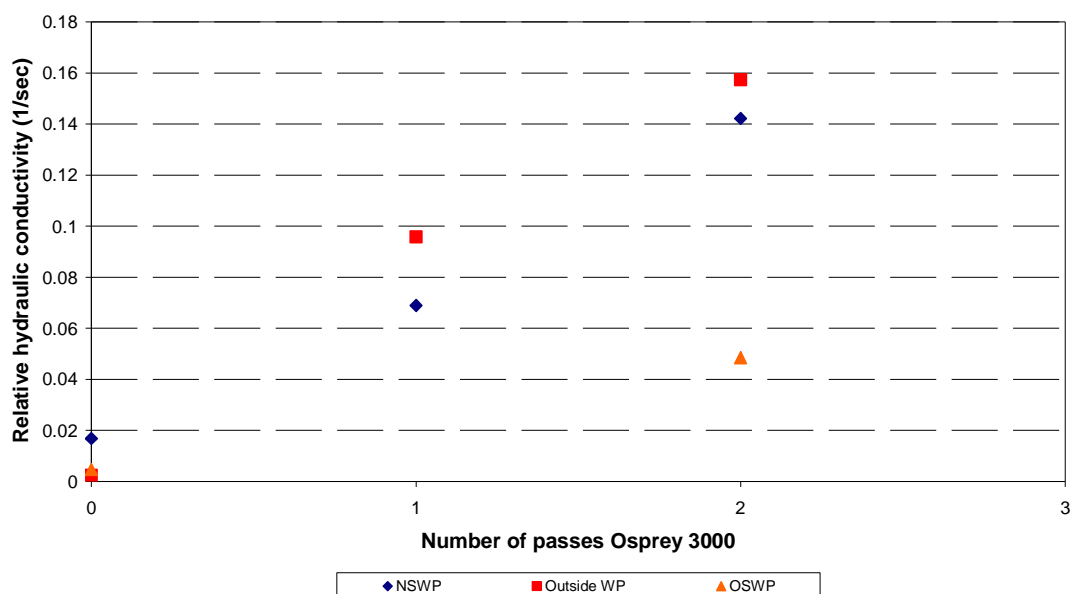
Figure 8.30: Variation of RHC with number of passes of Osprey 3000 for porous asphalt surface PAS-B

Figures 8.29 and 8.30 show continued improvement in the RHC of the porous asphalt surfaces with the number of passes of the Osprey 3000 equipment. With the initial equipment set-up, there is an indication from the NSWP tests on PAS-A that there is minimal improvement after two passes. The operators considered that, with adjustment of the equipment set-up, it was likely that the porous asphalt could be cleaned sufficiently well in one pass.

