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Laboratory Assessment of Water Flow Simulator for Porous Parking Lots Reservoir and Soil Layers

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Abstract: Porous parking lots were implemented to fill the scarcity and strengthen the sustainable development of impervious surfaces in Malaysia to reduce surface runoff. The new methodology proposed enable simulation of reservoir course at stipulated air voids despite details study on water levels and infiltration of porous parking lots system. A uniformly graded choker and reservoir stones functioned as reservoir structure for temporary storm water detention. A specially fabricated water flow simulator allows laboratory simulation of the porous parking lot system. The ability to simulate 1.24 to 59.89 cm h^{-1} rainfall intensities enable laboratory testing to verify water level and discharge time correlations with different soil infiltration rate at various rainfall intensities. The laboratory tests of vertical infiltration were conducted under conditions of saturated soils. At 59.89 cm h^{-1} simulated rainfall intensity, the highest water level recorded inside the water flow simulator's body without reservoir course is 55 cm from the surface of 0.254 cm h^{-1} soil infiltration rate. A total of 80.5 h duration was required to completely discharge the stored water between large aggregate particles due to low soil infiltration rate. Utilizing the same laterite soil as bottom layer, the depth increased by approximately 60% after installation of reservoir course with approximately 40% air voids. Over a 60 min duration, the soil with coefficient of permeability equaled to 25.4 cm h^{-1} had recorded water level 20 cm in height and completely dried within 32.2 min only.

Key words: Rainfall intensity, stone recharge bed, existing soils, rainfall-runoff, water flow simulator

INTRODUCTION

A porous asphalt pavement is a brilliant invention in modern built environment that allows storm water to permeate through the porous matrix efficiently. The storm water which passes through the permeable surface might completely or partially permeate into the existing soil. Other methods of disposing water entering into porous asphalt pavement include incorporation of either on or off-site disposal or re-use of the water, with or without further treatment before disposal. However, these approaches implicate high cost for constructions of detention basins and piping system. As an alternative, porous parking lots system incorporating porous asphalt as a surface layer provides an effective solution to mitigate storm water as the water will penetrate into the existing soil naturally (Cahill *et al*., 2003).

Porous parking lot system consists of porous paved surface, overlying an aggregate base course, functioned as a reservoir for temporary storm water detention which will infiltrate into the existing ground (Backstrom, 2000). Porous surfaces enable short term storage of rainfall which provided additional infiltration, thereby increasing base flow. The air voids between the coated aggregates reduced the velocity of water, thus diminishing the sediment load carried into receiving water (Leming *et al*., 2007).

The choker course is placed beneath the porous asphalt layer. The smaller size aggregate filled the void space, in turn provided stable surfaces for paving overlying asphalt and at the same time eliminate porous asphalt mixes from getting into large air voids between reservoir course's material (McNally *et al*., 2005; Rogers and Faha, 2007). Researchers had recommended a fix 2.54 cm thickness for this layer (McNally *et al*., 2005; NAPA, 2008). The typical aggregate gradations used for choker and reservoir courses are shown in Table 1.

According to NAPA (2008) the choker course, usually laid using AASHTO No. 57 gradation, consists of clean uniformly graded crushed aggregate which is smaller than uniformly graded aggregate base material (Rogers and Faha, 2007). Located under the choker course is a uniformly graded, clean crushed rock (AASHTO No. 2 gradation) recharge bed

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Table 1: Typical gradations for choker and reservoir course

U.S standard Sieve size (mm)	Choker course (AASHTO No. 57)	Reservoir course (AASHTO No. 2)	Reservoir course (AASHTO No. 3)
150	-	-	-
75	-	100	-
63	-	90-100	100
50	-	35-70	90-100
37.5	100	0-15	35-70
25	95-100	-	0-15
19	-	0-5	-
12.5	25-60	-	0-5
9.5	-	-	-
4.75	0-10	-	-
2.36	0-5	-	-

(McNally *et al*., 2005; Rogers and Faha, 2007) which was designed to receive, temporarily store and infiltrate the incoming runoff. On the contrary, researcher claimed that in many constructions, AASHTO No. 3 gradation was specified and has functioned well with AASHTO No. 57 choker course overlaid (NAPA, 2008).

