Abstract

MATA, LUIS ALEXANDER. Sedimentation of Pervious Concrete Pavement Systems. (Under the direction of Michael L. Leming).

Pervious concrete pavement systems (PCPS) are a unique and effective means to address important environmental issues and support green, sustainable growth, by capturing stormwater and allowing it to infiltrate into the underlying soil. Sedimentation leading to clogging is a potential problem in serviceability of PCPS. The sedimentation rates of pervious concrete with 20% porosity were examined with three different soil types: sand, clayey silt, and clayey silty sand. Pervious concrete beam and cylinder specimens were exposed to sediments mixed in water to simulate runoff with heavy and typical load of soil sediments. Falling head permeability tests were performed in the specimens before and after exposure. Results show that storage capacity will be minimally affected by sediment. Exfiltration rate, however, can be affected by sediment characteristics in some situations. A simple, economical test for estimating exfiltration rates of the system in these situations was also developed. The results of this study were used to develop design guidelines that complement the hydrological design of PCPS considering the effects of sedimentation of the system at end of service.

The effects of realistic freezing rates on frost resistance of pervious concrete, including the effects of sedimentation were also examined. Pervious concrete disk specimens were
subjected to freezing and thawing cycles using a unique and innovative test that considers realistic, slow freezing rates of partially saturated, pervious concrete disk specimens. Results confirmed previously published reports that sand must be included in the mixture to be frost resistant when saturated or normally saturated, regardless of the addition of air entraining admixture (AEA).
Sedimentation of Pervious Concrete Pavement Systems

by
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Biography

Luis Alexander Mata was born in Caracas, Venezuela. He graduated from San Ignacio de Loyola School in 1993. He received his BS (2002) from the Department of Civil Engineering at Universidad Católica Andrés Bello, Caracas, Venezuela.

In January of 2003, he enrolled in graduate program in the Department of Civil, Construction and Environmental studying Construction Engineering and Management and started taking classes towards his Master of Science degree. He worked in the research project Implementation of Self-Consolidating Concrete (SCC) for Prestressed Concrete Girders (NCDOT Project 8.1170903), and his thesis was based on the data obtained from this study.

In January 2005, he continued his education at North Carolina State University towards his PhD degree. In 2006, he received the Portland Cement Association Education Foundation Research Fellowship to investigate the effects of sedimentation in pervious concrete pavement systems. His doctorate dissertation was based on the data obtained from this study.
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Chapter 1. Introduction

1.1. Background

Pervious Concrete Pavement Systems (PCPS) have rapidly gained popularity with designers, owners, developers, and permitting agencies. Reasons include amendments to the Clean Water Act (US EPA 1999), which require both mitigation of the quantity of stormwater runoff and maintenance of water quality. Both of these requirements can be achieved by the use of pervious concrete pavements.

A properly designed PCPS can store some or all of the rainfall in a design storm for a site and therefore reduce the quantity of stormwater runoff. PCPS also have other benefits. PCPS capture the “first flush” of the runoff (Tennis, Leming, and Akers 2004), which carry a significant portion of the pollutants on the pavement. PCPS rely on natural treatment of runoff by holding much of the runoff until it can infiltrate into the underlying soil or subgrade (Tennis, Leming, and Akers, 2004). Other benefits of pervious pavements include reducing road noise, reducing road spray, improving skid resistance and minimizing the heat island effect in large cities (Ferguson 2005, Tennis, Leming, and Akers, 2004). A significant benefit of PCPS is to capture rainfall heated by conventional pavements on building surfaces so that warm runoff is not discharged into receiving waters. The PCPS has advantages to the owner of providing stormwater detention within the parking space and eliminating an attractive nuisance (Tennis, Leming, and Akers, 2004; Leming, Malcom, and Tennis, 2007).
The hydrological performance of PCPS is usually the characteristic of most interest to agencies granting permission-to-build permits. Traditional structural Best Management Practices (BMPs) addressed by the Environmental Protection Agency (EPA) and permitting agencies such as the North Carolina Department of Environmental and Natural Resources (NCDENR) include wet detention basins, extended detention wetlands, pocket wetlands, sand filters, bioretention areas, grassed swales, extended dry detention, filter strips and infiltration devices (US EPA 1999, NCDENR 2007). Both EPA and NCDENR consider PCPS to be a structural BMP infiltration system to capture a volume of runoff and infiltrate it into the ground (US EPA 1999, NCDENR 2007).

North Carolina passed new laws that affect sustainable building requirements in 2007. These laws include water use limitations, capture and consumption requirements, and elimination of potable water for irrigation (NC Session Law 2007-546), and limitation of impervious surfaces for developed North Carolina owned or administered sites (NC Session Law 2007-323).

Although pervious concrete pavements have been used successfully in numerous geographical locations and many of the fundamental properties of pervious concrete have been established, a number of issues are still not completely resolved. Many of these issues are related to long term behavior of the pavement in service.
Procedures to establish pavement and base depths appropriate for both load carrying and hydrologic design needs have been established reasonably well at this point (Meininger 1998, Ferguson 2005, Tennis, Leming, and Akers 2004, Leming Malcom and Tennis, 2007). Several concerns exist, however, in four primary areas: (1) blockage of the pores by sediments (sedimentation), resulting in loss of storage, or reduction in permeability or exfiltration, (2) selection of design exfiltration values particularly as they may change in service, (3) frost resistance due to sedimentation, and (4) standards for the fabrication of specimens for testing, although consensus standards are currently in development.

PCPS often exposed to freezing and thawing cycles and must be frost resistant. Current research reports that the addition of 7% sand by weight as replacement for coarse aggregate increases the freeze-thaw resistance (Kevern, Schaefer, Wang and Suleiman, 2008). Most of this research, however, focuses on rapid freezing and thawing tests that does not reflect realistic situations to which a PCPS is subjected in the field, and do not include the effects of sedimentation.

Sediment can fill the voids in the pervious concrete or the stone base and form layers on the surface or along the bottom of the PCPS. Sedimentation can therefore reduce permeability and the ability of the PCPS to store runoff. The quantitative effects of loss of storage have not been rigorously assessed and guidelines such as those proposed in the Portland Cement Association (PCA) Engineering Bulletin (EB) 303 (Leming, Malcom, and Tennis, 2007) are only general.
The most critical aspect of sedimentation may be the formation of a layer of fine-entrained material on the bottom of the PCPS. This layer could significantly compromise hydrological behavior of the PCPS by reducing the effective exfiltration rate of the system over time. A significant reduction in the ability to drain captured runoff could result in either additional runoff or excessive recovery time, that is, the time required to return to full capacity, or both. This possibility was noted in PCA EB 303 (Leming, Malcom and Tennis, 2007) but no experimental data or published guidelines exist to provide the Designer-of-Record with a rigorous methodology or appropriate design values.

1.2. Problem Statements

Additional study is urgently needed to quantitatively determine the effects of sedimentation on permeability, surface capacity, and hydrological behavior of PCPS, including end of service (EOS) performance, so that both design professionals and permit-granting agencies can rationally develop or evaluate likely PCPS performance in site specific applications. A key element of such a study is the development of a rational design methodology based on recognized and well established engineering principles. Another important element is to assess the effects of sedimentation on frost resistance of PCPS.
1.3. Objectives

This study is concerned primarily with sedimentation issues, including the related problem of establishing appropriate design exfiltration values for the life of the pavement structure. This study has two primary objectives:

1. Identify and analyze the effects of sediment deposition and segregation, and the effects of sediment transport within the PCPS to develop appropriate design guidelines for storage capacity analysis including the effect of changes in exfiltration rate. This analysis can also be useful in selecting appropriate maintenance strategies.

2. Examine the effects of realistic freezing rates on frost resistance of pervious concrete, including the effects of sedimentation.

1.4. Methodology

This study was divided into three phases. In Phases I and II, the sedimentation rates of three (3) different pervious concrete were examined with three different soil types: sand, clayey silt, and clayey silty sand. In Phase I, two pervious concrete mixtures were subjected to a low flow of water with a high sediment load, a relatively worst case scenario in terms of load. In Phase II, one concrete mixture was subjected to an initial high flow of water which was reduced to a lower flow with sediment loads that were estimated based on the service life of the pavement. The pervious concrete was also subjected to repetitive sediment loads after washing. In Phase III, frost resistance of pervious concrete was evaluated using a test that incorporates realistic, slow freezing rates of partially saturated, pervious concrete disk specimens. Pervious concrete mixtures were examined to confirm published data on the
effects of sand and air entrained admixtures on frost resistance, and, in a very limited fashion, examine the effects of a likely sediment type and load of frost resistance of pervious concrete.
Chapter 2. Literature Review

2.1 Pervious Concrete Pavement systems (PCPS)

Pervious Concrete Pavement Systems (PCPS) have been used in some areas for decades, however, recent interest in sustainable development and recognition of PCPS as a best management practice (BMP) for stormwater management has increased the interest and use in the U.S. as described in Portland Cement Association (PCA) Engineering Bulletin (EB) 303 (Leming, Malcom, and Tennis, 2007). PCPS support sustainable development initiatives such as EPA Heat Island Reduction Initiative (USEPA 2008a) and Low Impact Development (USEPA 2008b) and provides several credits in the LEED (Leadership in Energy and Environmental Design) rating system for sustainable building construction (US Green Building Council 2005).

PCPS can store some or all of the rainfall in a site and therefore, reduce the quantity of stormwater runoff (Bean, Hunt and Bidelspach, 2007a). A PCPS can capture the “first flush” of runoff (Tennis, Leming, and Akers 2004), in which a significant portion of the pollutants on the pavement is present. A PCPS can treat the water naturally by promoting infiltration through the underlying subsoil (Tennis, Leming, and Akers, 2004). Other benefits of pervious pavements include reducing road noise, reducing road spray, improving skid resistance and minimizing the heat island effect in large cities (Ferguson 2005, Tennis, Leming, and Akers, 2004).
2.2 System Definition

Pervious concrete is typically described as open-graded material consisting of portland cement, coarse aggregate, little or no fine aggregate, admixtures and water. The combination of these components produce a hardened material with interconnected pores, ranging in size from 0.08 to 0.32 in. (2 to 8 mm), that allow water to pass through easily (ACI 522, 2006).

A PCPS used as an active mitigation structure is usually placed with the pervious concrete on top of a clean stone aggregate base with a filter fabric often placed beneath the base, depending on subgrade, that is, the underlying soil (Tennis, Leming, and Akers, 2004; Leming, Malcom and Tennis, 2007). The depth of pervious concrete and aggregate base will be determined by the hydrological and structural requirement of the system. Filter fabric is placed between the base and the underlying soil when the PCPS is placed over fine grained soils to prevent the fines from migrating into the base under load. Aggregate base and filter fabric are optional components of the system and the use of each will be determined by the soil subgrade and intended application (Figure 2.1) [Tennis, Leming, and Akers 2004; Leming, Malcom and Tennis, 2007].
The exfiltration rate can be defined as the rate at which runoff leaves the PCPS and enters the underlying soil. Once stormwater runoff is captured by the system, runoff will be stored until it exfiltrates the system (Figure 2.2). The time it takes for the stormwater to exfiltrate the system will depend on the infiltration rate of the underlying soil (Tennis, Leming, and Akers 2004; Leming, Malcom and Tennis, 2007). Soil subgrade controls the exfiltration rate of the system and it is also the controlling factor in the hydrological design of PCPS. EB 303 (Leming, Malcom and Tennis, 2007) recommend that the PCPS be designed to drain fully in no more than five (5) days. The exfiltration rate is assumed to be constant, and given the variability in actual values it can be used in simple flow analysis for short such as a few days periods of time (EB 303 [Leming, Malcom and Tennis, 2007]).
Figure 2.2. PCPS Exfiltration Mechanism
Table 2.1. Recommended Infiltration Rates for Preliminary Design of PCPS

<table>
<thead>
<tr>
<th>General Soil Type</th>
<th>Infiltration rate, in./h (cm/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy soils</td>
<td>0.5 to 1 (1.3 to 2.5)</td>
</tr>
<tr>
<td>Silty soils</td>
<td>0.1 (0.3)</td>
</tr>
<tr>
<td>Clayey soils</td>
<td>0.01 (0.03)</td>
</tr>
</tbody>
</table>

2.3. Pervious Concrete Materials

Pervious concrete uses the same materials as conventional concrete, with the exceptions that the fine aggregate is nearly or entirely eliminated, and the coarse aggregate is typically more uniform (ACI 522, 2006; Tennis, Leming, and Akers 2004). Mixture proportions for pervious concrete are usually less forgiving than conventional concrete mixtures and tight controls are usually required in order to provide the required characteristics (Tennis, Leming, and Akers 2004). Mixture proportions will vary depending on local concrete materials. Trial batching and experience is required to develop acceptable mixtures, however, typical ranges of materials proportions are shown in Table 2.2.

Table 2.2 Typical Ranges of Materials Proportions in Pervious Concrete

<table>
<thead>
<tr>
<th>Materials</th>
<th>Proportions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cementitious materials, lb/yd&lt;sup&gt;3&lt;/sup&gt; (kg/m&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>450 to 700 (270 to 415)</td>
</tr>
<tr>
<td>Aggregate</td>
<td>2,000 to 2,500 (1,190 to 1,480)</td>
</tr>
<tr>
<td>Water: cement ratio (by mass)</td>
<td>0.27 to 0.34</td>
</tr>
<tr>
<td>Aggregate: cement ratio (by mass)</td>
<td>4 to 4.5:1</td>
</tr>
<tr>
<td>Fine: coarse aggregate ratio (by mass)</td>
<td>0 to 1:1</td>
</tr>
</tbody>
</table>
The aggregate-cement factor, and water cementitious materials ratio (w/c) are the two most important factors in the mixture that affect the mechanical characteristics (ACI 522, 2006). Water to cementitious materials ratios (w/c) between 0.30 and 0.40 have been used successfully in pervious concrete mixtures (ACI 522, 2006; Tennis, Leming, and Akers 2004). Chemical admixtures are usually used to influence workability and setting time of mixtures (Tennis, Leming, and Akers 2004; ACI 522, 2006).

Cementitious Materials

Portland cement conforming with ASTM C 150 is used as the main binder (ACI 522, 2006). Blended cements conforming with ASTM C 595 and C 1157 may also be used in pervious concrete (Tennis, Leming, and Akers 2004). Supplementary cementitious materials such as fly ash, slag cement, and silica fume conforming with ASTM C 618, ASTM C 989, and C 1240 respectively, can also be used. (ACI 522, 2006; Tennis, Leming, and Akers, 2004).

Aggregate

Fine aggregate content is nearly or entirely eliminated in pervious concrete mixtures. Aggregate gradings used in pervious concrete are typically either single-sized coarse aggregate or grading between 3/4 and 3/8 in. (19.0 and 9.5 mm) [ACI 522, 2006]. Commonly used gradations of coarse aggregate include ASTM C33 # 67 (3/4 in. to # 4), # 8 (3/8 in. to # 16), or # 89 (3/8 in. to # 50) sieves [in metric units: # 67 (19.0 to 4.75 mm), # 8 (9.5 to 2.36 mm), or # 89 (9.5 to 1.18 mm), respectively] (Tennis, Leming, and Akers 2004).
**Admixtures**

Chemical admixtures are used in pervious concrete for the same reasons as in conventional concrete. Due to the rapid setting time in pervious concrete mixtures, retarders or hydration-stabilizing admixtures are commonly used (Tennis, Leming, and Akers 2004). High range water reducers admixtures or medium range water reducers are commonly used depending on the w/cm (ACI 522, 2006). These admixtures should meet the requirements of ASTM C 494. Air-entraining admixtures in accordance with ASTM C 260 can be used in environments susceptible to freezing and thawing (Tennis, Leming, and Akers 2004).

**2.4. Fresh and Hardened Properties of Pervious Concrete**

Fresh pervious concrete is typically stiff, with low workability compared to conventional concrete. Slump values are usually less than 3/4 in. (20 mm) [ACI 522, 2006, Tennis, Leming, and Akers 2004]. The slump is rarely a useful or appropriate method to determine mixture consistency. ACI 522 (2008) recommends fresh density as the preferred measurement for quality control or quality assurance purposes.

Working time for fresh pervious concrete is usually shorter than for conventional concrete. Current recommended specifications (ACI 522, 2008) state that the discharge of the fresh pervious concrete shall be completed within 60 minutes of the introduction of mixture water to the cement, and within 90 minutes when using an extended set control admixture, such as a retarder or hydration stabilizer, depending on the temperature.
Due to unique characteristics, current standards in fabricating specimens and testing materials for conventional concrete do not apply for pervious concrete. ASTM standards for the fabrication of pervious concrete specimens, compressive and flexural strength are currently in development. ASTM Committee C09.49, focused on pervious concrete, is currently working on the development on consolidation techniques and also in the evaluation of tests methods to be incorporated into ASTM standards (ASTM 2007).

2.4.1. Density and Porosity

The density and porosity of pervious concrete are a function of the mixture proportions and the consolidation of the mixture. In-place densities of 100 lb/ft$^2$ to 125 lb/ft$^2$ (1,600 kg/m$^2$ to 2,000 kg/m$^2$) are common (Tennis, Leming, and Akers 2004).

Porosity affects compressive and flexural strength, and is the property that controls the storage capacity of pervious concrete. Low porosity is considered to be less than 15% voids, while high porosity estimated to be around 30% voids. A reasonable average value for preliminary structural and hydrological design is 20% (Tennis, Leming, and Akers 2004).

Different methods are being considered by ASTM to consolidate pervious concrete to determine density and void content, including (1) Marshal hammer (10 lbs, 18 in. drop, 3.875 in. tamper foot) at 5 blows per lift for 2 equal lifts; (2) Marshal hammer at 10 blows per lift for 2 equal lifts; (3) Proctor hammer (5.5 lbs, 12 in. Drop, 2 in. tamper foot) at 20 blows per lift for 2 equal lifts, and modifications for the Proctor hammer method (Obla, 2008). Other
methods of consolidating include roller-compacted concrete in cylinder molds as described in ASTM C 1176 and ASTM C 1435 (Obla, 2008).

ACI specifications for pervious concrete (ACI 522, 2008) call for the use of ASTM C138 following the jigging consolidation procedure described in ASTM C 29 to determine the density of fresh concrete. The density of the concrete, however, is likely to be higher in service.

The effective porosity can be obtained using ASTM C 140, Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units, or ASTM D 7063, Standard Test Method for Effective Porosity and Effective Air Voids of Compacted Bituminous Paving Mixture Samples. Using ASTM D 7063. Concrete specimens are dried to a constant mass at a temperature that does not exceed 40 °C (104 °F). The specimens are then placed into a plastic bag, the air is evacuated from the sample and the bag sealed. The mass of the sample and bag submerged in water is determined. The plastic bag is then cut, allowing the water to saturate the sample, and the mass of the saturated sample is determined. The effective porosity is then calculated as described in ASTM D 7063.

Porosity of pervious concrete specimens can also be determined using the difference in weight between the dry sample and the weight of the immersed sample, similar to the method described in ASTM C 140 as shown in equation 2.1
\[ V_r = \left(1 - \frac{W_2 - W_1}{\rho_w \cdot Vol}\right) \times 100 \]  \hspace{1cm} (2.1)

where

- \( V_r \) = total void ratio, \( \% \),
- \( W_1 \) = weight immersed (lbs or kg),
- \( W_2 \) = dry weight (lbs or kg),
- \( Vol \) = nominal sample volume based on dimensions of the sample (ft\(^3\) or m\(^3\)),

and

- \( \rho_w \) = density of water (lbs/ft\(^3\) or kg/m\(^3\)).

ASTM D 7063 uses the difference of the density of the specimen, obtained by a water displaced method, with an apparent maximum density obtained by the saturated weight of the specimen to determine the fraction of total number of voids that are accessible to water. This effective porosity can be used as storage capacity in the PCPS and can be considered as the preferred method to evaluate the hydrological performance of the system.

### 2.4.2. Permeability

Permeability or percolation rate, or hydraulic conductivity is one of the most important properties of pervious concrete and can be defined as the flow rate of water through the pervious concrete. Typical flow water through pervious concrete are from 290 in./hr (740 cm/hr) to 770 in./hr (1,960 cm/hr) and values higher than 1650 in./hr (4,190 cm/hr) have been measured in the laboratory (Tennis, Leming, and Akers 2004). Bean, Hunt and
Bidelspach (2007a) reported field permeability rates of pervious concrete pavements from 240 in./hr (600 cm/hr) to 2,760 in./hr (7,000 cm/hr).

Permeability of pervious concrete can be measured in the laboratory by several means. A falling head permeameter is commonly used for convenience. The cylindrical sample is prepared such that no water flows along the side of the specimens and water flows from top to bottom of the specimen. Water is added to the graduated cylinder to fill the specimen and the drain pipe, up to the initial head level, with the valve closed. The valve is then opened and the time required for water to fall from an initial head to a final head is measured (Figure 2.3) [Neithalath, Weiss and Olek, 2003; Kevern, Schaefer, Wang and Suleiman, 2008]. The average coefficient of permeability \( k \) is determined using equation 2.2, based on Darcy’s law principle of flow in homogeneous porous material.

\[
k = \frac{aL}{At} \ln \left( \frac{h_1}{h_2} \right)
\]  

(2.2)

where

\( k \) = coefficient of permeability, in./s or cm/s,

\( a \) = cross-sectional area of the pipe, in\(^2\) or cm\(^2\),

\( L \) = Length of sample, in. or cm,

\( A \) = cross-sectional area of the sample, in\(^2\) or cm\(^2\),

\( t \) = time for water to drop from \( h_1 \) to \( h_2 \), s,

\( h_1 \) = initial water level, in or cm, and

\( h_2 \) = final water level, in or cm.
On-site permeability can be measured using a single or double ring field infiltration rate test (Mata and Leming, 2008; Bean, Hunt and Bidelspach, 2007a). When using the single ring, a single ring of a known volume is attached to the surface of the pavement. A short pre-wetting period is followed testing, consisting of adding a known volume of water to the ring and measure the time for the water to fall between two elevations. The test is affected by lateral infiltration but is considered as a good test to evaluate surface permeability (Obla 2008).
2.4.3. Frost Durability

Frost durability of concrete is typically evaluated using two standard methods, which assess durability for two different phenomena, rapid freezing and thawing resistance to deicer salt scaling. ASTM C 666 involves rapid freezing and thawing cycles in the presence of water. Frost durability determined using ASTM C 666 Resistance of Concrete to Rapid Freezing and Thawing primarily measures the effectiveness of the air void system for concrete with frost durability aggregate. ASTM C 666 may not be the recommended method to evaluate resistance to freezing and thawing since it does not reflect the performance of the pavement in the field. The deicer salt test described in ASTM C 672 may be better suited for evaluation of durability of concrete in paving applications because it involves slower, more realistic freezing cycles in the presence of a deicing salt solution and attempts to more realistically simulate frost exposure conditions in a pavement (Dilek and Leming, 2007).

Pervious concrete pavements exposed to freezing and thawing cycles in cold climates must be frost resistant. The paste must contain adequate entrained air and the effects of saturation of the large pores must be considered. The expansion of the water in the large pores may affect frost durability significantly. The open structure of pervious concrete allows free ingress of water into the specimen. Subsequent freezing of saturated PCPS could cause rapid deterioration. Although freeze-thaw damage has been reported to develop in pervious concrete primarily as paste deterioration of lower porosity pervious concrete (Yang and Jiang 2002), Schaefer, Wang, Suleiman and Kevern (2006) reported failure due to splitting and
deterioration of aggregate, as well as paste deterioration. They also reported that deterioration of the paste appeared to depend on the aggregate type and size.

Korhonen and Bayer (1989) reported that samples of pervious concrete without air entrainment showed some resistance to freezing and thawing in a damp condition but not in a ponded condition. Pervious concrete beams with dimensions of 3 in. by 3 in. by 15 in. (76 mm by 76 mm by 381 mm) and cylinders with dimensions of 4 in. by 8 in. (102 mm by 203 mm) subjected to freezing in air and thawing in air, or freezing in air and thawing in water, or freezing in water and thawing sustained over 160 freeze-thaw cycles when damp or in air, but failed within 45 freeze-thaw cycles when submerged in water.

Frost resistance was also evaluated by Neithalath, Weiss and Olek (2003). Pervious concrete specimens were subjected to freezing and thawing resistance using either rapid freezing and thawing in water, in accordance with ASTM C 666 at 5 to 6 cycles per day, or slow freezing and thawing in water, in a controlled temperature chamber where the specimens were subjected to one cycle per day. Specimens without air entraining admixture (AEA) subjected to ASTM C 666 exposures had 50% to 70% loss of dynamic modulus after 65 cycles. Specimens with AEA suffered a dynamic modulus loss of 20% to 60% after 70 cycles. Specimens subjected to the alternative method experienced a dynamic modulus loss of 5% to 7% after 80 cycles.
Schaefer, Wang, Suleiman and Kevern (2006) reported that pervious concrete mixtures containing sand showed better freezing and thawing resistance that those without sand. Tests were performed in accordance with ASTM C 666. The same study showed that a mixture containing sand and an air entraining admixture suffered only 2% mass loss after 300 freeze-thaw cycles.

Kevern, Schaefer, Wang and Suleiman (2008) reported that the addition of 7% sand by weight as replacement for coarse aggregate increases the freeze-thaw resistance of pervious concrete tested in accordance with ASTM C 666. Mixtures with sand and air entraining admixture were also found to have better freeze-thaw resistance than those with sand only. The addition of class C fly ash to mixtures with sand, however, decreased freeze-thaw durability (Kevern, Wang and Schaefer 2008).

Sedimentation and frost resistance may be directly related in wet cold climates. If clogging of either the pervious concrete on PCPS occurs, water may accumulate in the pavement and freeze. Field observations of PCPS located in cold weather climates, with an average of 5 years of service, have not shown signs of freezing-thaw damage (Delatte, Miller and Mrkajic 2007). It appears that the technology to protect pervious concrete itself from the effects of freezing and deicing salts already exists (Kevern, Wang and Schaefer 2008; ACI 522, 2006); but the combined effects of clogging on potential saturation, reduced infiltration of the subsoil, changes in the depth of freezing and seasonal differences in storm intensity, including snow melt have not been fully resolved.
2.4.4. Flexural Strength and Dynamic Modulus of Elasticity

Flexural strength and dynamic modulus of elasticity are important for the structural behavior of pavements. Flexural strength of pervious concrete generally ranges between 150 psi (1.0 MPa) and 550 psi (3.8 MPa) [Tennis, Leming, and Akers 2004]. Flexural strength is influenced by many factors, especially the degree of compaction and porosity. A relationship between compressive strength and flexural strength can be established (ACI 522, 2006), however the lack of standards in specimen fabrication and testing limit wide acceptance. Flexural strength in pervious concrete can be determined in accordance with ASTM C 78. Dynamic modulus of elasticity can be measured non-destructible using beam specimens as described in ASTM C 215. Beam specimens used for both tests permit compaction comparable to the compaction applied in the field.

2.4.4. Compressive Strength

Compressive strength in pervious concrete is in general lower than conventional concrete due to the high porosity. Compressive strengths are in the range of 500 psi to 4000 psi (3.5 MPa to 28 MPa), with typical values of 2500 psi (17 MPa) [Tennis, Leming, and Akers 2004]. Since no standards for compaction in the fabrication of specimens have been developed yet, drilled cores can be considered as the most reliable measurement of in-placed concrete (ACI 522, 2006).
Compressive strength is typically used for quality control purposes simply for convenience, however, it has little practical use. Compressive strength will most likely be determined using specimens compacted with a standard Proctor hammer (Obla, 2008). Review of methodologies are under way by ASTM in order to determine the specimen size and specimen preparation for testing, including capping requirements.

2.5 Construction and Consolidation Effects of PCPS

As with conventional pavement, a uniform subgrade at the correct elevation is essential in the construction of the PCPS. The subgrade should be moistened before placing the concrete in order to prevent water from being removed from the lower part of the concrete too soon (Tennis, Leming, and Akers 2004). For consistent subgrade support, it is recommended that a minimum compaction of 90% to 95% of theoretical density be attained in accordance with AASHTO T 180. Increasing the subgrade density decreases the permeability of the soil (Tennis, Leming, and Akers 2004), however, and some concern has been expressed about compaction needs in areas with light traffic (Ferguson, 2005).

Placement of pervious concrete is commonly done continuously, spreading and striking-off immediately, commonly with vibrating screeds (Tennis, Leming, and Akers 2004). The pervious concrete is placed in forms to which a temporary wood spacing strip has been placed on top of the form. The strip is removed after strike off and before compaction (Paine, 1992). Consolidation is usually performed by rolling the concrete surface with a steel roller,
compacting the concrete to the height of the forms after removing the wood spacing strip (Tennis, Leming, and Akers 2004). The compaction of the concrete is recommended to be performed with enough weight to provide a minimum of 10 psi vertical stress. Other consolidation methods include either slip forms or a revolving steel cylinder that combines strike-off and compaction.

2.6 Compaction Gradients in Pervious Concrete

The pervious concrete in the system is denser on the surface due to compaction techniques used to place the concrete. Haselbach and Freeman (2006) report that porosity increased significantly from top to bottom in pervious concrete slabs 6 in. (15 cm) in height and placed with a surface compaction provided by a static roller after removal of the spacer strip.

The pressure distribution with depth due to an applied load at the surface for compaction can be estimated using the Boussinesq equation (Bowles, 1996). The Boussinesq solution for vertical stresses beneath an element depends on the pressure applied to the surface and the area of contact between the element and the surface (Figure 2.4). A steel roller compactor with an outer diameter of 10 in. (254 mm), commonly used for pervious concrete compaction, has a contact area with the surface of the pavement about 1.6 in. (40 mm) times the longitudinal length of the roller, and produces a pressure of approximately 10 psi (69 kPa) at the surface. This pressure on the surface creates a pressure about 5, 2, 1, 0.7, 0.3, and 0.002 psi (34.5, 13.8, 6.9 4.8, 2.1, and 0.01 kPa) at depths of 2, 4, 6, 8, and 10 in. (5, 10, 15
20, and 25 cm), respectively. This pressure gradient is generally consistent with the density gradient reported by Haselbach and Freeman (2006). This implies that since pervious concrete will have a much denser surface larger sediment particles will tend to be trapped in the surface or near surface region of a PCPS. This in turn implies that surface washing and vacuum sweeping may significantly reduce clogging of PCPS, which is consistent with the findings reported by Haselbach and Valavala (2006).

Boussinesq analysis also suggests that the porosity and tensile strength of the bottom of a pervious concrete may be relatively low once the depth exceeds about 6 in. (15 cm), and both very low and essentially constant at a depth of about 8 in. (20 cm) or greater. Boussinesq analysis further suggests that a practical maximum depth for pervious concrete is 6 in. (15 cm) for pavements exposed to significant traffic. Additional study on the effect of these porosity and strength distributions on PCPS design methods is recommended.
Figure 2.4. Pressure Isobars Based on the Boussinesq Equation for Square and Long Footings.
2.7 Hydrological Design of the PCPS

The most important characteristic of the PCPS is the capacity to capture and store all or some significant part of anticipated runoff in a given design storm. A PCPS can be designed as a “passive” or an “active” mitigation system. Passive mitigation is used only to reduce the quantity impervious surfaces by replacing it with pervious surfaces. Active mitigation is designed to capture not only the rain falling directly on the pavement, but also part, or all, of the stormwater runoff originated from adjacent zones (EB 3030 [Leming, Malcom and Tennis, 2007]). Active mitigation systems may be more susceptible to sedimentation due to the stormwater runoff from adjacent vegetated or bare areas.

Two critical factors in the hydrological design of PCPS are the amount of runoff anticipated from the site and the infiltration rate of the soil. These two characteristics are related since runoff will depend in part on the type of soil. Areas with clayey soils will tend to produce more runoff due to low infiltration; sandy soils will likely produce less runoff because of relatively high infiltration and less soil erodibility (Wurbs and James, 2000; ASCE, 2006). Recommended infiltration rates from EB 303 (Leming, Malcom and Tennis 2007) for preliminary design are shown in Table 2.1.

Permeability of the pervious concrete is not, in general, a limiting factor in the design of the system (EB 303 [Leming, Malcom and Tennis, 2007]). Permeability of pervious concrete and aggregate base is usually higher by several orders of magnitude than the infiltration of the
underlying soil. The exfiltration rate of captured runoff from the PCPS into the underlying soil is controlled by the soil infiltration (EB 303 [Leming, Malcom and Tennis 2007]).

Storage capacity in the PCPS is determined by the depths and effective porosities of the pervious concrete and the aggregate base. Pervious concrete typically has a porosity from 15% to 30%, and an aggregate base produced from a # 57 or # 67 stone usually has a design porosity of 40% (EB 303 [Leming, Malcom and Tennis 2007]). Runoff stored in the system will drain through the underlying soil to restore some part of the capacity during the storm, and to remove the water captured by the system once the storm has passed (EB 303 [Leming, Malcom and Tennis 2007]).

Two hydrological design methods are usually considered for PCPS: 1) the NRCS (SCS) Method, or “Curve Number,” and 2) the Rational Method. The main difference in these two methods is that the NRCS (SCS) method estimate runoff volume while the Rational Method estimates the peak runoff flow. Since a PCPS is used to capture runoff and hold it for infiltration, the NRCS (SCS) method is considered the more appropriate alternative (EB 303 [Leming, Malcom and Tennis 2007]). The Portland Cement Association (PCA 2007) offers software originally developed by Malcom and Leming, based on the NRCS (SCS) Method outlined in Technical Release 55 (TR-55) [SCS 1986] to analyze the hydrological behavior of a PCPS (PCA 2007).
2.7.1. The NRCS Curve Number Method

The Curve Number method includes the use of coefficients (the “Curve Numbers”) to estimate runoff based on types of soil and cover conditions. Adjustments to coefficients, based on the amount of impervious surface and how much of that impervious area is connected, are also possible. This method utilizes a 24-h design storm. This design approach considers the significant effects of infiltration and long term storage capacity of the PCPS, and it also incorporates the effects of both impervious surfaces and other surfaces (EB 303 [Leming, Malcom and Tennis 2007]) on PCPS behavior.