The effectiveness of each of the various cleaning treatments is shown in Figures 8.31 and 8.32 where RHC in the wheelpaths is plotted against time for the porous asphalt surfaces PAS-A and PAS-B respectively. The individual measurements and their average values are plotted as open and solid symbols respectively. The results for the Osprey 3000 device are the final RHC values after its use on a particular pavement was concluded.
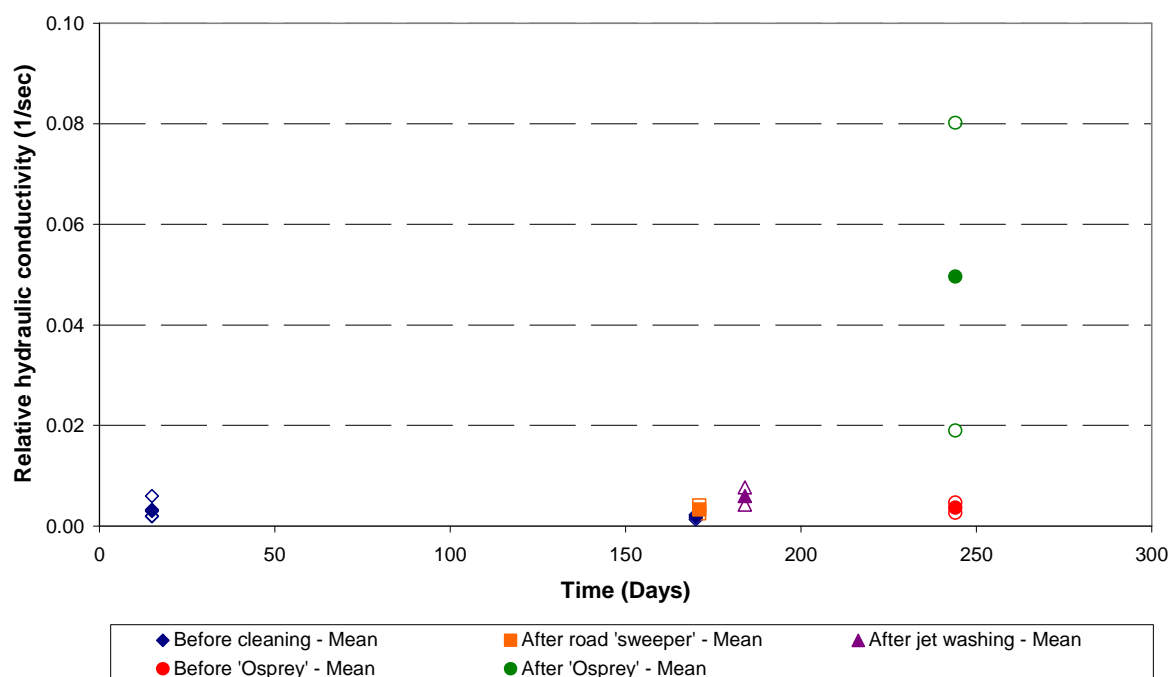
Figure 8.31: Variation of RHC with time and cleaning procedures in the wheelpaths of porous asphalt PAS-A
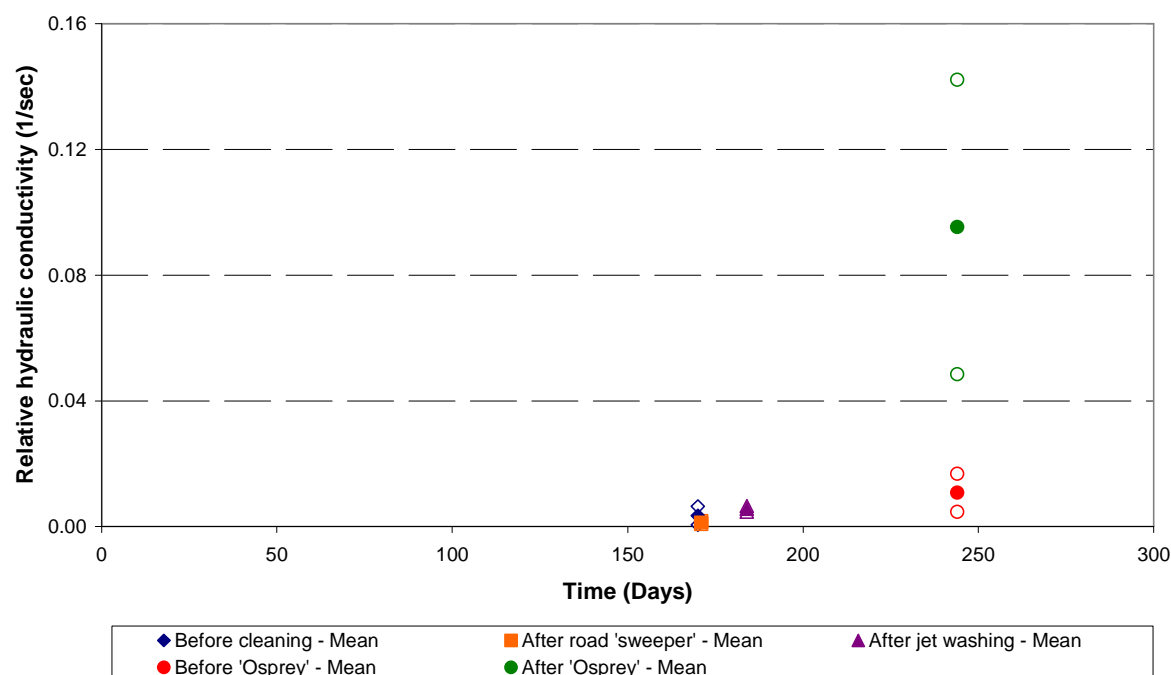

Figure 8.32: Variation of RHC with time and cleaning procedures in the wheelpaths of porous asphalt PAS-B

The road sweeper when used in a water jetting and suction mode and the hand water jetting were almost always shown by the RHC measurements to increase the hydraulic conductivity, but only by modest amounts. The specialist cleaning by the Osprey 3000 equipment, however, showed a marked improvement. Table 8.7 gives the increases in

mean RHC values based on the latest RHC value measured before the treatment. The Osprey 3000 increased the relative hydraulic conductivity by about 10 times. The equipment also works more rapidly than the other equipment.

