MODELLING THE IMPACT OF WATER SENSITIVE URBAN DESIGN ON PLUVIAL FLOOD MANAGEMENT IN A TROPICAL CLIMATE

BY

MISS LIHOUN TEANG

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE
DESIGN, BUSINESS AND TECHNOLOGY MANAGEMENT
FACULTY OF ARCHITECTURE AND PLANNING
THAMMASAT UNIVERSITY
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was approved as partial fulfillment of the requirements for the degree of Master of Science (Design, Business and Technology Management)

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Thesis Title	MODELLING THE IMPACT OF WATER SENSITIVE URBAN DESIGN ON PLUVIAL FLOOD MANAGEMENT IN A TROPICAL CLIMATE
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ABSTRACT

Increasing urbanization and population growth in Thailand have altered the natural urban environment and waterways, requiring metropolitan Bangkok and the surrounding areas to develop a more sustainable and effective stormwater management plan. Pluvial flooding has become a particular challenge for these areas, especially due to frequent, high intensity rainfall events associated with a tropical climate that conventional stormwater management is not always able to effectively accommodate. This study therefore aims to introduce a newer stormwater management approach, Water Sensitive Urban Design (WSUD), which is supported by dynamic mathematical modelling, to explore and emphasize its ability in managing pluvial flooding for a mixed residential, commercial, and industrial area of peri-urban Bangkok. Different landscape architectural designs from the Thammasat Nava Nakorn Smart District Project, including bioswales, bioretention cell, raingarden and detention pond were used as the basis of the performance evaluation. A Personal Computer version of the Stormwater Management (PCSWMM) was applied to understand flooding behaviour under a 2-year design storm (72 mm for 24 hours). Design options to improve performance of the individual designs and their connectivity with respect to runoff volume and flood, frequency, duration, and severity were then evaluated. WSUD performance was varied from one feature to another depending on the design size and its ability to capture runoff volume. The WSUD scenarios had less impact on surface flood duration and peak surface flood rate, but arguably volume reduction is more important than peak reduction for pluvial flooding. Flood reduction benefits under each feature were smaller for the entire system (Nava Nakorn) compared to the localized catchment area which is attributed to the fact that WSUDs area shared a small proportion of the entire study area. Simulation result for the localized impact showed that flooding volume was reduced by between 6 – 78 % for all scenarios when bioretention cells were considered and up to 100 % for all scenarios that included bioswale, raingarden and detention pond designs. Volume reduction for the entire Nava Nakorn Estate ranged between (-14 – 3.2 %) for the highest impact scenarios under each WSUD design. Furthermore, the findings indicated that flood volume and its severity were greatly decreased when all WSUDs being established together with the maximum reduction of 7.5 % for the entire Nava Nakorn Estate. Increasing percentage of pervious area (surface area occupied by the design) would result in better performance of the designs but would also require higher investment cost.

Keywords: Pluvial flooding, PCSWMM, WSUD, Tropical climate, Thailand

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Miss Lihoun Teang

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CHAPTER 1
INTRODUCTION

1.1 Background Information

Rapid urbanization modifies hydrologic processes, with impacts including more extreme floods, possible lower (and in some cases increased) baseflows, declining groundwater levels, and poorer water quality (Leopold, 1968; Price, 2011; Boyd, 2011; Suriya and Mudgal, 2012; Miller et al., 2014; Ren et al., 2014; Lorphensri et al., 2016; Patra et al., 2018; Kumar et al., 2020). The process of urbanization generally involves the transformation of vegetated land area into impervious surface area where the natural infiltration systems are replaced with a conventional (i.e., hard engineering) drainage system to manage water flow. However, with both frequent and extreme rainfall, the conveyance capacity of a conventional system can be exceeded, which results in localized flooding (Irvine, 2013; Moftakhari et al., 2018). The conventional drainage system is used to control extreme flood events and can be classified into two types, combined sewer and separate sewer systems. In general, the combined system consists of only one pipe functioning for both stormwater runoff and sanitary flow, in contrast to the separate system where stormwater and sanitary sewer are designed to flow in different pipes (Boyd, 2011). As for stormwater runoff, the minor drainage systems are designed for 2-5-year return periods. Minor systems generally are considered a pipe place along the curbside or property boundaries to control just runoff from the streets. On the other hand, major drainage systems are designed to convey flow up to 100-year return periods and likely consist of natural waterway, interceptor, and water impoundment (Melbourne Water, 2017).

Sustainable stormwater management as opposed to the conventional urban drainage approach addresses sustainable issues by integrating the GI (Green Infrastructure) approach into the urban landscape. With GI, the hard engineering system is incorporated with the natural areas such as wetland and pervious surfaces to manage urban flooding. This not only helps to reduce flooding but also increases landscape amenity values, groundwater recharge and stormwater quality improvement (Beza et al., 2019; Chow et al., 2014; Hartman et al., 2019) which then creates both monetary (i.e. food provisioning and economic benefits of fisheries, vegetation harvesting, and other water resources retrieving; avoided costs from flood damage) and non-monetary ecosystem service valuation (i.e. climate change mitigation, cultural and recreational services) (Sharma et al., 2016; Weber et al., 2015).

Many studies were conducted to understand effectiveness of nature-based solutions, a form of GI, with regard to urban flood management and those include the studies by Davis and Naumann (2017); Santoro et al. (2019); Oral et al. (2020); Hamel and Tan, 2021; Ruangpan et al. (2020). With proper design and collaboration between related stakeholders, the nature-based solutions were i considered as a promising sustainable approach to reduce urban flooding with potential cost-saving and additional environmental benefits compared to the conventional stormwater management. A range of terminologies has been developed and adopted to represent and describe sustainable stormwater management according to local and regional perspective and is driven by principle and context (includes existing infrastructure, water cycle, and climatic condition), focused development, and social expectation that each country wants to achieve. However, the mutual aim of adopting these terminologies is conveying the objectives, designs, and benefits to be more specific and in a new holistic approach. These terminologies include Low Impact Development (LID) which is commonly used in North America and New Zealand, Sustainable Urban drainage System (SuDS) which is a UK term, Water Sensitive Urban Design (WSUD) which was initiated in Australia, Best Management Practices (BMPs) and Green Stormwater Infrastructure (GSI) which was originally drafted and emerged in US, respectively, Alternative Techniques (ATs) which has begun to be used in France, and Sponge City which is popularly used in China (Fletcher et al., 2014; Zevenbergen & Pathirana, 2018; Lashford et al., 2019; Radcliffe, 2019).

WSUD was introduced around the 1990s and is a parallel initiative to LID. It has widely been implemented by many municipalities across Australia and internationally

(Radcliffe, 2019). Beside focusing on sustainability, which is to minimize the cost of stormwater management by taking the benefit of nature as a core, WSUD considers proactive processes rather than the reactive one and generally involves all the processes of urban planning and design, partnerships between planners, architects, landscape architects and engineers. The WSUD philosophy is not only to help minimize the drainage infrastructure development costs but also protect and enhance the natural water system in urban developments, integrate stormwater treatment into the landscape and maximize the visual and recreational amenity of development, reduce runoff and peak flows through local detention measures and minimize impervious areas (Fletcher et al., 2014).

Design and implementation of WSUD technologies such as green roofs, rainwater harvesting system, bioswale, raingarden, pervious pavement, cleansing biotopes, and constructed wetland, for the urban design to control flooding and stormwater quality was already investigated by many studies (Lloyd et al., 2002; Dietz, 2005; Dietz and Clausen, 2005; Wong, 2006; Khastagir and Jayasuriya, 2010; Guo, 2012; Wang, 2015), but there is limited information on the relationship between investment cost and options to optimize design and performance, particularly in tropical climates. Furthermore, there is a distinct gap in practice between the design professions (landscape architecture, urban design, urban planning) and the engineering community that implements the designs (Irvine et al., 2021).

The use of stormwater modelling as a decision-making tool for sustainable urban drainage development has been widely recommended due to its capability in reducing uncertainty and improving the effectiveness of the focused development techniques. Some of the major modelling programs for urban stormwater modelling such as SWMM, STORM, MOUSE, MIKE-urban, SOBEK, InfoWater, and DR3M-QUAL were developed around the 1970s. Among these, SWMM has gone through many updates and adjustments since it was publicly available (Obropta and Kardos, 2007). The explicit modelling of LID features was introduced into the 2010 version of SWMM 5 (Rossman, 2010), allowing a variety of GI to be simulated through LID components (Niazi et al., 2017). While SWMM is freely available from the U.S.EPA (https://www.epa.gov/water-research/storm-water-management-model-swmm), a number of SWMM-based packages with graphical user interfaces to facilitate data input, management, and visualizations also have been developed (e.g. PCSWMM, XPSWMM, Mike-Urban). PCSWMM which is known as a window's implementation of SWMM that is fully integrated within an open GIS platform, comprises tools for modelling, simulating and graphing to understand the water behaviors in each urban sub catchment and the infiltration associated with land use areas. This application is effective in evaluating the hydrological responses and the quantity and quality of water under the developing areas. Furthermore, PCSWMM allows for hydraulic modelling which is the ability to track runoff and external flow of water through the pipes or drainage system. Also, the function of pollution load estimation and GI as LID/WSUD control helps to assess the amount of pollutants in runoff or land use area and allows engineers to determine effective infrastructure for runoff and pollution management, respectively (https://www.pcswmm.com/).

The application of PCSWMM will allow this study to explore the performance of particular WSUD designs with regard to improving pluvial flood management inside the Nava Nakorn Industrial Estate. Additionally, it will enable this study to determine the best practice of WSUD connected as a whole system to reduce flooding and at the same time to help Nava Nakorn to improve its environment and community wellbeing.

1.2 Significance of Study

Rapid urbanization and population growth in Thailand require massive infrastructure projects, especially in Bangkok and the surrounding areas of the city which has led to transformation of the natural urban environment and waterways and creating stress even more on urban drainage systems. Bangkok also called "The Venice of the East" is substituting its old canals and green land areas with highways, roads, buildings and other construction to accommodate the increase of the population. The process is degrading the value of grey land areas and gives the city a significant concern regarding water management and environmental sustainability in the future. The city regularly floods, not just due to overpopulation, urbanization and climate change but also due to insufficient drainage capacity and unplanned development (Detchphol, 2016; Friend et al., 2016).

Climate change threatens decades of development and endangers sustainable growth (Zhenmin, 2019), especially by creating more frequent and intense rainfall events and flooding. Neslen (2018) noted that “Global flooding and extreme rainfall events have increased by more than 50% this decade and now are occurring at a rate 4 times higher than in 1980”. Many cities around the world have already been affected by a changing climate and for Bangkok this issue is exacerbated by sea level rise (estimated as approximately 4 millimeters a year) and the subsidence of the Chao Phraya delta (reportedly sinking between 1 to 2 centimeters a year) (Deviller, 2018).

Flooding impacts society in various ways, ranging from the loss of life, physical injuries and mental health effects to the destruction of assets and resources. The 2011 major flood in Thailand showed how the flooding disaster affected one country as a whole. The damaged areas were scattered in 69 regions in each locale of the nation. More than 13 million people were impacted in some way, while 680 deaths were reported. The total damage and loss were USD 46.5 billion, with most damage and loss centered in the industrial estate and residential areas located in Bangkok and the adjacent provinces to the north and west of the city (Nipon and Pitsom, 2013).

Located in peri urban Pathum Thani, 46 km north of downtown Bangkok, Nava Nakorn Industrial Estate is one of the oldest industrial estates in Thailand, and is surrounded by communities, malls, marketplaces and universities such as Thammasat University (TU), Asian Institute of Technology (AIT), and Bangkok University. With an area of approximately 6,500 rais (1040 hectares), the estate is wholly bounded by a permanent flood prevention system and rainfall drainage that protects the estate from both fluvial flooding (large scale flooding) and pluvial/surface water flood (localized flood from a high intensity storm exceeding drainage capacity) (NNCL, 2006). However, fluvial flooding is still a major concern for the estate as it is located in the area that is relatively flat, lying on the flood plain of the Chao Phraya River, which is easily affected by flood. The historic

2011 flood caused the estate to incur around 86,500 million baht in damage, making it the most impacted industrial estate in Thailand for this event (Marsh and Mclennan, 2012). In response to this, Nava Nakorn strengthened its flood protection to address possible flooding in the future, as indicated in Nava Nakorn annual reports (http://nncl.listedcompany.com/ar.html). The existing flood prevention system is comprised of 5.5 meters (MSL) flood protection wall with a total length of 20.6 km, a man-made canal in the area of 300 rais, and 5 flood prevention-water pump stations with the total capacity of 1,320,000 m3/day. Aside from fluvial flooding, a 45 km rainfall drainage system was constructed along the internal roads to the estate and has a capacity to accommodate rainfall runoff up to 400,000 m^3/day. However, a majority of local residents reported experiencing pluvial flooding, recently, with a typical duration of 4-6 hours (see "TUNN Smart District report", 2020) even though the flood prevention system was stated to have sufficient capacity to control flooding. Hence, flood management plans should regularly be updated to prepare for a range of possible flood scenarios, including uncertainty related to climate change and greater frequency of more intense storm events as the estate continues to develop and mature.

Nava Nakorn divides its total area into 4 main zones which are industrial, commercial and residential, free zone, and infrastructure and green zone. As of 2019, remaining unsold land totals 208 rais. Meanwhile, the remaining 35% of infrastructure and green zone area was sold out for the development projects (NNCL, 2019). Limited green space and recreational spots should be addressed as these are also the public needs (see "TUNN Smart District report", 2020).

Apart from pervious area transformation issues, Nava Nakorn also must address road maintenance issues, as a number of roads have been damaged during the past flooding events. In 2019, the ongoing repairing of the roads used asphalt and reinforced concrete, while several roads are still under the restoration plan (NNCL, 2019). Site observation also indicated that most streets do not have a proper walkway, and combined with very busy traffic activity, and poorly regulated vehicle parking patterns, results in unsafe pedestrian and traffic conditions during the start and end of daily working hours. Transit-oriented development and new urbanism concepts are needed to improve better and safer transportation within the area while at the same time, WSUD features incorporated into the design can function to infiltrate stormwater run-off.

It is important to note that while there are numerous publications focused on demonstration of WSUD implementation, the cost and benefits of it are still debated in the literature. Performances of WSUD technologies in various settings of urban context (climatic and soil condition) are not always well documented resulting in a lack of comprehensive source information that can be used to make comparisons between each urban practice. This is also to be noted for cost data which generally are mixed between fixed cost and variable cost for individual scale of WSUD technology. It was suggested that defensible cost and proof-of-concept studies that document the benefit of WSUD contributing to environmental sustainability and ecological quality improvement are needed to reduce uncertainty in WSUD implementation (Roy et al., 2008). This study therefore aims to explore the use of WSUD technologies and emphasize the effectiveness of different features of this technology to control pluvial flooding in Nava Nakorn. Four features of WSUD such as bioretention cell, raingarden, bioswale and detention pond are selected for this study as they should result in flood reduction and may also provide an enormous environmental benefit to the study area. These environmental benefits include reduction of urban heat island, reduction in noise, improving water and air quality, landscape, and providing greenspace.

1.3 Research Questions

1) What are the optimum WSUD design characteristics to maximize flood mitigation in Nava Nakorn?

2) How can a dynamic model be used to guide development of optimum WSUD design?

3) What are the differences in looking at design from individual feature scale compared to a system wide scale?

1.4 Aim and Objectives

This research ultimately aims to assess the performance of different WSUD features for pluvial flood management in the Nava Nakorn Industrial Real estate:

1) To apply the PCSWMM model to assess the performance of individual WSUD design features with respect to reducing runoff, flood frequency, duration, and severity

2) To assess the performance of individual WSUD features to optimize design as well as the holistic performance of an integrated WSUD system within Nava Nakorn

3) Illustrate the value of dynamic modelling as decision support tool for WSUD design

4) To develop guidelines for best practices to ensure optimum design and performance of WSUD features.

1.5 Scopes of Study

1) Selected WSUD features for the study includes bioretention cell, rain garden, bioswale and detention pond

2) The designs of WSUD are based on year 3 Landscape Architecture (LA) and Urban Design and Development (UDDI) studio classes for the Thammasat-Nava Nakorn smart city project (see "TUNN Smart District report", 2020). In this sense the mathematical modelling of WSUD performance is explicitly integrated with landscape architectural visioning to optimize the design presentation and evaluation, thereby illustrating the multidisciplinary bridges to the combined design and performance-based design-making

3) The study focuses only on pluvial flooding. Flooding simulation is based on the 24 hours, 2-year design storm with the total rainfall of 72mm.

1.6 Expected Output

It is intended that this study will provide design criteria and guidance that can be used as a decision-making tool for Nava Nakorn to better manage pluvial flooding inside the estate by optimizing WSUD design in addition to its current flooding management system. Following the principle of dynamic modelling, this study will enable decision makers and designers to evaluate the system performance and use the result to improve the design and benefit of WSUD features to achieve the optimum result

CHAPTER 2
REVIEW OF LITERATURE

This literature review focuses on six main sections to understand the need, impact, and process of urban stormwater management as well as the importance of modelling approach in the role of stormwater management. The first two sections describe the hydrological impacts of urbanization and traditional versus sustainable stormwater management. The frequent use of urban land areas for multiple purposes such as infrastructure developments affect the urban waterways and its quality. A major focus of traditional stormwater management is hard engineering drainage systems which are costly and considered obsolete to the new change in environment. Therefore, a new integrated and sustainable stormwater management approach is needed. The last four sections of the literature review include (1) urban hydrologic modelling and model selection, (2) urban watershed delineation, (3) model calibration and validation and (4) water sensitive urban design. The role of modelling in urban stormwater management has been recognized by scientists and some practitioners as the catalyst in the successful implementation of water management projects. However, modelling techniques require technical expertise and are not always easily understood by the other related stakeholders such as urban planners, landscape architects, and the public. Further research and development should be directed into friendly decision support tools that are convenient and readable by all these people, so that they could have a mutual understating, which is the key in bridging the gap between the stakeholders.

2.1 Hydrologic Impacts of Urbanization

Urbanization entails a variety of environmental concerns ranging from the local, regional, and larger scales as a direct result of biochemical and physical alteration to the hydrologic cycle including the surface and groundwater systems. The process of urbanization contributes to the change in topography, natural vegetation, stream flows and flooding characteristics (frequency of recurrence, volume and duration), temperature above and below the land surface, and surface and groundwater quality. Specifically, the rise of cities increases paved surface and roofs which in turn affect the natural storage and infiltration and has put cities at risk of flooding and water scarcity due to water quality deterioration (Matt et al., 2001; O'Driscoll et al., 2010; Mcgrane, 2016; Rezaei et al., 2019).

Many studies have been done to explain hydrologic response to urbanization including s impacts on peak flow characteristics (peak discharge and timing), runoff water quantity and quality (Matt et al., 2001; Irvine et al., 2005; Visoth et al., 2010; Miller et al., 2014; Irvine et al., 2015; Gaut et al., 2019; Bulti & Abebe, 2020). Human activities involving change in land use pattern and hydraulic disturbance were indicated as the major factors affecting hydrologic processes. Surface runoff volumes and peaks are highly influenced by stormwater system connectivity. Miller et al. (2014) found that modifying catchment surface with more impervious cover could have greater impact on peak flow and duration of flood. Bulti and Abebe (2020) also indicated that increase in urban build up would result in temporal change of stormwater runoff. In this case, the study revealed that runoff volume and its depth are linearly associated with the rise of imperviousness ratio. The runoff response to increased urbanization traditionally has been envisioned as a hydrograph shown in **Figure 2.1**. Generally, the identified impacts include a higher peak, earlier peak, greater volume and shorter (i.e., flashier) duration (Thompson, 2009) associated with the increased per cent imperviousness and enhanced (i.e., piped) drainage.