In general, the Curve Number method consists of mathematically applying the hourly distribution of rainfall for the design storm to the various surfaces of the site that discharge onto the pervious concrete pavement system. For an active mitigation system, this can include impervious surfaces such as building footprint, paved islands, and bus or truck lanes, and surfaces with natural cover such as planted traffic islands, vegetated areas on site, and adjacent properties that drain naturally onto the site under design. For a passive mitigation system, this would typically include only the surface of the pervious concrete pavement, but may also include border features associated with the pavements, such as curbs or impervious decorative borders (EB 303 [Leming, Malcom and Tennis 2007])

The volume of rain for each hourly increment of the design storm falling on the pervious concrete and the impervious surfaces, and the excess surface runoff from adjacent areas less the volume infiltrated into the soil, is stored in the pervious concrete pavement system.
Overland flow occurs very rapidly for small sites, so no adjustment is made for travel times for contiguous areas. This is both computationally convenient and conservative. This process continues until the rainfall of all of the 24 hourly increments has been applied or until the storage capacity of the system has been exceeded, in which case the remaining rainfall is considered to be excess surface runoff (EB 303 [Leming, Malcom and Tennis 2007]).

Infiltration maintains some fraction of the effective storage capacity of the pervious concrete pavement system by removing some of the captured runoff over time. The effect of infiltration on storage capacity, and therefore excess surface runoff, is a critical element in the analysis. Infiltration continues until the pervious concrete system is emptied and the storage capacity returned to its original value. The total recovery or drawdown time, the time until 100% of the storage capacity has been recovered, is also an important performance factor and should not exceed 5 days (EB 303 [Leming, Malcom and Tennis 2007]).

2.8 Structural Design of PCPS

PCPS can be designed using either standard pavement design procedures such as AASHTO, ACI 325.12R for streets and roads, or ACI 330R for parking lots, or using structural numbers from a flexible pavement design procedure (ACI 522, 2006; Tennis, Leming, and Akers 2004). The practical range of design thickness is from 4 to 10 in. (125 to 250 mm) for plain pavements (ACI 522, 2006). Table 2.3 shows pavement depths for type of traffic based on a fundamental approach using the modified Westergaard Analysis for flexural stress.
### Table 2.3. Pervious Concrete Pavement Depths for Type of Traffic

<table>
<thead>
<tr>
<th>Depth of pavement, in. (cm)</th>
<th>Traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 (10)</td>
<td>Cars and light trucks</td>
</tr>
<tr>
<td>6 (15)</td>
<td>No heavy trucks, residential driveways</td>
</tr>
<tr>
<td>8 (20)</td>
<td>Acceptable for low truck volumes</td>
</tr>
</tbody>
</table>

#### 2.9 Sedimentation Effects of PCPS

Clogging has been listed as one of the primary methods of failure of pervious concrete pavements (EPA, 1999), and as the primary concern that has prevented wide acceptance of pervious concrete (ACI 522, 2006). Wingerter and Paine (1989) reported that pervious concrete pavements showed small amounts of clogging after approximately 13 years of service. Successful maintenance and cleaning by sweeping and pressure washing have been reported by the Florida Concrete and Products Association (no date). These studies, however, were based in Florida where the soils are composed primarily of sand. Bean, Hunt and Bidelspach (2007a) reported that PCPS subjected to a potential source of fines showed significant effects in permeability when compared with PCPS with no fines exposure. Published studies examining sedimentation potential on pervious concrete pavements in areas with silty or clayey soils, however, are limited. Effects of (1) sediment deposition and segregation, (2) sediment transport within the pervious concrete, (3) methods to estimate sediment load and, (4) deposition and transport effects on design methodologies have not been fully established.
The storage capacity of a pervious concrete or aggregate base is the effective porosity, that is the available void content per unit volume, times the volume of the pervious concrete or aggregate base (EB 303 [Leming, Malcom and Tennis 2007]). Significant changes in porosity of either the pervious concrete or the aggregate base can affect the hydrological behavior of PCPS in three ways: (1) clogging can reduce storage capacity, (2) clogging can reduce surface permeability of the system and (3) clogging can affect the infiltration rate into the subgrade, that is, exfiltration from the PCPS to the underlying soil.

Leming, Malcom and Tennis (EB 303, 2007) report that fine grained sediments deposited in the pavement will most likely occupy less than 1/2 in. (12 mm) of the depth of the aggregate base in 20 years of service, resulting in only a few percent loss in storage capacity assuming typical but conservative deposition rates, but the analysis was based on simple assumptions not confirmed with experimental data. An extra inch (25 mm) of aggregate base is most likely more than adequate to supply sufficient storage capacity for those cases in which sedimentation of fine grained particles are estimated to be high (EB 303 [Leming, Malcom and Tennis 2007]); the marginal cost of the 1 in. (25 mm) of base is minimal. For sites with depositions estimated to be 1,000 lbs/ac/yr (1,125 kg/ha/yr) or higher, and additional inch (25 mm) of aggregate base was suggested as a standard design feature.

The most significant effect on hydrological behavior of the PCPS is likely to be the introduction of another element, a layer of material (sediment) which could affect the “exfiltration” of stormwater runoff from the PCPS into the underlying soil. Sediment
accumulation at the base of the PCPS could reduce the exfiltration of the system if the layer of sediment is fundamentally different from the underlying soil.

2.9.1. Volume of Sediments

2.9.1.1. General Estimation of Sediment Production

The specific effects of sedimentation on the PCPS will depend on the volume of sediment. Precise estimation of the quantity of sediment is very difficult, if not impossible, due to the numerous factors involved in the process such as climate, soil properties, soil surface conditions, topography, and human activities (ASCE, 2006; Wurbs and James, 2001). Modeling erosion and sedimentation processes is highly approximate and necessary empirical, being based mainly on field observation rather than on theoretical considerations (Wurbs and James, 2001).

The concentration of pollutants found in urban runoff is directly related to the land use (US EPA, 1999). Solids in urban stormwater originate from many sources including the erosion of pervious surfaces, dust, litter and other particles; and are one of the most common contaminants in urban storm water (US EPA, 1999). General estimates of the quantities of Total Suspended Solids (TSS) is shown in Table 2.4 (US EPA, 1999). Commercial sites are estimated to produce about 1,000 lbs/acre/yr of total suspended solids (TSS) (1,125 kg/ha/yr), while construction sites can produce the highest TSS loading, up to about 6,000 lbs/acre/yr (6,750 kg/ha/yr).
Table 2.4. Typical Pollutants Loadings from Runoff by Urban Land Use.

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Total Suspended Solids (TSS), lbs/acre-yr (Kg/ha-yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commercial</td>
<td>1,000 (1,125)</td>
</tr>
<tr>
<td>Parking Lot</td>
<td>400 (450)</td>
</tr>
<tr>
<td>High Density Residential</td>
<td>420 (473)</td>
</tr>
<tr>
<td>Medium Density Residential</td>
<td>190 (214)</td>
</tr>
<tr>
<td>Low Density Residential</td>
<td>10 (11)</td>
</tr>
<tr>
<td>Industrial</td>
<td>860 (968)</td>
</tr>
<tr>
<td>Construction</td>
<td>6,000 (6,750)</td>
</tr>
</tbody>
</table>

2.9.1.2. Universal Soil Loss Equation (USLE)

The Universal Soil Loss Equation (USLE) is a commonly used technique that provides a more site specific estimate of erosion and sedimentation quantities. The USLE is used to predict the most likely average annual soil loss in specific situations, and has been used for more than 40 years, although mostly for agricultural practices (Renard, Foster, Weesies, McCool, and Yoder, 1997; Wurbs and James, 2001). Advantages of the USLE includes:

1. The use of a single number to represent each factor;
2. Estimation is based on hydrological, soil, or erosion research data for each location, and
3. It is free from any geographically oriented base.

A revised version of the USLE (RUSLE) that provides expanded information for estimating parameter values has also been developed, however, the original USLE is sufficiently adequate and simple to estimate soil loss due to erosion (Wurbs and James, 2001).
The USLE is given as:

\[ E = A R K L S C P \]  \hspace{1cm} (2.3)

where

- \( E \) = annual soil loss in tons/yr,
- \( A \) = area in acres
- \( R \) = rainfall-runoff erosivity in (ft·tons·in.)/(ac·hr·yr),
- \( K \) = soil erodibility in tons/ac,
- \( L \) = flow length (dimensionless when used as \( LS \) factor),
- \( S \) = slope (dimensionless when used as \( LS \) factor),
- \( C \) = surface cover (dimensionless), and
- \( P \) = erosion control practice (dimensionless).

The value of \( R \) reflects the climate factor in the USLE and will depend on the selected region. General values for the rainfall and runoff erosivity index \( R \) for the continental U.S. in (ft·tons·in.)/(ac·hr·yr) are provided in Figure 2.5 (Wurbs and James, 2001).

Soil characteristics are represented by the erodibility factor \( K \), which is the average soil loss. Various alternative methods for estimating \( K \) have been developed; however, Wurbs and James (2001), based on data by Walesh (1989), provide general \( K \) values in tons/acre based on soil types (Table 2.5).
Figure 2.5 The Rainfall and Runoff Erosivity Index R in Hundreds of (ft⋅tons⋅in)/(ac⋅hr⋅yr)

Table 2.5. USLE K Factor Based on Soil Types

<table>
<thead>
<tr>
<th>Soil</th>
<th>Soil erodibility (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand and gravel</td>
<td>0.1</td>
</tr>
<tr>
<td>Loamy coarse sand, sand and fine sand</td>
<td>0.15</td>
</tr>
<tr>
<td>Loamy fine sand and loamy sand</td>
<td>0.17</td>
</tr>
<tr>
<td>Fine sandy loam and sandy loam</td>
<td>0.22</td>
</tr>
<tr>
<td>Loam, clay loam and sandy clay loam</td>
<td>0.3</td>
</tr>
<tr>
<td>Silt-loam and silty clay loam</td>
<td>0.34</td>
</tr>
<tr>
<td>subsoil</td>
<td>0.43</td>
</tr>
<tr>
<td>Clay and silty clay</td>
<td></td>
</tr>
<tr>
<td>&lt;50% clay</td>
<td>0.32</td>
</tr>
<tr>
<td>&gt;50% clay</td>
<td>0.28</td>
</tr>
</tbody>
</table>
The topography effect is incorporated in the USLE by the flow length and slope factors, usually represented as a dimensionless topographic factor $LS$ (Figure 2.6). The slope length $L$ is measured from the point where surface flow originates to the downslope point where the deposition begins.

Land cover factor $C$ is a dimensionless ratio of soil loss from a given combination of vegetative cover and management practices to the soil loss resulting from tilled, continuously fallow earth. Values of $C$ for different land covers are shown in Table 2.6.

![Dimensionless Topographic Factor LS for USLE](image)

**Figure 2.6** Dimensionless Topographic Factor LS for USLE
### Table 2.6. Values of C in USLE for Different Land Covers

<table>
<thead>
<tr>
<th>Land Cover</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous fallow</td>
<td>1</td>
</tr>
<tr>
<td>Undisturbed forest land</td>
<td></td>
</tr>
<tr>
<td>100-75% canopy cover, 100-90% duff cover</td>
<td>0.0001-0.001</td>
</tr>
<tr>
<td>70-45% canopy cover, 85-75% duff cover</td>
<td>0.002-0.004</td>
</tr>
<tr>
<td>35-20% canopy cover, 70-40% duff cover</td>
<td>0.003-0.009</td>
</tr>
<tr>
<td>Permanent pasture and brush cover</td>
<td></td>
</tr>
<tr>
<td>0% canopy, 80% ground cover</td>
<td></td>
</tr>
<tr>
<td>Grass</td>
<td>0.013</td>
</tr>
<tr>
<td>Weeds</td>
<td>0.043</td>
</tr>
<tr>
<td>50% brush, 80% grass cover</td>
<td>0.012</td>
</tr>
<tr>
<td>Bare soil</td>
<td></td>
</tr>
<tr>
<td>Undisturbed except scraped</td>
<td>0.66-1.30</td>
</tr>
<tr>
<td>Compacted, smooth</td>
<td>1.00-1.40</td>
</tr>
<tr>
<td>Compacted, root racked</td>
<td>0.90-1.20</td>
</tr>
<tr>
<td>Disk tillage, fresh</td>
<td>1</td>
</tr>
<tr>
<td>Disk tillage, after one rain</td>
<td>0.89</td>
</tr>
<tr>
<td>Straw mulch, 0.5 tons/acre</td>
<td>0.3</td>
</tr>
<tr>
<td>Straw mulch, 1.0 tons/acre</td>
<td>0.18</td>
</tr>
<tr>
<td>Straw mulch, 2.0 tons/acre</td>
<td>0.09</td>
</tr>
<tr>
<td>Straw mulch, 4.0 tons/acre</td>
<td>0.02</td>
</tr>
</tbody>
</table>
The erosion control practice factor $P$ is the dimensionless ratio of soil loss under a certain erosion control practice such as contouring and terracing. Specific values for most of the USLE factors can be found in the U.S. Department of Agriculture (USDA) Handbook 703 (Renard, Foster, Weesies, McCool, and Yoder, 1997; Wurbs and James, 2001).

### 2.9.1.3. Sediment for Urban Land Use

The sedimentation effects of PCPS will depend partially on the volume of sediments, especially those produced in the adjacent areas that will drain into the system. For a developed site, the sediment yield will depend on the area of the vegetated zones, which produce the sediments that will be deposited on the pavement as shown in Figure 2.7.

The relation between the vegetated zone areas with the PCPS area can be defined in Equation 2.4.

$$\frac{A_{VZ}}{A_{PCPS}} = R_{VP} \quad (2.4)$$

where

- $A_{VZ} = \text{Area of continuous vegetated zones draining onto PCPS},$
- $A_{PCPS} = \text{Area of PCPS},$ and
- $R_{VP} = \text{ratio of vegetated area to PCPS area, all in consistent units}.$

With Equation 2.4, Equation 2.3 can also be written in function of the PCPS area as

$$E = R_{VP} A_{PCPS} R K L S C P \quad (2.5)$$
or

\[ E = R_{VP} A_{PCPS} A \]  

(2.6)

where

\[ E \] = annual soil loss in tons/yr,

\[ R_{VP} \] = ratio of vegetated area to PCPS area

\[ A_{PCPS} \] = area of PCPS in acres, and

\[ A \] = average annual soil loss per unit of area in tons/ac/yr.

Parking lots are usually the main application of PCPS (Tennis, Leming, and Akers 2004), therefore parking lots for commercial sites can be used to estimate the production and deposition and the effect they would have when deposited in a PCPS.

Parking lot sizes for commercial sites in the United States tend to be similar for similar applications. Portland Cement Association (PCA) recommends different numbers of parking spaces per 1,000 square feet of building floor area, depending on the land use, such as restaurants, offices, hospitals, etc. (PCA, 2002). Commercial or industrial sites, for example, will have roughly 65% parking space and 35% building floor area (Figure 2.8). This estimation does not include either the impervious zones of the site, such as sidewalks and streets, or the pervious, vegetated zones such as channelizing islands or contiguous, unpaved areas, which produce soil sediments that can potentially be deposited on the pavement.
Figure 2.7. General Developed Commercial Site Using PCPS as the Parking Lot.

Figure 2.8. Recommended Parking Per 1,000 Square Foot of Building Area
Based on data from Arnold and Gibbons (1996), Cappiella and Brown (2001), and Wells (1994), Ferguson (2005) determined the average percent of land area covered with roofs and pavements based on the land use in urban developments (Figure 2.9). The pavement areas were classified as streets, sidewalks or parking areas. Using estimates from PCA (2002) and Ferguson (2005) industrial, commercial, or shopping center sites will average about 25% impervious zones, such as roofs and sidewalks, 60% pavements, including streets and parking, and 15% vegetated zones, which are usually responsible of producing the soil sediments coming from erosion (Figure 2.10). These average percentages can be used with Equation 2.6 to estimate the production of sediments for a general site with a PCPS using a value of 0.25 for $R_{VP}$.

**Figure 2.9.** Built Cover in Contemporary Urban Land Uses
2.9.2. Hypotheses on the Effects of Different Type of Sediments

Specific effects will depend on the depth and the nature of the sediment load. The sediments that will be carried on to the pavement by stormwater runoff are generally the same type as those on which the pavement was placed. Bean, Hunt and Bidelspach (2007a) confirm this basic understanding for sandy soils. Consider, for example, the cases in which PCPS are installed over a sandy soil, a silty soil and a silty sand soil (Figure 2.11).
2.9.2.1. Sand Sediments

Sandy sediments or at least the coarser particles, are likely to be deposited for the most part on the surface and in the top inch or so of the pervious concrete, not going completely through the PCPS to be deposited at the bottom (Figure 2.11-a).

The exfiltration rate of stored runoff from the PCPS into the underlying soil is not likely to be significantly reduced for a PCPS constructed on sand since, but sand particles may be concentrated at the surface such that flow into the pervious concrete is reduced (Leming, Malcom, and Tennis, 2007), and surface permeability noticeably affected (Bean, Hunt and Bidelspach, 2007a). In this case, permeability may be largely restored by routine maintenance operations (Haselbach, Valavala, and Montes, 2006).
Figure 2.11. Different Cases of Sediment Deposition in Pervious Concrete Pavements Depending on Soil Type
2.9.2.2. Fine Grained Sediments

If the PCPS is placed over a silty soil, fine grained sediments transported by the stormwater are small enough to be washed to the bottom of the PCPS, and retained on the filter fabric of the pavement system (Figure 2.11-b) typically placed under the stone base to prevent intrusion of fine grained subgrade material into the base.

Exfiltration of stored runoff from a PCPS will not likely be significantly changed for a system over a silt subgrade. The potential layer of silt sediments that may deposited at the bottom of the PCPS between the stone sub base and the filter fabric would not be as compacted as the underlying soil. The permeability of the sediment layer, therefore, will probably be significantly higher than that of the compacted silt subgrade and the infiltration rate of the compacted subgrade, that is, the underlying soil, will still control system behavior as in the original design of the PCPS. Loss of storage capacity must still be checked, however. In order to evaluate the likely effects of such a layer, the depth and the permeability of the deposit must be estimated. The depth may be estimated based on volume of deposits and likely density. This value is also useful in estimating the storage capacity loss.

The permeability of fine grain soils is functionally related to the void ratio, which is also a function of density as compacted. A practical relationship for estimating the permeability for clayey-silty soils is obtained from the relationship between infiltration rates of the same fine-grained soil at different void ratios (Braja, 2007):
\[
\log k = \log k_o - \frac{\varepsilon_o - \varepsilon}{C_k}
\]  \hspace{1cm} (2.7)

where:

\(k\) = permeability at a void ratio \(\varepsilon\),

\(k_o\) = in situ permeability at a void ratio \(\varepsilon_o\), and

\(C_k\) = conductivity change index.

Since \(C_k \approx 0.5\varepsilon_o\) for fine grained soils (Braja, 2007);

\[
\log k = \log k_o - \frac{2\varepsilon_o + 2\varepsilon}{\varepsilon_o}
\]  \hspace{1cm} (2.8)

\[
\log k = \log k_o - 2 + 2 \left(\frac{\varepsilon}{\varepsilon_o}\right)
\]  \hspace{1cm} (2.9)

If the void ratio of clayey silt sediments is estimated to be at least 10% higher than the underlying soil

\[
\frac{\varepsilon}{\varepsilon_o} = \frac{\varepsilon_{sed}}{\varepsilon_{insitu}} = \frac{1.1}{1}
\]  \hspace{1cm} (2.10)

and,

\[
\log k = \log k_o - 2 + 2 \left(\frac{1.1}{1}\right)
\]  \hspace{1cm} (2.11)

or

\[
\log k \geq \log k_o + 0.2
\]  \hspace{1cm} (2.12)

or

\[
\frac{k}{k_o} = 10^{0.2} = 1.68
\]  \hspace{1cm} (2.13)

or 60% higher permeability. This indicates that uncompacted silty sediment deposited on the
bottom of the PCPS should not affect exfiltration with silty subgrade.

### 2.9.2.3. Combined Sediments

A combination of these two phenomena may occur in a pavement placed over a silty sandy soil. Sediments would be “filtered” with the coarser sand fraction deposited on top or in the surface of the pavement while the silt fraction would accumulate at the bottom of the PCPS (Figure 2.11-c). A significant difference in design exfiltration values may occur in service in a PCPS placed over a clayey sand, a silty sand or a clayey silty sand, and will be exposed to runoff carrying both clay or silt or both, and sand. The infiltration of a silty sand is moderate, often about 0.5 in./hr (1.3 cm/hr) [Leming, Malcom, and Tennis, 2007]. The coarser particles of sediment may become trapped in the near surface region of the pervious concrete and the fine grained silty material can be deposited on the filter fabric, forming a layer of unconsolidated but very fine grained material between the PCPS and the moderately draining silty sand. The presence of a silt layer, even if un-compacted, over the filter fabric at the bottom of the PCPS may result in substantially lower exfiltration rates from the PCPS (Figure 2.11-c). The presence of even relatively small amount of clay could clearly affect exfiltration significantly. Precise effects are not known at this time, however, another of the primary objectives of this study was to evaluate the effects experimentally and provide design guidelines to the Designer-of-Record faced with this situation.
2.9.3. Layers and Multiple Soil Horizons

The effective permeability of two or more layers of soil can be estimated when the permeability and the depth for each layer is known. Permeability in horizontal and vertical directions is calculated using Equations 2.14 and 2.15 (Cervica, 1995). Both equations assume flow to a normal, cross section and a constant hydrostatic head for each layer, which are reasonable conditions for a PCPS.

\[
k_{xe} = \frac{\sum (k_x H)_n}{\sum H_n}
\]

(2.14)

\[
k_{ye} = \frac{\sum (H)_n}{\sum (H/k_y)_n}
\]

(2.15)

where

- \( k_{xe} \) = equivalent infiltration in the horizontal direction,
- \( k_{ye} \) = equivalent infiltration in the vertical direction,
- \( k_x \) = horizontal infiltration,
- \( k_y \) = vertical infiltration,
- \( H \) = depth of the \( n^{th} \) layer, and
- \( n \) = number of layers.

In order to evaluate the effect this layer of sediments can produce in the hydraulic performance of the system, a family of curves was plotted using Equation 2.15 with different values of permeability and depth of the layer of sediments (Figure 2.12). The changes in the
effective permeability of the system, and therefore, the exfiltration rate, will be affected by the permeability and he depth of this new layer of sediments.

The hydrological behavior of the PCPS must be evaluated at the end of service using a different value of exfiltration to estimate total runoff, peak runoff and recovery time. It is important to develop a method for estimating the exfiltration rate of the system including a layer of fines deposited at the bottom of the system. This need was examined in this study and is addressed in Chapters 3, 4, and 5, and reviewed theoretically in Section 2.9.4.

The effective exfiltration of the subgrade can be estimated using Equation 2.15 when a relatively shallow fill with a higher infiltration rate than the existing material is used in the system. This approach could also be used to determine needed soil characteristics in fill sections or when investigating potential improvement options in a marginal site.

2.9.4. Sediment Deposition Pattern
The presence of silt or clay sediments may also affect exfiltration depending upon the deposition patterns at the bottom of the PCPS. These fine grained soil sediments may have one of at least two different patterns of deposition: (1) sediments may be located close to the point of runoff entering the system, or (2) more or less uniformly distributed as a layer deposited between the stone sub-base and the filter fabric (Figure 2.13). The first case will most likely result in the localized loss of some area of exfiltration, while the second case could result in a reduction of the exfiltration of the entire system. Whether these fine grained
particles will be washed horizontally through the system or be deposited close to the point of entry of the stormwater runoff to the PCPS has not been established. Analysis of particle size, settling velocities and typical stormwater runoff velocities in the PCPS indicates sediment will be carried horizontally, but testing is needed to confirm this analysis.
Figure 2.12. Effective Permeability of a PCPS Affected by the Depth and Permeability of a Layer of Sediment
Figure 2.13. Different Patterns of Sediment Deposition in PCPS
### 2.9.4.1 Settling Velocities of Discrete Particles

A discrete particle is defined as one that, in settling, is not altered in size, shape, or weight. In falling freely through a quiescent fluid, such a particle accelerates until the frictional resistance, or drag, of the fluid equals the weight of the particle in the suspending fluid (Fair, Geyer, and Okum 1971). Therefore, the particle will settle at a uniform, or terminal velocity according to

\[
F_I = (\rho_s - \rho)gV
\]  

(2.16)

where

- \(F_I\) = impelling force,
- \(g\) = gravity constant,
- \(V\) = volume of the particle,
- \(\rho_s\) = density of the particle, and
- \(\rho\) = density of the fluid, all in consistent units.

The drag force \(F_D\) of the fluid is a function of the dynamic viscosity \(\mu\) and mass viscosity \(\rho\) of the fluid, and the velocity \(v_s\), and a characteristic diameter \(d\) of the particle (Fair, Geyer, and Okum 1971). The relationship for the frictional drag can be expressed as:

\[
F_D = \frac{C_D A_c \rho v_s^2}{2}
\]  

(2.17)

where,

- \(C_D\) = Newton’s drag coefficient,
- \(A_c\) = cross sectional area,
\[ \rho = \text{density of the fluid, and} \]

\[ v_s = \text{velocity of the particle, all in consistent units.} \]

Equations 2.16 and 2.17 can be combined to establish a general relationship for the settling of free and discrete spherical particles as follows (Fair, Geyer, and Okum 1971):

\[
v_s = \left[ \frac{4}{3} \frac{g}{C_D} (s_s - 1) d \right]^{1/2} \tag{2.18}
\]

where

\[ v_s = \text{velocity of the particle}, \]
\[ g = \text{gravity constant}, \]
\[ C_D = \text{Newton’s drag coefficient}, \]
\[ s_s = \text{specific gravity of the particle}, \]
\[ d = \text{diameter of the particle}. \]

For viscous resistance at low Reynolds numbers (R < 0.5), CD = 24/R, and Equation 2.18 is known as Stoke’s law (Fair, Geyer, and Okum 1971):

\[
v_s = \frac{g}{18} \frac{(s_s - 1)}{v} d^2 \tag{2.19}
\]

Where,

\[ v_s = \text{velocity of the particle}, \]
\[ g = \text{gravity constant}, \]
\[ s_s = \text{specific gravity of the particle}, \]
\(d = \) diameter of the particle, and

\( \nu = \) kinematic viscosity, all in consistent units.

Figure 2.14 shows curves of settling velocities of discrete spherical particles that span the region between the Stokes range and the turbulent range (Fair, Geyer, and Okum 1971). Settling velocities of coarse particles such as medium to fine sand, with an average a diameter of 0.04 in. (0.1 cm) are in the range of 91,000 in./hr (231,000 cm/hr), settling almost immediately.

For finer particles passing the #200 sieve size (75 µm), settling velocities will be lower than 9.0 in./hr (23 cm/hr). Water through pervious concrete has been reported to flow at speed ranging from 290 in./hr (740 cm/hr) to 770 in./hr (1,960 cm/hr) [Tennis, Leming, and Akers, 2004], and from 240 in./hr (600 cm/hr) to 2,760 in./hr (7,000 cm/hr) [Bean, Hunt and Bidelspach, 2007a]. Since the flow of the water in the pervious concrete is higher than the settling velocities, it will be very likely that these finer particles will not settle until the flow through the pervious concrete stops. If all flow is vertical, a more or less uniformly distributed layer of sediments will be deposited at the bottom of the pervious concrete or between the stone sub-base and the filter fabric if used.

If flow is horizontal, sediments will be deposited where horizontal flow stops, typically at a boundary or possibly at an internal check dam such as those used in some PCPS in sloped pavements. This implies that sedimentation and maintenance strategies must be carefully
considered in sloping PCPS with internal check dams. EB 303 (Leming, Malcom and Tennis, 2007) show an example of this type of construction, which is apparently rarely used.

Figure 2.14. Settling and Riding Velocities of Discrete Spherical Particles in Quiescent Water
Most flows will be vertical but if sediment is carried from off-site onto the PCPS, it is not clear how the sediment may be distributed. This study examined both vertical and horizontal flow characteristics of sediment through pervious concrete.

2.9.5. Sensitivity Analysis

In order to determine the impact that a layer of fine grained sediments deposited uniformly at the bottom of a system would have on the performance of the PCPS placed in a clayey silty sand, a scenario of a single developed site with a PCPS was analyzed.

The proposed development consists of 300,000 ft$^2$ (27,870 m$^2$) of a PCPS parking lot, onto which the runoff of 125,000 ft$^2$ (11,600 m$^2$) of impervious roof and impervious pavement structures drain. Vegetated islands, side slopes occupying 75,000 ft$^2$ (6,970 m$^2$) will also drain into the pervious concrete pavement system. The islands and slopes will be landscaped with grass and some bushes.

A pavement depth of 6 in. (150 mm) will be used in this example. The design porosity of the pervious concrete in the example is 20% and the PCPS is assumed to be level. The effects of base course are examined by using 4 in. (100 mm) of clean stone. A compacted aggregate base consisting of size #57 or #67 stone has a design porosity of 40%. The levels of precipitation used in this analysis are relatively conservative for temperate areas away from a coastline: the precipitation in the 2-year storm is given as 4 in. (100 mm) and the
precipitation in the 10-year storm is given as 6 in. (150 mm). This example is similar to the one used in Leming, Malcom, and Tennis (2007).

The soil used in the example analysis of the site is a loamy sand with some silt, with an intermediate infiltration rate of 0.5 in./hr (1.3 cm/hr), classified as HSG A with a Curve Number of 39 (SCS 1986). In order to determine the effect of the exfiltration rate on the system performance during the service life, different exfiltration values were selected. The exfiltration rate at the beginning of the service life is considered to be the same as the underlying soil. Different exfiltration rates, closer to the value of a clayey silt were used to analyze the effects an uniform deposition layer of sediments at the bottom would have in the performance of the system during the service life.

The PCA software originally developed by Malcom and Leming to analyze the hydrological behavior of a PCPS (CD063, PCA 2007) was used for this analysis. Based on the soil classification provided for pasturage in good condition with a Curve Number (CN) of 39, the pre-development runoff are estimated to be 0.3 in. (8 mm) and 1.1 in. (28 mm) for the 2-year and 10-year storm events respectively. The post-development runoff and CN estimates for the site with a PCPS with different exfiltration rates, are shown in Table 2.7 and 2.8. The storage recoveries of the system after 24 hours are shown in Table 2.9.
### Table 2.7. Runoff for Different Exfiltration Rates at the Beginning and End of Service

<table>
<thead>
<tr>
<th>Case</th>
<th>Exfiltration rate</th>
<th>Runoff, in. (mm)</th>
<th>2-year storm: 4.0 in. (100 mm)</th>
<th>10-year storm: 6.0 in. (150 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5</td>
<td>0.0 (0)</td>
<td>0.5 (12)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.4</td>
<td>0.0 (0)</td>
<td>0.7 (18)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>0.0 (0)</td>
<td>1.0 (25)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.2</td>
<td>0.0 (0)</td>
<td>1.4 (36)</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>0.1</td>
<td>0.1 (3)</td>
<td>2.1 (23)</td>
<td></td>
</tr>
</tbody>
</table>

### Table 2.8. Equivalent Curve Numbers for Different Exfiltration Rates at the Beginning and End of Service

<table>
<thead>
<tr>
<th>Case</th>
<th>Exfiltration rate</th>
<th>Equivalent CN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5</td>
<td>&lt;36*</td>
</tr>
<tr>
<td>2</td>
<td>0.4</td>
<td>&lt;36*</td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>&lt;36*</td>
</tr>
<tr>
<td>4</td>
<td>0.2</td>
<td>39</td>
</tr>
<tr>
<td>5</td>
<td>0.1</td>
<td>47</td>
</tr>
</tbody>
</table>

*Calculated values of the equivalent Curve Number should not be given below about 36.

### Table 2.9. Storage recovery for Different Exfiltration Rates at the Beginning and End of Service

<table>
<thead>
<tr>
<th>Case</th>
<th>Exfiltration rate</th>
<th>Recovery Time &lt; 5 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>0.4</td>
<td>Yes</td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>Yes</td>
</tr>
<tr>
<td>4</td>
<td>0.2</td>
<td>Yes</td>
</tr>
<tr>
<td>5</td>
<td>0.1</td>
<td>Yes</td>
</tr>
</tbody>
</table>
Case 1 shows the PCPS hydrological performance at the beginning of service with an exfiltration rate equal to that of the compacted subgrade. Cases 2 to 5 show the PCPS performance at different stages of the service life with a change on the exfiltration rate due to finer grain sediment uniformly deposited at the bottom of the system. The analysis demonstrates the sensitivity of the PCPS to changes in exfiltration rate such as could occur during sedimentation.

The hydrological performance of the PCPS can be affected significantly by changes in the exfiltration rate during the service life of the pavement. As expected, the increase of runoff during the 10-year storm would be higher, by as much as 1.6 in., than the runoff at the beginning of service. The PCPS with the 4 in. (100 mm) base, would be able to capture most, if not all, of the stormwater for the 2-year storm event, however, for the 10-year event the site would produce 1 in. (50 mm) more runoff at the end of the system life than that produced initially. Equivalent CN for the site would also increase consistently at the exfiltration rate decreases.