The clean-washed uniformly graded underlying reservoir course (also known as stone recharge bed) utilized stone sizes ranging from 3.0 to 6.4 cm (Cahill *et al*., 2003). There were several variations of both larger and smaller size stones, relying on local aggregate sources. Large void space between the aggregates, ensure good permeability and achieved by using uniformly graded stones. The air voids between the stones provided critical storage volume for the storm water. The reservoir course is usually between 45.7 to 91.4 cm deep depending on storm water storage requirements, frost depth considerations and site grading (Cahill *et al*., 2003). The reservoir course was filled with washed, uniformly graded granite aggregates that allow infiltration and provide structural support for the porous asphalt pavement (Rowe *et al*., 2008). The depth required for reservoir course is typically between 30.5 and 91.4 cm (NAPA, 2008). The minimum thickness suggested was 15.2 cm lower than the depth proposed by other researcher (Cahill *et al*., 2003). The lowest reservoir course depth suggested, ranging from 20.3 to 22.9 cm (CAPA, 2006). With about 40% air voids between the aggregates, this would mean that the recharge bed was capable of storing between 12.2 and 36.6 cm of precipitation (NAPA, 2008). Other researcher suggested 30 to 40% air voids for reservoir course (Ferguson, 2005). Meanwhile, Pennsylvania Storm water Best Management Practices Manual (PSBMPM, 2006) described minimum 40% air voids for reservoir course design. Up to date, there was no exact tolerance for air voids requirement. The thickness of the stone recharge bed was determined by the amount of water that needs to be stored and the infiltration rate of the soil. The reservoir course provides engineers the opportunity to infiltrate storm water into existing soils with recommended discharge time between 12 and 72 h (NAPA, 2008).

Table 2: Infiltration rates based on general HSG classification

HSG	Soil textures	Typical infiltration rate (cm h^{-1})
A	Sand, loamy sand or sandy loam	>0.75
B	Silt loam or loam	0.38-0.75
C	Sandy clay loam	0.013-0.38
D	Clay loam, silty clay loam, sandy clay, silty clay or clay	<0.013

The guideline has established soil infiltration rate between 0.254 to 25.4 cm h^{-1} as a permissible infiltration rate for porous parking lot construction (NAPA, 2008). However, some reports suggested that soils with permeability less than 0.64 cm h^{-1} were most likely not suitable for porous parking lot system without ample additional facilities (NAPA, 2008). The claim is reasonable as most references suggested the underlying soils shall have a minimum infiltration rate of 1.27 cm h^{-1} for proper exfiltration system. However, the confusion on this issue has been solved as researcher whom proved that the implementation of porous parking lot system above existing ground with infiltration rate as low as 0.254 cm h^{-1} was a success (NAPA, 2008). As summarized, the Hydrologic Soil Group (HSG) was divided into four main groups as illustrated in Table 2 (Viessman and Lewis, 2003).

The values given in Table 2 were according to soil surveys carried out by the Natural Resources Conservation Service (NRCS), previously known as Soil Conservation Service (SCS). Researcher has ascertained that HSG A and B were ideal for porous parking lot system; meanwhile potential areas in HSG C and D entailed more attention (NAPA, 2008).

This study aimed to quantify the effects of soil infiltration rate to the water level subjected to various rainfall intensities. Thus, the soil has been prepared to simulate both low and high infiltration rate conditions. A locally made Water Flow Simulator (WFS) was fabricated for laboratory simulation of the porous parking lot system. This paper describes a new methodology to simulate the reservoir course using granite aggregate. The preparation of the soil layer with infiltration rates 0.254 and 25.4 cm h^{-1} inside the WFS's body are highlighted in this paper as well. The paper also elaborates on the laboratory testing to verify water level and discharge time correlations with soil infiltration rate, at the same time ascertain the accuracy of the proposed methodology to simulate a reservoir course with sufficient air voids. The study aims at verifying the appropriate height of recharge bed depth that can temporarily store water and prevent the porous parking lot system from flooding or causes an overflow onto the pavement course.

MATERIALS AND METHODS

Test materials: Aggregate type granite supplied by Kuad Quarry Sdn. Bhd in Penang was used for the choker and reservoir courses. Crushed granite were sieved, washed and dried into selected size frictions. The courses should contain very clean and uniformly graded aggregates that conform to standard local sieve sizes. The basic properties of the aggregates are tabulated in Table 3.

In this study, the reservoir course has been assembled based on following size fractions. Aggregate sieve sizes 63 to 37.5 mm follows U.S standard, meanwhile 28 to 10 mm sieve sizes adopted as outlined in specification (JKR, 2008).

The reservoir course consists of aggregates that were cleaned and uniformly graded which functioned to provide temporary storm water storage and able to support vehicle loads. This study recommends thicker reservoir course since Malaysia is subjected to high rainfall intensity throughout the year. The proposed depth was 110 cm, about 20.4% higher than the depth proposed by other researcher (NAPA, 2008). The stone recharge bed was made with clean single-sized crushed aggregate with average Saturated Surface Dry (SSD) Specific Gravity of 2.628. The percentages of aggregates used for reservoir course are shown in Table 4.

To simulate 0.254 and 25.4 cm h^{-1} coefficient values, a combination of laterite soil and sand was used. The laterite soil was taken from Bukit Sungai Rambai, Relau, Kedah. Throughout the test, the soil was initially prepared at Optimum Moisture Content (OMC). The soil was compacted at its OMC according to required number of blows. A sieve analysis was used to quantify the particle-size distribution of soil with the smallest sieve size of 0.075 mm (Das, 1998). The particle size distribution curve obtained is shown in Fig. 1.

