An analysis of stormwater runoff rehabilitation for integrated BIOECODS using EPA-SWMM

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Abstract. Urbanization that is becoming a trend around the globe, including in Malaysia, has led to an increase in impervious areas. This, in turn, has led to severe flood events in most countries. To alleviate the problem, management actions are required to produce the desired result in flow mitigation. Software and modelling tools can offer support in the specific selection of suitable options. This paper shall focus on a study that was conducted at the academic complex of Universiti Teknologi PETRONAS (UTP). The chosen study location was known to experience flooding as the surface runoff did not drain efficiently on rainy days. An integrated BIOECODS was designed in front of Block 13 and Block 14 of the academic complex using EPA-Storm Water Management Model (SWMM). The existing drainage system was simulated using a model to identify the potential flash flood area. A new diversion flow was then designed to allow surface runoff to discharge into artificial lakes acting as temporary storage before the water flows out through the outlet. The interconnection of the drainage network and artificial lakes in UTP was also analysed in this project. For the simulation model input parameters, rainfall of 50-year ARI with a 30-minute duration was assumed as rain intensity. The hyetograph was converted to input time-series data during model development. The analysis showed that node RS12-274 and node RS113-275 exceeded the maximum depth of surface runoff during storm events. Additionally, hotspot flooding area was determined at a lower elevation level of the roadside drain. Hence, mitigation was proposed by realigning the drain slope and providing a new diversion flow channel. The new model analysed using EPA-SWMM, including the bioretention pond, showed that overflow issues would no longer occur. A recommendation could be proposed when using the simulation model, such as considering some modifications to the EPA-SWMM to adopt best management practices (BMPs) in Malaysia.

1. Introduction

Flooding is the most destructive natural disaster experienced in Malaysia. Across the country, including Sabah and Sarawak, there is about 29,800 km² of total land area susceptible to flood hazard that would affect the livelihood of 4.82 million people [1]. Urbanization has been identified as one of the main factors contributing to the rise of stormwater runoff as the storage capacity of urban soils is reduced [2]. The rapid increase in urbanization, along with population growth, has increased Southeast Asia’s regional vulnerability to thunderstorm related hazards such as flash floods [3, 4]. Furthermore, the rise in impervious surfaces and removal of vegetation due to land use modification have brought a massive impact to the characteristics of surface runoff hydrograph where the stormwater runoff volumes and peak flows have risen [5]. In order to alleviate the problem, the Department of Irrigation...
and Drainage (DID) Malaysia has published the Urban Stormwater Management Manual for Malaysia (MSMA). The manual promotes the concept of control-at-source and adopts Best Management Practices (BMPs) for flood control and management of surface runoff. BMPs include facilities such as ecological swale, retention pond, detention pond, etc.

1.1 Sustainable Urban Drainage System (SUDS)
Sustainable drainage should prevent local environmental pollution apart from maintaining an excellent public health barrier, by being able to operate for a long period [6]. Developed in the United Kingdom, the Sustainable Urban Drainage System (SUDS) is a new approach to manage environmental risks from urban runoff [7]. SUDS is widely known as a green engineering approach and design solution that could mimic the natural process of rainwater drainage. The basic idea behind SUDS is the integration of a system to manage storm runoff and handle the flow during extreme rain events. Other than that, it contains a feature of delivering amenity and maintaining ecological function through stormwater management and ease of maintenance. The SUDS concept is presented in three ways, namely Water Quantity, Water Quality, and Amenity and Biodiversity. The concept aims to minimize the impacts of development to runoff quantity and quality, and to maximize amenity as well as biodiversity opportunities [8]. Figure 1 shows the three-way concept of SUDS.

