Department of Civil and Construction Engineering

Investigation into the Ability of Pervious Pavement to Treat Stormwater for Aquifer Storage and Recovery

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To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgement has been made. This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

Signed:______________________ Date:______________

Abstract

Stormwater management has been given far more attention in recent years due to the pollution and excessive flows generated by traditional pipe systems. In addition, it is being looked to increasingly as a water resource, as population grows. One such approach is to store the stormwater underground using Managed Aquifer Recharge (MAR). For confined aquifers, Aquifer Storage and Recovery (ASR) is appropriate, where it is piped directly into the aquifer, but this requires pre-treatment. Porous pavement, a type of pavement that allows stormwater to infiltrate through it, could be one way to pre-treat the stormwater. This is being investigated by City of Canning, Western Australia, using stormwater for supplementing the local confined Leederville aquifer with extraction for later use for irrigation. The aim of this research was to find a pervious pavement system that would treat stormwater to a level suitable for MAR into the Leederville aquifer and subsequent extraction for irrigation. Laboratory testing has determined that modifications to the typical construction of pervious pavement will be required to treat the stormwater to an acceptable level, with nutrients being the focus of this research. In particular, adding a layer of Granular Activated Carbon (GAC) immediately below the bedding layer and adding a sand layer to the base were investigated. Both arrangements were shown to reduce ammonium from 1.46 mg (N)/L to roughly 0.04 – 0.06 mg (N)/L, at times lower than threshold limits for injection to the Leederville aquifer. In addition, the test rig with the sand layer was able to reduce phosphate from 0.84 mg (P)/L to roughly 0.05 mg (P)/L and at times met the required guideline value of 0.04 mg (P)/L. Suspended solids and the sum of nitrite and nitrate were not reduced to guideline values and should be the focus of further research.

Acknowledgements

As an old saying goes, no man is an island, and this research was certainly no exception. The guidance, support and great patience of my supervisor A/Prof Arumugam Sathasivan is much appreciated and his sound experience was a most appropriate and needed basis for my research. We worked together with forming ideas and he provided supervision for my whole thesis, continuing even after he left Curtin. I also give thanks to my associate supervisor Adj Prof Colin Leek and the City of Canning who gave me this inspiring topic to research as well as substantial material support, without which this project would not have been possible. I am also grateful to my co-supervisor (later supervisor) Prof Hamid Nikraz for his support as well as his help with pavement construction. Lecturer Dr Peerapong Jitsangiam also gave much needed and appreciated guidance with the construction of the pavements. The great assistance from all the laboratory technicians and fellow research students are greatly appreciated and I do not know where I would be without their help.

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1 INTRODUCTION

1.1 Background

Regulating authorities are placing emphasis on stormwater management. This is because it has been recognised that pollution and damaging flows from stormwater runoff pose an environmental risk. Particularly in heavily populated built up areas, runoff from impervious areas contains significant pollution (Linforth, Vorreiter, Constandopoulos, & Biddulph, 1994; Ports, 2009). Excessive stormwater flows can cause damage to water bodies as well as flooding (Fan & Li, 2004).

Water Sensitive Urban Design (WSUD) is a water management approach to limit impact on the environment, including waterways (Donofrio, Kuhn, McWalter, & Winsor, 2009). In terms of stormwater, both the contaminants and flows are managed.

Pervious pavement is one such Water Sensitive Urban Design solution, as it both reduces runoff (Bean, Hunt, & Bidelspach, 2007) and reduces contaminants (Hatt, Fletcher, & Deletic, 2007). It allows rainwater to infiltrate through the pavement, which can then be infiltrated into the natural ground (Hunt, Stevens, & Mayes, 2002) or collected via subsoil pipes (Watanabe, 1995).

Concurrently to the problems caused by stormwater, water scarcity is a pressing global issue (Barker, Scott, De Fraiture, & Amarasinghe, 2000). Underground aquifers can provide useful water storage for supply. These aquifers are often over-extracted (Nelson, 2012; Rodell, Velicogna, & Famiglietti, 2009; Wada et al., 2010), so to balance this, water could be placed back into the aquifer via the scheme of Managed Aquifer Recharge (MAR). For confined aquifers the Aquifer Storage and Recovery (ASR) system is appropriate, where water is directly pumped into the aquifer via a bore and recovered at the same point. Aquifer Storage Transfer and Recovery (ASTR) involve recovery at a different point, for additional treatment in the aquifer. Treated stormwater can be a water source for MAR and is in fact used in an ASTR scheme in South Australia, with the treatment performed by wetlands (Page et al., 2010).

Stormwater treatment requirements for MAR are substantial (Dillon et al., 2010). Pervious pavement often struggles to treat stormwater to the required quality. It can reduce ammonium in the order of 80-100% compared to inflow or asphalt runoff (Bean et al., 2007; Collins, Hunt, & Hathaway, 2010; Scholz & Grabowiecki, 2009) and can be very good at removing motor oil (Brattebo & Booth, 2003). However, the ionic oxides of nitrogen from nitrification of ammonium can be an issue (Bean et al., 2007; Collins et al., 2010) as can metals (Brattebo & Booth, 2003; Fassman & Blackbourn, 2010; J. J. Sansalone, 1999) and suspended solids (Bean et al., 2007; Fassman & Blackbourn, 2010; Rowe, Borst, O'Connor, & Stander, 2009). In fact, metals and suspended solids can leach from the pervious pavement materials (Brattebo & Booth, 2003; Fassman & Blackbourn, 2010).

Additional treatment measures would be needed where MAR requirements are not met by a typical pervious pavement design, such as iron oxide coated sand (J. J. Sansalone, 1999) or granular activated carbon / charcoal (Kanjo, Yurugi, & Yamada, 2003). Overall, use of pervious pavement for MAR will require testing to find the best system, with reference to investigations of pervious pavement performed by others.

1.2 Objectives

Pervious pavement is being considered for treatment of stormwater for Aquifer Storage and Recovery in the City of Canning, Western Australia, using the local Leederville aquifer. The ultimate aim is to ensure enough water is in the Leederville aquifer so it can be used for irrigation. Multiple objectives stem from this consideration:

- Establish the concentrations of pollutants in the Leederville aquifer as well as guideline concentrations (performed by others).
- Perform laboratory testing to find a pervious pavement that will reduce pollutant concentrations to target levels.
- Validate that this pervious pavement, or pavements, reduce pollutants to the required level by performing field testing.

1.3 Scope

Due to time constraints, not all of the objectives in Section 1.2 could be met within a Masters by Research degree. Specifically, the scope of this research project was:

- Find typical concentrations of the pollutants listed in Section 1.2 in stormwater and decide what concentrations to use for testing. An investigation into end-of-pipe stormwater pollutant concentrations was undertaken by GHD (GHD, 2008, 2009), however concentrations in direct runoff is more relevant in the case of treatment by pervious pavement.
- Perform laboratory testing on four pervious pavement rigs, using granular activated carbon in one, pavers with gravel filled gaps in at least one and porous concrete in at least one to determine how these perform at pollutant reduction. A yellow sand layer at the base of one test rig was also trialled. It was planned to test porous asphalt and Permecocrete (a porous concrete containing magnesium in the cement), but due to availability issues this was not possible.
- Test for reduction of ammonium, phosphate and dissolved organic carbon. The dissolved organic carbon was in the form of a single compound (glucose) to simplify analysis.
- Test for nitrate, nitrite and suspended solids, which would be produced or come from within the pavements.
- Add metals aluminium, copper and zinc at a later point to see how they affect nutrient reduction performance.
- Investigate any variations in pollutant reduction over time, attempting to determine the cause.
- Perform a detailed analysis to determine which pervious pavement produces the highest quality output and under what conditions.

1.4 Significance of Research

The focus of this research was designing a pervious pavement to treat stormwater for MAR, however, it will be useful for determining the applicability of pervious pavement for WSUD generally. Most of the literature reviewed was on field testing of pollutant reduction by pervious pavement, which by its very nature will be site specific. Laboratory test results were also found in the literature, of widely varying quality and scope. In particular, this research will:

- Provide a better understanding of how granular activated carbon within a pervious pavement setup will contribute to removal of dissolved organic carbon and ammonium, as well as any side effects
- Give insight into the effects a sand layer on the base of the pavement will have on pollutant removal
- Provide understanding of pollutant reduction performance using local aggregates
- Provide better insight into how easy or difficult it is to design a pervious pavement to treat stormwater for a proposed ASR scheme
- Initiate the process of finding a pervious pavement that will treat stormwater to a level suitable for MAR, thus providing an additional water resource. The City of Canning in particular requires additional water resources for irrigation of open spaces.

1.5 Research Approach

Laboratory testing was performed to provide a more reliable comparison of pavement designs, compared to field testing. Subject to availability constraints, four pervious pavement test rigs were designed and constructed. The scale of the testing system allowed the collection of sufficient samples for laboratory testing.

It was intended to protect these test rigs from the weather as much as possible, but due to space constraints they were situated outside. Covers were provided to protect from natural rain and direct sun as much as possible, but with ventilation provided, to mimic cloudy conditions. All four pavements were directly adjacent to each other so that they all experienced virtually the same conditions. Other sources of uncontrolled variability were limited where practical.

It was intended to remove as much clay material from the aggregate as possible during construction to reduce variability, but as construction commenced and testing progressed this proved difficult. However, as an unintended consequence the different extents of washing did provide insight into how the clay contributed to pollutant reduction. The hydraulic analysis in Section 4.1.2 gives an indication as to how the base conditions varied over time, with outflow becoming slower as more fines migrated to the base.

After a certain period of time it became clear that rigs 1, 2 and 4 were not treating the input to the required quality, so rig 1 was modified to include a sand layer, in an effort to improve treatment. Emission of nitrification by-products (nitrate and nitrite) worsened but ammonium and phosphate reduction performance improved. At the end of testing pavement layers were removed before each run to see how they contributed to pollutant reduction.

2 LITERATURE REVIEW

2.1 Background

2.1.1 Environmental Impact of Stormwater Runoff

Regulating authorities are placing emphasis on stormwater management. This is because it has been recognised that pollution and damaging flows from stormwater runoff pose an environmental risk. This is especially the case in heavily populated areas (Linforth et al., 1994; Ports, 2009). These places have large impervious areas that cause much higher rates of runoff after rainfall (Boyd, Bufill, & Knee, 1993), with higher pollutant transport (Ichiki, Ido, & Minami, 2008).

Excessive stormwater flows can cause damage to water bodies (Fan & Li, 2004). Nutrients can produce algal blooms (Shukla, Misra, & Chandra, 2008) and pollutants such as metals can harm biota (El-Nady & Atta, 1996). Even stormwater disposed of to the ocean can be a health risk according to University of Western Australia (UWA) modelling (Bennett, 2010). With such risks posed by stormwater runoff, mitigation is recommended.

2.1.2 Water Sensitive Urban Design

The main objective of Water Sensitive Urban Design (WSUD) is to maintain as near as possible, the pre-development water cycle. This need not purely be a liability. There are potential gains to be made by 'streamlining' water management and, say, recycling wastewater or utilising stormwater. At a minimum, WSUD is protection of waterways and groundwater, with pollution control a significant consideration (Chanan, Vigneswaran, & Kandasamy, 2010).

WSUD is a broad term, covering all types of urban water flows – water supply, wastewater and stormwater (Donofrio et al., 2009).

In addition to the impacts to the natural environment of stormwater runoff, excessive runoff within developed areas can be an issue.

2.1.3 The Problem of Inadequate Flood Control

In some cases, the existing infrastructure for controlling floods is inadequate. This is largely due to the combined impact of lack of planning, aging infrastructure and climate change.

Particularly in the case of Perth, Western Australia, in past times, most roads were unkerbed and water drained to the verge. In more recent times, local governments have kerbed roads mainly for aesthetic purposes, and water that used to infiltrate then required draining (C. Leek, personal communication, 2012).

For reasons unknown, rather than re-establishing soakage systems, pipe systems were developed, in many cases to large soaks established on housing lots. Land values rose, and local governments saw the advantage of extending pipe networks to rivers, so that the land previously set aside could be sold and developed. As suburbs expanded, pipe systems were extended, often without upgrading existing sections to cater for additional impervious areas of runoff (C. Leek, personal communication, 2012).

Changes in public acceptance and liability issues have seen changes in the stormwater drainage design standards, but even in Perth today, there are differences in standards between local governments, and between local governments and the Water Corporation (C. Leek, personal communication, 2012).

Some local governments, for example City of Canning, design for a 1 in 5 year recurrent storm, but where flooding of properties cannot be prevented by available overland flow paths, will design those sections for a 1 in 100 recurrent storm, whilst discharging to a Water Corporation main drain designed for a 1 in 10 year recurrent storm (C. Leek, personal communication, 2012).

Inadequate drainage planning is particularly a problem for poorer countries such as Brazil (Soares, Parkinson, & Bernandes, 2005), but developed countries are not immune. In places like Hong Kong, where in older areas drainage was designed to lesser protection standards and is deteriorating, flooding is an issue (Chan, Mak, Tong, & Ip, 2011). Increases in impervious area due to unforeseen development also potentially causes problems (Brach & Zeytinci, 2005).

Usefully, Australia has a rating system for infrastructure. Engineers Australia's 2010 Infrastructure Report Card gave Australia a 'C' for stormwater, indicating major changes were required. According to the report, a long dry spell had shifted focus away from flood management to water supply. Therefore, as urban infill projects developed, impervious areas in older suburbs increased without sufficient upgrades to stormwater drainage designed for the original developments with less impervious area. Problems arising from infrastructure reaching the end of its life were also highlighted. Stormwater pipes undergo deterioration and this impacts on hydraulic performance. Due to these combined factors, older areas were predicted to experience overland flooding with the return to 'more normal rainfall patterns' (Tran, Perera, & Ng, 2009).

The report card for Western Australia gave a 'C' for stormwater. It stated the performance could not be assessed due to lack of data and a drying climate (Yates, 2010).

Climate change is known to impose greater flood risk in many cases. This was investigated in detail in the United States (Thomas Jr, Kollat, & Kasprzyk, 2010). Climate change has already been observed to cause a dramatic change in hydrology in the South Western portion of Western Australia (Pitman, Narisma, Pielke, & Holbrook, 2004). Despite a notable downwards trend in annual runoff in this region (Bari, Berti, Charles, Hauck, & Pearcey, 2005), it is possible that the magnitudes of extreme floods may increase due to climate change (Yu & Neil, 1993). Modelling indicated that for Perth the 100 year 24 hour storm could increase in rainfall depth from 115 mm to 150 mm, under CO_2 concentrations twice that of 'control' (Evans & Schreider, 2002). A revision of Australian Rainfall and Runoff is underway, with a preliminary analysis of national hydrological data for trends (Ishak, Rahman, Westra, Sharma, & Kuczera, 2010). Any significant increase in design floods could mean an upgrade of existing infrastructure is required.

Stormwater runoff does pose significant issues, but if controlled wisely can actually be a resource.

2.1.4 Stormwater Harvesting

Water shortage is a pressing global issue, in part due to a growing population (Barker et al., 2000). In response, desalination is becoming much more common as a means of providing potable water (Stover, 2009). Australia is one such country increasingly relying on seawater desalination for water use (Crisp, Swinton, & Palmer, 2010). Large scale desalination plants in Australia cost around \$0.50-\$2.00/m^3 of water produced (Wittholz, O'Neill, Colby, & Lewis, 2008).

Wastewater recycling is an alternative to desalination, but public acceptance is a major obstacle (Dolnicar & Saunders, 2006). Western Australian Water Corporation's Perth Groundwater Replenishment Trial uses recycled wastewater to recharge the Leederville aquifer via managed aquifer recharge (see Section 2.1.4.1), which is then proposed as drinking water (Water Corporation, 2009). Their own survey on public perception found 31% of respondents requested further information before supporting it and 10% were firmly opposed. In 2007, respondents quoted an opinion that there were other "safer" sources that should be pursued first. Therefore, use of rainfall-originating water could possibly be attractive in terms of cost and perceived quality.

In Perth, the unconfined Gnangara Mound aquifer, providing about 60% of Perth's scheme water (Marsden & Pickering, 2006), is being depleted by reducing rainfall, abstraction for water supply and pine plantations (Yesertener, 2008). This has a number of environmental impacts, such as water acidification (Silva, 2009), impacts on lakes such as Perry Lakes (McFarlane, Smith, Bekele, Simpson, & Tapsuwan, 2009), cave ecosystems like Yanchep Caves (Yesertner, 2006), wetlands and other ecosystems (Arrowsmith & Carew-Hopkins, 1994). Groundwater depletion is a significant issue internationally (Nelson, 2012; Rodell et al., 2009; Wada et al., 2010).

Stormwater runoff in urban areas is an under-utilised resource. More rain falls on many Australian cities than water is used (Prime Minister's Science Engineering and Innovation Council, 2007). River systems are significantly drawn on instead, typically via dams. However, inflows to these dams from the rivers have been decreasing in some cases. In Perth, the average total annual inflow for dams existing in 2001 dropped from 338 GL for the period 1911 to 1974 to 177 GL for the period 1975 to 2000. For the more recent period 2006 to 2010 annual inflow has averaged only 57.7 GL (Water Corporation, 2010). In other cases, overdrawing is a significant issue. For some rivers, environmental flows are dangerously low. The Murray River is a good example, where 74% of the floodplain is at high risk of reduced flood frequency, compared to in 1977 when most of the floodplain had a low risk of reduced flood frequency (Overton & Doody, 2009). This flood reduction risk was linked to vegetation decline, indicating the effects the reduced flood frequency has on ecosystems.

Using stormwater sustainably can actually be beneficial for the environment, by reducing flows to near pre-development levels (T. D. Fletcher, Mitchell, Deletic, Ladson, & Seven, 2007). However, enough stormwater should be allowed to enter natural watercourses for the benefit of flora and fauna. This allowance should take into account upstream water infrastructure, including dams. To an extent, stormwater harvesting is already being used residentially on a local level through on-site stormwater infiltration and garden bores (Bott & Evangelisti, 2006). There is potential for expansion of this approach. In fact, in several cases stormwater is deliberately placed into underground aquifers for later use.

2.1.4.1 Managed Aquifer Recharge

Due to depletion of groundwater reserves, Managed Aquifer Recharge (MAR) is one approach currently in use to restore groundwater levels. Water is intentionally infiltrated into the aquifer for later use, with the groundwater reserve then acting as a large underground storage. Such an approach is the focus of much research (Dillon et al., 2010; Wang, Sun, & Xu, 2010).

There are environmental benefits to MAR, in many cases intentional. For example, Perry Lakes in Perth was drying and required pumped groundwater to maintain its ecological habitats and social values. A trial was established where treated wastewater was infiltrated into the ground as a MAR project. Measurements and modelling on groundwater levels and water quality indicated the project is feasible with low to medium risk to groundwater quality (McFarlane et al., 2009).

Using stormwater for MAR can also help restore the hydrology to before development. Since the 19th century in Iowa and California, MAR for stormwater has been performed via infiltration basins and river diversion, with many other similar schemes in existence. In New Mexico, a development making use of unlined retention ponds increased the effective recharge from less than 1% of precipitation before development, i.e. in natural conditions, to about 40%; due to the removal of trees. According to the study, if a predevelopment level of runoff was allowed, this would support downstream native habitat, stream flows, wetlands and estuaries without the damage associated with increased flooding from full impervious runoff. In addition, if predevelopment effective groundwater recharge were maintained (after artificial extraction for water supply) this would also help maintain wetlands and lakes (Stephens, Miller, Moore, Umstot, & Salvato, 2012).

MAR is also used to prevent seawater intrusion and ground subsidence. Increasing the hydraulic head relative to the seawater prevents seawater intrusion; ensuring salt water cannot intrude back into the aquifer. Subsidence often occurs when groundwater is lowered significantly over time, maintaining the long-term water table depth prevents this effect (Wang et al., 2010).

One way to perform aquifer recharge is Aquifer Storage and Recovery, arguably suitable for confined aquifers. This involves directly pumping water into the aquifer via a bore. The water is later withdrawn from the same site for reuse. Aquifer Storage Transfer and Recovery (ASTR) involves recovering the water from a different well for additional treatment. An ASTR project using stormwater is already in operation in Parafield, South Australia, which makes use of wetland pre-treatment (Page et al., 2010). For injection of stormwater, pre-treatment is required, with wetland treatment, microfiltration or granular activated carbon filtration suggested (Environment Protection and Heritage Council, 2009a). Porous pavement is one possible pre-treatment option and is in fact being pursued by City of Canning, Western Australia, for recharging the local confined Leederville aquifer. This is the focus of this research.

2.1.5 Pervious Pavement

There are many ways to deal with stormwater, but arguably, the most elegant is to directly replace impervious areas with pervious areas. Not only would this reduce runoff (Bean et al., 2007), it would increase stormwater treatment, as infiltration is effective at removing pollutants (Hatt et al., 2007). Pavement takes up a considerable portion of impervious area; it can be about 67% of the total impervious area in a typical urban setting (Zhou & Troy, 2008). Such an area could be made pervious by using either porous or permeable paving. Similar in trafficability to conventional pavement (depending on construction), these allow the ingress of stormwater directly through the pavement, to be infiltrated into the natural ground (Hunt et al., 2002) (Figure 1), stored *in-situ* (Myers, Sagi, van Leeuwen, & Beecham, 2007) or collected via subsoil pipes (Watanabe, 1995). When stored, the stormwater can be used for purposes such as irrigation (Oost, 2004).

Pervious pavement has the dual benefit of reducing runoff and pollution (Ball & Rankin, 2010; Brattebo & Booth, 2003). Reducing runoff and either reusing or infiltrating the stormwater substantially reduces flash flooding (Collins, Hunt, & Hathaway, 2008; Guan, Kong, & Zhang, 2009; Nishiyama, Ohnishi, Yano, Yamamoto, & Wada, 2010). Pervious pavement can also cause a significant reduction in stormwater pollutant concentrations (Calkins, Kney, Suleiman, & Weidner, 2010; Collins et al., 2010; Fassman & Blackbourn, 2010; Scholz & Grabowiecki, 2009). This is believed to be through a combination of sorption of heavy metals to the media and decomposition by bacteria (Scholz & Grabowiecki, 2009). The bacteria are in an oxygen rich environment due to the voids in the pavement, promoting their activity and growth.

2.1.5.1 Types and Materials

Pervious paving requires the use of materials that are both load bearing and allow rainwater to flow through them. The surface layer is the interface between the applied traffic (pedestrian or otherwise) and the pavement. It can be monolithic and porous in construction. Porous concrete is a common example (Beecham & Myers, 2007; Scholz & Grabowiecki, 2007), where the fines are removed from the concrete to give it its porosity and hence permeability. Similarly, asphalt can be made in the same manner (Brown, 2003; Liu & Cao, 2009). Gravel can also be bound with a resin, with its typical use around tree surrounds (Ayton Products, 2005). The aggregate does not even have to be stone – materials such as recycled glass or rubber can be used (M. B. Chopra, Stuart, & Wanielista, 2010; Meiarashi, 2004; Sandberg, 1999). Filterpave is an example of a recycled glass product.

There are other ways to construct pervious pavement. Impervious units (segmental paving blocks) can be laid to form gaps, where the gaps can be filled with permeable material such as gravel or soil (Bean, Hunt, & Bidelspach, 2004; Collins et al., 2008) or plastic grids can reinforce the fill material (Figure 1). These segmental approaches are often termed permeable pavement. Two examples of segmental paving blocks are Concrete Grid Pavers (Figure 2), where each typically non-rectangular unit is laid to form relatively large square holes, and Permeable Interlocking Concrete Pavers, where individual units form relatively smaller gaps when laid in the appropriate pattern. Such approaches are typically termed permeable paving. Voids may be filled with turf (Oost, 2004; Starke, Gobel, & Coldewey, 2011), which in the case of plastic grids can make the pavement into a continuous lawn (Figure 1).

Loose single sized gravel can also be used as a surface layer, however only in very low speed, low traffic situations such as infrequently used parking lots (Yudelson, 2007, p. 133).

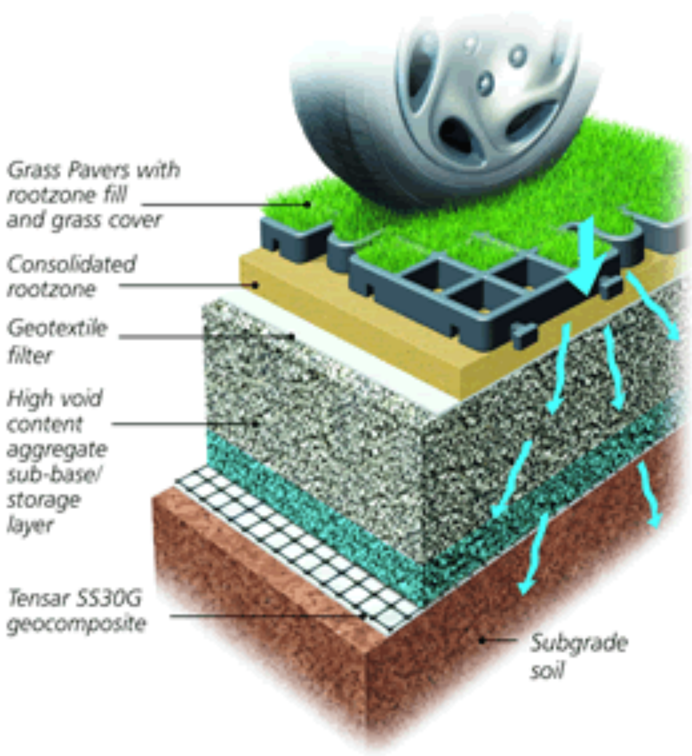

Figure 1: Grass pavers with infiltration to subgrade (Tensar International, 2011)

Figure 2: Concrete Grid Pavers near Canning Bridge, Western Australia

Underlying the surface layer is typically a bedding layer, which can be sand (Fassman & Blackbourn, 2011; Hunt et al., 2002) or 3-5 mm gravel (Andersen, Foster, & Pratt, 1999; Lucke & Beecham, 2011). Its thickness can be 30-50 mm.

Geotextile is sometimes placed directly under the bedding layer (Bean et al., 2007; Beecham & Myers, 2007) or just above the subgrade (Boving, Stolt, Augenstern, & Brosnan, 2008; Rowe, Borst, O'Connor, & Stander, 2010). When placed under the bedding layer, it traps fines coming from the stormwater runoff, as well as fines coming from the pavement materials above it. When placed above the subgrade, it controls erosion that would result from flow of water along the base. The example shown in Figure 1 includes both geotextile layers.

A gravel base is typically used in conjunction with pervious pavements, with its thickness depending on structural or hydraulic requirements. This base must be load bearing and open graded (Beecham & Myers, 2007; Shackel & Pearson, 2003). Figure 1 indicates the location of this gravel base.

For clarity, pavement that deliberately allows rainwater through it is generally termed pervious pavement herein. Pavement consisting of impervious pavers arranged to create gaps is termed permeable pavement. Where the paving units themselves are porous in nature at the coarse aggregate level, or a monolithic porous surface is used, this is termed porous pavement.

2.1.5.2 Design Considerations

Pervious pavement is a novel technology and there are many aspects to its design. Due to its gap-graded nature, it is potentially weaker than conventional pavement. This is especially the case with porous pavement (Scholz & Grabowiecki, 2009). An open graded concrete mix design is much more difficult to achieve than a conventional dense graded concrete, due to the required balance between porosity and adequate inter-bonding of the aggregates (Beecham & Myers, 2007). To mitigate this, it has been found that a combination of small aggregate size (4.75 mm) as well as the addition of some sand and latex increases the strength considerably (Huang, Wu, Shu, & Burdette, 2010), with compressive strengths ranging from 5 to 15 MPa. For porous asphalt, the Cantabro loss for new, dry mixes ranged from 1% to 18% (Alvarez, Epps-Martin, Estakhri, & Izzo, 2010). The Cantabro loss is an indirect test of mixture cohesion, resistance to disintegration and aggregate interlock. This involved placing each compacted specimen in a Los Angeles abrasion machine without an abrasive load, applying 300 revolutions, then calculating the percentage of mass broken away from each specimen.

As for permeable pavement, the durability can vary according to the system. Plastic grid systems can collapse under load, shift around and lift out, while the concrete grid pavers and permeable interconnecting concrete pavers are somewhat more durable (Brattebo & Booth, 2003). While a tank system, Figure 3 indicates the potential vulnerability of plastic load bearing systems.

Figure 3: Collapsed Plastic Grid System (courtesy of City of Canning)

The base material typically has aggregate fractions finer than 2 mm removed, with cement addition a possibility to improve strength (Shackel & Pearson, 2003).