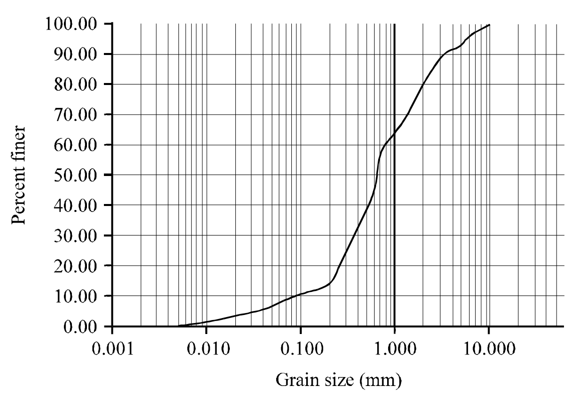

Fig. 1: Particle size distribution plot for soils adopted in this study

The gradation of a soil is determined by inferring from the particle size distribution curve produced from the results of laboratory tests on the soil. Based on the criteria set, the particle-size distribution of the soil is uniformly graded as the soil used has most of its grains of about the same size (Das, 1998). The percentages of gravel, sand, silt present in this soil have been adopted according to the Unified Soil Classification System (USCS) and are determined to be 19.68, 73.65 and 6.67%, respectively. The classification tests were carried out to describe the soil properties and the results shown in Table 5.

The sand used to simulate soil condition with higher permeability was supplied by a local supplier. The air voids between sand particles was higher compared to laterite soil due to larger particle sizes. In this study, the wet construction and with average coefficient of permeability, k equaled to 0.037 cm sec^{-1} was used for compaction process prior to the testing. The k was determined in accordance to BS 1377:5:1990 procedures utilizing standard constant-head permeameter that only suitable for soil with k ranging from 0.01 to 10 cm sec^{-1}. The k was computed from three samples (three readings each). The sand particle utilized has only 1.31% mass of material passing the 63 μm sieve size, conformed to the standard that allows only soils with less than 10% by mass of material passing the 63 μm sieve to be tested using constant-head method (BSI, 1990).

Table 3: Properties of crushed granite used in this study

Courses	Size fraction (mm)	Specific Gravity, SSD ASTM C127 (ASTM, 2004)	Water absorption (%) ASTM C127 (ASTM, 2004)	LAAV test (ASTM C 131) (ASTM, 2003b)
Reservoir	63-50	2.633	0.314	(Grading A) 28.6%
	50-37.5	2.634	0.372	
	37.5-28	2.621	0.421	
	20-14	2.633	0.5	
Choker	14-10	2.646	0.647	-

Table 4: Percentages of aggregates proposed in this study

Size fraction (mm)	63-50	50-37.5	37.5-28	28-20	20-14	14-10	10-5	5-2.36
Percentages of aggregates used	5	42.5	45	-	7.5	-	-	-

Table 5: Laterite soil properties

Liquid limit (LL) (%)	Plastic limit (PL) (%)	Plasticity index (PI) (%)	Specific gravity (Gs)	Maximum dry density (MDD) (Mg m^{-3})	Optimum moisture content (OMC) (%)	Compression index (Cc)
46.29	35.68	15.61	2.554	1.86	10.98	0.443

EXPERIMENTAL PROCEDURE

Preparation of reservoir course: The suitable weight of aggregates required to fill in the WFS's body, reaching 110 cm in height as depicted in Fig. 2 was determined using Eq. 1 (ASTM C29) (ASTM, 2003a, b). The density of water was assumed to be about 995.83 kg m^{-3}, corresponding to 29.4°C pipe water temperature:

$$\%\ \text{Air Voids} = \frac{[(S\times W)-M]}{(S\times W)}\times 100 \qquad (1)$$

where, M is bulk density of the aggregate (kg m^{-3}), S is bulk specific gravity (dry basis), W is density of water (998 kg m^{-3}).

Before the aggregates were filled into the structure according to the percentages of each size fractions, a trial test was carried out to estimate the amount of aggregates required. The formula has been used to determine the exact amount of aggregate required to fill in a stainless steel container with 40×40×45.5 cm dimensions. The total volume of the container was 0.0728 m^{-3}. By assuming 40% air voids, the bulk dry density of the aggregates calculated was 2606.6 kg m^{-3}. The mass of aggregates was determined by multiplying the bulk dry density of aggregates with the volume of aggregates. Thus, the estimated mass of aggregates was approximately 113.9 kg. The aggregates inside plastic container were dropped from 20 cm in height from the steel container surface, to ensure better aggregate particles packing and self compacting. Packing can be defined as the arrangement of the particles which fit together to fill the voids. The aggregates remain without any compaction effort during the pouring process. The observation has shown that the container was nicely filled by the combined multi-sizes aggregates.

