Evaluating Porous Pavement for the Mitigation of Stormwater Impacts

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Abstract. Urban stormwater management is one of the biggest challenges facing modern cities. The current stormwater networks are likely to be insufficient under continuous urbanisation and the development of residential areas in addition to increases in rainfall intensity, especially in areas with relatively high slopes, which may thus be subject to flooding. This study aimed to mitigate the overflow rate of the selected case study area using a pervious inlets pavement technique based on porous asphalt concrete pavements with a reservoir layer. The current stormwater network system of the Al-Yarmouk quarter (located at the centre of Karbala City, Iraq) was evaluated by exposing it in simulation to a rainfall intensity equivalent to the 5-year return period and applying storm and sanitary analysis (SSA) and GIS software. The simulation results showed that constructing 50% of the road surface area (5% of the total studied area) with porous asphalt concrete with air voids of about 20% and variable reservoir thicknesses, could reduce water runoff entering the network significantly. Moreover, placing 240 mm, 328 mm and 400 mm of No.75 aggregate layer within reservoir depths would result in further mitigation of surface runoff quantity by about 15%, 30%, and 45%, respectively. A reservoir depth of 240 mm was found to reduce flooding quantities and times by about 71% and 18.29%, respectively, while creating a 328 mm reservoir depth under the porous asphalt results in quantity and time reductions of about 98.5% and 81%, respectively. No surface flooding was observed when reservoir depth was changed to 400 mm.

Keywords: infiltration sump; layer porosity; permeability, porous media; stormwater network.

1. Introduction

Most urbanised areas have suffered from recent increases in impervious areas due to increases in paved roads, building roofs, and sidewalks, and modifications to vegetation [1-5]. All of these factors cause a significant reduction in storm water infiltration and thus increase both the volume and rate of storm water runoff [6, 7]. Moreover, most research suggests that the general trend of rainfall intensity in the future will increase due to climate change effects [8-11], and, according to Liu et al. [12], it is estimated that by 2050, urban areas will contain 85.9% and 64.1% of the population of developed and developing countries, respectively. Such factors explain the increase in observed flooding in some urban areas, particularly where the stormwater network capacity is insufficient to absorb additional runoff, as shown in Figure 1.

Several solutions have been discussed to mitigate flooding impact on urbanised area, and some have already been tested. Permeable pavement technology has been shown to be among the most effective ways of resolving such matters [13-22]. Porous or permeable pavement can be defined as any paved surface which permits water runoff to infiltrates to a reservoir layer structure underneath, which in turns exfiltrates water...
into the surrounding and underlying soil media \[23\]. Many previous studies have reported that permeable pavements show good effectiveness in reducing the volume of stormwater runoff, as well as enhancing water quality \[24-27\]. Permeable pavement may be used as a road surface or as a green roof layer \[4, 6, 28\], and the strategic location of permeable pavement and its routine maintenance are critical to achieving high surface infiltration rates \[14\].

The structure of permeable pavement mainly consists of a pervious surface layer on pervious soil media resting on subgrade. The surface layer may be either pervious concrete or porous asphalt concrete, though the requirements of permeable pavement limit the properties of such materials to ensure both the required porosity and high strength performance \[29, 30\]. The pervious soil underneath the surface layer plays an important role in mitigating and lagging stormwater runoff, as pervious soil properties such as structure, porosity, density, permeability have significant effects on water exfiltration rates \[31\].

Figure 1. Effects of surface flooding on local street traffic movement, Al-Yarmouk sector, Karbala, Iraq

Collins et al. \[18\] investigated the hydrological effects of four different permeable pavement types on the mitigation of stormwater runoff: these were pervious concrete, permeable interlocking concrete pavement with 12.9% and 8.5% openings, and concrete grid pavers with 28%. All of these solutions were significantly better than asphalt pavement in terms of reducing runoff quantity and mitigating stormwater peak flows. In general, the success of a permeable pavement derives from its structural and hydrologic integrity. Structural design must consider the need for pavement strength to support loads \[32\], yet, to date, no standard structural design procedure has been adopted for all permeable pavement types \[31\]. Hydrologic design must consider the capacity required for infiltration, storage, and detention of water to facilitate a sustainable stormwater management approach.

This study aimed to investigate and evaluate the performance of permeable pavement technology with regard to mitigating stormwater surface runoff. The selected case study was a part of the stormwater network in Al-Yarmouk sector, Karbala city, Iraq, which suffers from flooding events in the winter periodically due to its topographical nature.