Infiltration rate is an important consideration of pervious pavement. The permeability of new porous pavement can range between about 162 and 1180 mm/hr (Kuang, Sansalone, Ying, & Ranieri, 2011). This is substantial, considering the 100-year average occurrence interval 5-minute duration storm for Perth is close to 200 mm/hr (Institution of Engineers, Pilgrim, Canterford, & Institution of Engineers, 1987). For Permeable Interconnecting Concrete Pavement with gaps between the pavers in the ‘unblocked’ condition, high infiltration rates of roughly 10 000 mm/hr were reported (Lucke & Beecham, 2011).

Reinforced turf has a much lower infiltration rate, with the design infiltration rate determined by taking into account the long term permeability of the turf itself and the percentage of impervious cover (Argue, Allen, Stormwater Industry Association, University of South Australia: Urban Water Resources Centre, & Australian Water Association, 2005). To give an indication, silty sand can have an infiltration rate of roughly 200 mm/hr (Chiu, Zhu, & Chen, 2009). Concrete grid pavers typically have much higher impervious cover than plastic reinforcing. Runoff is likely during storm events and has in fact been measured. In a hurricane, runoff was produced at about 30 mm/hr during 40 mm/hr rainfall in one study on a concrete grid paver system (Bean et al., 2007).

The base gravel needs to be designed to have both sufficient permeability and storage volume, depending on the native subgrade permeability (Shackel & Pearson, 2003).

2.1.5.3 Maintenance

An important consideration in terms of the hydraulics of pervious pavement is clogging. Clogged pervious pavement can have an infiltration rate as low as 10 mm/hr after 6 to 18 years of service (M. Chopra, Kakuturu, Ballock, Spence, & Wanielista, 2010), with similar results for permeable interconnecting concrete pavers after seven years (Lucke & Beecham, 2011). For such deterioration in hydraulic performance, maintenance is necessary.

Several cleaning options are available for pervious pavement, such as vacuuming, sweeping, power washing, power blowing and rinsing. In one study it was found that using a large diameter hose, like a fire hose, was found to be the most effective method to restore the permeability of porous concrete. This was found to restore permeability up to 94% of the permeability just after construction. For debris gathering on the surface not yet washed into the voids, the other methods were indicated to likely be sufficient (Henderson & Tighe, 2011). It is unclear whether the pavement has a base course or is permeable concrete constructed directly on subgrade.

A similar study used porous concrete constructed on subgrade (M. Chopra et al., 2010). In this study, cores were taken from the pavement, tested for permeability then the clogged ones were vacuum swept, pressure washed or both and re-tested. Results found combining both methods was the most effective approach, with similar results to that reported in Henderson and Tighe (2011). It should be noted that cleaning was performed on the cores, not the in-situ pavement. Maintenance that removes the top layer of loose material was also able to improve the performance of concrete grid pavers and permeable interconnecting concrete pavers (Bean et al., 2004).

No study was found in the literature on the use of combination jet vacuum systems to clean pervious pavement. These have been used successfully in sewers and are cost effective (Anon, 1998).

Frimokar Pty Ltd offers road authorities, local governments and other owners of road and infrastructure assets an environmentally safe process to restore surface texture and porosity. This unit involves a high-pressure unit blasting water onto the pavement to remove fines, and vacuums the water and sediment into a recycling tank. This may be suitable for permeable pavers, concrete and asphalt, but where jointing material is the main media, this material will be dislodged and removed ("The Frimokar Process: How it works. - Frimokar Home").

2.1.5.4 Further Research

As indicated in the previous section, maintenance of pervious pavement may require further research and even consideration in the design phase. Research into pollutant removal by pervious pavement is covered in the following section, with potential for further research indicated.

2.2 Pollutant Reduction by Pervious Pavement

Pervious pavement has varying pollutant reduction rates. Fletcher and others performed a review of pervious pavement and found pollutant reduction to be roughly in the range of 40 to 90% depending on the pollutant type. The important point was made that inflow concentration, hydraulic loading and pavement properties are variables to consider. At the time of that review, there was a lack of studies reporting performance as related to these variables (T. Fletcher, Duncan, Poelsma, & Lloyd, 2004).

Pervious pavement precipitates out and removes dissolved metals by a combination of raising the pH, precipitation by metal carbohydrate complexes, adsorption and co-precipitation (Bean et al., 2007; Calkins et al., 2010; Myers et al., 2007). Particulate bound metals are typically strained out, but it is possible that most of the metals are dissolved (J. Sansalone & Teng, 2004). Bacteria also colonise within pervious pavement, with the ability to remove nutrients and hydrocarbons (Collins et al., 2010; Newman, Nnadi, Duckers, & Cobley, 2011; Scholz & Grabowiecki, 2007). In addition, hydrocarbons have a tendency to sorb to the aggregates and geotextile (Newman, Nnadi, et al., 2011; Newman et al., 2004).

In Perth, Western Australia, targeting the 3-month Average Recurrence Interval (ARI) storm will cover about 97.7% of rainfall volume. It has been argued that the majority of storm events with the potential to mobilise pollutants are of relatively low rain intensity (Wong, Wootton, Argue, & Pezzaniti, 1999). The Stormwater Management Manual specifies the 1-year ARI storm for pollutant removal purposes, covering 99.5% of rainfall volume (Department of Environment, 2004). The 1-year ARI, 5-minute duration storm is about 60 mm/hr in intensity (Institution of Engineers et al., 1987).

TKN refers to total Kjeldahl nitrogen in this section. Similarly, NO_x-N refers to nitrite-N + nitrate-N, for both concentration and total mass. TSS means total suspended solids.

Pollutant removal refers to the case where the influent and effluent concentrations of a pollutant were measured or known, and the removal is the difference between the two. Pollutant reduction is a more general case, and covers the above, as well as the case where the influent concentration is unknown or ambiguous. This is commonly the case with field testing. Instead of knowing the influent concentration, the runoff from an impervious surface such as conventional asphalt is measured, and the pollutant reduction is taken as the difference between the concentration of the pollutant in the impervious surface runoff and the concentration of the pollutant in the runoff/effluent from the pervious pavement. Generally, the effluent from the pervious pavement is measured and not the runoff, unless otherwise stated. All removal/reduction percentages are relative to the influent or impervious surface concentration.

2.2.1 Pavers with Gravel Filled Gaps

A common surface construction for pervious pavement is impervious pavers with gravel filled gaps. Much literature was found on studies of the pollutant reduction capacity of this type of pavement, commonly termed Permeable Interconnecting

Concrete Pavers (PICP). These are listed below.

An experimental test rig using C&M Ecotrihex block pavers was set up at the Royal Melbourne Institute of Technology University (Kadurupokune & Jayasuriya, 2009). It had construction details as listed in Table 2 and Table 3. Synthetic stormwater with 141 mg/L TSS, 0.24 mg/L total phosphorus (ortho-phosphate), 2.63 mg/L total nitrogen, 0.1 mg/L lead, 0.06 mg/L copper, 0.63 mg/L zinc, 0.007 mg/L cadmium and 20 mg/L of used oil from a motor vehicle was used as influent for pollutant removal testing. The total nitrogen was said to consist of nitrate and ammonium but the proportions were not mentioned. As the purpose of the test was to model long term clogging, a simulated rainfall intensity of 90 mm/hr was applied for a significant time of 1.5 hours (Table 4), for a total volume of 135 L (simulated rain area was 1 m^2). Collected volumes ranged from 100 L to about 115 L.

Over a simulated duration of 17 years, pollutant removal was determined. This was calculated based on pollutant load as opposed to concentration. Total nitrogen removal efficiency started at 63% and dropped to 12% over the simulation period. Similarly, PO_4^{3-} went from 53% removal down to 22% removal, TSS stayed at about 95% removal, oil increased from 86% to 97% removal and copper increased from 94% to 96% removal. Zinc results were unreliable, as the galvanised box used to hold the pavement test rig seemed to be corroding. Cadmium and lead were undetectable in the filtrate.

In Auckland, New Zealand, a PICP system was constructed in a road reserve then monitored over 3 years. It had construction details as listed in Table 2 and Table 3. Rainfall intensity varied from 0.3 mm/hr to 79.2 mm/hr overall (Table 4). Compared to the asphalt section and by concentration, reductions of about 56% of TSS, 57% of total copper and 93% of total zinc were achieved. The bedding sand was found to be a significant source of TSS, copper and zinc from wash tests (Fassman & Blackbourn, 2010), which is evident from the lower reduction percentages compared to Kadurupokune and Jayasuriya (2009) at over 90% TSS removal and over 90% copper removal by concentration (calculated). Hydrological information was provided but only in aggregate, i.e. no relationship between water quality and rainfall (rate or depth) was provided.

Also to test pollutant reduction, a trial carpark was set up in Washington, United

States. Four pervious pavement types were tested. The testing included determination of pollutant concentrations in the infiltrate coming from the pervious pavements, as well as runoff from an asphalt control. These pavements were constructed in 1996. Other than the surface type, no construction details were given (Table 2 and Table 3). Details were given for two storm events, including the most intense storm. Overall, the maximum rainfall intensity was 7.4 mm/hr (Table 4). Water quality data was presented in aggregate only, with the focus being on comparison between the pavements studied rather than between storm events. Stormwater samples were taken in both 1996 and 2001-2002, giving estimates of both immediate and longer-term (6 year) reduction rates. One of the four pavements used was UNI Eco-Stone, which is permeable interconnecting concrete pavers with about 90% impervious coverage and spaces filled with gravel (Brattebo & Booth, 2003).

The Uni Eco-Stone appeared to perform the best long-term zinc reduction, with an infiltrate event mean concentration of 6.8 μg/L compared to the asphalt control's 21.6 μg/L, or 69% lower. Copper concentration was 0.86 μg/L compared to 7.98 μg/L in the asphalt runoff, or 89% better, somewhat lower than Kadurupokune and Jayasuriya's at 96% by concentration after five years (2009). In 1996, the copper concentration was 79% higher than the asphalt runoff, suggesting it was a copper source. The zinc concentration was 44% lower than from asphalt, suggesting the reduction actually improved with age. Motor oil was not tested in 1996, but was below the detection limit of 0.1 mg/L for all pervious pavements in 2001-2002. The average concentration from the asphalt surface was about 0.164 mg/L. This is broadly consistent with Kadurupokune and Jayasuriya's results (2009). While suggesting pervious pavement is good for removing pollutants, the mixed results indicate further testing needs to be done to find the 'best' pervious pavement. In addition, the rainfall intensity in the above tests did not exceed 7.4 mm/hr. The soil was 'permeable', however clay fractions may have aided with coagulation and hence the pollutant reduction rate.

Pollutant reduction by a PICP setup was also investigated by Bean and others (Bean et al., 2007). This was tested in the field in Goldsboro, North Carolina after about 2 years of completion of construction. It had construction details as listed in Table 2 and Table 3. Hydrological data was not provided for the Goldsboro site (Table 4). Compared to asphalt runoff, total nitrogen was reduced on average by about 42%, total phosphorus by 63%, copper by 62% and zinc by 88%. However, TSS was only reduced by 33% relative to asphalt, suggesting loss of fines from the media. The total nitrogen result was slightly lower than Kadurupokune and Jayasuriya's at 53% removal by concentration after 2 years of simulation (2009). Whether the measured metal concentrations were dissolved or not was not specified.

Looking in more detail at the nutrients, orthophosphate was reduced by 42%, compared to bound phosphate at 72%. Again, this is less than Kadurupokune and Jayasuriya's result at 59% removal by concentration of orthophosphate after 2 years. The ammonia was reduced by 84%, compared to 50% for organic nitrogen. The NO_x-N concentration increased by 47%, because of nitrification of the TKN by nitrifying bacteria. Some denitrification would have occurred as well, due to the lower total nitrogen concentration in the effluent compared to the influent.

Water quality was also tested at a site in Swansboro, using pavement of the same construction. The concentrations in the exfiltrate were somewhat less, with the notable exception of phosphate, but no asphalt control samples were taken at this site, obscuring the reasons for the differences.

Another similar study was performed in Goldsboro, North Carolina, USA. Four pervious pavement and two asphalt parking sections were tested for nitrogen reduction a year after construction (Collins et al., 2010) (Table 2 and Table 3). This provides a valuable comparison with the previously cited article (Bean et al., 2007). These six bays were constructed adjacent to each other. Rainfall events varied in depth from 3.1 to 88.9 mm (Table 4). Neither intensity nor duration data were provided. One of the bays, named PICP1, had 80 mm deep Octabrick permeable interlocking concrete pavers, having 12.9% open area and other construction details as per Table 2 and Table 3. In addition, PICP2 was constructed with Rima brand PICP of 80 mm depth, having 8.5% open area, otherwise the same in construction as PICP1.

All four pavements had about 85% less ammonia by concentration than from runoff from the first asphalt bay (two were tested). (See below for pavement with concrete grid pavers filled with sand, see Section 2.2.5 for pavement with porous concrete.) The investigation by Bean et al. (2007) had almost the same result. Both impermeable asphalt sections tested had similar runoff concentrations of measured pollutants. Both PICP’s achieved about 30% reduction in organic nitrogen, while in Bean et al.’s study a 50% reduction was achieved, with both a higher concentration in asphalt runoff and lower concentration in PICP exfiltrate.

The NO_x-N concentration increased by over 300% for PICP1 and over 200% for PICP2, compared to asphalt runoff. However, in Bean et al.’s study it only increased on average by 47%, even though in both studies asphalt runoff concentrations of NO_x-N were about 0.3 mg/L. This is reflected in the effluent total nitrogen results, where it was about 40% higher for PICP1, 11% higher for PICP2 and 42% lower in Bean et al.’s study compared to asphalt runoff. Collins et al. attribute the increased nitrogen to either decaying biomass or algal nitrogen fixation. Their site was in fact monitored for water quality from January to July, i.e. mid winter to mid summer, with such growth well established, while in Bean et al.’s study it was monitored June 2003 to December 2004, with a bias towards the summer-to-winter growth phases.

The pervious pavements performed well at removing ammonium, the results suggest this was mostly converted to nitrites and nitrates. The reduction rate relative to asphalt was around 85%, and a lower 45% reduction for TKN. One bay was constructed of concrete grid pavers; this performed the best total nitrogen reduction. It was suggested this was due to ammonium being sorbed into the sand, as it was the only pervious pavement constructed with a sand layer.

Collins et al. also tested concrete grid pavers filled with sand. These were 80 mm deep and had 28% void area, with other construction details as per Table 2 and Table 3.

Ammonium concentration in the exfiltrate was on average slightly lower than the other pavements, at 88% less than in the asphalt runoff. Organic nitrogen was more or less the same as from the other pavements, at 28% lower than from asphalt. However, the NO_x-N concentration was only about 60% higher than from asphalt, at 0.46 mg/L. In addition, total nitrogen was 23% lower than from asphalt. This was suggested to be because of the higher surface area of the sand, allowing more microorganism colonisation, as well as possible assimilation of some ammonium before it can be nitrified. It is also suggested this could be because of the sand layer creating anoxic zones for denitrifying bacteria to colonise.

To test the pollutant removal of a PICP pavement, an experimental heat pump system was set up in the UK. The heat pump system is expected to be useful in frost-prone areas, due to higher temperatures promoting microbial decomposition. Crushed rock aggregate with an unsaturated depth of about 400 mm was used in the experimental test rigs (Figure 4) (Scholz & Grabowiecki, 2009). On top of this were proposed pavers with 3 mm pea gravel between. However, the pavers were omitted in the actual test rigs. Geotextile was installed under a 50 mm bedding layer (Table 2 and Table 3).

With an inflow of combined gully pot liquor, tap water and dog faeces, very good nutrient removal was observed. A gully pot is a drainage pit with a sealed base, constructed in a way to trap sediments and hence protect downstream drainage from clogging. It should be noted that this was a laboratory test; therefore, the set flow rate is important. It was stated that 2.2 L of solution was 'slowly' collected from each bin, but the time or method of application was not specified (Table 4). Reductions of 98 to 100% were observed for biological oxygen demand, 95% for ortho-phosphate and 99 to 100% for ammonia. However, nitrate concentration increased by up to four times, due to incomplete nitrification-denitrification. Anoxic conditions would be needed to remove nitrate; the nitrate concentration in the outflow was up to 3.2 mg/L. Increases in total dissolved solids and TSS were recorded due to the aggregate. Pathogen removal was evident but highly variable.

Microbes are believed to be chiefly responsible for the removal of nutrients. Carbon dioxide concentrations were monitored to qualitatively estimate microbial activity levels. The highest CO_2 concentration and hence highest microbial activity was near the geotextile layer, at this point the carbon dioxide concentration was about 1700 ppm. Figure 4 indicates carbon dioxide concentrations, the redder the higher the concentration. While the concentrations were not entirely clear from the graph in the paper, they are believed to be as indicated.

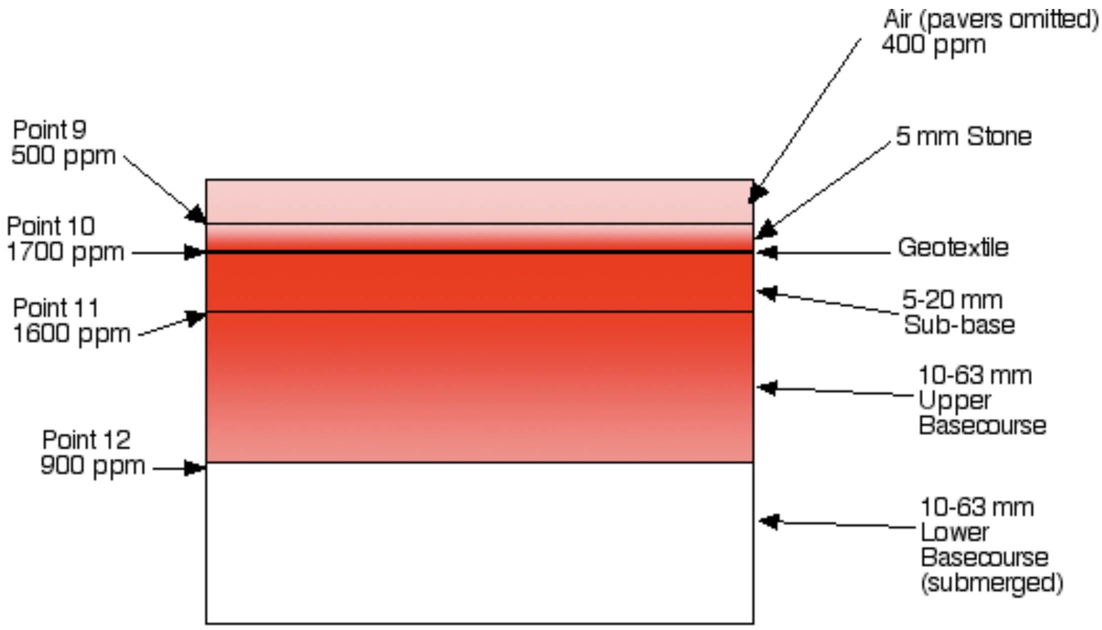

Figure 4: Indicative carbon dioxide levels in outside bin 1 for UK heat pump testing, February 2007

Another investigation involving test rigs looked at suspended solids in more detail. Four rigs with 600 mm by 900 mm bins were used, with 76 mm deep Permeable Interconnecting Concrete Pavers clear spaced a maximum of 12.7 mm and with the gaps filled with 10 mm crushed stone. The same stone was used for a 25 cm bedding layer, on top of 127 mm of 16 mm angular crushed stone and a slotted PVC pipe. Two rigs had a woven geotextile immediately below the bedding. Another four boxes were tested, but with non-woven geotextile instead of woven geotextile (Rowe et al., 2009) (Table 2 and Table 3).

Rain simulation involved 17 L of stormwater being drained onto each pavement in less than an hour. This came from holes drilled in a 19 L bucket (Rowe et al., 2009); therefore the entire pavement was not covered. The simulated rain depth is stated by Rowe et al. as 3 cm, assuming even coverage, but assuming a bucket diameter of 300 mm it would have been more like 270 mm. In Figure 5 the initial, maximum inflow rate is indicated as 1.5 cm/min, or 900 mm/hr (Rowe et al., 2009). This appears inconsistent with the stated loading depth of 3 cm. The inflow rate reduced over time (Rowe et al., 2009). Application occurred twice every workday for 12 weeks. In total, the rain loading was said to be equivalent to 3 years (Rowe et al., 2009), however with the concentrated loading area it was likely to be much higher, but over a small pavement area (Table 4).

The influent contained 18 to 228 mg/L of TSS. TSS removal was strongly correlated to the initial concentration, with R^2 given as over 0.98. For the bins with woven geotextile, this equation was given as

$$TSS_{removed} = 0.95 \times TSS_{input} - 7.4$$

For the bins with no geotextile, this was given as

$$TSS_{removed} = 0.74 \times TSS_{input} - 0.98$$

For these equations TSS is the total suspended solids concentration (input and removed, removed being input minus output) in mg/L. An increase in removal effectiveness was also noticed over time, with clogging taking place (Rowe et al., 2009).

Pervious pavement was tested as a stormwater holding medium (Gomez-Ullate, Novo, Bayon, Hernandez, & Castro-Fresno, 2011). Forty-five parking bays each of dimensions 4.2 m by 2.4 m plan area, with a depth of 0.5 m were installed. Six different surfaces were trialled, one of them being Aquaflow concrete blocks of 80 mm depth. Different types of geotextile were installed, with combinations such that there were bays with no geotextile for each surface type. Clean limestone sub-base was used for all bays. It was not specified whether a bedding layer was incorporated.

Preliminary results were given. Chemical oxygen demand varied from about 2 mg/L to 28 mg/L, depending on rain input, achieving the lowest of the six surface types in many cases. Similarly, total nitrogen varied from about 0.8 mg/L to 1.7 mg/L, roughly median performance. Total phosphorus was also about the median value, ranging from 0.01 to 0.05 mg/L. TSS was in some cases the best of the six surface types, ranging from 15 mg/L to 90 mg/L. Unfortunately these preliminary results cannot readily be compared with the laboratory study by Myers et al. (2007) on rainwater storage in gravel media, because in Gomez-Ullate et al.'s study there are rain events between every testing date that results were given for.

Stormwater treatment efficiency of pervious pavement has been researched outside of Australia; however little has been found applying to Australian conditions. Nonetheless, an article was found focussing on runoff from pervious pavement. A study performed in Sydney found the water quality of runoff from PICP pavement was similar to asphalt, but at the lower end of pollutant levels (Ball & Rankin, 2010). This is particularly relevant to Perth, as the site had sandy soils, like Perth. The PICP used was Rocla Ecoloc. Table 1 gives an overview of Event Mean Concentrations (EMC's):

Table 1: Event mean concentrations for Smith Street, Sydney (Ball & Rankin, 2010)

Constituent	Smith Street EMC range (µg/L)	Smith Street median EMC (µg/L)
Total phosphorus	102-1800	222
Copper	2-18	5
Zinc	19-120	29
Lead	3-40	9
pH	6.3-6.7	-
Conductivity (µS/cm)	29-233	-
TSS (mg/L)	8-77	-

While these results do not compare readily to the pavements used in the United States, due to the different climate, the pollutant concentrations were in fact closer to that of asphalt runoff in the US, with the possible exception of copper, which was closer to that from PICP pavement exfiltrate. However, comparing to runoff in southeast Queensland, only phosphorus was within the range of concentrations for impervious runoff, agreeing with Ball and Rankin's finding that the runoff quality was better than from impervious surfaces.

Runoff hydrology was investigated in detail, it was found that there was an initial loss of about 4 mm and at least 20 mm/hr was required to produce runoff. During runoff periods, the volumetric runoff coefficient was suggested to be 3% (Ball & Rankin, 2010). It is suggested here that the likely reasons for this low result were that the permeability of the soil was 145 mm/hr and the PICP pavement was about a year old at the time of testing. The pavement was released to traffic early 2002 (Shackel, Ball, & Mearing, 2003) and the testing was performed June 2002 to April 2003. A runoff coefficient range of 5 to 35% is more typical for pervious surfaces (American Society of Civil Engineers, 1969). Runoff is generally significantly lower than infiltration for pervious pavement. With higher levels of pollutants expected in runoff than infiltrate from pervious pavement, the lower the runoff, the lower the total exported pollutants.

In summary, pollutant removal of TSS (Table 5) was found to vary linearly with the input concentration (Rowe et al., 2009), which potentially justifies the use of TSS removal percentage as an indicator of performance. Pavements incorporating geotextile fared significantly better at TSS reduction than those without (59-95% compared to 33-60%, excluding the study by Scholz and Grabowiecki (2009) with the submerged base). A major source of variation between studies is the emission of TSS from the pavement itself, typically from the base layers. Reduction varied overall from 32% to 94%. For both laboratory studies providing loading rates, the removal rates are around 90% (with the use of geotextile in both cases), despite Kadurupokune and Jayasuriya (2009) using a loading rate of 90 mm/hr and Rowe et al. (2009) using an effective (concentrated) loading rate of roughly 270 mm/hr. With such similar removal rates despite such different hydraulic loading rates, this implies loading rate has little effect on TSS removal. Rowe et al.'s study demonstrates the effectiveness of using a geotextile layer for removing TSS. Wash tests would greatly assist in removing uncertainty, which was not done in all cases.

The studies by Bean et al. (2007) and Collins et al. (2010) on nutrient reduction gave similar results, especially for ammonia (Table 6). The main difference was in oxidised nitrogen reduction, possibly because of the different testing periods. Sand can improve oxidised nitrogen removal by providing more surface area for microbes, as indicated by Collins et al's study. Scholz and Grabowiecki's study had close to 100% removal of biological oxygen demand, phosphate and ammonia, suggested to be because of highly favourable conditions for bacterial growth in the laboratory compared to the above two field studies. Phosphorus removal appeared to be more influenced by loading rate than the other nutrients, as reflected by the much lower removal rates in Kadurupokune and Jayasuriya's study using a high 90 mm/hr loading rate. Due to the highly variable nature of nutrient reduction, many studies would need to be sourced to confirm findings, each of which will need to detail input concentration of each species, output concentration, loading duration, loading rate, loading history, other inputs, temperature etc.

Metal reduction rates varied significantly (Table 7). Leaching tests would be useful to account for the variation, which Fassman and Blackbourn did provide, but different base materials sorb the metals at different rates and this is believed to have contributed to the variation as well.

All reduction percentages in Table 5 through Table 7 are by concentration, with the values for the study by Kadurupokune and Jayasuriya (2009) calculated. Field reduction rates are relative to asphalt runoff.

Table 2: Surface and General Details of Permeable Pavements with Solid Pavers

		General					Surface			
Author	Date	Testing	Location	[Simulated] Age (years)	Case	Surface Type	Surface Thickness (mm)	Bedding Gravel Size (mm) *	Bedding Thickness (mm)
Kadurupokune and Jayasuriya	2009	Laboratory	Melbourne, Victoria, Australia	1	1 year	C&M Ecotrihex	80	2-5	30
Kadurupokune and Jayasuriya	2009	Laboratory	Melbourne, Victoria, Australia	17	17 years	C&M Ecotrihex	80	2-5	30
Fassman and Blackbourn	2010	Field	Auckland, New Zealand	3	-	Stevenson PICP	80	2-5	25
Brattebo and Booth	2003	Field	Washington, USA	0	As New	UNI Eco-Stone	80	?	?
Brattebo and Booth	2003	Field	Washington, USA	6	6 years	UNI Eco-Stone	80	?	?
Bean et al.	2007	Field	Goldsboro, North Carolina, USA	2	-	UNI Eco-Stone	75	ASTM No. "72" Pea Gravel	75
Collins et al.	2010	Field	Goldsboro, North Carolina, USA	1	Octabrick	Octabrick	80	ASTM No. 78 (2.36-12.5)	100
Collins et al.	2010	Field	Goldsboro, North Carolina, USA	1	Rima	Rima	80	ASTM No. 78 (2.36-12.5)	100
Collins et al.	2010	Field	Washington, USA	1	CGP	CGP filled with sand	80	Sand over ASTM No. 78 (2.36-12.5)	25, 100
Scholz and Grabowiecki	2009	Laboratory (rig 1, inside)	Edinburgh, Scotland, UK	0	-	None	None	5	50
Rowe et al.	2009	Laboratory	Edison, New Jersey, USA	0	Geo.	PICP	80	10	25
Rowe et al.	2009	Laboratory	Edison, New Jersey, USA	0	No Geo.	PICP	80	10	25

* Material used to fill gaps in and between pavers is the same as the bedding material in all cases.