![Figure 1. Three-way concept of SUDS.](image)

1.2 Best Management Practices (BMPs)
Stormwater Best Management Practices (BMPs) are one of the alternatives or actions taken to lessen the effects of land use changes in urban areas on both surface discharge quantity and quality. It adheres to the principle of integrating stormwater management strategies in the early stage of site planning while simultaneously managing the stormwater as close as possible to its source. Urban BMPs such as the porous pavements, bioretention system, permeable patios, green roofs, wet ponds, and dry ponds are common practices implemented in urban areas to address stormwater runoff quantity and quality [9-11]. BMPs are designed to decrease the discharge and flow velocity through the increase of infiltration rate and filtration of the soil layer. Another goal of BMPs is to enhance the water quality from impervious surfaces in developed areas [12]. Traditionally, BMPs facilities such as wet ponds and dry ponds have been employed in a centralized manner where they are located far away from development to mitigate peak discharge and minimize hydrologic alterations. However, this effort is yet to solve hydrologic issues [13]. Recently, BMPs have also been implemented in a decentralized manner to manage stormwater runoff in the landscape and be closer to its source with an emphasis on infiltration, water retention of the landscape, and integration with the urban design.

1.3 Bio-Ecological Drainage System (BIOECODS)
The Bio-Ecological Drainage System (BIOECODS) was launched in 2002 to fulfill the requirements of the Urban Stormwater Management Manual for Malaysia (MSMA). The approach was designed by the River Engineering and Urban Drainage Research Centre (REDAC), Universiti Sains Malaysia to
improve control of stormwater runoff quality and quantity. BIOECODS application has been designed to deal with overflooding, river pollution, and lack of water uses in Malaysia. Its implementation has been successful in flow attenuation and pollutant removal from stormwater discharge in pre-developed and developed areas. Such a technique reduces the discharge of stormwater from impervious areas through infiltration while simultaneously managing and treating the stormwater quality [14]. This pilot project was constructed on the engineering campus of Universiti Sains Malaysia (USM), and has been the prime example of a sustainable urban drainage system [1]. BIOECODS combines three techniques based on control-at-the-source approach, namely storage, conveyance, and infiltration to manage stormwater. The drainage system consists of three major components including ecological swale, retention pond, and detention pond to enhance the infiltration rate and performance in volume, as well as maximum velocity of stormwater runoff attenuation. With a combination of these facilities, the runoff lag time will increase along with the reduction of runoff rate and volume [12]. Besides that, pollutants in stormwater runoff can be removed through settling and bioinfiltration through the integration of BIOECODS [15].

The integrated BIOECODS has been effective in controlling stormwater quantity and quality, and has also acted as a flood protector and runoff controller. However, there is no standardized solution that is effective across all locations as many constituents such as the watershed-scale, natural characteristics, and human activities may differ notably from one place to another. A major difficulty faced by decision makers is in selecting the best combination of practical management strategies. Software and modeling tools are therefore required to support the specific selection of applicable options. Management actions are also required throughout the watershed to accomplish the desired results on flow mitigation and, at the same time, reduce the pollutant.

In this project, two new bioretention ponds were integrated using BIOECODS to mitigate the flash flood behind Block 13 and Block 14 in Universiti Teknologi PETRONAS (UTP). A new diversion channel was proposed to divert the discharge water into these two ponds. Existing drainage of the UTP campus and the artificial lakes were evaluated using EPA-Storm Water Management Model (SWMM) to identify the interconnected system. Relevant data were collected and used to assess the performance of a new drainage system and bioretention ponds to mitigate the stormwater runoff.

2. Methodology

2.1 Site location

The site of the project was situated at the Universiti Teknologi PETRONAS (UTP) academic complex which was known to experience flooding issues as the surface runoff did not discharge efficiently on rainy days. The road near Blocks 13 and 14 that was built with permeable block pavement systems encouraged some infiltration of rainwater but was unable to stop the flooding due to the high intensity of rainfall within a short duration. This problem was compounded by the construction works of Blocks 11 and 12 beside the existing buildings with soil and mud from the construction site possibly blocking the discharge inlet of the roadside drain, along the road behind Blocks 13 and 14.

2.2 Data collection

2.2.1 Site investigation. A site investigation was conducted to collect the relevant data and information before modelling was performed on the computer. Information including the location of the sump and current drain, characteristics of the drainage system (length, invert level, size), and sub-catchment area of the project site was collected. The existing road in front of Blocks 13 and 14 had a small drainage system that catered to the road surface runoff and parts of runoff from the surrounding buildings. The inlet of the drain was about 20 cm in diameter and the drain was located 6.5 m to the center along the roadside. The hilly area nearby had a large monsoon drain measuring 1600 mm wide x 900 mm deep, which only catered to the discharge runoff from the hillside and the surface runoff from the Pocket C open space parking. Based on observation of the topographic map, the catchment area collected
surface runoff from nearby commercial buildings (45% of the area), the road (35% of the area), and grass which accounted for about 20% of the total area, which flowed into the roadside drain.