2. Study area

The scope of the studied area in Karbala City (in the middle of Iraq), covers the Al-Yarmouk district, which has a total area of about 243,750 m\(^2\) (625 m in length, and 390 m in width) and which is divided into four segments, as illustrated in Figure 2. The elevation at ground level varies from 37.37 m to 33.8 m above sea level, as illustrated in the topographical map shown in Figure 3. The total number of designated storm water network manholes is 68 and there are 69 pipes. The minimum and maximum pipe diameters within the network are 315 mm and 500 mm, respectively, and the total length of the network is 3,178 m.
3. Materials and methods

AutoCAD storm and sanitary analysis software (SSA) was utilised in this study to analyse the stormwater network. Information of the existing stormwater network components such as pipes (lengths, slopes, elevations, diameters, and types) and manholes (types and elevations), were collected along with and metrological data, and AutoCAD software was used to transfer the data to an Excel sheet. The catchment area types, areas, and slopes were extracted from a satellite image and a digital elevation model raster was created in GIS. The network component data was refined, and each segment supplied with the required data. Finally, the stormwater network components were imported as shapefiles, using the GIS tool import method,
for analysis. The intensity duration frequency curves (IDF) of Karbala City for various rainfall return periods were implemented, based on previous local research [33].

4. Analysis criteria for stormwater network systems
A modified rational method was used in this study to predict the quantity of surface runoff generated from each sub catchment area. The rational method is traditionally a peak flow method used for storm sewer assessment, as it predicts peak flows using the formula \( Q = CIA \). Here, the intensity (I) was taken from an Intensity-Duration-Frequency (IDF) curve for Karbala City, using the time of concentration of each drainage area as illustrated in Figure 4.

The traditional rational method cannot be used with dynamic wave routing, however, as it is only applicable to a peak flow/steady state condition. Instead, most stormwater analysis software offers a "Modified Rational" runoff method, which incorporates key Rational Method concepts \( (Q = CIA) \) while simultaneously generating a hydrograph.

The Carter formula (Equation 1 [34]) was also adopted in the current work to estimate the time of concentration \( (T_c) \) of water surface runoff, which is divided into two parts: time of inlet and time of flow.

\[
T_c = 0.0977L^{0.6}S^{-0.2}
\] (1)

where \( L \) is the length of flow (m), and \( S \) is the slope of the subbasement.

The head loss produced from water flow in pipes and manholes approaches was calculated using the Hazen-Williams formula (Equation 2 [35]) for a circular pipe section as follows:

\[
h_f = \frac{10.7L}{D^{4.87}} \left(\frac{Q}{C}\right)^{1.85}
\] (2)

where \( H \) is Head loss (m), \( Q \) is flow rate (m³/sec), \( L \) is the length of pipe (m), \( D \) is pipe diameter (m), and \( C \) is the Hazen-Williams coefficient

The Intensity-Duration-Frequency curves for Karbala city, as illustrated in Figure 4, were inserted added to SSA for each return period.

![Figure 4. IDF curves for Karbala City [33]](image-url)
The collected stormwater network data, including details about the pipes, manholes, and catchment areas, were inserted and refined using GIS and Excel software, which made it easy to assign specific data to each part of the network and thus to generate a fully informed shapefile for each segment. The resulting data was then imported into SSA software, as illustrated in Figure 5. The red points represent the manholes (with an approximate surface area of 1.68 m²), while the black lines represent the network pipes and the blue hatched segments are the divided catchments areas. Pipe (conduit) data includes pipe invert elevation for upstream and downstream, manning roughness, length, and diameter, while each manhole (junction) is defined by ground elevation and invert elevation. The catchment area parameters are length of flow, slope, and runoff coefficients. Each contributing sub catchment is connected to the nearest manhole to the downstream side, as water runoff flows upstream to downstream.

Figure 5. System component configuration with SSA

After the required metrological data was defined, a 5-year rainfall intensity return period was selected for the study area, giving a rainfall intensity of about 14.3 mm/hr. The selected duration of storm was 1 hr, and the simulation process was continued for 2 h, to allow observation of how network capacity changes over time after a storm. The minimum time of concentration (5 min) was selected for all catchment areas. A ponding effect with constant flow was enabled in the simulation process to enable any excess inflow ponds that formed on the top of the manholes to re-enter the network when overflow capacity became available. A hydrodynamic routing method with a 30-second time step was selected to capture the peak flow value in the simulation process, while all other hydrodynamic routing parameters were set to defaults except where necessary modification was required to achieve system stability.