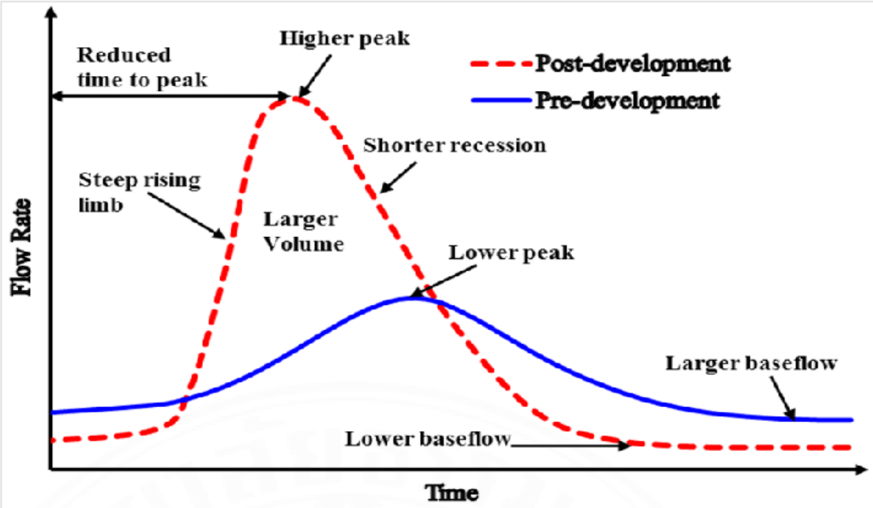

Figure 2.1 Schematic diagram showing the impact of urbanization on runoff (Thompson, 2009)

Apart from the quantity concerns, increasing in urban runoff could lead to fast transport of pollutants and nutrients which resulted in water quality degradation at the downstream water bodies of urban areas (Carle et al., 2005; Cerqueira et al., 2019; Irvine et al., 2005; 2015; Ren et al., 2014). Since hydrologic impacts are different between the catchments as well as varied between storm events, some studies have suggested that more detailed urban land use typologies that could represent hydrologic pathway both above and below ground are important to consider in developing sustainable mitigation measures such as WSUDs for an area. Prioritizing areas with greater per cent imperviousness, for example, may require appropriate land use planning that would be helpful to minimize possible flood risk and improve efficacy of stormwater management both quality and quantity control within the area (Matt et al., 2001; Miller et al., 2014; Irvine et al., 2015).

2.2 Traditional Stormwater Management Vs. Sustainable Stormwater Management

Over the last few decades, there has been an increasing recognition of the importance of stormwater as a resource that should be included in urban planning. This increased awareness has been driven by a number of factors, including an increasingly urbanized population, and increased water consumption, increased environmental consciousness, the risk of storm damage aggravated by climate change, and growth in urban areas and related impermeable surfaces. To manage the storm resource more effectively, many countries have merged the traditional urban stormwater management with a sustainable approach. Thus, a range of terminologies has been developed including Low Impact Development (LID) in North America and New Zealand, Sustainable Urban drainage System (SuDS) in the UK, Water Sensitive Urban Design (WSUD) in Australia, Best Management Practices (BMPs) and Green Stormwater Infrastructure (GSI) in US, Alternative Techniques (ATs) in France, and Sponge City in China (Fletcher et al., 2014; Zevenbergen & Pathirana, 2018; Lashford et al., 2019; Radcliffe, 2019).

Prior to the adoption of these key terms of sustainable urban stormwater management, cities generally followed hard engineering approaches (also called conventional stormwater management) which entirely were built based on the historical climate conditions. However, the practices and built environment are no longer compatible with the rapid development of places (Pyke, 2011). The traditional, hard engineering approach and planning is not flexible to support stormwater management as excessive use of pipe drainage contradicts the goal of sustainable development (Hood et al., 2007) while neglecting the potentially valuable source of water resource in the area (Mekonnen & Hoekstra, 2011). Sustainable stormwater management approaches such as LID/WSUD could reduce runoff volume and improve runoff water quality through incorporating nature and green infrastructure into the design (Sparkman et al., 2017). Diet and Clausen (2008) found that in urban watersheds where runoff is entirely managed by traditional approaches, increases in impervious surface percentage would lead to an increase in stormwater runoff volume (**Figure 2.2**) and the runoff coefficient (**Figure 2.3**). In contrast, urban watersheds where LID existed, increasing impervious surface showed unchanged level of runoff volume (**Figure 2.2**) while the runoff coefficient tends to be lower by greater impervious surface added (**Figure 2.3**). Similar trends applied for runoff water quality related to nutrient exports of NO3–N, NH3–N, TN, and TP (see **Figure 2.4**). The increase in flow in the traditional subdivision was identified as the key driver of increased pollutant exports. Pollutant export from the traditional subdivision was generally comparable to that from urbanized watersheds. However, pollutant exports from the LID subdivision were more comparable to that from forested watersheds. Figure 2.4 indicated that NH3–N significantly decreased with increasing impervious area, however no trend was found for TP and no relationships were found for NO3-N and TN under the LID scenarios. In Singapore, the PUB showed that the LID, rain garden, could reduce TN and TP levels in runoff going to a surface drain (Irvine et al., 2014), although Wang et al. (2017) found the same rain garden was a net exporter of nitrate. This is one of the reasons rain gardens in Singapore are now designed with a wood chip layer, to help produce anoxic conditions to aid in the denitrification process. These findings suggest that a sustainable approach has considerable advantages over a traditional approach, and that it may be able to assist cities in reducing the effects of development on downstream water bodies, if designed appropriately.

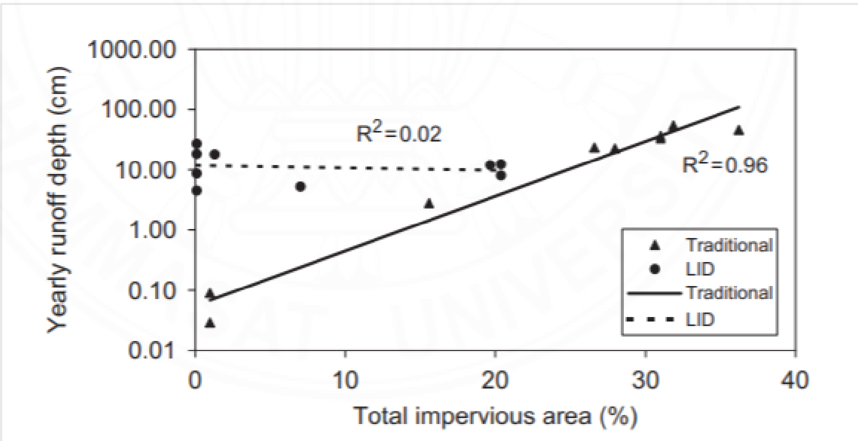

Figure 2.2 Annual runoff depth vs. total impervious area, traditional and LID subdivisions 1996–2004, (Dietz and Clausen, 2008)

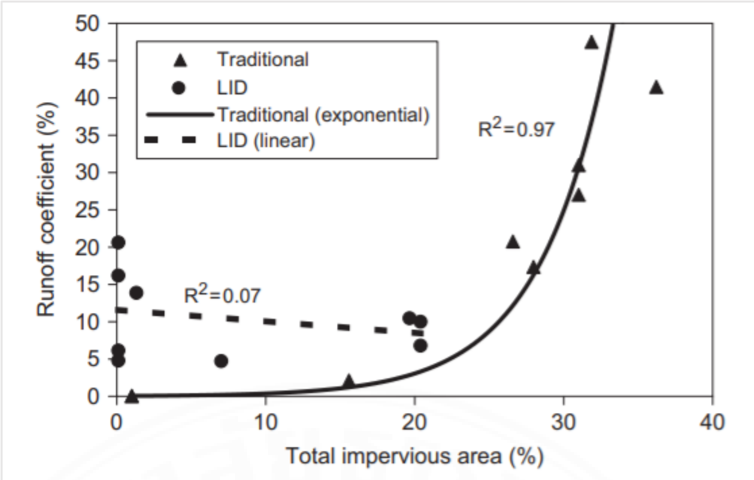

Figure 2.3 Total impervious area vs. runoff coefficient, traditional and LID subdivision, 1996-2004, (Dietz and Clausen, 2008)

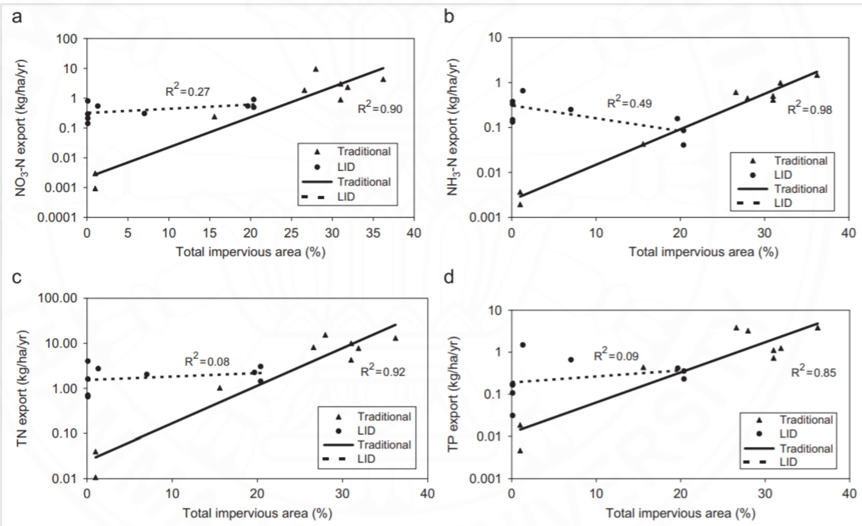

Figure 2.4 Traditional and LID subdivision nutrient export (1996–2004): (a) NO3–N, (b) NH3–N, (c) TN, and (d) TP, (Dietz and Clausen, 2008)

2.3 Urban Hydrologic Modelling and Model Selection

Hydrologic modelling has gained more and more interest from researchers as well as urban planners due to its potential benefits to maximize options for sustainable and cost-effective stormwater management. Initially, modelling techniques were developed just (1) to understand the effect of urbanization on natural water systems, (2) to improve reliability of data as the key measurement for the challenging, strongly heterogeneous urban environment, and (3) to predict the future of urban risk uncertainties as such flood, land use pattern and climate change scenario. The basic need of urban population to access clean and sufficient water supply, however, has added to the earlier purposes of modelling techniques to also cover the problem of safety and pollution risk assessment (Salvadore et al., 2015). For highly urbanized catchments where water resource depletion and flood occurrence are the major problems reliable assessment of water movement to protect human life, environment, and infrastructure is needed, and includes knowledge and a clear understanding of water behavior as related to landscape structure and watershed characteristics (i.e., soil type, land cover, and land use) (Korgaonkar et al., 2018).

Hydrologic modelling in the urban context needs elaboration between several groups such as urban planners, engineers, environmental scientists, public health researchers and practitioners, and sociologists to develop and improve the accuracy of the models (Obrota & Kardos, 2007; Fletcher et al., 2013; Irvine et al., 2014). Eventually, the ability to deliver accurate model results is still limited for many urban catchments. Salvadore et al. (2015) outlined this issue related to (1) spatio-temporal gaps between physical scales and model resolutions (2) limited data (3) limited number of physical processes described (4) inconsistent level of details and (5) high complexity which easily led to high uncertainty. The improvement of computer development capability and remote sensing data accessibility makes the hydrologic model become a more common practice with uncertainty levels being reduced. The integration of GIS and remote sensing with hydrologic modelling help improve the modelling processes, spatial viability of watershed, as well as effectiveness of studying the hydrologic responses, but most importantly is cost-efficient (Sina & Bahram, 2014; Kainat et al., 2020).

Models are simplified representations of the real-world system. In the hydrologic system, models are intended for the understanding and characterization of the hydrologic features and their processes. By considering the mathematical structures, hydrologic models can be either deterministic or stochastic. The deterministic category describes the catchment process using mathematical rules in which output is the part of a determinate system. Deterministic modelling does not consider randomness, therefore will always produce the same output for the simulation based on the same input. On the other hand, the stochastic category can have different results because this type of model is probability-based, considering random occurrences that evolve in time or space. However, its applications are specific and mode-based which means it cannot be extended to analyze the alternate scenarios of stormwater or runoff. The alternative approach for stormwater modelling is the hybrid between the two models: deterministic and stochastic. The combination of the two models enhances the process of stormwater modelling and reduces any uncertainty and error during the modelling process. Not only does the new hybrid model include all the two model's characteristics, but it will also add more complex features to support the accuracy of the modelling (Jajarmizad et al., 2012; Farmer and Vogel, 2016).

Many modelling software packages have been developed to facilitate the modelling processes. Some of the common modelling programs for the urban hydrologic system include the Rational Method, HEC-HMS, TR20, WIN TR-55, HEC-RAS, WSPRO, HydroCAD, SWMM based programs (SWMM5, PCSWMM, InfoSWMM, MikeUrban, SUSTAIN, XPSWMM) where details with respect to types, functions, and purposes are provided in Minnesota Stormwater Manual (2020). Among these modelling programs, SWMM has gone through many updates since it is available for public use. The modelling of EPA SWMM is applicable for both combined hydraulic and hydrologic and water quality models as well as the capability to integrate with LID application and traditional BMPs (i.e., rain barrels, permeable pavers, vegetative swales, bioretention cells, infiltration trenches, wetlands, ponds). The detail level of SWMM's conceptual model and its overall computational parsimony is well balanced, making it a sufficient model either for large or medium scale hydrologic application. Though, the new mechanistic algorithm method and user guidance to couple with the other model were suggested to improve the realistic simulation on some of the applications like diffuse pollutant sources and their fate and transport as well as the effectiveness of LID implementation scenarios (Niazi et al., 2017). The use of EPA SWMM was presented in many studies (Irvine et al., 2005; Chaosakul et al, 2013; Irvine et al., 2015; Wang et al., 2017) with the role of model in the decision support system for water resource management and planning also being addressed (Silva et al., 2008; Serrat et al., 2011; Niazi et al., 2017; Irvine et al., 2021).

The roles of the model, in general, are considered important in the decision support system, however, this belief is only within the academician and some practitioner groups. In reality, not everyone, and particularly urban planners and landscape architects, and the public are fully convinced of the value of modelling. And as stated earlier, the role and function of hydrologic modelling could serve a great benefit for the urban development, but it needs a mutual understanding and collaboration between these related stakeholders and the public. This barrier, however, can be overcome by integrating the design and modelling thereby creating a friendly-user decision support system (Serrat et al., 2011; Irvine et al., 2021).

2.4 Urban Watershed Delineation

Watershed is defined as an area of land that functions as a catchment for water. This also can be defined by process in which surface water flows from the catchment into outlets (or from high elevation to low elevation land area) either in the form of river, lake, pond, stream, or infiltrates into the groundwater. In urban areas, the natural ground surfaces are replaced by impervious surfaces such as buildings, concrete roads, parking lots and roofs. These impervious surfaces can generate overland flow that could cause many problems including pollutant transport into water bodies and erosion that caused by large volume of water flow (Dixon and Uddameri, 2016). This complexity of surfaces, as well as the piped drainage, in the urban environment can create particular challenges in delineating the watershed boundaries as compared to natural watersheds (Parece and Campbell, 2015).

Delineated watersheds are required for urban hydrologic modelling, mainly, because the result can be used to characterize and investigate the hydrological process of an area of study versus another. To date, delineating the watershed can be done through DEM (Digital Elevation Model) and manual based. The accuracy level of a DEM in representing topographic features of the land area affects the simulation modelling performance. Inaccurate DEM representation can lead to distorted drainage pattern in the model as the boundaries are generated automatically using a grid cell-based approach. The manual method, in contrast, is done based on the existing watershed boundaries and stream layers. This method is very time-consuming and demands a quality resolution topographic map as the marked drainage is divided on the scale of 1: 2,6000 topographic quadrangles. With the advancement of reliable DEM and GIS (Geographic Information System), the automated DEM based has become very popular among scientists and practitioners as it can be applied to various and larger scales while less time being consumed (Islam, 2004; Gusta et al., 2011; Parece and Campbell, 2015).

In PCSWMM, the automated delineation function is significantly fast and versatile. The integration with GIS makes this computer modelling program able to simulate stormwater sources and assess hydrological responses to changes in development areas. The existing watershed delineation tool in PCSWMM delineates the watershed into a network of sub-watersheds or subcatchments, junctions, and conduits, partially parameterized from DEM features. Some of the major parameterizes that are elevation-based attributes such as slope and overland flow length also can be generated using a DEM layer and a user-defined target discretization value (James et al., 2017).

2.5 Model Calibration and Validation

Calibration and validation are crucial steps in proving that a hydrologic and water quality model can deliver appropriate findings in a particular application. As for rainfall-runoff models, these processes have become more challenging to modelers due to the increase in size and complexity of the modelling applications. Theoretically, Model calibration is the act of modifying model input parameters to make modeled hydrographs match observed hydrographs using model calibration criteria. Meanwhile, the validation process (or else known as verification) involves using recent rainfall and flow data to examine the accuracy and reliability of an existing or previously calibrated model. Result of the validation is then used to verify whether the existing model can anticipate current conditions, or it needs to be updated and re-calibrated to match the current system (USEPA, 1999).

The two common practiced calibration techniques including event-based calibration and continuous calibration were discussed in comparison by Shamsi and Koran (2017). The event-based approach considering two steps of calibration and validation was adopted to achieve specific model accuracy criteria. Roughness, imperviousness, and soil permeability, for example, are adjusted until the difference between the predicted and real event values reaches acceptable accuracy criteria. The validation process consists of running the calibrated model with one or more observed events to ensure that the calibrated model is accurate. The accuracy is generally determined by disparity between observed and modeled flow depth, volume, peak flow rate, time-to-peak, and hydrograph shape. In most cases, these five hydrograph events are assessed in comparison to demonstrate the adequacy of model calibration. Different design applications would result in different prioritization of hydrograph events that used for calibration. As such, peak flow is more important in sewer design and capacity analysis application whereas flow volume is more relevant in combined sewer overflow (CSO) and storage sizing application. It is recommended that the 10% accuracy is acceptable for calibration while 25% is acceptable during validation (Shamsi, 2016).

Rather than involving both calibrating and validating processes, the continuous calibration approach uses statistical criteria, covers the entire observation period therefore the validation process using the independent events input to the model is unnecessary. Continuous calibration can be done using software like PCSWMM, which computes calibration error on the fly and saves time by eliminating the need for tedious spreadsheet pre-and post-processing of observed rainfall and flow data outside of the model. The statistic error functions have been developed to measure the model goodness-of-fit between the long term continuous measured and a modeled hydrograph, which result somehow can be used to evaluate the adequacy of calibration. Various objective functions and statistical measures are available in CHI's PCSWMM software and those includes Integral Square Error (ISE), Nash–Sutcliffe Efficiency (NSE), Coefficient of Determination (COD or R2), Standard Error of Estimate (SEE), Least Squares Error (LSE), Least Squares Error Dimensionless (LSED), Root Mean Square Error (RMSE), and Root Mean Square Error Dimensionless (RMSED) (Shamsi et al., 2016, Shamsi and Koran, 2017).

Oftentimes, the goodness-of-fit measured functions rating the performance of the model based on indicator values are solely used for the hydrologic modelling (i.e., NSE value range between 0.5 – 1 indicating excellence model simulation result). However, some studies suggested using benchmarks to evaluate the model simulation instead as it would help for model selection and development in the future (Schaefli and Gupta, 2007; Pappenberger et al., 2015; Seibert et al., 2018; Lane et al., 2019). With benchmarking, the indicator value represents what could be achieved in a catchment based on the data available. This allows modelers to make a more objective assessment of how well their model is doing. Climate data, mean observed discharge, and the performance of a simple, lumped hydrological model for the same conditions are all examples of benchmarks against which models can be assessed. The benchmark value however should be varied between the catchments (some are lower, and some are upper) as different catchments would generally contain different characteristic (i.e., size, width, %imperviousness).