The volume of water (mL) needed to fill the air voids between the aggregates has been recorded to ascertain the existence of designed air voids. The results showed that the amount of water required to fill the voids was 32100 mL or 44% of container's volume. It seems that the air voids between the aggregate particles exceeded the design air voids by 4% since the true air voids can never be determined (Crouch *et al*., 2007). This is due to the arrangement of aggregate orientation, stable skeleton, shape, surface texture and angularity variations among the aggregates. However, the findings comply to requirement of at least 40% air voids proposed the manual (PSBMPM, 2006). Thus, the amount of aggregates needed to fill in the water flow simulator's body was determined using the same method as proposed. From Eq. 1, the mass of aggregates needed to fill up the WFS's body up to 110 cm in height was determined. With 110×40×30 cm dimensions, the total volume calculated was 0.176 m^3. The volume of aggregates was determined as 60% of total volume. By assuming 40% air voids, the bulk dry density of the aggregates calculated was 2606.6 kg m^{-3}. By multiplying the bulk dry density of aggregates with 60% of total volume, the estimated mass of oven-dried aggregates was approximately 206.5 kg.

Fig. 2: The aggregates inside the WFS's body

Laboratory preparation for soil layer with k equals to 0.254 cm h^{-1}: The guidelines provided for porous asphalt pavement design requires soil infiltration rates of 0.254 to 25.4 cm h^{-1} appropriate for porous parking lot system (NAPA, 2008). Based on the proposed values, This laboratory scale study involved the determination of the water level recorded in the reservoir course corresponding to various rainfall intensities according to minimum infiltration rate of 0.254 cm h^{-1}. The soil with 25.4 cm h^{-1} infiltration rate was prepared to observe the effect of water level to the bottom layer with higher infiltration rate. Both sand and laterite soils have been used to simulate affirmed infiltration rates. The water level was measured from the datum (soil medium surface) after being subjected to simulated rainfall over a 60 min duration with 15 min interval. A series of tests with controlled number of compaction efforts had been implemented to achieve a target infiltration rate of 0.254 cm h^{-1}. The laterite soil used was compacted layer by layer at 10% moisture content in rectangular stainless steel container with 40×30 cm base with 45 cm in height.

The first portion of soil was placed into the container and compacted using 4.5 kg hollow metal rammer having a 7 cm circular surface diameter with 15.5 cm height, until the layer occupied about one-fifth of the total height of the soil medium. The soil medium thickness proposed in this study was 20 cm. The procedures adopted followed closely the modified proctor test method with adjustment in the number of blows. In this test, the hollow stainless steel rammer was placed to fall from a height of 45.7 cm, hitting on a rectangular solid steel base plate to compact the soil. The 400×300 mm solid steel base plate, with 10 mm thickness was used to ensure that the blows were evenly distributed over the soil surface.

Once the steel rod with two meters length and 1.2 cm diameter was tightened into the stainless steel base circular surface, the hollow metal rammer was carefully inserted into the rod and placed on top of the rectangular base plate.

To simulate a soil layer with 0.254 cm h^{-1} infiltration rate, laterite soil was compacted with different number of blows. Initially, the soil was placed in a rectangular stainless steel container and compacted in five layers. The compaction started with 4 blows. Subsequently, each layer (approximately 4 cm) was compacted with 4 blows until the 5th layers. The process was repeated with increment of 4 blows for each layer until the estimated infiltration rate is reached. By using falling-head permeability test, the permeability of the compacted soil was tested. The falling head test was used to measure the permeability of fine grained soil (Attom, 1997). Two circular moulds with 10.2 cm diameter and 12.5 cm height were placed side by side and hammered into the compacted soil. The compacted soils inside the circular moulds were soaked for about two h before testing to ensure saturated soil conditions. Two samples were tested for each different number of blows.

The k was determined according to the head differences observed in 100 cm height burette and the volume of water collected in a glass beaker. The whole process was repeated with increased compaction efforts, starting from 4 up to 32 blows.

The soil's coefficient of permeability was then calculated using Eq. 2 (Das, 1998):

$$k = \frac{QL}{Aht} \tag{2}$$

where, Q is volume of water collected; A is area of cross section of the soil specimen (81.72 cm^2); t is duration of water collection (3 min); L is length of the specimen (12.5 cm) and h is head difference (cm).

Laboratory preparation for soil layer with k equals to 25.4 cm h^{-1}: In this study, the sand was utilised to simulate a media with 25.4 cm h^{-1} k due to coarser particles size. The preliminary tests using sand has been carried out inside the WFS's body. The sand was compacted layer by layer inside the WFS body. In determining the k of the compacted soil equals to 25.4 cm per h, the sand was compacted with different compaction blows. A pulley system combined with L shape solid steel arm has been designed and attached to the WFS structure to start the test.

The sand was placed in a rectangular stainless steel body and compacted in 4 layers as describe in BS 1377-Part 5 (BSI, 1990). The standard recommends the placement of the material in at least 4 layers if the test samples utilized hand tamping (BSI, 1990). In this study, the proposed number of layer is four layers as outlined in the standard. The first portion of sand was placed into the stainless steel body and compacted until the layer occupied about one-fourth (approximately 5 cm) of the total medium height. The proposed height for soil medium was 20 cm, followed the height of the sand sample proposed for the laboratory constant-head permeability test. The sand used was compacted using 4.5 kg hollow metal rammer tied with a rope.

