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Clogging of pervious concrete pile caused by soil piping: An approximate experimental study

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ABSTRACT: Due to their high permeability and high strength, pervious concrete piles (PCPs) can improve the ground bearing capacity. However, clogging of pervious concrete in practice is a potential disadvantage. To investigate the clogging mechanism of PCPs due to sand piping, a series of laboratory simulation tests is conducted on a developed hydraulic conductivity test system. It demonstrates the effects of pervious concrete porosity, grading of fine movable particles, mix ratio of skeleton particles to movable particles, relative density of soil, and distance between PCPs on PCP clogging. The experimental test results show that the hydraulic conductivity of PCP decreases for approximately 70 min and then becomes relatively stable. In addition, it is observed that PCP clogging rarely occurs in cases of low pervious concrete porosity, small movable sand particle size, high sand relative density, and large pile distance. The results also show that measurement of electrical conductivity can be an alternative method of hydraulic conductivity measurement. Based on the test results, preliminary clogging models are proposed.

KEYWORDS: Pervious concrete pile; Clogging; Soil piping; Permeability

45 Introduction

46 In recent years, composite foundation technologies have been widely used to enhance the bearing
47 capacity of foundations and reduce the settlement and liquefaction potential of the ground (Ariyaratne
48 et al. 2013; Haldar and Babu 2010; Shen and Xu 2011). Vertical reinforcements of composite
49 foundations may be divided into granular piles and rigid piles (Shen et al. 2013, 2016, 2017). Granular
50 piles have been widely used in engineering, as they can accelerate the rate of consolidation and reduce
51 the liquefaction potential of a sand or silt foundation (Hughes and Withers 1974; Lee and Pande 1998;
52 Ferreira Pinto and Delgado Rodrigues 2008; Poorooshab and Meyerhof 1997). However, the stiffness
53 and strength of granular piles are low and are greatly related to the confining pressure of the
54 surrounding soil (Guetif et al. 2007). When soft clay, organic soil, or peat soil is subjected to vertical
55 loading and granular piles are applied, the shallow part of granular piles is prone to fail by bulging into
56 the surrounding soil. This limits their use in soft clay, organic soil, or peat soil (Suleiman et al. 2014).
57 When subjected to lateral loading, granular piles fail in direct shear at the location of loading or along
58 the slope failure surface (Ni et al. 2016). Although rigid piles, such as low-grade concrete piles and
59 cement fly-ash gravel piles, can overcome the weak bonding problem of granular piles (Sariosseiri and
60 Muhunthan 2009; Le Hello and Villard 2009; Zheng et al. 2008; Jia et al. 2011), they have poor
61 permeability and induce a small foundation consolidation rate.

62 Recently, an innovative ground improvement method that utilizes pervious concrete piles (PCPs)
63 was proposed by Suleiman et al. (2011), who developed an installation process and a creative method
64 to investigate the soil–structure interaction under vertical and lateral loadings (Suleiman et al. 2014;
65 Lin et al. 2016; Ni et al. 2016). Pervious concrete, also referred to as porous concrete, is a mixture of
66 Portland cement, gap-graded aggregate, and water with or without a small amount of fine aggregate.

67 Many connective pores exist within the aggregate skeleton. Generally, the porosity of pervious
68 concrete is between 15% and 25%, and the hydraulic conductivity is typically between 2 and 6 mm/s
69 but can be as high as 10 mm/s (Tennis et al. 2004; Montes et al. 2005; Luck et al. 2006). Because of its
70 high hydraulic conductivity, pervious concrete can also provide a compressive strength between 3.5
71 and 28 MPa (Schlüter and Jefferies 2002). Therefore, piles made of pervious concrete offer fast
72 drainage and have a high bearing capacity, i.e., PCPs have the advantages of both granular piles and
73 rigid piles (Zhang et al. 2016). The fast drainage capacity of PCPs could not only accelerate the
74 dissipation of excess pore water pressure and subsoil consolidation but also reduce the
75 post-construction settlement of an upper construction, such as a road embankment. However, fine
76 particles from the soil matrix can progressively get into a PCP via the seepage flow induced by excess
77 pore water pressure and clogged PCP pores, leading to a gradual reduction of the PCP hydraulic
78 conductivity.

79 Clogging is a common phenomenon that has occurred in many drainage projects. Several studies on
80 the clogging of permeable materials such as gravel and pervious concrete have been performed
81 (Jimenez et al. 1990; Kayhanian et al. 2012; Deo et al. 2010; Neithalath et al. 2006; Haselbach 2006;
82 Haselbach et al. 2010; Reddi et al. 2000; Reddi et al. 2005). The variation of the hydraulic
83 conductivity of pervious concrete pavement with time was studied by Jimenez and Perez (1990).
84 Kayhanian et al. (2012) assessed the clogging of pervious concrete pavements in parking lots via the
85 falling head method using a National Center for Asphalt Technology device in the field. To evaluate
86 the contribution of pores to clogging susceptibility, Deo et al. (2010) developed a falling head
87 hydraulic conductivity cell to simulate clogging cycles. For each clogging cycle, 25 g of sand was
88 initially spread evenly over the specimen surface, and a hydraulic conductivity test was conducted at

the end. Such experiments were repeated until further sand additions did not result in noticeable changes in hydraulic conductivity. In addition to the process of clogging development, clogging criteria have also been studied (Ghidaglia et al. 1996; Narayan et al. 1997). Tan et al. (2003) derived an empirical theoretical formulation from the Kozeny–Carman equation to predict the reduction in hydraulic conductivity of a permeable road base.

Similar to PCPs, prefabricated vertical drains (PVDs) are used to accelerate the consolidation process and rapidly increase the strength and stiffness of soil. Many investigations have demonstrated that fine particles in the soil could be entrapped within the fibers of a filter, i.e., the geotextile, surrounding a PVD. If the pores of a filter sleeve become clogged, the discharge capacity of the PVD decreases, and the consolidation process is consequently impeded (Rollin and Lombard 1988; Basu and Madhav 2000).

Currently, few investigations on PCP clogging have been reported (Lin et al. 2016), although the issue of clogging has been considered since the technique was first proposed. There are two possible reasons for PCP clogging. First, during the installation of a PCP via vibratory construction methods, the natural subsoil adjacent to the pile may experience local liquefaction due to strong vibration induced by the migration of excess pore pressure fine soil particles into the piles during pervious concrete hardening (Zou et al. 2010; Adalier and Elgamel 2004). Second, clogging can occur during the operation period, the mechanism of which is shown in Fig. 1. The upper load can cause excess pore water pressure to develop in the layer where soil piping can occur. Because of the poor drainage capacity of the upper and lower clay layers, the excess pore water pressure is mainly dissipated by the permeable piles. For a non-cohesive soil, a micro bridge network can form near the contact surface with contact piping, as shown in Fig. 2 (Rollin and Lombard 1988). At first, large particles are stopped

at the contact surface. In turn, these particles retain smaller particles to form a dense filter zone, and this process continues until the soil stabilizes. The fine particles in the bridging network migrate into the drain. Once soil piping occurs, the permeable piles may become clogged by soil particles (Boulanger et al. 1998).