2.2.2. Land survey. It was important to understand the dimension and topography of the site to ensure the design would fit the site. In planning a project, accurate base information would provide durable construction and prevent costly mistakes, while at the same time, ensure the highest quality control. Due to the limited sources, some of the elevations in the study area were unknown. This could affect the reading and result of the analysis of simulation. A land survey was required to gather essential information on the topography of the study area, its upstream and downstream, invert levels of channels, channel slopes, and other necessary data. The surveyed data were then combined with existing AutoCAD drawing to form a detailed topographic map of the project study area.

2.3 EPA Storm Water Management Modeling (SWMM)

EPA-Storm Water Management Model (SWMM) was a dynamic rainfall-runoff simulation model. It has been used for a single event or long-term simulation of water runoff quality and quantity. This simulation was developed to help support all stormwater management at different stages including local, state, and national. The aim was to reduce runoff through infiltration and retention, as well as to help minimize the discharge that caused impairment of water bodies.

2.3.1. Model Development. In EPA-SWMM, a reference or guide of the location was needed before inserting any nodes or conduits. A backdrop image obtained using Google Maps was loaded as a reference image in EPA-SWMM. The hydraulic component such as nodes and links was later inserted. These physical components would allow users to gain a clear visual image of the drainage system. Afterwards, the hydrology component including rain gauges, sub-catchment and BMPs control was inserted. Figure 2 shows the model created using SWMM.

![Figure 2. Layout Plan of Drainage in Block13 and Block14.](image)

Figure 3 shows the existing drainage details including the inlet level of the drain and sump, and drain flow direction. The existing roadside drain had a diameter of 300 mm in a pipe buried below the ground. Parts of rainwater flowed from the gutter drain on top of Block 13 and Block 14 buildings and discharged into the roadside drain. The outlet of the roadside drain flowed into an underground culvert and eventually diverted out into a retention pond in UTP eco-park.
2.4 Inputs and parameters

2.4.1. Design Rainfall Estimate. Intensity-Duration-Frequency (IDF) curves represented the standard form of design rainfall data required in peak flow rate estimation. The IDF was developed based on the historical rainfall data, and these data were available for most geographical locations in Malaysia. Since the project area in this study was in Bandar Seri Iskandar, Perak, the nearest rainfall station was Rumah Pam Kubang Haji. The average return period of rainfall was assumed at 50 years, and the design rainfall duration was about 30 minutes. The designed rainfall intensity was determined using the formula below:

\[ i = \frac{10T^k}{(d+\theta)^\eta} \]  

Where,
- \( i \) = Average rainfall intensity
- \( T \) = Average Return Interval (ARI) of 50 years
- \( d \) = Storm duration of 30 minutes (in hour)
- IDF constants: \( \lambda = 52.343 \)
- \( k = 0.164 \)
- \( \theta = 0.177 \)
- \( \eta = 0.840 \)

By inserting the constant parameters into the formula, the calculated rainfall intensity was 137.97 mm/hr for a rainfall duration of 30 minutes.

2.4.2. Time Series- Temporal Pattern of Rainfall Intensity. In EPA-SWMM, the rainfall data was inserted using the time series method. According to Hydrological Procedure No. 1 (Revised 2015), the regional storm profile was divided into 5 regions. The final region storm profiles were obtained using average mass curves from states in each derived region. Thus, this normalized design rainfall temporal pattern adopted the MSMA 2nd Edition to create a rainfall hyetograph.