Table 3: Base and Geotextile Details for Permeable Pavements with Solid Pavers

			Base			
Author	Case	Geotextile	Base Gravel Size (mm)	Base Thickness (mm)	Sub-base Gravel Size (mm)	Sub-base Thickness (mm)
Kadurupokune and Jayasuriya	1 year	1000 gauge PE	5-20	200	None	None
Kadurupokune and Jayasuriya	17 years	1000 gauge PE	5-20	200	None	None
Fassman and Blackbourn	-	Yes, unspecified	?	150	?	230
Brattebo and Booth	As New	Yes, unspecified	?	?	?	?
Brattebo and Booth	6 years	Yes, unspecified	?	?	?	?
Bean et al.	-	None	ASTM No. 57 (4.75 to 25)	200	None	None
Collins et al.	Octabrick	None	ASTM No. 5 (12.5-25.0)	250	None	None
Collins et al.	Rima	None	ASTM No. 5 (12.5-25.0)	250	None	None
Collins et al.	CGP	None	ASTM No. 5 (12.5-25.0)	225	None	None
Scholz and Grabowiecki	-	2 mm thick Inbitex + impermeable composite	5-20	100	10-63	500 (250 submerged)
Rowe et al.	Geo.	Woven, 0.425 mm apparent opening size	16	127	None	None
Rowe et al.	No Geo.	None	16	127	None	None

Table 4: Hydraulic Details for Permeable Pavements with Solid Pavers

Author	Case	[Simulated] Rainfall Intensity (mm/hr)	[Simulated] Rainfall Duration (hr)	[Simulated] Rainfall Depth (mm)
Kadurupokune and Jayasuriya	1 year	90	1.5	135
Kadurupokune and Jayasuriya	17 years	90	1.5	135
Fassman and Blackbourn	-	0.3-79.2	?	2-152
Brattebo and Booth	As New	0-7.4	≤72	13-121
Brattebo and Booth	6 years	0-7.4	≤72	13-121
Bean et al.	-	?	?	?
Collins et al.	Octabrick	?	?	3.1-88.9
Collins et al.	Rima	?	?	3.1-88.9
Collins et al.	CGP	?	?	3.1-88.9
Scholz and Grabowiecki	-	?	?	5.2 (calculated)
Rowe et al.	Geo.	0-900 (1.5 cm/min) reducing	~1	30 (quoted, likely closer to 300)
Rowe et al.	No Geo.	0-900 (1.5 cm/min) reducing	~1	30 (quoted, likely closer to 300)

Table 5: TSS Reduction Efficiencies of Permeable Pavements with Solid Pavers

Author	Case	TSS Reduction
Kadurupokune and Jayasuriya	1 year	94%
Kadurupokune and Jayasuriya	17 years	95%
Fassman and Blackbourn	-	59%
Brattebo and Booth	As New	-
Brattebo and Booth	6 years	-
Bean et al.	-	33%
Collins et al.	Octabrick	-
Collins et al.	Rima	-
Collins et al.	CGP	-
Scholz and Grabowiecki	-	32%
Rowe et al.	Geo.	>90%
Rowe et al.	No Geo.	>60%

Table 6: Nutrient Reduction Efficiencies of Permeable Pavements with Solid Pavers

		Nutrient											
Author	Case	DOC	COD	BOD	Motor Oil	TP	TN	TKN	NH_4-N	ON	NO_2-N	NO_3-N	NO_x-N
Kadurupokune and Jayasuriya	1 year	-	-	-	86%	53%	63%	-	-	-	-	-	-
Kadurupokune and Jayasuriya	17 years	-	-	-	97%	22%	12%	-	-	-	-	-	-
Fassman and Blackbourn	-	-	-	-	-	-	-	-	-	-	-	-	-
Brattebo and Booth	As New	-	-	-	ND	-	-	-	-	-	-	-	-
Brattebo and Booth	6 years	-	-	-	ND	-	-	-	-	-	-	-	-
Bean et al.	-	-	-	-	-	63%	42%	60%	84%	50%	-	-	-47%
Collins et al.	Octabrick	-	-	-	-	-	-40%	49%	85%	30%	-	-	-331%
Collins et al.	Rima	-	-	-	-	-	-11%	49%	85%	30%	-	-	-210%
Collins et al.	CGP	-	-	-	-	-	23%	49%	88%	28%	-	-	-59%
Scholz and Grabowiecki	-	-	-	99%	-	99%	-	-	100%	-	-	-	-55%
Rowe et al.	Geo.	-	-	-	-	-	-	-	-	-	-	-	-
Rowe et al.	No Geo.	-	-	-	-	-	-	-	-	-	-	-	-

ND: Not detected

Table 7: Metal Reduction Efficiencies of Permeable Pavements with Solid Pavers

		Metal											
Author	Case	Cd_T	Cd_D	Cd_P	Cu_T	Cu_D	Cu_P	Pb_T	Pb_D	Pb_P	Zn_T	Zn_D	Zn_P
Kadurupokune and Jayasuriya	1 year	-	ND	-	-	94%	-	-	ND	-	-	-46%	-
Kadurupokune and Jayasuriya	17 years	-	ND	-	-	96%	-	-	ND	-	-	25%	-
Fassman and Blackbourn	-	-	-	-	50%	49%	51%	-	-	-	80%	96%	62%
Brattebo and Booth	As New	-	-	-	-	-79%	-	-	-	-	-	44%	-
Brattebo and Booth	6 years	-	-	-	-	89%	-	-	ND	-	-	69%	-
Bean et al.	-	-	-	-	62%	-	-	-	-	-	88%	-	-
Collins et al.	Octabrick	-	-	-	-	-	-	-	-	-	-	-	-
Collins et al.	Rima	-	-	-	-	-	-	-	-	-	-	-	-
Collins et al.	CGP	-	-	-	-	-	-	-	-	-	-	-	-
Scholz and Grabowiecki	-	-	-	-	-	-	-	-	-	-	-	-	-
Rowe et al.	Geo.	-	-	-	-	-	-	-	-	-	-	-	-
Rowe et al.	No Geo.	-	-	-	-	-	-	-	-	-	-	-	-

T subscript refers to total concentration, similarly D means dissolved and P means particulate.

ND: Not detected

2.2.2 Plastic Reinforced Gravel

One of the four pavements tested by Brattebo and Booth was Gravelpave, a plastic grid filled with gravel, with a general setup as described in Table 8. Neither geotextile nor base details were reported, but hydraulic details were given (Table 9). TSS was not tested for. Motor oil was never detected in the exfiltrate, even though 0.164 mg/L was detected in the asphalt runoff. The same applied for the other three pavements (Section 2.2.1). In 1996, the dissolved copper concentration from the exfiltrate was on average 79% lower than the asphalt runoff, but the dissolved zinc concentration was 25% higher, suggesting the gravel was a source of zinc (Table 10). In 2001-2002, copper was 89% lower than asphalt and zinc 62% (Brattebo & Booth, 2003). All pollutant reduction percentages are by concentration.

Table 8: Surface and General Details for Plastic Reinforced Gravel

		General				Surface			
Author	Date	Testing	Location	[Simulated] Age (years)	Case	Surface Type	Surface Thickness (mm)	Bedding Gravel Size (mm)	Bedding Thickness (mm)
Brattebo and Booth	2003	Field	Washington, USA	0	As New	Gravelpave	25 (manuf.)	?	?
Brattebo and Booth	2003	Field	Washington, USA	6	6 years	Gravelpave	25 (manuf.)	?	?

Table 9: Hydraulic Details for Plastic Reinforced Gravel

Author	Case	Rainfall Intensity (mm/hr)	Rainfall Duration (hr)	Rainfall Depth (mm)
Brattebo and Booth	As New	0-7.4	≤72	13-121
Brattebo and Booth	6 years	0-7.4	≤72	13-121

Table 10: Metal Reduction Efficiencies of Plastic Reinforced Gravel

		Metal											
Author	Case	Cd_T	Cd_D	Cd_P	Cu_T	Cu_D	Cu_P	Pb_T	Pb_D	Pb_P	Zn_T	Zn_D	Zn_P
Brattebo and Booth	As New	-	-	-	-	79%	-	-	-	-	-	-25%	-
Brattebo and Booth	6 years	-	-	-	-	89%	-	-	ND	-	-	62%	-

T subscript refers to total concentration, similarly D means dissolved and P means particulate.

ND: Not detected

2.2.3 Reinforced Turf

Turf can be used as a pervious pavement surface, as long as it is reinforced by some structure. Two of the pavements tested by Brattebo and Booth (2003) were Grasspave, which is plastic reinforced turf, and Turfstone, which are concrete grid pavers with about 60% impervious area filled with turf. Construction details are given in Table 11. No base or geotextile details were reported, but Table 12 gives hydraulic details. TSS was not tested for. In 1996, on average the Grasspave discharged 138% more dissolved copper by concentration (at 21.4 μg/L) than the asphalt, making Grasspave the largest copper source of the four pavements, whereas the Turfstone discharged 82% less compared to asphalt. As for zinc, the concentration was below the detection limit of 5 μg/L from the Grasspave and 63% lower than asphalt for the Turfstone. In 2001-2002, the copper coming from the Grasspave was below the detection limit of 1 μg/L, providing the best reduction of all four pavements, and from the Turfstone was 83% lower than the asphalt. As for zinc, the concentration was 39% lower from the Grasspave than from the asphalt, for Turfstone 64% lower.

A novel approach of creating a structural soil consisting of turf and gravel was investigated (Sloan, Hegemann, & George, 2008). Nine combinations of varying contents of expanded shale, sand, sphagnum peat moss and zeolites were used. These were placed in pots 140 mm diameter, 110 mm depth and functional volume of 1.15 L. The expanded shale overall was graded 1 to 6 mm; sand 0.25 mm to 1.0 mm. Zeolites used were clinoptilolite as $Ca_x(Na, K)_{6-2x}(Al_6Si_{30}O_{72})$, to retain fertilizers and heavy metals.

Bermudagrass sprigs were grown in all the pots. Fertiliser was initially added at 0.48, 0.21 and 0.40 g per pot of nitrogen, phosphorus and potassium respectively, then every 175 days with slow release fertiliser at 1.08, 0.16 and 0.60 g per pot of nitrogen, phosphorus and potassium respectively. Grass was clipped to 38 mm height whenever necessary. After the grass was established, 10 mL of solution containing 250 µg of each of dissolved cadmium, copper, lead and zinc were added to each pot, then 250 mL of deionised water was leached through each pot at 1, 3, 7 and 14 days after metal addition. Leached metals were mostly undetected, so 30 mL of solution, 750 µg of each metal and 375 mL of deionised water were tried, with similar result. Two months later, 10 mL of solution containing 10 mg of each metal and 375 mg of deionised water was leached through after 1 day and 4 days.

Phosphorus leaching was the most influenced by peat moss content, where more peat moss meant more phosphorus leaching. The widely graded shale (1-6 mm) resulted in the least phosphorus leaching, possibly due to the increased particle surface area and lower permeability. Zeolites increased phosphorus levels in exfiltrate. Generally, phosphorus levels decreased over time, but in some cases increased. After the first 5 days, concentrations ranged from about 0.2 to 1.3 mg/L but after 162 days ranged from about 0.2 to 0.4 mg/L.

All the mixes performed well at removing metals. After 1 mg total addition of each metal the leachate amount of each metal was 'very low'. After 11 mg addition, the most total cadmium collected was about 27 µg. Similarly; at most 28 µg of copper, 17 µg of lead and 84 µg of zinc was collected. The role of peat moss, zeolites and shale particle size distribution are judged to be complex at best. Depending on the heavy metal, different combinations performed best. For cadmium, this was no peat moss, some zeolites and shale graded 1-6 mm. For copper, this was possibly 10% peat moss, 10% zeolites and 1-3 mm graded shale. For lead, 10% peat moss, 10% zeolites and 1-6 mm graded shale. Zinc removal worked best at a different composition again of some peat moss, no zeolites and 1-6 mm graded shale. It was reported that the 1-6 mm graded shale leached the least solution, as well as mixes containing peat moss. Analysis on a concentration basis may have yielded clearer results. Metal concentrations in grass clippings clearly increased with metal addition, but were not significantly different between pot mixes.

Two of the surface types tested by Gomez-Ullate et al. (2011) were plastic and concrete reinforced turf (see Section 2.2.1). As discussed earlier, the sub-base was used to store rainwater. The concrete reinforced turf fared similarly to the concrete block pavement in terms of chemical oxygen demand, with concentrations ranging between 5 mg/L and 37 mg/L. It also had similar total nitrogen concentrations to the concrete block pavement, ranging from about 0.2 mg/L to 3 mg/L, achieving the lowest concentration on three occasions. Total phosphorus results were mixed, ranging from the lowest recorded value of 0.005 mg/L to 0.045 mg/L. It performed fairly average in terms of TSS, ranging from 12 mg/L to 150 mg/L.

Pollutant performance by the plastic reinforced turf was relatively poor. Chemical oxygen demand ranged between 7 mg/L and the second highest recorded value of 46 mg/L, total nitrogen 1 mg/L and the highest recorded value of 8 mg/L, total phosphorus 0.005 mg/L and the highest recorded value of about 0.2 mg/L and TSS between 10 mg/L and the highest recorded value of nearly 600 mg/L. These substances were reasoned to leach from the turf.

To summarise, there are no specific results for TSS reduction, but the study by Gomez-Ullate et al. (2011) indicates the soil within the surface layer may be a significant source of TSS in reinforced soil structures. Additional layers of geotextile may be necessary to mitigate this.

There is insufficient information sourced to compare nutrient reduction between studies employing reinforced turf. Nevertheless, motor oil reduction was significant, as indicated in the study by Brattebo and Booth (2003). In addition, the results from the study by Sloan et al. (2008) on expanded shale reinforced turf are promising, with benign concentrations of phosphate in the leachate. Use of a leaching test on the fertiliser alone would have clarified whether this is the case.

Metal reduction results were again highly variable (Table 13). Leaching tests would have helped account for the variability, to determine how much was coming from the pavement structure. Sloan et al.'s study gave promising results here too, but presenting the results by concentration as well as load would have given a clearer picture, due to the different water absorption rates.

All reduction percentages in Table 13 are by concentration. Field reduction rates relate to asphalt runoff.

Table 11: Surface and General Details for Reinforced Turf

		General				Surface			
Author	Date	Testing	Location	[Simulated] Age (years)	Case	Surface Type	Surface Thickness (mm)	Bedding Gravel Size (mm)	Bedding Thickness (mm)
Brattebo and Booth	2003	Field	Washington, USA	0	As New	Grasspave (plastic)	25 (manuf.)	?	?
Brattebo and Booth	2003	Field	Washington, USA	6	6 years	Grasspave (plastic)	25 (manuf.)	?	?
Brattebo and Booth	2003	Field	Washington, USA	0	As New	Turfstone (concrete)	90 (manuf.)	?	?
Brattebo and Booth	2003	Field	Washington, USA	6	6 years	Turfstone (concrete)	90 (manuf.)	?	?

Table 12: Hydraulic Details for Reinforced Turf

Author	Case	Rainfall Intensity (mm/hr)	Rainfall Duration (hr)	Rainfall Depth (mm)
Brattebo and Booth	As New	0-7.4	≤72	13-121
Brattebo and Booth	6 years	0-7.4	≤72	13-121
Brattebo and Booth	As New	0-7.4	≤72	13-121
Brattebo and Booth	6 years	0-7.4	≤72	13-121

Table 13: Metal Reduction Efficiencies of Reinforced Turf

		Metal											
Author	Case	Cd_T	Cd_D	Cd_P	Cu_T	Cu_D	Cu_P	Pb_T	Pb_D	Pb_P	Zn_T	Zn_D	Zn_P
Brattebo and Booth	As New	-	-	-	-	-138%	-	-	-	-	-	ND	-
Brattebo and Booth	6 years	-	-	-	-	ND	-	-	ND	-	-	39%	-
Brattebo and Booth	As New	-	-	-	-	82%	-	-	-	-	-	63%	-
Brattebo and Booth	6 years	-	-	-	-	83%	-	-	ND	-	-	64%	-

T subscript refers to total concentration, similarly D means dissolved and P means particulate.

ND: Not detected

2.2.4 Porous Concrete

Porous concrete can remove metals, probably from a combination of precipitation, ion exchange and other sorption mechanisms (Calkins et al., 2010). This study found that the aggregate, sand and cement contribute, with cement contributing the most. They also found that adding about 1.2 kg/m^3 of nylon fibre increases the rate of sorption of copper, possibly by increasing the surface area available.

Collins et al. (2010) performed a study on four pervious pavements, including porous concrete. Construction details are given in Table 14 and Table 15, hydraulic details in Table 16. Like the other three pavements, the concentration of ammonia in the exfiltrate was on average about 85% less than in the asphalt runoff. The organic nitrogen was only 18% lower compared to around 30% lower for the other pavements, but considering it was 0.5 mg/L compared to around 0.45 mg/L for the other pavements, it was deemed insignificant in the discussion. The NO_x-N concentration increased similarly to the PICP pavements and total nitrogen had about the same concentration as from asphalt runoff.

One of the surface types tested by Gomez-Ullate et al. (2011) was porous concrete (see Section 2.2.1). As discussed earlier, the sub-base was used to store rainwater. For chemical oxygen demand, it performed the worst out of the six surface types, with concentrations ranging from 5 mg/L to over 50 mg/L. This was reasoned to be because of an additive in the concrete. Total nitrogen was at about 1.2 mg/L to 4.6 mg/L. Total phosphorus was at 0.01 mg/L to 0.10 mg/L. TSS was at 30 mg/L to 160 mg/L.

In summary, porous concrete is a good surface layer for iron oxide coated sand (OCS), an engineered material. This is because the concrete raises the pH of the influent and makes the OCS more effective at capturing dissolved metals (J. J. Sansalone, 1999) (see Section 2.2.7). The OCS is installed below the porous concrete.

Porous concrete appears to perform similarly to other pavement types in terms of nutrient reduction, according to the study by Collins et al. (2010). Pollutant reduction percentages in Table 17 are by concentration and are relative to asphalt runoff.

Cement in porous concrete aids in the removal of heavy metals, as evidenced in the study by Calkins et al. (2010).

Table 14: Surface and General Details for Porous Concrete

		General				Surface			
Author	Date	Testing	Location	[Simulated] Age (years)	Case	Surface Type	Surface Thickness (mm)	Bedding Gravel Size (mm)	Bedding Thickness (mm)
Collins et al.	2010	Field	Washington, USA	1	-	Porous concrete	150	ASTM No. 78 (2.36-12.5)	50

Table 15: Base and Geotextile Details for Porous Concrete

			Base			
Author	Case	Geotextile	Base Gravel Size (mm)	Base Thickness (mm)	Sub-base Gravel Size (mm)	Sub-base Thickness (mm)
Collins et al.	-	None	ASTM No. 5 (12.5-25.0)	230	None	None

Table 16: Hydraulic Details for Porous Concrete

Author	Case	Rainfall Intensity (mm/hr)	Rainfall Duration (hr)	Rainfall Depth (mm)
Collins et al.	-	?	?	3.1-88.9

Table 17: Nutrient Reduction Efficiencies of Porous Concrete

		Nutrient											
Author	Case	DOC	COD	BOD	Motor Oil	TP	TN	TKN	NH_4-N	ON	NO_2-N	NO_3-N	NO_x-N
Collins et al.	-	-	-	-	-	-	-2%	42%	85%	18%	-	-	-152%

2.2.5 *Porous Asphalt*

Asphalt can be constructed in such a way as to provide voids to let rainwater through, termed porous asphalt. A porous asphalt parking area was constructed in 2002 at the University of Rhode Island campus (Boving et al., 2008). On the subgrade 50 mm of sand was constructed, followed by geotextile, 600 mm of cobble-sized crushed granite, 300 mm of pebble-sized gravel then 150 mm of non-polymer modified porous asphalt (from bottom to top). Water sampling probes were installed in-situ just above the geotextile (shallow) and 600 mm below the geotextile (deep) (Table 18 and Table 19). Runoff samples were also collected from a nearby impervious asphalt car park. Only monthly rainfall data was given (Table 20).

A tracer test was performed to determine the actual attenuation of some pollutants. About 53.5 L of solution was pumped through over 38 days, during a wet month. There was a 27% reduction by mass of nitrate, which had an initial concentration of 990 mg/L. A 27% reduction of phosphate was also observed, with an initial concentration of 382 mg/L. Similarly, copper, zinc and sodium salicylate reduced by 92.5%, 92.2% and 52.2% respectively, with initial concentrations of 500 mg/L, 1300 mg/L and 1587 mg/L respectively.

Unfortunately, insufficient data was collected from the impervious carpark for comparison. What was measured was a concentration of 1.4 µg/L of polyaromatic hydrocarbons (PAH) in September 2004, compared to about 1.7-1.8 µg/L from the shallow probes at about the same time. Insufficient water was collected from the deep probes and a probe located outside the car park.

Porous asphalt was investigated in China (Xie, Xu, & Wang, 2009). Both continuously graded and gap graded asphalt were studied. Cement stabilised, 13.2 mm single sized bedding was used. Thicknesses were not given and a complete pavement setup comprising all the usual layers was not investigated (Table 18 and Table 19). Therefore results can only be compared within this one study. Also, hydraulic information such as loading rates was not provided (Table 20).

Individual layers were investigated. Gap graded asphalt removed far more chemical oxygen demand (COD) than continuously graded, at 54% removal compared to 22% removal. Initial concentrations were not given for the tests on individual layers. Both gap graded and continuously graded asphalt performed similarly for TSS, at about 63-65% removal. Layers were also tested in combination, but not forming a complete pavement. Initial pollutant concentrations of 180 mg/L COD and 4000 mg/L TSS (both calculated) were used. For the asphalt and bedding layer combination, gap graded asphalt performed slightly better at both COD and TSS removal in this test. The enhanced COD removal was reasoned to be because of the increase in air voids, providing more oxygen to bacteria.

Porous asphalt samples were tested for TSS removal by Kuang, Sansalone, Gnecco, Berretta, and Lanza (2007). Runoff containing both sand and silt from Baton Rouge, Louisiana was infiltrated through 97 mm deep samples at an unspecified flow rate. The samples removed nearly all the 'sediment' sized particles and 50% of the suspended particles, at a total rate of 80%. The initial concentrations were not given.

One of the surface types tested by Gomez-Ullate et al. (2011) was porous asphalt (see Section 2.2.1). As discussed earlier, the sub-base was used to store rainwater. Chemical oxygen was fairly average at 5 mg/L to 40 mg/L. Total nitrogen was probably the best out of the six surface types, at 0.5 mg/L to 1.5 mg/L. Total phosphorus ranged from 0.005 mg/L to 0.065 mg/L. TSS ranged from 20 mg/L to the second highest recorded value of over 500 mg/L. The reason for this is unknown.

Permeable friction course (PFC) is similar to porous asphalt pervious pavement, except the porous asphalt is constructed atop an impermeable membrane over conventional impervious basecourse and subbase. While this compromises the additional storage and infiltration capacity of porous asphalt pervious pavement, it still provides superior hydraulic performance to conventional impervious asphalt (Barrett, Klenzendorf, Eck, & Charbeneau, 2009).

A highway with a PFC was investigated in Austin, Texas (Barrett et al., 2009). The depth of friction course was not given, neither was hydrological data (Table 18, Table 19 and Table 20). Stormwater pollutant concentrations were measured before and after the installation of the friction course. TSS was most effectively removed, at around 90% less than originally. Sediment bound pollutants were lower as well, at over 80% lower for bound phosphorus, about 85% for bound copper, 88% for bound lead and over 90% for bound zinc. Dissolved pollutants were not significantly reduced; the best example was dissolved zinc at 30-50% reduction. This is expected to be because of the flow regime preventing adequate microbe establishment to remove nutrients and lack of available calcium from gravel to remove dissolved metals, compared to pervious pavement.

A study in North Carolina found TSS in stormwater exfiltrate from PFC to be lower than fully impermeable asphalt (Winston, Hunt, & Wright, 2010). The average TSS was 10-31 mg/L depending on testing location, compared to over 100 mg/L from impervious highways from other studies. However, TKN, NO_x-N, total nitrogen, ammonium, organic nitrogen and total phosphorus concentrations were all similar to from an impervious carpark in the same state. The depth of friction course was about 50 mm.

In summary, not enough information on TSS reduction was found for porous asphalt (Table 21), but PFC removed over 90% in the study by Barrett et al. (2009).

From the limited data collected nutrient reduction ability appeared to be similar to pavement with gravel filled gaps, considering the study by Boving et al. (2008) looked at pollutant load rather than concentration with a tracer test, but PFC fared poorly due to the limited opportunity for bacterial growth from the lateral flow (Section 2.2.1 Table 6, Table 22).

Similarly, metal reduction of porous asphalt is probably comparable to pavement with gravel filled gaps. Dissolved metal removal of PFC is poor because of the lack of available material for it to sorb to (Section 2.2.1 Table 7, Table 23).

Pollutant reduction percentages in Table 21 through Table 23 are by concentration. Field reduction rates are relative to asphalt runoff.

Table 18: Surface and General Details for Porous Asphalt

		General				Surface			
Author	Date	Testing	Location	[Simulated] Age (years)	Case	Surface Type	Surface Thickness (mm)	Bedding Gravel Size (mm)	Bedding Thickness (mm)
Boving et al.	2008	Field	Rhode Island, USA		-	Porous asphalt	150	None	None
Xie et al.	2009	Laboratory	Harbin, Heilongjiang, China	0	Cont. HMA	Porous asphalt – continuous graded	?	? (Cement stabilised)	?
Xie et al.	2009	Laboratory	Harbin, Heilongjiang, China	0	Gap HMA	Porous asphalt – gap graded	?	? (Cement stabilised)	?
Barrett et al.	2009	Field	Austin, Texas, USA	3	-	PFC (original site)	?	NA	NA

Table 19: Base and Geotextile Details for Porous Asphalt

			Base			
Author	Case	Geotextile	Base Gravel Size (mm)	Base Thickness (mm)	Sub-base Gravel Size (mm)	Sub-base Thickness (mm)
Boving et al.	-	Below shallow sampler (which was below sub-base), unspecified	'Pebble sized'	300	'Cobble sized'	600
Xie et al.	Cont. HMA	None	None	None	None	None
Xie et al.	Gap HMA	None	None	None	None	None
Barrett et al.	-	NA	NA	NA	NA	NA

Table 20: Hydraulic Details for Porous Asphalt

Author	Case	[Simulated] Rainfall Intensity (mm/hr)	[Simulated] Rainfall Duration (hr)	[Simulated] Rainfall Depth (mm)
Boving et al.	-	?	?	42-215 (per month)
Xie et al.	Cont. HMA	?	?	?
Xie et al.	Gap HMA	?	?	?
Barrett et al.	-	?	?	?

Table 21: TSS Reduction Efficiencies of Porous Asphalt

Author	Case	TSS
Boving et al.	-	-
Xie et al.	Cont. HMA	36%
Xie et al.	Gap HMA	40%
Barrett et al.	-	92%

Table 22: Nutrient Reduction Efficiencies of Porous Asphalt

		Nutrient												
Author	Case	DOC	COD	BOD	Motor Oil	TP	TN	TKN	NH_4-N	ON	NO_2-N	NO_3-N	NO_x-N
Boving et al.	-	-	-	-	-	27%	-	-	-	-	-	27%	-
Xie et al.	Cont. HMA	-	43%	-	-	-	-	-	-	-	-	-	-
Xie et al.	Gap HMA	-	56%	-	-	-	-	-	-	-	-	-	-
Barrett et al.	-	-	-	-	-	40%	-	9%	-	-	-	-	9%

Table 23: Metal Reduction Efficiencies of Porous Asphalt

		Metal											
Author	Case	Cd_T	Cd_D	Cd_P	Cu_T	Cu_D	Cu_P	Pb_T	Pb_D	Pb_P	Zn_T	Zn_D	Zn_P
Boving et al.	-	-	-	-	-	93%	-	-	-	-	-	92%	-
Xie et al.	Cont. HMA	-	-	-	-	-	-	-	-	-	-	-	-
Xie et al.	Gap HMA	-	-	-	-	-	-	-	-	-	-	-	-
Barrett et al.	-	-	-	-	49%	-78%	84%	88%	ND	88%	82%	51%	93%

ND: Not detected

T subscript refers to total concentration, similarly D means dissolved and P means particulate.

2.2.6 Role of Sub-base Aggregate

The type of sub-base can also influence stormwater pollutant removal performance. A laboratory study comparing calcite and dolomite was performed (Myers et al., 2007). This was performed mainly to investigate the effect of storage in the sub-base, so the surface layer was omitted. Bedding of 5 mm dolomite underlain with A34 Bidim geotextile was provided for all test rigs, followed by sub-base of either calcite or dolomite gravel (Table 24 and Table 25). Input solution used had organic material from dried leaves, 1.05 mg/L of phosphate (as phosphorus), 0.1825 mg/L copper, 0.748 mg/L zinc and 0.9432 mg/L lead. A control rig contained only this solution.

UV absorbance and other parameters were used to qualitatively indicate the organic content of the water. The UV absorbance started at 0.3 and decreased to 0.1 for the dolomite and just less than 0.2 for the control and calcite after 144 hours. Very little change had occurred after 2 hours. Total nitrogen reduced by 18% in the calcite and 67% in the dolomite rigs, the time period was not clearly specified but taken to be 144 hours. Similarly, phosphate reduced by 95%. After just 2 hours, zinc dropped to around 0.1 mg/L in both types of gravel but slightly lower in the dolomite. Similar results were seen for lead and copper. It was demonstrated that dolomite is more reactive, but no reason was given for the apparent superior nutrient removal ability of dolomite (Myers et al., 2007).