2.6 WSUD (Water Sensitive Urban Design)

The concept of water sensitive urban design seems to have been officially started in Australia since the 1990s. Evolving from its early foundation of low impact development, WSUD provides a wider framework for urban water resources management. Integrating the matter of urban stormwater stream, potable water, and wastewater in the WSUD concept could create system efficiency and resilience while protecting the environment (Donofrio et al., 2009). Often times, WSUD is used in parallel with the concept of water sensitive cities which aims to close the loop, bringing back the nature-oriented water cycle in the city. The process explicitly works across all scales and encourages a collaboration between engineers, architects, planners, social scientists, and ecologists. Even though the key element of WSUD is for stormwater management, it involves consideration on all parts of water cycle process. For that reason, it could help the city in improving its resilience both in the event of worsening climate change and rising urbanization (Fletcher et al., 2014). The main objectives of WSUD in detail according to Victorian Stormwater Committee 1999 (The Urban Stormwater Best Practice Environmental Management Guidelines) are:

1) To protect and enhance the natural water system in urban development
2) To integrate the stormwater treatment into landscape and maximize the visual and recreational amenity of development
3) To protect water quality draining from urban areas
4) To reduce runoff and peak flows through local detention measures and minimizing impervious areas and add value while minimizing the drainage infrastructure development cost.

There are many technologies (approximately 16 up to now) adopted in the WSUD for urban stormwater management. Hoyer et al. (2011) categorized these technologies based on the primary function of the WSUD. Those include (1) rainwater use (2) treatment (3) detention and infiltration (4) conveyance and (5) evapotranspiration. However, some of the commonly used technologies in WSUD are infiltration trench, permeable pavement, bioretention cell, vegetative swale/bioswale and rainwater harvesting system (Ahammed, 2017). The importance and the use of these technologies are discussed as follows:

Infiltration trench is a shallow excavation filled with porous materials, which captures stormwater runoff. The design allows runoff water to infiltrate into the underlying soil while particulate and some dissolved pollutants are retained in the porous media (see **Figure 2.5**). As recommended, the trench approach should be implemented for the small drainage areas where it is feasible (i.e., residential lots, commercial areas, parking lots, and roadway) (https://www.melbournewater.com). Clogging is the main problem of a trench due to the settling of fine sand particles in the interstices of the soil. Additionally, there can be issues related to the emptying time for the trench. Therefore, design should also be incorporated with porous pavement, grass swale and even with the detention pond in order to prevent the clogging and create sufficient emptying time in the case of excessive storm event (Chahar et al., 2012). Improper design would result in reduction of both runoff quantity and quality control. As demonstrated by Emerson et al. (2010) the infiltration trench that piloted in his study resulted in only runoff water quality improvement with a 36% TSS removal rate. However, no runoff reduction data was reported as flooding overwhelmed the trench capacity. This issue was linked to some of the major factors such as undersized design, oversized contributing area and no pre-treatment was incorporated into the design therefore causing the issue of clogging.

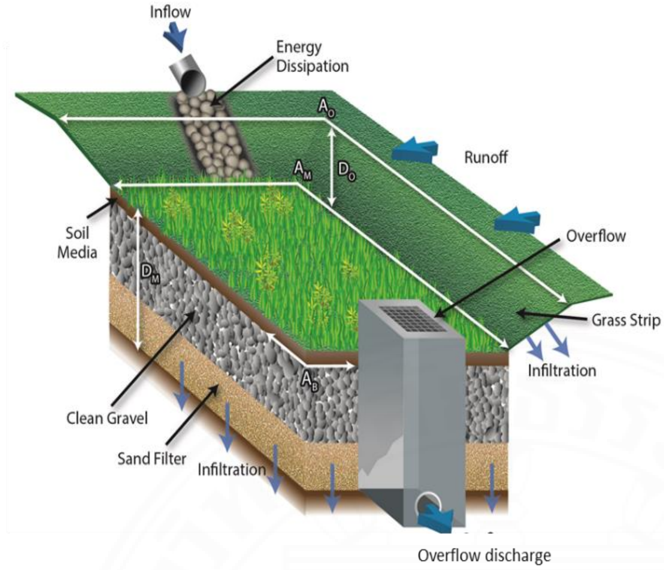

Figure 2.5 Vertical view of an infiltration trench representation (left) and example of this technology in the practice. Retrieved from (https://sswm.info/ar/water-nutrient-cycle/)

Permeable Pavement is a modified form of asphalt or concrete whose top layer is pervious to water due to voids intentionally created during mixing or installation. Permeable pavements include porous concrete, asphalt, grid pavers, and interlocking pavers with joints (**Figure 2.6**). Runoff water infiltrates at a rate of 12.7 to 76.2 mm/hour with this typical design, thus it normally is very efficient for runoff control. Design ratio should be as much as 2:1 between contributing catchment and total permeable pavement area. With this, it also can be used to capture the runoff from the surrounding areas of the site until the amount of runoff reaches the underground reservoir capacity. Like infiltration trench, permeable pavement is also used in the runoff water quality application and is applicable even in the low-traffic setting (residential area and parking lots). Technically, water flows through the pavement and is then treated by the sub-base gravel and soil under the pavement before percolating into the ground (https://sswm.info/ar/water-nutrient-cycle/). Proper design can be effective for the removal of sediments, nutrients, and some metal substances. However, this type of design requires periodic vacuuming and maintenance to prevent the failure of the system due to sediment clog (Scholz and Grabowiecki, 2007; Freeborn et al., 2012).

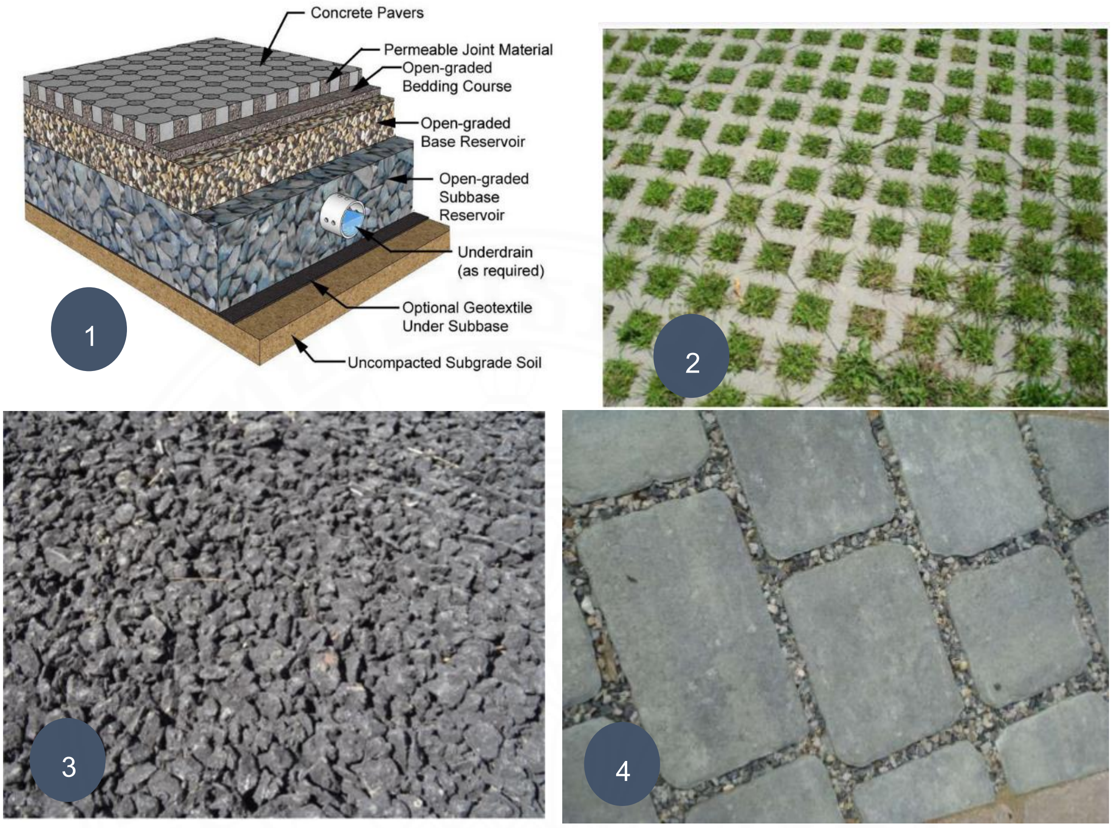

Figure 2.6 Schematic representation of permeable pavement (picture 1) and some examples of the permeable pavement surface (picture 2, 3, 4). Retrieved from (Freeborn et al., 2015)

Bioretention cell (also referred to raingarden) is a shallow depression structured with sandy soil, thick layer of mulch, and planted with dense vegetation on top. It usually is installed in the lawns, along the edge of the road, and in the median of parking lots to manage and treat storm runoff from those areas (see **Figure 2.7**). Storm runoff drains into the cell by pipes, swales or open curbs. The depression captures and stores the first flush temporarily before it infiltrates into the cell layers below (https://megamanual.geosyntec.com/npsmanual/).

Bioretention cell is one of the most practical WSUDs in urban areas due to its design flexibility that could fit with different climates and variety of soil type from clays to sands. The size and design of a bioretention cell depends on the total drainage area contributing to the site and soil characteristics of that area. When properly designed, bioretention cell could provide both runoff quantity reduction and water quality improvement. Reduction in runoff could reach between 85 – 90% with impact on runoff water quality, between 95 – 98 % of metals (Cd, Zn, Pb) removal, 40 % of total nitrogen and 15 to 75% of nitrate-nitrogen reduction. It has also been shown to reduce phosphorus by as much as 65% (Jarrett, 2016).

In tropical climates with frequent and high rainfall intensity and average temperature greater than the temperate regions, design and construction of bioretention cells requires some modification over the conventional design. For example, the combination of the cell with pervious concrete with an internal water storage (IWS) layer could improve the hydrologic performance. Brown and Hunt. (2011) found that this integration could result in total runoff volume reduction of 87 % (1.03 m IWS depth) for sandy soil and 75% (0.73 m IWS depth) for the sandy clay loam in the underlying soil. The other example is the Balam Estate raingarden, located in Singapore for which the design included a 0.3 m thick layer of woodchip and sand. This inclusion was to provide the anoxic condition which appeared to improve the nitrogen removal. The same raingarden reported in an average reduction for total nitrogen of 46%, total phosphorus of 21% and total suspended solid of 57% (Ong et al.,2012).

Figure 2.7 Schematic representtaion of bioretention cell design (above) and examples of this application (below) for the residential area (left) and padestrian roadway (right). Retrieved from (https://megamanual.geosyntec.com/npsmanual/)

Vegetated Swales (also known as bioswales) are wet or dry swales made of grass, rocks, and other types of vegetation (see Figure 2.8). Like the other types of WSUD, bioswale is applied in the urban area for storm runoff control purpose. As outlined in Xiao and McPherson (2011), bioswale installed in the parking lots of University of California Davis (UCD) campus reduced runoff by 89% and total pollutant loading by 95.4% under the total rainfall of 564 mm during February 2007 and October 2008. Bioswale with perforated underdrain allows this typical design to manage runoff from a larger event. For instance, a bioswale with a rip-rap lined forebay located along state highway NC 211 in

Bolivia, North Carolina, USA, exfiltrated runoff up to 100% for a storm event of 86 mm. The larger event of 146 mm rainfall resulted in 85 % of the volume being captured. The high treatment performance was also likely due to the high infiltration rate of the media and underlying soil, longer forebay underlain with media, and shallow slope (Purvis et al., 2019).

Figure 2.8 Cross-sectional view of a bioswale design and example of curbside bioswale in practice. Retrieved from (https://www.reliance-foundry.com/blog/bioswale-design)

Rainwater harvesting system (also known as a cistern tank) refers to any devices that captures, diverts, stores, and releases collected storm runoff from the roof as an alternative water use (see **Figure 2.9**). Harvesting roof runoff can be done either in small containers or large containers according to the need. As illustrated in **Figure 2.9**, the storage container can be put above or under the ground and, in general, is equipped with a pump to facilitate domestic uses (Freeborn et al., 2012). Rainwater harvesting systems have gained more and more recognition due to their ability to alleviate water pressure on centralized systems, reduce or delay storm runoff, and easily fit with either centralized or decentralized infrastructure. To optimize the benefit and performance of the rainwater harvesting system, it is crucial to consider appropriate sizing for the system while balancing it with the life-cycle cost implementation (Semaan et al., 2020).

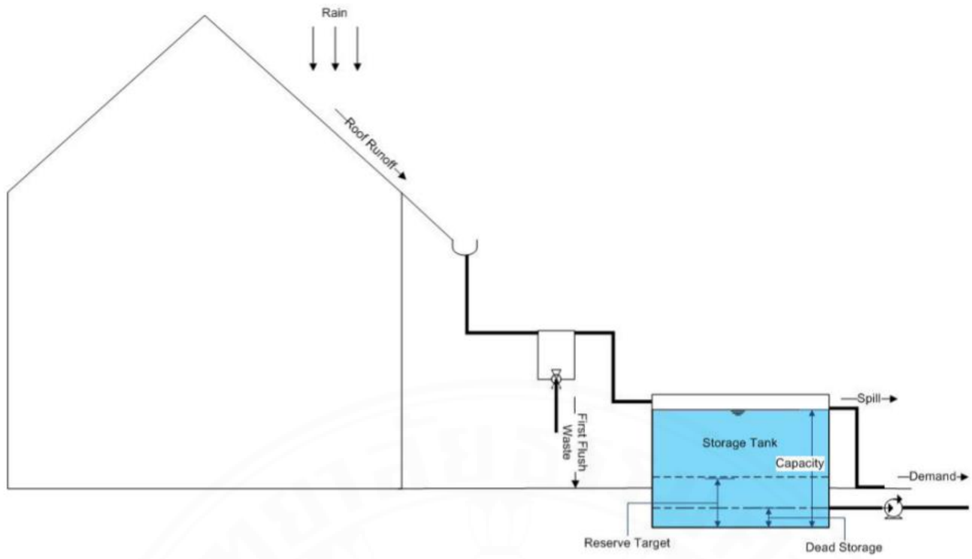

Figure 2.9 Schematic representation of rainwater harvesting system (above) and example of this technology in the residential practice (below). Retrieved from (Freeborn et al., 2012)

CHAPTER 3
METHODOLOGY

3.1 Introduction

This This study uses PCSWMM to simulate pluvial flooding inside the Nava Nakorn Industrial Estate and assess the possibility of adopting potential WSUDs to maximize flood reduction capacity in the study area. While it is recognized that WSUD technologies have a potential benefit of improving water quality, the focus of this study is on management of runoff quantity. This chapter explains the study design including study area, data collection, and method for analyzing the data. The major process consists of 5 parts in addressing the research objects, as summarized in **Figure 3.1**. Information needed to operationalize PCSWMM included rainfall data, site characteristics (e.g., location boundary, road system, land use, soil type, and land slope), and characteristics of the storm sewer system (e.g., location and geometry). This information was input to the model to represent current, or baseline conditions, and subsequently a number of WSUD scenarios were developed and run in comparison, with the objective of identifying the likely possible range of hydrologic benefits (minimum to maximum) associated with different WSUD designs. Performance of the connectivity between the designs was then evaluated to figure out the possible flood volume and severity reduction benefits in comparison between the conventional design (represented as scenario 1) and the optimum one (represented as scenario 3) (see **Table 1**). The overall costs of the individual designs were then estimated afterward.

Table 3.1 WSUD scenarios in this study

WSUD Feature	Scenario Development	WSUD Connectivity		
		Scenario 1	Scenario 2	Scenario 3
Bioretention cell	1, 2, 3	1	2	3
Bioswale	1, 2, 3, 4	1	3	4
Raingarden	1, 2, 3	1	2	3
Detention pond	1, 2	1	1	2

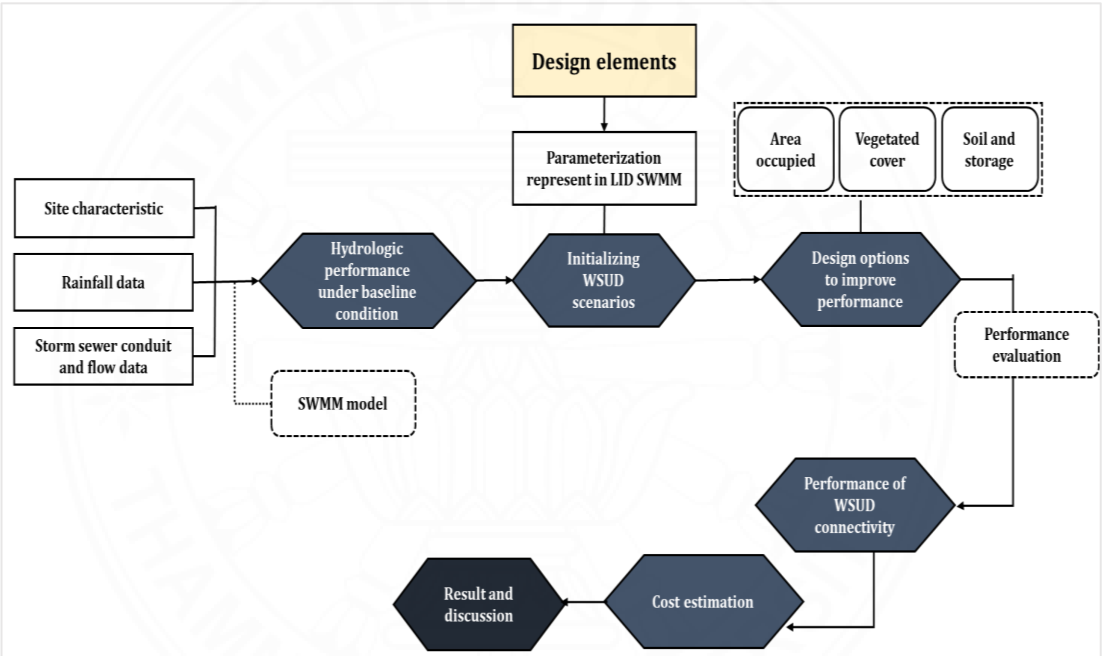

Figure 3.1 Research framework of this study

3.2 Description of Study Area

Nava Nakorn Public Company Limited was created on 26 March 1971 with the vision of undertaking industrial estate development and becoming an industrial city with new innovation in Thailand. Nava Nakorn Public Co., Ltd. currently operates industrial estates at two locations. The oldest industrial estate is in peri urban Pathum Thani, 46 km north of downtown Bangkok, Klong Luang District while the other industrial estate is located in Sung Noen District, Nakhorn Ratchasima Province which is about 230 kilometers north east of Bangkok.

Nava Nakorn Pathum Thani is considered one of the oldest industrial estates within the country and is home to approximately 200 domestic and international companies within a current area of about 6,500 rai (1,040 ha). It is surrounded by communities, malls, marketplaces, and universities such as Thammasat University (TU), Asian Institute of Technology (AIT), and VRU. Industry within Nava Nakorn is diverse, but generally is considered light, high value industry. Nava Nakorn divides its total area into 4 main zones which are industrial, commercial and residential, free zone, and infrastructure and green zone with the estimated population of 150,000. As of 2019, unsold land at the Pathum Thani site is 208 rais (33.3 ha). The building and land use categories within Nava Nakorn as well as the viewscape of the estate are demonstrated in **Figure 3.2** and **Figure 3.3**, respectively.

Nava Nakorn Public Company gains its principal revenue sources by being as a “super utility” to its customers through providing public municipal services including water, wastewater treatment, electricity, telecommunication service, security, solid waste management and urban infrastructure. Nava Nakorn also acts as a leader in community development and wellbeing by organizing bi-monthly town hall meetings of community leaders representing the different residential districts and developing an app which allows community to address and report their concerns in regard to water leaks, road disrepair, lighting and electrical problems.