The hollow rammer was pulled up to 45.7 cm from the base plate circular surface and released for a single compaction. The steel rod with two meters length and 1.2 cm in diameter (with one meter separated portion each) was to control the rammer's direction, at the same time ensuring that the base plate perfectly placed on the sand surface for the compaction process. The compaction started with 4 blows. Thus, each layer was compacted with 4 blows until the 4th layer. The process was repeated with increment of 4 blows for each layer until the estimated infiltration rate was reached. The whole process was then repeated with increased compaction efforts, starting from 4 up to 28 blows.

The k was determined by recording the volume (mL) of water recorded corresponding to constant water level inside the WFS's body at 20 cm. Three sets of readings were taken for each compaction effort. Equation 2 was used to calculate the k with following criteria; Area of cross section of the soil specimen, A (1200 cm^2), duration for water storage, t (3 min) and length of the specimen, L (20 cm). Using the same apparatus, the procedures were repeated to simulate lower infiltration rate value inside the WFS's stainless steel body, following laboratory preparation using laterite soil.

RESULTS AND DISCUSSION

In this study, the infiltration rate and coefficient of permeability, k are assumed identical. According to ASCE (2009), the term infiltration rate, as defined by the measured and calculated k, refers specifically to the assumed rate at which water will infiltrate vertically into a saturate soil. The infiltration rate differs between under saturated and unsaturated conditions. Because storm water management design must consider long-term conditions, the saturated infiltration rate is of primary interest to engineering professionals. Under saturated soil conditions, the infiltration rate is essentially equivalent to k (ASCE, 2009).

Coefficient of permeability (also called Hydraulic Conductivity) in horizontal and vertical directions is not similar. Due to the effects of gravity, the vertical coefficient of permeability is the primary component of interest in the soil infiltration process (ASCE, 2009). As described above, the saturated vertical soil coefficient of permeability may be assumed to equal the soil infiltration rate. Therefore when discussing saturated vertical flow conditions in paper, the terms coefficient of permeability and infiltration rate are equal. The computed k is the infiltration rate.

Soil compaction: Compaction is the process by which loose soils particles are densified to form a compact mass. The objective of compaction is to remove air voids from the soil. The k of compacted laterite soil as a function of number of blows is presented in Fig. 3.

In general, the k decreased as compaction effort increases. The permeability of soil decreases drastically at 4 to 8 blows. At number of blows equal to 4, the soil particles have random orientation that results in larger pore spaces resulting higher k. Increasing the number of blows to 8 per layer reduces the air voids between the particles due to the additional compactive energy that in turn orient the particles normal to its direction of application to fill in the air voids.

The additional compactive energy continues to densify the soil by packing the particle even closer up to a stage where the air voids between the soil particles is very low or minimal, thus limits the compressibility of the soils which in turn reduces the soil permeability. It is observed that the increment of compaction forces gradually minimizes the pore spaces between the soil particles, hence reducing the permeability. Three readings were recorded for each number of blows. The number of blows required to achieve the target k is determined by interpolation. At 32 blows, the corresponding k is 0.33 cm h^{-1} which exceeds the target value. At interpolated 30 times compaction efforts, the recorded k is 0.26 cm h^{-1} as shown in Fig. 3. Hence, the number of compaction equals to 30 is applied in simulating proposed k inside the WFS.

Sand compaction: The relationship between coefficient of permeability and number of blows for sand is shown in Fig. 4. The figure indicates that k is inversely proportional to the number of blows. At 4 blows, the permeability for

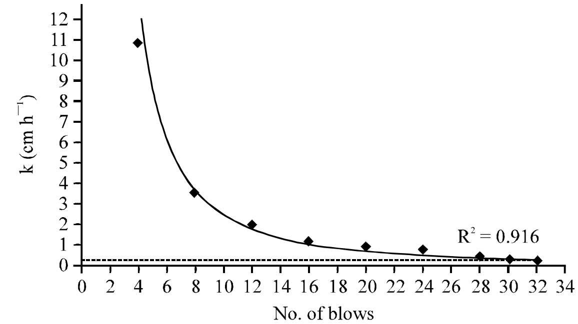

Fig. 3: k as a function of number of blows for soil

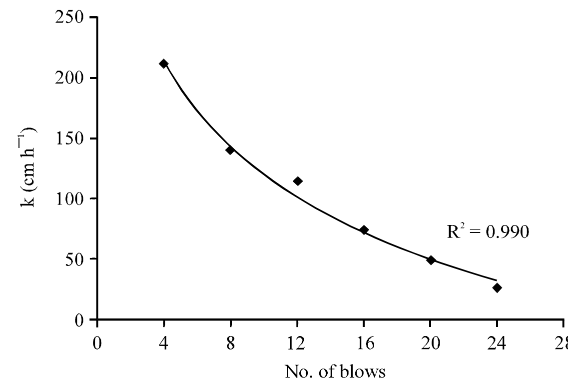

Fig. 4: k as a function of number of blows for sand

sand was approximately 20 times higher than the laterite soil. As the compaction blows continue, the permeability gradually reduced due to air voids reduction. The compacted soils exhibit improved load bearing capacity. After 24 blows, the measured k is approximately 25.42 cm h^{-1}. Thus, to simulate a medium with k of 25.4 cm h^{-1}, the very wet sand is subjected to 24 No. of blows per layer, up to the fourth layer. The compaction inside WFS, body utilizes same laboratory apparatus.