The storage recovery of the system would also be affected as the exfiltration rate decreases, taking longer time to regain full storage capacity. For all of the cases in this example, however, full capacity is predicted to be recovered within the next day of the event at the most, as shown in Table 2.9.
This analysis indicates that the sediments can affect the hydrological performance of the system during the service life. As the analysis shows, there is a need for a rational design approach based on fundamental principles, accepted standards, and commonly used methodologies to estimate the effects of this type of sedimentation to 1) estimate the volume of sediment; 2) estimate the depth of the layer of fine grained material; and 3) estimate the effects on these volume losses and layers on long term hydrological behavior. The confirmation of the uniform layer of sediments deposited at the bottom and a procedure to estimate its permeability are necessary to include these effects in the PCPS design.
Chapter 3. Experimental Research Methodology

3.1. Introduction

The experimental part of this study examined sedimentation effects on permeability, vertical and horizontal distribution patterns of sediment, storage capacity effects, and exfiltration rate differences using pervious concrete specimens. The findings of this part of the study were used to develop design guidelines and methodologies, which are discussed in Chapter 5. Frost durability of pervious concrete was also examined.

In this chapter, an overview of the test program used in the study, the development of test parameters, and the concrete materials and mixtures and are provided. Descriptions of the mixing procedures, preparation and curing of the specimens, and the test methodologies are also reviewed.

An on-site test to estimate the permeability of a layer of fine particle sediments uniformly deposited at the bottom of the pavement was developed. The simple procedure would allow to evaluate the exfiltration rate of the system that can affect the hydrological performance of the system during the service life.
3.2. Overview of Experimental Tests

The experimental part of the study was conducted in three phases. The test matrix for each phase of this study is shown in Table 3.1. In Phases I and II, the sedimentation rates using three (3) different pervious concrete mixtures were examined with three different soil types: sand, clayey silt, and clayey silty sand.

Estimates of sediment for a developed commercial site were based on EPA recommendation values for urban land use and the USLE. Sediment volumes for a reasonable worst case erosion rate and for a reasonable typical rate erosion during the PCPS service life were estimated for phase I and II respectively. The sediments were mixed with water and the fluid (“dirty water”) forced to flow either vertically through cylindrical specimens or horizontally through prismatic, beam specimens.

In Phase I, two pervious concrete mixtures were subjected to a low flow with a high sediment load, as a reasonable but worst case scenario. In Phase II, one concrete mixture was subjected to a series of water flows including both high and low flow, with sediment loads estimated to be produced during the service life of the pavement. The pervious concrete was also subjected to repeated sediment loads after washing. Permeability was measured at different stages of sedimentation and washing, and the sediment depositions were observed after testing on all specimens. Porosity, compressive strength, splitting tensile strength, modulus of rupture, and dynamic elastic (Young’s) modulus were determined.
In Phase III, the frost resistance of pervious concrete was determined using a unique test method based on realistic, slow freezing rates of partially saturated, pervious concrete disk specimens. Frost resistance of four different pervious concrete mixtures were produced with a variety of sand and air entrained admixture content was evaluated. This part of the study was limited in scope. It was conducted to provide fundamental understanding of phenomena reported by others and to provide preliminary findings used in additional studies.

3.2.1. **Volumes of Sediment and Runoff**

Estimates of the volumes of sediments and the volume of runoff were required that would represent reasonable worst case volume loadings for PCPS applications in a variety of geographical areas. In addition, different types, that is, particle sizes of sediments, would permit analysis of clogging potential and sediment movement through PCPS of a variety of different soil types. Three different soil types: sand, clayey silt, and clayey silty sand were used in this study. These soil types were selected to provide a range of different effects on sedimentation rates and hydrological behavior. The soil used in this study had been previously examined as part of an unpublished feasibility study at North Carolina State University.

3.2.2. **Phase I: High Sediment Load with Low Runoff Volume**

In Phase I, two pervious concrete mixtures with similar typical porosities, around 20%, were used to produce standard 6 in. (150 mm) by 6 in. (150 mm) by 20 in. (500 mm) beam
specimens, and 4 in. by 8 in. (100 mm by 200 mm) cylinder specimens. The specimens were exposed to the three type of sediments mixed in water to simulate stormwater runoff. Water with sediments was poured to the specimens in one single load and the specimens left to drain for a minimum of 24 hours. Volumes of sediments were estimated considering a reasonable worst case scenario for a high load of sediments with a low water flow. Two cylinder and two beam specimens were used for each pervious concrete in combination with each sediment type. Falling head permeability tests were performed in the specimens before and after sedimentation, that is, exposure to the selected sediment load. After sedimentation, permeability was determined before and after washing with pressurized water. Other tests included total and effective porosity, compressive strength, flexural strength and dynamic elastic (Young’s) modulus based on resonant frequency of the beams.

3.2.3. Phase II: Typical Sediment Load with High and Low Runoff Volume

In Phase II, one pervious concrete mixture with a porosity of approximately 25% was used to produce 6 in. (150 mm) by 6 in. (150 mm) by 40 in. (1000 mm) beam specimens, and 4 in. by 8 in. (100 mm by 200 mm) cylinder specimens. After Phase I, it was determined that most of the sand grains after the sedimentation were either deposited on the surface of the pavement or trapped within the concrete specimen, not affecting the initial exfiltration rate of a PCPS. For this reason, the beam specimens in this phase were exposed to the clayey silt and and clayey silty sand sediments only, mixed in water to simulate stormwater runoff. Cylinder specimens, however, were subjected to sand, clayey silt, and clayey silty sand sediments to confirm the findings from Phase I with more realistic sediment loads. Water
with sediments was added to the specimens at different times, leaving an interval of 24 hours between exposures. Volumes of sediments were estimated considering a relatively common scenario for a intermediate load of sediments during the service life of the pavement. Two cylinder and two beam specimens were subjected to the clayey silt sediments; two cylinder and 4 beam specimens were subjected to clayey silty sand sediments; and two cylinders were used with sandy sediments. Falling head permeability tests were performed in the specimens before, during and after the exposure. Other tests included total and effective porosity, compressive strength, and flexural strength.

### 3.2.4. Phase III: Frost Resistance

In Phase III, the frost resistance of pervious concrete and pervious concrete containing sediment was determined using a unique test based on realistic, slow freezing rates of partially saturated pervious concrete disk specimens. Previous testing had been reported to have been conducted at high rates using ASTM 666 (Kevern, Schaefer, Wang and Suleiman, 2008). Four different pervious concrete mixtures were used to study the effects of sand and air entraining admixtures on frost resistance. Disk specimens with 4 in. (100 mm) diameter and 1 in. (25 mm) depth were partially saturated in a solution containing 1% by mass of calcium chloride, and subjected to one freezing (low temperature) and thawing (high temperature) cycle per day. The mass and elastic dynamic (Young’s) modulus based on resonant frequency of the disks were measured every 10 cycles. Microscopic observation to determine entrained air was performed on selected specimens. Other tests included effective porosity, permeability and compressive strength.
Table 3.1. Test Matrix for the Three Phases of the Study

<table>
<thead>
<tr>
<th>Test</th>
<th>Standard/type</th>
<th>Specimen type</th>
<th>Mixture 1</th>
<th>Mixture 2</th>
<th>Mixture 3</th>
<th>Mixture 4</th>
<th>Mixture 5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Phase I: sedimentation with low flow and high sediment load</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Porosity</td>
<td>ASTM C 140</td>
<td>4 x 6 cylinders</td>
<td>7</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 x 6 x 20 beams</td>
<td>7</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>ASTM D 7063</td>
<td>4 x 6 cylinders</td>
<td>2</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permeability</td>
<td>Falling head</td>
<td>4 x 6 cylinders</td>
<td>6</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flow rate</td>
<td>Single ring</td>
<td>6 x 6 x 20 beams</td>
<td>7</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sedimentation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td></td>
<td>4 x 6 cylinders</td>
<td>2</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 x 6 x 20 beams</td>
<td>2</td>
<td>2</td>
<td></td>
<td></td>
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<tr>
<td>Clayey silt</td>
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<td>4 x 6 cylinders</td>
<td>2</td>
<td>2</td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 x 6 x 20 beams</td>
<td>2</td>
<td>2</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Clayey silty sand</td>
<td></td>
<td>4 x 6 cylinders</td>
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<td>2</td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 x 6 x 20 beams</td>
<td>2</td>
<td>2</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Compressive strength</td>
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<td>2</td>
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<td></td>
<td></td>
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<tr>
<td>Modulus of Rupture</td>
<td>ASTM C 78</td>
<td>6 x 6 x 20 beams</td>
<td>7</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic modulus</td>
<td>ASTM C 215</td>
<td>6 x 6 x 20 beams</td>
<td>7</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Phase II: Sedimentation with high to low flow and moderate sediment load</strong></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Porosity</td>
<td>ASTM C 140</td>
<td>4 x 6 cylinders</td>
<td>6</td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>ASTM D 7063</td>
<td>4 x 6 cylinders</td>
<td>6</td>
<td></td>
<td></td>
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<td>12</td>
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</tr>
<tr>
<td>Flow rate</td>
<td>Single ring</td>
<td>6 x 6 x 40 beams</td>
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</tr>
<tr>
<td>Sedimentation</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td></td>
<td>4 x 6 cylinders</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clayey silt</td>
<td></td>
<td>4 x 6 cylinders</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 x 6 x 40 beams</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clayey silty sand</td>
<td></td>
<td>4 x 6 cylinders</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 x 6 x 40 beams</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>ASTM C 39</td>
<td>4 x 6 cylinders</td>
<td>2</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus of Rupture</td>
<td>ASTM C 215</td>
<td>6 x 6 x 20 beams</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Phase III: Frost durability</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Porosity</td>
<td>ASTM C 140</td>
<td>4 x 6 cylinders</td>
<td>7</td>
<td>7</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 x 1 disks</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ASTM D 7063</td>
<td>4 x 6 cylinders</td>
<td>2</td>
<td>2</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 x 1 disks</td>
<td>5</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permeability</td>
<td>Falling head</td>
<td>4 x 6 cylinders</td>
<td>7</td>
<td>7</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Dynamic modulus</td>
<td>4 x 1 disks</td>
<td>4</td>
<td>4</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Compressive strength</td>
<td>ASTM C 39</td>
<td>4 x 6 cylinders</td>
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<td>3</td>
<td>2</td>
<td>2</td>
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</tr>
<tr>
<td>Freezing/thawing</td>
<td>4 x 1 disks</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
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<td></td>
</tr>
</tbody>
</table>
3.3 Materials and Concrete Mixtures

Two sets of pervious concrete mixtures were prepared for Phase I. Both mixtures were provided with a uniformly graded coarse aggregate with a nominal maximum size of 3/8 in. (9.5 mm). Sand was not included in either mixture. White cement (Type I) was used in both mixtures to try to improve color contrast with the sediment, although this was later found to provide no real improvement in visual discrimination.

Mixture 1 had a water-cementitious materials ratio (w/c) of 0.33 and Mixture 2 had a slightly lower w/c ratio of 0.32. Cementitious materials were kept constant and water content was reduced slightly in the second mixture, which also had a higher dose of Air Entraining Admixture (AEA) and a slightly higher coarse aggregate content. Mixture 1 and Mixture 2 were intended to have different porosities, however, they were very similar as shown in Section 4.2.2.

Two batches of Mixture 3 were required to fabricate the total number of specimens. Mixture 3 used in Phase II, unintentionally had a slightly higher coarse aggregate content than Mixture 1 and Mixture 2. A water-reducing admixture was also added to improve workability. Type I conventional gray cement was used for Mixture 3. The coarse aggregate used for all the mixtures showed irregular sharp and flat characteristics (Figure 3.1).

Two additional pervious concrete mixtures were prepared for Phase III of the study. Both mixtures substituted 7% by mass of coarse aggregate with fine aggregate based on
recommendations by Kevern, Schaefer, Wang and Suleiman (2008). Cement, fly ash, and water proportions were kept equal to those in Mixture 1. Type I, conventional gray cement was used instead of the white cement. A higher dose of AEA was also used in Mixtures 4 and 5, and the AEA used in mixture 5 was twice the dose used in Mixture 4. The summary for all five mixtures are shown in Table 3.2.

3.3.1. Mixing Procedures

The concrete for Mixture 1, Mixture 2, and Mixture 3 were prepared in 4 cubic foot (0.11 m$^3$) batches using a rotating-drum concrete mixer with a 6 cubic foot (0.17 m$^3$) capacity. The mixer was coated with a representative mortar (“buttered”) immediately prior to batching. The mixing sequence consisted of premixing half of the water, with the AEA, and the coarse aggregate in the mixer. The cement and fly ash were then added, the drum started and the remaining water added. The mixture was mixed for 3 minutes, allowed to rest for 2 minutes and then remixed for 2 minutes in general accordance with ASTM C 94.

Mixture 4 and Mixture 5 were mixed using a 0.5 ft$^3$ (0.01 m$^3$) open pan. The coarse, fine aggregate, fly ash, and a small amount of cement, less than 5% by mass, were dry mixed thoroughly. The remaining cement and the water, already containing the AEA, were added and the concrete. The concrete was then mixed for at least three minutes, allowed to rest for three minutes, and then mixed again for two minutes. This procedure was similar to that used by Kevern, Schaefer, Wang and Suleiman (2008) to study frost durability in pervious concrete.
Table 3.2. Pervious Concrete Mixture Proportions

<table>
<thead>
<tr>
<th>Materials</th>
<th>Mixture 1</th>
<th>Mixture 2</th>
<th>Mixture 3</th>
<th>Mixture 4</th>
<th>Mixture 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement - Type I, lb/yd(^3) (kg/m(^3))</td>
<td>600 (356)</td>
<td>600 (356)</td>
<td>600 (356)</td>
<td>600 (356)</td>
<td>600 (356)</td>
</tr>
<tr>
<td>Fly Ash - Type F, lb/yd(^3) (kg/m(^3))</td>
<td>120 (71)</td>
<td>120 (71)</td>
<td>120 (71)</td>
<td>120 (71)</td>
<td>120 (71)</td>
</tr>
<tr>
<td>F. Aggregate</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>177 (105)</td>
<td>177 (105)</td>
</tr>
<tr>
<td>C. Aggregate #78, lb/yd(^3) (kg/m(^3))</td>
<td>2,532 (1,500)</td>
<td>2,554 (1,514)</td>
<td>2,712 (1,607)</td>
<td>2,355 (1,393)</td>
<td>2,355 (1,393)</td>
</tr>
<tr>
<td>Water, lb/yd(^3) (kg/m(^3))</td>
<td>237 (140)</td>
<td>228 (135)</td>
<td>237 (135)</td>
<td>237 (140)</td>
<td>237 (140)</td>
</tr>
<tr>
<td>AEA oz/cwt (mL/100 kg)</td>
<td>1.14 (74)</td>
<td>2.28 (148)</td>
<td>1.14 (74)</td>
<td>1.60 (104)</td>
<td>3.20 (208)</td>
</tr>
<tr>
<td>Water reducing adm. oz/cwt (mL/100 kg)</td>
<td>-</td>
<td>-</td>
<td>4.0 (260)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
3.3.2. Fabrication and Curing of Specimens

Single use, plastic cylinder molds, 4 in. by 8 in. (100 mm by 200 mm) conforming to ASTM C 192 were filled in one lift to a depth of approximately 1.0 to 1.5 in. (25 to 38 mm) from the top such that approximately 6 in. (150 mm) tall cylinders would be produced after compaction. Compaction was performed using a standard proctor hammer on top of a steel 10.1 lbs (4.5 kg) steel cylinder, 3¾ in. (96 mm) in diameter and 4 in. (100 mm) tall on top of the fresh concrete. This was done to distribute the compaction effort evenly over the surface of the concrete. The concrete was compacted with seven drops of the standard proctor hammer. Although no standards for consolidation or pervious concrete have been developed, current recommendations for number of drops described in Section 2.4.1 were considered to over consolidate the specimen. The count of seven was determined as the point in which the level of the steel cylinder placed on top of the fresh concrete remained essentially constant. The porosity attained with this process was consistent with that attained in practice with similar materials and in the range desired.

Four in. (100 mm) diameter by 2 in. (50 mm) tall molds were filled in one lift up to approximately 0.5 in. (12 mm) from the top such that approximately 1 in. (25 mm) in depth concrete disks would be produced after compaction. Compaction was performed using the same procedure as for the cylinder specimens. Since the volume was less for these specimens the compaction energy per unit volume was greater than for the other cylindrical specimens.
A total of seven (7) prism specimens were cast from Mixture 1 and Mixture 2. Test specimens were formed in previously lubricated steel 6 in. (152 mm) by 6 in. (152 mm) by 20 in. (508 mm) flexural strength molds. The concrete was compacted to simulate field compaction to the extent possible.

Figure 3.2. Fabrication of Cylinder Specimens Using the Proctor Hammer.
Figure 3.3 Fabrication of Beam Specimens Using Steel Cylinders for Compaction.

Figure 3.4. Fabrication of Beam Specimens Using Steel Cylinders for Compaction in Phase II.
The specimens were fabricated by first slightly overfilling the forms in one lift using a jigging procedure similar that described in ASTM C 29. A steel cylinder, 10 lbs (4.5 kg), 2 in. (50 mm) in diameter by 10.1 in. (254 mm) long steel, which fit across the top of the beam, was used as a screed to remove excess concrete from the top of the beam and provide a slight compaction (Figure 3.3-a). Another steel cylinder, 17.2 lbs (7.8 kg), 4 in. (100 mm) in diameter by 5 in. (127 mm) long, that fit inside the beam mold edges was used with slight hand pressure to provide approximately 10 psi (69 kPa) contact pressure was applied to compact the surface of the specimen (Figure 3.3-b). The compaction process began by placing the steel cylinder in the middle of the specimen and rolling to one end, replacing the cylinder in the middle and then rolling to the other end of the specimen. Each specimen was “rolled” two times.

Beam specimens twice the long as standard beams were used in the evaluation of longitudinal sediment deposition. These specimens were cast in steel 6 in. (150 mm) by 6 in. (150 mm) by 40 in. (1,000 mm) molds that had been lightly lubricated. The procedure to fabricate the specimens was similar to the standard beams, with the difference that two fiberboard shim-type spacers, 3 in. (75 mm) wide, 45 in. (1,143 mm) long, and 0.25 in. (6 mm) thick were used on top of each side of the mold to permit more precise control of screed depth and final compaction depth (Figure 3.4). This procedure closely resembles field fabrication techniques as described in Section 2.5.
Mixture 1 and Mixture 2 specimens were protected from moisture loss for 5 days and then placed in a moist room conforming with ASTM C 192 for 21 days. Mixture 3 specimens were protected from moisture for 3 days and then placed in moist room for 19 days. Mixture 4 and Mixture 5 specimens were protected from moisture loss for 24 hours and then immersed in a curing bath for 7 days. The differences in the curing procedures were due in part to laboratory installation availability. Curing was not critical for this study since extended moist curing would have no effect on porosity or permeability.

3.4. Test Methods

3.4.1. Mechanical Properties

Mechanical properties were determined for completeness, and the data is provided for general information only.

The modulus of rupture was determined using 6 in. by 6 in. by 20 in. (152 mm by 152 mm by 508 mm). Seven (7) prism specimens from Mixture 1 and seven (7) prism specimen from Mixture 2 were tested. Modulus of rupture for Mixture 3 was determined using six (6) 6 in. by 6 in. by 40 in. (152 mm by 152 mm by 1,016 mm) beam specimens. All beam specimens were tested after 90 days in general accordance with ASTM C 78, with the exception that the specimens were dry. All tests were performed after sedimentation testing.
The dynamic (Young’s) modulus of elasticity ($E_d$) was determined using two different types of specimens. Beam specimens, 6 in. by 6 in. by 20 in. (150 mm by 150 mm by 500 mm) were used to determine $E_d$ in accordance with ASTM C 215. Seven (7) beams from Mixture 1 and seven (7) beams from Mixture 2 were tested.

The dynamic modulus of concrete disks was conducted as described in Leming, Nau and Fukuda (1999), using the fundamental cyclic natural frequency of the disks. The fundamental cyclic natural frequency of the disk was determined experimentally using a small, piezoelectronic accelerometer connected to one side of the specimen with a soft, adhesive wax with good acoustical properties. The disk was exited by striking it with a steel ball. To allow free free-vibration, the disk was air suspended vertically.

The signal from the accelerometer was captured at a sampling rate of 100 kilohertz. The signal was analyzed after removing the portion of the signal immediately following the impact of the ball-bearing. The fundamental frequency was identified using Fast Fourier Transform (FFM) techniques. A minimum of three tests was conducted on each disk. The thickness and diameter of each disk was determined with micrometers. Measurements were taken at several points. The bulk mass density of the concrete specimens was determined from air-dry and immersed mass measurements, following the procedures outlined in ASTM C 29.
The elastic (dynamic) modulus can be determined as described by Leming, Nau and Fukuda (1998) as:

\[
E_d = 2(1 + \nu)\rho \left[ \frac{\pi f d^2}{\Omega_0} \right]
\]  

(3.2)

where,

\(E_d\) = the elastic (dynamic) modulus,

\(\nu\) = Poisson’s ratio,

\(\rho\) = the mass density of the disk,

\(f\) = the fundamental cyclic natural frequency in Hz,

\(d\) = the diameter of the disk, and

\(\Omega_0\) = the frequency parameter associated with the fundamental mode of vibration.

Compressive strength was determined using cylinders specimens of 4 in. (100 mm) by approximately 6 in. (150 mm); testing was after 90 days for all mixtures. Two (2) cylinder specimens from each mixture were tested in general accordance with ASTM C39, including the size factor of the specimens. In order to have plane ends, the cylinders were ground on the top end.

Tensile strength was determined using cylinder specimens of approximately 4 in. by 6 in. (100mm by 150 mm); testing was after 90 days for all mixtures. One (1) cylinder specimen from each mixture was tested in general accordance with ASTM C 496.
3.4.2. Hydrologic Properties

3.4.2.1. Porosity

Effective porosity, that is, the percent of pores easily filled with water by submersion at atmospheric pressure, was measured using two different approaches. There is no universally accepted standard at this point, but most researchers and practitioners use one or both of these methods. As discussed in Section 2.4.1, the effective porosity will be less than the total porosity because some small pores will not be filled during the immersion. Effective porosity will also therefore tend to be somewhat more variable in repeated testing. This value is preferred for hydrological analysis however since it provides a more realistic estimate of storage capacity, although the effects are generally not significant in design (EB 303 [Leming, Malcom, and Tennis, 2007]).

Effective porosity was obtained in selected specimens using ASTM D 7063, Standard Test Method for Effective Porosity and Effective Air Voids of Compacted Bituminous Paving Mixture Samples, utilizing an Instrotek Corelok System (Instrotek, 2003). The specimens were dried to a constant mass at a temperature that did not exceed 40 °C (104 °F). The specimens were then placed into a plastic bag, the air was evacuated from the sample and the bag sealed. The mass of the specimen and bag submerged in water was determined. The plastic bag was then cut, allowing water to normally saturate the specimen, and the mass of the saturated specimen was determined. The effective porosity was then calculated as described in ASTM D 7063 (Figure 3.5).
Porosity of the specimens was also determined using Equation 3.1, which is based on the difference in weight between the dry specimen and the weight of the immersed specimen similar to the method described in ASTM C 140. Specimens were tested at ages greater than 21 days.

\[ V_r = \left( 1 - \frac{W_2 - W_1}{\rho \cdot Vol} \right) \times 100 \]  

(3.3)

where

- \( V_r \) = total void ratio (%),
- \( W_1 \) = weight immersed (lbs or kg),
- \( W_2 \) = dry weight (lbs or kg),
- \( Vol \) = nominal volume of sample calculated using dimensions of the sample (ft\(^3\) or m\(^3\)), and
- \( \rho \) = density of water (lbs/ft\(^3\) or kg/m\(^3\)).

Porosity was determined using Equation 3.1 for nine (9) cylinder specimens and seven (7) beam specimens from Mixture 1 and Mixture 2. The effective permeability in accordance with ASTM D 7063 was determine for two (2) cylinder specimens from Mixture 1, and two (2) cylinder specimens from Mixture 2 for comparison purposes.

In Phase II of the study, twelve (12) cylinder specimens form Mixture 3, six (6) from each batch, were used to determine the effective porosity in accordance with ASTM D 7063. Porosity was also determined in the same specimens using Equation 3.3.
Three (3) cylinder specimens and four (4) disk specimens from Mixture 4, and three (3) cylinder specimens and four (4) disk specimens from Mixture 5 were used in Phase III to determine the effective porosity in accordance with ASTM D 7063. Porosity was also determined in the same specimens using Equation 3.3.

**Figure 3.5.** Determination of Effective Porosity in Accordance with ASTM D 7063
3.4.2.2. Permeability

Permeability tests of cylinder and flow rate in beam specimens were conducted based on conventional falling head permeameter principles. The permeability in the cylindrical pervious concrete specimens was measured using a classical, one-dimensional flow approach. Both sediment and water was carried from the top of the cylinders to the bottom. The same approach was used with beam specimens to evaluate sediment deposition in two dimensions. Since flow was not uniform, however, an accurate permeability measure was not possible with beam specimens. Flow rates in the permeameter could be used for comparison, however.

The apparatus used to evaluate permeability of pervious concrete cylinder specimens in Phase I of the study consisted of a 4 in. (100 mm) diameter PVC riser pipe 16 in. (405 mm) long, with a marked transparent covering over a portion of the PVC pipe that had been removed so that measurement of the water level could be obtained over time. The concrete specimen was enclosed in a rubber sleeve, directly attached to the PVC pipe as shown in Figure 3.6-a and Figure 3.7-a. The samples were covered on the sides with heavy water proof tape so that water flow was not allowed through the sides of the specimen. A filter fabric was placed at the bottom of the specimen to hold the soil sediments that would be carried by runoff and percolate through the system (see Figure 3.7). The filter fabric was a non-woven needle-punched geotextile made of polypropylene staple filaments, with a nominal opening size of 0.0083 in. (0.212 mm) and a flow rate of 13,440 in/hr (9.33 cm/s) similar to that commonly used in PCPS construction.
The wrapped specimen was clamped to the riser pipe on one end and a PVC reducer on the other end. The reducer was fitted with a 90° elbow and a PVC cap that could be attached to keep the water in the specimen before running the permeability test (see Figure 3.6-b). A wooden frame was used to maintain the setup in a vertical position at all times.

One gallon (3.79 L) of water was placed in the riser pipe to a depth of about 18 inches (47 cm). The water was allowed to set for at least 2 minutes to fill pores, allow air in the specimen to escape and to ensure the test setup was not leaking prior to removing the cap.

The cap on the bottom was then removed and the time for 9 in. (23 cm) of water to permeate through the concrete recorded. The 9 in. (23 cm) of water used in the test was from the 5th inch to the 14th in. markings to provide a consistent head and to permit more accurate measurement of elapsed time.

Permeability of cylinder specimens for Phase II was determined using a 7.5 in. (175 mm) riser pipe instead of the 16 in. (405 mm) pipe used in Phase I (Figure 3.6-c). The procedure was similar with the difference that 0.44 gal (1.67 L) of water [8 in. (203 mm)] were added. The time required for 3 in. (76 mm) of water to flow through the concrete was recorded. The water levels were used for the test the 2nd to the 5th in. (50.8 mm to 127.0 mm) in the calibrated opening to allow the water level to stabilize once it was added. At least two measurements for each specimen were taken and the average calculated each time.
Specimens were tested at ages greater than 28 days. This test device, with either riser pipe, would only be appropriate for relatively high permeability materials or layers of materials such as found in PCPS. The change of permeameter in Phase II was to reduce the water pressure pattern produced by the high level of the water in the taller permeameter that could affect the sediment deposition pattern during the sedimentation test (Section 3.6.3).

The average coefficient of permeability \((k)\) was determined using Equation 3.3, which is based on Darcy’s law principle of flow in a homogeneous, porous material.

\[
k = \frac{aL}{At} \ln \left( \frac{h_1}{h_2} \right) \tag{3.4}
\]

where

\(k\) = coefficient of permeability, in/s or cm/s,

\(a\) = cross-sectional area of the pipe, in\(^2\) or cm\(^2\),

\(L\) = Length of sample, in or cm,

\(A\) = cross-sectional area of the sample, in\(^2\) or cm\(^2\),

\(T\) = time for water to drop from \(h_1\) to \(h_2\), s,

\(h_1\) = initial water level, in or cm, and

\(h_2\) = final water level, in or cm.

Flow rate of the beam specimens was determined based on a falling head, single ring principle using a 7.5 in. (109.5 mm) long, 4 in. (101.6 mm) inner diameter PVC pipe with a calibrated opening for measurement of the water level, similar to that used with the
cylindrical specimens. Heavy, water proof tape was placed on all faces of the beam except on one end to allow the water drain freely (Figure 3.7-b,c). Filter fabric was installed at the bottom of the specimen to hold any sediment carried by the water. The specimens were placed over two plastic supporters on each end to keep them elevated and let the water flow freely from both the bottom and the uncovered end. The relatively slower rate of flow of the filter fabric ensured flow both vertically and horizontally. The specimens were tested at ages greater than 28 days.

Approximately 0.44 gal (1.67 L) [8 in. (203 mm)] of water were added at the top of the specimen inside the pipe. The time required for 4 in. (100 mm) of water to flow through the concrete was recorded. The water levels used for the test were the 2nd to the 5th in. (50 mm to 127 mm) in the calibrated opening to allow the water level to stabilize once it was added (see Figure 2.9) since flow through the specimen was immediate. At least two measurements for each specimen were taken and the average time calculated.
Figure 3.6 Basic Schematics of Permeameters Used in Phase I and Phase II

Figure 3.7. Preparation of Cylinder and Beam Specimens for Sedimentation Tests
Figure 3.8. Test Setup to Determine Permeability of Cylinder Specimens

Figure 3.9. Falling Head Single Ring Test to Determine Water Flow of Beam Specimens
3.5. Sedimentation Test

3.5.1 Sediment Loads

The sediment load used in the Phase I of the study was considered a worst case scenario with a sediment production of 8,000 lb/acre (9,000 kg/ha) per year for 20 years of service. This sediment load is higher than the 6,000 lb/acre (6,750 kg/ha) per year estimated by EPA for construction sites. The sediment load for Phase II used EPA estimations of 1,000 lb/acre (1,125 kg/hr) per year for a commercial site for 20 years of service.

Since parking lots are usually the main application of PCPS (Tennis, Leming, and Akers 2004), these quantities were assumed to be produced by the vegetated zones of a developed site as described in Section 2.9.1.3. Equation 2.6 for Phase I and Phase II, with a value of 0.25 for \( R_{VP} \) can be written as

\[
E_{\text{Phase I}} = 0.25 A_{PCPS} (8,000) \quad (3.5)
\]
\[
E_{\text{Phase II}} = 0.25 A_{PCPS} (1,000) \quad (3.6)
\]

It was also considered that the distribution of the sediments onto the pavement may not be uniform but localized in areas close to the point of runoff entering the system as shown in Figure 2.7. These areas will depend on specific characteristics of the site, and a precise estimate for a general case could not be possible due to the nature of the estimation (sediments) and the different variables of the site in consideration. For this test, an area of 5% of the PCPS was considered to receive the sediment load in Phase I and Phase II. This is a
very conservative estimate since a larger area of PCPS will result in a lower load of sediments per area in the site; however, it was a sufficient load to determine the deposition patterns of the sediments in the concrete specimens.

If the sediments deposit on 5% of the area of the PCPS, then Equations 3.5 and 3.6 can be written as

\[ E_{\text{Phase I}} = \frac{2,000A_{\text{PCPS}}}{0.05A_{\text{PCPS}}} = 40,000 \frac{lbs}{ac} = 0.92 \frac{lb}{ft^2} \]  
(3.7)

\[ E_{\text{Phase I}} = \frac{250A_{\text{PCPS}}}{0.05A_{\text{PCPS}}} = 5,000 \frac{lbs}{ac} = 0.12 \frac{lb}{ft^2} \]  
(3.8)

For a service life of 20 years, and for the surface area corresponding to the sedimentation test described in Sections 3.3.3. and 3.3.4, 1.76 lbs (0.8 kg) and 0.22 lbs (0.1 kg) were used for each test in Phase I and Phase II respectively.

### 3.5.2 Soil Sediment Characteristics

The type of soils used in this study were selected to have the most significant effects on reduction of exfiltration from the system by deposition of a layer of clayey or silty material on top of the filter fabric. In addition, a soil with both coarse and fine particles will likely segregate on a PCPS with the larger sized fraction remaining on or in the near surface region, and the finer particle susceptible to transport through the pervious concrete both horizontally and vertically. ACI 522 (ACI 2006), Tennis, Leming, and Akers (2004), and EB303 (Leming, Malcom, and Tennis, 2007), and Ferguson (2005) recommend the use of PCPS
over well draining soils. Leming, Malcom, and Tennis (2007) also note that it is possible to
design a PCPS in a silty soil (infiltration about 0.1 in/hr [0.3 cm/hr] or less) but that
performance may be barely acceptable due to extended recovery time.

Particle size analysis of the soil used in the tests was conducted in general accordance with
ASTM D 422. The sand used was a medium to fine sand as classified by the Unified Soil
Classification, with particles retained between 0.08 in. (2 mm) and the #200 sieve size (75
µm). Approximately 40% of the sand was classified as medium sand and the remaining 60%
as fine sand. Finer grained particles, those passing the #200 sieve size (75 µm), contained
approximately 25% clay as measured by the hydrometer test in general accordance with
ASTM D 422. The clayey silty sand was composed of approximately 60% sand and 40%
clayey silt. Sediments of the three types of soil used in this study are shown in Figure 3.10.