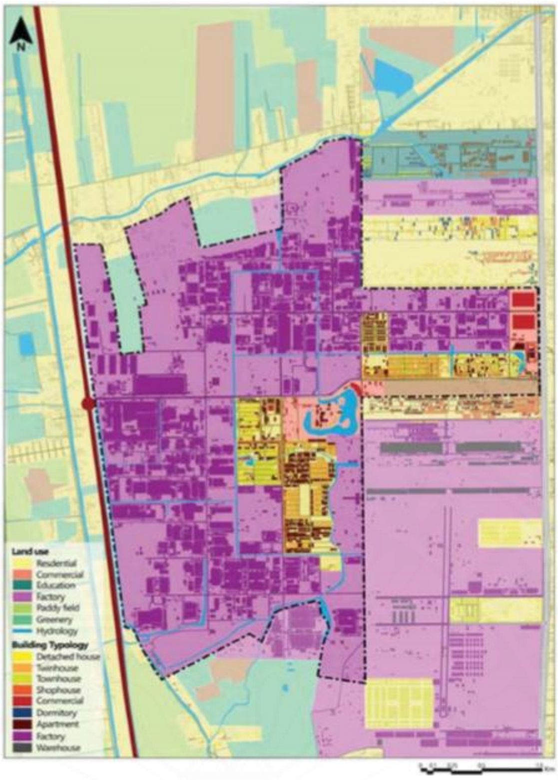

Figure 3.2 Land use and building categories in Nava Nakorn, “TUNN Smart District report”, 2020)

Figure 3.3 General area of Nava Nakorn, from https://www.navanakorn.co.th/en

3.3 Climate Data

In the Koppen Koppen climate classification system, the province of Pathum Thani is categorized as a Tropical Savanna (Aw) type climate with an average temperature of 27.8 °C and approximately 1,301 mm of rainfall per year. According to Thailand's Department of Meteorology, the annual mean temperature of Thailand rose by one-degree Celsius from 1981-2007. The closest meteorology station to Nava Nakorn, is located at Rangsit, has an average temperature of 27.7 °C. Precipitation is the lowest in January, with an average of 5 mm. April is the warmest month of the year with an average temperature of 30.3 °C while the coldest is December when the average temperature drops to 25.4 °C. Precipitation difference between the driest and the wettest months is 311 mm while the variation in annual temperature (as indicated in **Figure 3.4**) is around 4.9 °C. Rainfall intensity (**Table 3.1**) representing the study area was extracted from the design rainfall storm study for the northern part of Bangkok conducted by Weesakul et al. (2017). The design was based on 21 years of rainfall records between 1990 and 2011 that were obtained from the meteorological station at AIT, Pathum Thani.

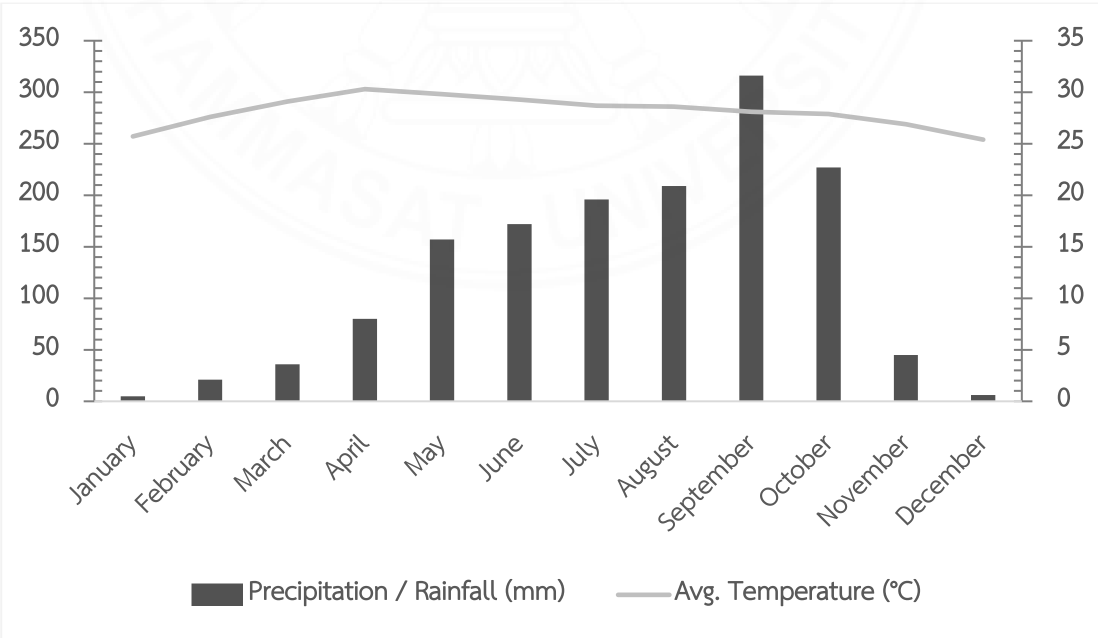

Figure 3.4 Pathum Thani climate data, https://en.climate-data.org/asia/thailand/

Table 3.2 Rainfall intensity for each design storm period from AIT rain gauge data

Return Period (years)	Intensity (mm/hr)								
	5min	10min	15min	30min	1hrs	2hrs	6hrs	12hrs	24hrs
2	130	117	105	81	53	30	11	6	3
5	148	133	123	100	67	41	15	8	5
10	160	143	135	113	75	47	18	10	5
25	175	156	150	129	87	56	21	12	7
50	186	166	161	141	95	63	24	13	7
100	198	175	172	152	103	69	26	15	8

Source: Weesakul et al. (2017)

3.4 Material and Method

3.4.1 Personal Computer Stormwater Management Model (PCSWMM)

PCSWMM is a simulation model used to investigate and evaluate the quantity and quality of surface and subsurface water in the same sub catchment areas. PCSWMM has the ability to model storm water sources and evaluate hydrological responses to changes in development areas using Geographic Information System (GIS) which consists of essential applications such as river modelling tools, real-time control analysis, time series management, Digital Elevation Model (DEM) support and native GIS support. These systems allow for automatic reporting, modelling, and visualization of storm water sources to manage the water supply.

PCSWMM also provides essential tools to record and plot data in graphs, run various water quality simulations which can be portrayed in any form, including graphs, color coding, tables, and statistics analyses. The major functional capabilities of PCSWMM include:

- **Hydraulic Modelling** provide user the ability to track runoff and external inflow that runs through the drainage system network of pipes, storage, or channels. This allows for the study of the water flow, mapping the locations and the level difference between groundwater and the drainage system

- **Hydrologic Processes** provides ability to investigate different types of water processes within the tested area such as time varying precipitation and evaporation, water infiltration to different ground layers and water runoff

- **Pollutant Load Estimation** giving the user the ability to estimate pollutant load associated with runoff from specific land uses, also including consideration of pollutant treatment through WSUD features or storage elements

- **Green Infrastructure as LID/WSUD Controls** enables engineers and planners to evaluate and determine infrastructure that are suited for effective runoff control and pollutant management.

An example of the PCSWMM user interface is shown in **Figure 3.5** indicating some of the major functions of this modelling tool, while **Figure 3.6** provides the schematic representation of PCSWMM including subcatchments to represent surface runoff fora drainage network system as a series of conduits and nodes to convey runoff water to a treatment wetland. Surface and groundwater hydrologic methods produce runoff based on rainfall input therefore allowing the user to estimate total runoff quantity for a certain catchment or sewershed area as well as the overall performance of the drainage system. Results then can be used, for example, to determine a specific design and evaluate its performance throughout WSUD control like bioswale or detention pond. The LID control editor available in PCSWMM allows the user to conduct the study and evaluate different features of WSUD technology with respect to water quality and quantity improvement. An example of an LID process in PCSWMM is provided in **Figure 3.7**.

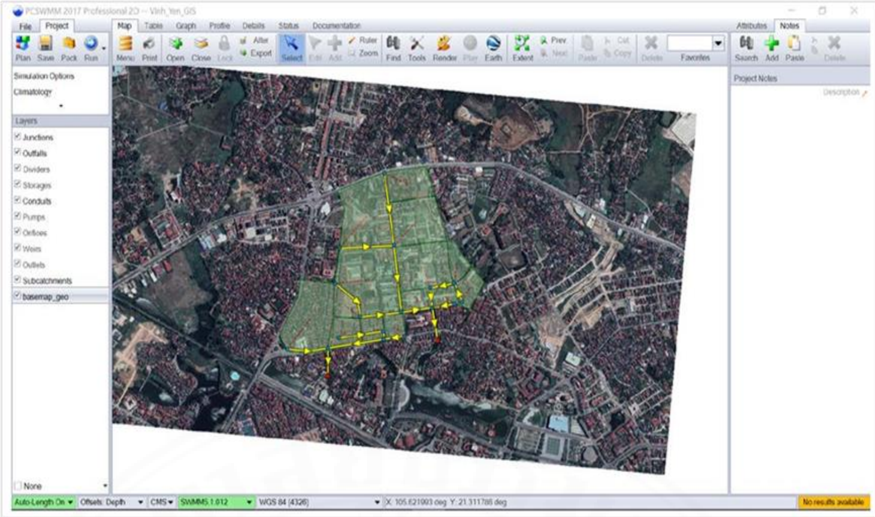

Figure 3.5 Example of PCSWMM interface (CHI, 2019)

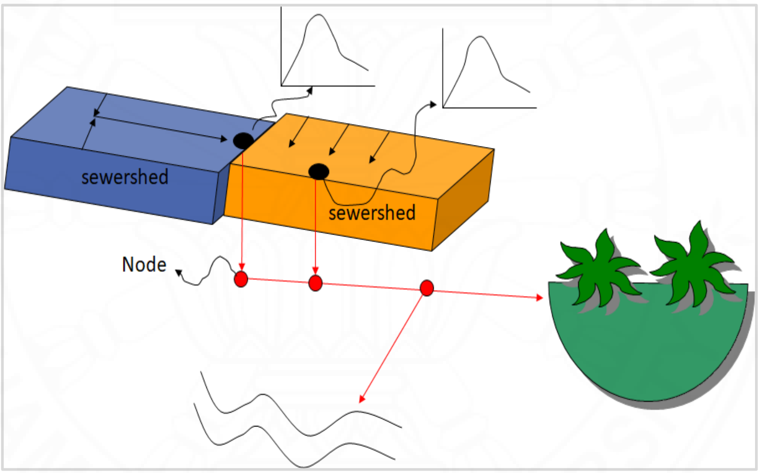

Figure 3.6 Schematic representation of PCSWMM 7 (Irvine, 2019)

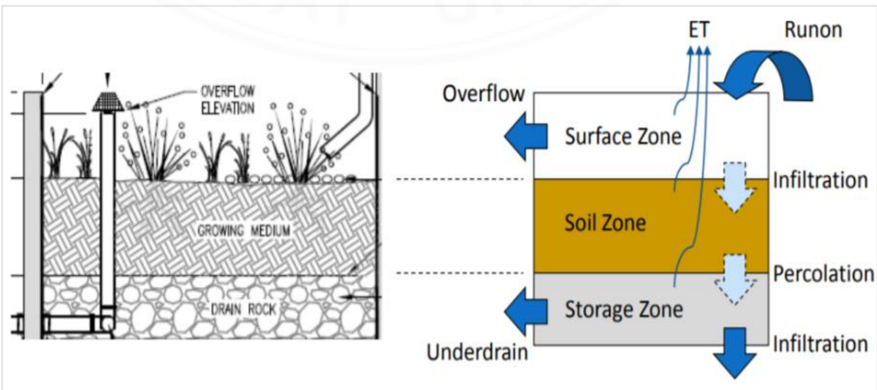

Figure 3.7 Conceptual model of a LID process in PCSWMM (Rossman, 2010)

3.4.2 PCSWMM Configuration for Baseline Condition

Important data types including topography, climate data and hydrology are required to set up the model in order to assess the hydrologic performance of the study area, specifically for pluvial flooding situations with respect to runoff volume, surface flood volume, peak flood discharge, and duration. In this case, a 2-year, 24-hour (72 mm depth) design storm obtained from Weesakul et al. (2017) was used to drive the study scenarios. We note here that for all model scenarios, runoff is assumed to drain within 24 hours so that the system would have enough capacity to deal with the next storm event. This assumption could be evaluated in the future using a continuous rainfall timeseries run of 1 year.

Generally, topography data describes the physical characteristics of any specific land area while also illustrating land elevation information. As indicated in **Table 3.2**, this data type is significant for subcatchment delineation and characterization of runoff drainage system and was obtained by using satellite imagery available in google earth and through field observation to determine land-use characteristics, location boundaries, road system, percentage of impervious area, drainage system and flow data. Surface flow direction was defined by using elevation data specific for Nava Nakorn Pathum Thani available at (https://www.floodmap.net/?gi=1607983). Drainage canal slope also was defined using the surface elevation data from (https://www.floodmap.net/?gi=1607983) and the slope and canal geometry data provided in the Nava Nakorn Master Plan report which is either 1:200 or 1:1000. Attribute values including rim elevation and invert elevation for individual junctions were then estimated considering elevation change and canal geometry between one junction to another. An engineering schematic of the drainage pipe definition, with accompanying attributes is provided in **Figure 3.8**.

Horton infiltration with dynamic wave routing were used to drive the model. The infiltration parameters for this study were based on the suggested infiltration values for clayey soil in the SWMM manual, with a maximum infiltration rate (f_0) of 50.8 mm/hour, minimum infiltration rate (f_∞) of 1.27 mm/hour, and decay coefficient (K_d) of 4.14/hour. The appropriate input values used for the study are provided in detail in Table 3.3.

Table 3.3 Important data for hydrologic assessment of current Nava Nakorn

N^0	Data	Specification	Sources of data
1)	Subcatchment division	Site characteristic, location boundary, road system and land-use	- Field observation satellite image in google earth
2)	Parameterization	Land use, land slope and soil type	- Nava Nakorn company reports - SWMM manual guidelines
3)	Outline sewer conduit	Junction node, conduit, outfall, pump, flow path	- Nava Nakorn company reports - Floodmap.net website
4)	Design storm selection	Design storm period from AIT rain gauge, 2-year design storm in 24 hours	- Weesakul et al., (2017)

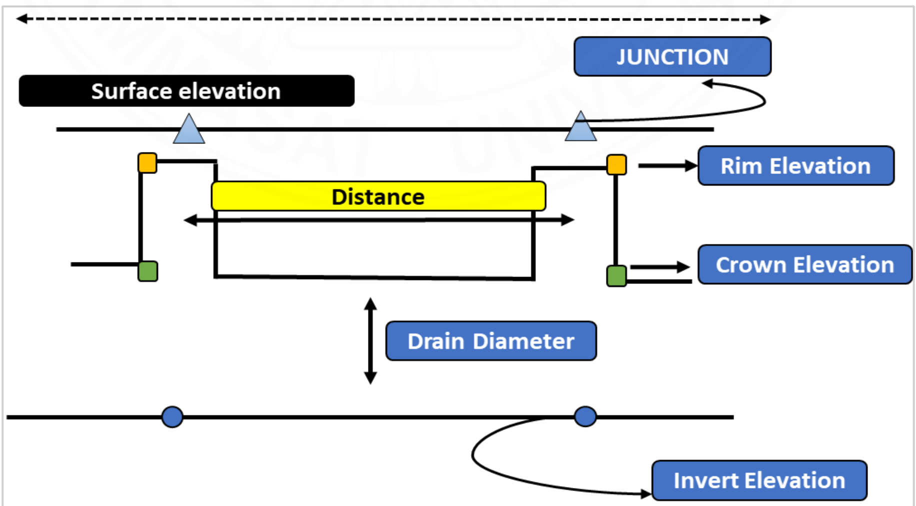

Figure 3.8 Diagram representing on surface view of connected junction nodes as a series of conduit and ground view of a pipe system

Table 3.4 Parameter inputs used for PCSWMM

Parameter	Typical value for Nava Nakorn	Chosen value
Land slope*	0-1 %	0.5 %
Percentage of impervious area*	Commercial and business areas 85% Industrial districts 72%	85% (for most of the sub-catchments)
Percentage of impervious area with no depression storage**		75%
Manning's N for impervious area**	Corrugated metal 0.024 Cement rubble surface 0.024	0.024
Manning's N for pervious area**	Short, prairie grass 0.15	0.15
Depth of depression storage on impervious area**	0.05 inches - 0.10 inches (1.27 mm - 2.54 mm)	1.27
Depth of depression storage on pervious area**	Lawns 0.1 inches - 0.2 inches (2.54 mm - 5.08 mm)	3.81
Conduit Manning's roughness coefficient **	Concrete 0.011 - 0.020 Excavated earth winding uniform	0.015 0.03
Conduit cross section geometry***	Closed rectangular conduit Trapezoidal open channel	0.5 × 0.5 2.2 m × 1m (slope 1 m left and 1 m right

*measured in this study; **from SWMM manual; ***from Nava Nakorn reports

3.4.3 WSUD Features and Scenario Development

Four common types of WSUD control, bioretention cell, bioswale, raingarden, and detention pond, were modelled in PCSWMM using the LID editor function. The fundamental designs of these were entirely based on those proposed by year 3 Landscape Architecture (LA) and Urban Design and Development (UDDI) studio classes for the Thammasat Nava Nakorn smart city project (see "TUNN Smart District report", 2020). All designs were assessed using various parameter inputs with regard to its performance in controlling flood generated under the 2-year storm condition. Each WSUD feature was developed with either 2, 3 or 4 scenarios representing a range of conditions, from the conservative design to the optimum level design. **Table 3.4** summarizes some of the fundamental design criteria that were proposed by the students, including area occupied by the WSUD control, surface width and number of replicated units each design needed. Some of the student design concepts (**Figure 3.9**) were impossible to implement in real life (i.e., the flow hydraulics for the community island, purifying island and gardening island concept were not feasible) while some of the designs involved reversing the current flow direction back to the upstream which is in the north of Nava Nakorn and which would be both costly and likely would overwhelm system capacity in that area, possibly leading to worse flooding. The gardening island in this case simply was assumed as the greenspace with the area occupying about 21,500 m^2 and the surface width of 214 m but was not included in discussion. Detail of the individual design and its scenario development are discussed in the following sections.

Table 3.5 Design criteria for WSUDs feature in this study

LID controls	Area occupied (m^2)	Surface width (m)	Replicated unit
Bioswale	26,500 / 53,000	1 or 2 / 2 or 4	n/a
Bioretention cell	35,500	2 or 2.2	n/a
Raingarden			
- S131	215	2	18
- S142	100	2	30
- S104	214	2	25
- S134	220	3	26
Greenspace	21,437	214	n/a

Note: n/a = not available

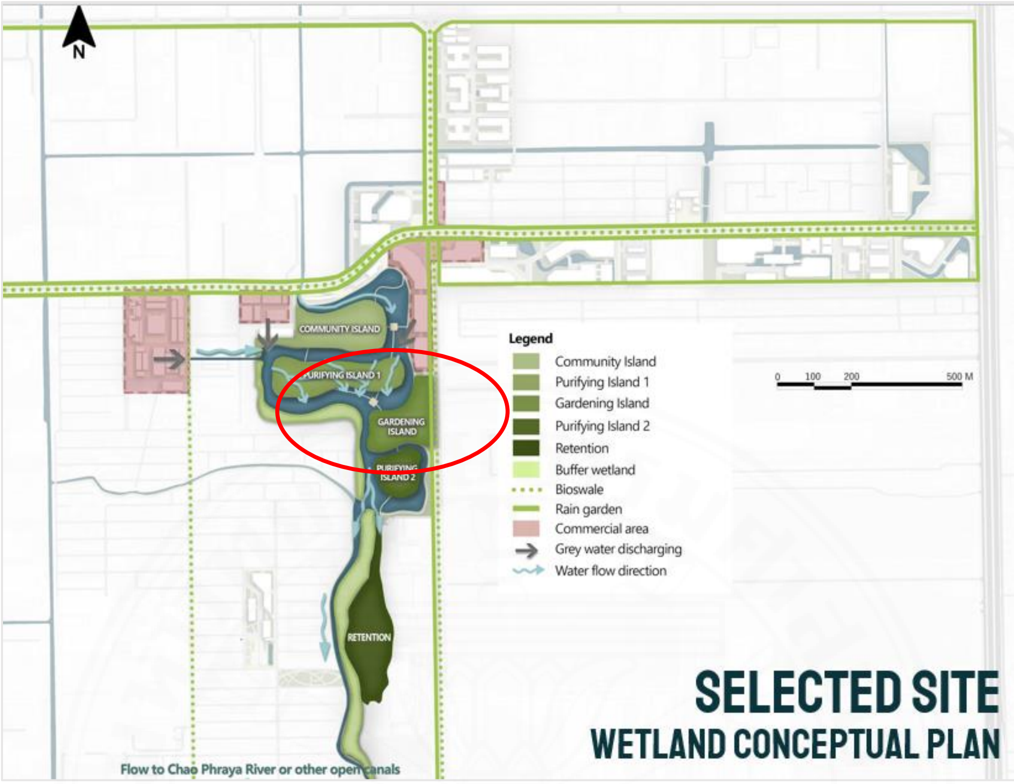

Figure 3.9 Wetland conceptual development plan for Nava Nakorn, meanwhile the purifying inland, and the gardening island which turned to greenspace are in the circle

3.4.3.1 Bioretention Cell

Four major roads of the study area were suggested to have bioretention cell with the objective of improving community livelihood and environmental quality of the area, while simultaneously producing a potential adaptive solution to control flooding. The four sections representing different roads in the area where traffic problems and flooding were raised as a concern, included two shared streets, mostly surrounded by residential and commercial land use, located in the main road or section A-A' with a length of 7,120 m (Figure 3.10), section B-B' referred to as a community main street with a 3,546 m long bioretention cell being placed on the road median (Figure 3.11). The other two sections (section C-C' and D-D') are surrounded by industry, and designs provided about 3500 m and 3250 m, of bioretention, lengthwise respectively, (Figure 3.12).