The higher compactive energy caused the sand particles to come closer to each other, thus reducing the air voids. Increased number of blows move the sand particle closer, thus decrease the volume of air voids available for the flow of water, eventually reduces the permeability. Prior to the compaction process, the sand is wetted with pipe water. The sands are compacted most easily at either very dry or very wet conditions. At intermediate water contents, capillary stresses in voids or bulking resist compaction. The water was added to ease the sand particles to slide against each other during compaction. The water acts as lubricating agent, facilitating the soil particles to slide against each other to form a compact mass.

Water level and rainfall intensity relationship: The relationship between water level and rainfall intensity without incorporating reservoir course is shown in Fig. 5. The water level increases slowly with the rainfall intensities which can be illustrated by the gentle slope of the curves. At highest rainfall intensity, the water level increases by almost 100% at fourth quarter.

With 25.4 cm h^{-1} sand permeability, the water starts to go up inside the stainless steel body only at 24.28 cm h^{-1} simulated rainfall intensity. It differs by 1.12 cm h^{-1} or 4.6% from the target k. Theoretically, the water should arise if the rainfall intensity exceeded the soil infiltration rate. If the rainfall intensity equals or lower than the soil infiltration rate, the inclination of water level can never be observed. The transient process of infiltration was complex due to the high nonlinearity of soil water characteristics, soil permeability and initial conditions (Yang *et al*., 2006). It is impossible to get constant or identical permeability rate for each repetition due to particle orientation changes. The graph shows that the water level escalates gradually with the ascending rainfall intensity. After 60 min, the 25.4 cm h^{-1} simulated rainfall records only 0.9 cm increment of water level. However, the water level increased up to 20 cm when subjected to 59.89 cm h^{-1} simulated rainfall over the same period. On the contrary, the increment of the water level is more significant due to lower soil infiltration rate as shown in Fig. 6.

The water level versus rainfall intensity relationship follows closely the trend but not necessarily identical with those of Fig. 5. The graph shows that the water level starts to increase at even the lowest simulated rainfall intensity. This is due to the low soil infiltration rate that limits the percolation of standing water. The soil is compacted to simulate 0.254 cm h^{-1} infiltration rate. Generally, the water level increased linearly with additional rainfall intensity rates. The highest water level recorded is at 59.89 cm h^{-1} simulated rainfall intensity after a 60 min duration. The depth recorded from the datum (saturated soil surface) was 55 cm. At 1.24 cm h^{-1} simulated rainfall intensity, the water level goes up by 0.6 cm. At the highest simulated rainfall intensity, the saturated soil medium with low k permits the water level to increase by 175% higher as compared to water level recorded when subjected to high permeability sand medium. The findings proved that the soil permeability rates considerably affects the water level. The comparison of the water level recorded for the medium with k equal to 0.254 and 25.4 cm h^{-1} is shown in Fig. 7.

Obviously, both soil and sand layers started to generate standing water just above the surface if the simulated rainfall intensity is greater than the medium

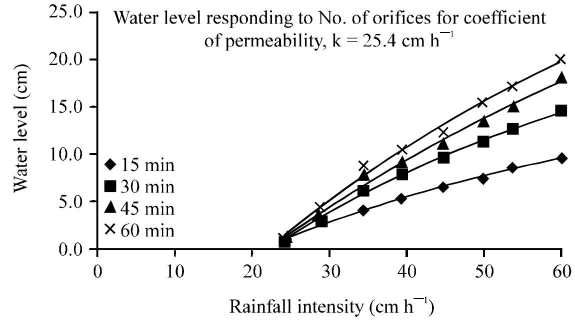

Fig. 5: Water level versus rainfall intensities (k = 25.4 cm h^{-1})

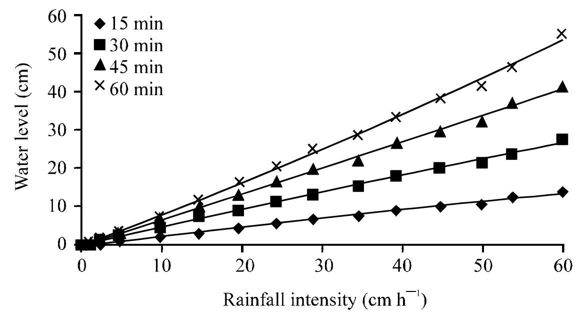

Fig. 6: Water level versus rainfall intensities (k = 0.254 cm h^{-1})