The project study area was located in the northwest region, and the temporal pattern of rainfall intensity for a 30-minute duration is shown in table 1.
Table 1. Temporal Pattern of Rainfall Intensity for 30 minutes duration

| Rainfall Intensity (mm/hr) | Time duration (minutes) | Block fraction | Average rainfall intensity (mm/hr) |
|----------------------------|-------------------------|----------------|-----------------------------------|
| 137.97                     | 5                       | 0.158          | 21.800                            |
|                            | 10                      | 0.161          | 22.214                            |
|                            | 15                      | 0.210          | 28.974                            |
|                            | 20                      | 0.173          | 23.869                            |
|                            | 25                      | 0.158          | 21.800                            |
|                            | 30                      | 0.141          | 19.454                            |

The results of estimated average rainfall intensity were inserted into EPA-SWMM and presented in graph form as shown in figure 4.

Figure 4. Time Series for 50-year ARI- 30minutes duration in EPA-SWMM.

3. Results and discussion

This project consisted of 48 sub-catchments, 39 nodes, 39 links, and three outflow nodes. All the outflow nodes would divert the surface runoff to UTP eco-park. However, in this project, only one outflow node was taken for calculating and determining the flooding area. Figure 5 shows the project site plan in EPA-SWMM.

Figure 5. Project Site Plan in EPA-SWMM

3.1 Determination of flooding area using EPA-SWMM

There was a total of 0.437 ha of sub-catchment area that contributed to the surface runoff. Along the total roadside length, about 89 m of culvert pipe was involved in discharging the surface runoff to the outlet. This culvert pipe was buried 0.5 m below the ground level and had a diameter of about 300 mm. By running the simulation, the nodes and links were displayed in different colors to represent the
water depth of runoff stormwater. EPA-SWMM was a dynamic tool to track the hydrological performance of hydraulic components from time to time. Figures 6 and 7 show the result of water depth condition after 15 minutes of a rain event. Node RS1-274 had water depth over 0.5 m, which exceeded the maximum inlet height of the inlet sump. Therefore, an overflow of rainwater during the first 15 minutes of rainfall event could be assumed based on this result.

![Figure 6. Hydrological performance after 15 minutes of rainstorm event.](image1)

![Figure 7. Water Elevation Profile after first 15 minutes of rainstorm event.](image2)

Figure 8 shows the result after 30 minutes of rain event. The color red indicated that Node RS12-274 was still experiencing overflow condition, and water was unable to discharge through the inlet sump. Besides that, the culvert pipe at link 57 also experienced critical condition as it was submerged in rainwater runoff. Figure 9 shows the water elevation profile where the conduit between RS12-274 and RS13-275 was fully saturated. Thus, overflooding could be predicted in this area.

![Figure 8. Hydrological performance after 30 minutes of rainstorm event.](image3)
Figure 9. Water Elevation Profile after first 30 minutes of rainstorm event.

Figure 10 is a result of simulation after 45 minutes of the rain event. The nodes and links area were indicated in the colors cyan and blue. This signified that the runoff flow was decreasing in amount. Figure 11 shows the water elevation profile where the water depth had dropped and water was discharged to the outlet.

Figure 10. Hydrological performance after 45 minutes of rainstorm event.

Figure 11. Water Elevation Profile after 45 minutes of rainfall event start.

3.2 Design of bioretention system
The goal of the bioretention pond was to treat large amount of sudden rainwater runoff by using biological uptake and porous media filtration processes. The existing pipe culvert drainage could not discharge the high amount of stormwater in a short time. Therefore, the pond was proposed to capture and control the rainstorm water before releasing it back into the outlet of the drain. MSMA 2nd Edition was used as a reference to design the bioretention pond.

3.2.1 Water quantity volume. Water quantity volume was estimated based on 24.94 mm precipitation calculated using EPA-SWMM. In the project study area, the total catchment area was about 0.2913 ha. The catchment area was divided into many sub-catchment areas with different land uses and sizes. The
runoff coefficient was based on the type of land use. Table 2 shows the water quality volume of this pond.