For the four sections, biorientation cells were modelled to occupy an area of 35,500 m^2 with a surface width of either 2 m or 2.2 m (**Table 3.4**). The model inputs for the three scenarios of this control were then determined according to the soil characteristics (infiltration rate, surface roughness, porosity, field capacity, conductivity) and are detailed in **Table 3.5**. Most of the input values were identical, except for some storage related controls (berm height, soil thickness and storage height) as we used these to explore the optimum performance result. Most of the inputs for the bioretention cells were based on the SWMM manual guidelines, however, the storage conductivity followed a study conducted by Irvine and Chua (2016) which modelled stormwater runoff from an urban park in Singapore with the value of 219 mm/hour as an average infiltration value was used. As for the underdrain (required when native soil has low infiltration capacity rate), a drain coefficient was calculated following **equation 3.1**.

$$q = C \times (h)n \qquad \text{(equation 3.1)}$$

Where:

Q = underdrain flow (mm/s)

C = underdrain discharge coefficient ($mm^{-(n-1)/sec}$)

H = hydraulic head seen by the underdrain (mm)

n = underdrain discharge exponent

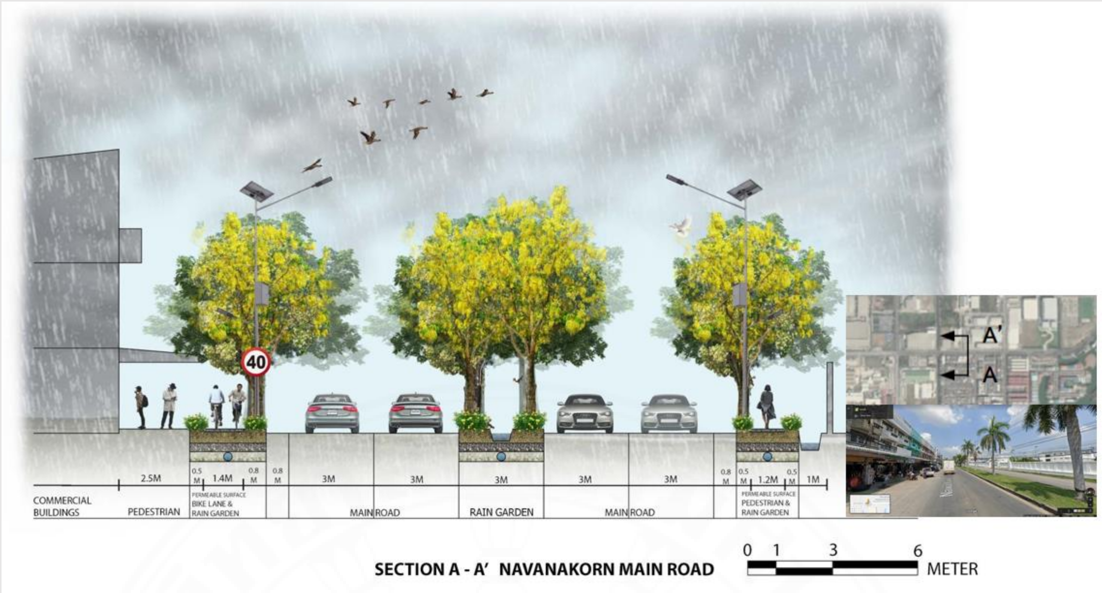

Figure 3.10 Cross sectional view of a typical raingarden for the major road inside Nava Nakorn or section A'A

Figure 3.11 Bioretention cell design on the community main street (section B'B) with underdrain illustrated as in streetscape view (Designed by Tanavara, T., Paveena, K., Manita, I., Yuto, M.)

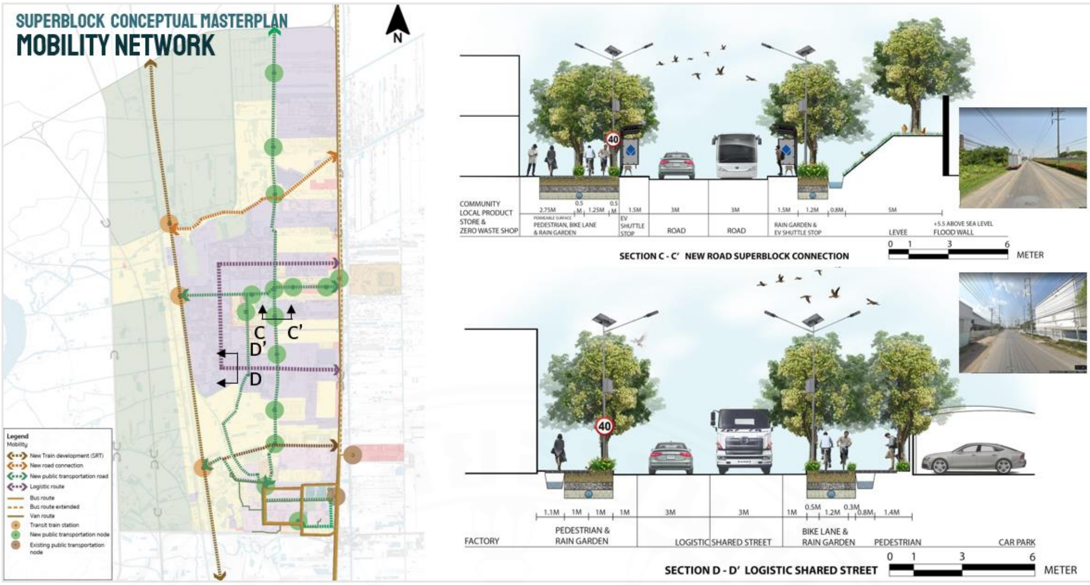

Figure 3.12 Bioretention cell design to improve 2 major transportation roads named as section C (above) and section D (below)

Table 3.6 Scenario design for bioretention cell in PCSWMM

LID controls	Scenario		
	1	2	3
Surface			
Berm height (mm)	150	300	300
Vegetative cover (fraction)	0.7	0.7	0.7
Manning's roughness	0.15	0.15	0.15
Surface slope (%)	0.5	0.5	0.5
Soil			
Thickness (mm)	450	650	1000
Porosity (volume fraction)	0.46	0.46	0.46
Field capacity (volume fraction)	0.15	0.15	0.15
Wilting point (volume fraction)	0.05	0.05	0.05
Conductivity (mm/hr)	108	108	108
Conductivity slope (%)	5	5	5

Table 3.6 Scenario design for bioretention cell in PCSWMM (Continued)

LID controls	Scenario		
	1	2	3
Storage			
Suction head (mm)	49.5	49.5	49.5
Height (mm)	150	450	850
Void ratio (voids/solids)	0.5	0.5	0.5
Conductivity (mm/hr)	219	219	219
Clogging factor	0	0	0
Underdrain			
Drain coefficient (mm/hr)	4.6	4.6	4.6
Drain exponent	0.5	0.5	0.5
Drain offset height (mm)	0	0	0

3.4.3.2 Bioswale

Some major roads were also recommended to have bioswale design. Bioswales in this case were covered on the three sections of Nava Nakorn main road (section A-A', 7120 m long, **Figure 3.13**), shared street in Phaholyothin road (section B-B', 3546 m long, **Figure 3.14**) and heavy and light route in Nava Nakorn 5 road (section C-C', 5,200 m long, **Figure 3.15**). Bioswales were designed to cover about 26,500 m^2 in area with a surface width of either 1 m or 2 m, and 4 different scenarios. All scenarios retain the same coverage area noted in **Table 3.4**, except for scenario 3 in which the coverage area was double the proposed one (i.e., the width was increased from 1 m to 2 m). The input values for the bioswale parameters were adapted from SWMM user manual and are given in **Table 3.6**.

Unlike bioretention cells, bioswales would only consider the surface design characteristic and these includes berm heigh, vegetative cover, manning's roughness, surface slope and swale side slope. In general, SWMM recommended a value of 150 mm to 300 mm for the berm height of a bioswale, as if the swale is used for stormwater runoff conveyance, with the swale side slope of 2:1 (rise/run) as the steepest and 4:1 (rise/run) as the flattest. However, design practice for bioswales in tropical Singapore can have berm heights of between ~600 mm and 1,000 mm in depth.

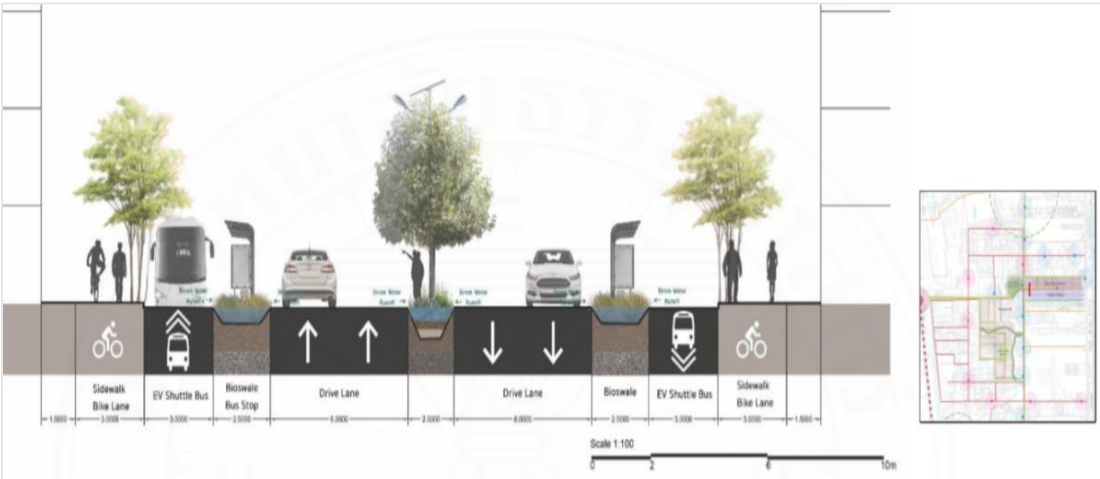

Figure 3.13 Section A-A’: Share street in Nava Nakorn main road with bioswale for storm drainage

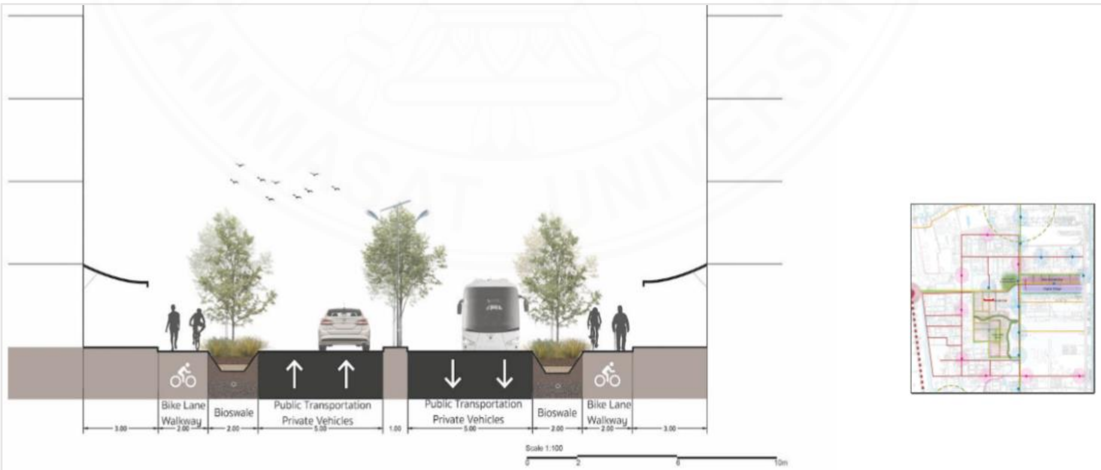

Figure 3.14 Section B-B’: Share street in Phaholyothin road

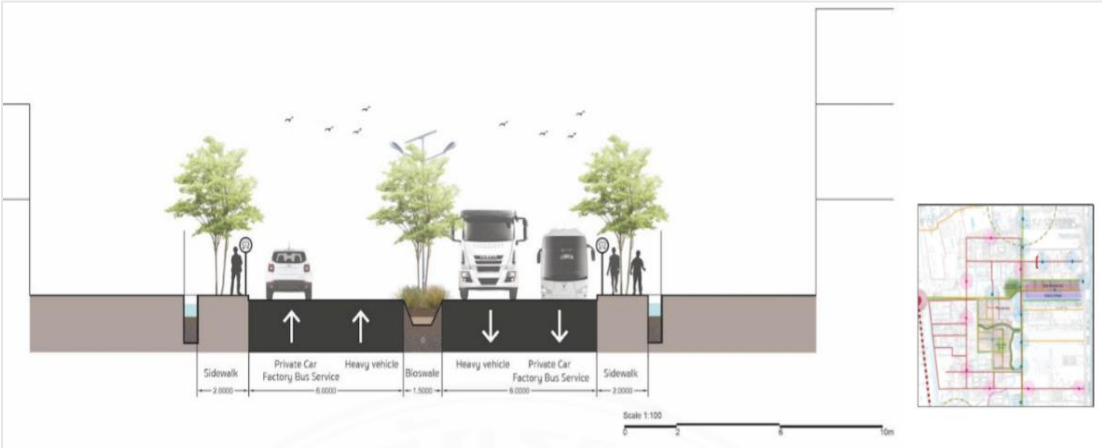

Figure 3.15 Section C-C': Heavy and light route in Nava Nakorn 5 road

Table 3.7 Scenario design for bioswale in PCSWMM

LID controls	Scenario			
	1	2	3	4
Berm height (mm)	150	300	300	1000
Vegetative cover (fraction)	0.7	0.8	0.9	0.9
Manning's roughness	0.1	0.15	0.15	0.15
Surface slope (%)	0.5	0.25	0.25	0.25
Swale side slop (rise/run)	3	4	4	4

3.4.3.3 Raingarden

Raingardens were assumed to be placed on some of the vacant lands that are available in the area with respect to the student's concept suggested in the TUNN smart city report. The reviews indicated that 4 land plots are still free from the constructions therefore remain a potential space to improve aesthetic, environmental quality and community wellbeing. Two of the plots nearby the Nava Nakorn main office which is at the central part of the estate were proposed to be an ecopark while the other two plots were suggested to be a gardenland park. The proposed designs, however, were not detailed for any specific values that could be used for the modelling therefore we assumed all these areas to be raingarden. An example of the student idea regarding the gardenland park development is illustrated in **Figure 3.16**.

Raingardens were positioned in the total area of 17,940 m^2 for the four plots proposed, either 2- or 3-meter width (see **Table 3.4** for the specific area breakdown). The area of raingarden was estimated following the Vermont raingarden manual for the gardening to absorb the storm (see Andreoletti, 2007). The process of sizing the area of the raingarden that was applied in this study is given in **Figure 3.17**. Basically, the raingarden area is estimated by multiplying the total draining area that would go to the raingarden with the sizing factor value. As discussed, a typical soil like clay has very low infiltration rate (between 1.27 and 50.8 mm/hour) compared to other soil types therefore the sizing factor for this type of soil is 0.1. The conductivity rate was chosen as a typical value for sand as it has better infiltration capacity and is normally amended with the native soil combined with organic compost fertilizer to support infiltration and plant growth. Scenario design for the raingarden is provided in **Table 3.7**. Most of the input values for the surface layer, soil and storage layers were the same as the values used for the bioretention cell design. Parameters that we used to assess performance of this control include berm height, soil thickness and storage.

Figure 3.16 Example of gardenland park design (referred as raingarden in the study) for a vacant land area represented as subcatchment 104 in the model (designed by Tanavara, T., Paveena, K., Manita, I., Yuto, M.)

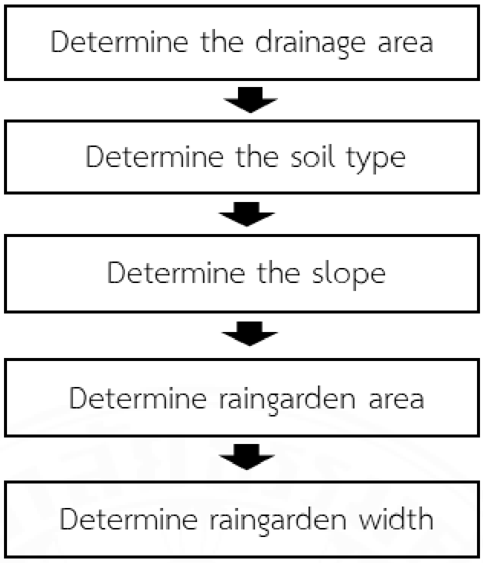

Figure 3.17 Process of sizing raingarden area for this study

Table 3.8 Scenario design for raingarden with underdrain in PCSWMM

LID controls	Scenario		
	1	2	3
Surface			
Berm height (mm)	150	200	300
Vegetative cover (fraction)	0.7	0.7	0.7
Manning's roughness	0.15	0.15	0.15
Surface slope (%)	0.5	0.5	0.5
Soil			
Thickness (mm)	450	650	1000
Porosity (volume fraction)	0.46	0.46	0.46
Field capacity (volume fraction)	0.15	0.15	0.15
Wilting point (volume fraction)	0.05	0.05	0.05
Conductivity (mm/hr)	108	108	108
Conductivity slope (%)	5	5	5
Suction head (mm)	49.5	49.5	49.5

Table 3.8 Scenario design for raingarden with underdrain in PCSWMM (Cont.)

LID controls	Scenario		
	1	2	3
Storage			
Height (mm)	150	450	850
Void ratio (voids/solids)	0.5	0.5	0.5
Conductivity (mm/hr)	219	219	219
Clogging factor	0	0	0
Underdrain			
Drain coefficient (mm/hr)	4.6	4.6	4.6
Drain exponent	0.5	0.5	0.5
Drain offset height (mm)	0	0	0

3.4.3.4 Detention Pond

One of the pluvial flooding control methods proposed by the students was the purifying island, a concept that used a wetland to control runoff discharge rate. This design would be implemented within the central part of Nava Nakorn to control pluvial flooding around that area (see **Figure 3.9**). The design as presented in **Figure 3.9**, however, does not have enough detail that can be used for implementation within the model and as such, the idea was replaced with a wet detention pond. The detention pond was modelled to manage runoff draining from the area of 17 ha using offset control (inlet elevation and outlet elevation) and flood storage.

In this case, the storage pond was expected to capture 65 % of the maximum total inflow from junction SU1 (0.43 m^3/s) that receives runoff from the 17-ha catchment area. With the maximum design outflow of 35 % (0.15 m^3/s), required a storage volume of about 2,615 m^3. Construction of a detention pond normally requires both permanent and temporary pools. As recommended, the minimum permanent pool depth is 0.9 meters while the deepest point is 3 meters for both quality and quantity control purpose (although only quantity control is included in this study) (Pitt, 2003). Offset control values were determined to manage the inflow and outflow for the pond. Two scenarios were developed for this feature where the required capture volume remains as the basis. The first scenario occupied on the area of 2,100 m^2 with an overall depth of 1.2 meters and a permanent pool depth of 0.9 meters which could store up to 630 m^3 of runoff water while the second scenario occupied an area of 1450 m^2 with an overall depth of 1.8 meters and a permanent pool depth of 1 meter deep that could store water up to 1,160 m^3.