This study focuses on an approximate experimental simulation of PCP clogging caused by soil piping during the operation period. A physical clogging simulation system combined with hydraulic conductivity and electrical conductivity measurements of pervious concrete is developed. A series of laboratory simulation tests is conducted using this system to demonstrate the effects of PCP porosity, grading of fine movable soil particles, mix ratio of skeleton soil particles to movable soil particles, relative soil density, and distance between PCPs on clogging. The clogging mechanism of a soil–PCP system is investigated, and a preliminary clogging model is proposed.

Development of PCP clogging simulation system

No standard test procedure for measuring and evaluating the hydraulic conductivity of pervious concrete caused by soil piping is available. The existing testing apparatuses cannot constantly record the continuous variation in hydraulic conductivity of pervious concrete specimens. To investigate the reduction in PCP hydraulic conductivity due to clogging caused by soil piping, a simulation cell that can continually record the hydraulic conductivity changes is necessary. Based on the simulation system mentioned in a previous study (Cui et al. 2016), a system is designed (Fig. 3). To approximately simulate the real situation in the composite foundation (as shown in Fig. 4), the sand column and pervious concrete specimen are indicated as the zone in the red frame shown in the figure after a clockwise rotation of 90°. The height of the pervious concrete specimen is equal to the radius of the pervious concrete pile, and the height of the sand column is equal to half of the pile distance minus

the pile radius. The downward flow is used in this study for two reasons. First, it is believed that the downward flow simulates the most adverse conditions, because in this case, gravity on the particles acts in the same direction as the seepage or drag force (Lafleur 1984). Second, migrations of gravity-driven particles are rather weak compared with migrations of particles induced by seepage forces (Tomlinson and Vaid 2000). The functions of the developed simulation system mainly include the following: (1) avoiding leakage on the pervious concrete specimen sidewalls during the hydraulic conductivity tests by using a waterproof daub and flexible rubber cushion (Cui et al. 2015); (2) simulating the PCP clogging caused by soil piping; (3) recording the continuous variation in hydraulic conductivities for both the pervious concrete and soil column; and (4) simultaneously monitoring the change in pervious concrete electrical conductivity during the clogging process. All the measurement modules employed have been calibrated.

Continuous recording of hydraulic conductivity changes during the clogging process

As shown in Fig. 3, two water pressure sensors and an ultrasonic velocity meter are installed to record the change in hydraulic conductivity of both the pervious concrete and soil column during the clogging process in real time. All the sensors are connected to a computer by an A/D converter. To measure the hydraulic conductivity of pervious concrete, the head loss between the upper and bottom surfaces of a specimen is acquired from the water pressure sensors, and the velocity of the water in the pipes is measured by the ultrasonic velocity meter. The hydraulic conductivity of the specimen is calculated through the method presented by Cui et al. (2015).

Continuous recording of change in electrical conductivity during clogging process

Electrical conductivity has already been used to characterize the porosity of soil, rock, and concrete (Neithalath et al. 2006; Samouëlian et al. 2005; Turesson 2006; Zhang et al. 2014). The electrical

conductivity and hydraulic conductivity are employed together to characterize the clogging process of pervious concrete.

Because fresh water is not a suitable conductor of electricity to form a closed circuit in the saturated pervious concrete specimen, 10% (Li et al. 2014) NaCl solution with a density of 1.070 g/cm³ is used as the circulating fluid in the continuous physical simulation system for clogging. Therefore, the density of salt water is used in the calculation of water head loss. As shown in Fig. 3, two copper meshes (13) are placed at both sides of the specimen as electrodes and are connected to an AC power (15) of 12 V. The pore size of the copper meshes is 10 mm. The changes in current and power are measured continually by a multimeter (14) connected to the computer (18) via the A/D converter (17). Therefore, the electrical conductivity of the pervious concrete specimen can be calculated as follows:

$$(1) \quad \sigma = \frac{A^2 L_p}{W S}$$

where σ is the electrical conductivity, W is the power, A is the current, S is the cross-sectional area of the specimen, and L_p is the length of specimen.

Test materials

Pervious concrete

Pervious concrete mixtures are prepared using ordinary Portland cement and coarse aggregates. The aggregates are composed of limestone rubble with particle sizes ranging from 4.75 mm to 9.5 mm. The physical indexes are listed in Table 1. A sulphamate series superplasticizer is used to decrease water usage. Its dosage contributes 0.8% of the cement mass. Three mixtures are prepared for three target porosities: 15%, 20%, and 25%. The mix design method used in this study is the same as that presented by Cui et al. (2015). The mix proportions of pervious concrete per cubic meter are given in Table 2.

The mixtures are prepared using a laboratory mixer, and specimens are cast in $\phi 100 \text{ mm} \times 100 \text{ mm}$ cylindrical molds and demolded after 24 h. All specimens are placed in a standard curing room for 28 days. When the porosity values are 15%, 20%, and 25%, the average values of compressive strength are 20.7, 18.1, and 14.3 MPa, and the average initial hydraulic conductivities are 0.3, 0.5, and 1.1 cm/s respectively. The properties of pervious concrete are similar to the published results (Suleiman et al. 2014; Ni et al. 2016).

Sand column

The sand column in which piping can occur is a mixture of skeleton particles and movable particles (Tables 3 and 4). The skeleton is composed of coarse gravel with particle sizes ranging from 4.75 mm to 9.5 mm. The movable particles include four types of fine sand with particle sizes ranging from 0.3 mm to 4.75 mm. Based on the criterion given by the Nanjing Hydraulic Research Institute of China (Liu 2006), piping can occur for all mixtures.

Test procedure and scheme

Test procedure

With the advantage of a newly developed device, a clogging test of pervious concrete is carried out according to the following procedure:

- 1) The pervious concrete specimens, after curing for 28 days in a standard curing room, are first submerged in 10% NaCl solution for 24 h. After the surface of a specimen is wiped, petroleum jelly is daubed on the side and the specimen is covered with a flexible rubber cushion (2 in Fig. 3). Next, specimens are installed in a polymethyl methacrylate (PMMA) sleeve (3 in Fig. 3) and mounting bolts are screwed in. Then, the sand mixture is added layer by layer above the pervious concrete specimen.