An investigation into four different sub-base materials was also performed (Pratt, Mantle, & Scholfield, 1995). The base materials tested were 10 mm rounded gravel, 40 mm blast furnace slag, 5-40 mm granite and 5-40 mm carboniferous limestone. Overall, each pavement tested had concrete block pavers filled with gravel. These were placed on top of gravel bedding, which was underlain by geotextile, with the base beneath this. The base was constructed on an impermeable membrane with a subsoil collection pipe. The pavements were installed in 1986.

Suspended solids were suggested to come almost entirely from the pavement structure itself, with solids from stormwater trapped within the upper layers of the pavements. Concentrations of lead in the output declined over time, this was said to be because it was in fact leaching from the aggregate. The limestone initially emitted the most TSS, at 164 to 615 mg/L in April 1987. The other aggregates emitted 11-370 mg/L. In two storm events following, in August and October, the TSS output from all pavements were broadly the same at 4-46 mg/L. Blast furnace slag consistently emitted the most lead, initially at 81-120 µg/L. Gravel performed the best, with 22-36 µg/L in the exfiltrate for the same storm. An impermeable asphalt control surface was not tested.

Performance of different gravel layers was also investigated by Xie et al. (2009). The layers tested included 13.2 mm cement stabilised aggregate, 13.2 mm basecourse and a sand-gravel bed course, with 4.75 mm gravel. Unfortunately thicknesses were not given (Table 24 and Table 25). Hydraulic loading rates were not provided either. Layers were individually tested, with initial concentrations not given. The bedding removed 48% of chemical oxygen demand (COD), the basecourse 42% and the sand gravel layer 36%. As for suspended solids, the bedding was the only gravel layer to achieve a net removal, with the other two layers contributing TSS due to the presence of attached fine particles during construction.

Layers were also tested in combination. The basecourse overlaid the sand-gravel layer in one test and in another fibre (taken to mean geotextile) was added, the location of this geotextile not specified. With initial concentrations of COD and TSS of 180 mg/L and 2000 mg/L respectively (calculated), the combination without geotextile removed 46% of COD and increased TSS by 108%, with the gravel layers being a source of fines as stated earlier. With geotextile, it performed much better at 65% removal of COD and a net removal of TSS at 51%. Geotextile acts as a filter and provides significant surface area for microbial growth, explaining the improvement.

In summary, gravel layers in pervious pavement, including the sub-base, can be a substantial source of TSS (Xie et al., 2009). Pratt et al. (1995) found that this can be particularly the case for limestone (Table 26).

As for nutrient removal, the type of aggregate used can also have an effect. Myers et al. (2007) found that dolomite is substantially better than calcite at aiding the reduction of UV absorbance (related to organic carbon), total nitrogen and total phosphorus, at least when using the sub-base as a storage layer (Table 27).

Gravel layers can either be a source of or a sink for metals. Leaching tests are required to clarify for each case (Section 2.2.1). Sorption mechanisms can include binding to calcium carbonate and precipitation through an increased pH, or combinations of the two (Myers et al., 2007). Little information was found for individual layers, but when a gravel layer is used as storage, significant reductions can be achieved (Table 28).

Pollutant reductions in Table 26 through Table 28 are by concentration.

Table 24: Surface and General Details for Different Bases

		General				Surface			
Author	Date	Testing	Location	[Simulated] Age (years)	Case	Surface Type	Surface Thickness (mm)	Bedding Gravel Size (mm)	Bedding Thickness (mm)
Myers et al.	2007	Laboratory (as reservoir)	Adelaide, South Australia	0	Calcite	None	None	Dolomite, 5	?
Myers et al.	2007	Laboratory (as reservoir)	Adelaide, South Australia	0	Dolomite	None	None	Dolomite, 5	?
Xie et al.	2009	Laboratory	Harbin, Heilongjiang, China	0	No Geo.	Base, sand-gravel	None	None	None
Xie et al.	2009	Laboratory	Harbin, Heilongjiang, China	0	Geo.	Base, sand-gravel, geotextile	None	None	None

Table 25: Construction and Geotextile Details for Different Bases

			Base			
Author	Case	Geotextile	Base Gravel Size (mm)	Base Thickness (mm)	Sub-base Gravel Size (mm)	Sub-base Thickness (mm)
Myers et al.	Calcite	A34 Bidim	? (Calcite)	?	None	None
Myers et al.	Dolomite	A34 Bidim	? (Dolomite)	?	None	None
Xie et al.	No Geo.	None	13.2	?	4.75 + sand	?
Xie et al.	Geo.	Yes, unspecified, location unknown	13.2	?	4.75 + sand	?

Table 26: TSS Removal Efficiencies of Different Bases

Author	Case	TSS
Myers et al.	Calcite	-
Myers et al.	Dolomite	-
Xie et al.	No Geotextile	-108%
Xie et al.	Geotextile	51%

Table 27: Nutrient Removal Efficiencies of Different Bases

		Nutrient											
Author	Case	DOC	COD	BOD	Motor Oil	TP	TN	TKN	NH_4-N	ON	NO_2-N	NO_3-N	NO_x-N
Myers et al.	Calcite	-	-	-	-	95%	18%	-	-	-	-	-	-
Myers et al.	Dolomite	-	-	-	-	95%	67%	-	-	-	-	-	-
Xie et al.	No Geo.	-	46%	-	-	-	-	-	-	-	-	-	-
Xie et al.	Geo.	-	65%	-	-	-	-	-	-	-	-	-	-

Table 28: Metal Removal Efficiencies of Different Bases

		Metal											
Author	Case	Cd_T	Cd_D	Cd_P	Cu_T	Cu_D	Cu_P	Pb_T	Pb_D	Pb_P	Zn_T	Zn_D	Zn_P
Myers et al.	Calcite	-	-	-	~97%	-	-	~97%	-	-	97%	-	-
Myers et al.	Dolomite	-	-	-	~99%	-	-	~99%	-	-	99%	-	-
Xie et al.	No Geo.	-	-	-	-	-	-	-	-	-	-	-	-
Xie et al.	Geo.	-	-	-	-	-	-	-	-	-	-	-	-

T subscript refers to total concentration, similarly D means dissolved and P means particulate.

2.2.7 Enhancing Treatment Efficiency

Additives to further enhance treatment efficiency were investigated. Prof. Simon Beecham in South Australia is developing pervious pavement systems. Depending on requirements, ferrous hydroxide is specified for additional metal removal and activated carbon for greater nutrient removal (Salleh, 2006).

Similar to ferrous hydroxide, oxide coated sand (OCS) – modified silica sand prepared by mixing silica sand with ferric nitrate and evaporating the solution to dryness was tested (J. J. Sansalone, 1999). The configuration investigated was a partial exfiltration trench, a trench along side a road pavement filled with granular material, capable of both infiltrating and conveying stormwater. Trench dimensions of 300 mm wide by 900 mm deep, with a sand porosity of 0.37 and a road pavement width of 20 m and length of 15 m in Cincinnati, Ohio, United States were used (Table 29 and Table 30).

OCS is good at sorbing heavy metals from stormwater. It was found that by raising the pH of the stormwater from 6.5 to 8.0 a pavement design life of about 15 years could be achieved. The pH can be raised by using porous pavement as the surface layer (J. J. Sansalone, 1999). The design life was defined as the time taken, determined by laboratory testing, before a 90% breakthrough of the critical pollutant (found to be zinc amongst the metals tested) occurs. Sansalone distinguished between metals already sorbed to sediment particles and dissolved metals. The oxide coating is specifically added to trap dissolved metals, which is expected to be useful for Perth conditions. A partial exfiltration trench with a relatively thin porous pavement strip was investigated (J. Sansalone & Teng, 2004); however, it is believed by this author the OCS could be used between the sub-base and the subgrade in a pervious pavement.

Pollutant removal during storm events was also investigated for the above partial exfiltration trench (J. Sansalone & Teng, 2004). A 600 mm wide by 90 mm deep section of porous concrete was underlain by non-woven geotextile, with the oxide coated sand section underneath. This had a depth varying linearly from 450 to 600 mm and a width of 300 mm, with an outflow pipe underlain by in situ clay soil. The section investigated had a length of 3.75 m.

Three storm events were analysed. The first was on 25 November 1996 with a runoff volume of 215 L over the full 15 m length and duration of 150 min, for a mean loading rate of 19 mm/hr onto the OCS (Table 31). The mass reduction in TSS by concentration was 81%, chemical oxygen demand 74% and zinc also 74%. The reduction in suspended solids by count (as opposed to mass concentration) was 2 orders of magnitude (Teng & Sansalone, 2004). On 16 December 1996 a 340 min rain event produced a mean loading rate onto the OCS of 10 mm/hr, with 87% by mass of TSS removed by concentration, 70% of COD and 64% of zinc. Number reduction in suspended solids was also roughly 2-log. Finally, a 20 minute rainfall event on 12 June 1997 imposed a significant mean loading rate of 304 mm/hr onto the OCS, resulting in a TSS mass removal of just 54% by concentration, COD by 37% and zinc by 37%. Suspended solids reduction by number concentration was only 1-log. It is suspected that suspended solids leached from the pavement, with higher loading rates exacerbating the leaching, rather than the higher loading rates causing significantly greater pass-through of incoming suspended solids. Leaching tests would be required to clarify this. In all three events, all runoff was intercepted.

These results further support the significance of hydraulic loading rate on pollutant removal.

Charcoal from waste wood was investigated as a treatment enhancer (Kanjo et al., 2003). The intended application was in the sub-base. Test cylinders of 150 mm diameter, 200 mm height using ‘crusher run’ stone were used (Table 29 and Table 30). Multiple parameters were tested, with one experiment per parameter. In the experiment comparing materials for pollutant removal, a control cylinder with no additives, a cylinder with granular activated carbon (GAC), a cylinder containing charcoal from crating materials (charcoal 1) and a cylinder containing charcoal from crating materials and a mixture of building wastes (charcoal 2) were tested. Where charcoal/GAC was added, it was typically at 2% by weight (unless otherwise stated). The charcoal had about 1/6th the specific surface area of the GAC. Washing solution from a porous pavement highway was used as the influent, with down flow through the samples (i.e. the flow was not likely to have been saturated). Hydraulic details are given in Table 31.

Leaching tests indicated that the source of the waste wood is important, as charcoal 2 leached copper and chromium, reasoned to be because the source contained treated wood. The chromium concentration in the leachate was above that permissible for soils in Japan.

Total organic carbon (TOC) in the influent appeared to be 40 mg/L (Figure 5), including 37 mg/L of oils and grease. In the control sample tested with no charcoal, TOC would momentarily increase before decreasing back down to around 40 mg/L over time. This is believed by the author to be because the sample leached organic substances, however leaching tests were not conducted on the crushed stone for organic carbon. For all samples, the effluent concentration tended towards minimum values over time (Figure 5).

The removal ratio was calculated based on the total organic carbon mass passing through a sample over time (the time not explicitly specified, taken to be 48 hours as indicated in Figure 5) versus the corresponding mass passing through the control sample. GAC performed the best, at just over 60% removal, followed by 50% for charcoal from mixed wood sources and 30% for charcoal from used crates, all at 2% addition and a flow rate of just over 50 mL/hour (corresponding to a loading rate of about 3 mm/hr).

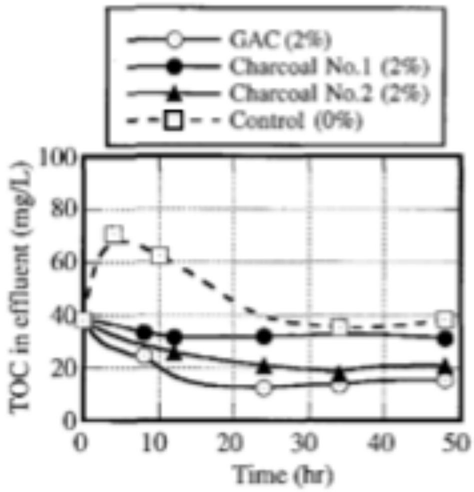

Figure 5: Change in TOC concentration over time (Kanjo et al., 2003)

Treatment performance with regards to flow rate was provided in a separate experiment. Increasing the flow rate reduced the removal rate, in the case of the charcoal from mixed sources from 50% to 20% with the flow increased from 50 mL/hour to about 210 mL/hour (about 12 mm/hr) respectively. Interestingly, when 1%, 2% and 4% addition of charcoal were tried in another experiment, 2% addition was found to be most effective, even when subtracting the TOC leaching from the charcoal.

To improve performance in the event of a catastrophic oil spill, an oil bund built into a PICP pavement sub-base was investigated (Newman et al., 2004). This was envisaged to perform better than an oil separator associated with a stormwater pipe, because of the lower water velocities. Laboratory testing was performed using two 490 mm barrels containing Formpave concrete blocks bedded on pea gravel (thickness not given) in turn placed on Terram 1000 geotextile and 290 mm of washed 50 mm granite (Table 29 and Table 30). In the second barrel, a plastic container was installed to trap water. A submerged bund was installed just above the container to trap the oil (Figure 6).

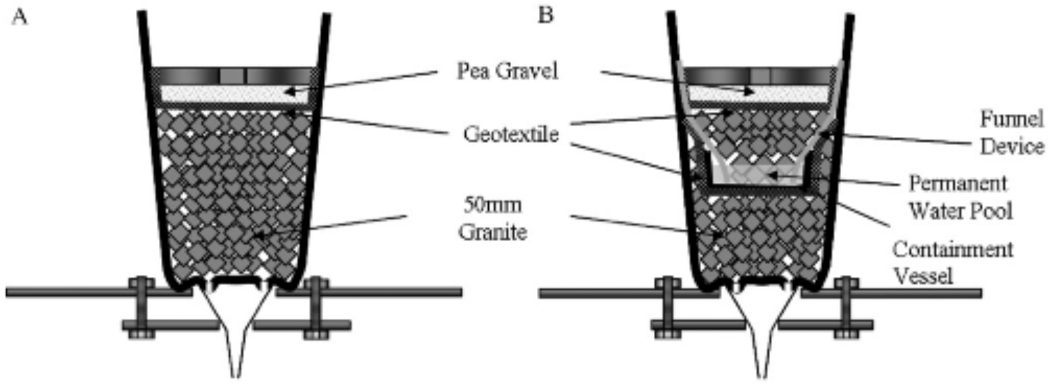

Figure 6: Schematic cross-section of oil bund experiments. (a) Without oil trap; (b) with oil trap. (Newman et al., 2004)

Two litres of oil was poured through each test barrel, after rinsing each barrel with distilled water. It was reported that over 830 mL of oil came through the barrel without the trap. As for the one with the trap, 800 mL of water was displaced, containing less than 1 mg/L of oil. Six simulated rainfall events were then applied, at 15 mm/hr and 50 minutes over 3 weeks (Table 31). From the barrel without the trap, oil concentrations were in excess of 4000 mg/L. As for the oil trap model, concentrations of less than 30 mg/L were recorded.

To improve degradation of oil, phosphate-leaching beads were tested in a PICP setup (Newman, Nnadi, et al., 2011). Test rigs were constructed with cross sections of 354 mm square. The pavement construction was similar to that used by Spicer, Lynch, Newman, and Coupe (2006), with concrete block pavers underlain by 20 mm of 10 mm gravel, then modified or unmodified geotextile. However, instead of using gravel for the sub-base, a Permavoid polymeric void forming unit was used (Table 29 and Table 30). Used motor oil was applied at 1.4 mL/m^2 every two weeks and each followed by 10 mm of simulated rain as well as 0.05 g/m^2 of nitrogen in the form of ammonium nitrate. The rate of water application was not given (Table 31). Oil degradation reached 450 mg/(/m^2•week) for the pavement with phosphate releasing beads and only about 230 mg/(/m^2•week) for the control. Other tests also showed that phosphate levels released by the beads did not exceed 0.5 mg/L.

A further study by Newman, Duckers, Nnadi, and Cobley (2011) was performed, using the same rigs and the same testing for the first three months. After that, no more oil was added for about five months, but simulated rain continued. Used oil loading of 14 mL was then added per pavement, followed by another several months later when bacterial activity had plateaued.

Carbon dioxide levels were monitored to indicate bacterial activities. With the rigs containing the phosphorus pellets, significantly enhanced activity was observed both immediately following the cessation of oil addition and after adding the large amounts of oil. It should be noted that activity was already rising sharply while the initial oil addition stopped. Higher activity in the test rigs containing phosphate pellets than in the rigs not containing phosphate pellets indicates superior hydrocarbon removal. Testing was ongoing at the time of printing.

Several materials were tested in the laboratory by Ree-Ho et al. (2006). Materials tested were urethane and recycled aggregate (Type A), aggregate with ash from wastewater sludge (Type B), recycled aggregate with tourmaline (Type C) and recycled aggregate with urethane and chemically modified lignocellulose for enhanced nutrient removal (Type D). All pavements are taken to be homogeneous, from the figure. They had plan dimensions of 500 mm by 500 mm and a depth of just 30 mm (Table 29 and Table 30). Rainfall runoff was used as the influent. For water quality testing, the simulated rain was taken to be 50 mm/hr, from the value used for temperature testing. The duration was 30 minutes (Table 31).

Parameters tested were turbidity, NO_x-N, ammonia, total nitrogen and total phosphorus. Turbidity was reduced the most out of the pollutant species, from 35 NTU down to 30 NTU for pavement type A, 14 NTU for B and 8 for D. Type D pavement reduced phosphorus but the final value is unclear (graph). Other parameters were not affected noticeably, which is expected because of the shallow thickness of the pavement.

In summary, to enhance removal of pollutants, there are a number of options, depending on the pollutant. Geotextile, considered a standard component of pervious pavement, improves TSS removal as discussed in Section 2.2.6. With additional measures taken such as oxide coating sand, care should be taken to quantify and control the washoff of iron oxide particles, or carbon in the case of charcoal or activated carbon. This washoff effect appears to be influenced by loading rate (Table 32).

For nutrient removal, several solutions are available. Granular activated carbon and charcoal are capable of absorbing organic carbon (Table 33). Building in an oil bund or adding phosphate beads enhances the capture or degradation of oil respectively. Chemically modified lignocellulose can also improve nutrient removal.

Oxide coated sand is effective for removing metals, especially when combined with porous concrete. While this is not clear from Table 34 in comparison to other field studies, due to the varying conditions, the laboratory tests performed confirm this (J. J. Sansalone, 1999).

Pollutant reductions in Table 32 through Table 34 are typically by concentration and are all relative to input, *including the field testing by Sansalone.* For the studies by Newman et al. however, the oil removal rates are by pollutant load, not concentration. For the oil bund tests the approximate cumulative oil removal rate after the third test is given, taking into account any initial leakage of oil. For the microbial degradation tests by Newman et al., the input loading and degradation during the sixth fortnight – the period when degradation was highest – was used.

Table 29: Surface and General Details of 'Enhanced' Pervious Pavements

		General				Surface			
Author	Date	Testing	Location	[Simulated] Age (years)	Case	Surface Type	Surface Thickness (mm)	Bedding Gravel Size (mm)	Bedding Thickness (mm)
Sansalone and Teng	2004	Field (25/11/96)	Cincinnati, Ohio, USA	0	25/11/96	Porous concrete	90	None	None
Sansalone and Teng	2004	Field (16/12/96)	Cincinnati, Ohio, USA	0	16/12/96	Porous concrete	90	None	None
Sansalone and Teng	2004	Field (12/06/97)	Cincinnati, Ohio, USA	0	12/06/97	Porous concrete	90	None	None
Kanjo et al.	2003	Laboratory	Osaka, Japan	0	Charcoal	Stone and charcoal (#2)	None	None	None
Kanjo et al.	2003	Laboratory	Osaka, Japan	0	GAC	Stone and GAC	None	None	None
Newman et al.	2004	Laboratory	Coventry, UK	0	No Bund	Formpave PICP	80?	Pea Gravel	?
Newman et al.	2004	Laboratory	Coventry, UK	0	Bund	Formpave PICP	80?	Pea Gravel	?
Newman et al.	2011	Laboratory	Coventry, UK	0	No Osmocote	Concrete PICP	80?	Pea Gravel	20
Newman et al.	2011	Laboratory	Coventry, UK	0	Osmocote	Concrete PICP	80?	Pea Gravel	20
Ree-Ho et al.	2006	Laboratory	Goyang-Si Gyeonggi-Do, Korea	0	Urethane	None	None	None	None
Ree-Ho et al.	2006	Laboratory	Goyang-Si Gyeonggi-Do, Korea	0	Ash	None	None	None	None
Ree-Ho et al.	2006	Laboratory	Goyang-Si Gyeonggi-Do, Korea	0	Urethane + lignocellulose	None	None	None	None

Table 30: Base and Geotextile Details of 'Enhanced' Pervious Pavements

			Base			
Author	Case	Geotextile	Base Gravel Size (mm)	Base Thickness (mm)	Sub-base Gravel Size (mm)	Sub-base Thickness (mm)
Sansalone and Teng	25/11/96	Nonwoven, spun-bonded polypropylene @ 120 g/m2	Oxide coated sand	450-600	None	None
Sansalone and Teng	16/12/96	As above	Oxide coated sand	450-600	None	None
Sansalone and Teng	12/06/97	As above	Oxide coated sand	450-600	None	None
Kanjo et al.	Charcoal	None	None	None	'Crusher run' stone and 2% charcoal (#2)	200
Kanjo et al.	GAC	None	None	None	'Crusher run' stone and 2% GAC	200
Newman et al.	No Bund	Terram 1000	50	290 (No Bund)	None	None
Newman et al.	Bund	Terram 1000	50	290 (Bund)	None	None
Newman et al.	No Osmocote	Inbitex Without Osmocote (P)	Polymeric Void Forming Unit	?	None	None
Newman et al.	Osmocote	Inbitex With Osmocote (P)	Polymeric Void Forming Unit	?	None	None
Ree-Ho et al.	Urethane	None	Urethane + recycled aggregate	30	None	None
Ree-Ho et al.	Ash	None	Aggregate + ash	30	None	None
Ree-Ho et al.	Urethane + lignocellulose	None	Urethane + recycled aggregate + lignocellulose	30	None	None

Table 31: Hydraulic Details for 'Enhanced' Pervious Pavements

Author	Case	Hydraulic Loading Rate (mm/hr)	Hydraulic Loading Duration (hr)	Hydraulic Loading Depth (mm)
Sansalone and Teng	25/11/96	0-131	2.50	47
Sansalone and Teng	16/12/96	0-46	5.67	59
Sansalone and Teng	12/06/97	0-1010	0.33	101
Kanjo et al.	Charcoal	12	48.00	~600
Kanjo et al.	GAC	12	48.00	~600
Newman et al.	No Bund	15	0.83	12.5
Newman et al.	Bund	15	0.83	12.5
Newman et al.	No Osmocote	?	336 (storage)	10
Newman et al.	Osmocote	?	336 (storage)	10
Ree-Ho et al.	Urethane	50	0.50	25
Ree-Ho et al.	Ash	50	0.50	25
Ree-Ho et al.	Urethane + lignocellulose	50	0.50	25

Table 32: TSS Removal Efficiencies of 'Enhanced' Pervious Pavements

Author	Case	TSS
Sansalone and Teng	25/11/96	75%
Sansalone and Teng	16/12/96	87%
Sansalone and Teng	12/06/97	54%

Table 33: Nutrient Removal Efficiencies of 'Enhanced' Pervious Pavements

		Nutrient											
Author	Case	DOC	COD	BOD	Motor Oil	TP	TN	TKN	NH_4-N	ON	NO_2-N	NO_3-N	NO_x-N
Sansalone and Teng	25/11/96	-	56%	-	-	-	-	-	-	-	-	-	-
Sansalone and Teng	16/12/96	-	70%	-	-	-	-	-	-	-	-	-	-
Sansalone and Teng	12/06/97	-	37%	-	-	-	-	-	-	-	-	-	-
Kanjo et al.	Charcoal	22%	-	-	-	-	-	-	-	-	-	-	-
Kanjo et al.	GAC	42%	-	-	-	-	-	-	-	-	-	-	-
Newman et al.	No Bund	-	-	-	56%	-	-	-	-	-	-	-	-
Newman et al.	Bund	-	-	-	log-4	-	-	-	-	-	-	-	-
Newman et al.	No Osmocote	-	-	-	33%	-	-	-	-	-	-	-	-
Newman et al.	Osmocote	-	-	-	64%	-	-	-	-	-	-	-	-
Ree-Ho et al.	Urethane	-	-	-	-	0%	0%	-	0%	-	-	-	0%
Ree-Ho et al.	Ash	-	-	-	-	0%	0%	-	0%	-	-	-	0%
Ree-Ho et al.	Urethane + lignocellulose	-	-	-	-	>50%	0%	-	0%	-	-	-	0%

Table 34: Metal Removal Efficiencies of 'Enhanced' Pervious Pavements

		Metal											
Author	Case	Cd_T	Cd_D	Cd_P	Cu_T	Cu_D	Cu_P	Pb_T	Pb_D	Pb_P	Zn_T	Zn_D	Zn_P
Sansalone and Teng	25/11/96	56%	57%	50%	53%	57%	40%	51%	44%	63%	91%	95%	64%
Sansalone and Teng	16/12/96	33%	33%	63%	60%	58%	67%	14%	0%	61%	91%	95%	64%
Sansalone and Teng	12/06/97	70%	72%	50%	70%	72%	54%	39%	30%	67%	91%	92%	37%

T subscript refers to total concentration, similarly D means dissolved and P means particulate.

2.3 Conclusion

Pervious pavement can be a good device to use for Water Sensitive Urban Design. Its ability to remove pollutants is important for protecting the environment or for treating water for water supply, as the stormwater contains significant pollution. Overall, pollutant reduction rates of pervious pavements in the literature were varied due to different conditions, such as input pollutant concentrations, inflow, pavement thicknesses, temperature etc. These conditions were not always explicitly stated.

As for TSS, it is understood that concentrations in the effluent depended mainly on the aggregates (gravel and/or sand) used for each layer, with these aggregates often being a significant source, especially for sand. One study found that removal of TSS by concentration varied linearly with input concentration. Reduction compared to asphalt runoff / influent varied from about -110% (net increase due to fines originating from the filter medium) to 94%.

In situations where stormwater is directly infiltrated to the soil, natural filtration will occur in the soil profile, but the soil may become clogged over time. Thus the importance of sediment removal will depend very much on the application and soil types.

Permeable Friction Course (PFC), which is porous asphalt with an impermeable surface immediately beneath it, fared similarly to other pavement types for TSS removal. Geotextile significantly improves performance. When adding materials such as charcoal, granular activated carbon or iron coated sand to the pavement structure, care must be taken to control washoff of particles.

Nutrient reduction is highly complex and depends on many factors, due to the reliance on microorganisms. Only one study was found that gave a good comparison of nutrient removal between different pavement types and revealed that PICP, sand filled concrete grid pavers and porous concrete all performed similarly, with the exception of the concrete grid pavers, with the sand apparently enhancing treatment of NO_x-N. A separate study determined PFC performs insignificant nutrient reduction, as opposed to full depth pervious pavement.

Motor oil was most easily removed, with it below detection limit in exfiltrate samples in most cases. No study was found testing for motor oil removal for porous asphalt however, which contains hydrocarbons in its structure. Excluding PFC, ammonium was easily reduced in concentration, by 84% to 100% compared to asphalt runoff / input.

The most difficult nutrient to remove was NO_x-N, at best 9% and at worst -331% (i.e. net gain, due to nitrification) compared to asphalt runoff. Storing the stormwater in the pavement sub-base over a period of several days using dolomite caused a large reduction in total nitrogen of 67%.

Geotextile contributes significantly to pollutant removal by providing the microbes with a high surface area medium to grow on. To improve nutrient removal there are many options: adding granular activated carbon, charcoal, phosphate beads, lignocellulose or even an oil bund.

Metal removal varied significantly, from non-detection in the output to -138% (net gain) compared to asphalt runoff concentration / input concentration. This depended on the metal species and likely the pavement materials. Particle bound metals are strained, while dissolved metals are sorbed to the pavement. A combination of porous concrete and iron oxide coated sand enhances the removal of dissolved metals. Leaching tests would be useful for identifying pavement materials that are a source of metals in the exfiltrate.

Laboratory testing has been undertaken at Curtin University to develop a greater understanding of the impact of different testing conditions and pavement constructions on pollutant removal ability. In light of the strong potential for washoff of solids from within the pavement structure, it was important to determine how much this occurs. Due to the complex nature of nutrient removal, simple, laboratory prepared compounds were used for ease of analysis of results. Some enhancement options such as granular activated carbon were trialled.