Table 8.7: Increases in relative hydraulic conductivity following various cleaning methods

Porous asphalt	Increase in RHC following cleaning by:		
	Road sweeper	Hand jetting	Osprey 3000
PAS-A – In wheelpaths	x1.9	X1.8	X13.6
PAS-B – In wheelpaths	X0.4	X4.5	X8.9
PAS-A – Outside wheelpaths	X1.2	-	-
PAS-B – Outside wheelpaths	X0.8	-	-

Overall the results showed that, in the wheelpaths on the porous asphalt, the relative hydraulic conductivity decreased with trafficking but increased on cleaning. The specialist cleaning by the Osprey 3000 equipment is markedly better than the other procedures.

The permeability of the porous asphalt surfacing of Bay 4 was sufficiently improved to permit a one hour standard storm of average rainfall intensity of about 46 mm/h to be drained without runoff.

The appearance of the block paving after one pass of the Osprey 3000 equipment is shown in Figure 8.33.

Figure 8.33: Specialist cleaning of block paving

8.3 Summary and conclusions

The findings of the hydraulic study of the trial pavements are as follows:

- The trial pavements for heavy traffic were subject to natural and simulated rainfall events and demonstrated the expected behaviours of full-scale reservoir pavements.

- The investigations showed the ability of a sandy gravel subgrade beneath a block paved, permeable pavement to drain infiltrated water from substantial rainfall events on the pavement's surface.

- The natural attenuation of the peak intensities of simulated rainfall events and the delays to the peak drain flows by various reservoir pavement materials and structures was quantified. Although this modification of water flow is advantageous, restrictor orifices in drains were shown to reduce and delay peak drain flows from tanked reservoirs more significantly, which would be more effective in mitigating against flooding incidents.

- The pervious surfaces of block paving and porous asphalt became significantly clogged with detritus within the eighteen month study. The adoption of pervious surfaces for a specific site should be dependent on their likelihood of clogging.

- Equipment to clean pervious surfaces was assessed and one device was shown to produce a marked improvement in water infiltration capacity.

- Although cheaper to construct, reservoir pavements with pervious surfaces have a regular maintenance requirement that will increase their whole life cost.

9   Other attributes of reservoir pavements

Reservoir pavements can also help to reduce pollution, noise, spray and glare.

9.1   Pollution control

Surface water runoff can wash pollutants into watercourses and soil. The nature and amount of these pollutants depends on the land use and human activities within the catchment area. Pervious pavements can be used to intercept runoff and so limit the direct discharge of pollutants into drains and waterways. CIRIA (2002) points out that the pollutants of most concern in highway and car park runoff are:

- Sediments;

- Metals (zinc, copper, cadmium);

- Hydrocarbons (oil and fuel) including polycyclic aromatic hydrocarbons (PAH);

- Pesticides and herbicides from landscaping maintenance;

- Chlorides from de-icing.

The impacts of these pollutants are many and varied and they have been extensively covered in both scientific and general media. The Water Resources Act (Environment Agency, 1991) and the Groundwater Regulations (Environment Agency, 1998) are the main items of legislation protecting waters in England and Wales. In Scotland, the policy relating to the use of sustainable drainage techniques is provided in Scottish Environment Protection Agency Policy No 1 (SEPA, 1996) and Policy No 15 (SEPA, 2001). Water policy and legislation is however evolving rapidly following the introduction of the Water Framework Directive (2000/60/EC), which sets out key objectives for the water environment including:

- Prevention of the deterioration of the status of all surface and groundwater bodies;

- The protection, enhancement and restoration of all bodies of surface water and groundwater with the aim of achieving good surface water and groundwater status by 2015; and

- A contribution to the mitigation of the effects of floods.