To ensure uniformity, the thickness of a single layer is set approximately 20 mm and each layer is compacted manually based on its relative density (Fig. 3).

2) A 10% NaCl solution is prepared as the circulating fluid. The valve (7 in Fig. 3) is turned off while the pump (11 in Fig. 3) is turned on. Fluid is dumped evenly into the PMMA sleeve and the water level in the sleeve is determined based on the height of the outflow (5 in Fig. 3). The sand and pervious concrete are immersed in the circulating fluid for 1 h so that the influence of bubbles in the sand or specimen can be minimized.

3) The valve (7 in Fig. 3) is turned on; fluid infiltrates through the pervious concrete and is discharged (8 in Fig. 3). Meanwhile, the AC power (15 in Fig. 3) is turned on to apply 12 V to the wire netting, which covers both ends of the specimen. Simultaneously, the sensors start recording the water pressure (by 16 in Fig. 3), flow rate (by 12 in Fig. 3), and electric current and power (by 14 in Fig. 3).

4) The clogging test continues until the flow rate of the specimen becomes relatively stable (the process lasts approximately 70 min).

5) After the test ends, the sand above the specimen is taken out layer by layer in increments of 90 mm. The sand is then dried and sieved to analyze the change in gradation.

Test cases

In this research, characterization methods are used to investigate the influence of pile distance, porosity, movable particle size, sand mixing proportion, and relative density on clogging. Three sand column heights are selected, namely 270 mm, 360 mm, and 450 mm (Cases 10, 3, and 12 in Table 5, respectively), i.e., the half pile distances are 370, 460, 550 mm respectively. Therefore, the values of

area replacement ratios are 0.06, 0.04, and 0.03 respectively. The height of the outflow controls the pore pressure in the composite foundation. Because the excess pore water pressure difference in the composite foundation of the Yellow River Delta is approximately 5 kPa between the subsoil at the center of two piles and the PCP at a depth of 5.5 m (Zhang et al. 2016), the height of the water level in this study is kept at 500 mm with equal proportion in real condition. The porosities of the specimens are set as 15%, 20%, and 25% (Cases 1, 2, and 3 in Table 5, respectively). M1, M2, M3, and M4 (Cases 4, 3, 5, and 6 in Table 5, respectively) are used as movable particles. The mixing proportions of movable and skeleton particles of 1:3, 1:4, and 1:5 are tested (Cases 7, 3, and 8 in Table 5, respectively). Three relative densities are selected, namely 0.3, 0.6, and 0.9 (Cases 9, 3, and 10 in Table 5, respectively). Cases 13 to 20 are the orthogonal test design supplement. For each case, three samples are tested. The initial hydraulic conductivity values of sand columns are also given in Table 5.

Experimental results and analysis

Influence of pervious concrete porosity on clogging

The porosity of the pervious concrete specimen is set as the independent variable in this test. The time history curves of the average normalized hydraulic conductivities (NP, acquired by dividing the instantaneous hydraulic conductivity by the initial value) of the sand columns are shown in Fig. 5(a). For all the three samples, the hydraulic conductivities of the sand columns decrease rapidly in the initial test phase. Twenty minutes later, the change in hydraulic conductivity slows down and becomes relatively stable after 40 min. After 70 min, the decrease in hydraulic conductivity is more obvious because the porosity is smaller. For specimens with porosity values of 15%, 20%, and 25%, the final average hydraulic conductivities are 0.28, 0.46, and 0.51 times the initial values, respectively. The

239 presence of noise in the curves are caused by the following. First, injecting water into the physical
240 simulation system causes pressure oscillation on the specimen surface. Second, the environment and
241 electrical circuits can cause noise.

242 When piping occurs, large particles are initially trapped at the contact surface. Then these particles
243 retain small particles, forming a dense filter zone. This process continues until the soil is stabilized.
244 The stability of the soil hydraulic conductivity in Fig. 5(a) depends on the stabilities of the soil
245 particles. On one hand, the dense filter zone decreases the hydraulic conductivity of the soil. On the
246 other hand, the transport of soil particles from the soil column increases the soil hydraulic
247 conductivity. The average soil hydraulic conductivity after stabilization is a comprehensive reflection
248 of two contradictory aspects. A specimen with large porosity has a greater seepage flow velocity,
249 which can form a large bridging network and reduce the filter zone. Therefore, the average soil
250 hydraulic conductivity will increase (as shown in Fig. 5(a)).

251 Fig. 5(b) shows the NP time history curves of pervious concrete specimens with different porosities.
252 Generally, there is no obvious change in the hydraulic conductivity of pervious concrete at the initial
253 phase. The decline starts approximately 20 min later, the same time at which the hydraulic
254 conductivity of the sand column becomes relatively stable (Fig. 5(a)). This phenomenon occurs
255 because movable particles in sand require a certain amount of time to reach the interface between the
256 sand column and the pervious concrete specimen. They later migrate into the pore structure via
257 seepage flow and result in clogging. Forty minutes after the test begins, the slope of the hydraulic
258 conductivity decreases. After 70 min, the hydraulic conductivity of pervious concrete becomes stable.
259 However, for the specimen with porosity of 20%, the NP value increases at the beginning of the
260 measurement. This could be the result of the undersaturation and the presence of bubbles. Along with

the flow, bubbles that are driven out of the system could cause an increase in the hydraulic conductivity.

Fig. 5(c) shows the normalized electrical conductivity (NEC, obtained by dividing the instantaneous electrical conductivity by the initial value) time history curves for specimens with different porosities. The variation tendency of the electrical conductivity is generally the same as that of the hydraulic conductivity. Compared with the measurement of permeability, electrical conductivity measurement is much more convenient. Therefore, the measurement of electrical conductivity can be an alternative method of hydraulic conductivity measurement. According to Figs. 5(b) and 5(c), specimens with large porosity are more easily clogged. This phenomenon occurs because the pore size and seepage velocity are larger in specimens with large porosity. Thus, more particles can migrate into the pore system via seepage flow, which increase the possibility of particles clogging the narrow part of the pathway.

After the tests, the sand layer is removed in layers (each layer is 90 mm high), and then dried and sieved to analyze the change in gradation (as indicated in Table 6). The proportion of fine particles in the bottom layer is greater than that in the upper layers for Cases 1–3. This is because movable particles carried via seepage flow move to the sand–pervious concrete interface. With increasing pervious concrete porosity, the proportion of movable particles in the bottom layer increases as well.

Influence of movable particle size on clogging

Three types of sand mixtures are used in investigating the influence of movable particle size on clogging. The movable particles are denoted as M and the skeleton particles are denoted as S (Table 5). The proportion of skeleton to movable particles is 4:1, the relative density is 0.6, the porosity of pervious concrete is 25%, and the height of the sand column is 360 mm.