3 MATERIALS AND METHODS

3.1 Construction

A testing setup was constructed at the civil engineering laboratory of Curtin University Western Australia, Bentley campus, outdoors near shipping containers at the south end of the yard. Four 160 L plastic boxes from Total Materials Handling of internal dimensions 600 mm by 600 mm in plan, 450 mm depth, were used for containment of pavement structures, forming test rigs. A photo of one completed setup is shown in Figure 7. These test rigs are numbered 1 to 4 and named accordingly (Figure 8). Holes of 20 mm diameter were drilled in each box for drainage, with slope to each hole.

For Test Rig 1, a crushed granite base was provided (Figure 8). This granite was sourced from BGC Quarries, from a quarry near York. It was sized 7 to 60 mm, with 17% from the 7-10 mm pile, 16% from the 14 mm pile, 7% from the 20 mm pile and 60% from the 40-60 mm (ballast) pile to try and achieve optimal grading for density. There is deviation from Fuller's curve (Figure 9) but it is still believed to be a satisfactory mix, given adequate mixing. The aggregate was found to have reddish fine material adhering to it. This base material was intended to mimic that of Scholz and Grabowiecki's (2009) test rigs, at least in size range. A 300 L mixer was used to mix the gravel, as segregation had occurred after pre-mixing by others. Unfortunately the mixer did not mix the gravel sufficiently. This was probably because it was designed for mixing concrete, not a ballast and coarse aggregate mixture. Sparks were in fact observed during mixing, indicating significant resistance.

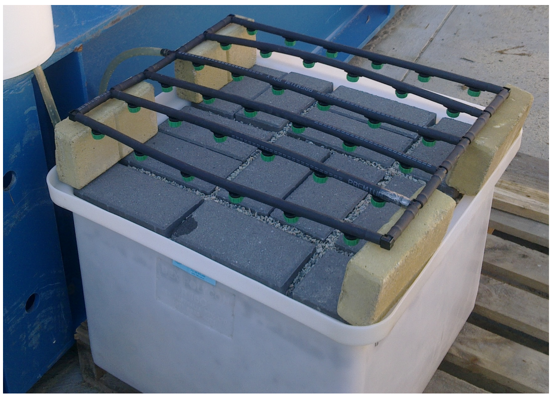

Figure 7: Test Rig 1 with Dripper Manifold (taken 14 July 2011, before water quality testing period began)

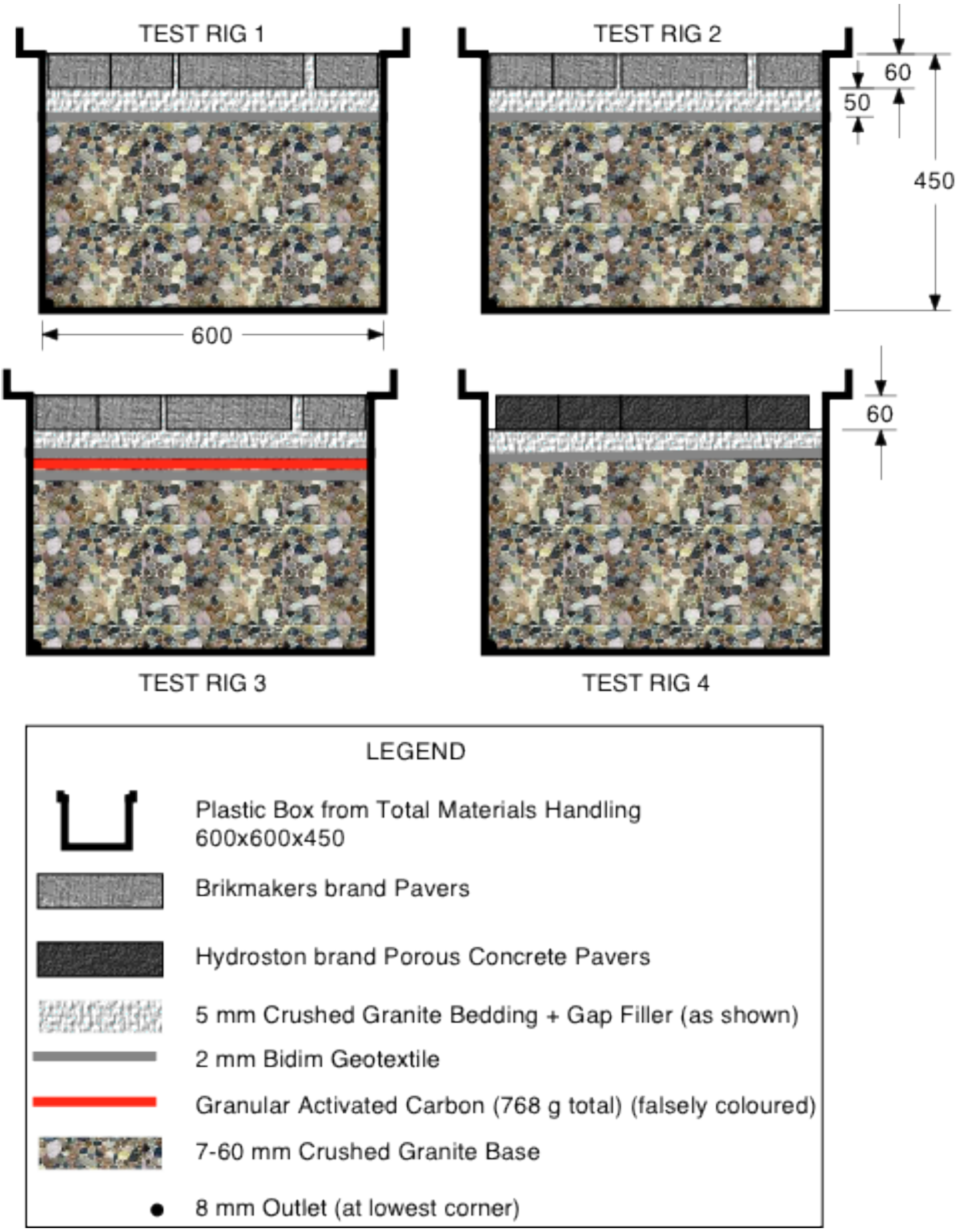

Figure 8: Cross Sections of Initial Constructions of Test Rigs

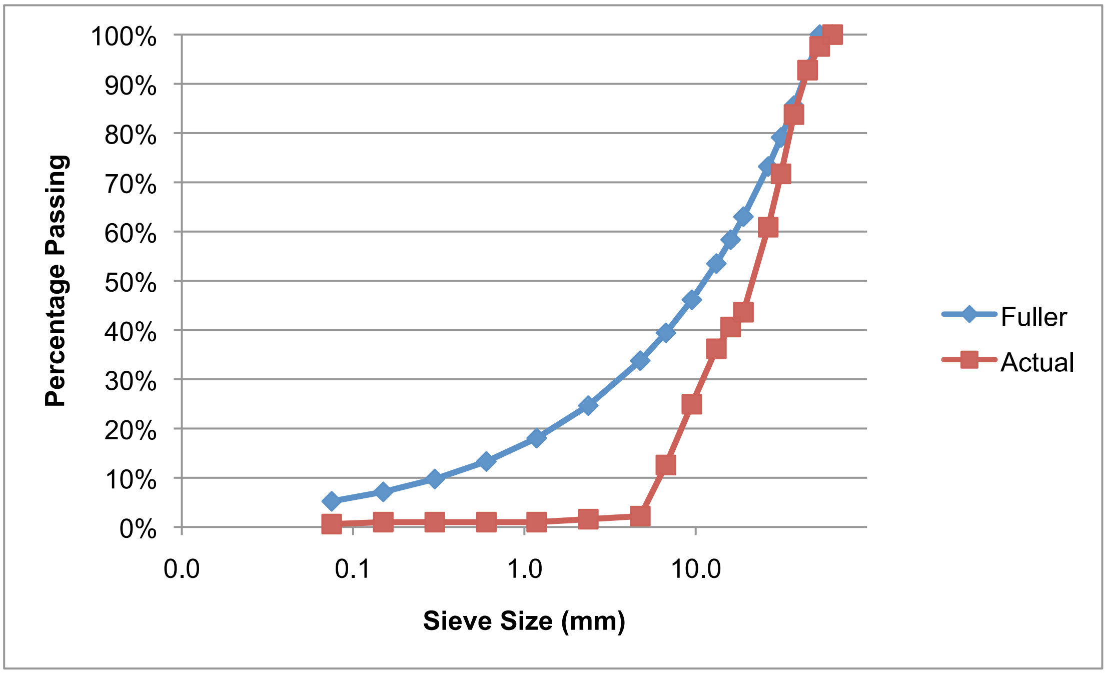

Figure 9: Grading Curve for Base Aggregate

After placement of the base layer, Rig 1 was hosed down until water leaving the outlet was observed to be clear, simply by observing the water as it flowed.

Bidim geotextile was laid on top of the base gravel for Rig 1. On top of the geotextile, a bedding layer was placed. This consisted of 50 mm of 5 mm crushed granite from Curtin University's stockpiles, in turn sourced from Holcim. As this material contained substantial fines it was washed with a sieve in batches by submerging in water and agitating.

Brikmakers pavers with gaps filled with 5 mm gravel (same as bedding material) were laid on the bedding material for Rig 1. All pavers were a nominal 60 mm thick.

Test Rig 2 was constructed the same as Rig 1 (Figure 8) as a check. Any differences in testing results would be investigated.

Test Rig 3 was constructed the same as Rig 1 except with the following differences. The pre-mixed aggregate had run out so more was sourced directly by the author. It was recognised that attempting to mix the aggregate using existing facilities was resulting in inadequate mixing and hence variation. Rather than using the mixer for the base material this time, small amounts from each aggregate stockpile were added at a time to the bins, which is argued to achieve the same effect as thorough mixing. The small amounts were carefully proportioned as per the overall mix design. Each amount consisted of 14.5 kg of 40-60 mm ballast, allowing the gaps between aggregate particles to be filled successively. To reconstruct the previous rigs in the same way would have required either re-sieving or replacing the aggregate. It was decided not to do either, considering the aggregate was being supplied gratis, and continue as-is. Sieving 250 L of base aggregate using University facilities would have been painstaking and time consuming.

The washing method for rigs 1 and 2 was deemed insufficient, as a cake of fines had built up on the base. Therefore, for Test Rig 3 the gravel was washed by hosing down the gravel in small batches directly into a drain. The completed test rig was also hosed down. Suspended solids tests were performed to see if these different washing methods had an effect (see Section 4.2.4). The outflow calculations also provide insight. Although suspended solids tests probably should have been performed for the first run for all test rigs, from the results and turbid appearance of samples it was apparent both sieving and hosing the completed test rigs was necessary, with washed sand at the base as well to trap any remaining fines (see Section 3.5). This is necessary for satisfactory Aquifer Storage and Recovery, i.e. without requiring excessive maintenance of the bore (see Section 3.4).

The most important feature of Test Rig 3 is that 768 g of granular activated carbon was placed in a single layer between two layers of geotextile (Figure 8).

Test Rig 4 was constructed the same as Rig 1, except the base material was placed in the same manner as for Rig 3, without hosing down the completed test rig (by mistake). Hydroston porous concrete pavers were laid on the bedding for Test Rig 4 instead of the Brikmakers pavers with gaps (Figure 8).

3.2 Statistical Analysis

All data was imported or directly entered into spreadsheets. Statistical analysis was performed using the R statistical package (R Development Core Team, 2012) as well as using OpenOffice Calc. All errors are to 95% confidence unless otherwise stated. Quantile-quantile plots in R, comparing with the normal distribution, were used to check for normality and remove outliers where indicated. Number of observations in a sample is denoted by ‘n’.

3.3 Hydraulics

A dripper manifold was constructed using garden hose and drippers, with the drippers spaced 100 mm both directions and spanning six in each direction (Figure 7). This was attached to a bucket at a fixed height above the pavement. The elevation of the bucket relative to the drippers was typically greater than 500 mm (unlike in Figure 7), to minimise flow variation during each test as the water surface in the bucket dropped. This flow variation was usually around 2% as a result (calculated).

Changing the bucket height every few days or so was necessary to compensate for different thermal effects on the drippers as the weather changed. Typically, the height was 500 – 1000 mm. As a result, the hydraulic loading rate was typically 76±9 mm/hr (n = 172). The dripper manifold was stored inside to attempt to minimise thermal effects on dripper loading rate.

Adjustment of the dripper heads proved difficult, with creep effects affecting the flow rate. Therefore, this was minimised where possible, but was necessary at the beginning of testing.

The volume of solution dripped per pavement per run was typically 2.25 L. Application was once every workday. Outflow was collected from each outlet, intended to be in 250 mL samples but with the actual volume and time of collection recorded. Three samples were taken each time, except in rare cases where not enough water came out.

All four test rigs were given timber spacers to increase the slope towards the outlet, detailed in Table 35. This had the unintended effect of causing the bases to sag, which was identified after testing had finished. For rigs 1 and 3, this in turn resulted in a trapped volume. Trapped volumes and areas given in Table 35 are rough calculations only, direct measurement was not possible.

Table 35: Geometry of Bases of Test Rigs

	Slope 1	Slope 2	Deflection (mm)	Ponding Area (m^2)	Trapped Volume (L)
Rig 1	2%	2%	21	0.14	0.4
Rig 2	2%	6%	19	0	0
Rig 3	4%	1%	17	0.08	0.2
Rig 4	3%	9%	35	0	0

The start time of inlet flow was roughly judged to be when the flow rate reached half of the typical flow rate for each run. Outflow was judged to begin when two consecutive drips occurred in an appropriately short period of time, since water would continue to slowly drip out from the previous run. The error of each is expected to be in the order of 10 seconds.

3.4 Water Quality

From the literature review, pollutant concentrations relevant to Australian runoff were used as in Table 36 to make a synthetic stormwater. Stock solutions were prepared by measuring out the solid masses (salts except for glucose) to an accuracy of 1 mg. These were then mixed with 1 L of reverse osmosis water to make concentrations of 1 g/L, except for aluminium, which was prepared as 500 mL of solution at 2 g/L. These were then diluted to the target concentrations each day immediately before testing. Sulphuric acid was added to each of the metal stock solutions after preparation to achieve a pH of below 4, with the bottles kept sealed to avoid atmospheric contamination from carbon dioxide. The acid was added to avoid precipitation caused by carbon dioxide (since the bottles had to be opened during use). Nutrient stock solutions were kept in the fridge. Reverse osmosis water used was from an Ibis Mini unit. Preparation of each synthetic stormwater solution involved adding small amounts of each stock solution to more reverse osmosis water, to achieve the target concentrations in Table 36.

Table 36: Pollutants used for Synthetic Stormwater

Pollutant	Concentration (mg/L)	Source
Dissolved Organic Carbon (DOC)	4.00	Glucose ($C_6H_{12}O_6$)
Phosphorus	0.84	$KH_2PO_4 \cdot 3H_20$
Nitrogen	1.46	NH_4Cl
Aluminium	1.47	$Al_2(SO_4)_3 \cdot 16H_20$
Copper (II)	0.34	$CuSO_4 \cdot 5H_20$
Zinc	1.85	$ZnSO_4 \cdot 7H_20$

For a period of time, only the organic carbon and nutrients (DOC, P, N) were added, until a consistent output quality was achieved. The three metals were then added, with metal addition starting at different dates, to see the effect on nutrient removal (see Section 3.5 for sequence).

Ultimately the aim of this research is to use pervious pavement to pre-treat stormwater for managed aquifer recharge into the Leederville aquifer, Perth, Western Australia. Nutrient levels from the test rigs were compared against guidelines from the Australian and New Zealand Environment and Conservation Council (ANZECC & ARMCANZ, 2000; Environment Protection and Heritage Council, 2006, 2009a, 2009b) and existing Leederville aquifer (GHD, 2008) values, with the guidelines only considered where the aquifer concentrations were ambiguous. Where guideline values were used, the value from the guideline giving the lowest value was adopted. These are presented in Table 37. (The ANZECC freshwater guideline for ammonium in a lowland river in southwest Australia is 0.08 mg/L.)

Table 37: Nutrient Concentration Quality Limits

Nutrient	Limit (mg/L)	Source
Ammonium	0.11	Aquifer
NO_x-N (nitrate + nitrite)	0.01-0.06	Irrigation Guideline
Phosphate	0.04	Freshwater Guideline
Dissolved Organic Carbon	1-10	MAR Guideline
Suspended Solids	1-10	MAR Guideline

Samples were stored in the fridge until testing was performed, typically on the same day. Testing for DOC was accomplished with a Sievers 5310C Laboratory TOC Analyzer with an experimental error for DOC of ±5% and a method detection limit of around 0.1 mg/L. This worked by reducing organic substances with ultraviolet light and persulphate and measuring the carbon dioxide emitted.

An Aquakem 200 machine was used to measure ammonium, nitrite, nitrate and reactive phosphorus, with nitrate-N being the difference between NO_x-N and nitrite-N. This machine measured ammonium by reacting with hypochlorite to form chloramine, then salicylate under certain conditions to form a blue compound measurable at light wavelength 660 nm (Thermo Fisher Scientific, 2011a), with a method detection limit of 0.002 mg/L. Nitrite was measured by reaction with sulfanilamide and N-(1-naphthyl)-ethylenediamine dihydrochloride to form an azo dye detectable with the 540 nm wavelength of light (Thermo Fisher Scientific, 2011b), with a method detection limit of 0.003 mg/L. The NO_x-N concentration was determined by reducing the nitrate to nitrite using hydrazine under alkaline conditions, then measuring the total nitrite as above (Thermo Fisher Scientific, 2011d). The method detection limit is reported as 0.001 mg/L despite the detection limit for nitrite being 0.003 mg/L. Reactive phosphorus was measured by reacting it with ammonium molybdate under acidic and catalytic conditions to form a 12-molybdophosphoric acid complex, which in turn was reduced with ascorbic acid to form a blue compound detectable with the 880 or 660 nm wavelength of light (Thermo Fisher Scientific, 2011c) with a method detection limit of 0.001 mg/L.

Suspended solids was tested using EPA method 160.2 with paper filters, then later using the HACH photometric method with a HACH DR2800 meter (Hach, 2010).

The temperature of the input water was adjusted to 20±2 ˚C until 7 December (corresponding to Day 142, see next section), when this was deemed futile due to the hot weather. Effort was made to keep the pavements in the shade as much as possible and to shelter from rain at all times.

For statistical analysis the Event Mean Concentration (EMC) was calculated for each test run and each pollutant, being:

$$EMC = \frac{C_2V_2 + C_3V_3}{V_2 + V_3} \qquad (1)$$

Where:

EMC = event mean concentration

$C_{2,3}$ = concentrations of pollutant of interest from samples 2 and 3

$V_{2,3}$ = volumes of samples 2 and 3.

For runs with less than three samples, the EMC was simply adopted as the concentration of the pollutant in question in sample 2. Similarly, for the start of Rig 1 testing when only sample 1 was tested, these measured concentrations from sample 1 are illustrated in graphs as the EMC's. They are only included in the graphs for completeness and this data from the first samples are not used in statistical analysis.

Sample 1 will normally be rejected from analysis when calculating the Event Mean Concentration. Plots of ammonium and phosphate output concentration with respect to date for each of samples 1, 2 and 3 indicated that sample 1 had a higher variance. In particular, the period Day 72 to Day 115 for Rig 1 had a fairly stable ammonium concentration. Samples 1, 2 and 3 had standard deviations in ammonium concentrations of 0.08, 0.05 and 0.06 mg/L respectively. Samples 1 and 2 had normally distributed ammonium concentrations, as indicated by normal quantile-quantile plots, but sample 3 did not. Applying the f-test for equality of variances, samples 1 and 2 had significantly different variances ($p = 0.04$, two-tailed).

Paired t-tests were used to compare pollutant removal performance of the test rigs, unless otherwise stated. The one-tailed t-tests were used to find the minimum magnitude of difference in output concentration of a pollutant between two test rigs, using a confidence of 95%. Only one test run was performed each time a pavement layer was stripped. Therefore, the sample standard deviation before layer stripping commenced was adopted as the 'pooled' standard deviation, with equal variance assumed. A similar approach was used for testing the effect of spiking the ammonium concentration, except some test runs after the spike occurred were included in the analysis as well.

Contamination and degradation of samples was somewhat of an issue. All runs that produced degraded or contaminated samples were excluded from analysis for the pollutant in question. For example, when a sample had a discontinuously high concentration of phosphate, the phosphate data for that run was excluded. Also, ammonium degradation was noted when longer storage in the fridge was used. Therefore, ammonium results for samples kept in the fridge longer than 24 hours were excluded.

There were significant problems with dissolved organic carbon degradation in samples, likely due to the use of glucose as the organic carbon input. Bacteria consume glucose readily. Only the samples for which the measured input sample was over 1 mg (C)/L are counted, reducing the available data set. A stricter limit would have reduced the data set too severely.

3.5 Sequence of Events

Multiple changes were undertaken during testing and the pavements were not all constructed on the same date. The sequence of events is as below. Day 1 refers to 19 July 2011 and the day numbers increment for each calendar day.

- Day 1: Testing began for Rig 1.
- Day 45: After some difficulties, the inflow was stabilised to 76±9 mm/hr (n = 172).
- Day 51: Testing began for Rig 2.
- Day 85: Testing began for Rig 3.
- Day 87: Rig 2 had its surface changed to porous concrete pavers, as in Rig 4, to directly observe any difference in results.
- Day 88: Testing begins for Rig 4.
- Day 98: Install plywood timber supports for the dripper manifold in rigs 2 and 4, acrylic supports in Rig 3 and one stainless steel plus one fibreboard support for Rig 1, for convenience of testing. The timber supports may have caused dissolved organic carbon contamination and were observed to have microorganisms growing on them.
- Day 120: In Rig 1 the bottom 50 mm of gravel was replaced with Rocla sand overlaid with the same geotextile as under the bedding layer (Figure 10). At the same time, the coarse aggregate base was washed using the more thorough method for rigs 3 and 4. Before washing, the base was observed to have a thick (roughly 50 mm) cake of clay.
- Day 130: Aluminium addition to input solution started for all four pavement rigs.
- Day 133: Inflow adjusted to 54±7 mm/hr (n = 47) for rigs 2, 3 and 4, similar to Perth's 1 year ARI 5 minute duration storm intensity of 60 mm/hr. This inflow was used from then on.
- Day 134: Inflow adjusted to above value for Rig 1. This inflow was used from then on.

- Day 137: Zinc addition to input solution started for all four pavement rigs.
- Day 142: Input solution volume adjusted to 1.8 L for each pavement, which corresponds to 60 mm/hr for 5 minutes over one pavement's 600 mm by 600 mm area.
- Day 143: Copper addition to input solution started for all four pavement rigs.
- Day 155: To test for effects from individual layers, the pavers were removed from all four test rigs before testing (retaining any gravel between the pavers).
- Day 156: Bedding gravel removed before testing for all four test rigs.
- Day 157: Geotextile removed before testing for all four test rigs. This day, 22 December, is the last run for all four test rigs.

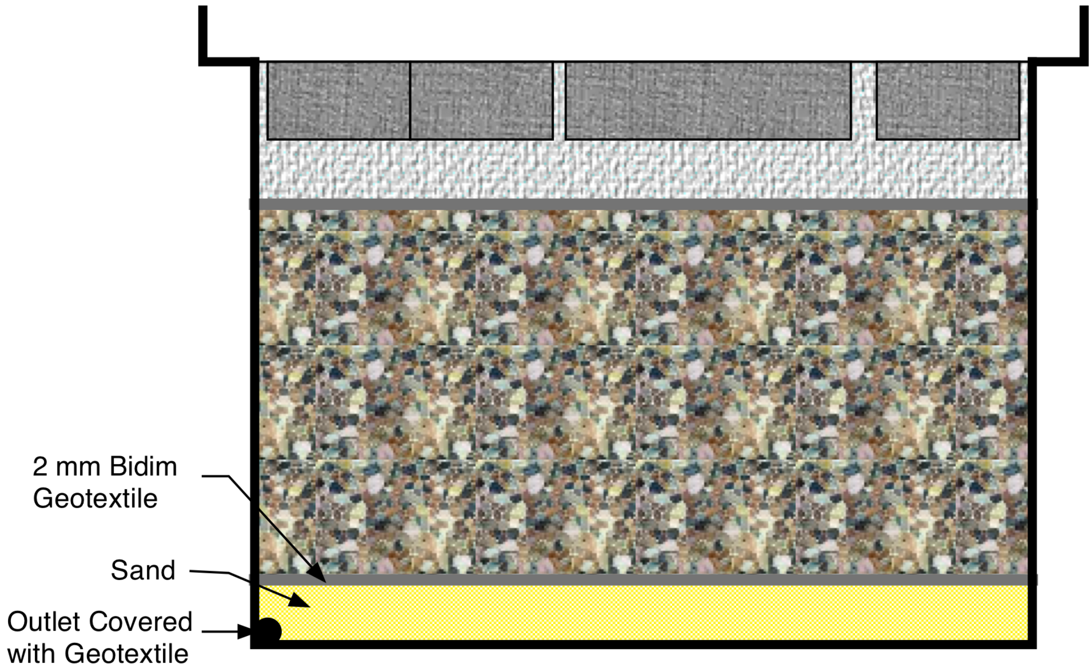

Figure 10: Cross Section of Rig 1 with Sand Layer

4 RESULTS AND DISCUSSION

Four test rigs were analysed for their hydraulic behaviour and treatment efficiency. Hydraulic behaviour was analysed in terms of the simulated rainfall intensity (or loading rate), outflow start time and outflow rates. To test the water treatment efficiency various parameters were analysed including removal of ammonium, dissolved organic carbon, nitrite, nitrate and phosphate.

4.1 Hydraulics

During experimentation a hydraulic loading rate of about 72 mm/hr was intended. However, achieving a stable loading rate was somewhat difficult. Even after it was stabilised the 95% variation was 9 mm/hr (n = 172). The significant error in loading rate probably arose from thermal effects on the plastic drippers, as the range in loading durations was nearly 2 minutes. In fact, in some instances the bucket height was adjusted (from 520 mm to 580 mm on Day 144 for example) to compensate for systematic error as the season progressed. From Day 36 to completion (n = 83), when the drippers were kept on the same setting, the multiple R^2 for linear correlation between bucket height above drippers and loading rate for Rig 1 was only 0.34. Normally flow would be expected to increase as in

$$Q \propto h^{0.5} \qquad (2)$$

Q represents flow rate and h represents elevation head. However, after transforming the elevation data as above, the R^2 was unchanged. From Day 36 to Day 45, for Rig 1 (n = 8), which covered five different bucket heights, the multiple R^2 was an improved 0.58, with a weak linear trend evident from a scatter plot. In reality, however, the rainfall intensity is never constant and hence the simulated conditions are sufficient enough to represent the intended rainfall intensity.

4.1.1 Outflow Start Time

In all cases there was a delay between beginning of inflow and outflow. This delay is termed the outflow start time. It is due to the water having to both wet and flow downwards through the pavement layers before reaching the outlet. Most of the water also had to flow horizontally along the base; this may have contributed to the measured outflow start times as well. The time the simulated rainfall takes to percolate through the gravel layers is of interest when calculating the time of concentration to an Aquifer Storage and Recovery bore and hence storage infrastructure. It also gives an indication as to the hydraulic retention time, which is relevant to pollutant removal.

Outflow start times, overall, remained fairly consistent. For Rig 1, outflow seemed to stabilise after Day 32, possibly after channel formation in the clay-like sediments on the base. Between this date and sand layer placement, the outflow started after 63±26 seconds. Doing a paired t-test with Rig 2 while it had pavers with gaps, ignoring the first two tests, the start times were not significantly different ($p = 0.14$).

Immediately after the sand layer was placed in Rig 1, the outflow start time increased significantly by over a minute for three days ($p = 0.04$, unpaired), likely due to the sand layer filling with solution. The start time then stabilised to a higher value than before ($p = 1\times10^{-7}$, unpaired t-test with 3 outliers removed) of 1 min 36 sec, due to the lower permeability of the sand. The outliers corresponded to tests performed after the rig was left untested for several hot days, suggesting drying affecting the results.

After the porous pavers were installed for Rig 2, there was still no significant difference in start time with Rig 1 ($p = 0.52$, paired t-test ignoring first 2 days with porous pavers). Interestingly, rigs 2 and 4 had significantly ($p = 0.009$, paired) different outflow start times, likely due to the different sub-base washing methods.

Rig 3 had a significantly longer start time than Rig 1 ($p = 3.4\times10^{-10}$, paired t-test for before sand layer was added), due to the water having to penetrate two layers of geotextile as well as granular activated carbon. These are summarised in Table 38:

Table 38: Outflow Start Times for Test Rigs

Situation	Mean (sec)	Error (sec)	Observations (ex. outliers)	Outliers Removed
Rig 1 Rock Base	63	26	61	13*
Rig 1 Sand Layer	96	29	18	3
Rig 2	66	44	68	1
Rig 3	117	33	44	1
Rig 4	72	36	39	3

* Up to and including Day 31, after which the outflow start times were judged to be sufficiently stable.