For Phase I, the clayey silty sand used for the sedimentation test was used without any prior
gradation or re-blending. Since the effect of the clayey silty sand sediment was hard to
determine during this phase as described in Section 4.3.1.3, the sediments for Phase II were
separated in particles retained and particles passing the #200 sieve size (75 µm), and re-
blended with 60% sand and 40% clayey silt.
Figure 3.10 Soil Sediment: Clayey Silt, Clayey Silty Sand, and Sand
3.5.3 Low Flow with High Sediment Load Test

The sedimentation characteristics of the pervious concretes were examined with three different soil types: a) sand, b) clayey silt, and c) a combined soil of clayey silty sand. Sedimentation rates of each of the soil were examined in one-dimensional and two-dimensional tests to observe vertical and horizontal patterns of sediment deposition in the pervious concrete.

Sediment load for the low flow test considered the reasonably worst case scenario as described in Section 3.3.1. A total of 1.76 lbs (0.82 kg), corresponding to the cylinder surface area, was mixed with 1 gallon (3.8 L) of water to simulate yearly runoff in one single load. This same concentration of water and sediments was used for the one-dimensional and two-dimensional tests.

Cylinder specimens were used for the one-dimensional test. The test method consisted of a vertical PVC pipe attached to the surface of the specimen on one end. The PVC pipe was attached to the pervious concrete specimen with a clamper rubber sleeve similar to the one used in the permeability test. The lower part of the specimen was also attached to a PVC pipe extension 4 in. (100 mm) long to keep the specimen suspended in the air and not making contact with the ground. One dimension sedimentation test setup is shown Figure 3.11-a.

Water with sediments was added to the pipe in one single load and then left to rest for a minimum of 24 hours, during which the water had completely drained through the specimen.
The flow rate was variable, depending on the type of sediment used in the test and affected by soil sediments deposited on the surface of the specimen.

Once the water with sediments has completely drained, the top pipe was detached, the remaining sediments on the surface were then removed with pressurized water and permeability was measured again. The filter fabric was then removed from the bottom of the specimen to observe the sediments deposited at the bottom. Permeability was measured again after the removal of the filter fabric. Permeability was measured using the 16 in. (41 cm) permeameter in all cylinder specimens. Two specimens from Mixture 1 and two specimens from Mixture 2 were tested with each sediment type.

The two-dimensional test method using beam specimens consisted of a vertical PVC pipe attached to the surface of the specimen on one end. The specimens were placed over two plastic supporters on each end to keep them elevated and let the water flow freely. The PVC pipe was attached to the pervious concrete specimen with a waterproof, moldable, adhesive caulk to ensure the water with the sediments penetrated the concrete instead of escaping through the gap between the pipe and the surface of the specimen (Figure 11-b).

Water with sediments was added to the pipe at one time and then left to rest for a minimum of 24 hours, by which time the water had completely drained through the specimen. The flow rate was variable with each sediment type, and was strongly affected by the surface clogging of each specimen.
The pipe was detached, the remaining sediments on the surface were then removed with pressurized water and the permeability was measured again. The filter fabric was then removed from the bottom of the beams to observe the deposition patterns of each sediment. Two specimens from each mixture were tested with each sediment type.

Figure 3.11. Low Flow Sedimentation Test in Phase I
3.5.4. High and Low Flow with Typical Sediment Load

The sedimentation characteristics of pervious concrete specimens from Mixture 3 were examined with the three different type of soil described in Section 3.2.3. The test was initially performed with a high flow of water with a lower concentration of sediments that would reduce the flow gradually as more sediments deposit into, or on the specimen.

Sediment load for the low flow test considered a typical scenario as described in Section 3.3.1. A total of 0.22 lbs (0.0.1 kg) was mixed with 5 gallon (18.9 L) of water. This volume of water was selected to provide a relatively small concentration of sediment per volume of water to permit a high water flow. The flow rate was variable with each sediment type, starting with a high flow and reducing the rate due to the accumulation of soil sediments deposited on the surface or within the specimen.

Cylinder specimens were used for the one-dimensional test. The test method used the 7.5 in. (190 mm) long permeameter attached to the top surface of the specimen. The water with sediments was added to the permeameter and the permeability was recorded during the last fraction of the total volume, approximately the last 0.3 gallons (1.14 L) of the mixture. The permeameter and the specimen were kept standing using the same frame used in the permeability test.

Once the specimen had completely drained, the permeameter was detached, and the specimen, including any trapped sediments, was left to air-dry for a minimum of 24 hrs.
Sediments on the surface were then removed with pressurized water and permeability was measured again. This procedure, using the same sediment load and water volume, was conducted a total of three times for each specimen to examine the effects of sedimentation and washing cycles. The filter fabric was removed after the third sedimentation test to observe the sediments deposited at the bottom of the beam. Permeability was measured again after the removal of the filter fabric. Four (4) specimens from Mixture 3 were tested with each sediment type.

Sediment deposits at the bottom of the standard length beams used in Phase I testing were essentially uniform (see Section 4.3.1.2). Longer beams were used in this phase to confirm if the sediment deposit would change with a longer flow length. As seen in Section 2.9.3 a non-uniform pattern of sediment deposits could affect design considerations compared to a uniformly deposited layer. Since fine sediments would form these deposits, two (2) beam specimens were tested with the clayey silty soil and four (4) beam specimens were tested with clayey silty sand. No long beam specimens were tested with sand.

For the two-dimensional testing, the 7.5 in. (190 mm) long permeameter was attached to one end of the surface of a long beam specimen. Water containing the typical sediment load was added to the permeameter and the flow rate was recorded during the last fraction of the mixture, approximately the last 0.3 gallons (1.14 L) of the mixture (Figure 3.12). The specimens were placed over two plastic supporters on each end to keep them elevated and let the water flow freely. The flow rate was variable with each sediment type, starting with a
high flow and reducing the rate due to the accumulation of soil sediments deposited on the surface or within the specimen. Once the specimen had completely drained, the permeameter was detached, and the specimen, including any trapped sediments, was left to air-dry for a minimum of 24 hrs. Sediments on the surface were then removed with pressurized water and flow rate was measured again. This procedure, using the same sediment load and water volume, was conducted a total of 5 times for each specimen to examine the effects of sedimentation and washing cycles. The filter fabric was removed after the fifth sedimentation test to observe the sediments deposited at the bottom of the beam. Flow rate was measured again after the removal of the filter fabric.

Figure 3.12. Sedimentation Test in Phase II
3.6. Sediment Permeability Tests

One of the primary objectives of this study was to develop design guidelines that could be used by practical engineers to analyze hydrological behavior of PCPS over the design life, including the effects of sedimentation. Early testing had confirmed that sediment fines would be transported through the PCPS to be deposited, either on filter fabric or at the bottom of a PCPS without filter fabric, as a layer between the PCPS and the underlying subgrade. The presence of such a layer could affect exfiltration characteristics of the PCPS and therefore play an important role in overall hydrological behavior, PCPC serviceability and PCPS serviceability. A simple test method was needed to estimate the exfiltration rate after the design sediment load has been applied. This section describes the test method used in this study.

![Figure 3.13. Sediment Hydraulic Conductivity Test Setup](image)

a)  
b)
The permeability of a layer of finer soil particles corresponding to the clay and silt fractions was measured using a simple test method that could be conducted on site or in a laboratory. The procedure was similar to the sedimentation test method described in section 3.5.4. The values obtained were compared to permeability results from other parts of this study. The values were also used in developing design guidelines including the potential changes of the exfiltration rate of the system during the service life.

The testing procedure was performed in three different test setups: 1) including only a layer of sediments at the bottom; 2) including a stone base in the system; and 3) including a layer of sand at the bottom of the pavement to act as a filter for the smaller particles. Filter fabric was attached to the bottom of a 4 in. (100 mm) diameter PVC riser pipe 8.0 in. (200 mm) long for all three setups.

The same procedures were followed in all three test setups. A total of 0.22 lb (0.1 kg) of sediment was mixed with 5 gallon (18.9 L) of water added to the pipe at different times, and the permeability was then determined. The flow rate started with a high flow, reducing the rate while sediments deposited on the filter fabric. The short permeameter used in the sedimentation test, 7.5 in. (190 mm) long, was attached to the top of the PVC pipe to determine the permeability of the sediments. The permeameter and the specimen were kept vertical using the same frame used in the permeability test. The sediments were left to air dry for a minimum of 24 hours to let the sediments to settle, and the water to completely flow out of the system. The coefficient of permeability was then determined using Equation 2.2.
For those tests including stone base, a layer 6 in. (150 mm) in depth of stone with a nominal maximum size of 3/4 in. (19 mm) was added inside the pipe. For those tests including sand, a layer 1 in. (25 mm) in depth of medium to fine sand was added inside the pipe on top of the filter fabric.

3.7. Frost Durability

Frost resistance testing using ASTM C 666 is considered severe. The specimens are typically completely immersed in water and exposed to rapid, unrealistically high freezing rates. Evaluation is based on changes in dynamic modulus over at least 300 cycles.

ASTM C 672 is an alternative method intended to evaluate deicer salt scaling of pavements. A slab specimen is ponded with a 1% calcium chloride and subjected to realistic freezing rates. Frost resistance is evaluated based on the visually extent of surface scaling after 50 cycles. The mass of scaled material is also often measured.

An adaptation of ASTM C 672 was used to evaluate frost resistance of 4 in. (100 mm) diameter by approximately 1 in. (25 mm) thick concrete disks. Evaluation was based on mass loss and changes in dynamic modulus. The results of this study indicate that small disks, tested at realistic freezing rates, could be successfully used to evaluate frost resistance of paving mixtures. This method had been used in other studies (Dilek, 2000).
This adaptation was selected for this study for several reasons including the use of freezing rates similar to real exposure, avoiding unrealistic high freezing rates and the convenient sample size.

The disk specimens were partially immersed in a solution containing 1% by mass of calcium chloride. The specimens were placed in a pan over three (3) or four (4) 0.75 in. (19 mm) stones to allow the solution to penetrate the bottom of the specimens (Figure 3.15). The pan was then placed in a freezing environment at 0 ± 5°F (-18 ± 3°C) for 16 to 18 hours. At the end of this time the specimens were taken out of the freezing environment and placed in laboratory air at 73.5 ± 3.5°F (23 ± 2°C), similar to the procedure given in ASTM C 672. The procedure was repeated daily, adding calcium chloride solution as necessary to maintain the solution level at approximately half the depth of the disk specimens. The solution was changed after every ten cycles. The specimens were kept frozen during any interruption in the daily cycle, except for testing as described below.

Four (4) disk specimens from Mixture 1, Mixture 2, Mixture 4 and Mixture 5 for a total of sixteen (16) were tested for frost durability. Specimens from Mixture 1 and 2 were sawn from the top portion of cylindrical specimens and disks from Mixture 4 and 5 were produced as described in Section 3.3.2. The mass and dynamic modulus using the fundamental cyclic natural frequency were determined for each specimen before the first cycle and after every
ten cycles. The specimens were left to air dry for 24 hours in lab air not exceeding 100°F (40°C) prior to testing.

One (1) disk from Mixture 4 and Mixture 5 were tested as described previously with the addition of sand sediments infiltrating the disk. The mass and $E_d$ were determined after 10 cycles. One (1) disk specimen from each of the selected mixtures was examined under a stereoscopic microscope with 50x and 100x lenses to qualitatively assess the entrained air system in the specimen.

Since only a small area of the paste was available, it was not possible to conduct analysis in accordance with ASTM C 457. The entrained air void system, however, could be interpreted as “inadequate,” “acceptable,” or “excellent” by an experience analyst. The specimen were prepared by a lightly grinding that would allow the qualitative examination.
Figure 3.14. Test Setup for Frost Durability Test.
Chapter 4. Sedimentation Results and Discussion

The results of the sedimentation tests, that is, results of Phase I and Phase II are reviewed first in this chapter. The results of frost resistance, Phase III, which comprised a limited part of this study, are presented in Chapter 6.

4.1. General

The results and analysis of the sedimentation tests for Phase I and Phase II using the one-dimensional sedimentation test, and the two-dimensional sedimentation test are shown and discussed. In general, the vertical distribution and segregation of sediments, and the horizontal distribution of the fine grained soil sediments anticipated \textit{a priori} was confirmed: the larger grain soil sediments were trapped in the top inches of the pavement while the fine grained soil sediments were deposited at the bottom, forming a uniformly distributed layer at the bottom.

Hydrologic properties of the concrete, including porosity and permeability, were determined. Porosity for Mixture 3 was slightly higher than Mixture 1 and Mixture 2. Mixtures 4 and 5 showed the lowest porosity due to the sand content. Permeability of the mixtures 1, 2 and 3 were comparable while mixtures 4 and 5 showed lower permeability than the other three mixtures.
Mechanical properties of the pervious concrete were not critical to this study, however, modulus of rupture, dynamic modulus of elasticity, compressive strength, tensile strength, were determined, and the information is shown in Appendix A for completeness.

4.2. Hydrological Properties of the Pervious Concrete

The specimen identification code for all the properties follows the following designation: a) first letter corresponds to the specimen type, c for cylinder, b for beams, and d for disks; b) first number corresponds to the concrete Mixture the specimen was fabricated; and c) second number correspond to the specimen identification for that mixture. For example, beam specimen number 6 from Mixture 3 is designated as B-3-6.

4.2.2. Porosity

The porosity for the cylinder specimens from Mixture 1 and Mixture 2 determined by Equation 3.1, the unit weight approach, or as described in ASTM D 7063, are shown in Table 4.1. The results show a slightly higher porosity for Mixture 2, about 18.9%, than Mixture 1, about 21.1%. The average porosity of both mixtures when evaluated together is about 20.0% with a standard deviation of 3.9%. The difference between the porosity determined by the unit weight approach and ASTM D 7063 was positive in some cases and negative in others. No indication of bias was found with these specimens but data were limited.

The porosity of the beam specimens from Mixture 1 and Mixture 2 determined by the unit weight difference are shown in Table 4.2. The average porosities were 22.0% and 24.2% for
mixtures 1 and 2 respectively, approximately 2.0% to 4.0% higher than the porosities of companion cylinders of each mixture. Similarly to the cylinder analysis, differences in porosity between mixtures is not statistically significant. The average of these specimens is 23.0% with an standard deviation of 3.3%. 
Table 4.1. Dry Density, Porosity, and Hydraulic Conductivity Coefficient for Mixture 1 and Mixture 2 Cylinder Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Height, in. (cm)</th>
<th>Dry density, lb/ft$^3$ (kg/m$^3$)</th>
<th>Porosity, % (diff. weights)</th>
<th>Porosity, % (ASTM D 7063)</th>
<th>Hydraulic Conductivity Coefficient k, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mixture 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-1-1</td>
<td>5.6 (14.3)</td>
<td>118.23 (1915.39)</td>
<td>10.4</td>
<td>-</td>
<td>970 (0.69)</td>
</tr>
<tr>
<td>C-1-2</td>
<td>5.2 (13.3)</td>
<td>112.72 (1826.05)</td>
<td>22.9</td>
<td>-</td>
<td>910 (0.65)</td>
</tr>
<tr>
<td>C-1-3</td>
<td>5.2 (13.3)</td>
<td>114.75 (1858.97)</td>
<td>21.7</td>
<td>-</td>
<td>890 (0.64)</td>
</tr>
<tr>
<td>C-1-4</td>
<td>5.8 (14.6)</td>
<td>111.38 (1804.37)</td>
<td>24.1</td>
<td>-</td>
<td>1030 (0.73)</td>
</tr>
<tr>
<td>C-1-5</td>
<td>5.3 (13.4)</td>
<td>113.97 (1846.31)</td>
<td>22.3</td>
<td>-</td>
<td>920 (0.66)</td>
</tr>
<tr>
<td>C-1-6</td>
<td>5.8 (14.6)</td>
<td>116.02 (1879.46)</td>
<td>13.5</td>
<td>-</td>
<td>980 (0.70)</td>
</tr>
<tr>
<td>C-1-7</td>
<td>5.4 (13.7)</td>
<td>114.70 (1858.08)</td>
<td>14.2</td>
<td>-</td>
<td>1000 (0.71)</td>
</tr>
<tr>
<td>C-1-8</td>
<td>5.8 (14.8)</td>
<td>114.70 (1858.08)</td>
<td>21.9</td>
<td>20.8</td>
<td>-</td>
</tr>
<tr>
<td>C-1-9</td>
<td>5.8 (14.8)</td>
<td>118.05 (1924.27)</td>
<td>18.9</td>
<td>19.9</td>
<td>-</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td><strong>Std. deviation</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mixture 2</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-2-1</td>
<td>6.1 (15.4)</td>
<td>115.16 (1865.56)</td>
<td>22.4</td>
<td>-</td>
<td>1020 (0.72)</td>
</tr>
<tr>
<td>C-2-2</td>
<td>5.8 (14.6)</td>
<td>115.14 (1865.21)</td>
<td>22.1</td>
<td>-</td>
<td>1020 (0.72)</td>
</tr>
<tr>
<td>C-2-3</td>
<td>5.3 (13.4)</td>
<td>122.78 (1989.06)</td>
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<td>-</td>
<td>1230 (0.87)</td>
</tr>
<tr>
<td>C-2-4</td>
<td>5.5 (13.9)</td>
<td>112.63 (1824.61)</td>
<td>23.4</td>
<td>-</td>
<td>1440 (1.02)</td>
</tr>
<tr>
<td>C-2-5</td>
<td>5.9 (14.9)</td>
<td>115.04 (1863.63)</td>
<td>22.3</td>
<td>-</td>
<td>1370 (0.97)</td>
</tr>
<tr>
<td>C-2-6</td>
<td>5.8 (14.6)</td>
<td>118.86 (1925.50)</td>
<td>19.6</td>
<td>-</td>
<td>830 (0.59)</td>
</tr>
<tr>
<td>C-2-7</td>
<td>5.5 (13.9)</td>
<td>115.47 (1870.57)</td>
<td>22.6</td>
<td>-</td>
<td>1360 (0.97)</td>
</tr>
<tr>
<td>C-2-12</td>
<td>5.9 (15.0)</td>
<td>114.87 (1860.97)</td>
<td>22.3</td>
<td>23.4</td>
<td>-</td>
</tr>
<tr>
<td>C-2-13</td>
<td>5.9 (15.0)</td>
<td>120.80 (1956.96)</td>
<td>18.4</td>
<td>16.5</td>
<td>-</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Std. deviation</strong></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4.2. Dry Density, Porosity, and Hydraulic Conductivity for Mixture 1 and Mixture 2 Beam Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dry density, lb/ft³ (kg/m³)</th>
<th>Porosity, % (diff. weights)</th>
<th>Flow Rate, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1-1</td>
<td>116.91 (1893.97)</td>
<td>18.5</td>
<td>-</td>
</tr>
<tr>
<td>B-1-2</td>
<td>115.59 (1872.57)</td>
<td>17.5</td>
<td>830 (0.59)</td>
</tr>
<tr>
<td>B-1-3</td>
<td>113.44 (1837.77)</td>
<td>23.2</td>
<td>1800 (1.28)</td>
</tr>
<tr>
<td>B-1-4</td>
<td>115.40 (1869.42)</td>
<td>22.6</td>
<td>900 (0.64)</td>
</tr>
<tr>
<td>B-1-5</td>
<td>111.17 (1800.89)</td>
<td>26.0</td>
<td>1200 (0.85)</td>
</tr>
<tr>
<td>B-1-6</td>
<td>110.56 (1791.01)</td>
<td>27.4</td>
<td>1080 (0.77)</td>
</tr>
<tr>
<td>B-1-7</td>
<td>116.45 (1886.54)</td>
<td>18.7</td>
<td>1800 (1.28)</td>
</tr>
<tr>
<td>Average</td>
<td>114.44 (1850.31)</td>
<td>22.0</td>
<td>1270 (0.90)</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>2.54 (41.22)</td>
<td>3.8</td>
<td>430 (0.31)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Flow rate, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1-1</td>
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<tr>
<td>B-1-2</td>
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<tr>
<td>B-1-3</td>
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<td>B-1-4</td>
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<td>B-1-6</td>
<td></td>
</tr>
<tr>
<td>B-1-7</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td></td>
</tr>
<tr>
<td>Std. deviation</td>
<td>2.35 (38.08)</td>
</tr>
</tbody>
</table>

Table 4.3. Flow Rate for Mixture 3 Beam Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Flow rate, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3-1</td>
<td>1200 (0.85)</td>
</tr>
<tr>
<td>B-3-2</td>
<td>1200 (0.85)</td>
</tr>
<tr>
<td>B-3-3</td>
<td>3600 (2.56)</td>
</tr>
<tr>
<td>B-3-4</td>
<td>2700 (1.92)</td>
</tr>
<tr>
<td>B-3-5</td>
<td>3600 (2.56)</td>
</tr>
<tr>
<td>B-3-6</td>
<td>2700 (1.92)</td>
</tr>
<tr>
<td>Average</td>
<td>2500 (1.78)</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>1080 (0.86)</td>
</tr>
</tbody>
</table>
The lower porosity of the cylinders is consistent with greater compaction energy. The energy provided by the proctor hammer combined with the restrain of the steel cylinder used to fabricate the specimens appears to be higher than that applied by rolling the beam surface as described in Section 3.3.2. This finding has important implications in pavement design and quality control, but that topic is outside the scope of this study.

Table 4.4 shows the porosity of the cylinder specimens from the two batches of Mixture 3. The porosity of these specimens were determined by the unit weight approach and in accordance with ASTM D 7063. The porosity determined with the unit weight approach shows a slightly higher porosity than that determined with ASTM D 7063, approximately 1.5% high. The average porosity combining both batches for the method using Equation 3.1 and ASTM D 7063 are 26.8% with a standard deviation of 3.9%, and 23.4% with a standard deviation of 3.2%, respectively. The difference of about 3.4% is very likely to be the non-interconnected pores that are not considered in ASTM D 7063, which determines the effective porosity. Effective porosity should be less than the total percent of air voids.

The difference between the unit weight and ASTM D 7063 is almost always positive, indicating a bias and therefore a real difference between the two test methods exist, even though the values are relatively close. The difference is especially noticeable with the specimens from Batch 2, in which a consistent difference of approximately 5.0% was found in most specimens.
Table 4.4. Dry Density, Porosities (Unit Weight and ASTM D 7063), and Hydraulic Conductivity Coefficients (16 in. long and 7in. long permeameter) for Mixture 3 Cylinder Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Height, in. (cm)</th>
<th>Dry density, lbs/ft³ (kg/m³)</th>
<th>Porosity, % (diff. weights)</th>
<th>Porosity, % (ASTM D 7063)</th>
<th>Hydraulic Conductivity Coefficient (k), in./hr (cm/s) [16 in. pipe]</th>
<th>Hydraulic Conductivity Coefficient (k), in./hr (cm/s) [7 in. pipe]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Batch 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-3-1</td>
<td>5.9 (15.0)</td>
<td>116.57 (1888.37)</td>
<td>26.6</td>
<td>25.3</td>
<td>1550 (1.10)</td>
<td>1450 (1.03)</td>
</tr>
<tr>
<td>C-3-2</td>
<td>5.6 (14.2)</td>
<td>120.39 (1950.27)</td>
<td>23.8</td>
<td>23.6</td>
<td>1460 (1.04)</td>
<td>1370 (0.97)</td>
</tr>
<tr>
<td>C-3-3</td>
<td>6.1 (15.4)</td>
<td>119.77 (1940.30)</td>
<td>23.8</td>
<td>18.3</td>
<td>1420 (1.01)</td>
<td>1500 (1.07)</td>
</tr>
<tr>
<td>C-3-4</td>
<td>5.5 (13.9)</td>
<td>116.33 (1884.59)</td>
<td>24.9</td>
<td>25.9</td>
<td>1770 (1.26)</td>
<td>1690 (1.20)</td>
</tr>
<tr>
<td>C-3-5</td>
<td>5.6 (14.1)</td>
<td>123.20 (1995.77)</td>
<td>20.4</td>
<td>22.0</td>
<td>1540 (1.09)</td>
<td>1230 (0.88)</td>
</tr>
<tr>
<td>C-3-6</td>
<td></td>
<td>122.13 (1978.51)</td>
<td>22.0</td>
<td>16.4</td>
<td>1300 (0.92)</td>
<td>1250 (0.89)</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>119.73 (1939.63)</td>
<td>23.6</td>
<td>21.9</td>
<td>1510 (1.07)</td>
<td>1420 (1.01)</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>2.82 (45.68)</td>
<td>2.2</td>
<td>3.8</td>
<td>160 (0.11)</td>
<td>170 (0.12)</td>
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</tr>
<tr>
<td>Batch 2</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>C-3-7</td>
<td>5.4 (13.7)</td>
<td>110.02 (1782.29)</td>
<td>32.6</td>
<td>26.5</td>
<td>3230 (2.29)</td>
<td>3320 (2.36)</td>
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<tr>
<td>C-3-8</td>
<td>5.9 (14.9)</td>
<td>113.48 (1838.36)</td>
<td>29.6</td>
<td>24.4</td>
<td>2730 (2.29)</td>
<td>2400 (1.71)</td>
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<tr>
<td>C-3-9</td>
<td>5.2 (13.3)</td>
<td>112.26 (1818.65)</td>
<td>29.9</td>
<td>24.5</td>
<td>2430 (1.73)</td>
<td>2500 (1.83)</td>
</tr>
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<td>C-3-10</td>
<td>5.8 (14.7)</td>
<td>111.00 (1798.16)</td>
<td>31.6</td>
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<td>3050 (1.16)</td>
<td>2870 (2.04)</td>
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<td>2920 (2.07)</td>
<td>2750 (1.95)</td>
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<tr>
<td>C-3-12</td>
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<td>113.08 (1831.93)</td>
<td>29.7</td>
<td>25.1</td>
<td>2850 (2.02)</td>
<td>2680 (1.90)</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>112.17 (1817.14)</td>
<td>30.1</td>
<td>24.8</td>
<td>2870 (2.04)</td>
<td>2770 (1.96)</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>1.38 (22.41)</td>
<td>1.9</td>
<td>1.6</td>
<td>270 (0.19)</td>
<td>310 (0.22)</td>
<td></td>
</tr>
</tbody>
</table>
This bias implies that some pores are not being filled with water during the ASTM D 7063 testing. The bias found with cylindrical specimens and the apparent lack of bias with beams is consistent with the findings related to compaction energy. The proctor hammer appears to be forming a larger number of discontinuous pores in the pervious concrete. Due to the very small differences in porosity found, the apparent bias should have no meaningful effect on sedimentation results on this study since the volume of discontinuous pores appears to be fairly small, but additional study is strongly recommended.

4.2.3. Permeability

Table 4.1 shows the coefficient of permeability for Mixture 1 and Mixture 2 cylinder specimens. Mixture 2 showed a slightly higher permeability than Mixture 1, with a difference of around 200 in./hr (0.16 cm/s). A pooled average of 1070 in./hr (0.97 cm/s) with a standard deviation of 200 in./hr (0.16 cm/s) was calculated using the specimens from both mixtures.

The lower permeability of Mixture 1 is consistent with the lower porosity. Although the hydraulic conductivities are not statically significantly different, analysis of sedimentation was initially conducted considering both separate mixtures and pooled values of Mixture 1 and Mixture 2.

The effects of slight differences in porosity and permeability were further explored with results using Mixture 3 specimens. The effects of differences in porosity and permeability on
sedimentation results do not appear to be very large for a wide range of porosities of pervious concrete found in practice, but these effects should be investigated with a wider range of materials.

The coefficient of permeability for Mixture 3, Batch 1 and Batch 2, are shown in Table 4.4. Permeability of Batch 1 specimens was similar to those specimens from Mixture 1 and Mixture 2, however, Batch 2 specimens show a considerable higher permeability than all the others, around twice that of Batch 1. The mixing, mixture, fabrication, and curing were the same for both batches and the reasons for the differences found are not known. For practical purposes, the permeabilities are well within the ranges normally encountered in service (ACI 522, 2006).

The coefficient of permeability was also determined in Mixture 3 specimens using the small permeameter. The results, shown in Table 4.4, were similar to those obtained with the large permeameter, showing, as expected, no significant difference between permeameter height for the permeameter configurations used in this study.

Flow rate for Mixture 1 and Mixture 2 beam specimens are shown in Table 4.2. A pooled apparent average flow rate of 1140 in./hr (0.91 cm/s) with a standard deviation of 380 in./hr (0.30 cm/s) was calculated using specimens from both mixtures. Flow rate for Mixture 3 beam specimens are shown in Table 4.3.
Water flowing out of the open end of the beam was observed while performing the flow rate tests before and after sedimentation in the standard 20 in. (500 mm) long and the 40 in. (1000 mm) long specimens. This end of the beam was not blocked or covered with the heavy adhesive. The filter fabric at the bottom of the specimens was found to retain a minimum head of water before flow. Although only minimal water was observed to be retained, the process is not immediate and the flow rate through the fabric is lower than through the pervious concrete. This finding is consistent with concerns discussed in ACI 522 committee meetings and implies that full recovery of the storage capacity may not be possible. This also implies that sediments deposited on the filter fabric may not fully dry in practice.

4.3. Sedimentation

In preliminary analysis, changes in permeability and flow rate due to sedimentation were used directly, and graphical representations were very difficult to interpret. Using percentages of initial permeability for each specimen allows much easier comparisons and identifications of trends. The discussion and graphical representation of the experimental findings of the effects of sedimentation are therefore presented using percent of initial permeability. It is recognized that major differences in porosity are likely to result in differences in void size distribution, which could result in different findings regarding sedimentation with substantially different pervious concrete porosities. The porosities and initial hydraulic conductivities of the specimens used in this study are believed to be
sufficiently close to those in practice that comparing percents of initial permeability provide
the most useful method of analysis.

Additionally, the findings of this study are therefore limited to PCPS in which the porosity of
the pervious concrete is about 20%. Pervious concrete with significantly different porosities,
such as 10%, or 30%, should be evaluated.

4.3.1. Phase I: Low Flow with High Sediment Load

The effects of the different types of sediment on the coefficients of permeability of cylinder
specimens are shown in Table 4.5. The cylinder specimens subjected to either sand, clayey
silt or clayey silty sand sediments are shown with the respective coefficient of permeability
determined initially, after sedimentation, after washing, and after removal of the filter fabric.
The remaining permeability in percentage after each stage are shown in Table 4.6.