There are no documented cases of the use of porous surfaces and associated reservoir pavements causing deterioration in the quality of receiving waters (CIRIA, 2002). All the evidence to date has demonstrated an improvement in water quality. Pollutants may be filtered from the percolating water. This may occur through entrapment (filtration), adsorption or biodegradation. Filtration can occur within the soil, the aggregate matrix or on geotextile layers within the construction. Adsorption occurs when the pollutant attaches or binds to the surface of soil or aggregate particles. Microbial communities can become established and biodegrade organic pollutants such as oil or grease (Pratt, 1999; Puehmeir, 2002). A number of research studies[1] have identified the benefits of pervious and reservoir pavements in attenuating pollutants in drained water. Reductions are recorded in, for example, suspended solids, oil, copper, lead, zinc and cadmium. Chemical oxygen demand (COD) was also reduced.

There is less direct evidence for the pollutant attenuation effects of edge drained, injected type reservoir pavement systems (i.e. Types IV- VI). Many of the attenuation processes described above, however, will still be active when water is injected into the subbase reservoir as opposed to percolating down from the surface. In addition, edge

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Footnote[1]: Baladés et al (1992), Hogland and Niemczynowicz (1986), Legret et al (1999), Newman et al (2002), Pratt (2004) and Ranchet et al (1993).

drained, injected systems provide the opportunity to fit pollution containment within the edge drain system. This design might include, for example, sediment or hydrocarbon traps. With an appropriate maintenance regime, these traps will both address pollution issues and mitigate one of the more significant disadvantages of pervious reservoir systems; their potential susceptibility to clogging by washed off sediments.

The ability of reservoir pavements to attenuate pollutants associated with road runoff or to trap these pollutants supports the aims of HD 45 (HA et al, 2009). This document provides guidance on methods to assess pollutant risks and mitigate their impacts, together with a description of the legislative setting, and roles and responsibilities of both Overseeing Organisations and the Environmental Protection Agencies.

9.2　Acoustic performance and spray and glare reduction

Full depth porous pavements should be at least as good as conventional pavements surfaced with porous asphalt in reducing traffic noise in their early life and the performance of these roads is well documented. Berengier et al (2000) monitored the acoustic behaviour of porous pavements and showed that, after 10 years, the acoustic absorption of drainage pavements was 10 to 15% better than conventional pavements. Also, because of their full depth air voids structure, they can also attenuate engine noise better.

As with acoustic properties, the spray and glare reduction properties of appropriately maintained porous pavements should be, at least, as good as those of conventional pavements surfaced with porous asphalt surfacings.

10 Implementation of reservoir pavements on the HA road network

10.1 Low risk locations

The findings of this research enable the use of reservoir pavements in more highly trafficked situations than has previously been possible. However, due to the limited experience in the use of this innovation, a low risk strategy for implementation is recommended. Also, for reservoir pavements in less highly trafficked situations, including occasionally trafficked and untrafficked uses, further information is required to optimise the design, construction and maintenance of these pavements. It is therefore recommended that initially reservoir pavements should be limited to the following locations:

Trafficked

- In hard shoulders of motorways (unless subject to hard shoulder running) and central reservations, where moisture susceptible subgrades beneath adjacent conventional pavements constructed with dense materials are isolated from wetted subgrades of reservoir pavements.

- Parking areas including motorway service areas.

- Isolated emergency refuge areas as well as emergency access and egress areas of motorways

Occasionally trafficked or untrafficked

- Lay-bys.

- Approaches to toll booths in occasionally or lightly trafficked areas.

- As a replacement of granular drains to avoid stone scatter.

- Within the confines of roundabouts.

- Within verges, footways and cycleways and other non-trafficked areas.

In addition, areas occupied by balancing ponds could be converted into paved reservoirs and use made of the overlying pavements.

The thickness of the pavement would be dictated by the magnitude of the envisaged traffic.

Apart from choice of location, risk could be further reduced by construction of reservoir pavements on non-moisture susceptible subgrades or, possibly, on subgrades that have been rendered non-moisture susceptible by virtue, for example, of stabilisation.