The time history curves of the average NP of sand columns with different movable particles are shown in Fig. 6(a). When the clogging processes reach equilibrium, the average hydraulic conductivity of the sand column clearly decreases in M1 (Case 4) and M2 (Case 3) with movable particles, particularly for Case 4 because the M1 movable particles in Case 4 are well-graded sand. Some of the large particles and fine particles in M1 can together more easily form a stable micro bridge network with seepage flow, as shown in Fig. 7(a). The micro bridge increases the resistance to movement of fine particles and accelerates the formation of the filter zone. For M3 (Case 5) and M4 (Case 6), the hydraulic conductivities of the sand columns increase during the test. This phenomenon occurs because these two types of movable particles mainly comprise fine sand that has difficulty in forming the bridge network. A mass of fine sand particles can pass through the sand-pervious concrete interface and migrate into the pores of the pervious concrete (Fig. 7(b)).

Fig. 6(b) shows the NP time history curves of pervious concrete specimens for different movable particles. The hydraulic conductivities of pervious concrete are also different for the four cases. In the initial test stage, the hydraulic conductivities decrease quickly for M3 and M4. Different from M1 and M2, the movable particles in M3 and M4 are mainly composed of fine particles. The fine particles can be quickly transported into the pervious concrete after the test starts and partial inner pores in the pervious concrete are clogged rapidly. As observed, the final hydraulic conductivity of pervious concrete for Case 6 (M4) is the largest. The reason for this is that M4 particles can easily pass through most of the pervious concrete pore channels.

Table 7 presents the particle size distributions of each sand layer after completion of the tests. The proportion of fine particles (less than 0.6 mm) in the bottom layer is greater than that in the upper layers for Cases 3, 4, 5, and 6. Note that for Case 4 (M1), because it is a well-graded mixture with

sizes between 1.18 and 4.75 mm, their incipient motion is more difficult compared with the fine movable particles. Therefore, most of them migrate slowly, resulting in a higher proportion at the top and lower proportion at the bottom.

Influence of sand mixing proportion on clogging

Three mixing proportions of movable and skeleton particles are used in this test. The relative density of the sand mixture is 0.6, the porosity of the pervious concrete is 25%, and the height of the sand column is 360 mm. The NP time history curves of the sand column and pervious concrete specimen are shown in Fig. 8.

As Fig. 8(a) shows, when the change in hydraulic conductivity becomes stable, the hydraulic conductivity of the sand column decreases as the mixing proportion of the movable particles decreases. This is because the resistance to movement decreases when there are less movable particles filling in the pores of the skeleton particles. Therefore, it is easier for movable particles to reach the sand–pervious concrete interface to form the dense filter zone, which decreases the hydraulic conductivity of the sand column. This supposition is supported by Table 8 and Fig. 9. Table 8 presents the particle size distributions in the sand layers after the completion of testing and Fig. 9 shows the proportions of movable particles. As illustrated in Fig. 9, the proportion of movable particles after testing is distributed more unevenly in the sand column as the mixing proportion of movable particles decreases.

As shown in Fig. 8(b), the final hydraulic conductivities of pervious concrete for Cases 3, 7, and 8 are relatively similar although the clogging processes are different. The probable reason is that the quantity of sand clogging the pervious concrete can be sufficiently provided by the sand column in these three cases. Moreover, the movable particles (clogging source) in these three cases are of the

same type. Under these circumstances, the seepage velocity is a consequential factor in controlling the clogging. As mentioned above, a small mixing proportion of movable particles can cause a large reduction in the hydraulic conductivity of the sand column and a low seepage velocity, which prolong the clogging process in pervious concrete.

Influence of relative density of sand on clogging

Three samples of distinct relative density are applied in the test. The porosity of the pervious concrete is 25%, M2-S-14 sand is used, and the height of the sand column is 360 mm. The NP time history curves of the sand column and pervious concrete specimen are shown in Fig. 10, respectively.

As Fig. 10(a) shows, the stable normalized hydraulic conductivity of the sand column increases as the relative density increases. When the relative density increases, the particles become more firmly packed and the number of gaps between particles decreases. Therefore, it is more difficult for movable particles to be carried off by the seepage flow, less movable particles are transported to the sand–pervious concrete interface, and the decrease in hydraulic conductivity is smaller. Table 9 presents the evidence showing that the content of movable particles varies from 14.97% to 23.05% for loose sand, while for dense packed sand, it varies from 17.82% to 21.00%.

As shown in Fig. 10(b), the pervious concrete in Case 3 and Case 9 experiences almost the same reduction in hydraulic conductivity, while that in Case 10 remains nearly unchanged. As analyzed above, fewer movable particles are transported to the sand–pervious concrete interface for sand with a large relative density. Thus, fewer particles can enter and clog the pores of PCPs. Case 3 and Case 9 both provide enough movable particles to cause clogging, while the particle holding capacity for pervious concrete is relatively limited. Therefore, they both exhibit the same clogging effect after the hydraulic conductivity becomes stable.

Influence of sand column height on clogging

Sand column heights of 270 mm, 360 mm, and 450 mm are used to investigate the influence of sand column height on clogging. The porosity of the pervious concrete is 25%, M2-S-14 sand is used, and the relative density is 0.6. The NP time history curves of the sand column and pervious concrete specimen are shown in Fig. 11, respectively.

As Fig. 11(a) shows, when the clogging processes reach equilibrium, the decrease in hydraulic conductivity of the sand column is most noticeable in Case 12 and least noticeable in Case 11. Based on Darcy's law, the seepage velocity has a negative correlation with the sand column height. Therefore, Case 11 has the largest hydraulic gradient, and the movable particles are transported to the interface at the highest speed via seepage flow. The most rapid decrease in hydraulic conductivity of the sand column in Case 11 occurs at the beginning of the test. However, Case 11 has the lowest quantity of movable particles that can be transported to the sand–pervious concrete interface, and the reduction in hydraulic conductivity is limited. The situation in Case 12 is the opposite of that in Case 11. As shown in Table 10, Case 12 has the largest number of movable particles that can accumulate in the bottom layer.

Fig. 11(b) shows that the hydraulic conductivity decreases least for the case with a sand column height of 450 mm. Case 12 has the smallest hydraulic gradient because it has the maximum height. Therefore, the seepage velocity is relatively small, and particles experience more difficulty in getting into the pores of the pervious concrete via seepage flow.