5. Pavement characteristics and proposed techniques
A typical porous pavement is shown in Figure 6. To complete the analysis of the proposed porous pavement for the study area, the design parameters and properties of each layer were estimated based on recommended values from previous research work. The existing subgrade soil type is loamy fine sand, with moderate permeability of about 0.001 cm/sec as an average value, as shown in Table 1. The air voids of the top pavement surface (assumed to be porous asphalt concrete) are at a level of about 20%, an average
referred value for porous concrete or asphalt layers [30, 36]. The area of paved streets forms about 8% of the total studied area. The selected porosity of the revisor layer is about 0.4 based on No.57 aggregate being utilised for this purpose [37]. Equation 3 was thus used to determine the depth of the revisor layer (dp), which also functions as a base course for resistance to traffic loads [38]:

\[
d_p = \frac{(P \times R \nu_i \times DA/Ap) - (i.Tf)}{\eta_r}
\]

where \( Ap \) = the permeable pavement surface area, the porosity (\( \eta \)) for No. 57 stone = 0.4., \( dp \) is the depth of reservoir, (i) is infiltration rate for the subgrade soils, \( P \) = The rainfall depth, RVi is either 0.95 (runoff coefficient for impervious cover) or 0.8 (for porous pavement), and DA is the total contributing drainage area. The time to fill the reservoir layer is assumed to be 1 hr.

![Figure 6. Typical cross-section of permeable pavement. Adapted from City of Rockville [39]](image)

**Table 1.** Typical saturated permeability of various soil types [40]

| Soil Layer          | Saturated K soil (cm/s) | Permeability Class |
|---------------------|-------------------------|--------------------|
| Pavement            | 0                       | Very low           |
| Silty loam          | 0.00019                 | Low                |
| Loam                | 0.00037                 |                    |
| Fine sandy loam     | 0.00052                 |                    |
| Sandy loam          | 0.00072                 |                    |
| Loamy fine sand     | 0.001                   |                    |
| Loamy sand          | 0.0017                  |                    |
| Sandy gravelly soils| 0.0058                  | Very high          |
6. Simulation results

Four scenarios were examined in terms of mitigating storm water load on the network depending on rainfall intensity: scenario 1 (the reference case, a normal stormwater system with a rainfall intensity of 14.3 mm/hr), scenario 2 (15% runoff mitigation with 12.15 mm/hr rainfall intensity), scenario 3 (30% of runoff mitigation with 10.01 mm/hr rainfall intensity), and scenario 4 (45% of storm water runoff mitigation or 7.86 mm/hr rainfall intensity). The final three scenarios were planned to determine the dp as a required variable limiting the mitigation percentage. The four scenarios’ results are shown in Figure 7, with black pipes and red pipes representing pipes in under capacity and overcapacity states, respectively, and blue and red manholes representing flooded and unflooded manholes, respectively. All reservoir thicknesses in all mitigating scenarios had areas of about 5% of the total studied area, which is about 50% of the pavement surface area.

The reference scenario results, where no solution for storm water mitigation was added, showed that about 78.5% of network pipes worked at overcapacity, where the ratio of water height to pipe diameter was equal or higher than 1, as shown in Figure 8. Simultaneously, the percentage of flooded manholes was about 11.5%, as shown in Figure 9.

Scenario 2, which incorporated porous asphalt concrete and No. 75 aggregate type in a reservoir layer with a depth of 240 mm, as determined according to Equation 3, resulted in mitigating the storm water runoff quantity by about 15%, which resulted in reducing the percentage of pipes working at overcapacity to 74.3% of the total system length, and reducing the percentage of flooded manholes to 5.8%.

Scenario 3, which involved treating the pavement layers with a reservoir depth of about 328 mm of No.57 aggregate, resulted in mitigating storm water runoff by about 30%. This water load reduction on the network resulted in a reduction in the percentage of the system with overcapacity to 54.35%, and the percentage of flooded manholes to 1.44%, as shown in Figures 8 and 9, respectively.