For moisture susceptible subgrades beneath reservoir pavements that drain into the soil, care must be taken to establish the design subgrade strength by, for example, conducting soaked CBR tests. Such subgrades beneath conventional pavements, however, must not be weakened by water infiltrating from an adjacent reservoir pavement that has been subsequently constructed. This risk is reduced by isolating the reservoir pavement from the traditional pavement with, for example, cut-off drains. Associated with this approach is the need for the layout of these drains to be designed for ease of investigation and maintenance so that their satisfactory performance can be maintained.

Reservoir pavements, which are built over undulating ground, say, alongside part of a trunk road, could require the reservoir to be divided up into compartments to prevent water flowing through the reservoir and out of the pavement at low points.

All of these issues suggest that the next stage in the development of reservoir pavements should be the construction of localised reservoir pavements, which receive

water from a wider area and can be hydraulically isolated from existing traditional pavement structures. This arrangement could be readily incorporated in emergency refuge areas and emergency access and egress areas of motorways. Also, with care, sections of the hardshoulder, both full and part width, could be designed to these requirements.

10.2   Implementation strategy

The proposed stages for full implementation of reservoir pavements on the Highways Agency (HA) road network are as follows:

- Issue an Interim Advice Note (IAN) as guidance that is based on current knowledge.

- Approve reservoir pavements for use in low risk situations on HA roads under Departures from Standards.

- As a condition of the approval of the Departure from Standards, require information to be provided on the construction and approval testing of the reservoir pavements.

- Monitor a series of full-scale trials of reservoir pavements in trafficked situations.

- Use information from the approved roads and the full-scale trials, which should aim to cover a range of subgrades and traffic levels, to identify limitations of the guidance and develop second generation advice.

- Construct reservoir pavements more widely on the HA road network to cover a greater range of situations, including those of higher risk, than is advised in this report.

In this way, it should be possible to use reservoir pavements, alongside other solutions, to maintain water discharges from trunk roads at current rates despite road widening and increased rainfall due to climate change.

11 Conclusions

1) Reservoir pavements have many advantages. They can be used to control storm water effectively and can also filter out pollutants. Those pavements with porous surfaces can reduce traffic noise and improve safety by reducing spray and standing water on the road. Reservoir pavements fulfil the requirement of Sustainable Drainage Systems (SuDS) by dealing with runoff at source and, in the infiltration mode, they help replenish groundwater. However, the locations for constructing these pavements require careful consideration and those pavements with pervious surfaces require more maintenance than conventional pavements.

2) Reservoir pavement technology is still developing and, in other parts of the world, many design configurations have been developed for use in lightly trafficked applications. However, it has been recognised that the technology needs to be developed for heavier traffic applications. A pilot-scale trial, which comprised four variants of reservoir pavements as flexible pavements with a hydraulically bound mixture (HBM) base of porous concrete, was therefore constructed. Pavement design theory suggests that these novel constructions have the potential to carry heavy traffic.

3) The trial was trafficked by heavy commercial vehicles and the structural performance of these pavements was evaluated. Laboratory mechanical tests and *in situ* structural tests demonstrated that the test pavements fulfilled many of the structural requirements that were assumed in their design. There was not any evidence of structural deterioration by cracking or of excessive deformation by traffic.

4) Two pavement variants included subbases of linked geocellular boxes. The contribution of this modular subbase system to foundation stiffness, which describes the foundation's support of the overlying pavement, should be explored in future research along with the development of a method of incorporating this type of subbase in traditional pavement design methods.

5) The hydraulic performance of each trial pavement under natural and simulated rainfall events has been investigated and the expected behaviours of full-scale reservoir pavements demonstrated. The effects of water infiltrating through a permeable pavement and draining into a sandy gravel subgrade were observed. The natural attenuations of peak rainfall intensities by the various pavement materials and structures were determined along with the associated delays to the peak water flows from the reservoirs. Greater control of water flows in outfall drains was obtained by the use of restrictor orifices that can be chosen to adjust flow rates to permitted values.

6) The pervious surfaces of block paving and porous asphalt became significantly clogged with detritus during the eighteen month study. The adoption of reservoir pavements with pervious surfaces for a specific site should be dependent on their likelihood of clogging. Equipment to clean pervious surfaces was assessed and one device was shown to produce a marked improvement in water infiltration capacity. Although cheaper to construct, reservoir pavements with pervious surfaces have a maintenance requirement that will increase their whole life cost. An alternative approach is to adopt the version of reservoir pavement that collects water runoff from conventional, impermeable surfaces at edge drains and injects the runoff into the reservoir layer via sediment traps.