Removing the geotextile for Rig 1 decreased the start time significantly ($p = 0.005$, assuming variance unchanged) by roughly 45 seconds, considering only one data point was collected after the geotextile was removed. Surprisingly, there was no significant difference for the other pavements, suggesting the geotextile may not have actually contributed significantly to the outflow start time. As only one test was performed per pavement without either the surface or bedding layers (but with geotextile), no significant contribution to start time from these layers was found. No hard conclusion can be drawn; in fact, it is entirely plausible the differences in start times between rigs (with the exception of adding the sand layer) were mostly due to the different placement and washing methods of the base courses.

For Rig 1, considering the data from commencement until just before the sand layer was added ($n = 74$), outflow start time was not correlated to loading rate, with an R^2 of just 0.06. Similarly, correlations between loading rate and outflow start time were not found for the other test rigs, considering the entire testing period for each. Also, changing the loading rate from 76 mm/hr to 54 mm/hr was not found to make a significant difference to the outflow start time, considering the few days in summer when significant drying was believed to take place.

Since testing was not performed on every single day during the testing period, correlation between outflow start times and the number of days since the last test was considered. For Rig 1, before adding the sand layer a weak ($R^2 = 0.22$) correlation was found ($n = 74$). Three days after adding the sand layer (when outflow was found to become consistent) to just before deconstruction commenced there was no correlation ($R^2 = 0.05$, $n = 21$). Weak correlations were also determined for the other test rigs. For longer time periods without running water through the pavement, it is envisaged a correlation could be found.

No correlation was found between outflow start time and either pan evaporation (roughly cumulative between tests) or average temperature, using Bureau of Meteorology weather data.

Typically, outflow start times were in the order of a minute, except for Rig 3, which was about 2 minutes. These times could be taken as a rough indicator of hydraulic retention time.

4.1.2 Outflow Rate

Flow of water coming out of each test rig was calculated. Outflow rate is of immediate relevance to sizing of Aquifer Storage and Recovery infrastructure but also gives insight into the base conditions, which may in turn affect pollutant removal performance. For comparison purposes, the 8-minute average outflow was calculated in all cases. No instantaneous outflow values are given due to lack of data. First, it is useful to understand the flow behaviour. For the period of time when inflow is occurring, and when the base consisted of rock (as opposed to sand), a power model was used. This is reasoned to be appropriate due to the form of most hydraulic flow equations, including the kinematic wave equation (Ragan & Duru, 1972). In a few cases this equation was directly used to calculate the total volume discharged after eight minutes and hence the eight minute average outflow.

$$V = B \bullet t^{A} + C \tag{3}$$

where:

V = total volume collected for a run after a specific point in time

t = time elapsed since outflow started

A = an index

B = a coefficient

C = the y-intercept.

In reality, the flow regime will change when water from one side of the box opposite the outlet reaches the outlet and again when water from the other side opposite the outlet reaches the outlet. This change could not be determined accurately, with the problem significantly compounded by the sagging bases. A value of 1.2 for A was adopted, simply because increasing it to 1.3 for Rig 2 caused one run to have its eight minute flow indeterminable. The y-intercept C was incorporated to take into account the changing flow behaviour as a run progressed. Using this crude model, values for *B* and *C* are given in Table 39. Values for *B* generally followed the eight-minute average outflow, for Rig 2 the R^2 was 0.74. Outliers can be explained by the inherent error in the model.

Table 39: Outflow power model factors *B* and *C*

Pavement	*B* (L/day$^{1.2}$)	*C* (L)	Observations (ex. outliers)	Outliers
Rig 1 Rock Base	673 ± 103	-0.28 ± 0.1	60	6
Rig 2	699 ± 320	-0.22 ± 0.2	65	3
Rig 3	490 ± 193	-0.12 ± 0.2	43	1
Rig 4	697 ± 453	-0.21 ± 0.2	38	2

There are two limits to consider with this power model – when down flow onto the box base stops and when the maximum outflow is reached. Rigs 2 and 4 had several runs at the beginning of testing when roughly constant outflow was achieved. The lag between inflow stopping and outflow being affected was estimated to be about 1 minute for these runs. For Rig 3, it was found setting the lag time to 1 min 14 sec gave the least runs for which the eight-minute outflow could not be determined; therefore this was adopted. Using the runs for which the maximum outflow was reached, the 'run through' coefficient (analogous to the runoff coefficient) was determined to be about 55%; the rest was believed to be taken up by the clay attached to the aggregate. Varying between 0% and 100% only made on average a 10% difference to the average outflow for Rig 3, with negligible effects for the other rigs. If the average outflow between two sample collection times was greater than the calculated maximum outflow, the former was adopted.

Several runs had all three samples taken within the period covering inflow and lag, two each for Rig 1 and Rig 2 and 3 for Rig 4. For these, outflow was taken to stop immediately after the lag period ended. For rigs 2 and 4, outflow was observed to decrease sharply to almost 0 roughly a minute after the inflow stopped for the first few runs. As for Rig 1, for both of these runs the down flow onto the base was calculated to stop no earlier than eight minutes after inflow commenced.

After the lag period after inflow stopped, total volume collected was found to increase according to a logarithmic plot, given in equation (4).

$$V = a\ln(t-b)+c \tag{4}$$

where:

V = total volume collected for a run after a specific point in time

t = time elapsed since outflow started

a = multiplier, expected to be related to conditions at the base (among other variables);

$b = t$ – axis shift, believed to be related to the inflow stop time;

$c = V$ – axis shift, expected to be related to the volume of water in the test rig after inflow stops.

Six samples were collected for the run performed on Day 39 for Rig 1. A log curve was fitted with an R^2 of about 0.9993 and standard error of 0.02 L. The final sample for this run, collected after 74 hours, was excluded from regression as the sample was overflowing. The first sample was collected just before inflow stopped and was therefore excluded from regression as well. This seemingly logarithmically varying volume is believed to be at least partly due to lateral flow occurring within the clay cake layer at the base. An exponential model was also trialled, detailed in Appendix A, and rejected.

In the vast majority of cases the log equation was used directly to find the total volume discharged after eight minutes and hence the eight-minute average outflow. Of these cases, the power model was used to calculate the volume just after down flow stopped when required. Again, the log equation was not found to apply to Rig 1 when it had a sand base. For this situation, and in all cases where a log equation could not be fitted, the eight-minute volume was linearly interpolated between two sample collection points. Generally, error in linear interpolation for the eight-minute outflow was up to 14%, depending on the time between sample collection points. For linear extrapolation this was up to 50% (overestimation compared to the power and log models). For Rig 1 with the sand layer this was believed to be generally no more than 3% for runs terminated after 10 minutes (using the log model as an approximation).

Values of *a, b* and *c* are given in Table 40:

Table 40: Values of *a*, *b* and *c* for outflow log model

Pavement	*a* (L/log(day))	*b* (min)	*c* (L)	Observations (ex. outliers)	Outliers
Rig 1 Rock Base	0.23±0.06	4.7±0.9	2.1±0.4	59	6
Rig 2	0.07±0.06	5.8±1.2	1.1±0.4	64	2
Rig 3	0.19±0.06	5.0±1.5	1.7±0.4	45	2
Rig 4	0.09±0.08	5.5±1.2	1.2±0.4	38	1

For Rig 1, outflow is only considered after the inflow was stabilised as at Day 45 unless otherwise stated. Average eight-minute outflows are given in Table 41. After adding the sand layer, only 13 mL was collected the first day then 6 mL the second day, both after 5 minutes, with flow beginning after about 4 min 30 sec each time. On subsequent days the flow increased. Only from four days after adding the sand layer onwards is the outflow counted for this construction. One outlier was removed for Rig 2, which was the first run. This run was performed on the same day as washing, producing an eight-minute outflow of 2.5 mL/sec.

Table 41: Average Eight-Minute Outflow Rates from Each Pavement

Situation	Input = 2.25 L (mL/sec)			n	Input = 1.8 L (mL/sec)			n	t-test p-value
Rig 1 Rock Base	1.4	±	0.3	51	N/A				N/A
Rig 1 Sand Base	0.9	±	0.3	12	0.5	±	0.2	7	5×10^{-06}
Rig 2	1.4	±	0.4	58	0.7	±	0.3	7	3×10^{-06}
Rig 3	1.1	±	0.3	38	0.9	±	0.3	7	1×10^{-02}
Rig 4	1.4	±	0.6	35	0.6	±	0.3	7	7×10^{-10}

As can be seen from Table 41, lowering the input volume from 2.25 L to 1.8 L made a significant difference in all cases, applying two-tailed t-tests assuming unequal variance. Changing the loading rate but keeping the input volume at 2.25 L only made a significant ($p = 0.02$, $n_1 = 8$, $n_2 = 3$) difference for Rig 1 with the sand layer, apparently by 0.2 mL/sec. It also made a significant difference for Rig 3 ($p = 0.001$, $n_1 = 34$, $n_2 = 4$) but it apparently caused an increase in eight-minute outflow of 0.2 mL/sec.

For Rig 1, no trend of outflow with calendar time was found. Adding the sand layer significantly reduced the average eight-minute outflow by over 0.4 mL/sec ($p = 0.07$).

Rig 2 showed a downward trend in outflow at the beginning. From the second run to

Day 79, average eight-minute outflow decreased from 1.9 mL/sec by 0.015±0.002 mL/(sec•day) ($R^2 = 0.67$, $p = 6x10^{-6}$). This was possibly due to fine particles migrating from the aggregate onto the base, where it would slow the flow down more significantly. Such an effect for Rig 1 was likely obscured by the unintentionally varying inflow. It remained fairly stable until Day 137 at 1.4±0.3 mL/sec and was significantly different to the outflow of Rig 1 (paired t-test, $p = 0.05$, $d_f = 24$), but with Rig 1's apparently higher by 0.06 mL/sec. This is despite the different box slopes, likely because of significant clogging at the base and is possibly because Rig 1 was hosed down for longer.

No correlations were found for Rig 3, likely with negligible washoff of fine material onto the base during testing. Its outflow was significantly lower than Rig 1's by 0.3 mL/sec (paired t-test, $p = 2x10^{-4}$, $d_f = 22$), likely due to trapped water at the base buffering flow, where for Rig 1 the sag in the base was filled by sediment.

A correlation was found for Rig 4. A linear model was fitted with an R^2 of 0.79, as in Figure 11, with outflow reducing at 0.020±0.002 mL/(sec•day) ($p = 2x10^{-11}$). This effect is again likely due to fines migrating from the aggregate onto the base. Correlation did not improve with cumulative number of testing days as opposed to calendar day, possibly due to migration continuing to occur between tests. As Rig 2 was only washed as a whole with a hose and Rig 4 only had its aggregate sieved in water, the result appeared to be more washable suspended solids in Rig 4. Rig 2's outflow was significantly higher by on average over 0.1 mL/sec (paired t-test matching the second and first runs of Rig 2 and 4 respectively, $p = 0.02$, $d_f = 32$). This implies that when the solids settled to the base the accumulation was greater for Rig 4 than for Rig 2, resulting in lower final outflows. Rig 4 actually had a higher slope (3% and 9% each direction) compared to Rig 2 (2% and 6%).

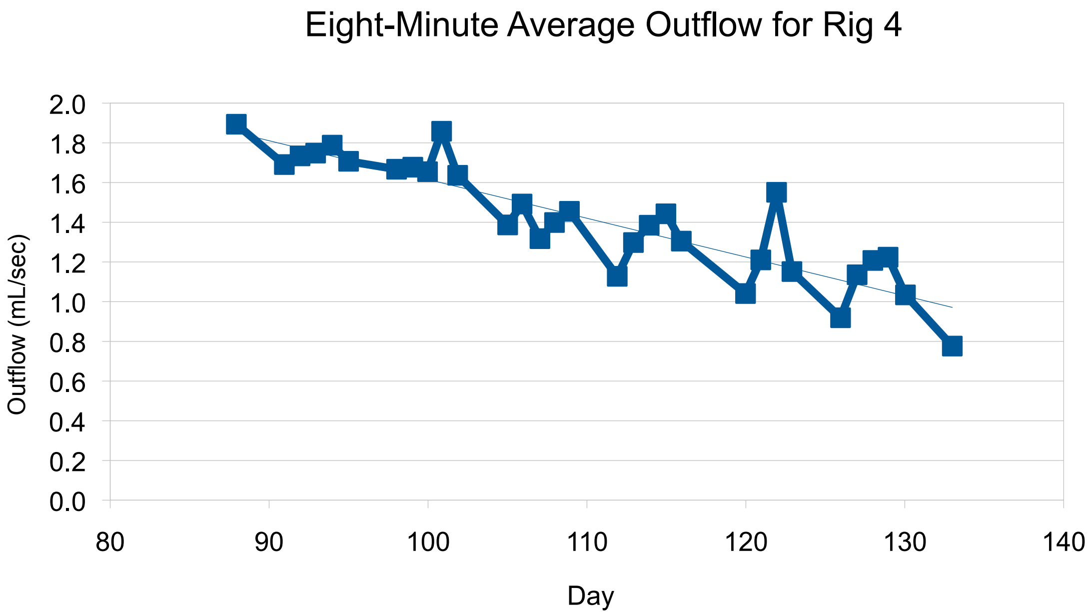

Figure 11: Eight-Minute Average Outflow for Rig 4

Stripping layers away produced some significant increases in outflow, as given in Table 42. For example, for Rig 2 removing all three layers significantly increased the outflow (p = 0.002). Using only the outflow data for when the input volume was 1.8 L, in all cases $n_1 = 7$, $n_2 = 1$, equal sample variance was assumed and the t-tests were one tailed. Surprisingly, stripping the layers away produced no significant increase in eight-minute outflow for Rig 3.

Table 42: Significance p-values for Increase in Flow Resulting from Stripping Pavement Layers

	Cumulative Flow Increase		
Pavement	No Pavers	No Bedding	No Geotextile
Rig 1 Sand Base	0.055	0.02	0.001
Rig 2	0.03	0.002	0.002
Rig 3	0.21	0.4	0.29
Rig 4	0.06	0.01	0.0002

Rigs 1, 2 and 3 had reasonably stable outflow rates possibly because they were all hosed down. Rig 4 had an outflow rate that varied significantly as the days progressed, likely because it was not hosed down, therefore the fines would have migrated from the aggregate to the base during testing. Formation of a sludge layer at the base is believed to affect pollutant removal performance.

4.2 Water Quality

Water quality analysis was undertaken on the samples collected from the four pervious pavement test rigs. After dripping through the input solution on each testing day, leftover solution was collected from the dripper manifold for testing. Ammonium concentration was at 1.3±0.4 mg (N)/L (n = 100) with the exception of three outliers when the concentration was over 2 mg/L on 30/8/2011, 12/12/2011 and 19/12/2011 possibly due to contamination. Phosphate concentration was at 0.78±0.14 mg (P)/L (n = 102), with the exception of one outlier where the concentration was 2.64 mg/L on 28/10/2011, possibly due to contamination. None of these outlier input concentrations impacted significantly on the output concentration, suggesting the input solution samples were contaminated instead of the solution put through the pavements.

First, it is useful to ascertain how water quality varies over time as the samples are collected for a single run. For Rig 1 there were two runs for which six samples were collected – one on Day 9 and one on Day 39. On Day 9, 9.5 L of solution was dripped through over 16 minutes (99 mm/hr) and samples were collected at roughly 1 L intervals. Maximum outflow was achieved before the first or second sample. Figure 12 indicates that the ammonium and phosphate concentrations appeared to approach the input concentration over time, while the nitrate and nitrite concentrations appeared to decay to zero exponentially (R^2 = 0.98). Shown are the samples taken before inflow stopped.

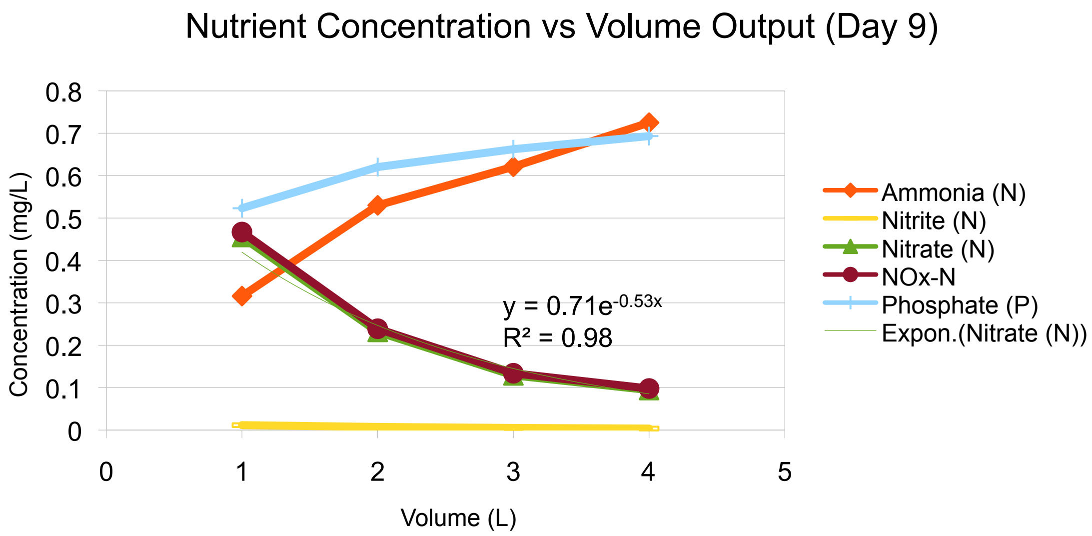

Figure 12: Nutrient Concentration vs Volume Output for Day 9

On Day 39, a run was performed and six samples were collected, with the sixth collected after about three days. The output concentrations decreased over time. Figure 13 presents nutrient loads over time, with log curves fitted with varying degrees of success.

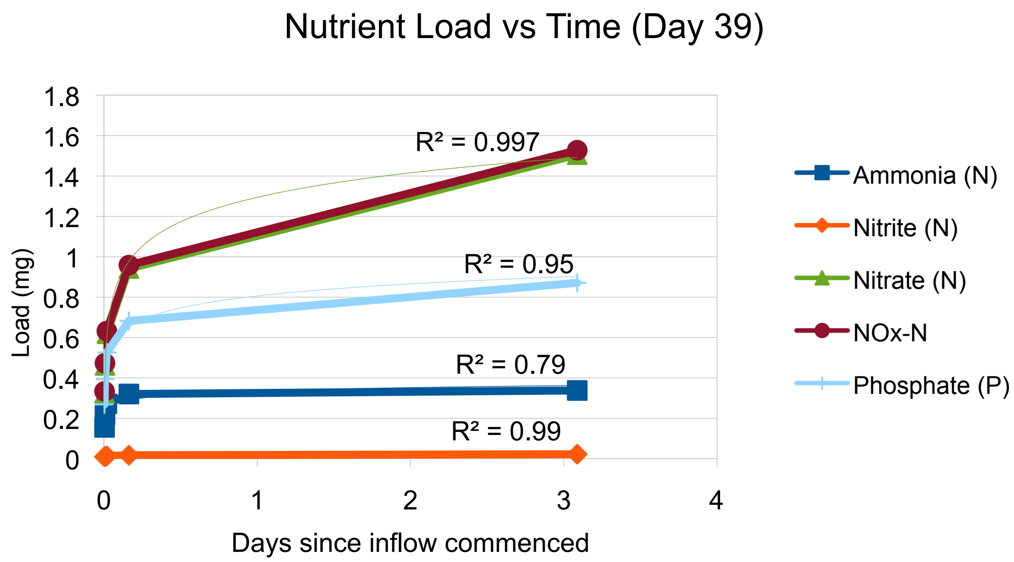

Figure 13: Nutrient Load vs Time for Day 39

Generally, no definite rule of concentration over time could be determined for either case of during inflow loading or after, with more data required. In light of this decreasing output concentration over time after inflow stops, only runs with samples collected within 11 minutes are considered for calculating the EMC. In the experiments performed, a constant input concentration was used, whereas in the field situation the input concentration will start high and decrease as pollutant is washed off the surface. This is especially the case after a long dry period, during which pollutants accumulate. This effect is commonly referred to as the first flush effect. It will affect results and future testing should take this variation in concentration into account.

4.2.1 Nitrogen

Ammonium, nitrite and NO_x-N concentrations were measured and compared to guideline values. The aquifer concentration requirement of 0.11 mg (N)/L is not met by rigs 2 (Figure 15) or 4 (Figure 17); yet it is improved on at times by Rig 1 (Figure 14) when it had the sand layer and Rig 3 (Figure 16), likely due to its granular activated carbon layer. Both needed time for the bacteria to develop.

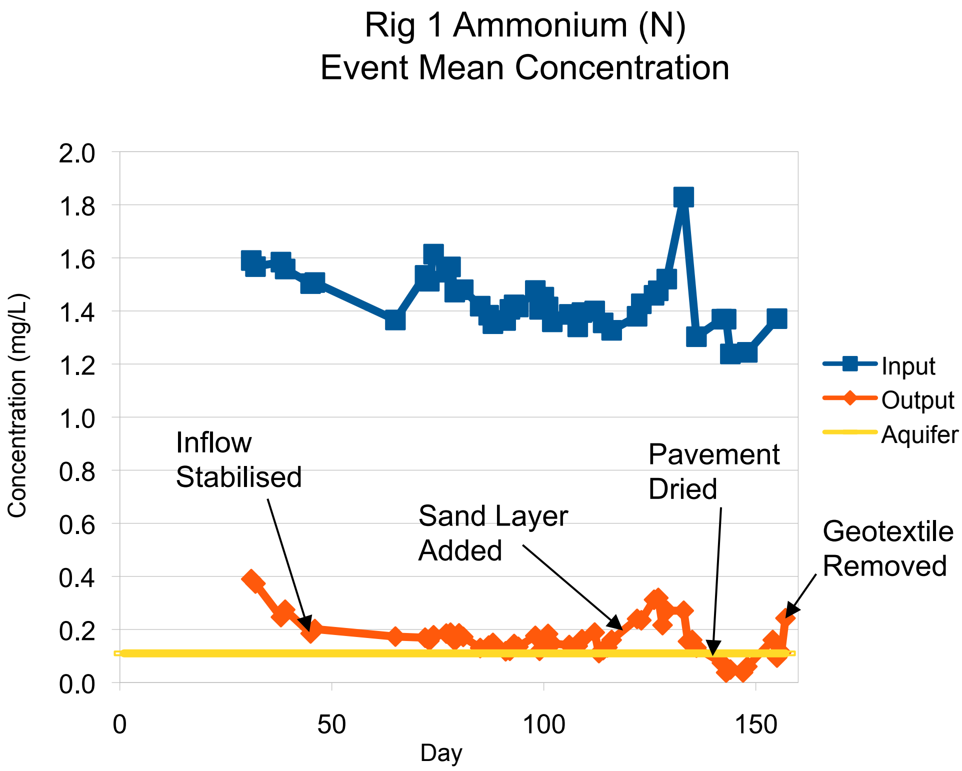

Figure 14: Rig 1 Ammonium Event Mean Concentration

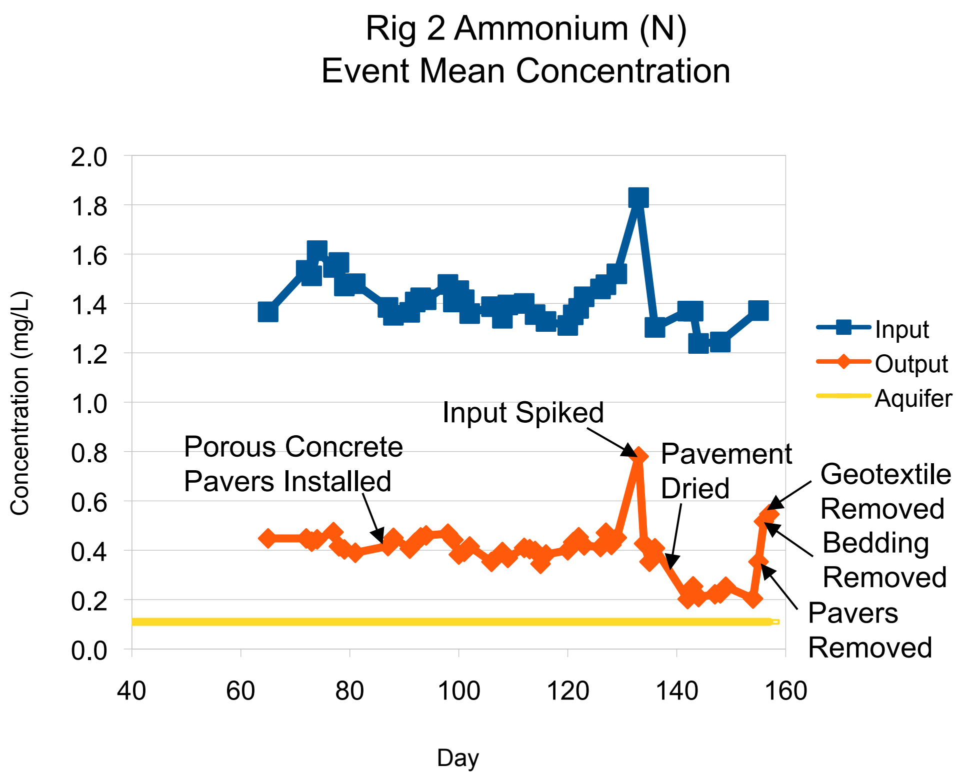

Figure 15: Rig 2 Ammonium Event Mean Concentration

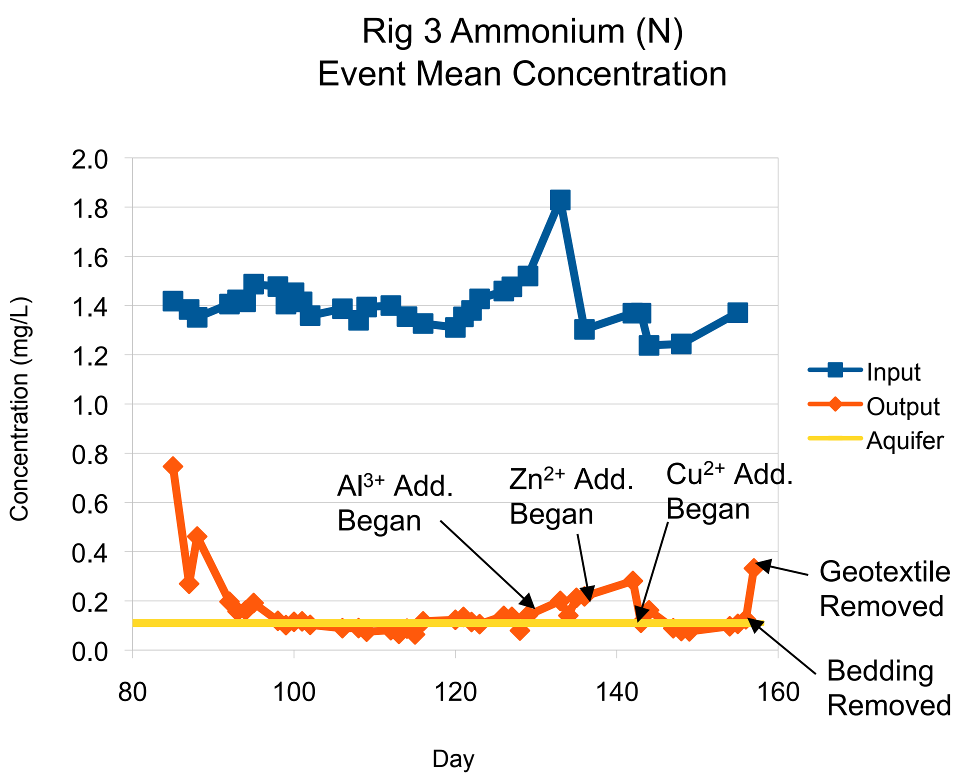

Figure 16: Rig 3 Ammonium Event Mean Concentration

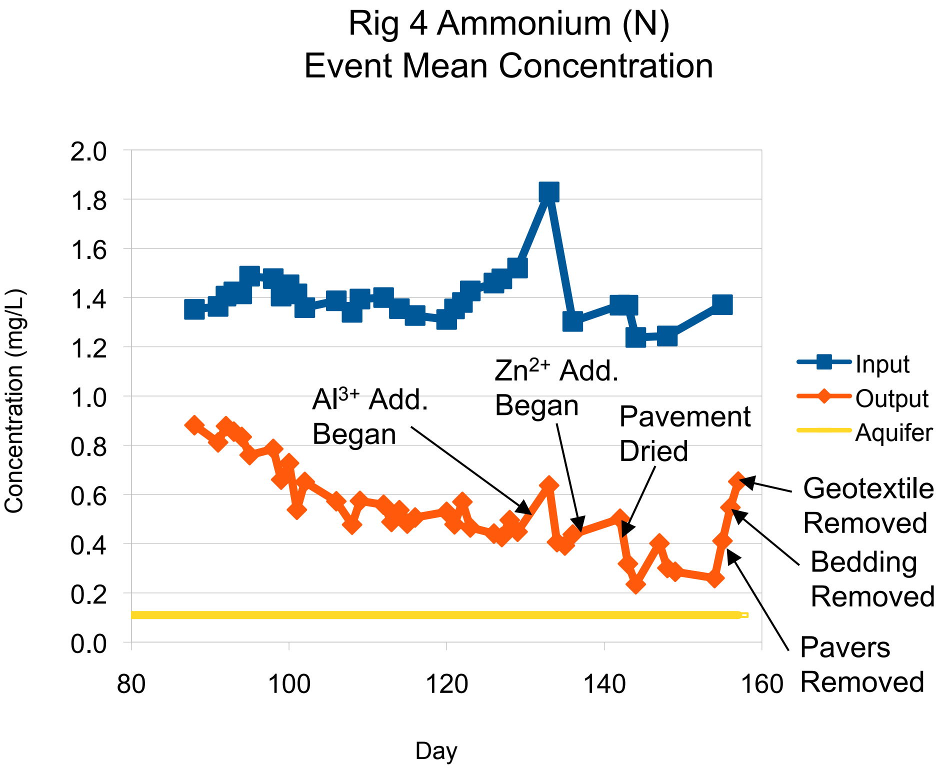

Figure 17: Rig 4 Ammonium Event Mean Concentration

As indicated in Table 43, Rig 3 and Rig 1 with the sand layer demonstrated the best ability to remove ammonium, followed by Rig 1 without the sand layer, Rig 2 and finally Rig 4. It should be noted this analysis ignores the first five runs for Rig 3, when the bacteria was developing. The poor performance of Rig 4 before the layer of fines formed on the base suggests this layer of fines has a significant ammonium removal capacity. The spike in ammonium concentration immediately after cleaning out the base in Rig 1 and adding the sand layer further demonstrates the ability of the fines layer to host microbes to remove/convert ammonium.