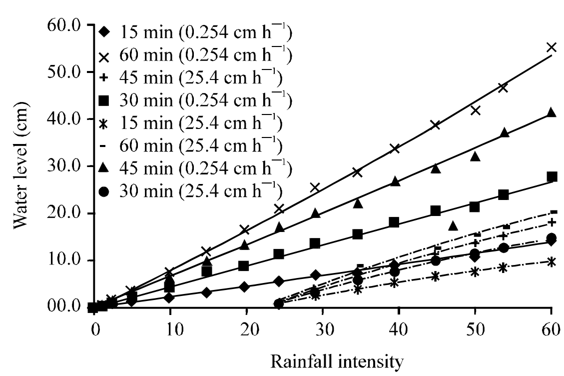

Fig. 7: Water level corresponding to stipulated rainfall intensities for both laterite and sand media

infiltration rates. Figure 6 indicates that the water levels are directly proportional to the rainfall intensities. For both laterite soil and sand bottom layers, the water level constantly increases with ascending simulated rainfall intensities up to 59.89 cm h^{-1} after a 60 min duration. The reason for bigger head differences with rainfall of larger intensities is due to the faster increase in the volume of the water. However, the observed water level is more significant when the simulated rainfall is subjected to lower soil infiltration rate. With the same rainfall intensities simulated, the soil medium with lower k shows higher responding water level and vice versa.

For sand medium with 25.4 cm h^{-1} infiltration rate, the depth of water level recorded between each 15 min interval is getting less as the duration increases. This is due to the effect of pore water pressure and sand permeability. As the water level gets higher, more water percolated through the sand medium due to the increment of pore water pressure. This is due to larger water head differences as the standing water continuously rises. Hence, larger air voids existed between the sand particles allows more water to permeate through. Conversely, the depth of water level recorded between each 15 min interval for 0.254 cm h^{-1} infiltration rate shows almost a constant increment. At each 15 min interval, the water level for soil medium is higher compared to the sand medium with 25.4 cm h^{-1} infiltration rate. From the first 15 min interval until the fourth interval, the depth increased by 13.8, 13.8, 13.9 and 13.5 cm, respectively. This is primarily due constant volume of water falling into the system, despite very small amount of water percolated through the 0.254 cm h^{-1} soil medium throughout the test period.

Water level and discharge time relationship: The proposed stone recharge bed or reservoir course should be able to drain within 12 and 72 h (NAPA, 2008). It is noted that the soil's coefficient of permeability affects discharge time. Figure 8 and 9 show the time taken for water to completely drain through soil medium with 25.4 cm h^{-1} and 0.254 cm h^{-1} soil infiltration rates, respectively.

Figure 8 shows that more discharge time is required as the water level increases. The water level defined as the maximum depth of water recorded after a 60 min rainfall duration incorporating intensities between 24.28 to 59.89 cm h^{-1}. At water level equals to 20 cm, the time taken to completely dry is 32.2 min. The lowest water level recorded is 0.9 cm with 16.7 sec discharge time. The discharge time gradually reduces as the water level gets lower. Similar trend can be observed for soil medium with lower k as illustrated in Fig. 9.

The discharge time recorded with k equals to 0.254 cm h^{-1} gradually rise as the water level increases, compared to soil with higher permeability. The discharge time of 80.5 h required to drain 55 cm water inside the stainless steel body. At 22.3 cm and 22 cm water level for lower and higher k respectively, the previous recorded discharge time approximately 73 times longer. For soil medium the lowest water level recorded is 0.6 cm with 54 min discharge time. The water levels shown are the maximum depth of water recorded after a 60 min rainfall duration incorporating rainfall intensities between 1.24 to 59.89 cm h^{-1}. The findings showed that soil k greatly impact the discharge time required. Thus, the permissible soil infiltration rate between 0.254 to 25.4 cm h^{-1} must be considered during construction of porous parking lots.

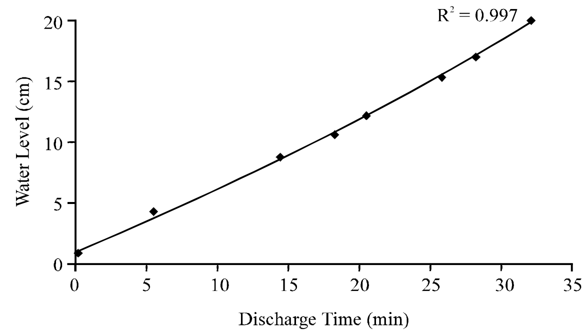

Fig. 8: Discharge Time according to Water Level for k equals to 25.4 cm h^{-1}

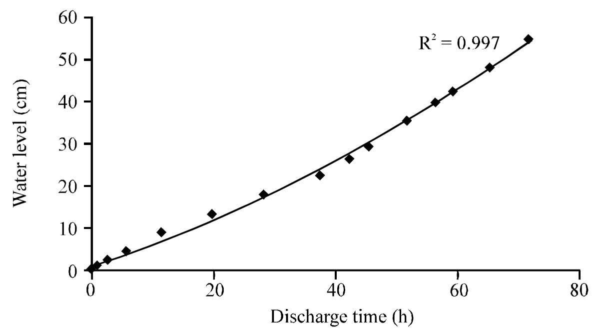

Fig. 9: Discharge Time according to water level for k equals to 0.254 cm h^{-1}

Confirmation on air voids: By introducing reservoir course inside the WFS's body, the water level inside the structure is expected to go up 60% higher than the initial water level tested without the reservoir course. The test utilizes only 0.254 cm h^{-1} soil infiltration rate. Previous study assumes a very low soil permeability of 0.254 cm h^{-1} to determine bed depth requirement for zero discharge (NAPA, 2008). The lowest infiltration rate soil medium was chosen since the critical depth obtained is reasonable for soil with higher infiltration rate as the water will percolate faster, thus reducing the standing water depth.