CHAPTER 4
RESULT AND DISCUSSION

4.1 Sub-catchment and Drainage System Delineation

Nava Nakorn was divided into 142 sub-catchments, 179 junction nodes, 179 conduit lines, and 5 pumping stations (**Figure 4.1**). Rainfall runoff system was combined as a series of connected pipes and excavated canals from one junction node to one junction node to convey the runoff to the pumping stations before being released to the outside natural waterway. Manholes or junctions are located along the road with elevation changes of 1:1000 and 1:200, while conduits are assumed made of cement with a closed rectangular geometry of 0.5m × 0.5m (based on Nava Nakorn reports).

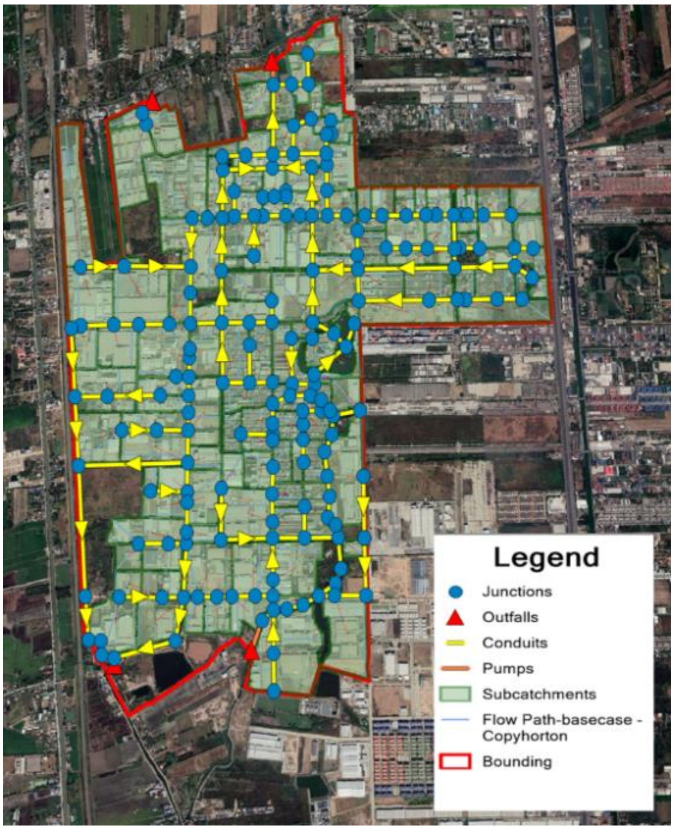

Figure 4.1 PCSWMM configuration for the runoff and drainage systems inside Nava Nakorn

4.2 Result of Rainfall Runoff Estimation

Based on the design storm shape, maximum rainfall would occur between 11:40 am to 11:55 am (see **Figure 4.2**). The runoff volume continuity error for the entire study area was an acceptable (–0.245 %) with the flow routing continuity error was –0.008 %. The maximum rainfall runoff rate for the entire Nava Nakorn study area was 119 m^3/s (**Figure 4.3**) with the total runoff volume being at 495,400 m^3 for the 24-hour rainfall event. The greatest runoff peaks appeared at subcatchments: S72, S97, S4, S73 and S137 with the values of 1.92, 2.07, 2.24, 2.26 and 2.47 (m^3/s), respectively. The lowest peak runoff was estimated with the values of 0.056, 0.15, 017, 0.228 and 0.229 (m^3/s) for the sub-catchments S142, S50, S3, S140 and S130. According to the simulation results, the factors influencing runoff rate include the size of the subcatchment, the percentage of impervious area, and the width of the subcatchment. Sensitivity analysis of these factors should be addressed in future work.

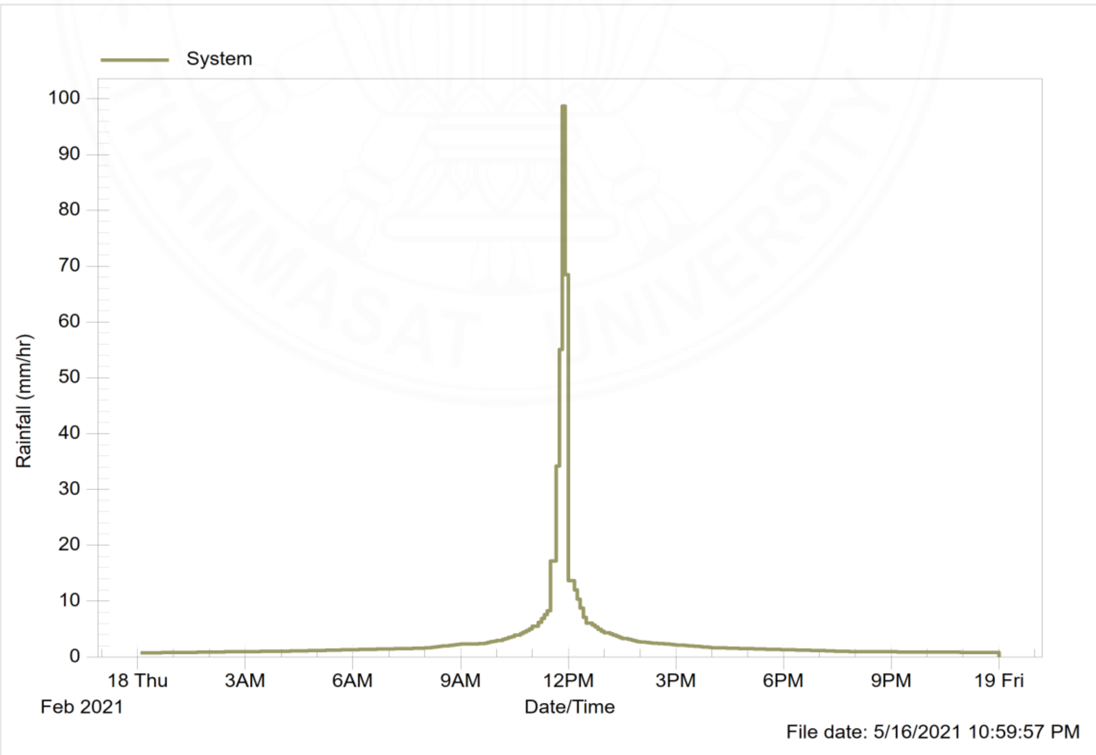

Figure 4.2 Behavior of selected rainfall event

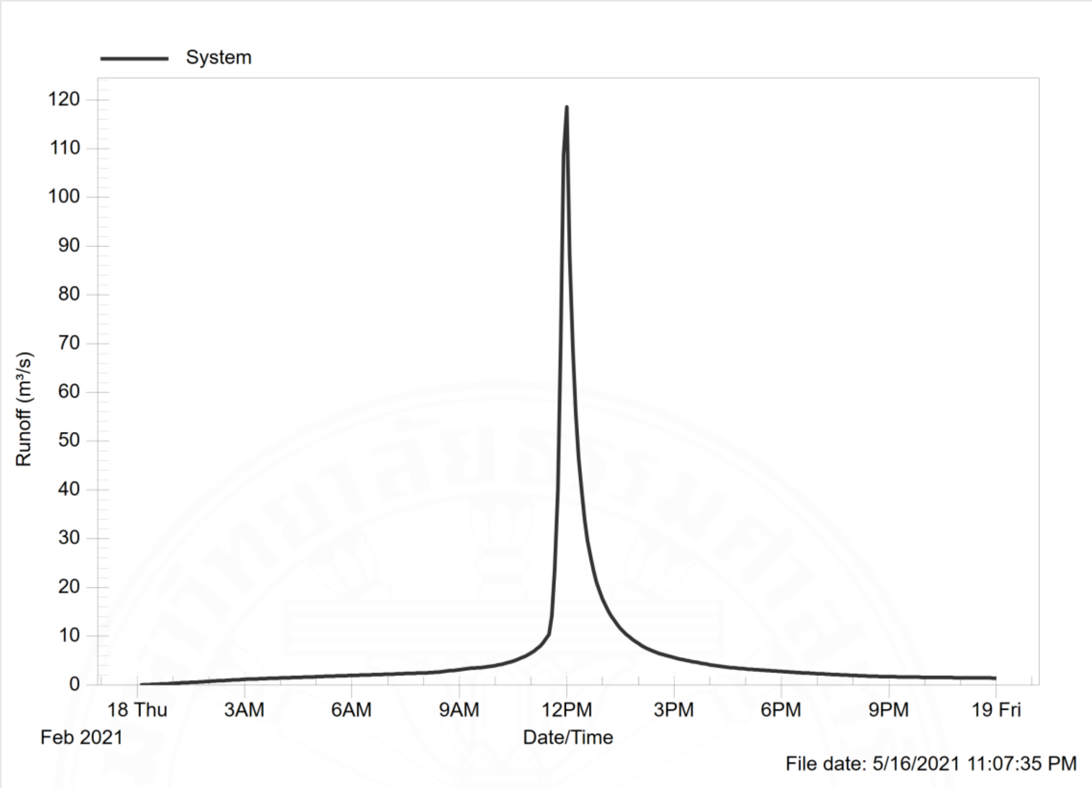

Figure 4.3 Total runoff volume produced under the 2-year design storm (72 mm, 24 hours) for the entire study area

4.3 Model Calibration and Validation

PCSWMM was calibrated based on an observed rainfall intensity which produced surface pluvial flooding inside Nava Nakorn. The observation indicated that 39-43 mm of rain in 2-3 hours caused pluvial flooding (see **Figure 4.4**) which could pose inconveniences to local people there. In this case, 39 mm of rain was selected for model calibration. Observed flow data were not available for the canals or catchments to calibrate the model and therefore satellite image data representing repetitive flooding event from https://www.gistda.or.th was used for calibration/validation. The calibration effort prioritized areas with occurrence of surface flooding (although information regarding flood depth and duration are not available). In PCSWMM, different parameters that could influence on the likelihood of flooding as such subcatchment imperviousness, pipe manning's roughness coefficient, flow path and conduit geometry were adjusted until it could represent the rainfall drainage system for the study area (see **Table 3.3** as the final input to the model). From the observed rainfall data as input, model showed areas with localized flooding matching to the satellite flood map (see **Figure 4.5**) with total runoff volume for the entire study area being 248,200 m^3 and maximum runoff rate of 51.6 m3/s. Of the 179 junctions that represented the drainage system for the entire Nava Nakorn, 76 junctions appeared with flooding events (displayed in the red circle). Flooding covered different parts of the area, including residential, commercial, and some major transportation routes (see **Figure 4.5**). Flooding would likely to increase with the bigger storm events both localized flooding and the whole system. Our model development appeared to match well with the pluvial flooding events estimation for Nava Nakorn.

Figure 4.4 Localized pluvial flooding inside Nava Nakorn during last wet month, 2020

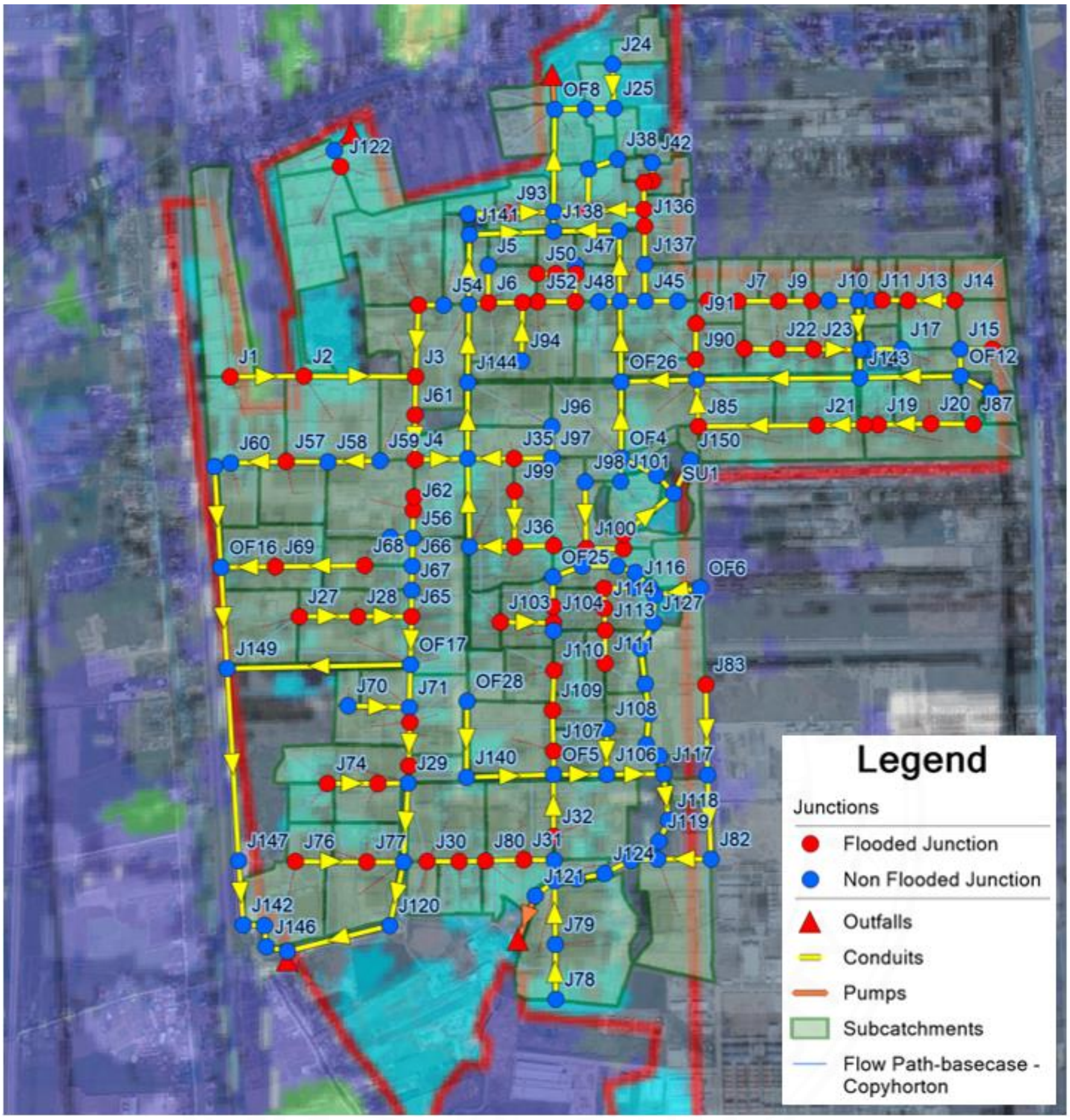

Figure 4.5 Overlapped observed and satellite flooding map to investigate localized flooding event, where flooded junctions are in red and non-flooded junction are in blue

4.4 Flooding Level and Duration

The total surface flooding (pluvial flooding) volume inside Nava Nakorn during the 2-year storm was 206,100 m^3, which is approximately 40% of the total runoff for the entire study area noted in the previous section. A complete result from the simulation showed that of the 179 junction nodes, 79 would experience surface flooding under the

2-year storm (see **Figure 4.6**). The maximum surface (pluvial) flooding for each flooded junction was 0.85 ± 0.7 m^3/s. The result showed that in general the larger the volume of flooding, the longer duration the area is flooded, which is noticeable for J40 (15,958 m^3, 18.9 hour), J2 (12,711m^3, 16.7 hour) and J1 (11,023m^3, 7.97 hour) and is attributed to the fact that these junctions are responsible for runoff from larger areas compared to other junctions. However, for some junctions, duration of flooding is shorter despite the amount of runoff from the contributing area being greater compared to some junctions that exhibited longer durations but smaller runoff volumes. These include: J3 (4,105m^3, 9.1 hours), J21 (3,471m^3, 5.91 hours), J80 (2,425m^3, 4.86 hours), and J34 (2,035m^3, 4.83 hours). This would result from the different characteristics of the contributing areas (imperviousness and overland flow length) to each junction as well as the drainage pipe flow characteristics.

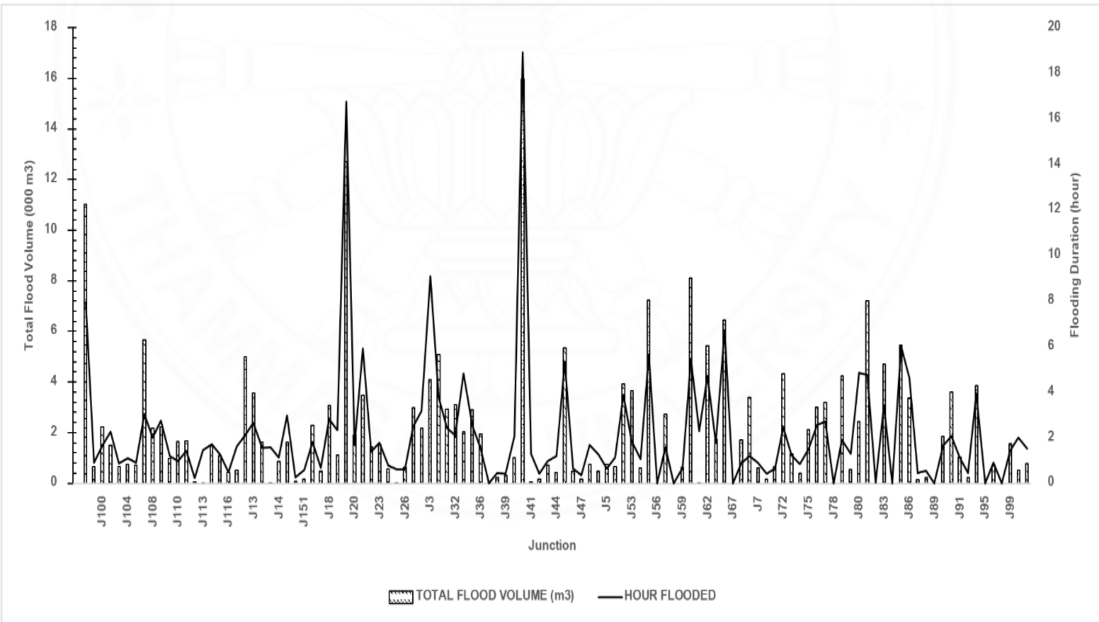

Figure 4.6 Comparison between flood duration and total flooding volume (surface flooding) by junctions

4.5 Result of Scenario Simulation

4.5.1 Bioretention Cell

4.5.1.1 WSUD impact on the entire Nava Nakorn Estate

Bioretention cells reduced total runoff by a greater percentage than surface flooding, with the reduction for the entire study area being 11.7% - 12.7% for scenario 1 to 3 (**Table 4.1**). The area ratio of control measure had a significant impact on the performance of the bioretention cell. Specifically, runoff volume reduction of each subcatchment varied between 6 – 72.5% for scenario 1, 9 – 78% for scenario 2, and 9.6 – 78% for scenario 3. With respect to surface flooding, bioretention cell design reduced flooding volume from the entire study area by 2.3 %, 3 %, and 3.2 % for scenarios 1, 2, and 3, respectively (see **Figure 4.7**). Similarly, reduction of flooding volume differed between junctions and scenarios with a reduction ratio of -965 – 100 % for scenario 1, -945 – 100 % for scenario 2, and -885 – 100 % for scenario 3 (see **Table 4.1**).

The results showed that when properly designed, the bioretention cell approach can be effective in managing flooding volume even though the implemented area was just about 0.4 % of the total study area. As shown, greater flooding volume decreased with a bigger design area as input. However, designs were less effective in terms of controlling peak surface flooding and flood duration, even with the scenario 3, peak flooding rate increased to 108.8 m^3/s compared to the base case which is about 93.2 m^3/s and as noted, the design in all scenarios decreased surface flooding by about 0.25 hour in average. It is possible that the larger storm volume of 72 mm for a 2-year storm, concentrated in a peak time (**Figure 4.2**) overwhelmed the peaking storage capacity for runoff. It was observed in the field that 39-43 mm of rain in 2-3 hours produced pluvial flooding and this was confirmed in the model. We would note that in running a smaller 39 mm rainfall event in PCSWMM as a trial, the best scenario design provided a greater control of flooding volume (by 11% flooding reduction for the whole study area), peak runoff remains uncontrollable. So, future efforts could focus on controlling the peaks of the smaller events like 30 mm or scaling the system up (and perhaps including additional pond storage) to manage the larger 2-year event.