The effects of the different type of sediments on the flow rate of beam specimens are shown
in Table 4.7. The beam specimens subjected to either sand, clayey silt or clayey sand
sediments are shown with the respective flow rate determined with no sediment, after
sedimentation, after washing, and removal of the filter fabric. The remaining flow rate in
percentages of initial after each stage is also shown in Table 4.7.
Table 4.5. Changes in Coefficient of Permeability in Cylinders at Different Stages of the Phase I Sedimentation Test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Initial coefficient of permeability $k$, in./hr (cm/s)</th>
<th>Coefficient of permeability $k$ after sedimentation, in./hr (cm/s)</th>
<th>Coefficient of permeability $k$ after washing, in./hr (cm/s)</th>
<th>Coefficient of permeability $k$ after removing fabric, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-1-1</td>
<td>Sand</td>
<td>970 (0.69)</td>
<td>110 (0.08)</td>
<td>210 (0.15)</td>
<td>690 (0.49)</td>
</tr>
<tr>
<td>C-1-2</td>
<td>Sand</td>
<td>910 (0.65)</td>
<td>270 (0.19)</td>
<td>610 (0.43)</td>
<td>680 (0.49)</td>
</tr>
<tr>
<td>C-2-3</td>
<td>Sand</td>
<td>1230 (0.87)</td>
<td>40 (0.03)</td>
<td>80 (0.06)</td>
<td>840 (0.59)</td>
</tr>
<tr>
<td>C-2-4</td>
<td>Sand</td>
<td>1440 (1.02)</td>
<td>40 (0.03)</td>
<td>240 (0.17)</td>
<td>920 (0.66)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-1-3</td>
<td>Clayey silt</td>
<td>890 (0.64)</td>
<td>20 (0.02)</td>
<td>80 (0.05)</td>
<td>800 (0.56)</td>
</tr>
<tr>
<td>C-1-4</td>
<td>Clayey silt</td>
<td>1030 (0.73)</td>
<td>10 (0.01)</td>
<td>20 (0.02)</td>
<td>720 (0.51)</td>
</tr>
<tr>
<td>C-2-1</td>
<td>Clayey silt</td>
<td>1020 (0.72)</td>
<td>10 (0.01)</td>
<td>40 (0.03)</td>
<td>690 (0.49)</td>
</tr>
<tr>
<td>C-2-2</td>
<td>Clayey silt</td>
<td>1020 (0.72)</td>
<td>30 (0.02)</td>
<td>60 (0.04)</td>
<td>770 (0.54)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-1-5</td>
<td>Clayey silty sand</td>
<td>920 (0.66)</td>
<td>370 (0.26)</td>
<td>500 (0.36)</td>
<td>620 (0.44)</td>
</tr>
<tr>
<td>C-1-6</td>
<td>Clayey silty sand</td>
<td>980 (0.70)</td>
<td>5 (0.01)</td>
<td>530 (0.38)</td>
<td>660 (0.47)</td>
</tr>
<tr>
<td>C-2-5</td>
<td>Clayey silty sand</td>
<td>830 (0.59)</td>
<td>10 (0.01)</td>
<td>770 (0.55)</td>
<td>1030 (0.73)</td>
</tr>
<tr>
<td>C-2-6</td>
<td>Clayey silty sand</td>
<td>1360 (0.97)</td>
<td>30 (0.02)</td>
<td>60 (0.04)</td>
<td>480 (0.34)</td>
</tr>
<tr>
<td>Specimen</td>
<td>Sediment type</td>
<td>Remaining permeability after sedimentation, %</td>
<td>Remaining permeability after washing, %</td>
<td>Remaining permeability after removing fabric, %</td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>---------------</td>
<td>-----------------------------------------------</td>
<td>------------------------------------------</td>
<td>-----------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>C-1-1</td>
<td>Sand</td>
<td>11%</td>
<td>21%</td>
<td>71%</td>
<td></td>
</tr>
<tr>
<td>C-1-2</td>
<td>Sand</td>
<td>30%</td>
<td>67%</td>
<td>75%</td>
<td></td>
</tr>
<tr>
<td>C-2-3</td>
<td>Sand</td>
<td>4%</td>
<td>6%</td>
<td>68%</td>
<td></td>
</tr>
<tr>
<td>C-2-4</td>
<td>Sand</td>
<td>3%</td>
<td>17%</td>
<td>64%</td>
<td></td>
</tr>
<tr>
<td>C-1-3</td>
<td>Clayey silt</td>
<td>2%</td>
<td>8%</td>
<td>89%</td>
<td></td>
</tr>
<tr>
<td>C-1-4</td>
<td>Clayey silt</td>
<td>1%</td>
<td>2%</td>
<td>70%</td>
<td></td>
</tr>
<tr>
<td>C-2-1</td>
<td>Clayey silt</td>
<td>1%</td>
<td>4%</td>
<td>68%</td>
<td></td>
</tr>
<tr>
<td>C-2-2</td>
<td>Clayey silt</td>
<td>3%</td>
<td>6%</td>
<td>75%</td>
<td></td>
</tr>
<tr>
<td>C-1-5</td>
<td>Clayey silty sand</td>
<td>40%</td>
<td>55%</td>
<td>67%</td>
<td></td>
</tr>
<tr>
<td>C-1-6</td>
<td>Clayey silty sand</td>
<td>1%</td>
<td>54%</td>
<td>68%</td>
<td></td>
</tr>
<tr>
<td>C-2-5</td>
<td>Clayey silty sand</td>
<td>1%</td>
<td>56%</td>
<td>75%</td>
<td></td>
</tr>
<tr>
<td>C-2-6</td>
<td>Clayey silty sand</td>
<td>3%</td>
<td>7%</td>
<td>58%</td>
<td></td>
</tr>
</tbody>
</table>
Table 4.7. Changes in Flow Rate in Beams at Different Stages of the Phase I Sedimentation Test, and Remaining Flow Rate in Beams at Different Stages of the Phase I Sedimentation Test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Initial flow rate, in./hr (cm/s)</th>
<th>Flow rate after sedimentation and washing, in./hr (cm/s)</th>
<th>Flow rate after removing fabric, in./hr (cm/s)</th>
<th>Remaining flow rate after sedimentation and washing, %</th>
<th>Remaining flow rate after removing fabric, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1-4</td>
<td>Sand</td>
<td>900 (0.64)</td>
<td>720 (0.51)</td>
<td>720 (0.51)</td>
<td>80%</td>
<td>80%</td>
</tr>
<tr>
<td>B-1-5</td>
<td>Sand</td>
<td>1200 (0.85)</td>
<td>1080 (0.77)</td>
<td>1080 (0.77)</td>
<td>90%</td>
<td>90%</td>
</tr>
<tr>
<td>B-2-2</td>
<td>Sand</td>
<td>1080 (0.77)</td>
<td>720 (0.51)</td>
<td>900 (0.64)</td>
<td>67%</td>
<td>83%</td>
</tr>
<tr>
<td>B-2-7</td>
<td>Sand</td>
<td>680 (0.48)</td>
<td>280 (0.20)</td>
<td>280 (0.20)</td>
<td>42%</td>
<td>42%</td>
</tr>
<tr>
<td>B-1-2</td>
<td>Clayey silt</td>
<td>830 (0.59)</td>
<td>430 (0.31)</td>
<td>450 (0.32)</td>
<td>52%</td>
<td>54%</td>
</tr>
<tr>
<td>B-1-3</td>
<td>Clayey silt</td>
<td>1800 (1.28)</td>
<td>720 (0.51)</td>
<td>720 (0.51)</td>
<td>40%</td>
<td>40%</td>
</tr>
<tr>
<td>B-2-5</td>
<td>Clayey silt</td>
<td>-</td>
<td>830 (0.59)</td>
<td>900 (0.64)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B-2-6</td>
<td>Clayey silt</td>
<td>720 (0.51)</td>
<td>630 (0.45)</td>
<td>680 (0.48)</td>
<td>88%</td>
<td>94%</td>
</tr>
<tr>
<td>B-1-6</td>
<td>Clayey silty sand</td>
<td>1080 (0.77)</td>
<td>1350 (0.96)</td>
<td>1350 (0.96)</td>
<td>100%</td>
<td>100%</td>
</tr>
<tr>
<td>B-1-7</td>
<td>Clayey silty sand</td>
<td>1800 (1.28)</td>
<td>1200 (0.85)</td>
<td>1350 (0.96)</td>
<td>67%</td>
<td>75%</td>
</tr>
<tr>
<td>B-2-3</td>
<td>Clayey silty sand</td>
<td>1080 (0.77)</td>
<td>1080 (0.77)</td>
<td>1080 (0.77)</td>
<td>100%</td>
<td>100%</td>
</tr>
<tr>
<td>B-2-4</td>
<td>Clayey silty sand</td>
<td>1540 (1.10)</td>
<td>1350 (0.96)</td>
<td>1540 (1.10)</td>
<td>88%</td>
<td>100%</td>
</tr>
</tbody>
</table>
4.3.1.1. Sand Sediments

The changes in permeability of the cylinder specimens subjected to sand sediments at the four different stages of the test procedure are shown in Figure 4.1. Three specimens lost around 90% of the initial permeability while one specimen, C-1-2 lost about 70%. The permeability recovery of the most affected specimens did not show a significant increase after the washing procedure, reaching, at the most, around 20% of the initial permeability. The permeability of the four specimens showed a recovery around 70% of the initial value after the removal of the filter fabric at the bottom of the specimens.

As expected, a significant quantity of sand was observed at the top of the specimens after the sedimentation test, which was completely removed from the top by washing, as shown in Figure 4.2. Visual observation after the removal of the filter fabric showed only a few sparsely distributed particles of finer sand (see Figure 4.3).
Figure 4.1. Remaining Coefficient of Permeability of Cylinder Specimens Subjected to Sand Sediments in Phase I
Figure 4.2. Cylinder Specimen Exposed to Sand Sediments in Phase I

Figure 4.3. Cylinder Specimen After Exposure to Sand
The permeability recovery of the specimens seems to indicate that the finer sand particles entered the specimens and most were deposited within the concrete with only small particles passing through, retained by the filter fabric. This observation is consistent with the size of the finer particles of sand, between about 0.08 in. (2 mm) and the #200 sieve size (75 µm), and the void size of the pervious concrete ranging from 0.08 to 0.32 in. (2 to 8 mm) [ACI 2006]. The recovery of the specimens after the removal of the filter fabric seems to indicate that most of these finer sand particles were retained by the fabric and were free to flow out of the specimen once it was removed, however, no significant amount of the particles were found deposited at the bottom. Many of the coarser particles were trapped on or near the surface and could be removed by washing. This finding is consistent with ACI report (2006) and Haselbach, Valavala, and Montes (2006). Some clogging occurs internal within the concrete and some clogging also occurs in the filter fabric, however, permeability is still much higher than 1 in./hr (2.5 cm/hr).

The change in flow rate of the beam specimens subjected to sand sediments at the three different stages of the test procedure is shown in Figure 4.4. The beam specimens did not seem to lose as much permeability as companion cylinder after washing although results are scattered. Specimen B-2-7 showed the higher loss of flow rate, remaining approximately 40% of the initial value while the other specimens recover up to 70% to 90% of their initial flow rate. Water flowing at the end of the beam was observed while performing the flow rate tests before and after sedimentation in all of the specimens. The removal of the filter fabric did not show any significant difference in the flow rate of the beam specimens.
A significant quantity of sand was observed at the top of the beam specimens after the sedimentation test, which was removed from the top by washing. Visual observation of the bottom when removing the filter fabric did not find many as with cylindrical specimens, only a few sparsely distributed grains of sand were observed (see Figure 4.5).

The loss of flow rate of the beams, especially specimen B-2-7, and the cylinders seem to indicate that some fraction of the finer sand grains penetrated the specimen and was deposited within the concrete, while the larger fractions on the sand was deposited on top of the specimen. Penetration all the way through the specimen was limited, however, the effect of this deposition will not affect the exfiltration of the system due to the lower infiltration of the sub-base.
Figure 4.4. Remaining Flow Rate of Beam Specimens Subjected to Sand Sediments in Phase I
Figure 4.5. Beam Specimens After Exposure to Sand
4.3.1.2. Clayey Silty Sediments

The changes in permeability of the cylinder specimens subjected to clayey silt sediments at the four different stages of the test procedure are shown in Figure 4.6. The four specimens used in this test showed similar results with each other, losing about 95% of the initial permeability. The permeability recovered after washing was less than 10% of the initial value for all specimens, however permeability recovered to about 70% of the initial permeability when the filter fabric was removed.

A significant quantity of clayey silt was observed at the top of the specimens after the sedimentation test. This material was completely removed from the surface after washing (see Figure 4.7) based on visual observations. Visual observation of the filter fabric during removal found a significant amount of clayey silt sediments deposited between the specimen and the fabric (see Figure 4.8).

These observations and test values, particularly the permeability recovery of the specimens, indicated that the clayey silt sediments penetrated the pervious concrete and were deposited for the most part at the bottom of the specimen, retained by the filter fabric. The void sizes of the concrete, ranging from 0.08 to 0.32 in. (2 to 8 mm), are certainly large enough to allow the sediments of clay and silt smaller than the #200 sieve size (75 µm), to be transported within the specimen. The deposition on the bottom is consistent with a largely interconnected void system. The recovery of the specimens after the removal of the filter fabric indicates that most of these particles were retained by the fabric and were free to flow out of the
specimen once it was removed. Recovery was not complete, indicating that some particles were trapped within the concrete.

The change in flow rate of the beam specimens subjected to clayey silt sediments at the three different stages of the test procedure is shown in Figure 4.9. As with the sand sediments, the beam specimens did not lose as much permeability as companion cylinder specimens after washing. Specimens B-1-2 and B-1-3 showed a higher loss of flow rate, retaining approximately 45% of the initial value. The third specimen recovered over 90% of its initial flow rate. The removal of the filter fabric did not show any significant difference in the flow rate of the beam specimens. Water flowing at the end of the beam was observed while performing the flow rate tests before and after sedimentation in all the specimens.

A significant quantity of clayey silt was observed at the top of the specimens after the sedimentation test. This material was completely removed from the surface after washing. Visual observation the bottom of the beam during removal of the filter fabric showed a significant amount of clayey silt sediments deposited uniformly at the bottom of all three beams (see Figure 4.10), similar to the observation with the cylinder specimens.
Figure 4.6. Remaining Coefficient of Permeability of Cylinder Specimens Subjected to Clayey Silty Sediments in Phase I
Figure 4.7. Cylinder Specimen Exposed to Clayey Silty Sediments in Phase I

Figure 4.8. Cylinder Specimen After Exposure to Clayey Silt
Figure 4.9. Remaining Flow Rate of Beam Specimens Subjected to Clayey Silty Sediments in Phase I
Figure 4.10. Beam Specimen After Exposure to Clayey Silt
4.3.1.3. Clayey Silty Sand Sediments

The changes in permeability of the cylinder specimens subjected to clayey silty sand sediments at the four different stages of the test procedure are shown in Figure 4.11. Three specimens lost more than 90% of the initial permeability after sedimentation while the remaining specimen, C-1-5, kept about 40%. Washing had mixed results, showing definite improvement in three specimens that recovered 50% and one specimen, C-2-6 that did not show a significant recovery. All four specimens recovered permeability after the removal of the filter fabric. Recovery ranged between almost 60% to well over 70% of the initial permeability.

A significant quantity of sand with a fine layer of the clayey silt sediments was observed on the surface of the cylinder after the sedimentation test. This material was completely removed from the top after washing (see Figure 4.12). Visual observation at the moment of the filter fabric removal did not show a significant amount of sediments deposited at the bottom, as shown in Figure 4.13, very similar to the observation with the sand sediment sedimentation test.

The permeability recovery seems to indicate that most of the soil particles did not enter the specimen but were deposited in the top few inches of the surface and then removed by washing. Some finer particles also penetrate deeper into the C-2-4 specimen, however. The test results in this specimen was reasonably similar to the test with sand sediments: the specimens recovered a good percentage of the initial permeability after removal of the filter.
fabric, however, no significant quantity of sediments were found at the bottom of the specimen once the filter fabric was removed.

The changes in flow rate of the beam specimens subjected to clayey silty sand sediments at the three different stages of the test procedure are shown in Figure 4.14. The sedimentation test did not have the same results as found with cylindrical specimens. The reduction in flow rate of these specimens was small compared to the cylinders. Visual observation of the bottom of the beam after removal of the filter fabric showed a thin layer of clayey silt sediments uniformly deposited at the bottom (see Figure 4.15). This layer was considerably thinner than the layer observed with the clayey silt tests.

The layer of clayey silty sediments on top of the sand at the end of the sedimentation test was also observed with the two-dimensional test. This observation suggests that the sand may have stopped some of the clayey silt sediments from reaching the surface of the specimen, creating segregation of the soil particles during the test. If that is the case, the low flow with high sediment load applied in a single load may not be the most appropriate test for assessing the effects of clayey silty sand sediments.

Recovery of permeability by washing, as measured by these tests, is probably unrealistically high unless accompanied by vacuum sweeping. Washing procedures causing flow of the diluted sediments over the surface of the pervious pavement would likely result in finer
particles being washed into the pervious concrete where they would migrate through the PCPS over time.
Figure 4.11. Remaining Coefficient of Permeability of Cylinder Specimens Subjected to Clayey Silty Sand Sediments in Phase I
Figure 4.12. Cylinder specimen Exposed to Clayey Silty Sand

Figure 4.13. Cylinder Specimens After Exposure to Clayey Silty Sand
Figure 4.14. Remaining Flow Rate of Beam Specimens Subjected to Clayey Silty Sand Sediments in Phase I
Figure 4.15. Beam Specimen After Exposure to Clayey Silty Sand
One of the interesting observations of this study is that flow of the finer fraction of sediments was very rapid. This implies that the formation of a sediment layer above the filter fabric will occur reasonably rapidly after sediment arrives at the pervious concrete pavement. As many others have noted (ACI, 2006; Tennis, Leming and Akers, 2004) it is critical to control erosion during construction.

Another interesting observation is that recovery of permeability is never complete with sediments containing fine grained particles. This observation is consistent with closure, or blocking, of small diameter voids that may be connecting much larger voids. Additional study of the relationships between void size distribution, porosity and sediment characteristics is recommended.
4.3.2. Phase II: High and Low Flow with Typical Sediment Load

The effects of different types of sediment on the coefficient of permeability of the cylinder specimens subjected to typical sediment loads are provided in Table 4.8. The coefficient of permeability determined before and after sedimentation, and after washing for two cycles, and after the removal of the filter fabric are given. The remaining permeability, in percentage of initial after each stage and cycle are shown in Table 4.9.

The effects of the different types of sediments on the flow rate of the beam specimens are shown in Table 4.10. The beam specimens subjected to either clayey silt or clayey sand sediments are shown with the initial flow rate and the permeability after sedimentation and washing for 5 cycles, and after the removal of the filter fabric. The remaining flow rate in percentage of the initial after each stage and cycle is also shown in Table 4.11.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Initial coefficient of permeability, in./hr (cm/s)</th>
<th>Coefficient of permeability after sedimentation 1, in./hr (cm/s)</th>
<th>Coefficient of permeability after washing 1, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-3-5</td>
<td>Sand</td>
<td>1230 (0.88)</td>
<td>160 (0.11)</td>
<td>370 (0.26)</td>
</tr>
<tr>
<td>C-3-6</td>
<td>Sand</td>
<td>1250 (0.89)</td>
<td>40 (0.02)</td>
<td>400 (0.29)</td>
</tr>
<tr>
<td>C-3-11</td>
<td>Sand</td>
<td>2750 (1.95)</td>
<td>150 (0.11)</td>
<td>650 (0.46)</td>
</tr>
<tr>
<td>C-3-12</td>
<td>Sand</td>
<td>2680 (1.90)</td>
<td>130 (0.09)</td>
<td>780 (0.56)</td>
</tr>
<tr>
<td>C-3-1</td>
<td>Clayey silt</td>
<td>1450 (1.03)</td>
<td>100 (0.07)</td>
<td>440 (0.31)</td>
</tr>
<tr>
<td>C-3-2</td>
<td>Clayey silt</td>
<td>1370 (0.97)</td>
<td>90 (0.09)</td>
<td>220 (0.15)</td>
</tr>
<tr>
<td>C-3-7</td>
<td>Clayey silt</td>
<td>3320 (2.36)</td>
<td>20 (0.01)</td>
<td>380 (0.27)</td>
</tr>
<tr>
<td>C-3-8</td>
<td>Clayey silt</td>
<td>2410 (1.71)</td>
<td>30 (0.02)</td>
<td>200 (0.14)</td>
</tr>
<tr>
<td>C-3-3</td>
<td>Clayey silty sand</td>
<td>1500 (1.07)</td>
<td>30 (0.02)</td>
<td>110 (0.08)</td>
</tr>
<tr>
<td>C-3-4</td>
<td>Clayey silty sand</td>
<td>1690 (1.20)</td>
<td>20 (0.01)</td>
<td>90 (0.06)</td>
</tr>
<tr>
<td>C-3-9</td>
<td>Clayey silty sand</td>
<td>2580 (1.83)</td>
<td>20 (0.01)</td>
<td>130 (0.09)</td>
</tr>
<tr>
<td>C-3-10</td>
<td>Clayey silty sand</td>
<td>2870 (2.04)</td>
<td>40 (0.03)</td>
<td>320 (0.23)</td>
</tr>
</tbody>
</table>
Table 4.8 (Cont.) Changes in Coefficient of Permeability in Cylinders at Different Stages of the Phase II Sedimentation Test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Coefficient of permeability after sedimentation 2, in./hr (cm/s)</th>
<th>Coefficient of permeability after washing 2, in./hr (cm/s)</th>
<th>Coefficient of permeability after removing fabric, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-3-5</td>
<td>Sand</td>
<td>20 (0.01)</td>
<td>310 (0.22)</td>
<td>400 (0.28)</td>
</tr>
<tr>
<td>C-3-6</td>
<td>Sand</td>
<td>6 (0.00)</td>
<td>350 (0.24)</td>
<td>290 (0.20)</td>
</tr>
<tr>
<td>C-3-11</td>
<td>Sand</td>
<td>50 (0.04)</td>
<td>290 (0.20)</td>
<td>980 (0.70)</td>
</tr>
<tr>
<td>C-3-12</td>
<td>Sand</td>
<td>80 (0.06)</td>
<td>700 (0.50)</td>
<td>1490 (1.06)</td>
</tr>
<tr>
<td>C-3-1</td>
<td>Clayey silt</td>
<td>20 (0.01)</td>
<td>100 (0.08)</td>
<td>770 (0.54)</td>
</tr>
<tr>
<td>C-3-2</td>
<td>Clayey silt</td>
<td>60 (0.05)</td>
<td>130 (0.09)</td>
<td>860 (0.61)</td>
</tr>
<tr>
<td>C-3-7</td>
<td>Clayey silt</td>
<td>20 (0.02)</td>
<td>350 (0.25)</td>
<td>1660 (1.18)</td>
</tr>
<tr>
<td>C-3-8</td>
<td>Clayey silt</td>
<td>30 (0.02)</td>
<td>200 (0.14)</td>
<td>1610 (1.14)</td>
</tr>
<tr>
<td>C-3-3</td>
<td>Clayey silty sand</td>
<td>2 (0.00)</td>
<td>40 (0.03)</td>
<td>300 (0.21)</td>
</tr>
<tr>
<td>C-3-4</td>
<td>Clayey silty sand</td>
<td>30 (0.02)</td>
<td>40 (0.03)</td>
<td>900 (0.64)</td>
</tr>
<tr>
<td>C-3-9</td>
<td>Clayey silty sand</td>
<td>10 (0.01)</td>
<td>40 (0.03)</td>
<td>1430 (1.02)</td>
</tr>
<tr>
<td>C-3-10</td>
<td>Clayey silty sand</td>
<td>20 (0.01)</td>
<td>90 (0.06)</td>
<td>1430 (1.02)</td>
</tr>
<tr>
<td>Specimen</td>
<td>Sediment type</td>
<td>Remaining permeability after sedimentation 1, %</td>
<td>Remaining permeability after washing 1, %</td>
<td>Remaining permeability after sedimentation 2, %</td>
</tr>
<tr>
<td>----------</td>
<td>----------------</td>
<td>-----------------------------------------------</td>
<td>------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>C-3-5</td>
<td>Sand</td>
<td>13%</td>
<td>30%</td>
<td>1%</td>
</tr>
<tr>
<td>C-3-6</td>
<td>Sand</td>
<td>3%</td>
<td>32%</td>
<td>0%</td>
</tr>
<tr>
<td>C-3-11</td>
<td>Sand</td>
<td>6%</td>
<td>24%</td>
<td>2%</td>
</tr>
<tr>
<td>C-3-12</td>
<td>Sand</td>
<td>5%</td>
<td>29%</td>
<td>3%</td>
</tr>
<tr>
<td>C-3-1</td>
<td>Clayey silt</td>
<td>7%</td>
<td>30%</td>
<td>1%</td>
</tr>
<tr>
<td>C-3-2</td>
<td>Clayey silt</td>
<td>6%</td>
<td>16%</td>
<td>5%</td>
</tr>
<tr>
<td>C-3-7</td>
<td>Clayey silt</td>
<td>1%</td>
<td>11%</td>
<td>1%</td>
</tr>
<tr>
<td>C-3-8</td>
<td>Clayey silt</td>
<td>1%</td>
<td>8%</td>
<td>1%</td>
</tr>
<tr>
<td>C-3-3</td>
<td>Clayey silty sand</td>
<td>2%</td>
<td>7%</td>
<td>0%</td>
</tr>
<tr>
<td>C-3-4</td>
<td>Clayey silty sand</td>
<td>1%</td>
<td>5%</td>
<td>2%</td>
</tr>
<tr>
<td>C-3-9</td>
<td>Clayey silty sand</td>
<td>1%</td>
<td>5%</td>
<td>0%</td>
</tr>
<tr>
<td>C-3-10</td>
<td>Clayey silty sand</td>
<td>1%</td>
<td>11%</td>
<td>1%</td>
</tr>
</tbody>
</table>
Table 4.10. Changes in Flow Rate in Beams at Different Stages of the Phase II

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Flow rate, in./hr (cm/s)</th>
<th>Flow rate after sedimentation 1, in./hr (cm/s)</th>
<th>Flow rate after washing 1, in./hr (cm/s)</th>
<th>Flow rate after sedimentation 2, in./hr (cm/s)</th>
<th>Flow rate after washing 2, in./hr (cm/s)</th>
<th>Flow rate after sedimentation 3, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3-1</td>
<td>Clayey silt</td>
<td>1200 (0.85)</td>
<td>30 (0.02)</td>
<td>900 (0.64)</td>
<td>30 (0.02)</td>
<td>1200 (0.85)</td>
<td>300 (0.21)</td>
</tr>
<tr>
<td>B-3-4</td>
<td>Clayey silt</td>
<td>3600 (2.56)</td>
<td>10 (0.01)</td>
<td>3600 (2.56)</td>
<td>3600 (2.56)</td>
<td>3600 (2.56)</td>
<td>44 (0.03)</td>
</tr>
<tr>
<td>B-3-2</td>
<td>Clayey silty sand</td>
<td>1200 (0.85)</td>
<td>2 (0.00)</td>
<td>980 (0.70)</td>
<td>30 (0.02)</td>
<td>680 (0.48)</td>
<td>20 (0.01)</td>
</tr>
<tr>
<td>B-3-3</td>
<td>Clayey silty sand</td>
<td>3600 (2.56)</td>
<td>20 (0.01)</td>
<td>1350 (0.96)</td>
<td>180 (0.13)</td>
<td>1540 (1.10)</td>
<td>90 (0.07)</td>
</tr>
<tr>
<td>B-3-5</td>
<td>Clayey silty sand</td>
<td>3600 (2.56)</td>
<td>50 (0.04)</td>
<td>3600 (2.56)</td>
<td>100 (0.07)</td>
<td>3600 (2.56)</td>
<td>10 (0.01)</td>
</tr>
<tr>
<td>B-3-6</td>
<td>Clayey silty sand</td>
<td>2700 (1.92)</td>
<td>10 (0.01)</td>
<td>2160 (1.53)</td>
<td>75 (0.05)</td>
<td>2700 (1.92)</td>
<td>230 (0.16)</td>
</tr>
</tbody>
</table>
Table 4.10 (Cont.) Changes in Flow Rate in Beams at Different Stages of the Phase II

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Flow rate after washing 3, in./hr (cm/s)</th>
<th>Flow rate after sedimentation 4, in./hr (cm/s)</th>
<th>Flow rate after washing 4, in./hr (cm/s)</th>
<th>Flow rate after sedimentation 5, in./hr (cm/s)</th>
<th>Flow rate after washing 5, in./hr (cm/s)</th>
<th>Flow rate after removing fabric, in./hr (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3-1</td>
<td>Clayey silt</td>
<td>1080 (0.77)</td>
<td>470 (0.33)</td>
<td>830 (0.59)</td>
<td>130 (0.09)</td>
<td>900 (0.64)</td>
<td>900 (0.54)</td>
</tr>
<tr>
<td>B-3-4</td>
<td>Clayey silt</td>
<td>3600 (2.56)</td>
<td>3600 (2.56)</td>
<td>2700 (1.92)</td>
<td>2160 (1.53)</td>
<td>2700 (1.92)</td>
<td>2700 (1.92)</td>
</tr>
<tr>
<td>B-3-2</td>
<td>Clayey silty sand</td>
<td>340 (0.24)</td>
<td>10 (0.01)</td>
<td>230 (0.16)</td>
<td>3 (0.00)</td>
<td>300 (0.21)</td>
<td>490 (0.35)</td>
</tr>
<tr>
<td>B-3-3</td>
<td>Clayey silty sand</td>
<td>140 (0.10)</td>
<td>4 (0.00)</td>
<td>300 (0.21)</td>
<td>20 (0.01)</td>
<td>260 (0.18)</td>
<td>260 (0.18)</td>
</tr>
<tr>
<td>B-3-5</td>
<td>Clayey silty sand</td>
<td>1540 (1.10)</td>
<td>120 (0.08)</td>
<td>980 (0.70)</td>
<td>6 (0.00)</td>
<td>1200 (0.85)</td>
<td>1350 (0.96)</td>
</tr>
<tr>
<td>B-3-6</td>
<td>Clayey silty sand</td>
<td>1080 (0.77)</td>
<td>80 (0.05)</td>
<td>270 (0.19)</td>
<td>2 (0.00)</td>
<td>490 (0.35)</td>
<td>900 (0.64)</td>
</tr>
</tbody>
</table>
Table 4.11. Remaining Flow Rate in Beams at Different Stages of the Phase II Sedimentation Test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Remaining flow rate sedimentation 1, %</th>
<th>Remaining flow rate washing 1, %</th>
<th>Remaining flow rate after sedimentation 2, %</th>
<th>Remaining flow rate after washing 2, %</th>
<th>Remaining flow rate after sedimentation 3, %</th>
<th>Remaining flow rate after washing 3, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3-1</td>
<td>Clayey silt</td>
<td>3%</td>
<td>75%</td>
<td>2%</td>
<td>100%</td>
<td>25%</td>
<td>90%</td>
</tr>
<tr>
<td>B-3-4</td>
<td>Clayey silt</td>
<td>0%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>1%</td>
<td>100%</td>
</tr>
<tr>
<td>B-3-2</td>
<td>Clayey silty sand</td>
<td>0%</td>
<td>82%</td>
<td>3%</td>
<td>56%</td>
<td>1%</td>
<td>28%</td>
</tr>
<tr>
<td>B-3-3</td>
<td>Clayey silty sand</td>
<td>0%</td>
<td>38%</td>
<td>5%</td>
<td>43%</td>
<td>3%</td>
<td>4%</td>
</tr>
<tr>
<td>B-3-5</td>
<td>Clayey silty sand</td>
<td>1%</td>
<td>100%</td>
<td>3%</td>
<td>100%</td>
<td>0%</td>
<td>43%</td>
</tr>
<tr>
<td>B-3-6</td>
<td>Clayey silty sand</td>
<td>0%</td>
<td>80%</td>
<td>3%</td>
<td>100%</td>
<td>8%</td>
<td>40%</td>
</tr>
</tbody>
</table>
**Table 4.11 (cont.)** Remaining Flow Rate in Beams at Different Stages of the Phase II Sedimentation Test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Remaining flow rate after sedimentation 4, %</th>
<th>Remaining flow rate after washing 4, %</th>
<th>Remaining flow rate after sedimentation 5, %</th>
<th>Remaining flow rate after washing 5, %</th>
<th>Remaining flow rate after removing fabric, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3-1</td>
<td>Clayey silt</td>
<td>39%</td>
<td>69%</td>
<td>10%</td>
<td>75%</td>
<td>75%</td>
</tr>
<tr>
<td>B-3-4</td>
<td>Clayey silt</td>
<td>100%</td>
<td>75%</td>
<td>60%</td>
<td>75%</td>
<td>75%</td>
</tr>
<tr>
<td>B-3-2</td>
<td>Clayey silty sand</td>
<td>1%</td>
<td>19%</td>
<td>0%</td>
<td>25%</td>
<td>41%</td>
</tr>
<tr>
<td>B-3-3</td>
<td>Clayey silty sand</td>
<td>0%</td>
<td>8%</td>
<td>1%</td>
<td>7%</td>
<td>7%</td>
</tr>
<tr>
<td>B-3-5</td>
<td>Clayey silty sand</td>
<td>3%</td>
<td>27%</td>
<td>0%</td>
<td>33%</td>
<td>38%</td>
</tr>
<tr>
<td>B-3-6</td>
<td>Clayey silty sand</td>
<td>3%</td>
<td>10%</td>
<td>0%</td>
<td>18%</td>
<td>33%</td>
</tr>
</tbody>
</table>
4.3.2.1. Sand Sediments

The change in permeability of cylinder specimens subjected to sand sediments at the three different stages of the test procedure, including two cycles each of sedimentation and washing, is shown in Figure 4.16. During the first cycle, the four specimens show practically the same loss of permeability, from about 10% of initial recovery to about 30% after washing. All the specimens showed a higher loss of permeability during the second application of sediment, to about 2% of the initial permeability. After the second washing, one of the specimens, C-3-11, only recovered 10% of the initial permeability while the other three specimens recovered between 25% and 30%. Removal of the filter fabric did not show any significant effect except for one of the specimens, C-3-12, that had a remaining permeability of 55%.

As expected, sand sediments were generally trapped at the top of the specimens after the sedimentation test, and were removed by washing in both cycles (see Figure 4.17). The permeability started being affected by sand sediments at about 0.50 lb/ft². Visual observation at the moment of the filter fabric removal did not show a significant amount of sand sediment deposited at the bottom, as shown in Figure 4.18. The permeability recovery of the specimens and the observation of the filter fabric at the end of the test seem to indicate that the finer sand particles penetrated the specimen and were trapped within the concrete.
Figure 4.16. Remaining Coefficient of Permeability of Cylinder Specimens Subjected to Sand Sediments at Different stages
Figure 4.17. Cylinder Specimens Exposed to Sand

Figure 4.18. Cylinder Specimen After Exposure to Sand
4.3.2.2. Clayey Silty Sediments

The change in permeability of the cylinder specimens subjected to clayey silty sediments at the three different stages of the test procedure, including two cycles each of sedimentation and washing, is shown in Figure 4.19. During the first cycle, the four specimens show practically the same lost of permeability, lower than 10% of the initial value. Permeability recovery of the specimens after washing increased the permeability, but results were varied. Permeability recovered from less than 10% to 30% and returned to values lower than 10% of the original value after the second sedimentation. The specimens were not able to recover much of the permeability after the washing procedure, ranging from 5% to 10% of initial permeability. The four specimens recovered about 55% of the original permeability after the removal of the filter fabric.

After each sediment test, silty clayey sediments were deposited on top of the specimen, covering most of the surface area (see Figure 4.20). Visual observation during removal of the filter fabric showed a significant amount of sediment deposited at the bottom (see Figure 4.21).

The visual observation and the permeability recovery of the specimens indicated that significant quantities of clayey silt sediments penetrated the pervious concrete and were deposited at the bottom of the specimen, retained by the filter fabric. In this case, the results were similar to the results with the low flow test. The recovery of the specimens after the removal of the filter fabric seems to confirm that most of these particles were retained by the
fabric and were free to flow out of the specimen once it was removed, however, some particles were trapped within the concrete, affecting the recovered permeability of the specimen without filter fabric present.

The change in flow rate of beam specimens subjected to clayey silty sediments is shown in Figure 4.22. Water flowing from the end of the beam was observed while performing the tests before and after sedimentation in all specimens. The sedimentation test did not seem to affect the flow rate of these specimens in the first three cycles, losing very little permeability after washing. After the third cycle, the flow rate loss appeared to become relatively stable. Approximately 75% of the initial flow rate remained. No difference in flow rate was observed after the removal of the filter fabric. This is not surprising since flow was forced to occur through the vertical end of the beam opposite the permeameter.

Clayey silty sediments deposited in the top of the specimen after each sediment load varied after each cycle. For those cases in which the specimen lost a considerable percent of the initial flow rate, a fraction of the sediment load was observed to be deposited on the surface, while in those cases that the specimens lost hardly any flow rate, only a small fraction was observed. Visual observation of the bottom of the beam during removal of the filter fabric found a uniformly distributed of sediment deposited at the bottom (see Figure 4.22).