The data tabulated in Table 6 shows the maximum water depth inside the water flow simulator's body after a 60 min simulated rainfall duration incorporating various rainfall intensities.

With various rainfall intensities simulated, most of the increments recorded approximately 60% inclination of

Table 6: Measured water level due to incorporation of reservoir course

Rainfall intensity (cm h^{-1})	59.90	53.95	49.84	44.69	39.28	34.46	28.9	24.28	19.69	14.72	9.86	4.92	2.44	1.24
Height (cm)														
Before	55.0	48.3	42.5	39.9	35.4	29.5	26.5	22.3	17.9	13.2	8.80	4.30	2.00	0.60
After	126.9	122.5	109.9	102.3	87.8	74.4	64.1	57.0	43.5	31.5	20.7	10.7	4.60	2.20
Increment (%)	56.7	60.6	61.3	61.0	59.7	60.3	58.7	60.9	58.9	58.1	57.6	59.8	56.8	72.3

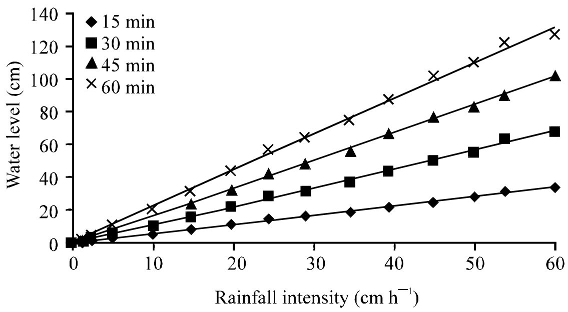

Fig. 10: Water level recorded after incorporating reservoir course

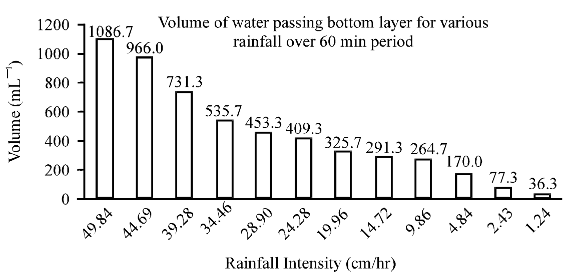

Fig. 11: Volume of water measured according to stipulated rainfall intensity

water level inside the WFS's body with reservoir course structure. The exact 60% rises is almost impractical since it is impossible to have absolute uniform consistency of air voids. The findings proved that the methodology adapted to simulate the reservoir course with designed air voids is reliable. The graph illustrated in Fig. 10 shows the water level increment with k equals to 0.254 cm h^{-1} inside the WFS's body incorporating reservoir course.

The water levels show a constant rise between each quarter due to almost impermeable bottom most layer. The highest water level recorded is at 59.89 cm h^{-1} intensity after a 60 min rainfall duration, recorded 126.9 cm from the datum (saturated soil surface) which exceeded the porous asphalt pavement by 6.9 cm. Given the proposed 110 cm depth of reservoir course, the system is able to capture rainfall with 49.84 cm h^{-1} intensity over a 60 min duration. The volume of water discharge from the soil layer is clearly shown in Fig. 11. At lowest rainfall intensity simulated, the volume measured is almost 30 times lower than 49.84 cm h^{-1} rainfall. At highest rainfall intensity, only 1086.7 mL of water measured attributable to very low soil infiltration rate. The volume measured is descending with decrement of rainfall intensities due to lower pore water pressure experienced inside WFS's body.

CONCLUSION

The laboratory tests of vertical infiltration were conducted using WFS subjected to various rainfall intensities simulated under conditions of saturated soils. Therefore, this newly fabricated WFS at laboratory scale enable studies on infiltration of porous parking lots systems. In addition, the WFS was able to simulate 1.24 to 59.89 cm h^{-1} rainfall intensities. The rainfall intensities had a major impact on the increment of water levels. At maximum 59.89 cm h^{-1}, without reservoir course the highest water level recorded was 55 cm from saturated laterite soil surface which required 80.5 h to completely discharge. The value increased up to 126.9 cm, approximately 60% increment as the reservoir course is introduced. At the same simulated rainfall intensity, the soil medium with higher infiltration rate recorded only 20 cm of water level and completely dry within 32.2 min. The study also proved that the simulated rainfall with higher intensity over a constant duration increased the pore water pressure in the soil more than rainfall with smaller intensity. The new method proposed to imitate reservoir course was reliable and enable prediction of suitable reservoir course depth for porous parking lot system corresponding to various rainfall intensities.

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