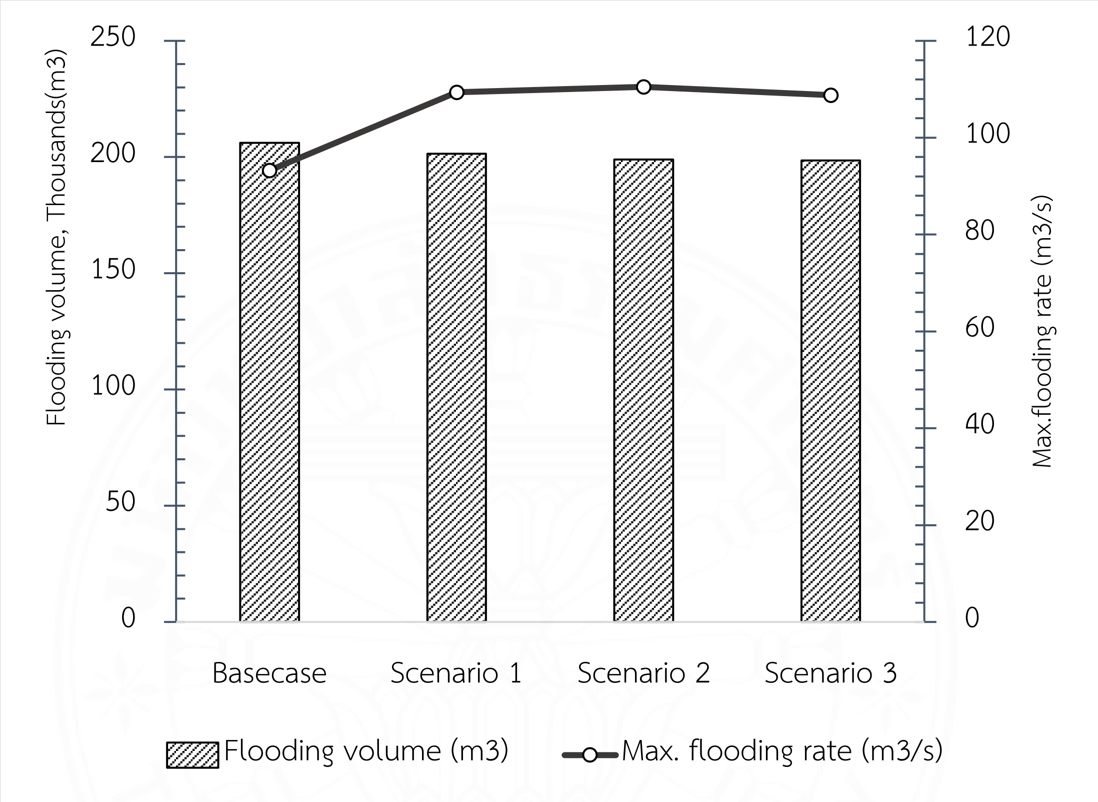

Figure 4.7 Performance of bioretention cell on flood reduction for the entire system

4.5.1.2 WSUD impact on the localized flooding

Undoubtedly, effectiveness of bioretention cell design to control localized flooding was higher compared to the entire system performance. Simulation result of the three scenarios indicated that between 49 – 53 % of 115 observed junctions associated with subcatchments with this control resulted in decreasing both flooding volume and duration. As indicated, average flood duration by each scenario was reduced by 0.25 hours with the least reduction of 0.01 hours and up to 5.2 hours the maximum reduction. The most impacted junctions were with J2 in which flooding decreased about 4.9 – 5.2 hours with volume reduction ratio between 15 – 15.3 % for scenario 1 – 3. The other high impact junctions including J80 (1.9 – 2 hours; 17 – 18%), J21 (1.8 hour; 17 – 17.4%), J86 (1.7 hour; 13 – 16.5%), J34 (1.5 – 1.7 hour; 1 – 6.6%), J62 (1.1 hour; 7 – 7.5%), J83 (0.5 - 0.56 hour; 10 – 10.5%), J82 (0.1 hour; 66 – 98%), and J84 (0.02 hour; 100%). These junctions were observed to be located along Nava Nakorn main road (section A-A') and heavy transportation road, factory and industry areas (section C-C' and D-D').

Bioretention was less effective in managing flooding volume from the residential complex (section B-B'), specifically junctions J116 (-965 to -885%), J112 (-228 to -223%), J37 (-200 to -100%). It is possible that a larger-sized bioretention cell would be required for these areas as from the result. For example, the bioretention cell servicing J116 was only about 0.07% of the total contributing area draining to this junction. Also, for some junctions along this section, the duration of flooding was shortened (up to 0.8 hour) despite flooding volume increased which could result from expanding the pervious area, hence improving infiltration.

Table 4.1 Performances of bioretention cell by different scenarios

Control	Baseline	Scenario 1	Scenario 2	Scenario 3
Impact of WSUD on the entire Nava Nakorn				
Total runoff reduction (%)	495,400 m3	11.7	12.5	12.7
Total flood reduction rate (%)	206,100 m3	2.3	3	3.2
Max. flooding rate (m3/s)	93.25	109.4	110.5	108.8
Impact of WSUD on localized flooding				
Runoff reduction varied by subcatchment (%)	-	6 – 72.5	9 - 78	9.6 - 78
Flooding volume reduced by junction (%)	-	-965 – 100	-945 - 100	-885 - 100
Average flood duration (hour)	1.7	1.47	1.46	1.45

4.5.2 Bioswale

4.5.2.1 WSUD impact on the entire Nava Nakorn

The finding indicated that scenario 4, in which the berm of the swale was assigned to 1 m appeared to provide the best performance compared to the other scenarios and this also included the runoff reduction benefit for the entire study area that was reduced by 3 % while scenario 1 – 3 could reduce only by between (0.7 – 2.4 %). Like the bioretention cell, performance of bioswales in managing runoff varied from one subcatchment to the next because the occupied area ratio of controlled bioswales varied between subcatchments, ranging from 0.3 to 4% for scenarios 1, 2, and 4 and from 0.6 to 8% for scenario 3. This results in a maximum runoff volume reduction for individual subcatchments up to 11%, 48.7%, 41% and 80% for scenarios: 1, 2, 3 and 4 (see **Table 4.2**).

Although all scenarios under this control had a benefit on runoff volume reduction (associated with total inflow and surface outflow), its benefit with respect to surface (pluvial) flooding was not so clear for the entire system comparison. As shown in **Figure 4.8**, the bioswale under all scenarios seems to be ineffective in controlling flooding under the 2-year return period design storm. As indicated, flooding volume, peak flooding rate and flood duration in all scenarios were higher than the baseline case (-15 % for scenario 1, -15.5 % for scenario 2 and -14 % for scenario 4) except scenario 3 that was relatively the same as the baseline scenario. And this would be a matter of undersized design as noted the total area of the bioswale being implemented was quite small to manage runoff generated for the bigger area like Nava Nakorn (as noted 0.25 – 0.5 %).

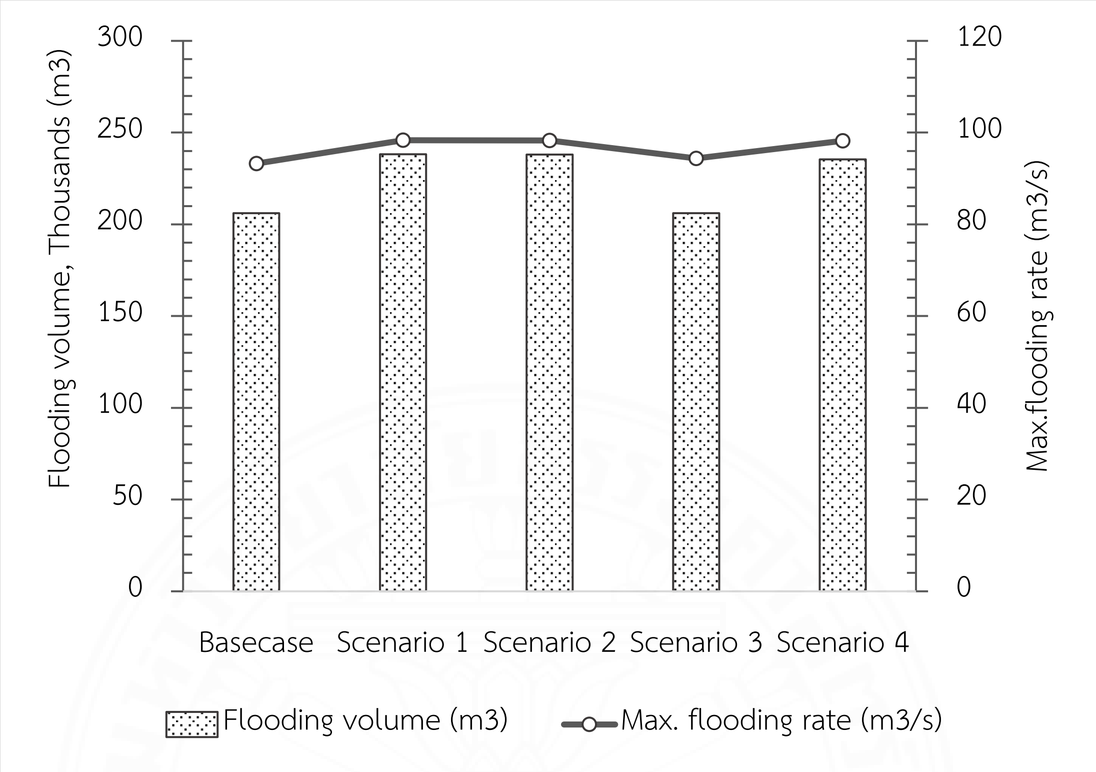

Figure 4.8 Performance of bioswale on flood volume reduction for the entire system by different scenarios

4.5.2.2 Impact of WSUD on localised flooding

Although it was not so clear with the bioswale performance regarding flood management for the entire study area, there were still some localised benefits produced under this control. From **Table 4.2**, bioswale under all scenarios could reduce surface flooding from the individual junctions by up to 100 % the maximum flood volume and up to 0.8-hour flood duration for scenario 1, 2 and 4 respectively. While the highest flood reduction for scenario 3 was about 0.5 hour. Reduction benefits under the scenario 3 were minor compared to the other scenarios but these positive impacts were observed to cover a greater number of junctions (36 % for scenario 3 while scenario 1, 2 and 4 were between 16 – 24% of the total 120 junctions that connected to this control). Bioswale was seen to be effective to control flooding partly for different sections under this control such as Nava

Nakorn main road (section A-A'), residential complex (section B-B') and heavy road industry area (section C-C'), also the downstream areas that linked to these sections. It is understandable that rainfall runoff drainage system connectedly works as a network therefore placing WSUD features in an area would impact the connected areas as a whole. However, there were parts that bioswale failed to control and this resulted in flooding greatly surging for some junctions along the section C-C'. For example, J45 under the design scenario 1 for which flooding increased by about 1,920 % flooding volume and 4-hour flood duration of the baseline scenario (260 m^3, 0.95 hours). It is likely that the design is undersized as comparison between scenarios indicated that flood surge occurred in scenario 1, 2 and 4 where bioswale is 1 meter wide while the double sized scenario 3 had some minor positive impact on these junctions. Increasing the size of the bioswale should be addressed further in the future as it may result in better flooding control for the 2-year storm. Additionally, investigating on the smaller storm event as suggested in bioretention cell (30 mm rainfall) should work for controlling peak events.

Table 4.2 Performances of bioswale by different scenarios

Control	Baseline	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Impact of WSUD on the entire Nava Nakorn					
Total runoff reduction (%)	495,400 m3	0.7	1	2.4	3
Total flood reduction rate (%)	206,100 m3	-15	-15.5	0	-14.2
Max. flooding rate (m3/s)	93.25	98.34	98.3	94.4	98.2
Impact of WSUD on localized flooding					
Runoff reduction varied by subcatchment (%)	-	0.1 – 11.4	0.2 – 48.7	0.3 - 41	0.6 - 80
Flooding volume reduced by junction (%)	-	Up to 100	Up to 100	Up to 100	Up to 100
Average flood duration (hour)	1.7	2.15	2.14	1.7	2

4.5.3 Raingarden

4.5.3.1 WSUD impact on the entire Nava Nakorn

Raingarden design under all scenarios improved runoff reduction rate by between 1.4 – 1.7 % for the study area despite the small area being occupied (18,000 m^2) with the volume reduction varying between 75 – 93 % for each subcatchment with this design. Beside its positive result on runoff volume reduction ratio, the raingarden also reduced flood volume and peak flooding rate. Raingarden with scenario 1 decreased flood volume for the entire system by 1.1% and reduced peak flow rate by 0.75 m^3/s from the baseline case, 93.25 m^3/s (**Figure 4.9**). While scenario 2 and 3 decreased flooding volume by 1.7 and 1.5 % and 1.67 m^3/s of its peak flow rate, equally. The average flood duration under raingardens were less different from the baseline case, as indicated by the highest performance scenario in this regard producing a reduction of just 0.02 hour. Based on flood volume, it seems that scenario 2 design had a higher value for the entire Nava Nakorn compared to scenario 1 and 3. However, as the main aim of placing WUD feature on the certain areas was to help improve flooding issues from those localized areas, it is even more important to consider WSUD benefits within these local areas. Impact of WSUD on the localised flooding are discussed in the next section 4.5.3.2.

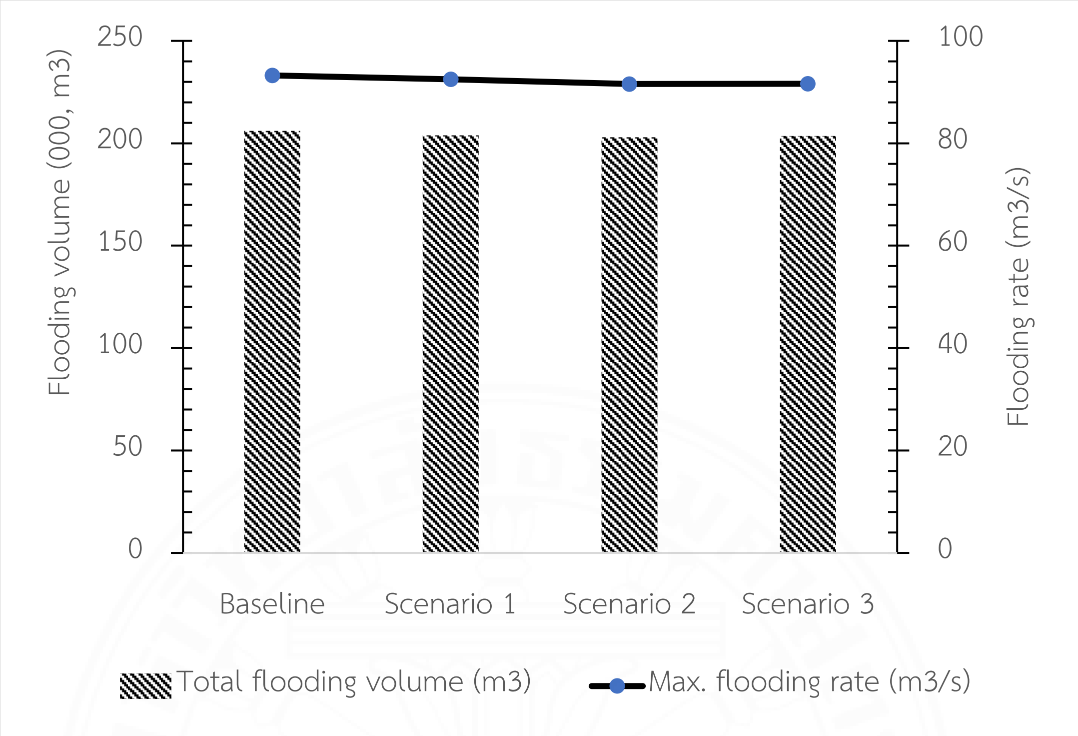

Figure 4.9 Performance of raingarden on flood reduction benefit for the entire system by different scenarios compared to the baseline

4.5.3.2 WSUD impact on localised flooding

Like the other controls in this study, performance of raingardens tends to be higher for localised areas compared to the whole system Nava Nakorn. As discussed, the maximum runoff volume reduced by between 75 – 93 % from the four subcatchments where raingardens were installed under all scenarios. Clearly, scenario 1 reduced runoff volume from the individual subcatchments in the range of 75 – 85 %, whereas scenario 2 and 3 resulted equally in the total reduction of 79 – 93 % (**Table 4.3**).

There were 119 junctions connected to this control in overall, however only 19 junctions were impacted under each scenario design. Flooding decreased by between 1 – 63 % and up to 0.4 hour under raingarden scenario 1, whereas flooding under scenario 2 and 3 reduced by up to 100 % and 82 % with duration control within the individual junctions by up to 0.6 hour and 0.5 hour, respectively. The most impacted areas were seen with the areas under the raingarden control and the nearby encompassing areas. For scenario 1, these include junctions J101 (57 %, 0.25 hour, 0.15 m^3/s), J108 (63 %, 0.4 hour, 0.13 m^3/s), J111 (33%, 0.25 hour, 0.3 m^3/s), J113 (11%, 0.3 hour, 0.02 m^3/s), and SU1 (14%, 0.1 hour, 0.03 m^3/s), flood volume, duration, and peak rate reduction, respectively. For scenario 2 and 3, reduction benefits were comparable with scenario 1 for most of these junctions except flooding volume in J101 and 108. Design under scenario 2 reduced flooding by up to 100 % from these junctions whereas scenario 3 reduced flooding by 82 % for J101 and 76 % for J108.

The results indicated that WSUD under the raingarden scenarios appeared to be effective in managing flooding from the localized areas which is a result of proper sizing of the design. However, among the three scenarios, raingarden performance happened to be the best with scenario 2 for both localised flood benefit and the entire area of Nava Nakorn (as discussed flooding for the entire study area reduced by 1.5 %). The bigger drainage area would require bigger sized design to capture runoff overflow effectively.

Table 4.3 Performances of raingarden by different scenarios

Control	Baseline	Scenario 1	Scenario 2	Scenario 3
Impact of WSUD on the entire Nava Nakorn				
Total runoff reduction rate (%)	495,400 m3	1.4	1.7	1.5
Total flood reduction rate (%)	206,100 m3	1.1	1.5	1.2
Max. flooding rate (m3/s)	93.25	92.5	91.58	91.59
Impact of WSUD on localized flooding				
Runoff reduction varied by subcatchment (%)	-	75 – 85	79 - 93	79 - 93
Flooding volume reduced by junction (%)	-	1 - 63	1 - 100	1 - 82
Average flood duration (hour)	1.7	1.69	1.68	1.68

4.5.4 Detention Pond

As discussed, two scenarios were created for a detention pond design in order to evaluate performance of this feature with respect to flood reduction benefit. The detention pond was placed close to the Nava Nakorn main office to capture runoff contributing from the surrounding area of 17 ha. The schematic representation of the WSUD design in the PCSWMM is provided in **Figure 4.10** while location of the pond is provided in **Figure 4.11**. All runoff produced from this total area were assigned to drain into the Nava Nakorn main drainage channel (represented as junction OF4 in the model, see **Figure 4.11**) in order to manage flooding from the area. However, flooding remained unmanageable and being highly concentrated in junction SU1 with the volume of 458 m^3 with the maximum flooding rate of 0.3 m^3/s and flood duration of 0.97 hour, therefore a pond was provided to manage flooding from this junction. From the result, detention pond under both scenarios could manage all the flooding events from this junction, however reduction benefit on the other connected junctions was slightly different between these two scenarios compared to the baseline scenario (see **Table 4.4**).