Clogging model of pervious concrete

The Kozeny–Carman equation forms the basis for deriving the empirical model used to predict the deterioration in hydraulic conductivity of pervious concrete due to clogging. Several researchers have

used the same equation to study similar phenomena. Blazejeski and Sadzide (1997) used the Kozeny–Carman equation to study the influence of decreasing porosity on sand hydraulic conductivity. Wu and Huang (2000) used this equation as a basis for deriving their theoretical model to study the effects of sediment intrusion into gravel beds. Tan et al. (2003) derived an empirical theoretical formulation from the same equation to predict the reduction in hydraulic conductivity of permeable bases.

A decreased hydraulic conductivity due to clogging can be represented as a function of the initial hydraulic conductivity, porosity, and specific deposit of clogging material (Tan et al. 2003):

$$(2) \quad k = k_0 \frac{(1 - P)^2}{P^3} \frac{(P - \alpha\sigma)^3}{[1 - (P - \alpha\sigma)]^2}$$

where k is the hydraulic conductivity of a clogged pervious concrete specimen, k_0 is the initial hydraulic conductivity of the specimen, P is the porosity of the specimen, σ is the specific deposit, which is the volume of sediment deposit divided by the bulk volume of the pervious concrete specimen, and α is an empirical coefficient.

Eq. (2) is a modified version of Wu and Huang's model (Wu and Huang 2000), which was used by Tan et al. (2003) to predict the reduction in hydraulic conductivity of permeable bases. From then on, many studies (Haselbach et al. 2006; Deo et al. 2010) began to use the coefficient α for indicating the rate of deterioration in permeability. The value of α also indicates the rate of deterioration of hydraulic conductivity. A high value of α implies a more rapid decrease in hydraulic conductivity. Therefore, α can be used as a means of evaluating the rate of deterioration and predicting the clogging effects. When the clogging particles completely fill up the entire void space within the pervious concrete, α is taken as 1 (Tan et al. 2003).

The values of the coefficient α can be calculated using Eq. (2). Table 11 presents the values of coefficient α for various cases at the end of each test.

Table 11 shows that when the specimens are clogged with sand, the α values are approximately between 2.7 and 6.0. Wu and Huang (2000) proposed that the depositional patterns of particles in a gravel matrix are governed by the gravel-sediment size ratio and the seepage flow rate. In this study, to analyze the value of α under different testing conditions, five dimensionless variables are used to represent the governing factors, namely the porosity of pervious concrete P , dimensionless height of the sand column H (the height of the sand column divided by the height of the pervious concrete specimen), relative density of the sand mixture D_r , aggregate-sediment size ratio $R_s = D_{15}/d_{85}$, and coefficient of uniformity of the clogging material C_u .

The gravel-sediment size ratio D_{15}/d_{85} in the study of Wu and Huang (2000) is easy to obtain. However, in this study, because the aggregates in pervious concrete specimens are coated with the hydration products of cement and water, the sizes of the solid grains “expand,” and the clogging process can change. Therefore, the expansion of the solid grain volume should be considered. The volumes of solid grains of pervious concrete specimens contain the volumes of the aggregate and the hydration product coating layer. The volume of the hydration product coating layer varies with the porosity of the specimen. The hydration product-induced volume expansion ratio of aggregates can be defined as the volume of solid pervious concrete divided by the volume of aggregates:

$$(3) \quad R_v = \frac{(1 - P)}{V_a} = \frac{(1 - P)\rho_a}{m_a}$$

where R_v is the volume expansion ratio, which, in the case of no hydration product coating layer on the aggregate, is taken as 1; V_a is the volume of aggregates per cubic meter of concrete, m_a is the mass of aggregates per cubic meter of concrete, ρ_a is the density of the aggregates, and P is the pervious concrete porosity.

Thus, the expansion ratio of the aggregate equivalent diameter R_d is

$$(4) \quad R_d = \sqrt[3]{R_v}$$

413 The equivalent characteristic sizes of solid grains (aggregate with the hydration product coating
414 layer) in pervious concrete are calculated using Eq. (5) and given in Table 12.

$$(5) \quad D_{15} = D_{15}' R_d$$

415 where D_{15}' is the equivalent characteristic size of aggregate used in the preparation of pervious
416 concrete.

417 The characteristic sizes of clogging materials are listed in Table 13. After obtaining the equivalent
418 D_{15} of the aggregate and d_{85} of the sediment, the aggregate-sediment size ratio R_s can be calculated, as
419 indicated in Table 11.

420 Five dimensionless variables are given in Table 11. To reflect the influence of each factor, the
421 multivariate nonlinear fitting was used. After comparisons, the multiparameter power function model
422 is found to fit well with the data. Wu and Huang (2000) also used this type of fitting model with 3
423 dimensionless variables to study the effects of sediment intrusion into gravel beds. The relationship of
424 the coefficient α with the five dimensionless variables is obtained, as indicated in Eq. (6), with $r^2 =$
425 0.87 (r is the correlation coefficient).

$$(6) \quad \alpha = P^{0.08} H^{-0.03} D_r^{-0.92} R_s^{-0.03} C_u^{0.44}$$

426 Eq. (6) indicates that the influences of pervious concrete porosity, piles distance, and
427 aggregate-sediment size ratio on the coefficient α have slight differences (the powers of P , H , and R_s
428 are 0.08, -0.03 , and -0.03 , respectively). Compared with the other parameters, the relative density of
429 sand and the coefficient of uniformity have major influence on the coefficient α (the power of D_r is
430 -0.92 , and that of C_u is 0.44). Fig. 12 shows that the regression equation is validated by the measured
431 data.

432 In view of Terzaghi's filter criterion (Terzaghi 1929), for cases in which $R_s = D_{15}/d_{85} < 4$, the
 433 physical implication of a small R_s is that the risk of soil piping through the granular filter is reduced.
 434 This implication indicates that if the clogging particle sizes are larger relative to the fine aggregate
 435 sizes within the specimen, there is a higher tendency for the clogging particles to be retained on the
 436 surface of the pervious concrete specimen, e.g., the clogging materials in Case 4 have an R_s of 3.41,
 437 and most sediments in this case are retained on the surface of the pervious concrete.

438 Meanwhile, for cases with very fine clogging materials, the amount of clogging decreases, e.g., the
 439 clogging of fine sediment in Case 6 is the least serious among four cases (Case 3–6), although Case 6
 440 has the largest R_s of 28.25, as shown in Fig. 9. Therefore, the relation between R_s and the coefficient α
 441 is complex when the range of R_s is greater than 4. Further research regarding the influence of R_s on the
 442 coefficient α is required.

443 The coefficient α of the stable stage is a characteristic resulting from the clogging process that
 444 depends on five variables. Using Eq. (2) and (6), the hydraulic conductivity reduction of a pervious
 445 concrete pile after clogging can be estimated. However, because the regression is based on
 446 experimental data, the applicability of Eq. (6) outside the range of tests is unwarranted (applicable
 447 range: $0.15 < P < 0.25$, $3 < H < 5$, $0.3 < D_r < 0.9$, $3.41 < R_s < 28.25$, and $10.16 < C_u < 33.73$).