Finally, scenario 4 mitigated storm water runoff in the network by about 45%, based on a 400 mm reservoir depth under permeable pavement. The percentage of system length with overcapacity was reduced to 22.93%, and no flooded manholes were observed in this scenario.
Figure 7. System statuses for various percentages of storm water runoff mitigation

Figure 8. Percentages of flooded pipes lengths

Figure 9. Percentages of flooded manholes

Figure 10 clarifies the curves for time-inflow discharges and time-outflow discharges for the four studied scenarios. All runoff inflow scenario curves cover 1.2 hr., which means that there was a time lag of about 0.2 hr, while about 1.8 hr was required for runoff to outflow through the system. The difference between inflow and outflow discharges represents flood water. Scenarios 1, 2, and 3 demonstrated inflow discharges higher than the outflow, suggesting that surface flooding would occur. However, scenario 4 displayed an inflow discharge roughly equal to the outflow discharge, suggesting that this scenario is critical, and that the system in that case works under surcharged pressure so that no surface flooding will occur. Figure 11 illustrates the surface flooding discharge with time for all studied scenarios, and scenario 4 has no flooding, while scenario 3 has very low flooding time and quantity as compared with both scenarios 1 and 2. Nevertheless, scenario 2 still significantly reduces flooding time and quantity as compared with scenario 1.
The area under the time discharge curve represents the quantity of flooded water over the pavement surface in each case.

As clarified by Figures 10 and 11, scenario 2 mitigates flooding quantity and time by about 71% and 18.29%, respectively, in comparison with scenario 1. Likewise, scenario 3 reduced these parameters by about 98.5% and 81%. However, scenario 4 had no flooding occurrence at all, reducing both quantity and time of flooding by 100%. Such surface flooding mitigation does not, however, mean that all pipes were not surcharged or at full capacity.

![Figure 10. System inflow-outflow curves for various scenarios](image)

![Figure 11. Total system flooding discharge for various scenarios](image)

Practically, an exponential relation was detected between reservoir layer depth and the mitigation of stormwater runoff, as illustrated in Figure 12. Thus, mitigation of stormwater runoff in the network system of about 15%, 30%, and 45% would be achieved by increasing reservoir depths to 240 mm, 328 mm, and 400 mm, respectively.
Figure 12, Relationship between reservoir layer thickness and mitigation percentage for various cases

A typical graphical comparison between scenarios 1 and 4 is illustrated in Figure 13, which highlights the difference in water level inside the network profiles in a main conduct pipe with a diameter of 600 mm. The figure clearly illustrates the total energy grade line reduction after the mitigation of 45% of stormwater runoff, indicating that such a reduction would allow the network to overcome a large portion of the surcharge pressure and reduce outflow discharge. During investigation of variations in the stormwater network, although some pipes were working over capacity, at higher runoff mitigation percentages, the surcharge pressure on the overall network components was reduced, as shown by the red line illustrated in Figure 13 that represents the total energy grade, which is significantly reduced in scenario 4.
Figure 13. Water profile comparison of the main conduit pipe: reference case (top) and case 4 (bottom)

7. Conclusions
Based on the simulated cases, the following conclusions can be drawn:

1. Covering 50% of the surface road area (5% of the total studied area) with porous pavement with air voids of about 20% and variable reservoir thickness can reduce water runoff entering the network significantly.
2. Placing 240 mm, 328 mm and 400 mm of No.75 aggregate base layers as reservoir depths resulted in mitigation of surface runoff quantity by about 15%, 30%, and 45%, respectively.
3. Mitigating stormwater runoff by 15%, 30%, and 45%, as noted above, resulted in substantial reductions in the quantity and time of surface flooding. A reservoir depth of 240 mm reduced flooding quantity and time by about 71% and 18.29%, respectively, while placing a 328 mm reservoir under a porous surface layer resulted in quantity and time reductions in flooding of about 98.5% and 81%, respectively. No surface flooding was observed when the reservoir depth was changed to 400 mm.
4. A noticeable advantage was obtained when increasing the reservoir layer in that the layer can function as a base course to withstand various traffic loads if a suitable thickness is selected.
5. Some pipes work over capacity even where a 400 mm reservoir depth is used; nevertheless, the surcharge pressure and total energy grade line of the flooded sections are reduced significantly.