These observations, including the recovery and deposition of sediments at the bottom of the specimen strongly suggest that the flow rate was affected for the most part by the clayey silty
soil sediments deposited on the surface of the concrete beam. There was, however a loss of about 30% of the initial flow rate that could be attributed to the soil particles remaining within the concrete that could not reach the bottom of the specimen.

Based on these findings, it is reasonable to suspect that continued vertical transport of these sediments will occur through the pavement and that continued accretion of sediment will occur at the bottom of the pavement. The long term permeability in the pervious concrete will be reduced by internal clogging. Since the permeability of the PCPS is not a controlling factor and the reduction is not likely to exceed 50%, the effects of sediment layers on top on the filter fabric remain the critical element in assessing hydrological behavior of PCPS at end of service life.
Figure 4.19. Remaining Coefficient of Permeability of Cylinder Specimens Subjected to Clayey Silty Sediments at Different Stages in Phase II
Figure 4.20. Cylinder Specimens Exposed to Clayey Silt

Figure 4.21. Cylinder Specimens After Exposure to Clayey Silt
Figure 4.22. Remaining Flow Rate of Beam Specimens Subjected to Clayey Silty Sediments at Different Stages
Figure 4.23. Beam Specimen After Exposure to Clayey Silt
4.3.2.3. Clayey Silty Sand Sediments

The change in permeability of cylinder specimens subjected to clayey silty sand sediments at the three different stages of the test procedure, including two cycles each of sedimentation and washing is shown in Figure 4.24. These specimens showed the worst permeability loss and lowest permeability recovery. During the first cycle, the four specimens show practically the same loss of permeability, less than 5% of the initial value. Recovery of permeability after washing was slight; values of permeability were 10% or less of the initial value. The second sedimentation cycle showed loss of permeability similar to the first cycle, with even lower recovery. The removal of the filter fabric increased the permeability up to 50% of the initial value for three specimens and 20% for one specimen.

After each sediment test, silty clayey sand sediments were observed on top of the specimen, covering most of the surface area (see Figure 4.25). Visual observation during removal of the filter fabric showed a significant amount of sediment deposited at the bottom, very similar to the case of clayey silt sediments (see Figure 4.26). These observations and the permeability values indicate some material remained in the pervious concrete but much had been transported through the specimen to form a layer at the bottom of the specimen.

The changes in flow rate permeability of beam specimens subjected to clayey silty sediments are shown in Figure 4.27. The specimens lost most, if not all, of their initial flow rate after each cycle of sedimentation. Two specimens recovered most of the flow rate within the two first cycles, while the other two recovered around 40% of the initial value. During the third
cycle, the remaining flow rate of the specimens was reduced to values around 35%, except for specimen B-2-3 which had a value around 5%. Cycles 4 and 5 show similar results with the specimen recovering around 10% to 25% of the initial flow rate. The removal of the filter fabric did not show a significant recovery of the flow rate.

Clayey silty sand sediments were observed to be deposited in the top of the specimen after each application of sediment. Visual observation of the bottom of the beam during the removal of the filter fabric found a uniformly distributed layer of sediment deposited at the bottom, very similar to that observed with the clayey silt sediments (see Figure 4.27). Water flowing from the end of the beam was observed while performing the flow rate tests before and after sedimentation in all the specimens.

These observations, including the recovery and the deposition of the sediments at the bottom of the specimen suggest that that the flow rate was affected by the soil grains deposited on top of the concrete, and that a high fraction of the finer clayey silt was deposited at the bottom, also affecting the flow rate of the specimen.
Figure 4.24. Remaining Coefficient of Permeability of Cylinder Specimens Subjected to Clayey Silty Sand Sediments at Different Stages
Figure 4.25. Cylinder Specimens Exposed to Clayey Silty Sand

Figure 4.25. Cylinder Specimens After Exposure to Clayey Silty Sand
Figure 4.23. Remaining Flow Rate of Beam Specimens Subjected to Clayey Silty Sand Sediments at Different Stages
Figure 4.24. Beam Specimen After Exposure to Clayey Silty Sand
4.3.3. Discussion of the Sedimentation Tests

Although the one-dimensional (cylinder) and two-dimensional (beam) tests show reasonably similar results, the test with the cylinder specimens seems to result in a higher loss of the hydraulic performance of the concrete than the two-dimensional test. In the one-dimensional test, the water with the sediments has a one directional flow through the specimen. The water, and the water with sediments, flow through a confined space tended to push the sediments deeper into the specimen. The high head of the water (18 in. [456 mm]) initially in the one-dimensional test may also have contributed to this effect. The water head was reduced in Phase II by using the 7.5 in. long permeameter.

The test with the beam specimens allowed the water to flow not only vertically through the concrete but also horizontally to the other end of the specimen. The water head in this test was a maximum of 7.5 in. [175 mm], reducing the water flow through the specimen. Even with the lower head, the horizontal velocity was sufficient to distribute the finer soil grains more or less uniformly along the filter fabric. The force applied by the water head may not push the sediments deeper within the specimen even in the one-dimensional test, and the water applied to evaluate flow rate may find an easier path to flow horizontally, not being partially retained by the finer sand particles within the top inches of the concrete as was common with the cylindrical specimens.
4.4. Implications of Sedimentation Effects on PCPS

4.4.1. Sedimentation Effects of Sand Sediments on PCPS

Both sedimentation tests confirmed that most of sand sediments will be trapped on top of the concrete, however, a part of the finer sand fraction will be deposited within the concrete, or travel through the concrete. The denser, less permeable surface acted like a coarse filter, passing small particles but trapping larger ones. This phenomenon will affect the apparent permeability of the PCPS by clogging the surface or near-surface region.

The permeability of a well designed and constructed PCPS will be much larger than the underlying soil infiltration. The denser layers of concrete at the top of the pervious concrete will be less permeable than the bulk of the pervious concrete and the apparent permeability of the PCPS will be affected by sand sediment deposited at the top. While the permeability of the pervious concrete, even when clogged, will rarely, if ever, be less than the infiltration of the underlying soil, if permeability becomes too low, more runoff may occur and less stormwater may be captured. More “ponding” can result in the perception of clogging with only a slight sand sediment load on top of the pavement. Surface permeability can be recovered to acceptable values by maintenance operations such as washing with pressurized water and vacuum (Haselbach, Valavala, and Montes, 2006). Changes in exfiltration due to sand sediments were demonstrated to be negligible as discussed in Sections 4.3.1.1 and 4.3.2.1. Storage capacity loss was also found to be negligible with this type of sediment.
On-site apparent permeability can be estimated using the Pervious Concrete Apparent Permeability Test (P-CAP). The P-CAP test is a simple, field procedure that involves pouring a known volume of water from a relatively small container, at a moderate, constant flow rate on a selected location on the pervious concrete pavement surface and measuring the dispersion or spread of the water over the surface of the pavement. The test is based on the sprinkler test used by Youngs (2006) to classify pervious concrete permeability. The full theoretical development by Mata and Leming (2008), is provided in Appendix A.

The apparent surface permeability can be expressed as:

\[
\mu_a = \frac{V_i}{A_p \cdot t}
\]

(4.1)

where

\(\mu_a\) = apparent surface permeability of pervious concrete,

\(V_i\) = volume of water poured onto the pavement,

\(A_p\) = area of the puddle, and

\(t\) = time to drain the water from the test container onto the pavement

with all values in consistent units. Typically, apparent surface permeability would be expressed in in./h or cm/h since those values are used in hydrologic design of pervious concrete pavements. The test can be used to evaluate the potential loss of apparent surface permeability due to sand sediments deposited on top of the pavement and effectiveness of maintenance in recovering surface permeability.
4.4.2. Sedimentation Effects of Clayey Silty Sediments

As expected, a layer of clayey silty sediments was deposited at the bottom of the concrete and retained by the filter fabric. This result was observed in both phases of the sedimentation tests. The layer was uniformly distributed at the bottom, instead of being deposited in a localized area, indicating that the sediments are maintained in suspension as the stormwater moves horizontally through the pervious concrete. The high flow rates possible in pervious concrete, at least those with a moderate porosity, with permeability greater than 9.0 in./hr (0.001 cm/s), the velocity below which particles passing the #200 sieve size (75 µm) settle in water, are sufficient to carry sediment horizontally. The high volume of solids used in the first phase and the repetitive application of a more typical amount of solids in the second phase both failed to create localized concentrations of deposits. Settlement occurred when vertical flow slowed significantly at the filter fabric. In practice, stormwater will retain fine sediments until horizontal equilibrium has been reached. These fine sediments are then likely to be largely, but not completely, carried down through the PCPS by subsequent storms until trapped by the filter fabric.

These findings and the estimated velocity for settlement are important since they indicate that a layer of finer grained sediments will be deposited along the entire surface of the filter fabric, creating a layer which could affect infiltration. This layer can affect serviceability at some point in the service life of the PCPS depending on the volume of sediment anticipated in service and the permeability of the sediment as deposited.
The filter fabric at the bottom of the concrete specimen may have contributed to the uniform deposition of the sediments. It was observed during testing that a minimum height, and thus weight, of approximately half an inch (12 mm) of water on top of the filter fabric was required for the water to vertically flow freely. This observation is comparable to the situation in the PCPS, when the water is retained in the system not only due to the filter fabric, but also by the underlying soil, which has a lower infiltration rate that the system. The retention of water due to either the filter fabric or the underlying soil, will tend to distribute the water uniformly at the bottom in a flat PCPS with no slope, creating a relatively uniformly deposited layer once the stormwater has been exfiltrated.

An additional test was performed after observing the sediment deposition pattern at the bottom of the specimen. The test was very similar to that described in Section 3.5 with the difference that all sides of the beam were covered with heavy tape, including the bottom of the specimen. Water without sediments was added to the specimen and kept within the beam approximately up to ¾ of the total storage capacity. Water with sediments was then added to the specimen and the heavy tape at the bottom of the specimen was removed allowing the water to flow freely through the specimen. Results of this test did not show much difference to those of both previous sedimentation tests. Sediments were observed to be horizontally deposited without any signs of localized deposition on the filter fabric.

In order to evaluate the likely effects of such a layer, the depth and the permeability of the deposit must be estimated. The depth may be estimated based on volume of deposits and
likely density as deposited. This value is also useful in improving the estimate of storage capacity loss. The permeability of fine grained soils is functionally related to the void ratio, which is also a function of density as deposited and compacted in service.

The clayey silt sediments deposited at the bottom will not affect the exfiltration of the system if the PCPS rests on a compacted clayey silty soil, since the layer of sediment deposited will not be compacted. The infiltration rate of the sediment layer will be higher than that of the subgrade, as shown in Section 2.9.2.2, and confirmed experimentally (see Section 4.3.1.2, and 4.4.1.2).

Traditional maintenance methods may not be efficient with clayey silt sediments. Deposition of the sediment at the bottom of the system will make it very difficult, if not impossible, to recover the clayey silt material using the same techniques as used with sand sediments, since the depth of the clayey silt sediments will likely be more than 6 in. (150 mm) from the surface of the pavement. These effects must therefore be considered in design. By estimating an effective exfiltration rate of the PCPS at the end of service, the Designer-of-Record can more accurately estimate the behavior of the PCPS at some future time using an estimated depth of sediment and an effective permeability estimated from analysis field trials.

4.4.3. Sedimentation Effects of Clayey Silty Sand Sediments

Silty sand sediments did not show any significant difference with the sand sediments initially trapping the finer particles in Phase I of the study in the one-dimensional test. Larger sand
particles in the water deposited earlier than smaller clay or silt, impeding these sediments from reaching the specimen and thus, finer particles did not flow through the pervious concrete to the bottom of the cylinder specimens. Additional rinsing visually confirmed eventual washing of the finer particles into the pervious concrete. This situation was similar in the two-dimensional test, in which some clayey silt sediments were able to enter the beams and form a thin layer of sediments at the bottom of the beam.

The vertical segregation of sediments, and the horizontal distribution of fine grained soil sediments was confirmed in the second phase of the sedimentation test, both with the one dimensional and two dimensional tests. A clayey silty sand will segregate, with time, with the sand trapped on the top of the concrete and, with time, clayey silty sediments eventually being deposited in a layer along the filter fabric.

The segregation observed in this case can lead to different permeabilities within the PCPS structure at the end of service as described in section 2.9.2.3 The permeability of this new layer of fine sediments deposited at the bottom of the system will affect the effective permeability of the system and, therefore, should be estimated and considered in the design of the PCPS.
4.5. Sediment Permeability Tests

4.5.1. Overview and Description

A simple test is needed to estimate the effective permeability of the layer of sediment deposited on filter fabric, or sand filter where used, over the service life of the PCPS. The effects of deposition and filtering, and the presence of larger aggregates fractions must be considered. It is desirable that such a test be able to be used in the field.

A simple test was developed in this study to provide the Designer-of-Record with a method to estimate the exfiltration value of the PCPS at end of service (EOS) conditions including the effects of sedimentation. This test was found to give values comparable to those found in one-dimensional testing in Phase I and II. The test method was developed as a modification of the methods used in both phases.

4.5.2. Procedure for Exfiltration Test

The test device, or “exfiltrometer,” consists of a PVC pipe 4 in. (100 mm) in diameter and 8 in. (20 cm) tall. Filter fabric is attached at the bottom of the pipe to retain the layer of sediments and the permeameter, which is similar to the one used in Phase II, is attached to the upper end of the pipe. A setup of the test is shown in Figure 4.25. A representative sample of soil for the location where the PCPS will be in service should be obtained and sieved to separate the finer particles passing the #200 sieve size (75 µm) that will form the layer at the bottom of the system. These finer particles will be used in the exfiltration test.
The volume of sediment can be estimated using USLE as described in Section 2.9.1.2 or by using approximate values given by EPA and shown on Table 2.4. Typical values based on USLE analysis are provided in tabular form in Appendix C as design aids. As shown in Sections 4.3 and 4.4 a fraction of the sediments are not transported to the bottom of the system but are trapped within the concrete. Any estimate of this fraction is subject to substantial variability and using 100% of the production is conservative.

The sediment load per unit area of PCPS is estimated and a load corresponding to the area of the exfiltrometer is mixed with water at an approximate concentration of 2% by weight. This concentration was used during Phase II and found to be a convenient and not unrealistic method for diluting the sediments in water. The water with the sediments is then added to the exfiltrometer and the sediments are left to air dry for a minimum of 24 hours to allow settlement and for the water to completely flow out of the device. The coefficient of
permeability is then determined using Equation 3.4. This coefficient of permeability can be used to estimate the exfiltration rate of the PCPS at EOS.

As shown in Section 2.9.2.3 the depth of the layer will affect exfiltration rate and the storage capacity of the system, thus it should be determined for the EOS. The depth of the layer of sediments at the EOS in a PCPS can be estimated by measuring it directly in the “exfiltrometer.” The depth is estimated as the average of at least two measurements. The depth of the layer can then be used to estimate the storage loss in the system at EOS.

4.5.3. Results of the Exfiltration Test

The coefficient of permeability for the exfiltration test was determined 1) including only a layer of sediments at the bottom; 2) including a stone base in the system; and 3) including a layer of sand at the bottom of the pavement to act as a filter for the smaller particles.

The depth of the layer of sediment was measured directly at the bottom of the pipe by a measuring tape, and was recorded to the nearest 0.20 in. (5 mm). The coefficient of permeability for Case 1, no filter, and Case 2, stone, were very similar, at about 0.14 in./hr (0.44 cm/hr) and 0.15 in./hr (0.48 cm/hr) respectively. The addition of a stone base did not result in any change in the exfiltration.

In Case 3, use of a sand filter, a layer of clayey silty sediments was observed on top of the sediment, suggesting that the finer particles will not penetrate the layer of sand. The
permeability was most likely controlled by this layer, showing a very similar flow rate to Case 1 and Case 2.

The coefficient of permeability in Phase I and II was measured as a system of the sediment and the concrete specimen. A more representative alternative is to compare those results with the exfiltration of the new test using flow rate. The flow rate of Phase II cylinders subjected to silty clayey sediments after the sedimentation test are shown in Table 4.12. These specimens were selected because the permeameter used in this phase was the same as used in the exfiltration test. As expected, the flow rate of the sediment layer is comparable to those of Phase II subjected to the same sediment type.

The results show that the proposed exfiltration rate can be used to estimate not only the exfiltration rate the sediment layer at the EOS but also its depth. These values are necessary when analyzing hydrological behavior as discussed in Chapter 5.

**Table 4.12. Flow Rate of Phase II Cylinder Specimens Subjected to Sedimentation with Clayey Silty Soil and the Layer of Sediment in Exfiltration Test**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sediment type</th>
<th>Flow rate, in./hr (cm/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-3-1</td>
<td>Clayey silt</td>
<td>12 (29)</td>
</tr>
<tr>
<td>C-3-2</td>
<td>Clayey silt</td>
<td>50 (128)</td>
</tr>
<tr>
<td>C-3-7</td>
<td>Clayey silt</td>
<td>18 (45)</td>
</tr>
<tr>
<td>C-3-8</td>
<td>Clayey silt</td>
<td>19 (49)</td>
</tr>
<tr>
<td>Layer</td>
<td>Clayey silt</td>
<td>11 (29)</td>
</tr>
</tbody>
</table>
Chapter 5. Design Guidelines

5.1. General

In this chapter, design guidelines including long term effects of sedimentation of a PCPS are developed and discussed. Applications of the design guidelines are examined and the implications of the analysis for some common situations are also discussed in this chapter. The guidelines complement the hydrological design of PCPS discussed in the Portland Cement Association (PCA) Engineering Bulletin (EB) 303 (Leming, Malcom and Tennis, 2007), and are based on the results and analysis of the sedimentation tests discussed in Chapter 4.

The recommended procedure consist of the five steps listed below:

1. Conduct hydrologic analysis of the PCPS as described by Leming, Malcom and Tennis in EB 303 (2007). This analysis will include determination of Curve Numbers and areas of parcels adjacent to or within the PCPS that drain onto the PCPS including contiguous, vegetated areas, buildings on-or offsite, and areas paved with conventional paving materials. Selection of an appropriate initial exfiltration value of the subgrade is an important design factor and is based on the soil characteristics of the site.

2. Determine the composition of the sediment. The characteristics of the sediment are assumed to be those of the subgrade unless analysis indicates otherwise.
3. Estimate the volume of sediment anticipated over the design life of the PCPS, using 20 years as the default life. Use either USLE, general EPA estimates or other rational method. A selected set of USLE design values are including in Appendix C.

4. Based on the area of the PCPS and the volume of the sediment anticipated, estimate the sediment load in mass per unit area. Using this value, a representative sample of the sediment, with coarser particles sieved out, and the “exfiltrometer” device described in Section 4.5, determine the exfiltration rate through the layer of sediment likely to form over the service life of the PCPS. In the absence of experimental data or as part of a preliminary design study, the nominal exfiltration rates developed in this study can be used.

5. Using the experimentally derived exfiltration value of the sediment layer or an initial estimate from published data, check hydrologic behavior, including runoff and recovery time of the PCPS using the method described in EB 303 (Leming, Malcom and Tennis, 2007). If the design is no longer acceptable, adjust the design of the PCPS, limit runoff if possible, design additional features to reduce or capture sediment before it arrives at the PCPS, or consider alternate BMPs to reduce discharge and maintain water quality.
5.2. Estimates of Sediment Volume

As discussed in section 2.9.1.1, a precise estimate of the quantity of sediment for a specific site is not possible. Accuracy and precision of the estimates obtained using USLE are highly dependant on the accuracy and precision of the input variables, which must be selected based on experience and engineering judgment. The designer must recognize that the estimated value is a reasonable but only approximate estimate of the sediment production for a specific site. Estimates used in design should be on conservative in general.

The USLE was used to generate a set of tables of estimates of the sediment production depending on the slope and the length of the slope, for sites with sand, clayey silt, or clayey silty sand and three types land cover: pasture, bare soil, and an intermediate cover between these two (see Appendix C). These tables may be used by the practicing engineer to rapidly determine a design value for amount of sediment on a given site. Alternately, design values may be selected from EPA (US EPA, 1999) based on general site characteristics as shown in Table 2.4. For example, the sediment estimated for an industrial site is 860 pounds per acre per year (lb/ac/yr) [970 kg/ha/yr].

When the site characteristics of a particular project indicate little likely production of sediments, a typical value of 1,000 lbs/acre/year (1,125 kg/ha/year) production may be used in analysis of end of service (EOS) hydrological performance to also account for deposition of sediments transported by other mechanism such as wind and vehicles. This approach is extremely conservative in many situations. If erosion control is strictly enforced during
construction, and hydrological behavior is still adequate at EOS, no further analysis may be required. When EOS behavior is marginal using this assumption, the Design Professional of Record may choose to examine behavior with a more rational estimate or modify the design of the PCPS.

The deposition patterns of the sediments in the PCPS may, in general, be different depending on the type of sediments. The horizontal and vertical distribution of the sediments may vary depending of the soil particle size and sediment load. The results of Phase I and Phase II testing in this study indicated that, in the case of sands, the coarser sand grains will be deposited on the surface of the pavement, while some of the finer grains will enter the system and be trapped within the concrete. It is not expected that the sand will be deposited evenly over the entire pavement area. Some locations will be more affected depending on the site layout.

Results of Phase I and II, discussed in Chapter 4, indicate that finer soil particles, those passing the #200 sieve size (75 µm), will be carried through the pervious concrete by runoff in both the vertical and horizontal directions. The deposits were essentially uniform at the bottom of the system, even in still water with no horizontal flow. This strongly suggests that design at EOS should include the effects of a uniformly deposit of sediment above the filter fabric.
5.3. Design Considerations for PCPS Subjected to Sand Sediments.

Hydrologic design procedures at EOS for PCPS subjected to sand sediments are not significantly different from those proposed in Leming, Malcom and Tennis in EB 303(2007). Sand sediment will not affect the exfiltration rate of the system, however, it can affect the surface permeability if a significant load of sediments is deposited on the pavement. The partial recovery of these locations to an acceptable performance can be achieved by maintenance procedures (Haselbach, Valavala, and Montes, 2006; Wingerter and Paine 1989).

The apparent permeability can be affected in localized zones that receive most of the sediments, however, the high rates of permeability of the pervious concrete will not likely significantly affect the overall performance of the system. The designer, however, should consider that a fraction of the surface permeability can be reduced during the service life of the system. Special consideration should be addressed in those cases in which the volume of sediments is relatively high and the area covered by the PCPS is relatively small.

Maintenance schedules will depend on the particular characteristic of the site. A simple and economical way to determine if a pavement needs maintenance is to use the P-CAP test (Mata and Leming, 2008), as described in Appendix B. PCPS are usually designed to have an on-site permeability of at least 290 in./hr (0.2 cm/s), therefore permeability values lower than this rate may indicate that the pervious concrete needs maintenance. Another approach to
determine the apparent permeability of the concrete is by performing a single ring permeability test. This and other permeability tests are in development by ASTM.

Pressurized washing is not the optimal alternative to maintain acceptable performance of the pavement. The sand particles will most likely enter the pavement, trapping the sand in the concrete instead of removing them. Pressurized washing can also contribute to surface raveling (ACI 522, 2006). Maintenance by sweeping or vacuum sweeping is preferred since it removes the sand.

5.4. Design Considerations for PCPS Subjected to Clayey Silty Sediments

5.4.1. Changes in Exfiltration Values with Sedimentation

As shown in Chapter 4, the clayey silty sediments will enter the PCPS and largely be deposited at the bottom of the system. It is expected that this layer will have depth differences due particular site characteristics. The use of a uniform layer of sediment in analysis is both reasonable and conservative.

The settlement of a uniform layer over the entire system is considered to be conservative analytically compared to localized sediment deposition. Consider, for example, the situations shown in Figures 5.1a and 5.2b, where two different cases are shown. Figure 5.1a shows a uniform deposition in which the exfiltration rate of the system at the end of service is approximately one third of the initial value, similar to the experimental results obtained with
the clayey silty sediment of this study. The exfiltration rate will be the same at all points since the layer is uniform.

Consider now the case in which there is a localized deposition in the system as shown in Figure 5.1-b. The section area with no sediment will remain with the same exfiltration rate estimated initially, that is, the sub-base infiltration rate. For a clayey silty sand, this value can be estimated as 0.5 in./hr (1.3 cm/hr). The exfiltration rate in the location with sediments will depend on the depth of this layer. A conservative value would be 0.1 in./hr (0.3 cm/hr), which is the infiltration rate estimation of a silty sub-base.

From Figure 5.1a:

\[ i_u = h_u k \]  \hspace{1cm} (5.1)

where:

\[ h_u = \text{depth of the uniform layer of sediment}, \]

\[ k = \text{unit change in exfiltration rate with depth}, \]
\( i_u = \) exfiltration rate of a uniformly deposited layer of sediment.

For these conditions, the exfiltration rate of the layer is lower than the exfiltration rate estimated at beginning of service. As discussed in Section 4.5.3, the exfiltration rate was found to vary approximately linearly with depth for the materials, depths and deposition densities likely to be encountered in practice.

From Figure 5.1b:

\[
V_w = \frac{1}{2} h_c L_c \tag{5.2}
\]

where:

\( V_w = \) is the volume of sediment per unit width of PCPS,

\( h_c = \) maximum depth of deposition, and

\( L_c = \) as defined above.

This volume must also be equal to the total deposited uniformly, that is,

\[
V_w = h_u L \tag{5.3}
\]

so

\[
h_u L = \frac{1}{2} h_c L_c \tag{5.4}
\]

and

\[
h_c = 2 h_u \frac{L}{L_c} \tag{5.5}
\]
The exfiltration rate in this zone can be estimated as:

\[ i_{\text{eff}} = \frac{(L - L_c)}{L} i_{sp} + \frac{L_c}{L} (i_{ls}) \]  

(5.6)

where,

- \( i_{\text{eff}} \) = effective exfiltration rate,
- \( i_{sp} \) = exfiltration rate estimated at beginning of service,
- \( i_{ls} \) = exfiltration rate of the section affected by localized sediment,
- \( L \) = total length of the system, and
- \( L_c \) = length of the section affected by sediment.

The exfiltration rate in the area of deposition can be estimated as

\[ i_{ls} = \frac{1}{2} \frac{h}{k} \]  

(5.7)

due to linearity of \( i \) and depth. Therefore

\[ i_{\text{eff}} = \frac{L - L_c}{L} i_{sp} + \frac{L_c}{L} \left( \frac{1}{2} \frac{h}{k} \right) \]  

(5.8)

and, due to volume restrictions

\[ i_{\text{eff}} = \frac{L - L_c}{L} i_{sp} + \frac{L_c}{L} \left[ \frac{1}{2} \left( \frac{2h}{L_c} \right) \right] \frac{k}{2} \]  

(5.9)

or, with simplification

\[ i_{\text{eff}} = i_{sp} - \frac{L_c}{L} i_{sp} + \frac{h}{k} k \]  

(5.10)

if \( i_{\text{eff}} > i_u \), then:
\[ i_{sp} - \frac{L_c}{L} i_{sp} + h_u k > h_u k \]  \hspace{1cm} (5.11a)

\[ i_{sp} - \frac{L_c}{L} i_{sp} > 0 \]  \hspace{1cm} (5.11b)

\[ i_{sp} > \frac{L_c}{L} i_{sp} \]  \hspace{1cm} (5.11c)

and

\[ 1 > \frac{L_c}{L} \]  \hspace{1cm} (5.11d)

This is true for all \( L_c \) since \( L_c < L \), therefore

\[ i_{eff} > i_u \]  \hspace{1cm} (5.12)

in all cases and a uniform deposition can be conservatively used in analysis.

As discussed in Chapter 4, the Designer-of-Record should consider the effect of filter fabric on exfiltration rate, but soils of this type will typically have low exfiltration rates and the filter fabric commonly required with those soils will have little effect on the exfiltration rate used in design. Design guidelines for PCPS at EOS subjected to clayey silt, or silt sediments are therefore not significantly different to that described by Leming, Malcom and Tennis in EB 303 (2007).
5.4.2. Changes in Storage Capacity with Sedimentation

The depth of the layer of clayey silty sediment that will be deposited at the bottom of the system can be estimated using the “exfiltrometer” device as described in Section 4.5. The sediment layer can affect the storage capacity of the system if a very small area of PCPS is subjected to a relatively high sediment load. For those cases, the Designer-of-Record may easily and economically accommodate the loss in storage capacity by providing additional depth of stone base; the marginal cost of an extra inch (25 mm) of stone is relatively low. The depth of additional stone required is the depth of sediment divided by 40%, the nominal porosity of the stone. A minimal depth of additional stone specification should be 1.0 in. (25 mm) when needed due to practical reasons including construction tolerances when the PCPS is exposed to fine particles. The effect of the sediment on storage capacity will clearly be reduced as the PCPS area increases with the same sediment production rate.

5.4.3. Maintenance for Clayey Silty Sediments

Traditional maintenance methods will have little effect on the hydrological behavior in PCPS overlying clayey silty or silty soils. Deposition of the sediment at the bottom of the system will make it very difficult, if not impossible, to recover and dispose of the clayey silt material using the same techniques as used with sand sediments.
5.5. Design Considerations for PCPS Subjected to Clayey, Silty Sand Sediments

Clayey silty sand sediments can affect the surface permeability, storage capacity, and, more importantly, the exfiltration rate of the system. The permeability may be affected more than in the case of sand sediments due to the wider gradation of soil particles that can be trapped, as discussed in Section 4.4.3, but the effects may still be ignored in many practical design considerations. The storage capacity is likely to be affected less than in the case of clayey silt sediments if the volume of sediments is the same since less of the total volume of clayey silty sand will reach the bottom of the PCPS.

5.5.1. Changes in Exfiltration with Sedimentation

The exfiltration can be affected in localized zones that receive most of the sediments or by formation of a more or less uniformly distributed deposit over the entire bottom area of the PCPS. The exfiltration rate of PCPS may be strongly affected by a layer of clayey silty sediments, particularly if a considerable volume is deposited at the bottom of the system. In general, the infiltration rate of this layer of clayey silt will be the controlling factor in the hydrologic behavior of a PCPS overlying a clayey silty sand. Careful analysis is required in those cases in which the volume of sediments is large and the area covered by the PCPS is relatively small.

The permeability of this layer can be estimated by using the test described in section 4.5.3. The quantity of the sediment and, therefore the depth of the layer, will depend on the
sediment production and zone distribution for the particular site. As shown in Figure 2.12, the permeability of the layer will be reduced at the depth increases.

5.5.2. Changes in Storage Capacity with Sedimentation

The inclusion of a clean stone base course is typically recommended for PCPS over clayey silty sands for both load carrying capacity and hydrological behavior. A PCPS subjected to this type of sediment may require a stone base even when analysis of behavior shows acceptable hydrological performance of the system without the base initially, if storage capacity reductions due to depositions of sediment on the filter are significant by EOS. The methodology for evaluation and correction is the same as discussed in Section 5.4.2.

5.5.3. Maintenance Procedures

A fraction of the surface permeability can be reduced during the service life of the system. The surface permeability can be recovered to acceptable values with the implementation of maintenance procedures as in the case with sandy sediments. The cleaning procedures, however, will remove the coarser sediments deposited on the surface of the pavement but not those smaller particles that are trapped in the concrete or deposited at the bottom of the system. The methodologies for evaluation and correction are the same as discussed in Section 5.3.
5.6. Examples and Discussion – Hydrological Performance at EOS

The implications of the design procedures including several common situations of a single developed site with a PCPS overlying different subgrades, and therefore subjected to various sedimentation conditions, were analyzed and the results discussed. Three cases of PCPS overlying and subjected to sand, clayey silt, or clayey silty sand were considered. The PCA software originally developed by Malcom and Leming (PCA 2007) to analyze the hydrological behavior of a PCPS was used for this analysis. Initial estimates of the infiltration rates for each case are based on TR-55 (SCS 1986) and EB 303 (Leming, Malcom and Tennis, 2007), and are shown in Table 5.1.

5.6.1. Soil and Sediment Characteristics

The three different soils used in the example analysis of the site are:

1. a sandy, well draining soil classified as HSG A;
2. a silty soil with an intermediate infiltration rate classified as HSG B; and
3. A silty sandy soil with some clay with a lower infiltration rate, classified as HSG B.

The sediments were assumed to be essentially those of the underlying soil. These clearly are the types of soils tested in this study. The experimental values obtained were used where appropriate.
Table 5.1. Initial Estimates of Infiltration Rate

<table>
<thead>
<tr>
<th>Case</th>
<th>Soil type</th>
<th>Classification</th>
<th>CN</th>
<th>Infiltration rate, in./hr (cm/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sandy, well draining</td>
<td>HSG A</td>
<td>58</td>
<td>1.0 (2.5)</td>
</tr>
<tr>
<td>2</td>
<td>Silt</td>
<td>HSG B</td>
<td>79</td>
<td>0.1 (0.3)</td>
</tr>
<tr>
<td>3</td>
<td>Clayey silty sand</td>
<td>HSG B</td>
<td>70</td>
<td>0.5 (1.3)</td>
</tr>
</tbody>
</table>

5.6.2. Site Characteristics

The development consists of 300,000 ft² (27,870 m²) of a PCPS parking lot, onto which the runoff of 125,000 ft² (11,600 m²) of impervious roof and impervious pavement structures drain. Vegetated islands and contiguous side slopes occupying 75,000 ft² (6,970 m²) will also drain into the pervious concrete pavement system as shown in Figure 5.1. The islands and slopes will be considered landscaped with grass and some bushes. The sloped, vegetated zones at one side of the developed site cover approximately 70,000 ft² (6,500 m²). The site is located in an SCS Type II rainfall regional such as the mid-Atlantic, United, with rolling terrain.