448 ***Clogging model of sand column***

449 The seepage path of fluid in soil is shortened by the installation of pervious piles, and the dissipation
 450 of the excess pore water pressure is accelerated. However, when water moves toward the pervious
 451 concrete piles, it must first pass through the surrounding soil. If the hydraulic conductivity of soil is
 452 reduced and blocked, then the access to the pile is also blocked. The drainage capacity of pile and soil
 453 composite foundation will reduce. The consolidation rate of soil will then decrease. Therefore, the

surrounding soil is also considered in the test. To approximately predict the hydraulic conductivity of the soil between PCPs due to clogging, an empirical equation similar to Eq. (6) is derived to predict the variation in hydraulic conductivity of the sand column above the pervious concrete specimen. Via the regression analysis method, the relationship between the variation in hydraulic conductivity and the five dimensionless variables given in Table 14 is obtained, as indicated in Eq. (7), with $r^2 = 0.95$ (r is the correlation coefficient).

$$(7) \quad \frac{k_s}{k_{s0}} = P^{0.46} H^{-1.51} D_r^{0.43} R_s^{0.40} C_u^{0.56}$$

where k_s is the hydraulic conductivity of the clogged sand column and k_{s0} is the initial hydraulic conductivity of the sand column.

Fig. 13 shows the fitting result of the regression equation. Eq. (7) indicates that the influences of the pervious concrete porosity, sand relative density, aggregate-sediment size ratio, and coefficient of uniformity of the clogging material on the reduction in sand column hydraulic conductivity have little differences (the powers of P , D_r , R_s , and C_u are 0.46, 0.43, 0.40, and 0.56, respectively), and the fitted power of H is -1.51 . The negative sign means that the pile distance has a negative influence on the hydraulic conductivity of the surrounding soil. As its absolute value is greater than the other variables, the pile distance has a relatively greater influence.

Conclusions

In this study, an innovative continuous hydraulic conductivity measurement system is developed to investigate the clogging mechanism of PCPs caused by sand piping. A series of laboratory simulation tests is conducted using this system to demonstrate the effects of pervious concrete porosity, grading of fine movable particles, mix ratio of skeleton particles to movable particles, relative density of soil,

and distance between PCPs on the PCP clogging. The main conclusions are as follows:

1. The average hydraulic conductivity of soil after achieving stability is a comprehensive reflection of two contradictory aspects: the dense filter zone near the soil–pervious concrete interface formed by seepage, which decreases the hydraulic conductivity of soil, and the transport of soil particles from the soil column, which increases the hydraulic conductivity of soil.

2. The variation in the electrical conductivity of pervious concrete closely follows its clogging process. Therefore, the measurement of electrical conductivity, which is much more convenient, can be used as an indicator of pervious concrete clogging.

3. In this study, PCP clogging rarely occurs in cases of small pervious concrete porosity, small movable sand particle size, high sand relative density, and large pile distances.

Based on the test results, preliminary clogging models of pervious concrete and sand columns are proposed. These models can approximately estimate the hydraulic conductivity of PCPs after clogging occurs.

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Fig. 12 Relationship between tested coefficient α and fitted coefficient α

Fig. 13 Relationship between tested and fitted k_s/k_{s0}

Fig. 1. Schematic of PCP clogging caused by soil piping

194x92mm (300 x 300 DPI)

Fig. 2 Micro bridge network formation (Rollin and Lombard, 1988)

160x121mm (300 x 300 DPI)

473x423mm (72 x 72 DPI)

Fig. 4 Sketch of experimental setup design principle

332x184mm (300 x 300 DPI)

Fig. 5 Influence of pervious concrete porosity on clogging: (a) NP time history curves of sand; (b) NP time history curves of pervious concrete; (c) NEC time history curves of pervious concrete

303x543mm (300 x 300 DPI)

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331x397mm (300 x 300 DPI)

Fig. 7 Clogging mechanisms of sands with different movable particles (a, Well-graded sand; b, Single size sand)

157x46mm (200 x 200 DPI)

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Fig. 8 Influence of sand mixing proportion on clogging: (a) NP time history curves of sand; (b) NP time history curves of pervious concrete

313x363mm (300 x 300 DPI)

Fig. 9 The proportions of movable particles for each layer after testing

769x423mm (72 x 72 DPI)

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321x384mm (300 x 300 DPI)

Fig. 11 Influence of sand column height on clogging: (a) NP time history curves of sand; (b) NP time history curves of pervious concrete

311x361mm (300 x 300 DPI)

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80x62mm (300 x 300 DPI)

Fig. 13 Relationship between tested and fitted k_s/k_{s0}
83x63mm (300 x 300 DPI)

Table 1 Physical indexes of coarse aggregate

Particle size	Apparent density	Stacking density	Porosity	Crushed value
(mm)	(g/cm ³)	(g/cm ³)	(%)	(%)
4.75~9.5	2.665	1.655	37.89	8.6

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Table 2 Mix proportions per cubic meter of pervious concrete

Target porosity	Cement	Water	Aggregate	Superplasticizer
(%)	(kg/m ³)	(kg/m ³)	(kg/m ³)	dosage (%)
15	335	121		
20	262	94	1622	0.8
25	189	68		

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Table 3 Characteristics of particles used in the tests

Particle type	Particle name	Mass percentage (%)					
		4.75-9.5mm	2.36-4.75mm	1.18-2.36mm	0.6-1.18mm	0.3-0.6mm	0.15-0.3mm
Movable particle	M1	0	11.6	28	35	4.5	20.9
	M2	0	0	0	97.4	2.6	0
	M3	0	0	0	17.2	78.2	4.6
	M4	0	0	0	0	0	100
Skeleton particle	S	100	0	0	0	0	0

Table 4 Different types of sand column mixtures

Name of mixture	Movable particle included	skeleton particle included	Mass ratio of movable and skeleton particle	Minimum dry density (g/ml)	Maximum dry density (g/ml)
M1-S-14	M1	S	1:4	1.40	1.79
M2-S-13	M2	S	1:3	1.39	1.80
M2-S-14	M2	S	1:4	1.38	1.79
M2-S-15	M2	S	1:5	1.35	1.77
M3-S-14	M3	S	1:4	1.49	1.89
M4-S-14	M4	S	1:4	1.59	1.97