7) The findings of the research informed an Interim Advice Note for the Highways Agency that was produced within the project as a draft document for further consideration and development.

8)   This trial is envisaged as the first stage of a programme of research to develop reservoir pavement technology and guide its application within the Highways Agency's (HA's) road network. Future research stages are intended to include the construction and monitoring of full-scale trials on the road network in, initially, low risk locations such as safe havens and where subgrade conditions are unlikely to cause problems. As experience is developed, the application of reservoir pavements could be widened. In this way, the HA's guidance for the design, construction and maintenance of reservoir pavements could be confirmed and adapted.

Acknowledgements

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Glossary of terms

CBR value	California Bearing Ratio; a measure of the stiffness and strength of soils, used in road pavement design.
Controlled waters	In England, Scotland and Wales, a term used to describe groundwater and surface waters.
Elastic modulus	Also known as Young's Modulus or stiffness modulus; the ratio of stress divided by strain for a particular material.
Geomembrane (membrane)	An impermeable plastic sheet, typically manufactured from polypropylene, high-density polyethylene or other geosynthetic material.
Geotextile	A polymeric fabric which is permeable.
Grading – Poorly and well sorted	A poorly sorted (well graded) soil contains an assortment of particles covering a wide range of grain sizes which reduce permeability through restriction of pore spaces and interconnectivity, whereas a well sorted (poorly graded) soil has particles that are more uniform in size.
Groundwater	All water that is below the surface of the ground in the saturation zone (below the water table) and in direct contact with the ground or subsoil.
Infiltration	The passage of water through a boundary, either the pervious surface or a sub-surface interface, into the underlying material.
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.
Porosity	The percentage of the bulk volume of a rock or soil that is occupied by voids, whether isolated or connected.
Porous pavement	A porous pavement allows water to infiltrate across its entire surface, for example porous concrete or porous asphalt.
Permeable pavement	A permeable pavement is formed of a material that is itself impermeable but which is laid to provide a void space through the surface to the subbase (e.g. concrete block paving designed to allow water at the surface to penetrate through joints or voids between the blocks into the underlying structure).

Pervious pavement	A pervious pavement is any type of pavement surface that allows direct downward water infiltration- the terms porous and permeable pavement (see above) are specific types of pervious pavement that comply to the definitions in the CIRIA SUDs Manual C697.
sa_{130}	Abbreviation for a standard 130 kN axle used in France.
sa_{80}	Abbreviation for a standard 80 kN axle used in UK.
Subbase	An unbound or bound layer laid on the soil (or capping layer) to provide a stable foundation for construction of the road pavement.
Subgrade	The soils onto which the road pavement is constructed.
SuDS	Sustainable Drainage System: a sequence of management practices and control structures designed to drain surface water in a more sustainable fashion than some conventional techniques.
Surface water	Generally, waters including rivers, lakes, lochs, loughs, reservoirs, canals, streams, ditches, coastal waters and estuaries; but specifically, in the context of this document, water runoff derived from rainfall on a pavement.

A pilot-scale trial of reservoir pavements for drainage attenuation

TRL

Infrastructure developments with hard paved areas prevent the natural dissipation of rainwater. Their adverse effects are cumulative and can lead to long-term problems of disposing of water that can result in flooding. The Highways Agency aims to maintain rainwater runoff rates from their roads at current levels, despite road widening and the increase in rainfall predicted by climate change. Sustainable Drainage Systems (SuDS), which deal with runoff at source by mimicking the natural processes of redistributing rainwater to the air and the ground, reduce the severity of these problems. One such system is a reservoir pavement that can either eliminate or reduce runoff, or just temporarily store water and reduce run-off flows. There are several configurations of these pavements to cope with different site specific issues. Reservoir pavements, however, have been used mainly for lightly trafficked applications. This report describes a pilot-scale trial of flexible pavements with porous concrete bases that have the potential to extend the technology to more demanding traffic levels. An assessment is made of the hydraulic and structural behaviours of a variety of reservoir pavement types that indicates the potential of these pavements. This research, sponsored by the Highways Agency, was undertaken by TRL with industry collaboration.

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