Table 43: Ammonium (N) Performance Comparison

Rig 1 Rock Base			Rig 1 Sand Layer		
Comparison	95% Difference (mg/L)	n	Comparison	95% Difference (mg/L)	n
Rig 4 – Rig 2	0.2	18	Rig 4 – Rig 2	0.02	22
Rig 2 – Rig 1	0.25	27	Rig 2 – Rig 1	0.18	19
Rig 1 – Rig 3	0.02	16	Rig 1 – Rig 3	None	19

Rig 4 shows a notable downwards trend in ammonium output concentration (R^2 = 0.84 before stripping layers, excluding when input was spiked, see next paragraph). This suggests that as fines built up on the base over time, a stable media was formed for microbes to grow in.

From Day 138 to Day 141 inclusive no solution was added to the test rigs. During this period, the pavements are likely to have dried out significantly. Hot summer conditions also prevailed during December, the month these days were in. These conditions are believed to have been favourable for ammonium conversion/removal. With 95% confidence and using unpaired t-tests assuming equal population variances, the ammonium concentration from Rig 1 dropped by at least 0.1 mg (N)/L (n_1 = 9, n_2 = 6), from Rig 2 by 0.17 mg (N)/L (n_1 = 38, n_2 = 7) and from Rig 4 by 0.16 mg (N) /L (n_1 = 30, n_2 = 7). No significant difference was found for Rig 3 (n_1 = 4 – during aluminium addition, n_2 = 7).

Adding metals to Rig 3 impacted on the ammonium removal performance, as indicated by the increase shown in Figure 16. This was only temporary, possibly due to the favourable weather conditions in December dominating. In particular, adding aluminium alone increased the output concentration by at least 0.04 mg (N)/L (unpaired t-test, 95% confidence, n_1 = 25 – ignoring first five runs, n_2 = 3).

Due to human error, on Day 133 the input solution concentration of ammonium was 1.8 mg (N)/L, with the distinct spike in output concentrations visible for Rig 2 (Figure 15) and Rig 4 (Figure 17). For Rig 2, the increase was at least 0.3 mg (N)/L with 95% confidence (unpaired, n_1 = 38, n_2 = 1), almost the entire increase in input concentration. For Rig 4, on Day 133 the 95% prediction interval for the estimated linear trend was 0.27 – 0.56 mg (N)/L. The spike exceeded the upper limit by 0.07 mg (N)/L.

Rig 1 without the sand layer and Rig 2 demonstrated stable output concentrations. Ignoring the period when inflow was unstable (to Day 44) and when the input concentration was spiked, Rig 1 had an output ammonium concentration of 0.16±0.05 mg (N)/L (n = 31). Before Day 142, Rig 2 had an output concentration of 0.42±0.06 mg (N)/L (n = 38).

Removing the top layers also caused a large spike in ammonium levels, implying these layers were important for ammonium conversion (Table 44). With a confidence of 95%, removing the pavers from Rig 2 made a significant difference. It is interesting that the porous concrete pavers significantly contributed to ammonium removal for Rig 2 but not Rig 4, perhaps because the bedding was less thoroughly washed for Rig 2, giving the bacteria more surface area to grow on. Only after removing the geotextile (including any granular activated carbon) from rigs 1 and 3 was a significant increase in ammonium concentration found, suggesting these layers had an important role.

Table 44: Cumulative Minimum Ammonium (N) Increase Due to Stripping Pavement Layers (95% confidence)

Pavement	Strip Pavers (mg/L)	Strip Bedding (mg/L)	Strip Geotextile (mg/L)	n_1	n_2
Rig 1	None	None	0.071	6	1
Rig 2	0.084	0.248	0.277	7	1
Rig 3	None	None	0.051	7	1
Rig 4	None	0.026	0.131	7	1

Overall, ammonium removal is highly variable, with the output varying from 0.04 mg (N)/L to 0.9 mg (N)/L (40% removal to 97% removal using intended input concentration), depending on conditions.

The NO_x-N concentration was even more difficult to manage, with output concentrations always higher than the ANZECC irrigation guideline of 0.02 – 0.06 mg (N)/L, as shown in Figure 18 through Figure 21. With the exception of Rig 1, where the varying inflow rate obscured the results (Figure 18), a peak in NO_x level is reached before it slowly reduces. Adding the sand layer caused two spikes in NO_x, the first when the layer was first placed and the second when it was left for four days in summer. Possibly due to the dry condition of the sand before these spikes, nitrifying bacteria would have nitrified ammonium to create the NO_x, but the required anoxic conditions were not present for denitrifying bacteria to convert the NO_x to nitrogen gas.

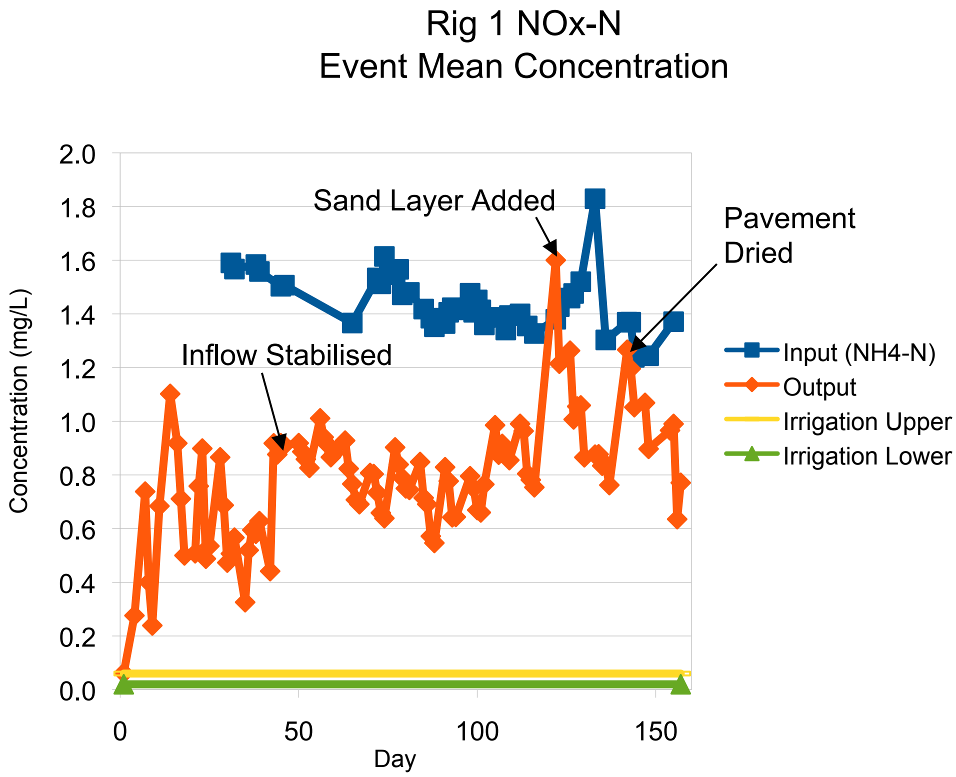

Figure 18: Rig 1 NO_x-N Event Mean Concentration

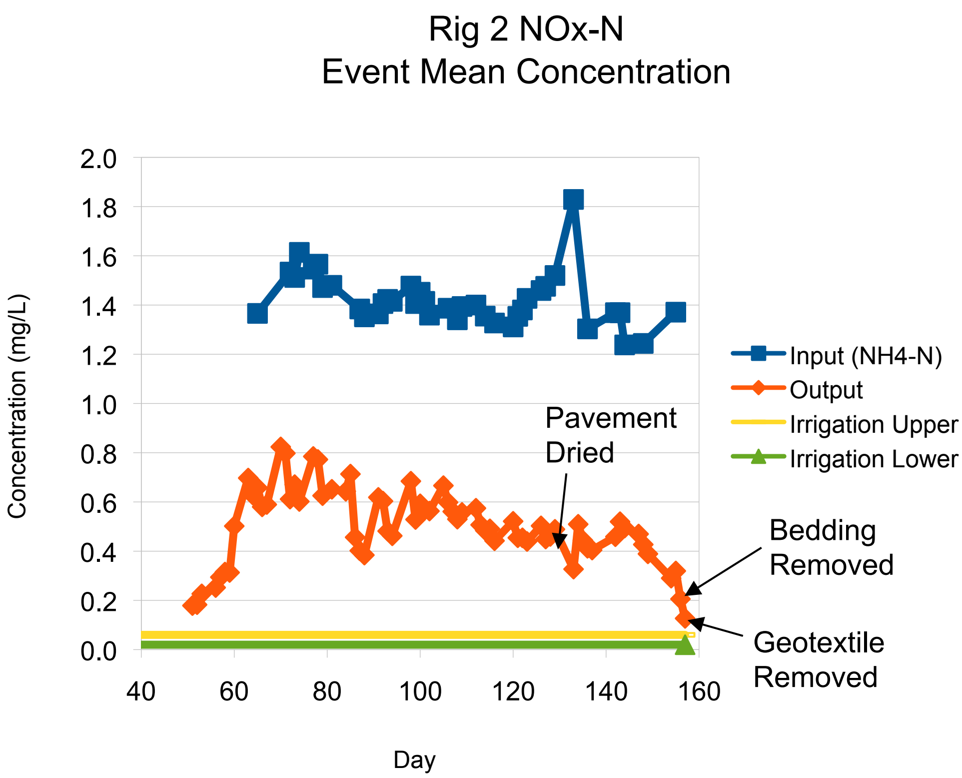

Figure 19: Rig 2 NO_x-N Event Mean Concentration

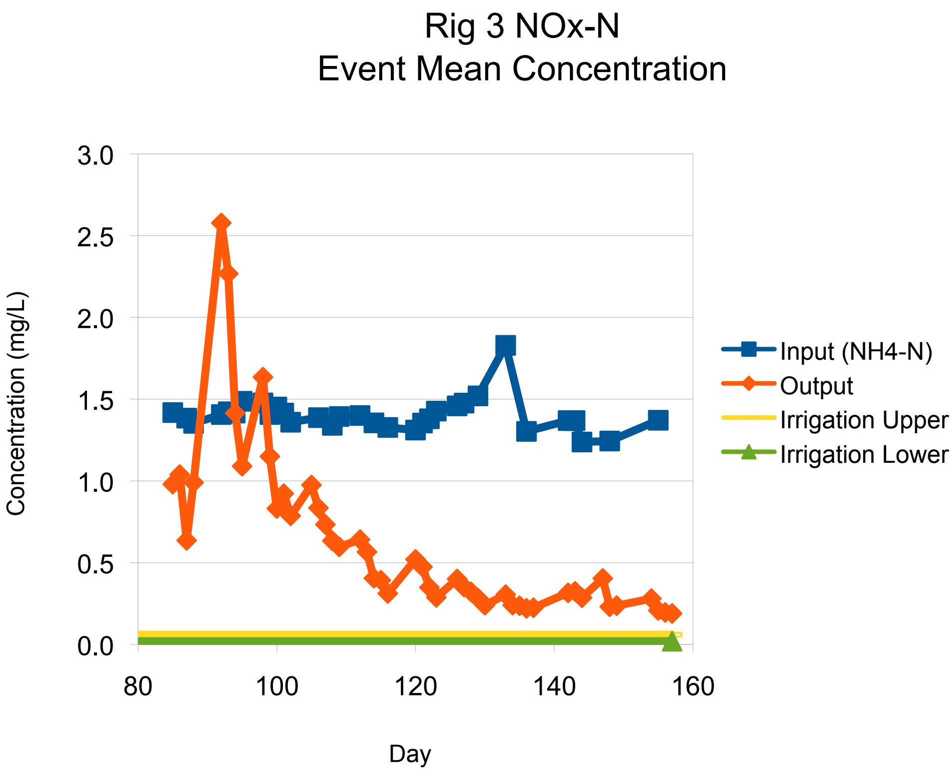

Figure 20: Rig 3 NO_x-N Event Mean Concentration

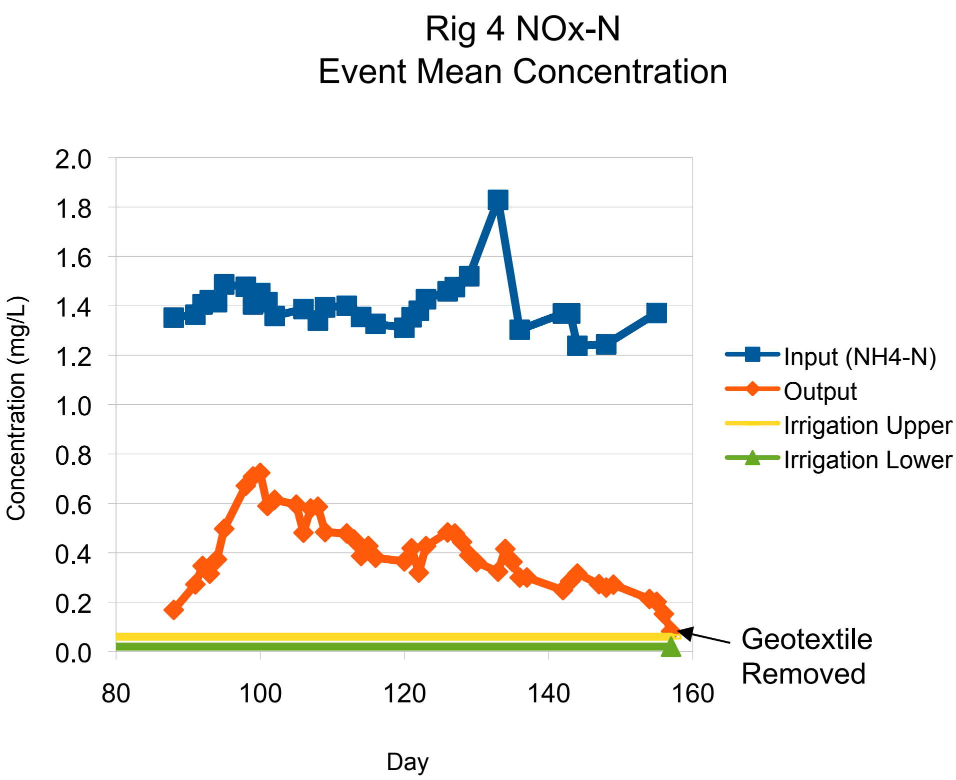

Figure 21: Rig 4 NO_x-N Event Mean Concentration

All rigs performed similarly for NO_x removal, as indicated in Table 45, with the exception of Rig 1 with the sand layer (Figure 18) and a large spike for Rig 3 at the start (Figure 20), with the latter due to the negatively charged granular activated carbon layer repelling the negative NO_x ions, out of the main source of bacteria. Again, this analysis ignores the first 11 runs for Rig 3, when the bacteria were developing. Removing the top layers did not significantly increase NO_x concentrations, implying the base was responsible for this role, with relatively anoxic conditions at this depth. In fact, concentrations decreased in some cases, possibly due to reduced conversion from ammonium to NO_x.

Table 45: NO_x-N Performance Comparison

Rig 1 Rock Base			Rig 1 Sand Layer		
Comparison	95% Difference (mg/L)	n	Comparison	95% Difference (mg/L)	n
Rig 1 – Rig 3	0.06	13	Rig 1 – Rig 2	0.53	20
Rig 3 – Rig 2	0.04	13	Rig 2 – Rig 3	0.08	23
Rig 2 – Rig 4	0.01	20	Rig 3 – Rig 4	None	24

Overall, NO_x proved challenging to remove and will require further effort to meet the guideline, unless it can be removed within the aquifer.

4.2.2 Phosphorus

Phosphate was measured from each of the test rigs. Iron in the clay attached to the aggregate is believed to be responsible for much of the phosphate removal. Figure 22 through Figure 25 show that after inflow stabilised, Rig 1 (Figure 22) performed the best phosphate removal, with the sand layer dramatically improving performance and even meeting the ANZECC freshwater guideline most of the time. Table 46 supports the conclusion that Rig 1 performed the best phosphate removal. The iron in the yellow sand is believed to be responsible for the substantially improved performance. Rig 2 (Figure 23) performed next best, likely due to its higher slope, with Rig 4 (Figure 25) performing slightly worse, perhaps due to the higher outflows at the beginning and the fact more clay was washed from it during construction. The granular activated carbon (Figure 24) appeared to be contaminated with phosphate and in fact emitted 14 mg (P)/L at the very beginning (sample 1 of the first run), eventually performing similarly to Rig 1 without the sand layer.

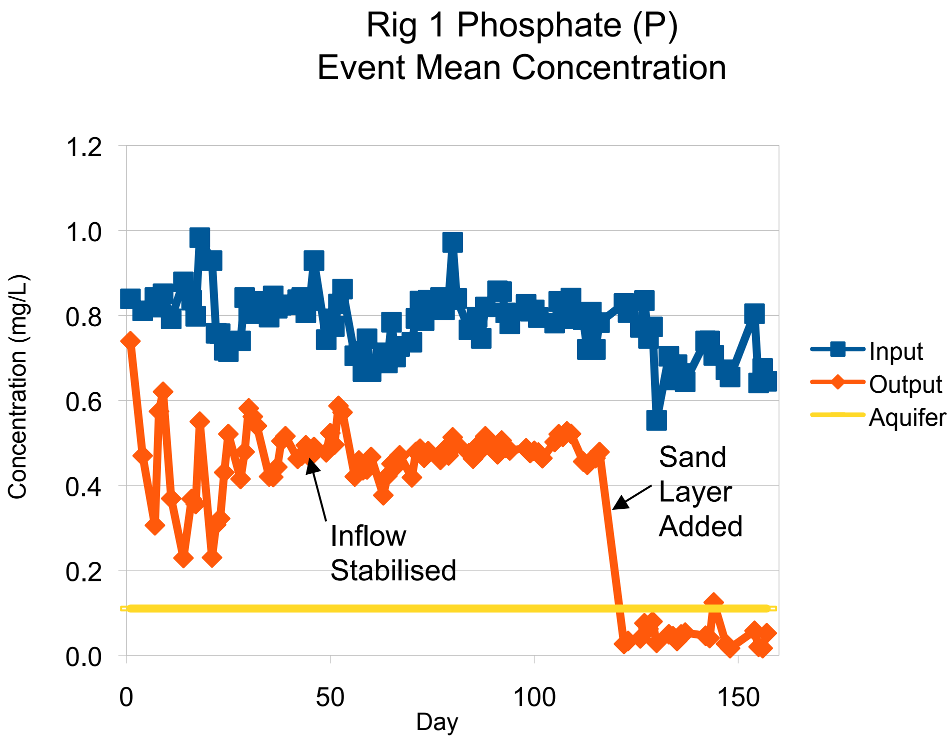

Figure 22: Rig 1 Phosphate Event Mean Concentration

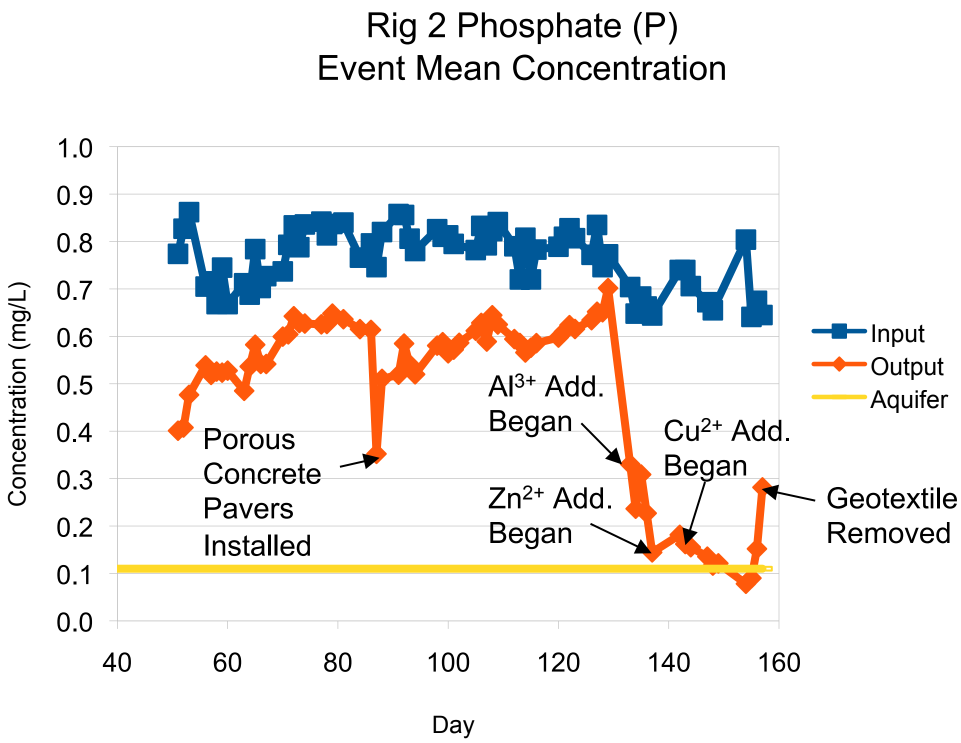

Figure 23: Rig 2 Phosphate Event Mean Concentration

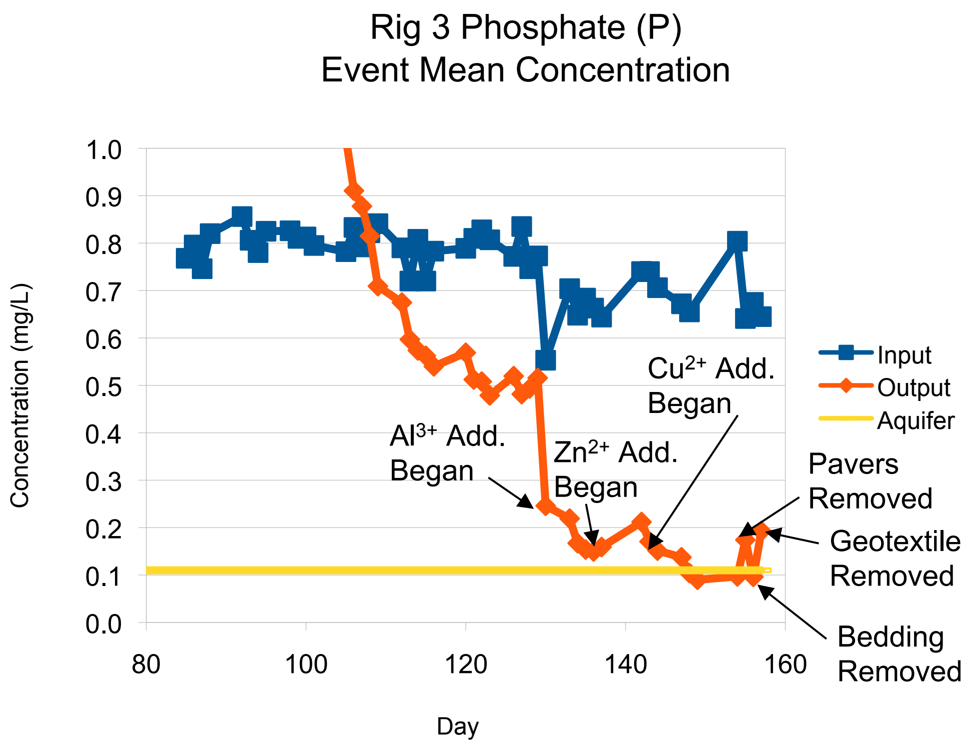

Figure 24: Rig 3 Phosphate Event Mean Concentration

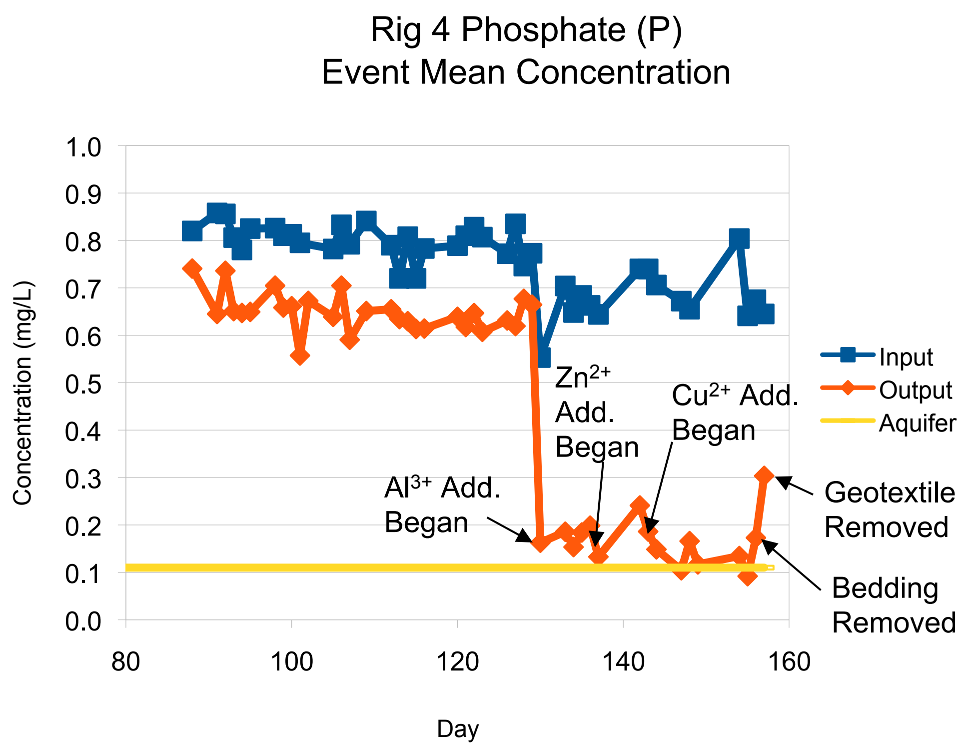

Figure 25: Rig 4 Phosphate Event Mean Concentration

Table 46: Phosphate (P) Performance Comparison

Rig 1 Rock Base			Rig 1 Sand Layer		
Comparison	95% Difference (mg/L)	n	Comparison	95% Difference (mg/L)	n
Rig 3 – Rig 4	0.33	19	Rig 2 – Rig 4	None	23
Rig 4 – Rig 2	0.05	19	Rig 4 – Rig 3	0.02	24
Rig 2 – Rig 1	0.09	42	Rig 3 – Rig 1	0.15	21

Two rigs had periods where output concentration of phosphate was stable. Before the sand layer was added and while outflow was stable, Rig 1 had an output concentration of 0.48±0.07 mg (P)/L (n = 51). Before adding metals, Rig 4 had an output concentration of 0.65±0.08 mg (P)/L.

Figure 23 indicates two significant low points in phosphate output concentration for Rig 2. Before adding metals, the output concentration was generally increasing, presumably with the capacity of the clay to absorb phosphate reducing over time. In particular, the first two points were lower by at least 0.12 mg (P)/L (95% confidence, unpaired t-test, $n_1 = 2$, $n_2 = 50$).