8. References

[1] E. Zachary Bean, W. Frederick Hunt, D. J. J. o. I. Alan Bidelspach, and D. Engineering, "Evaluation of four permeable pavement sites in eastern North Carolina for runoff reduction and water quality impacts," vol. 133, no. 6, pp. 583-592, 2007.
[2] A. Goonetilleke, E. Thomas, S. Ginn, and D. J. J. o. E. m. Gilbert, "Understanding the role of land use in urban stormwater quality management," vol. 74, no. 1, pp. 31-42, 2005.
[3] R. Cronshey, "Urban hydrology for small watersheds," US Dept. of Agriculture, Soil Conservation Service, Engineering Division1986.
[4] N. D. VanWoert, D. B. Rowe, J. A. Andresen, C. L. Rugh, R. T. Fernandez, and L. J. J. o. e. q. Xiao, "Green roof stormwater retention: effects of roof surface, slope, and media depth," vol. 34, no. 3, pp. 1036-1044, 2005.

[5] G. E. J. o. H. E. Moglen, "Hydrology and impervious areas," vol. 14, no. 4, pp. 303-304, 2009.

[6] G. Ercolani, E. A. Chiariad, C. Gandolfi, F. Castelli, and D. J. J. o. H. Masseroni, "Evaluating performances of green roofs for stormwater runoff mitigation in a high flood risk urban catchment," vol. 566, pp. 830-845, 2018.

[7] B. K. J. A. i. C. E. Nile, "Effectiveness of hydraulic and hydrologic parameters in assessing storm system flooding," vol. 2018, 2018.

[8] W. H. Hassan, B. K. Nile, B. A. Al-Masody, W. H. Hassan, B. K. Nile, and B. A. J. E. R. Al-Masody, "Climate change effect on storm drainage networks by storm water management model," vol. 22, no. 4, pp. 393-400, 2017.

[9] N. Al-Ansari, M. Abbaldatif, S. Ali, and S. J. O. E. Knutsson, "Long term effect of climate change on rainfall in northwest Iraq," vol. 4, no. 3, pp. 250-263, 2014.

[10] Y. Osman, N. Al-Ansari, M. Abbaldatif, S. B. Aljawad, and S. J. E. Knutsson, "Expected future precipitation in central Iraq using LARS-WG stochastic weather generator," vol. 6, no. 13, pp. 948-959, 2014.

[11] N. Al-Ansari, M. Abdellatif, M. Ezelden, S. S. Ali, S. J. J. O. C. E. Knutsson, and Architecture, "Climate Change and Future Long Term Trends of Rainfall at North-Eastern Part of Iraq," vol. 8, no. 6, pp. 790-805, 2014.

[12] J. Liu, D. J. Sample, C. Bell, and Y. J. W. Guan, "Review and research needs of bioretention used for the treatment of urban stormwater," vol. 6, no. 4, pp. 1069-1099, 2014.

[13] S. Abdollahian, H. Kazemi, T. Rockaway, and V. J. E. Gullapalli, "Stormwater quality benefits of permeable pavement systems with deep aggregate layers," vol. 5, no. 6, p. 68, 2018.

[14] E. Z. Bean, W. F. Hunt, D. A. J. J. o. I. Biilepsch, and D. Engineering, "Field survey of permeable pavement surface infiltration rates," vol. 133, no. 3, pp. 249-255, 2007.

[15] S. Beecham, D. Pezzaniti, and J. Kandasamy, "Stormwater treatment using permeable pavements," in Proceedings of the Institution of Civil Engineers-Water Management, 2012, vol. 165, no. 3, pp. 161-170: Thomas Telford Ltd.

[16] F. Boogaard, T. Lucke, N. Van de Giesen, and F. J. W. Van de Ven, "Evaluating the infiltration performance of eight Dutch permeable pavements using a new full-scale infiltration testing method," vol. 6, no. 7, pp. 2070-2083, 2014.

[17] A. S. Braswell, R. J. Winston, and W. F. J. o. e. m. Hunt, "Hydrologic and water quality performance of permeable pavement with internal water storage over a clay soil in Durham, North Carolina," vol. 224, pp. 277-287, 2018.

[18] K. A. Collins, W. F. Hunt, and J. M. J. o. H. E. Hathaway, "Hydrologic comparison of four types of permeable pavement and standard asphalt in eastern North Carolina," vol. 13, no. 12, pp. 1146-1157, 2008.

[19] J. A. Drake, A. Bradford, and J. J. W. Q. J. o. C. Marsalek, "Review of environmental performance of permeable pavement systems: state of the knowledge," vol. 48, no. 3, pp. 203-222, 2013.

[20] E. A. Fassman, S. D. J. o. I. Blackbourn, and D. Engineering, "Road runoff water-quality mitigation by permeable modular concrete pavers," vol. 137, no. 11, pp. 720-729, 2011.