Table 2. Water Quality Volume.

| Land use     | Runoff Coefficient, C | Precipitation (mm) | Area (m²) | WQv (m³) |
|--------------|-----------------------|--------------------|-----------|----------|
| Commercial   | 0.95                  | 24.94              | 585       | 13.86    |
| Open Space   | 0.50                  | 24.94              | 449       | 5.599    |
| Permeable Road | 0.725             | 24.94              | 1879      | 33.975   |
| **Total water quality volume** | **53.434** |                     |           |          |

3.2.2. Soil mixture of planting bed. Planting soil bed would provide nutrients and water to allow plant life in the bioretention pond. According to MSMA 2nd Edition, the total depth of planting soil bed should range between 450 to 1000 mm, and the planting soils were recommended to be sandy loam, loam texture with clay content ranging from 10 to 25%. In designing the planting bed, 450 mm depth was adopted, while porosity was considered at 25%. The maximum clay content was 5%, and the coefficient of permeability was 26 mm/hr, which was about 0.624 m/day.

3.2.3. Surface area of filter bed. The surface area of bioretention system should follow the physical specification and geometry as shown in Table 3 below:

Table 3. Physical Specification and Geometry.

| Parameter                              | Specification                  |
|----------------------------------------|-------------------------------|
| Minimum Size                           | 3 m wide by 6 m long          |
| Length and Width Ratio                 | 2:1                           |
| Maximum Emptying Time                  | < 24 hours                    |
| Permeability of Planting Bed           | > 13 mm/hr                    |
| Ponding Depth                          | 150mm - 300 mm                |
| Depth to Groundwater Table             | > 0.6m                        |

The required filter bed area ($A_f$) was calculated based on Darcy’s Law, and the equation was:

$$A_f = \frac{(WQ_v)(d_f)}{[(k)(h_f+d_f)(t_f)]} \quad (2)$$

Where,

- $A_f$ = The surface area of filter bed (m²) ($102.76$)
- $WQ_v$ = Water Quality Volume (m³) ($53.434$)
- $d_f$ = Filter bed depth (m) ($0.45$)
- $k$ = Coefficient of permeability of filter media (m/day) ($0.312$)
- $h_f$ = The average height of water above the filter bed (m) ($0.3$)
- $t_f$ = Design filter bed drain time (day) ($1.0$)

Based on the equation above, the calculated surface area of the filter bed was 102.76 m². The recommended location of the bioretention pond was in front of Block 13. An estimated size of 8 m
wide x 13 m long (Total surface area 104m²) was proposed as the size of the pond. However, this size was too big and would cause inconvenience to road users. Thus, the bioretention pond was later divided into two smaller ponds with the size of 7 m x 8 m each (Total 2 pond 112m²). Inflow divider was provided to allow the runoff to discharge into two ponds simultaneously.

3.2.4. Maximum infiltration rate. In the case of an impermeable bioretention system, the maximum infiltration rate through filtration media must be determined for the subsoil drain to be sized. The maximum infiltration rate reaching the perforated pipe at the base of the soil media was calculated using the equation:

\[ Q_{max} = kL_b W_b \frac{h_f + d_f}{d_f} \]  

Where,

- \( Q_{max} \) = Maximum infiltration rate (m³/s) 
- \( k \) = Hydraulic conductivity of the filter bed (m/s) 
- \( W_b \) = Base Width of the ponded cross-section above the filter bed (m) 
- \( L_b \) = The base length of the bioretention zone (m) 
- \( h_f \) = Height of the water above the filter bed (m) 
- \( d_f \) = Depth of filter media (m)

The calculated maximum infiltration rate was 6.74 x 10⁻⁴ m³/s. However, the suitability of the above formula needed to be assessed for each site based on the influence of both annual maximum groundwater level and infiltration capacity of surrounding natural soils on the bioretention system.

3.2.5. Subsoil / Underdrain Pipe. Subsoil pipes or perforated pipes would be placed at the base of the impermeable bioretention system to cater to and collect the infiltrated water for conveyance downstream. In order to calculate the capacity of flow through the perforations, orifice flow conditions were assumed, and the number and size of perforations would be determined. The impermeable bioretention systems employed a UPVC perforated sub-soil pipe to convey the stormwater runoff downstream. Table 4 shows some of the details needed to determine the flow per perforation:
### Table 4. Details of flow per perforation

| Type                                      | Value   |
|-------------------------------------------|---------|
| Diameter of Perforated Pipe               | 100 mm  |
| Diameter of Perforations                  | 5 mm    |
| Spacing of Perforations                   | 25 mm   |
| Circumference of Perforated Pipe          | 314.2 mm c/c |
| Number of Rows (of Perforation)           | 12.57   |
| Number of Column (of Perforation/m)       | 40.00   |
| Number of Perforations (assumed 50% blockage) | 502.72 |
| Number of Perforations                    | 251.36  |
| Area of each Perforation                  | 19.63 mm² |
| Total Area of Orifices, A                 | 0.0049 m² |