When the pavers with gravel filled gaps were replaced with porous concrete on Day 87 for Rig 2, with 95% confidence a one off reduction in phosphate concentration of at least 0.15 mg (P)/L occurred (unpaired t-test, $n_1 = 50$, $n_2 = 1$). Figure 23 suggests a more sustained reduction in output concentration, with the porous concrete possibly continuing to absorb phosphate. This is believed to be due to the calcium from the cement precipitating phosphate out, similar to the iron from the clay. Excluding these two low points, the output phosphate concentration was 0.59±0.1 mg (P)/L (n = 50). The two low points were also excluded from the analysis in Table 47.

Adding metals to the input solution precipitated some of the phosphate out, explaining the drop in output concentration for rigs 2-4 (Figure 23 through Figure 25). The measured input concentration dropped to roughly 0.7 mg (P)/L. Using unpaired t-tests, adding aluminium alone caused a decrease by at least 0.2 mg (P)/L, except for in Rig 1 (Table 47). Phosphate will form aluminium phosphate ($AlPO_4$) with aluminium, copper phosphate ($Cu_3(PO_4)_2$) with copper and zinc phosphate ($Zn_3(PO_4)_2$) with zinc, having solubility products of 6.3×10^{-19}, 2.05×10^{-35} and 9.0×10^{-33} respectively, all at 25˚C (California State University; Generalic, 2003) (California State University solubility products taking preference). After aluminium addition alone, the dissolved phosphate concentration in the input would have been undetectable at $7x10^{-13}$ g (P)/L, suggesting that either incomplete precipitation occurred or the Aquakem machine was detecting precipitated phosphate.

Table 47: Incremental Minimum Decrease in Phosphate (P) Output from Adding Metals (95% Confidence)

	Add Al^{3+}			Add Zn^{2+}			Add Cu^{2+}		
Pavement	Dec. (mg/L)	n_1	n_2	Dec. (mg/L)	n_1	n_2	Dec. (mg/L)	n_1	n_2
Rig 1	None	6	5	None	5	2	None	2	5
Rig 2	0.19	85	4	0.03	4	2	None	2	6
Rig 3	0.28	7	5	None	5	2	0.01	2	6
Rig 4	0.44	28	5	None	5	2	None	2	6

Of the layers removed, it was only after removing the geotextile that a significant (95% confidence) increase in phosphate was found, implying it likely contributed to phosphate removal. This only applied to rigs 2 and 4 ($n_1 = 7$, $n_2 = 1$) (Figure 23 and Figure 25). For Rig 1, most of the phosphate removal appeared to be performed in the sand layer and for Rig 3 the granular activated carbon must have precluded any significant phosphate absorption within the geotextile.

Overall, phosphate output concentrations ranged from 0.02 to 0.74 mg (P)/L, excluding Rig 3 with its contaminated GAC (12% to 98% removal overall assuming the intended input concentration).

4.2.3 Dissolved Organic Carbon

Dissolved Organic Carbon (DOC) was tested for as well and compared to guideline values. According to the managed aquifer recharge guideline (Environment Protection and Heritage Council, 2009a), a DOC level of 1-10 mg/L would require 'moderate' maintenance. All rigs' output was generally within this range, according to Figure 26 through Figure 29. This is not surprising as the input was only 4 mg/L. The lower the DOC, the less bacteria have a chance to grow and therefore less maintenance will be required at the injection well performing managed aquifer recharge.

Addition of timber supports for the dripper manifold may have resulted in contamination. In fact, for Rig 1 the addition of the timber support seems to have caused an increase in DOC of at least 2.8 mg/L after a week (95% confidence, unpaired, $n_1 = 3$, $n_2 = 3$) (Figure 26), using the data after inflow stabilised and excluding when the geotextile was removed. The other support was stainless steel and would not have affected results noticeably. For the other test rigs there is not enough good data for comparison.

According to Table 48, Rig 3 (Figure 28) appears to have performed the best with its granular activated carbon layer, followed by Rig 2 (Figure 27) and Rig 4 (Figure 29). However, it is possible that Rig 3 performed well simply because timber supports for the dripper manifold were never installed in it (with acrylic ones used instead), while they were for the other test rigs. Adding the sand layer to Rig 1 appears to have worsened DOC removal (Figure 26), whether due to inadequate time for the bacteria to develop or the sand being a carbon source is unclear. When Rig 1 did not have the sand layer, no significant differences between rigs were found due to lack of valid data.

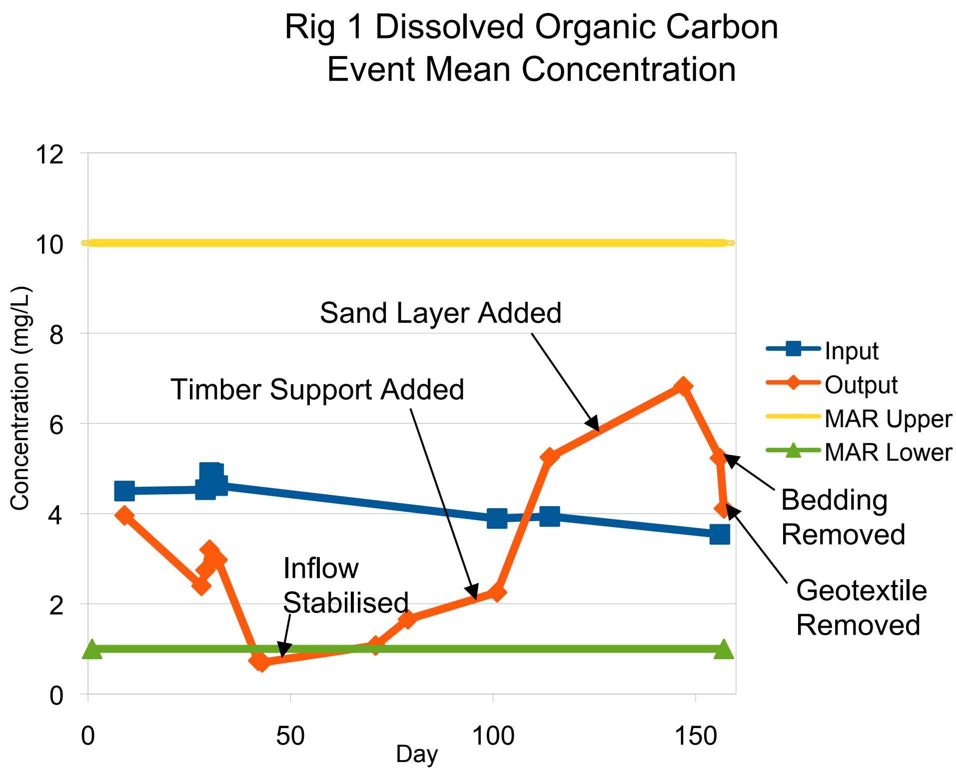

Figure 26: Rig 1 Dissolved Organic Carbon Event Mean Concentration

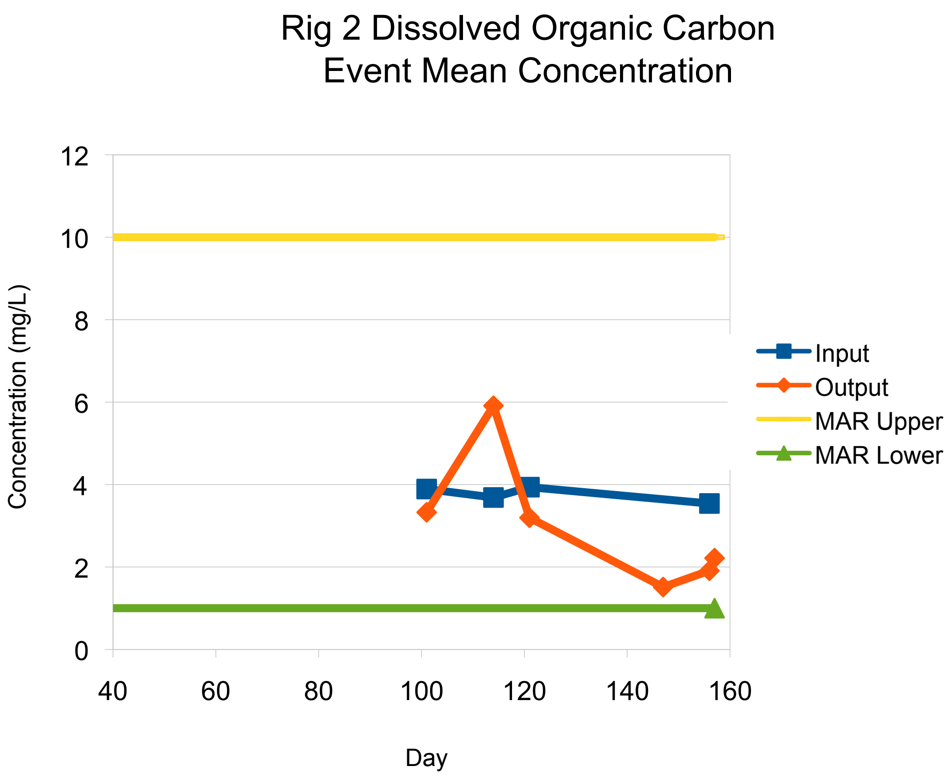

Figure 27: Rig 2 Dissolved Organic Carbon Event Mean Concentration

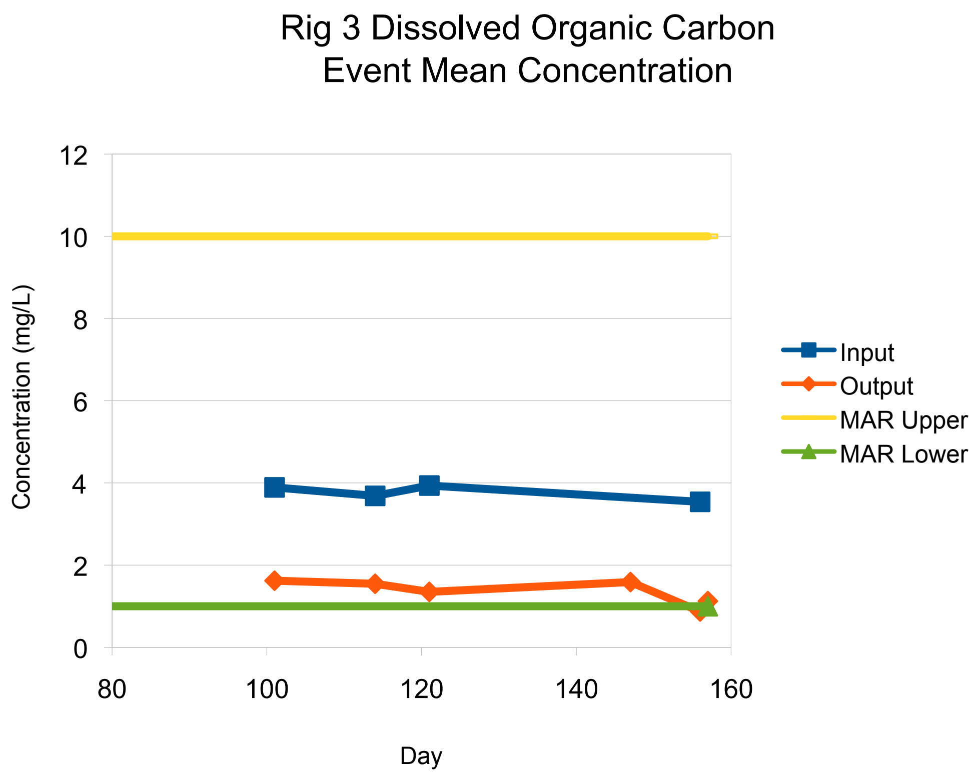

Figure 28: Rig 3 Dissolved Organic Carbon Event Mean Concentration

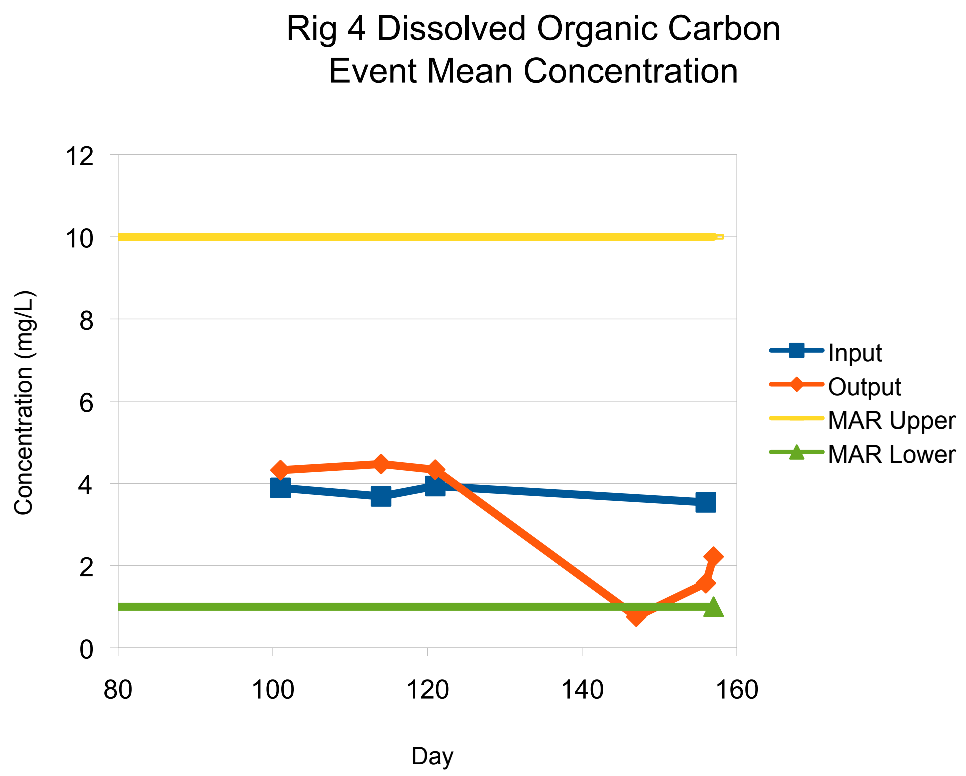

Figure 29: Rig 4 Dissolved Organic Carbon Event Mean Concentration

Table 48: Dissolved Organic Carbon Performance Comparison

Rig 1 Sand Layer		
Comparison	95% Difference (mg/L)	n
Rig 1 – Rig 4	0.34	3
Rig 4 – Rig 2	None	4
Rig 2 – Rig 3	0.04	4

Overall, it is unclear how the pavers, bedding layer or geotextile impacted on DOC concentrations, due to lack of data. Nonetheless, microbial activity will have occurred within all these layers, especially the bedding and geotextile, removing DOC from solution.

Contamination from the timber supports seemed to impact materially on results for rigs 1, 2 and 4. Treatment of dissolved organic carbon is beneficial for reducing maintenance of the Aquifer Storage and Transfer bore and where practical, sources of contamination should be eliminated.

4.2.4 Total Suspended Solids

Total suspended solids were a significant problem. The managed aquifer recharge guidelines (Environment Protection and Heritage Council, 2009a) suggest a suspended solids level of 1-10 mg/L will require 'moderate' maintenance at the injection well. All rigs' output consistently exceeded 10 mg/L. Nevertheless, Rig 1 (Figure 30) always emitted the least suspended solids, possibly due to the stability of the clay / sand layer at the base and the shallow slope of this base. Rig 3 (Figure 32) performed similarly, with the same slope as Rig 1, but the data is inconclusive. No significant differences were found between any of the rigs for TSS, due to the lack of data.

Rigs 2 (Figure 31) and 4 (Figure 33) fared poorly, likely due to the high slope of the base. More tests were performed for Rig 2 because of its high initial reading. For rigs 2 and 4, removing the top two layers worsened the TSS output, possibly due to the increase in outflow. Washing more thoroughly would have alleviated the suspended solids problem. Rig 1 performed considerably better because of the shallow slope and because it had been rinsed thoroughly with a hose before testing, despite sieving having thought to be a more thorough method. For Rig 1, washing only the sand layer may have been adequate, as the suspended solids were observed to be the same yellow as the sand rather than the typical reddish orange. This implies the sand was trapping clay.

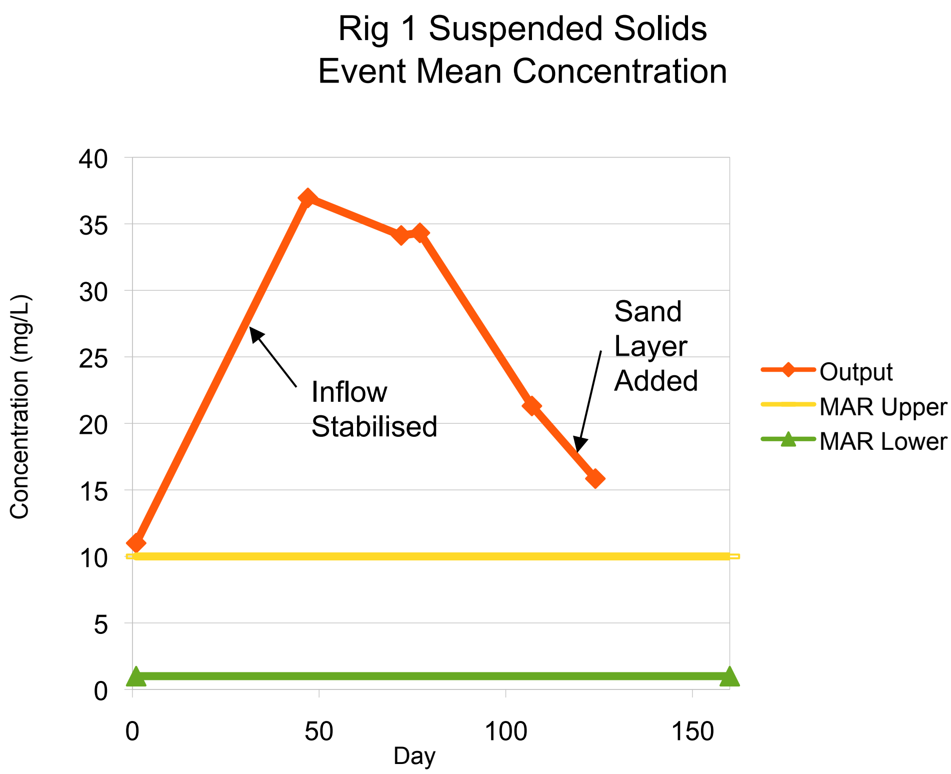

Figure 30: Rig 1 Suspended Solids Event Mean Concentration

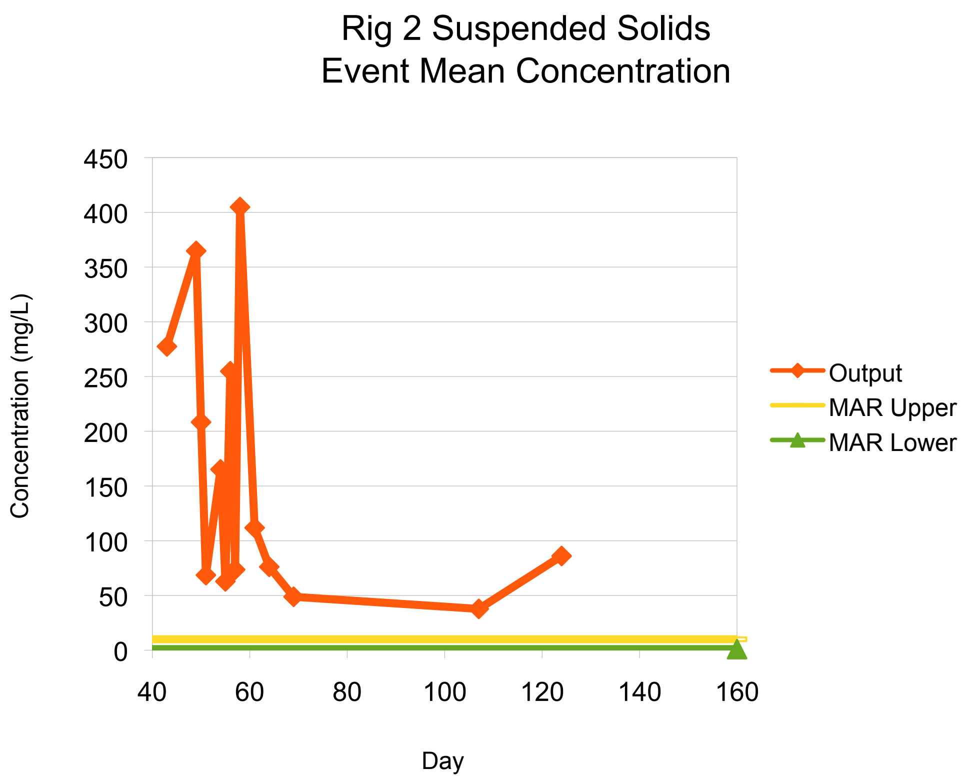

Figure 31: Rig 2 Suspended Solids Event Mean Concentration

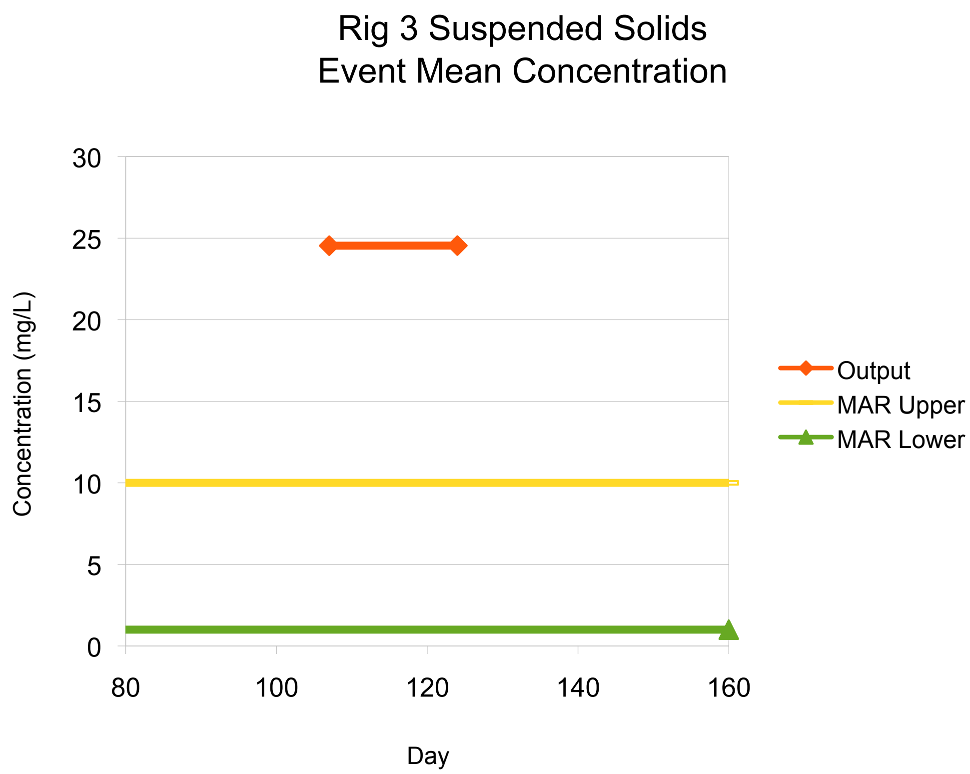

Figure 32: Rig 3 Suspended Solids Event Mean Concentration

Figure 33: Rig 4 Suspended Solids Event Mean Concentration

Different washing methods and slopes are likely to have affected suspended solids concentrations in the output. In no case was it considered acceptable for Aquifer Storage and Transfer. There was significant washoff of solids from the aggregate within the pavements, with more treatment required. Adding a layer of washed sand at the base could provide this treatment, by filtering out the solids from the aggregate above it.

5 CONCLUSIONS

Stormwater management is seeing increasing attention in recent times. With the traditional method of piping stormwater directly to the nearest watercourse no longer appropriate, alternatives are being sought. Water Sensitive Urban Design is providing new ways of dealing with stormwater, reducing exported pollutants and attenuating both runoff volumes and flows. In addition, stormwater is being looked to as a resource. Aquifers are commonly drawn on for water supply and these can be recharged with stormwater via managed aquifer recharge. Aquifers are commonly used for irrigation. When the direct injection approach is taken, as can be the case with confined aquifers, stormwater must be treated prior to injection. Pervious pavement could possibly be one such treatment method.

Pervious pavement is a type of pavement that allows rainwater to infiltrate directly through it with significantly reduced runoff compared to traditional impermeable pavements. The surface can come in many forms, including porous concrete, pavers with gravel filled gaps, plastic reinforcing grids enclosing turf or gravel and more. Beneath are layers of open graded or gap graded gravel for storage or conveyance, often with a geotextile layer included.

Laboratory testing was undertaken to evaluate the nutrient removal performance of four test rigs, using porous concrete and pavers with gravel filled gaps for the surfaces. One test rig had granular activated carbon sandwiched between two layers of geotextile immediately below the bedding (Rig 3), while the rest had a single layer of geotextile at this depth. Another test rig had a sand layer topped with geotextile added at the base after a period of time (Rig 1). All had gravel base courses.

Rig 1 with the sand base and Rig 3 with the activated carbon performed similarly for ammonium removal and removed the most ammonium out of the four rigs. For each of these two rigs there were periods where the output ammonium concentration was better than in the Leederville aquifer in Perth Western Australia. This suggests these pavements could treat ammonium to a level suitable for Managed aquifer recharge. Ammonium removal was sensitive to many factors including inflow rate, metal addition, extent of washing of the gravel layers, input concentration, establishment of nitrifying bacteria over time, moisture content and temperature.

Products of nitrification, namely nitrite and nitrate, were more difficult to reduce to a level acceptable for managed aquifer recharge. If this is not substantially removed in the aquifer itself, additional measures will be required to remove it, ideally within the pervious pavement.

Phosphate could be removed to a level suitable for managed aquifer recharge, but applying the strict ANZECC guideline for freshwater, this was only achieved by adding the sand layer to Rig 1. Overall, phosphate removal was quite stable, but only with stable inflow and contamination from the granular activated carbon was an exception.

Results from dissolved organic carbon testing were somewhat inconclusive but granular activated carbon gave a clear advantage. The lower DOC is, the less maintenance will be required at the injection well for Managed aquifer recharge. The same applies for suspended solids. The results from suspended solids testing was also inconclusive but if the sand layer were washed, suspended solids coming from the pavement may become acceptable, with the sand layer straining solids coming from the gravel layers. Evidence of this is that the suspended solids in solution were the same yellow as the sand and not the typical reddish orange.

Using porous pavement for stormwater reuse is somewhat more difficult than simply for stormwater disposal, with higher water quality standards required depending on the use. However, with the right techniques and construction it could be done. Further testing will be required to find a porous pavement that will treat stormwater to a level suitable for managed aquifer recharge and subsequent withdrawal for irrigation. If such a pavement is found, it could be used to help enhance the security of water supplies. In cases where stormwater would have reached water bodies untreated, it will also reduce the impact on the environment from polluted and fast flowing stormwater.

6 RECOMMENDATIONS

With the aim of pre-treating stormwater to a level suitable for aquifer storage and recovery in the Leederville aquifer, the following are recommended:

- Washoff testing by placing concentrated pollutants directly onto the pavement surface
- Further laboratory testing on pavement rigs, incorporating the sand layer at the base but in winter time
- Investigation into the use of sawdust in this sand layer to reduce export of NO_x-N from the pervious pavement
- Phosphate retention index testing on the sand used, to determine the design treatment life for effective phosphate removal
- Testing for removal efficiency of motor oil and related hydrocarbons (e.g. polyaromatic hydrocarbons)
- Testing for metal removal efficiency and determination of design treatment life
- Testing for removal of forms of dissolved organic carbon other than glucose, closer in chemical properties to rotting leaf matter for example
- Investigation of the effect of long duration, low intensity simulated storms
- Field testing at a suitable scale and location, testing for the extent of pollutant loading onto the road surface, rainfall patterns and the quality of exfiltrate from the pervious pavement.

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APPENDIX A: TRIAL OF EXPONENTIAL HYDRAULIC MODEL

An exponential model was fitted to the hydraulic data for the run on Day 39 for Rig 1, since this was initially believed to be more appropriate, with down flow reasoned to be proportional to the volume still flowing over the aggregate at any given time. This was of the form:

$$V = V_f - \frac{Q_0}{k} e^{-k(t-t_0)} \qquad (5)$$

where:

V = total volume collected after time t

V_f = total final ouput volume

Q_0 = initial outflow rate, i.e. the instantaneous outflow rate when inflow stops

k = decay rate

t_0 =time when inflow stops

This had an R^2 of about 0.998 and standard error of 0.18 L. The calculated total final output volume was 1.54 L, compared to 2.33 L of input solution actually used. However, the total volume collected was 2.03 L with some overflow. The logarithmic model predicted the total volume output at this time of collection (74 hours after commencement) was 2.15 L, which seems more accurate. Also, the exponential model's implied apparent loss of about 0.8 L cannot be justified, as the test rig was washed heavily during construction and evaporation would have been minimal due to it being winter at the time. The sample was protected from rain. Therefore, the exponential model is rejected in favour of the logarithmic model.