[21] B. Hunt, S. Stevens, and D. Mayes, "Permeable pavement use and research at two sites in Eastern North Carolina," in Global Solutions for Urban Drainage, 2002, pp. 1-10.

[22] L. Moretti, P. Di Mascio, and C. J. W. Fusco, "Porous Concrete for Pedestrian Pavements," vol. 11, no. 10, p. 2105, 2019.
[23] S. USEPA, "Stormwater technology fact sheet: porous pavement," ed: United States Environmental Protection Agency Washington, DC, 1999.

[24] B. O. Brattebo and D. B. J. W. r. Booth, "Long-term stormwater quantity and quality performance of permeable pavement systems," vol. 37, no. 18, pp. 4369-4376, 2003.

[25] M. Legret, V. J. W. S. Colandini, and Technology, "Effects of a porous pavement with reservoir structure on runoff water: water quality and fate of heavy metals," vol. 39, no. 2, pp. 111-117, 1999.

[26] B. Nile, W. Hassan, and B. J. M. Esmaeel, "An evaluation of flood mitigation using a storm water management model [SWMM] in a residential area in Kerbala, Iraq," vol. 433, no. 1, p. 012001, 2018.

[27] A. Hussein, S. Shahid, K. Basim, and S. Chelliapan, "Modelling stormwater quality of an arid urban catchment," in Applied Mechanics and Materials, 2015, vol. 735, pp. 215-219: Trans Tech Publ.

[28] H. A. Obaid, "Modelling Sewer Overflow of Karbala City with Large Floating Population," Universiti Teknologi Malaysia, 2015.

[29] A. Abdulwahid, N. Al-Shafi’i, and S. F. Al-Busaltan, "Evaluating the Effect of Porous Concrete Pavement Characteristics on Beneath Pavement Layers," in IOP Conference Series: Materials Science and Engineering, 2020, vol. 870, no. 1, p. 012064: IOP Publishing.

[30] O. A. Al-Jawad and S. J. o. U. o. B. f. E. S. Al-Busaltan, "Statistical Modeling for the Characteristics of Open Graded Friction Course Asphalt," vol. 27, no. 1, pp. 366-381, 2019.

[31] U. Kuruppu, A. Rahman, and M. A. J. E. S. Rahman, "Permeable pavement as a stormwater best management practice: a review and discussion," vol. 78, no. 10, p. 327, 2019.

[32] J. Monrose and K. J. C. S. Tota-Maharaj, Air, Water, "Technological review of permeable pavement systems for applications in small island developing states," vol. 46, no. 9, p. 1700168, 2018.

[33] B. K. N. a. H. A. M. A.-H. Maha Obaid Nayel, "Estimation of the Floods that Occur in the Drainage Network During the Rainy Season," Journal of Engineering and Applied Sciences, vol. 13, no. 4, pp. 8178-8187, 2018.

[34] A. J. A. J. E. A. S. O. R. p. h. d. o. a. Azizian, "Uncertainty analysis of time of concentration equations based on first-order-analysis (FOA) method," vol. 341, 2018.

[35] J. C. Guo, Urban flood mitigation and stormwater management. CRC Press, 2017.

[36] S. A.-B. Rand Mahdy, Ola Al-Jawad, "FUNCTIONALITY PROPERTIES OF OPEN GRADE FRICTION COURSE ASPHALT MIXTURES USING SUSTAINABLE MATERIALS: COMPARISON STUDY," Proceedings of the LJMU 19th Annual International Conference on: Highways and Airport Pavement Engineering Asphalt, Pavement Engineering and Infrastructure vol. 19, March 2020.

[37] Y.-J. Choi, D. Ahn, T. H. Nguyen, and J. J. S. Ahn, "Assessment of field compaction of aggregate base materials for permeable pavements based on plate load tests," vol. 10, no. 10, p. 3817, 2018.

[38] K. A. Stephens, P. Law, L. Maclean, D. Reid, and P. J. P. o. t. W. E. F. Graham, "BRITISH COLUMBIA’S STORMWATER MANAGEMENT GUIDEBOOK: INTEGRATING POLICY, SCIENCE AND SITE DESIGN," vol. 2002, no. 2, pp. 1724-1747, 2002.

[39] C. T. Agouridis, M. J. A. Villines, and M. J. D. J. U. o. K. C. o. A. Luck, Lexington, "Permeable pavement for storm water management," vol. 40546, 2011.

[40] S. Ghosh, "Flood Control and Drainage Engineering."