The maximum head of water above the pipe (h) was determined at 1.05 m, and the flow per perforation was estimated using the equation below:

\[
Q_{\text{perf}} = \frac{CA\sqrt{2gh}}{\psi} \quad (4)
\]

Where,
- \(Q_{\text{perf}}\) = Flow per perforation (m³/s) \(= 0.00672\)
- \(A\) = Total area of orifice (m²) \(= 0.0049\)
- \(H\) = Maximum Hw above pipe (m) \(= 1.05\)
- \(C\) = Orifice Coefficient \(= 0.6\)
- \(\psi\) = Blockage factor \(= 2\)
- \(G\) = Gravity constant (m²/s) \(= 9.81\)

Given the calculated flow per perforation per meter was 0.00672 m³/s, the length of base pond was 8 m. Hence the total inlet capacity: 0.00672 x 8 = 0.05346 m³/s. The total inlet capacity (0.05346 m³/s) was larger than the maximum infiltration rate (0.000674 m³/s). Therefore, a single pipe of 100 mm diameter was enough to pass the flow into the perforation. The flow of the circular perforated pipe had to be determined to ensure the drain capacity was enough to cater to the infiltrated flow. This was done using Manning Equation:

\[
Q_{\text{pipe}} = \frac{0.312}{n} D^{2.67} S_0^{0.5} \quad (5)
\]

Where,
- \(Q_{\text{pipe}}\) = Flow of perforated pipe (m³/s) \(= 0.0061\)
- \(n\) = Manning Coefficient of UPVC \(= 0.011\)
- \(D\) = Pipe Diameter (m) \(= 0.1\)
- \(S_0\) = Slope Hydraulic Grade Line (m/m) \(= 0.01\)
The flow of perforated pipe was \((0.0061 \text{ m}^3/\text{s})\), which was higher than the maximum infiltration rate \((0.000674 \text{ m}^3/\text{s})\), hence the 100 mm diameter perforated pipe was adequate for the bioretention system.

3.2.6. Size of Overflow Sump. Given the peak flow calculated using EPA-SWMM, the flow as discharged from the roadside drain was about \(0.045 \text{ m}^3/\text{s}\). When it entered the flow divider, it would divide half of the discharge, where each bioretention pond shall receive \(0.0225 \text{ m}^3/\text{s}\) of flow from the roadside drain. For pond 1 and pond 2, the gutter drain of Block 13 also contributed to the inflow. Total inflow capacity for the first pond was \(0.0285 \text{ m}^3/\text{s}\), while it was \(0.0265 \text{ m}^3/\text{s}\) for the second pond. The overflow sump size was determined by checking for drowned outlet conditions using the orifice equation:

\[
A = \frac{Q_{\text{overflow}}}{c \sqrt{2gh}}
\]

Where,
- \(Q_{\text{overflow}}\) = Overflow (orifice) discharge \((0.0285)\)
- \(C\) = Flow Coefficient \((0.6)\)
- \(h\) = Headwater above weir crest \((0.10)\)
- \(A\) = Orifice Area \((m^2)\) \((0.034)\)
- \(g\) = Gravity constant\((m^2/s)\) \((9.81)\)

The discharge area required was \(A = 0.034 \text{ m}^2\), which could be provided by 300 mm x 300 mm overflow sump \((\text{Area} = 0.09 \text{ m}^2)\). The size of the overflow sump had to consider an allowance for the grate bars, where it would affect the value of perimeter and area.