Table 5 Test cases

Cases	Porosity of pervious concrete (%)	Name of sand mixture	Relative density D_r	Height of sand column (mm)	Initial hydraulic conductivity of sand column (m/s)
1	15	M2-S-14	0.6	360	0.019
2	20	M2-S-14	0.6	360	0.015
3	25	M2-S-14	0.6	360	0.013
4	25	M1-S-14	0.6	360	0.011
5	25	M3-S-14	0.6	360	0.019
6	25	M4-S-14	0.6	360	0.064
7	25	M2-S-13	0.6	360	0.011
8	25	M2-S-15	0.6	360	0.023
9	25	M2-S-14	0.3	360	0.028
10	25	M2-S-14	0.9	360	0.010
11	25	M2-S-14	0.6	270	0.013
12	25	M2-S-14	0.6	450	0.015
13	15	M1-S-14	0.3	270	0.014
14	15	M3-S-14	0.9	450	0.013
15	20	M1-S-14	0.6	450	0.013
16	20	M2-S-14	0.9	270	0.010
17	20	M3-S-14	0.3	360	0.023
18	25	M1-S-14	0.9	360	0.009
19	25	M2-S-14	0.3	450	0.021
20	25	M3-S-14	0.6	270	0.018

Table 6 Size distributions of sand in each layer of sand column after the test

Case	Layers (from up to bottom)	Mass percentage of sand in each layer				
		Skeleton particle 4.75-9.5mm	Movable particle			
			1.18-4.75mm	0.6-1.18mm	0.3-0.6mm	<0.3mm
1	1st layer	80.55% ↑	-	19.05% ↓	0.40% ↓	-
	2nd layer	80.17% ↑	-	19.41% ↓	0.42% ↓	-
	3rd layer	79.82% ↓	-	19.70% ↑	0.48% ↓	-
	4th layer	75.25% ↓	-	20.18% ↑	0.57% ↑	-
2	1st layer	81.27% ↑	-	18.40% ↓	0.34% ↓	-
	2nd layer	80.12% ↑	-	19.43% ↓	0.45% ↓	-
	3rd layer	79.63% ↓	-	19.89% ↑	0.48% ↓	-
	4th layer	78.11% ↓	-	21.39% ↑	0.50% ↓	-
3	1st layer	82.69% ↑	-	16.86% ↓	0.44% ↓	-
	2nd layer	82.35% ↑	-	17.20% ↓	0.46% ↓	-
	3rd layer	78.58% ↓	-	20.89% ↑	0.53% ↑	-
	4th layer	77.35% ↓	-	21.99% ↑	0.65% ↑	-

Note: arrows indicate the changes in the proportions compared with initial values before test.

Table 7 Size distributions of sand in each layer of sand column after the test

Case	Layers	Mass percentage of sand in each layer					
	(from up	Skeleton	Movable particle				
	to	particle					
	bottom)	>4.75mm	2.36-4.75mm	1.18-2.36mm	0.6-1.18mm	0.3-0.6mm	0.15-0.3mm
3	1st layer	82.69% ↑	-	-	16.86% ↓	0.44% ↓	-
	2nd layer	82.35% ↑	-	-	17.20% ↓	0.46% ↓	-
	3rd layer	78.58% ↓	-	-	20.89% ↑	0.53% ↑	-
	4th layer	77.35% ↓	-	-	21.99% ↑	0.65% ↑	-
4	1st layer	82.18% ↑	2.41% ↑	5.73% ↑	5.2% ↓	0.71% ↓	3.77% ↓
	2nd layer	81.93% ↑	2.31% ↓	5.62% ↑	5.74% ↓	0.80% ↓	3.69% ↓
	3rd layer	78.48% ↓	2.22% ↓	5.45% ↓	8.51% ↑	0.89% ↓	4.45% ↑
	4th layer	78.07% ↓	2.17% ↓	4.74% ↓	8.89% ↑	1.22% ↑	4.90% ↑
5	1st layer	85.23% ↑	-	-	3.50% ↑	10.57% ↓	0.70% ↓
	2nd layer	83.79% ↑	-	-	2.48% ↓	12.94% ↓	0.80% ↓
	3rd layer	83.07% ↑	-	-	3.41% ↓	12.69% ↓	0.83% ↓
	4th layer	80.02% ↑	-	-	3.31% ↓	15.68% ↑	0.99% ↑
6	1st layer	98.65% ↑	-	-	-	-	1.35% ↓
	2nd layer	87.86% ↑	-	-	-	-	12.14% ↓
	3rd layer	86.74% ↑	-	-	-	-	13.26% ↓
	4th layer	80.50% ↑	-	-	-	-	19.50% ↓

Note: arrows indicate the changes in the proportions compared with initial values before test.

Table 8 Size distributions of sand in each layer of sand column after the test

Case	Layers (from up to bottom)	Mass percentage of sand in each layer				
		Skeleton particle	Movable particle			
		>4.75mm	1.18-4.75mm	0.6-1.18mm	0.3-0.6mm	<0.3mm
7	1st layer	76.67% ↑	-	22.82% ↓	0.51% ↓	-
	2nd layer	76.58% ↑	-	22.88% ↓	0.54% ↓	-
	3rd layer	75.50% ↑	-	23.87% ↓	0.63% ↓	-
	4th layer	73.86% ↓	-	25.50% ↑	0.64% ↓	-
3	1st layer	82.69% ↑	-	16.86% ↓	0.44% ↓	-
	2nd layer	82.35% ↑	-	17.20% ↓	0.46% ↓	-
	3rd layer	78.58% ↓	-	20.89% ↑	0.53% ↑	-
	4th layer	77.35% ↓	-	21.99% ↑	0.65% ↑	-
8	1st layer	87.31% ↑	-	12.35% ↓	0.34% ↓	-
	2nd layer	85.42% ↑	-	14.19% ↓	0.39% ↓	-
	3rd layer	81.21% ↓	-	18.27% ↑	0.52% ↑	-
	4th layer	80.32% ↓	-	18.76% ↑	0.92% ↑	-

Note: arrows indicate the changes in the proportions compared with initial values before test.

Table 9 Size distributions of sand in each layer of sand column after the test

Case		Layers (from up to bottom)	Mass percentage of sand in each layer					
			Skeleton particle	Movable particle				
				>4.75mm	1.18-4.75mm	0.6-1.18mm	0.3-0.6mm	<0.3mm
9	1st layer	85.03% ↑	-	14.58% ↓	0.39% ↓	-		
	2nd layer	80.46% ↑	-	19.11% ↓	0.43% ↓	-		
	3rd layer	78.52% ↓	-	20.97% ↑	0.51% ↓	-		
	4th layer	76.95% ↓	-	22.41% ↑	0.64% ↑	-		
3	1st layer	82.69% ↑	-	16.86% ↓	0.44% ↓	-		
	2nd layer	82.35% ↑	-	17.20% ↓	0.46% ↓	-		
	3rd layer	78.58% ↓	-	20.89% ↑	0.53% ↑	-		
	4th layer	77.35% ↓	-	21.99% ↑	0.65% ↑	-		
10	1st layer	82.18% ↑	-	17.30% ↓	0.52%	-		
	2nd layer	81.69% ↑	-	17.98% ↓	0.33% ↓	-		
	3rd layer	79.74% ↓	-	19.84% ↑	0.42% ↓	-		
	4th layer	79.00% ↓	-	20.41% ↑	0.59% ↑	-		

Note: arrows indicate the changes in the proportions compared with initial values before test.