This particular configuration was selected for analysis for several reasons. It has areas of buildings and pavement that are reasonably typical for a small, independent, shopping center, as described in Section 2.9.1.3; but with slightly less area of PCPS so that the site could be considered challenging for the purposes of design. This configuration is also similar to the site used to demonstrate hydrological design in EB 303 (Leming, Malcom and Tennis, 2007), so that comparisons with previously published findings are more direct.
The effects of sedimentation on the hydrological behavior of the PCPS were assessed by considering different types of sediment as discussed above. In order to provide some basis for estimating flow rates and sediment loads, a variety of conditions of the contiguous, vegetated areas were examined. One case used vegetated zones with a slope of 5% and 100 ft (30 m) width (length of runoff) that drains directly to the PCPS.

A pervious concrete pavement depth of 6 in. (15 cm) was used in this example. The design porosity of the pervious concrete in the example is 20% and the PCPS is assumed to be level. These are commonly used values in practice. The pavement depth is assumed to be adequate for the anticipated traffic loads. The effects of base course were examined by using 6 in. (15 cm) of clean stone. A compacted aggregate base consisting of size #57 or #67 stone has a porosity of about 40%. The depths of both the pervious concrete and the clean stone based
were selected to be reasonable but minimal for the type of site to accentuate any hydrologic
deficiencies or problems in performance due to sedimentation.

The levels of precipitation used in this analysis are typical for a Type II storm in the mid-
Atlantic region. The precipitation in the 2-year storm was selected to be 3.5 in. (9 cm) and
the precipitation in the 10-year storm is given as 5.1 in. (13 cm) in 24 hours. The PCPS
design was constrained, somewhat arbitrarily, to capture all of the 2-year, 24-hour storm and
keep runoff in the 10-year, 24-hour storm at or below the 10-year 24-hr runoff estimated for
the site prior to development throughout an anticipated service life of 20 years.

Estimates of runoff at pre-development and post-development for the example case are
shown in Tables 5.2 and 5.3 respectively. Equivalent CN's and recovery time of storage
capacity after 5 days are shown in Table 5.4 and 5.5.

<table>
<thead>
<tr>
<th>Case</th>
<th>Infiltration rate, in./hr (cm/h)</th>
<th>2-year storm: 3.5 in. (90 mm)</th>
<th>10-year storm: 5.1 in. (130 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0 (2.5)</td>
<td>0.5 (13)</td>
<td>1.2 (30)</td>
</tr>
<tr>
<td>2</td>
<td>0.1 (0.3)</td>
<td>1.6 (41)</td>
<td>2.9 (74)</td>
</tr>
<tr>
<td>3</td>
<td>0.5 (1.3)</td>
<td>1.0 (25)</td>
<td>2.1 (53)</td>
</tr>
</tbody>
</table>

Table 5.2. Estimates of Pre-Development Runoff for Example Case
Table 5.3. Estimates of Post-Development Runoff for Example Case, initial

<table>
<thead>
<tr>
<th>Case</th>
<th>Infiltration rate, in./hr (cm/h)</th>
<th>2-year storm: 3.5 in. (90 mm)</th>
<th>10-year storm: 5.1 in. (130 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0 (2.5)</td>
<td>0.0 (0)</td>
<td>0.0 (0)</td>
</tr>
<tr>
<td>2</td>
<td>0.1 (0.3)</td>
<td>0.0 (0)</td>
<td>1.2 (30)</td>
</tr>
<tr>
<td>3</td>
<td>0.5 (1.3)</td>
<td>0.0 (0)</td>
<td>0.0 (0)</td>
</tr>
</tbody>
</table>

Table 5.4. Estimates of Post-Development Equivalent CN for Example Case, initial

<table>
<thead>
<tr>
<th>Case</th>
<th>Infiltration rate, in./hr (cm/h)</th>
<th>2-year storm: 3.5 in. (90 mm)</th>
<th>10-year storm: 5.1 in. (130 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0 (2.5)</td>
<td>&lt; 36*</td>
<td>&lt; 36*</td>
</tr>
<tr>
<td>2</td>
<td>0.1 (0.3)</td>
<td>36</td>
<td>58</td>
</tr>
<tr>
<td>3</td>
<td>0.5 (1.3)</td>
<td>&lt; 36*</td>
<td>&lt; 36*</td>
</tr>
</tbody>
</table>

*Calculated values of the equivalent Curve Number should not be given below about 36.

Table 5.5. Recovery Time After 5 days for Example Case, initial

<table>
<thead>
<tr>
<th>Case</th>
<th>Infiltration rate, in./hr (cm/h)</th>
<th>2-year storm: 3.5 in. (90 mm)</th>
<th>10-year storm: 5.1 in. (130 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0 (2.5)</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>2</td>
<td>0.1 (0.3)</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>3</td>
<td>0.5 (1.3)</td>
<td>yes</td>
<td>yes</td>
</tr>
</tbody>
</table>

5.6.3. Results and Analysis

The production of sediments produced by the erosion of the vegetated zones was estimated using the USLE. The rainfall and runoff erosivity index selected was 300 erosion index units per year using Figure 2.5 (Wurbs and James, 2001). Using the tables in Appendix C, for permanent pasture, the sediment production is estimated as 1,240 lbs/acre/year (1,395
kg/ha/year) for sand; 4,000 lbs/acre/year (4,500 kg/ha/year) for silt; and 2,480 lbs/acre/year (2,790 kg/ha/year) for silty sands. The difference in sediment production is due to different soil erodibility factors for each type of soil.

The sediment yields on a yearly basis for each case were estimated using Equation 2.3. These are shown in Equations 5.13, 5.14 and 5.15 for sand, clayey silt, and clayey silty sand sediments respectively.

\[
E_s = 1,240 \text{ lb/ac} \cdot 1.72\text{ac} = 2,130 \quad (5.13)
\]

\[
E_{cm} = 4,000 \text{ lb/ac} \cdot 1.72\text{ac} = 6,880 \quad (5.14)
\]

\[
E_{cms} = 2,480 \text{ lb/ac} \cdot 1.72\text{ac} = 4,270 \quad (5.15)
\]

where

\[
E_s = \text{sand sediment yield per year (lb)},
\]

\[
E_{cm} = \text{clayey silty sediment yield per year (lb)},\] and

\[
E_{cms} = \text{clayey, silty sandy sediment yield per year (lb)}.\]

5.6.3.1. Sand Sediment – Effects on hydrological Performance

The sand sediments will likely affect the surface permeability of only those areas located close to the point of runoff from adjacent sites. The extent of these areas will depend on particular site characteristics. Test results in Phase II (see Section 4.3.2.1) show that surface permeability will start being affected by sand sediments at about 0.50 lb/ft\(^2\) (2.50 kg/m\(^2\)) of
PCPS. Once this load is reached, cleaning maintenance by sweeping and vacuum should be necessary, however, surface permeability tests can be performed periodically to evaluate the performance of the system, particularly after storms with more runoff. As discussed above, no meaningful difference in hydrological behavior of the PCPS is otherwise anticipated.

5.6.3.2. Clayey Silty Sediment – Effect on Hydrological Performance

In this example, the load of sediment per area of PCPS is estimated to be about 0.023 lb/ft$^2$ (0.11 kg/m$^2$) per year, as shown in Equation 5.16:

$$E_{m/PCPS} = \frac{6,880\ lb}{300,000\ ft^2} = 0.023\ lb/ft^2$$  \hspace{1cm} (5.16)

After 20 years of service, the sediment accumulation would be around 0.46 lb/ft$^2$ (2.2 kg/m$^2$). The depth of this sediment layer at EOS can be estimated from results obtained using the “exfiltrometer” device as described in Section 4.5.3. For the clayey silt sediments used in this study, the depth of a layer produced by a load of this clayey silty sediment of 2.5 lb/ft$^2$ (12 kg/m$^2$) is estimated to be approximately 0.2 in. (5 mm) (see Section 4.5.3.).

Table 5.6 shows the estimates of the exfiltration rate and depth of the layer for loads of sediment per area for the clayey silty soil used in this study. For 0.46 lb/ft$^2$ (2.2 kg/m$^2$), the depth is estimated to be approximately 0.05 in. (1 mm). The loss of storage in this example is therefore negligible for a PCPS subjected to this amount of sediment load.
As discussed in Section 5.4, the effective exfiltration rate will not change with this type of sediment and subgrade. The runoff and recovery time for this case will not change from those values estimated initial (See Table 5.3 and 5.5).

The surface permeability of the system can also be affected in this case, especially in those areas located close to the point of runoff from adjacent sites. Test in Phase II show that a percentage of permeability, between 40% and 50% can be lost (see Figure 4.19) if the system is subjected to severe cases of sedimentation. With this type of sediment, maintenance by sweeping vacuuming will not be effective since the sediment trapped within the PCPS will be very difficult, if not impossible, to be recovered. In most practical cases, the permeability of the pervious concrete is so much greater than that of the subgrade, a reduction of even 100% would have little meaningful effect on hydrological behavior.

5.6.3.3. Clayey Silty Sand Sediment – Effect on Hydrological Performance

The surface permeability can be affected during the service life of the PCPS exposed to clayey silty sand, particularly in those locations close to the point of runoff from adjacent sites. Permeability lost and recovery for this case can be higher than those shown in the case with sand and clayey silty sediments as discussed in Section 4.4.3.

The fraction of the finer particles produced in this case was estimated as 40% of the total sediment production consistent with the material used in the experimental phases of this study. It is very likely that the runoff will have a higher content of fines than determined in a
particle size analysis since larger particles may drop out before hitting the PCPS. When runoff occurs over vegetated surface, it is not expected that all of the finer sediments will be deposited at the bottom of the system but a fraction will be trapped within the concrete, however. Using the total volume of finer grained sediment as the design value of a layer at the bottom of the PCPS is believed to be conservative for assessment of hydrological performance at EOS. Given the variability of the estimates, this approach should provide acceptable results.

To determine the nominal exfiltration rate of the system at EOS, values developed with the “exfiltrometer” device were used. For the clayey silt sediments used in this study, the depth of a layer produced by a load of this clayey silty sediment of 2.5 lb/ft\(^2\) (12 kg/m\(^2\)) is estimated to be approximately 0.2 in. (5 mm) with a nominal exfiltration rate of 0.15 in./hr (0.4 cm/hr). An inverse linear relation between the effective exfiltration rate and depth of the layer can be used to estimate the effective infiltration rate used in the EOS analysis. Although this relation is not linear, the approximation is considered to be adequate due to the accuracy of the data and the estimation of the infiltration rate in service. For this case, as shown in Table 5.6, an effective exfiltration rate is about 2.8 in./hr (7 cm/hr), larger that the initial 0.5 in./hr (1.3 cm/hr) [see Table 6.8], hence the underlying soil stills controls the design and initial estimations.
Table 5.6. Selected Estimate Values of Exfiltration Rate and Depth of Layer Based on Sediment Load

<table>
<thead>
<tr>
<th>Sediment load per area, lb/ft² (kg/m²)</th>
<th>Exfiltration rate, in./hr (cm/h)</th>
<th>Depth of layer, in. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8 (3.8)</td>
<td>0.5 (1.3)</td>
<td>0.05 (0.13)</td>
</tr>
<tr>
<td>0.9 (4.7)</td>
<td>0.4 (1.0)</td>
<td>0.06 (0.15)</td>
</tr>
<tr>
<td>1.3 (6.3)</td>
<td>0.3 (0.8)</td>
<td>0.09 (0.23)</td>
</tr>
<tr>
<td>1.9 (9.4)</td>
<td>0.2 (0.5)</td>
<td>0.13 (0.33)</td>
</tr>
<tr>
<td>3.8 (18.8)</td>
<td>0.1 (0.3)</td>
<td>0.26 (0.66)</td>
</tr>
</tbody>
</table>

Using this same approach, the sediment layer for this case is estimated to start to be lower than the initial exfiltration rate of 0.5 in./hr (1.3 cm/hr) when it reaches a depth of 0.05 in. (1.3 mm), that is, when the PCPS receives sediment load larger than 0.80 lb/ft² (3.8 kg/m²). This sediment load corresponds to a rate of approximately 18,000 pounds per acre per year (lb/ac/year; 20,200 kg/ha/year) in a 20 year design life. This value is extremely high when compared with EPA estimation for a construction site at about 6,000 lbs/ac/year (6,750 kg/ha/year).

5.6.3.4. Effective Default Values: Clayey Silty Sands

The typical value used in design of a PCPS over clayey silty sand is 0.5 in./hr. due to the differences in thickness of the deposit. It is reasonable to expect that the exfiltration rate at EOS in this particular situation would be between 0.1 in./hr (0.5 cm/hr) and 0.5 in/hr (1.3 cm/hr). The Designer-of-Record in this situation could choose to use the very conservative value of 0.1 in./hr (0.5 cm/hr) or elect to conduct and “exfiltrometer” test to estimate this value.
In this study, the “exfiltrometer” test was conducted to develop a better estimate of nominal exfiltration rate for analysis and to confirm the depth of the unconsolidated sediment. The exfiltration rate determined experimentally for the clayey silt was approximately 0.14 in./hr (0.4 cm/hr), with a depth of approximately 0.2 in. (5 mm). In practice, if analysis indicates satisfactory EOS behavior using a conservative estimate, additional experimental testing may not be needed.

5.7. Effect of Erosion Control

Erosion control on the site and hence, the production of sediments will directly affect the long term behavior of the PCPS. Consider the example of section 6.6.1, but the vegetated areas are covered with a straw mulch with a surface cover, c, estimated to be around 0.18 instead of permanent pasture. Sandy soils are not considered in this example.

Using the tables from Appendix C, for medium cover, the sediment production is estimated as 18,000 lbs/acre/year (20,250 kg/ha/year) for silt; and 11,200 lbs/acre/year (12,600 kg/ha/year) for silty sands. Following the same analysis as the example of Section 5.6.3, the PCPS will receive an estimate of 0.12 lb/ft$^2$ (0.6 kg/m$^2$) of sediment, and 0.03 lb/ft$^2$ (0.1 kg/m$^2$) per year for clayey silt and clayey silty sand respectively.
After 20 years of service, the accumulation of clayey silty sediments for case of clayey silt will be around 2.4 lb/ft\(^2\) (11.4 kg/m\(^2\)). Using the data from the “exfiltometer” test similar to the example in Section 5.6.3, the depth of the layer at the bottom of the system is estimated to be around 0.20 in. (5 mm). For a clean stone base with 40% porosity, this layer will result in approximately 0.5 in. (13 mm) loss of storage capacity after 20 years of service. For this type of situations, the addition of 1 in. (25 mm) of stone will cover the loss of storage capacity during the service life of the structure.

After 20 years of service, the accumulation of clayey silty sediments for the case of clayey silty sand will be around 0.60 lb/ft\(^2\) (3.0 kg/m\(^2\)), hence the exfiltration rate will not likely be significantly reduced from the value used in the original analysis by this sediment load.

5.8. Effect of Changes in Exfiltration Rate During Service

The size of the PCPS can affect the load of the silty sediments per area of PCPS. Consider a similar example from section 5.7, but in this case, the PCPS was 225,000 (20,900 m\(^2\)); the impervious zones are 180,000 ft\(^2\) (16,700 m\(^2\)); and the off-site area 90,000 ft\(^2\) (8,400 m\(^2\)). The pavement depth remains at 6 in. (150 mm), but the depth of the base was increased to 10 in. (200 mm). Porosity of the pervious concrete in the example is 20% and the PCPS is assumed to be level. The precipitation in the 2-year storm was selected to be 3.5 in. (9 cm) and the precipitation in the 10-year storm is given as 5.1 in. (13 cm) in 24 hours. The PCPS design was constrained to capture all of the 2-year, 24-hour storm and keep runoff in the 10-year,
24-hour storm at or below the 10-year 24-hr runoff estimated for the site prior to development throughout an anticipated service life of 20 years. The runoff and equivalent CN’s for the proposed development for clayey silt and clayey silty sand soils are given in Table 5.8 and 5.9. Keeping the same sediment production as in Section 5.7, the sediment load per unit area of PCPS are estimated around 0.15 lb/ft² and 0.04 lb/ft² per year for clayey silt and clayey silty sand respectively. Storage capacity for both cases was recovered within 5 days.

**Table 5.7. Initial Estimates of Infiltration Rate**

<table>
<thead>
<tr>
<th>Case</th>
<th>Soil type</th>
<th>Classification</th>
<th>CN</th>
<th>Infiltration rate, in./hr (cm/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Silt</td>
<td>HSG B</td>
<td>79</td>
<td>0.1 (0.3)</td>
</tr>
<tr>
<td>2</td>
<td>Clayey silty sand</td>
<td>HSG B</td>
<td>70</td>
<td>0.5 (1.3)</td>
</tr>
</tbody>
</table>

**Table 5.8. Estimates of Post-Development Runoff in Example of Section 5.8**

<table>
<thead>
<tr>
<th>Case</th>
<th>Infiltration rate, in./hr (cm/h)</th>
<th>Runoff, in. (mm)</th>
<th>2-year storm: 3.5 in. (90 mm)</th>
<th>10-year storm: 5.1 in. (130 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1 (0.3)</td>
<td></td>
<td>0.0 (0)</td>
<td>1.2 (30)</td>
</tr>
<tr>
<td>2</td>
<td>0.5 (1.3)</td>
<td></td>
<td>0.0 (0)</td>
<td>0.0 (0)</td>
</tr>
<tr>
<td>2 EOS</td>
<td>0.3 (9)</td>
<td></td>
<td>0.0 (0)</td>
<td>0.1 (6)</td>
</tr>
</tbody>
</table>

**Table 5.9. Estimates of Post-Development Runoff for Example 5.8**

<table>
<thead>
<tr>
<th>Case</th>
<th>Infiltration rate, in./hr (cm/h)</th>
<th>Equivalent CN</th>
<th>2-year storm: 3.5 in. (90 mm)</th>
<th>10-year storm: 5.1 in. (130 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1 (0.3)</td>
<td>36</td>
<td>36</td>
<td>58</td>
</tr>
<tr>
<td>2</td>
<td>0.5 (1.3)</td>
<td>&lt; 36*</td>
<td>&lt; 36*</td>
<td>&lt; 36*</td>
</tr>
<tr>
<td>2 EOS</td>
<td>0.3 (9.0)</td>
<td>36</td>
<td>36</td>
<td></td>
</tr>
</tbody>
</table>

*Calculated values of the equivalent Curve Number should not be given below about 36.
5.8.1. Effects on Hydrological Performance: Clayey Silt

After 20 years of service, the accumulation of clayey silty sediments will be around 3.6 lb/ft\(^2\) (17.0 kg/m\(^2\)). Using the data from the “exfiltometer” test similar to the example in Section 5.6.3, the depth of the layer at the bottom of the system is estimated to be around 0.25 in. (6 mm). For a clean stone base with 40% porosity, this layer will result in 0.5 in. (25 mm) loss of storage capacity after 20 years of service. As in Section 5.7, the addition of an extra 1 in. (25 mm) will cover the loss of storage capacity during the service life of the structure.

5.8.2. Effects on Hydrological Performance: Clayey Silty Sand

After 20 years of service, the accumulation of clayey silty sediments will be around 0.8 lb/ft\(^2\) (4.0 kg/m\(^2\)), the pavement For the clayey silt sediments used in this study, the depth of a layer produced by a load of this clayey silty sediment of 2.5 lb/ft\(^2\) (12 kg/m\(^2\)) is estimated to be approximately 0.2 in. (5 mm) with a nominal exfiltration rate of 0.15 in./hr (0.4 cm/hr) [see Section 4.5.3]. Using the same approach as in Section 5.6.3.3, it can be estimated the sediment layer for this case will have a depth around 0.1 in. (0.25) with an exfiltration rate of 0.3 in./hr. (0.8 cm/hr) at EOS.

The nominal exfiltration rate at EOS is lower than the initially estimated. Table 5.8 and 5.9 show the runoff and CN of the site at the EOS. All of the 2-year, 24-hour storm is still captured by the system, and the 10-year 24-hr runoff is reaching its limit at this point of not generating any runoff. However, the 10-year, 24-hour is still kept below the 10-year 24-hr runoff estimated in the pre-development stage.
5.9. Conclusions and Recommendations

PCPS subjected to typical loads of clayey silt or sandy sediments will most likely not be affected either in the storage capacity or the exfiltration rate at the EOS. The addition of one inch (25 mm) of the base layer should be sufficient in most cases to overcome storage capacity losses. The effect of heavy loads of sediment may be marginal in these situations, and the particular effects on storage capacity and exfiltration at EOS will depend on the sediment production and the size of the PCPS where the sediment in the stormwater runoff will drain. The Designer-of-Record should routinely include EOS analysis to ensure adequate behavior however.

A simple test was developed in this study to provide the Designer-of-Record with a method to estimate the exfiltration value of the PCPS at EOS conditions including the effects of sedimentation. This test was found to give values comparable to those found in one-dimensional testing in Phase I and II.

Analysis did not include the effects of significant quantities of organic material that could be washed onto the PCPS from the application of mulch on additional top soil to adjacent areas. Runoff in these conditions must be strictly controlled until these ground covers have stabilized if small particles or organic could be transported through the PCPS. As a minimum, accumulations of this type of material must be swept up immediately to prevent sedimentation problems with these materials causing significant changes in the effective exfiltration rate. Additional research in this area is recommended.
Some dust will inevitably be deposited on all surfaces. This fine grained material must be considered in design and analysis. The Designer-of-Record must include the effects of this material if the site will be exposed to unusually large quantities of dust, such as at or near an industrial site with limited dust abatement controls.

The substantial loss in permeability found in Phase II with multiple applications of sediments and washing indicates internal clogging. It is not known if this clogging will be different in pervious asphalt pavements or if warmer temperatures in service resulting in viscous flow of the asphalt, in conjunction with the clogging make pervious asphalt more susceptible to clogging. Additional research to address this possibility is recommended.
Chapter 6. Frost Durability Results and Discussions

6.1. General

In this chapter, the results of the frost durability testing are given for the four different mixtures used in the test. Testing confirmed previously published reports (Schaefer, Wang, Suleiman and Kevern, 2006; Kevern, Schaefer, Wang and Suleiman, 2008) conducted using a harsher testing environment, that sand must be included in the mixture to be frost resistant when saturated or normally saturated, regardless of the addition of air entraining admixture (AEA). None of the specimens without sand passed more than eight freezing and thawing cycles, showing clear signs of deterioration including aggregates debonding and visible cracking of the paste. Microscopic analysis of these specimens did not find any evidence of entrained air in the paste.

Specimens from Mixture 4 and Mixture 5, those with some sand in their proportioning, lost approximately 9% and 4% of mass respectively after 40 cycles. The dynamic elastic (Young’s) moduli ($E_d$) were reduced approximately 19% and 15% for Mixture 4 and Mixture 5 respectively. Microscopic examination of these specimens showed evidence of entrained air in the paste, however, the amount was minimal based on qualitative assessment.
6.2. Frost Resistance

6.2.1. Dynamic Elastic (Young’s) Modulus and Mass

Disk specimens from Mixture 1 and Mixture 2 were obtained by sawing cylinder specimens C-1-8 and C-1-9 from Mixture 1; and C-2-12 and C-2-13 from Mixture 2. The hydraulic properties of these cylinder specimens are shown in Table 4.1. The mass and $E_d$ of the disk specimens from Mixture 1 and Mixture 2 specimens are shown in Table 6.1. These properties could be measured only prior to the test since none of the specimens passed 10 cycles, the first scheduled interim testing. The $E_d$ of the disks showed a considerable higher value, approximately twice that of the respective beam companions. This difference may have been caused by the different compaction techniques used for the cylinder and the beam specimens. The energy applied by the proctor hammer in fabricating the cylinders is higher than the rolling procedure used for the beams, making a denser material. Effective porosity also showed a denser material in the cylinder specimens than in the beam specimens. The difference in $E_d$ was not expected based on differences in porosity. The effects of compaction energy or method on $E_d$ are not well established. Additional study on the differences in mechanical properties due to compaction method is recommended.
Table 6.1. Dry density, Mass, and Dynamic Elastic (Young’s) Modulus of Mixture 1 and Mixture 2 Disk Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dry density, lb/ft$^3$ (kg/m$^3$)</th>
<th>Mass, lbs (kg)</th>
<th>Dynamic Modulus, ksi (Gpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-1-1</td>
<td>120.47 (1951.61)</td>
<td>0.739 (0.336)</td>
<td>3,300 (23.0)</td>
</tr>
<tr>
<td>D-1-2</td>
<td>117.69 (1906.58)</td>
<td>0.680 (0.309)</td>
<td>3,100 (21.0)</td>
</tr>
<tr>
<td>D-1-3</td>
<td>125.59 (2034.56)</td>
<td>0.696 (0.317)</td>
<td>3,900 (26.5)</td>
</tr>
<tr>
<td>D-1-4</td>
<td>123.40 (1999.08)</td>
<td>0.711 (0.323)</td>
<td>3,700 (25.5)</td>
</tr>
<tr>
<td>Average</td>
<td>121.79 (1973.00)</td>
<td></td>
<td>3,500 (24.0)</td>
</tr>
<tr>
<td>D-2-1</td>
<td>122.63 (1986.61)</td>
<td>0.733 (0.333)</td>
<td>3,300 (23.0)</td>
</tr>
<tr>
<td>D-2-2</td>
<td>116.70 (1890.54)</td>
<td>0.726 (0.330)</td>
<td>2,800 (19.5)</td>
</tr>
<tr>
<td>D-2-3</td>
<td>121.31 (1965.22)</td>
<td>0.697 (0.317)</td>
<td>3,500 (24.0)</td>
</tr>
<tr>
<td>D-2-4</td>
<td>116.85 (1892.97)</td>
<td>0.667 (0.303)</td>
<td>2,900 (20.0)</td>
</tr>
<tr>
<td>Average</td>
<td>119.37 (1933.79)</td>
<td></td>
<td>3,100 (21.5)</td>
</tr>
</tbody>
</table>

Table 6.2. Dry Density and Porosity of Mixture 4 and Mixture 5 Disk Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dry density, lb/ft$^3$ (kg/m$^3$)</th>
<th>Porosity, % (ASTM D 7063)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-4-1</td>
<td>122.38 (1982.56)</td>
<td>9.90</td>
</tr>
<tr>
<td>D-4-2</td>
<td>115.75 (1875.15)</td>
<td>9.90</td>
</tr>
<tr>
<td>D-4-3</td>
<td>120.84 (1957.61)</td>
<td>11.10</td>
</tr>
<tr>
<td>D-4-4</td>
<td>119.27 (1932.17)</td>
<td>10.40</td>
</tr>
<tr>
<td>Average</td>
<td>119.56 (1936.87)</td>
<td>10.33</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>2.84 (46.00)</td>
<td>0.57</td>
</tr>
<tr>
<td>D-5-2</td>
<td>119.89 (1942.22)</td>
<td>13.00</td>
</tr>
<tr>
<td>D-5-3</td>
<td>118.47 (1919.21)</td>
<td>15.10</td>
</tr>
<tr>
<td>D-5-4</td>
<td>121.12 (1962.14)</td>
<td>10.30</td>
</tr>
<tr>
<td>Average</td>
<td>120.31 (1949.06)</td>
<td>12.80</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>1.45 (23.57)</td>
<td>2.41</td>
</tr>
</tbody>
</table>
The disk specimens from these mixtures after 8 freezing and thawing cycles are shown in Figures 6.1 and 6.2. The failure of the disks included chipping of the aggregates and cracks in the paste. Most of the specimens deteriorated considerably after 8 cycles, leaving only aggregate and paste debris as shown in Figure 6.2-b. Microscopic examination of the disk specimens found no evidence of AEA, even with the very high dose of air entraining admixture used in Mixture 2.

The change in mass for the disk specimens from Mixture 4 and Mixture 5 are shown in Table 6.3. and Figures 6.4 and 6.5 respectively. Mixture 4 specimens kept approximately 91% of the initial mass after 40 cycles, while Mixture 5, with twice the dose of AEA had, 96% of the initial mass. The dynamic elastic (Young’s) modulus for these disks and the percentage of original $E_d$ after 40 cycles are shown in Table 6.3. The $E_d$ for Mixture 4 and Mixture 5 after 40 cycles were 81% and 85% of their initial value respectively. Figure 6.3 shows the specimens from Mixture 4 and Mixture 5 after 40 cycles respectively.

Figures 6.6 and 6.7 show the change in $E_d$ every 10 cycles for each specimen of Mixture 4 and Mixture 5 respectively. ASTM C 666 considers “Failure” of a specimen if the $E_d$ is lower than 40% of the initial value by 300 cycles. Using this criteria, specimen D-4-2 failed after 20 cycles, even though the mass did not show any significant change. The three other specimens had similar losses in $E_d$. Disks from Mixture 5 had a slightly higher relative loss of $E_d$ than those from Mixture 4 after 40 cycles.
Figure 6.1. Mixture 1 and 2 Disk Specimens After Eight Freezing and Thawing Cycles
Figure 6.2. Mixture 1 and 2 Disk Specimens After Eight Freezing and Thawing Cycles
Figure 6.3. Mixture 4 and Mixture 5 Disk Specimens After 40 Freezing and Thawing Cycles
Table 6.3. Initial and Remaining Mass Every 10 Cycles of Disk Specimens Mixture 4 and Mixture 5

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Initial mass, lb (kg)</th>
<th>10 cycles</th>
<th>20 cycles</th>
<th>30 cycles</th>
<th>40 cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-4-1</td>
<td>1.00 (0.45)</td>
<td>99%</td>
<td>96%</td>
<td>94%</td>
<td>93%</td>
</tr>
<tr>
<td>D-4-2</td>
<td>1.03 (0.47)</td>
<td>98%</td>
<td>92%</td>
<td>91%</td>
<td></td>
</tr>
<tr>
<td>D-4-3</td>
<td>1.23 (0.56)</td>
<td>98%</td>
<td>94%</td>
<td>92%</td>
<td>90%</td>
</tr>
<tr>
<td>D-4-4</td>
<td>1.19 (0.54)</td>
<td>97%</td>
<td>95%</td>
<td>93%</td>
<td>90%</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>98%</td>
<td>94%</td>
<td>92%</td>
<td>91%</td>
</tr>
<tr>
<td>D-5-1</td>
<td>1.02 (0.46)</td>
<td>99%</td>
<td>98%</td>
<td>96%</td>
<td>96%</td>
</tr>
<tr>
<td>D-5-2</td>
<td>1.18 (0.53)</td>
<td>98%</td>
<td>97%</td>
<td>96%</td>
<td>96%</td>
</tr>
<tr>
<td>D-5-3</td>
<td>1.12 (0.51)</td>
<td>100%</td>
<td>99%</td>
<td>98%</td>
<td>98%</td>
</tr>
<tr>
<td>D-5-4</td>
<td>1.09 (0.49)</td>
<td>99%</td>
<td>97%</td>
<td>95%</td>
<td>95%</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>99%</td>
<td>98%</td>
<td>96%</td>
<td>96%</td>
</tr>
</tbody>
</table>

Table 6.4. Initial and Remaining $E_d$ Every 10 Cycles of Disk Specimens Mixture 4 and Mixture 5

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$E_d$, Mpsi (GPa)</th>
<th>10 cycles</th>
<th>20 cycles</th>
<th>30 cycles</th>
<th>40 cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-4-1</td>
<td>2.8 (17.5)</td>
<td>100%</td>
<td>97%</td>
<td>100%</td>
<td>84%</td>
</tr>
<tr>
<td>D-4-2</td>
<td>2.2 (14.0)</td>
<td>89%</td>
<td>31%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D-4-3</td>
<td>2.8 (17.5)</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>70%</td>
</tr>
<tr>
<td>D-4-4</td>
<td>2.5 (15.5)</td>
<td>99%</td>
<td>99%</td>
<td>94%</td>
<td>78%</td>
</tr>
<tr>
<td>Average</td>
<td>2.6 (16.5)</td>
<td>98%</td>
<td>85%</td>
<td>104%</td>
<td>81%</td>
</tr>
<tr>
<td>D-5-1</td>
<td>2.7 (17.0)</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>71%</td>
</tr>
<tr>
<td>D-5-2</td>
<td>2.5 (15.5)</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>82%</td>
</tr>
<tr>
<td>D-5-3</td>
<td>2.4 (15.0)</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>96%</td>
</tr>
<tr>
<td>D-5-4</td>
<td>2.8 (17.5)</td>
<td>100%</td>
<td>96%</td>
<td>99%</td>
<td>91%</td>
</tr>
<tr>
<td>Average</td>
<td>2.6 (16.5)</td>
<td>100%</td>
<td>99%</td>
<td>100%</td>
<td>85%</td>
</tr>
</tbody>
</table>
Figure 6.4. Mass Loss for Mixture 4 Specimens After Every 10 Cycles
Figure 6.5. Mass Loss for Mixture 5 Specimens After Every 10 Cycles
Figure 6.6. Dynamic Elastic (Young’s) Modulus Loss for Mixture 4 Specimens After Every 10 Cycles
Figure 6.7. Dynamic Elastic (Young’s) Modulus Loss for Mixture 5 Specimens After Every 10 cycles
6.2.2. Air Void Evaluation

Concrete is not considered to be adequately air entrained if the total air content is less than about 3%, the entrapped air content, for conventional concrete with a nominal maximum size aggregate size of \( \frac{1}{2} \) in. (12.5 mm) [ACI 201, 1992]. This is percent of the entire volume, however. The microscopic evaluation in this study considered only the paste fraction. The specified air content of concrete with lower w/c may be decreased by 1% (ACI 318, 2005). Since w/c ratio in pervious concrete is low, it may require less air than conventional concrete, however, these effects are not fully established and additional research is recommended.