3.3 Proposed bioretention system

Two bioretention ponds would be provided to cater to the surface runoff by installing a diversion flow drain. To ensure all runoff would come into these ponds before reaching the outflow node, two conduit links together with inlet sump were required to readjust the slope direction to ensure the runoff would go to node RS11-273. From node RS11-273, a culvert pipe with 300 mm diameter was proposed to discharge the runoff to the pond. From the simulation result, there was a total of \(0.045 \text{ m}^3/\text{s}\) of peak discharge reaching node RS11-273, and the flow divider diverted half the discharge to pond 1 and another half to pond 2, respectively. For pond 1, there was another inlet flow about \(0.006 \text{ m}^3/\text{s}\), thus taking the total to \(0.0285 \text{ m}^3/\text{s}\). On the other hand, pond 2 received an inlet flow of \(0.0225 \text{ m}^3/\text{s}\) from the roadside drain and second inlet flow of \(0.004 \text{ m}^3/\text{s}\) from the gutter drain. Figure 12 shows the layout plan of the bioretention system. Both pond size was 8 m long x 7 m wide to prevent soil erosion. Overflow sump with the of size 300 mm x 300 m was provided in both ponds to cater to the excess flow. The discharge would eventually discharge out from two ponds into the existing underground pipe, connected to the inlet of UTP eco-park lake. Figures 13, 14, and 15 show the details of bioretention pond 1 and 2 and a cross-section view of the pond, respectively.
3.4 Analysis of new drainage network

By inserting the properties of the bioretention system component, analysis of the model was done to measure hydrological performance. Figure 16 shows the interface of BMPs Control Editor in EPA-SWMM; where the model could perform infiltration analysis of the soil layer. Figure 17 shows the
result of water depth after 15 minutes of the rainstorm event. Node 36 was the flow divider where it diverted the discharge into separate flow to pond 1 and pond 2. The nodes and links were within the maximum depth of the drainage. Therefore, no overflooding of runoff would occur. Figure 18 shows the water elevation profile in the first 15 minutes of the rainstorm event.

Figures 19 and 20 represent data for rainstorm events after 30 minutes of the rainstorm event. This was the most critical part, as the highest surface runoff was received in this period. The nodes and links were still below the maximum depth. Subsequently, figures 21 and 22 show the result after 45 minutes of the rainstorm event. The depth of water would start to reduce as the discharge was stored in the bioretention pond before it diverted out to the outlet. Thus, the overflow problem did not occur.
Figure 19. Hydrological performance in the first 30 minutes of the rainfall event.

Figure 20. Water Elevation Profile in the first 30 minutes of rainstorm event.

Figure 21. Hydrological performance in the first 45 minutes of the rainfall event.

Figure 22. Water Elevation Profile in the first 45 minutes of rainstorm event.

4. Conclusion and recommendation
To conclude, it has been shown that stormwater runoff analysis for the project study area was able to be performed using EPA-SWMM. In this project, the interconnection of the drainage network and
artificial lakes in UTP was assessed through a simulation model. Flooding issue would normally occur in front of Blocks 13 and 14 in UTP after each rainstorm event. The situation was compounded by the undergoing construction works for UTP Block 11 and Block 12. Debris from the construction site had possibly blocked parts of the drain flow and restricted the discharge from diverting into its original culvert. By using this simulation, the possible hotspots areas of flash flood were able to be determined. The potential area with flash flood issues was at node RS12-274 and node RS13-275. Thus, a mitigation plan was proposed by realigning the existing slope of node RS11-273, RS12-274, and RS13-275 to lead all surface runoff to node RS11-273. Next, a new diversion flow channel would also be installed where the flow would be discharged to a newly integrated bioretention system. From the simulation result, a new diversion flow was determined in the design of the bioretention system. Data of the peak discharge from the realigned drainage system were required to design components of the bioretention system using MSMA 2nd Edition. Two ponds with the size of 8 m long x 7 m wide were required to cater to the surface runoff from UTP Block 13 and Block 14. The integrated bioretention system was taken into consideration for an analysis using EPA-SWMM. The result showed that flash floods would no longer occur. A recommendation could be proposed when using the simulation model, such as considering some modifications to the EPA-SWMM by adopting BMPs in Malaysia to produce better result to meet MSMA standards.

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