Table 10 Size distributions of sand in each layer of sand column after the test

Case	Layers (from up to bottom)	Mass percentage of sand in each layer					
		Skeleton particle	Movable particle				
			>4.75mm	1.18-4.75mm	0.6-1.18mm	0.3-0.6mm	<0.3mm
11	1st layer	80.99% ↑	-	18.63% ↓	0.38% ↓	-	
	2nd layer	79.97% ↓	-	19.57% ↑	0.46% ↓	-	
	3rd layer	80.39% ↑	-	18.93% ↓	0.68% ↑	-	
	4th layer	79.38% ↓	-	20.27% ↑	0.35% ↓	-	
3	1st layer	82.69% ↑	-	16.86% ↓	0.44% ↓	-	
	2nd layer	82.35% ↑	-	17.20% ↓	0.46% ↓	-	
	3rd layer	78.58% ↓	-	20.89% ↑	0.53% ↑	-	
	4th layer	77.35% ↓	-	21.99% ↑	0.65% ↑	-	
12	1st layer	81.26% ↑	-	18.32% ↓	0.42% ↓	-	
	2nd layer	80.57% ↑	-	18.96% ↓	0.47% ↓	-	
	3rd layer	79.94% ↓	-	19.58% ↑	0.48% ↓	-	
	4th layer	77.18% ↓	-	22.27% ↑	0.55% ↑	-	

Note: arrows indicate the changes in the proportions compared with initial values before test.

Table 11 Values of coefficient α for various clogging tests

Cases	Coefficient α	P	H	D_r	R_s	C_u
1	2.77	0.15	4	0.6	7.37	10.16
	3.51					
	3.72					
2	3.66	0.20	4	0.6	7.21	10.16
	3.45					
	4.03					
3	4.96	0.25	4	0.6	7.06	10.16
	4.21					
	3.98					
4	4.52	0.25	4	0.6	3.41	10.45
	4.13					
	3.78					
5	4.77	0.25	4	0.6	13.41	17.26
	4.56					
	4.36					
6	4.02	0.25	4	0.6	28.25	33.73
	4.75					
	5.53					
7	5.04	0.25	4	0.6	7.06	11.05
	4.53					
	3.66					
8	5.07	0.25	4	0.6	7.06	10.01
	4.25					
	3.52					
9	5.26	0.25	4	0.3	7.06	10.16
	6.33					
	6.75					
10	3.88	0.25	4	0.9	7.06	10.16
	3.51					
	3.96					
11	5.93	0.25	3	0.6	7.06	10.16
	5.24					
	4.79					
12	3.83	0.25	5	0.6	7.06	10.16
	3.27					
	2.95					
13	5.97	0.15	3	0.3	3.56	10.45
	5.84					

	5.72					
	4.03					
14	3.85	0.15	5	0.9	13.98	17.26
	4.08					
	5.86					
15	5.97	0.2	5	0.6	3.48	10.45
	5.48					
	3.19					
16	2.85	0.2	3	0.9	7.21	10.16
	3.06					
	6.02					
17	5.59	0.2	4	0.3	13.69	17.26
	5.86					
	4.68					
18	4.35	0.25	4	0.9	3.41	10.45
	4.52					
	6.24					
19	6.18	0.25	5	0.3	7.06	10.16
	5.87					
	3.93					
20	3.99	0.25	3	0.6	13.41	17.26
	4.14					

Table 12 Equivalent characteristic sizes of solid grain in pervious concrete

P	R_v	R_d	D_{10}	D_{15}	D_{60}	D_{85}
			(mm)	(mm)	(mm)	(mm)
15%	1.40	1.12	7.93	8.25	10.38	11.58
20%	1.31	1.10	7.77	8.08	10.17	11.35
25%	1.23	1.07	7.60	7.91	9.96	11.11

Draft

Table 13 Characteristic sizes of clogging materials

Clogging materials	d_{10} (mm)	d_{15} (mm)	d_{60} (mm)	d_{85} (mm)
M1	0.24	0.28	1.17	2.32
M2	0.68	0.71	0.92	1.12
M3	0.32	0.33	0.47	0.59
M4	0.15	0.16	0.25	0.28

Draft

Table 14 Values of hydraulic conductivity of sand column for various clogging tests

Cases	k_s/k_{s0}	P	H	D_r	R_s	C_u
	0.23					
1	0.28	0.15	4	0.6	7.37	10.16
	0.36					
	0.44					
2	0.55	0.20	4	0.6	7.21	10.16
	0.48					
	0.51					
3	0.41	0.25	4	0.6	7.06	10.16
	0.37					
	0.36					
4	0.29	0.25	4	0.6	3.41	10.45
	0.24					
	0.72					
5	0.99	0.25	4	0.6	13.41	17.26
	0.87					
	1.21					
6	1.15	0.25	4	0.6	28.25	33.73
	1.32					
	0.42					
7	0.64	0.25	4	0.6	7.06	11.05
	0.53					
	0.32					
8	0.28	0.25	4	0.6	7.06	10.01
	0.38					
	0.31					
9	0.34	0.25	4	0.3	7.06	10.16
	0.41					
	0.39					
10	0.62	0.25	4	0.9	7.06	10.16
	0.47					
	0.72					
11	0.68	0.25	3	0.6	7.06	10.16
	0.57					
	0.29					
12	0.23	0.25	5	0.6	7.06	10.16
	0.33					
	0.31					
13	0.22	0.15	3	0.3	3.56	10.45

		0.35					
		0.41					
14	0.56	0.15	5	0.9	13.98	17.26	
	0.61						
	0.29						
15	0.22	0.2	5	0.6	3.48	10.45	
	0.31						
	0.74						
16	0.65	0.2	3	0.9	7.21	10.16	
	0.82						
	0.55						
17	0.51	0.2	4	0.3	13.69	17.26	
	0.64						
	0.32						
18	0.29	0.25	4	0.9	3.41	10.45	
	0.49						
	0.28						
19	0.43	0.25	5	0.3	7.06	10.16	
	0.39						
	0.87						
20	0.95	0.25	3	0.6	13.41	17.26	
	0.93						