Microscopic photographs shown the air entrained voids for Mixture 4 and Mixture 5 specimens are shown in Figures 6.8 and 6.9 respectively. Microscopic evaluation of Mixture 4 disk specimens found evidence of entrained air in the paste. Based on qualitative assessment, the quantity of entrained air voids was very low. The air content of Mixture 4 appeared to be less than what would be 2% in conventional concrete. The concrete would not be considered to have the minimum entrained air content required for frost durability of conventional concrete.

Mixture 5, which had twice the dose of air entraining admixture, had a larger quantity of entrained air voids in the paste. The air content of Mixture 5 appeared to be that would be 4% of conventional concrete, a minimally acceptable entrained air content.
Figure 6.8. Microscopic Photograph of Mixture 4 disk (50x)
Figure 6.9. Microscopic Photograph of Mixture 5 disk (50x)
The frost durability of Mixture 4, with a very low entrained air content, was found to be marginally acceptable based on $E_d$. The frost durability of Mixture 5, which had a higher entrained air void and was found to have acceptable based on $E_d$.

The frost durability of specimens from mixtures 4 and 5, and the catastrophic failure of specimens from mixtures 1 and 2 confirms the need for sand in pervious concrete. The sand may be required to establish and stabilize the entrained air in the mortar of the pervious concrete mixture. This observation is speculative, however, and additional research to fully establish the reasons for improvement of frost durability with the inclusion of sand is recommended.

The method used to assess frost durability has not been standardized but was used to provide assessment on frost resistance under more realistic conditions of temperature cycling, and saturation. The acceptable criteria for this test has not been fully established, however, and additional research is recommended.

### 6.3. Freezing and Thawing Resistance with Sand Sediments

The specimen from Mixture 4 tested with sand sediments infiltrating the disk failed within 8 cycles of freezing and thawing. The early failure of the disk specimen may be related to the minimum air entrained voids and marginal frost durability found with Mixture 4 specimens. Specimens from Mixture 5 did not show any significant change in mass or dynamic elastic
(Young’s) modulus, showing very similar results to those of Mixture 5 disk specimens. This disk kept 100% of both, the initial $E_d$ and mass after 10 cycles.

This finding indicates that the frost durability of otherwise frost resistance pervious concrete is not compromised by sand sediment, although clearly this study was limited and additional study would be useful.

### 6.4. Conclusions – Frost Resistance

The proposed test had more realistic freezing and thawing cycles, with freezing occurring in a deicing salt solution, simulating frost exposure conditions in a pavement and similar to the exposure in ASTM C 672. The results from the test largely confirmed earlier studies in which ASTM C 666 was used. Mass loss results were similar to those of Schaefer, Wang, Suleiman and Kevern (2006) when results are compared at 40 cycles. The mass loss percentage of the disk specimens appear to be slightly higher, however, since the debonding of an aggregate was a larger percentage of the smaller volume of the test specimen.

Microscopic examination showed that Mixture 4 probably had less than the minimum recommended entrained air content and Mixture 5 probably had the minimum recommended entrained air at most to provide frost resistance.
The rapidly deterioration of the specimens without any sand in their mixture confirmed the findings of previous studies that saturated or nearly saturated pervious concrete without any sand in the mixture is not frost resistance, even with air entraining admixture. The cement paste alone does not appear to be sufficient to develop the air entrained air voids required to protect the concrete to water expansion on freezing. These types of mixtures will not likely resist the effects of cold weather climates and different mixture proportions should be specified in areas with (1) significant freezing cycles and (2) for sites in which analysis indicates the pervious concrete will be saturated during periods of freezing.

The results of the freezing and thawing tests confirmed that the addition of 7% sand by weight as replacement for coarse aggregate increases the frost resistance of pervious concrete significantly and, when used with an adequate amount of AEA can provided adequate frost resistance as measured by the method used in this study.

Sedimentation appeared to have, at most, marginal resistance effects on frost resistance of pervious concrete, but results were less than conclusive.

**6.5. Recommendations**

Additional research is needed to determine the percentage of entrained air voids related to air entraining admixture and sand content in a low w/c pervious concrete mixture.
The $E_d$ with of pervious concrete cylinder specimens or disks will not likely be the same as the concrete in service and additional research is also needed to evaluate the effect of consolidation techniques on the relationship between $E_d$ and porosity.
7. Conclusions and Recommendations

7.1. Conclusions

7.1.1. Phase I and Phase II Conclusions

1. Sedimentation tests in Phases I and II confirmed that most sand sediments will be trapped on top of the concrete, however, a part of the finer sand fraction will be deposited within the concrete or travel through the concrete. The denser, less permeable surface acts as a coarse filter, passing small particles but trapping larger ones. This phenomenon can affect the apparent permeability of the PCPS by clogging the surface or near-surface region, and can be partially recovered by maintenance involving a vacuum sweeping. No exfiltration problem was found with this type of sediment.

2. Sedimentation test in Phase I and II confirmed that a layer of clayey silty sediments was deposited at the bottom of the concrete and retained by the filter fabric when exposed to clayey silt or clayey silty sand sediments. The layer was uniformly distributed at the bottom, instead of being deposited in a localized area, indicating that the sediments are maintained in suspension as the stormwater moves horizontally through the pervious concrete. The high flow rates possible in pervious concrete, at least those with a moderate porosity, with permeability much greater than 9.0 in./hr (0.001 cm/s), the velocity below which particles passing the #200 sieve size (75 µm)
settle in water, are sufficient to carry sediment horizontally. The high volume of solids used in the first phase and the repetitive application of more typical amounts of solids in the second phase both failed to create localized concentrations of deposits. Settlement occurred when vertical flow slowed significantly at the filter fabric. In practice, stormwater will retain fine sediments until horizontal equilibrium has been reached. These fine sediments will then be largely but not completely carried down until trapped by the filter fabric.

3. As expected, silty sand sediments segregated with larger size particles, that is, sand, trapped on or in the surface of the pervious concrete and finer grained sizes washing through to be trapped on the filter fabric.

4. The clayey silty sediments deposited at the bottom will not affect the exfiltration of the system when the PCPS rests on a compacted clayey silty soil, since the layer of sediment deposited will not be compacted. Storage capacity may be affected but the addition of at least one inch (25 mm) of coarse base will most likely account for this situation in almost all practical situations.

5. The permeability of this layer of fine sediments deposited at the bottom of the system can affect the effective exfiltration when the system rests on a silty sand subgrade. This effect must be estimated and considered in the design of the PCPS.
6. A simple test method was developed in this study to provide the Designer-of-Record with an estimate of the exfiltration value of the PCPS at EOS conditions specifically including the effects of sedimentation. This test was found to give values comparable to those found in one-dimensional testing in Phase I and II. The test method was a modification of the methods used in both phases.

7. Results from Phase I and Phase II were used to develop design guidelines for PCPS subjected to sedimentation. The guidelines complement the hydrological design of PCPS discussed in EB 303 (Leming, Malcom and Tennis, 2007), and are based on the results and analysis of the sedimentation tests discussed in this study.

7.1.2 Phase III Conclusions

1. The proposed test to evaluate frost resistance had relatively slow freezing and thawing cycles, with freezing occurring in a deicing salt solution, simulating frost exposure conditions in a pavement and similar to the exposure in ASTM C 672. The results from the test largely confirmed earlier studies in which ASTM C 666, with relatively rapid cycle was used. Mass loss results were similar to those of Schaefer, Wang, Suleiman and Kevern (2006) when results are compared at 40 cycles. The mass loss percentage of the disk specimens appear to be slightly higher, however, since the debonding of an aggregate was a larger percentage of the smaller volume of the test specimen.
2. Microscopic examination showed that Mixture 4 probably had less than the minimum recommended entrained air content and Mixture 5 probably had the minimum recommended entrained air, at most, to provide frost resistance.

3. The rapid deterioration of the specimens without any sand in the mixture confirmed the findings of previous studies that saturated or nearly saturated pervious concrete without any sand is not frost resistance, even with air entraining admixture. These types of mixtures will not likely resist the effects of cold weather climates and different mixture proportions should be specified in areas with (1) significant freezing cycles and (2) for sites in which analysis indicates the pervious concrete will be saturated during periods of freezing. E_d and mass loss show comparable results with previous studies.

4. The results of the freezing and thawing tests confirmed that the addition of 7% sand by weight as replacement for coarse aggregate increases the frost resistance of pervious concrete significantly and, when used with an adequate amount of AEA can provided adequate frost resistance as measured by the method used in this study.

5. Frost resistance of pervious concrete did not show signs to be affected by sand sediment.
7.2. Recommendations

1. Additional studies are strongly recommended to analyze the effects of consolidation on pervious concrete beam and cylinder consolidation. The consolidation method can directly affect the porosity and \( E_d \) of the specimens, not representing the properties of the pervious concrete in service. The \( E_d \) with of pervious concrete cylinder specimens or disks will not likely be the same as the concrete in service and additional research is also needed to evaluate the effect of consolidation techniques on the relationship between \( E_d \) and porosity.

2. Recovery of permeability is never complete with sediments containing fine grained particles. This observation is consistent with closure, or blocking, of small diameter voids that may be connecting much larger voids. Additional study of the relationships between void size distribution, porosity and sediment characteristics is recommended.

3. The sedimentation effects in this study were analyzed using specimens with an approximate 20% porosity. Additional research is recommended to study the effects with pervious concrete of lower and higher porosity, for example 10% to 30%.
4. Additional research is needed to determine the relationship between the percentage of entrained air voids, air entraining admixture dosage and sand content in a low w/c, pervious concrete mixture.
8. List of References


ACI Committee 201, *Guide for Making a Condition Survey of Concrete in Service*, 201.1R-92 American Concrete Institute, Farmington Hills, Michigan, 1992, 16 pages.


ACI Committee 522, *Pervious Concrete*, 522R-06, American Concrete Institute, Farmington Hills, Michigan, 2006, 25 pages.

ACI Committee 522, *Specification for Pervious Concrete Pavement*, 522.1-08, American Concrete Institute, Farmington Hills, Michigan, 2008, 7 pages.


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Appendices
Appendix A. Mechanical Properties

Modulus of rupture for Mixture 1 and Mixture 2 are shown in Table A.1. Specimen B-2-4 was not considered for the average of the Mixture 1 modulus of rupture. The specimen, which broke at a very low load, may have been damaged during handling, transportation or both. Table A.1 shows the flexural modulus for Mixture 3. The flexural modulus for the three mixtures were very similar, not showing any significant difference. Dynamic elastic (Young’s) modulus for Mixture 1 and Mixture 2 are also shown in Table A.1. The $E_d$ for the mixtures 1 and 2 were very similar, not showing any significant difference.

Compressive strength for all five mixtures are shown in Table A.2. Compressive strength were similar between mixtures, not showing any significant difference with each other. Mixture 3 showed the greater compressive strength with the greater variability.
Table A.1. Flexural and Dynamic Elastic (Young’s) Modulus for Beam Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Flexural modulus, psi (Mpa)</th>
<th>Dynamic Young’s Modulus of Elasticity, ksi (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mixture 1</td>
</tr>
<tr>
<td>B-1-1</td>
<td>440 (3.0)</td>
<td>1,600 (11.0)</td>
</tr>
<tr>
<td>B-1-2</td>
<td>415 (2.9)</td>
<td>1,600 (11.0)</td>
</tr>
<tr>
<td>B-1-3</td>
<td>355 (2.4)</td>
<td>1,600 (11.0)</td>
</tr>
<tr>
<td>B-1-4</td>
<td>125 (0.9)*</td>
<td>1,500 (10.0)</td>
</tr>
<tr>
<td>B-1-5</td>
<td>455 (3.1)</td>
<td>1,800 (12.5)</td>
</tr>
<tr>
<td>B-1-6</td>
<td>485 (3.3)</td>
<td>2,100 (14.5)</td>
</tr>
<tr>
<td>B-1-7</td>
<td>425 (2.9)</td>
<td>1,800 (12.5)</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>430 (3.0)</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>40 (0.3)</td>
<td>200 (1.5)</td>
</tr>
<tr>
<td>Mixture 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-2-1</td>
<td>370 (2.6)</td>
<td>2,400 (17.0)</td>
</tr>
<tr>
<td>B-2-2</td>
<td>460 (3.2)</td>
<td>2,000 (13.5)</td>
</tr>
<tr>
<td>B-2-3</td>
<td>410 (2.9)</td>
<td>1,900 (13.0)</td>
</tr>
<tr>
<td>B-2-4</td>
<td>510 (2.5)</td>
<td>2,200 (15.0)</td>
</tr>
<tr>
<td>B-2-5</td>
<td>485 (3.3)</td>
<td>1,900 (13.0)</td>
</tr>
<tr>
<td>B-2-6</td>
<td>450 (3.1)</td>
<td>1,800 (12.5)</td>
</tr>
<tr>
<td>B-2-7</td>
<td>505 (3.5)</td>
<td>2,100 (14.5)</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>455 (3.1)</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>50 (0.4)</td>
<td>210 (1.6)</td>
</tr>
<tr>
<td>Mixture 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-3-1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B-3-2</td>
<td>510 (3.5)</td>
<td>-</td>
</tr>
<tr>
<td>B-3-3</td>
<td>360 (2.5)</td>
<td>-</td>
</tr>
<tr>
<td>B-3-4</td>
<td>430 (3.0)</td>
<td>-</td>
</tr>
<tr>
<td>B-3-5</td>
<td>360 (2.5)</td>
<td>-</td>
</tr>
<tr>
<td>B-3-6</td>
<td>400 (2.8)</td>
<td>-</td>
</tr>
<tr>
<td>Average</td>
<td>410 (2.9)</td>
<td>-</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>60 (0.4)</td>
<td>-</td>
</tr>
</tbody>
</table>

*Not considered in average
Table A.2. Compressive Strength for The Five Mixtures

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mixture 1</th>
<th>Mixture 2</th>
<th>Mixture 3</th>
<th>Mixture 4</th>
<th>Mixture 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2,380 (16.4)</td>
<td>3,220 (22.2)</td>
<td>2,456 (16.9)</td>
<td>2,520 (17.4)</td>
<td>2,390 (16.5)</td>
</tr>
<tr>
<td>2</td>
<td>2,940 (20.3)</td>
<td>3,010 (20.8)</td>
<td>4,120 (28.4)</td>
<td>2,620 (18.1)</td>
<td>2,660 (18.4)</td>
</tr>
<tr>
<td>3</td>
<td>2,440 (16.8)</td>
<td>2,520 (17.4)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Average</td>
<td>2,590 (17.8)</td>
<td>2,920 (20.1)</td>
<td>3,290 (22.7)</td>
<td>2,570 (17.7)</td>
<td>2,530 (17.4)</td>
</tr>
<tr>
<td>Stad dev</td>
<td>310 (2.1)</td>
<td>360 (3.5)</td>
<td>1,180 (8.1)</td>
<td>200 (1.4)</td>
<td>70 (0.5)</td>
</tr>
</tbody>
</table>
Appendix B. Pervious Concrete Apparent Permeability Test (P-CAP Test)

On-site permeability can be estimated using a simple procedure that involves pouring a known volume of water from a relatively small container, at a moderate, constant flow rate on a selected location on the pervious concrete pavement surface and measuring the dispersion or spread of the water over the surface of the pavement. The apparent surface permeability of the pervious concrete pavement can be expressed as

\[
\mu_a = \frac{V_i}{A_p \cdot t}
\]  

(B.1)

where

\( \mu_a \) = apparent surface permeability of pervious concrete (in/h or cm/h),

\( V_i \) = volume of water poured onto the pavement (in\(^3\) or cm\(^3\)),

\( A_p \) = area of the puddle (in\(^2\) or cm\(^2\)), and

\( t \) = time to drain the water onto the pavement (h).

Derivation:

Applying the principle of mass balance, a volume of water (\( V_i \)), poured onto the pervious pavement surface from a relatively small container, will be equal to the volume of the resulting “puddle” (\( V_p \)) less the volume infiltrated through the surface into the pavement (\( V_o \)) (see Fig B.1), or:
\[ V_i = V_o + V_p \]  \hfill (B.2)

and, therefore:

\[
\frac{d}{dt} V_i = \frac{d}{dt} V_o \cdot \frac{d}{dt} V_p
\]  \hfill (B.3)

**Figure B.1.** Elevation and Plan View for On-site Permeability Index

\( V_i \) is given by the area \((A_h)\) and depth \((h)\) of the water inside the container:

\[ V_i = A_h \cdot h \]  \hfill (B.4)
The volume of water in the puddle \( (V_p) \) is the product of the area \( (A_p) \) and height \( (y) \) of the puddle. There is no expansion of the puddle at equilibrium, so the area of the puddle is the same as the area of penetration; the volume of water that infiltrates the pavement is given by the area of the puddle and the depth of penetration \( (Pen) \) of the water at that point in time.

\[
V_p = A_p \cdot y \quad \text{(B.5)}
\]

\[
V_o = A_p \cdot Pen \quad \text{(B.6)}
\]

The derivatives with respect to time of Equations B.4, B.5 and B.6 are:

\[
\frac{dV_i}{dt} = A_h \cdot \frac{dh}{dt} \quad \text{(B.7)}
\]

\[
\frac{dV_p}{dt} = A_p \cdot \frac{dy}{dt} + y \frac{dA_p}{dt} \quad \text{(B.8)}
\]

\[
\frac{dV_o}{dt} = A_p \cdot \frac{dPen}{dt} + Pen \cdot \frac{dA_p}{dt} \quad \text{(B.9)}
\]

Examining Equation B.8, the height of the puddle is approximately constant at a constant, moderate flow from the container, and the change in height of the puddle is very small, so:

\[
A_p \frac{dy}{dt} \approx 0 \quad \text{(B.10)}
\]
and therefore:

\[
\frac{dV_p}{dt} = y \frac{dA_p}{dt}
\]  

(B.11)

Examining Equation B.9, the instantaneous depth of penetration will be much lower than the rate of penetration \( (\mu) \), for any real pervious concrete pavement found in service:

\[
Pen \frac{dA_p}{dt} \ll \frac{A_p dPen}{dt}
\]  

(B.12)

Substituting \( \mu \), Equation (B.9) can therefore be given as:

\[
\frac{dV_o}{dt} = A_p \cdot \mu
\]  

(B.13)

Combining Equations B.7, B.11, and B.13, Equation B.3 can now be restated as:

\[
A_h \frac{dh}{dt} = y \frac{dA_p}{dt} + A_p \mu
\]  

(B.14)

Letting \( r \) be the average radius of the puddle,

\[
A_p = \pi \cdot r^2
\]  

(B.15)
and, therefore:

\[
\frac{dA_p}{dt} = 2\pi \cdot r \cdot \frac{dr}{dt}
\]  
(B.16)

and Equation (B.3) can be given as:

\[
A_h \frac{dh}{dt} = 2y\pi r \frac{dr}{dt} + A_p \mu_a
\]  
(B.17)

Let \( D \) be the average diameter of the puddle and \( d \) be the diameter of the container. Then \( D = 2r \), and:

\[
A_p = \frac{\pi D^2}{4}
\]  
(B.18)

and

\[
A_h = \frac{\pi d^2}{4}
\]  
(B.19)

Equation B.17 can be written as:

\[
\frac{\pi d^2}{4} \frac{dh}{dt} = y\pi r \frac{dr}{dt} + \frac{\pi D^2}{4} \mu_a
\]  
(B.20)

This equation can be simplified to:
\[ d^2 \cdot \frac{dh}{dt} = 4Dy \cdot \frac{dr}{dt} + \mu_a D^2 \]  

(B.21)

where,

- \( d \) = diameter of the container,
- \( \frac{dh}{dt} \) = change in head over time of the container,
- \( \mu_a \) = the apparent surface permeability of the pervious concrete,
- \( D \) = the diameter of puddle,
- \( y \) = depth of puddle, and
- \( \frac{dr}{dt} \) = horizontal velocity (rate of puddle spread) (all in consistent units)

In practical situations, the horizontal velocity of the water on top of the pervious concrete pavement, that is, the rate of the spread of the puddle, is much lower than the vertical velocity, that is, the rate of penetration (\( \mu_a \)), and, with a very small \( y \) (depth of the puddle) compared to \( D \):

\[ 4Dy \frac{dr}{dt} \ll \mu_a D^2 \]  

(B.22)

Equation (B.20) can now be given as:

\[ d^2 \frac{dh}{dt} \approx \mu_a D^2 \]  

(B.23)
The apparent permeability ($\mu_a$) can therefore be obtained as a function of the diameter of the puddle, the diameter of the container being used and the rate the water is poured on the pavement, or, solving Equation (B.23) for $\mu_a$:

$$\mu_a = \frac{d^2}{D^2} \frac{dh}{dt}$$  \hspace{1cm} (B.24)

With a relatively small depth or head ($h$) and a relatively high flow rate from the test container, the time derivative of the head in the container can be replaced by simple $h/t$. It is convenient to convert $d^2 h$ back to $V_i$ and $D^2$ back to $A_p$. The apparent surface permeability of the pervious concrete pavement can now be defined as:

$$\mu_a = \frac{V_i}{A_p \cdot t}$$  \hspace{1cm} (B.25)

where

- $\mu_a =$ apparent surface permeability of pervious concrete,
- $V_i =$ volume of water poured onto the pavement,
- $A_p =$ area of the puddle, and
- $t =$ time to drain the water from the test container onto the pavement,

with all values in consistent units. Typically, apparent surface permeability would be expressed in in./h or cm/h since those values are used in hydrologic design of pervious concrete pavements.
Appendix C. USLE Tables

The USLE was used to generate a set of tables of estimates the sediment production depending on the slope and the length of the slope, for sites with sand, clayey silt, or clayey silty sand and three land covers: pasture, bare soil, and an intermediate cover between these two. These tables may be used by the practicing engineer to rapidly determine a design value for amount of sediment on a given site. Figures 2.5 and 2.6; and Tables 2.5 and 2.6 where used to generate these tables.
**Universal Soil Loss Equation**

\[ A = R \times K \times LS \times C \times P \]

- **Land Cover**
  - Continuous fallow: 1.00
  - Undisturbed forest: 0.00
  - 70-75% grass cover: 0.05-0.10
  - 75-85% grass cover: 0.05-0.10
  - 90-100% grass cover: 0.05-0.10
- **Percentage of ground cover**
  - Grass: 0.012
  - Forage grass: 0.063
  - 50% brush, 40% grass cover: 0.012
  - Bare soil: 1.00
  - Undisturbed, not exposed: 1.00
  - Compacted, smooth: 1.00
  - Compacted, rough: 1.00
- **Slope**
  - Steep, higher than 15°: 1.00
  - Steep, lower than 15°: 0.60
  - Steep, lower than 15°: 0.60
  - Steep, higher than 15°: 0.60
  - Steep, lower than 15°: 0.60
  - Steep, higher than 15°: 0.60
  - Steep, lower than 15°: 0.60
- **Soil**
  - Sand and gravel: 0.15
  - Loamy sand soil, sand and loam soil: 0.17
  - Fine sandy loam: 0.22
  - Loamy, clay loam: 0.30
  - Silt loam and silt loam: 0.34
  - Clay: 0.45
- **Crop**
  - **Surface**
    - Field crops: 0.10
    - Hay crop, pasture: 0.10
    - Orchard, vineyard: 0.10
  - **Efficiency**
    - N/A

**USLE Sand Production in tons/ac·yr. for Regions with R = 100 (ft·tons·in)/(ac·hr·yr)**

- **Rainfall Intensity (in/hr)**
  - 10
  - 20
  - 40
  - 100
  - 200
  - 400
  - 800
  - 1000
  - 2000

**Sand**

- **Rainfall Intensity**
  - **Surface**
    - Field crops: 0.10
    - Hay crop, pasture: 0.10
    - Orchard, vineyard: 0.10
  - **Efficiency**
    - N/A

**Soil**

- **Surface**
  - Sand and gravel: 0.15
  - Loamy sand soil, sand and loam soil: 0.17
  - Fine sandy loam: 0.22
  - Loamy, clay loam: 0.30
  - Silt loam and silt loam: 0.34
  - Clay: 0.45
- **Efficiency**
  - N/A

**Crop**

- **Surface**
  - Field crops: 0.10
  - Hay crop, pasture: 0.10
  - Orchard, vineyard: 0.10
  - **Efficiency**
    - N/A

**Figure C.1. USLE Sand Production in tons/ac·yr. for Regions with R = 100 (ft·tons·in)/(ac·hr·yr)**
Figure C.2. USLE Sand Production in tons/acre·yr for Regions with $R = 150$ (ft·tons/in)/(ac·hr·yr)
Figure C.3. USLE Sand Production in tons/ac·yr. for Regions with R = 200 (ft·tons-in)/(ac·hr·yr)
Figure C.4. USLE Sand Production in tons/ac·yr. for Regions with R = 250 (ft·tons-in)/(ac·hr·yr)
Figure C.5. USLE Sand Production in tons/ac-yr. for Regions with R = 300 (ft⋅tons/in)/(ac-hr-yr)
Figure C.6. USLE Sand Production in tons/ac·yr. for Regions with R = 350 (ft·tons-in)/(ac·hr·yr)

<table>
<thead>
<tr>
<th>Soil Slope</th>
<th>High</th>
<th>Medium</th>
<th>Low</th>
<th>Very Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clayey soil</td>
<td>0.80</td>
<td>0.70</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>Loamy sand</td>
<td>0.90</td>
<td>0.85</td>
<td>0.80</td>
<td>0.75</td>
</tr>
<tr>
<td>Sandy loam</td>
<td>0.95</td>
<td>0.90</td>
<td>0.85</td>
<td>0.80</td>
</tr>
<tr>
<td>Silty loam</td>
<td>1.00</td>
<td>0.95</td>
<td>0.90</td>
<td>0.85</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>High</th>
<th>Medium</th>
<th>Low</th>
<th>Very Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clayey soil</td>
<td>0.80</td>
<td>0.70</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>Loamy sand</td>
<td>0.90</td>
<td>0.85</td>
<td>0.80</td>
<td>0.75</td>
</tr>
<tr>
<td>Sandy loam</td>
<td>0.95</td>
<td>0.90</td>
<td>0.85</td>
<td>0.80</td>
</tr>
<tr>
<td>Silty loam</td>
<td>1.00</td>
<td>0.95</td>
<td>0.90</td>
<td>0.85</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Universal Soil Loss Equation</th>
<th>R (ft tons in/ac·hr·yr)</th>
<th>V (in)</th>
<th>K</th>
<th>A (ft tons in/ac·yr)</th>
<th>P (ft yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Universal soil loss equation</td>
<td>R (ft tons in/ac·hr·yr)</td>
<td>V (in)</td>
<td>K</td>
<td>A (ft tons in/ac·yr)</td>
<td>P (ft yr)</td>
</tr>
<tr>
<td>Universal soil loss equation</td>
<td>R (ft tons in/ac·hr·yr)</td>
<td>V (in)</td>
<td>K</td>
<td>A (ft tons in/ac·yr)</td>
<td>P (ft yr)</td>
</tr>
<tr>
<td>Universal soil loss equation</td>
<td>R (ft tons in/ac·hr·yr)</td>
<td>V (in)</td>
<td>K</td>
<td>A (ft tons in/ac·yr)</td>
<td>P (ft yr)</td>
</tr>
<tr>
<td>Universal soil loss equation</td>
<td>R (ft tons in/ac·hr·yr)</td>
<td>V (in)</td>
<td>K</td>
<td>A (ft tons in/ac·yr)</td>
<td>P (ft yr)</td>
</tr>
<tr>
<td>Universal soil loss equation</td>
<td>R (ft tons in/ac·hr·yr)</td>
<td>V (in)</td>
<td>K</td>
<td>A (ft tons in/ac·yr)</td>
<td>P (ft yr)</td>
</tr>
</tbody>
</table>
Figure C.7. USLE Clayey Silt Production in tons/ac-yr. for Regions with R = 100 (ft-tons in)/(ac-hr-yr)
Figure C.8. USLE Clayey Silt Production in tons/ac-yr. for Regions with $R = 150$ (ft⋅tons-in)/(ac-hr-yr)
Figure C.9. USLE Clayey Silt Production in tons/ac·yr. for Regions with R = 200 (ft·tons/in)/(ac·hr·yr)
Figure C.10. USLE Clayey Silt Production in tons/ac-yr. for Regions with R = 250 (ft·tons/in)/hr·yr
### Universal Soil Loss Equation

\[ A = R \times K \times S \times C \times P \]

<table>
<thead>
<tr>
<th>Land Cover</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous fallow</td>
<td>1.00</td>
</tr>
<tr>
<td>Undisturbed forest</td>
<td>0.000001</td>
</tr>
<tr>
<td>70-35% canopy cover, 150-60% stubble cover</td>
<td>0.000001</td>
</tr>
<tr>
<td>35-15% canopy cover, 85-75% stubble cover</td>
<td>0.000001</td>
</tr>
</tbody>
</table>

| Bare soil | 0.32 |
| Grass | 0.84 |
| Silt | 0.43 |

### Soil Erodibility (K)

- **Sand and gravel:** 0.10
- **Loamy fine sand, sand, and fine sand:** 0.15
- **Loamy fine sand and loamy sand:** 0.17
- **Loamy sand and clay loam:** 0.22
- **Silt loam and silty clay loam:** 0.34
- **Clayey silt and clay:** 0.43

### Figure C.11. USLE Clayey Silt Production in tons/ac⋅yr. for Regions with R = 300 (ft⋅tons/in)/(ac⋅hr⋅yr)

#### Permanent pasture

<table>
<thead>
<tr>
<th>R (ft tons/in/hr⋅yr)</th>
<th>10</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>100</th>
<th>200</th>
<th>400</th>
<th>600</th>
<th>1000</th>
<th>2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>0.22</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.84</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil loss (tons/ac⋅yr)</td>
<td>0.00</td>
<td>0.01</td>
<td>0.01</td>
<td>0.02</td>
<td>0.04</td>
<td>0.06</td>
<td>0.09</td>
<td>0.11</td>
<td>0.16</td>
<td>0.22</td>
</tr>
</tbody>
</table>

#### Bare soil

<table>
<thead>
<tr>
<th>R (ft tons/in/hr⋅yr)</th>
<th>10</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>100</th>
<th>200</th>
<th>400</th>
<th>600</th>
<th>1000</th>
<th>2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>0.80</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>1.10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil loss (tons/ac⋅yr)</td>
<td>0.00</td>
<td>0.01</td>
<td>0.01</td>
<td>0.02</td>
<td>0.04</td>
<td>0.06</td>
<td>0.09</td>
<td>0.11</td>
<td>0.16</td>
<td>0.22</td>
</tr>
</tbody>
</table>
Figure C.12. USLE Clayey Silt Production in tons/ac·yr. for Regions with R = 350 (ft·tons/in)/(ac·hr·yr)
Figure C.13. USLE Clayey Silty Sand Production in tons/acre-yr. for Regions with R = 100 (ft·tons-in)/(ac·hr·yr)
### Universal Soil Loss Equation

\[ A = R \times K \times LS \times C \times P \]

**Land Cover**
- C: 1.00
  - Critical falling
  - Undisturbed forest land
    - 50-100% native cover, 100-50% native cover: 0.957
    - 50-75% native cover, 75-25% native cover: 0.996
    - 25-5% native cover, 5-0% native cover: 0.995
- Permanent pasture and brush cover
  - 6% grass, 30% ground cover
    - grass: 0.63
    - vines: 0.42
    - 50% brush, 30% grass cover: 0.912
- Bare soil
  - Undisturbed except tilled
    - 1.3 0.05-1.0
  - Compacted, root-eroded
    - 1.0 0.0-1.0
  - Disk tillage, farm
    - 1.0 0
  - Disk tillage, after one rain
    - 0.5
  - Straw mulch, 0.5 tons/acre
    - 0.5
  - Straw mulch, 2.8 tons/acre
    - 0.15
  - Straw mulch, 4.6 tons/acre
    - 0.02

### Silty Sand

<table>
<thead>
<tr>
<th>Soil</th>
<th>Soil erodibility (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand and gravel</td>
<td>0.10</td>
</tr>
<tr>
<td>Loamy coarse sand, sand and fine sand</td>
<td>0.15</td>
</tr>
<tr>
<td>Loamy fine sand and loamy sand</td>
<td>0.17</td>
</tr>
<tr>
<td>Fine sandy loam and sandy loam</td>
<td>0.22</td>
</tr>
<tr>
<td>Loam, silt loam and sandy clay loam</td>
<td>0.30</td>
</tr>
<tr>
<td>Silt loam and silt loam</td>
<td>0.34</td>
</tr>
<tr>
<td>Clay loam and silt loam</td>
<td>0.45</td>
</tr>
<tr>
<td>Clay and silt clay</td>
<td>0.50</td>
</tr>
</tbody>
</table>

### Figure C.14. USLE Clayey Silty Sand Production in tons/ac-yr for Regions with R = 150 (ft·tons-in)/(ac·hr·yr)

**Universal Soil Loss Equation**

\[ A = R \times K \times LS \times C \times P \]

**Slope Length (ft)**

<table>
<thead>
<tr>
<th>10</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
<th>200</th>
<th>400</th>
<th>600</th>
<th>800</th>
<th>1000</th>
<th>2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5%</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1%</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2%</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3%</td>
<td>0</td>
<td>0</td>
<td>0</td>
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Figure C.15. USLE Clayey Silty Sand Production in tons/ac-yr. For Regions with R = 200 (ft⋅tons/in)/(ac⋅hr⋅yr)
Figure C.16. USLE Clayey Silty Sand Production in tons/ac-yr. for Regions with $R = 250$ (ft-toms in)/(ac-hr-yr)
Figure C.17. USLE Clayey Silty Sand Production in tons/ac-yr. for Regions with \( R = 300 \) (ft- tons-in)/(ac-hr-yr)
Figure C.18. USLE Clayey Silty Sand Production in tons/ac-yr. for Regions with R = 350 (ft·tons/in)/(ac·hr·yr)