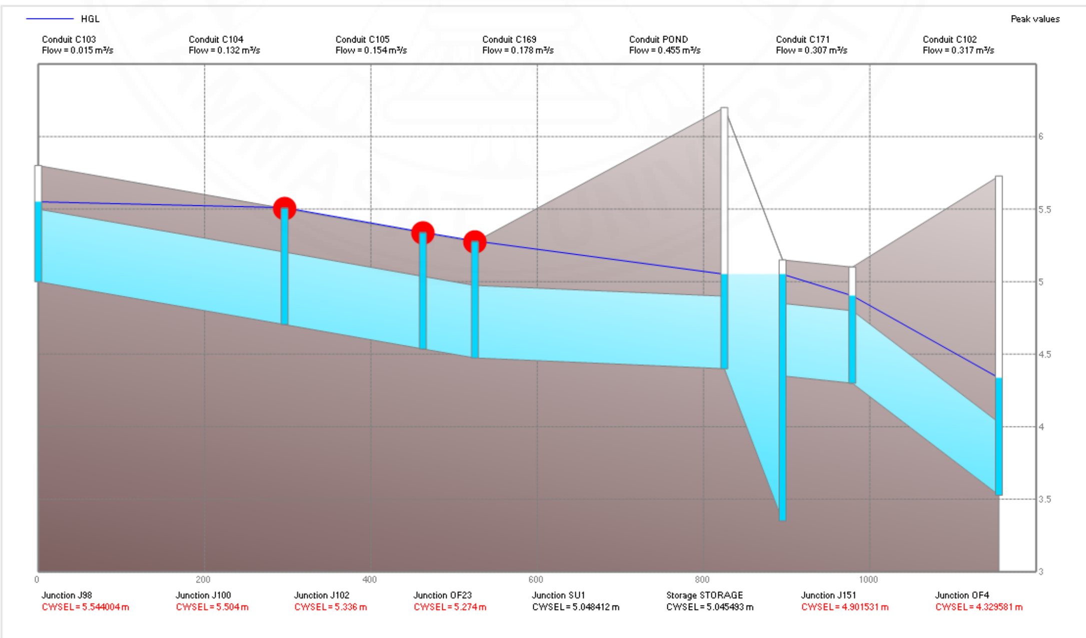

Figure 4.10 Schematic representation of detention pond design in the model

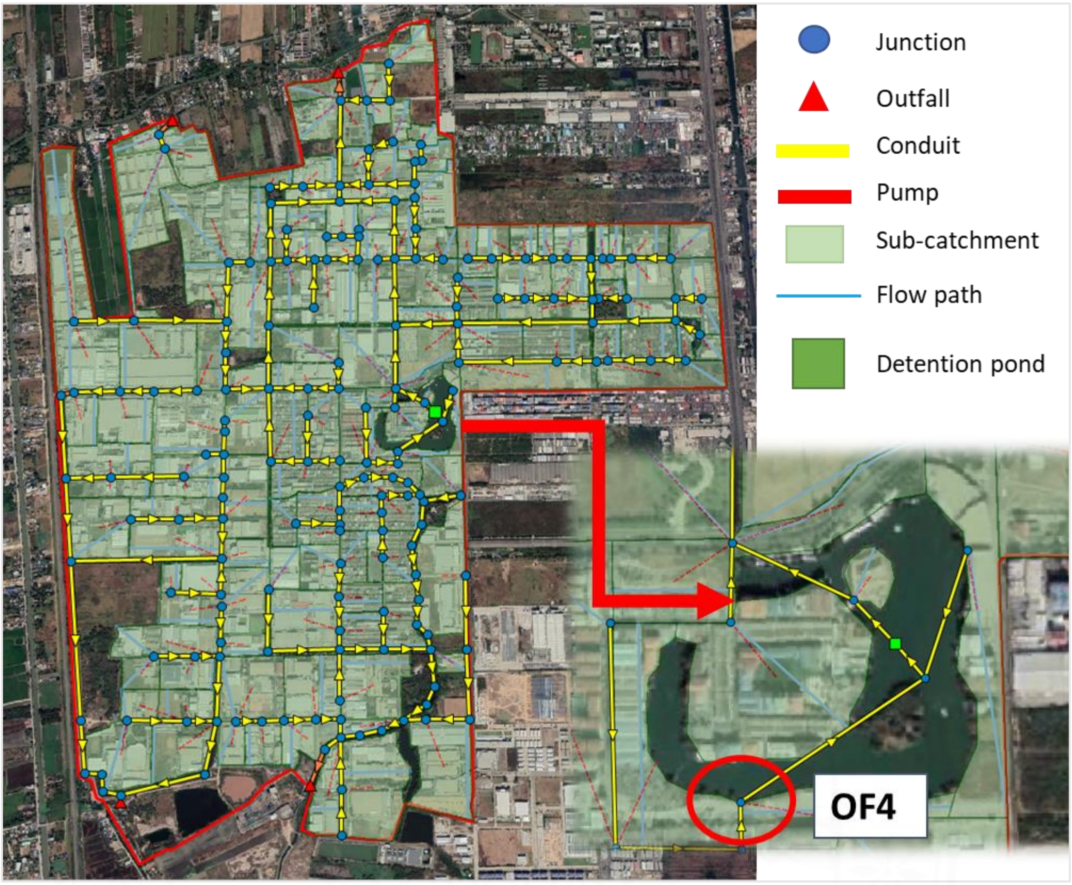

Figure 4.11 Detention pond set up in the model

Table 4.4 Performance of detention pond on the connected junctions by scenarios

Junction	Basecase		Scenario 1		Scenario 2	
	Flood volume	Peak flooding rate	Flood volume	Peak flooding rate	Flood volume	Peak flooding rate
J100	2243	1.35	2254	1.36	2255	1.36
J101	458	0.5	459.5	0.5	459	0.5
J102	1484	0.7	1572	0.7	1516	0.7
J150	77.5	0.13	106	0.16	54	0.1
J151	0	0	0	0	0	0
OF23	436.5	0.23	868	0.26	470	0.2
SU1	498	0.3	0	0	0	0

Figure 4.12 and **4.13** illustrate the storage capacity of the pond to manage runoff generated from the 2-year design storm. The result showed that the bigger storage pond (1160 m^3) would result in flooding control better than the smaller one (630 m^3). As indicated, the capacity of the pond to manage runoff was higher for scenario 2 compared to scenario 1. The pond under scenario 2 temporarily stored the water up to 7000 m^3 of total inflow with maximum inflow rate of 0.45 m^3/s, subsequently, pond depth increased to 1.7 meter (**Figure 4.13**) during this peak event. However, with scenario 1, the maximum water temporarily stored in the pond was about 6000 m^3 with the peak inflow rate of 0.36 m^3/s therefore water level in the pond reached up to 1.2 meter. As a result, a shallower pond like scenario1 produced a backwater effect, resulting in flooding volume double that of the baseline scenario for junction OF23.

Although the detention pond could eliminate the flooding problem efficiently from the area of concern, impact of this feature on surface flooding problem was minor for the entire Nava Nakorn (-0.1% for scenario 1 and 0.15% for scenario 2). Increasing the size of the pond or considering having a decentralized set of ponds would be required to cope with runoff from a greater drainage area to the pond.

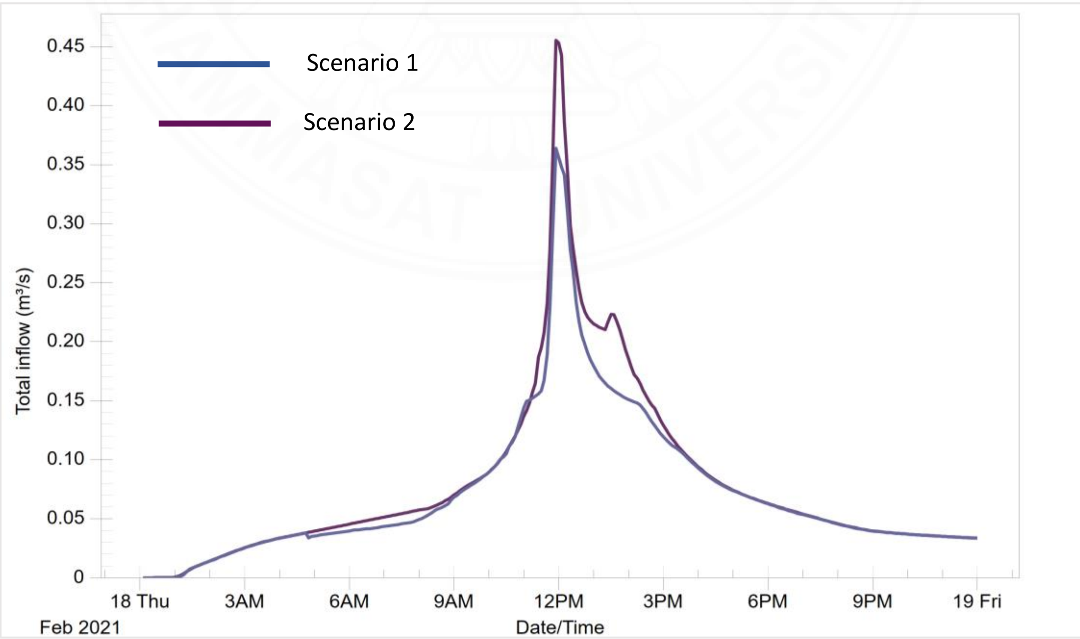

Figure 4.12 Maximum total inflow (m3/s) for the storage pond

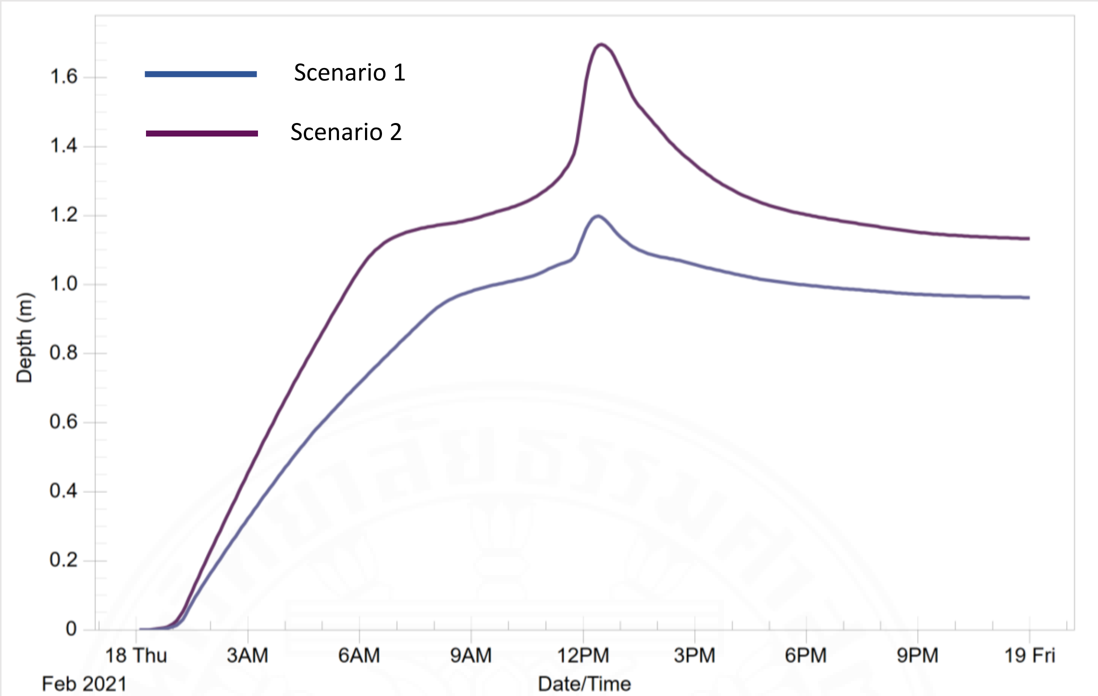

Figure 4.13 Maximum water depth (m) in the storage pond

4.6 WSUD Connectivity

Performance of WSUD connectivity to manage pluvial flooding was assessed with all WSUD features combined. This resulted in three different scenarios in which the first scenario was the combination of the conservative with least benefit scenarios, the second was from the aggressive design that maximize the benefit but higher cost, and the third one used the optimized design scenarios. With all WSUD such as bioretention cell, bioswale, raingarden and detention pond simultaneously occupying different sections of Nava Nakorn, surface runoff volume was reduced by between 12.5 – 15.8 % for scenario 1 – 3. Runoff reduction rate varied between subcatchments in the range of (-15 – 100%) for scenario 1, (-10 – 100%) for scenario 2, and (-8 – 100%) for scenario 3. Figure 4.14 provides total surface flooding volume and peak flooding rate data for the different combined WSUD scenarios compared to the baseline. As indicated, the optimized design scenario 3 made the biggest impact on flooding volume reduction (7.7%) while the poorest was from scenario 1 (4.2%). However, none of the scenarios resulted in peak flooding rate improvement which was noted as the matter of undersized design in some features like bioswale and bioretention cell along with the large storm 2-year return period that overtopped the system capacity.

Like the individual WSUD design, the performance of WSUD connectivity was better with the localized area than the entire system comparison. From **Table 4.5**, reduction of flooding volume at the individual junctions was up to 48.7 %, 100 % and 76.7 % for scenario 1, 2, 3 respectively. Although, the maximum reduction under scenario 2 was greater compared to scenarios 1 and 3, flooding flood volume in general was greater for many junctions in this scenario. As expected, most junctions under scenario 3 appeared to have the best result in regard to flood duration reduction with the maximum reduction of up to 5.7 hours while the poorest was from scenarios 1 and 2 which was about 4 hours similarly for the highest impacted junctions.

From these results, the optimum designs combined together appeared in scenario 3 which provided the best result in terms of runoff and flooding event control compared to the other two scenarios. Scenario 2 with aggressive designs collectively used as input presented not so much different compared to scenario 1 in which the designs in individual WSUD feature were assigned smaller (or using more conservative values). Inclusion of multiple WSUD features within the study area would be of benefit to improve pluvial flooding problem to be less severe, however, selecting appropriate designs to implement would also require a careful consideration between the cost and benefit. For example, it is possible that the cost would be higher for scenario 2 with the design taking up a larger area while the modelling indicated that it would be less effective compared to scenario 3 in which the design cost would be lower. The next section would provide the preliminary cost estimation of these different scenarios based on past study estimates.

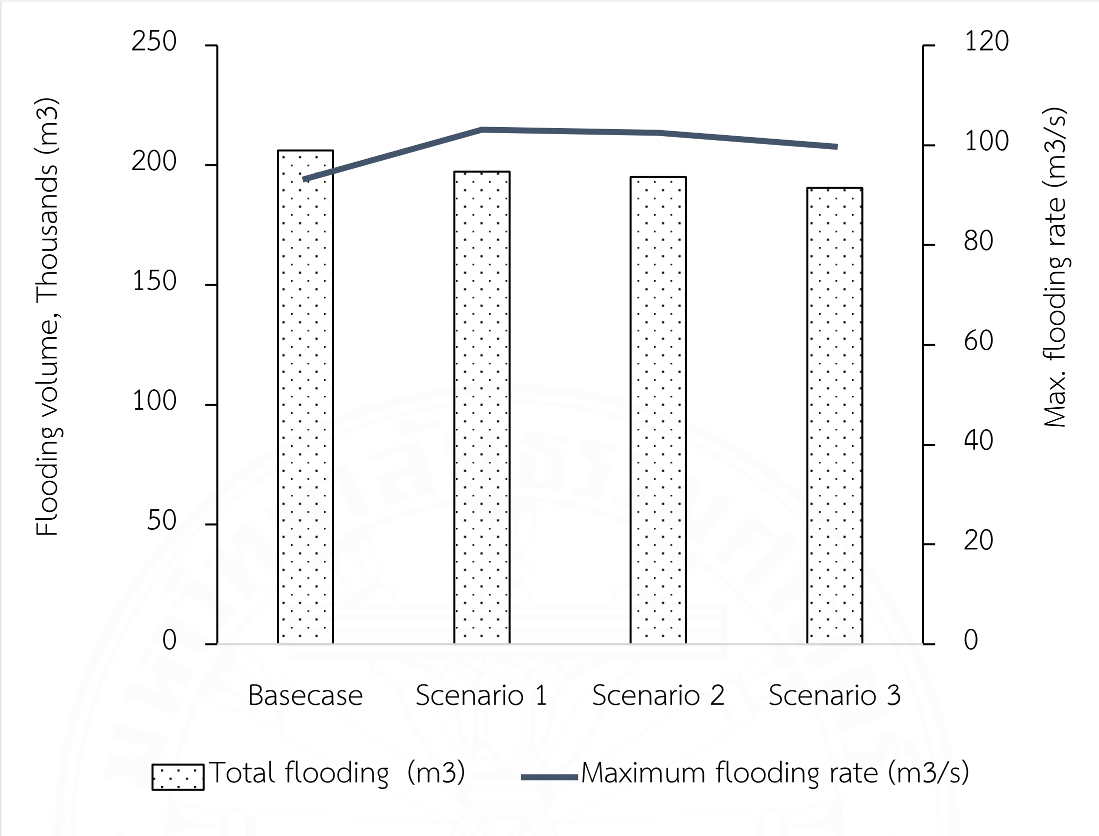

Figure 4.14 Comparison between total flooding and peak flooding rate between scenarios

Table 4.5 Performance of WSUD connectivity

Control	Baseline	Scenario 1	Scenario 2	Scenario 3
Impact of WSUD on the entire Nava Nakorn				
Total runoff reduction rate (%)	495,400 m3	12.5	13	15.8
Total flood reduction rate (%)	206,100 m3	4.2	5.4	7.7
Max. flooding rate (m3/s)	93.25	101.3	100.6	99
Impact of WSUD on localized flooding				
Runoff reduction varied by subcatchment (%)	-	-15 – 100	-10 - 100	-8 - 100
Flooding volume reduced by junction (%)	-	Up to 48.7	Up to 100	Up to 76.7
Average flood duration (hour)	1.7	1.67	1.67	1.6

4.7 Cost Estimation

Cost estimation were made on the different design scenarios under the WSUD connectivity to understand better with regard to cost and benefit design relationship. In this study, the estimation was made based on past studies conducted in Thailand, literature review of local contacting company reports and internet searches. The possible costs of each design are illustrated in **Table 4.6**.

Table 4.6 Cost estimation for combined WSUD design scenarios

WSUD system	Unit cost (baht/m^2 or baht/m^3)	Total cost (baht)		
		Scenario 1	Scenario 2	Scenario 3
Bioretention cell	947 - 1,015*	33,614,800	34,537,600	36,039,600
Bioswale	300***	7,950,000	15,900,000	7,950,000
Raingarden	955 - 1,015*	17,199,950	17,609,000	18,273,600
Detention pond	640**	1,673,600	1,673,600	1,209,600
Total costs		60,438,350	69,720,200	63,472,800

* Estimated based on Chaosakul et al. (2013); ** Estimated based on EPA estimates report; *** Auhtor estimate

CHAPTER 5
CONCLUSION AND RECOMMENDATION

5.1 Conclusion

In this study, we explored the design and benefits of implementing bioretention cell, bioswale, raingarden and detention pond to illustrate the benefits of WSUD technology with respect to reducing runoff and flooding volume, flood duration and its severity. Each feature was given different design criteria to test for its performance and effectiveness in controlling these problems. A range of scenarios produced for individual features giving the possible implemented design options cover on three of the major aspects include conservative design with minor benefit, aggressive design that would maximize the benefit but higher cost, and the optimized system design. Results of each scenario by different features were then evaluated and categorized based on its performance in regard with flooding problem improvement.

Runoff was estimated using PCSWMM for Nava Nakorn under a 2-year design storm rainfall to examine total runoff volume, surface flooding volume, maximum surface runoff rate, and surface flooding duration. The simulation result showed that 72 mm of rain would produce 495,400 m^3 runoff volume for the entire area of Nava Nakorn whereas total flooding volume was estimated at 206,100 m^3 with the average flood duration of 1.7 hour. To put this flood volume in perspective, it would be equivalent to 2 cm of flood spread all over the area of 6,500 rai (33.3ha). PCSWMM allows for detailed exploration of flooding for the localized areas. Severity level of flood differed between the areas and this would be due to different characteristics of the area itself that could influence runoff overflow rate, runoff coefficient, and infiltration loss. As discussed, the existing rainfall runoff drainage system using canals and ponds was built to prevent possible flooding. Furthermore, as of 2019, more land plots were sold out for development and this results in a majority of the area of Nava Nakorn (~ 85%) to have impervious surfaces. It is understandable that the likelihood of surface pluvial flood tends to increase with more intense rainfall events and urbanization making cities as well as real estate companies in Thailand become susceptible to flooding (Vojinovic et al., 2015; Laeni et al., 2019). As for Nava Nakorn, existing flood prevention system from rainfall showed to be ineffective to manage runoff, therefore many areas suffered from localized flooding.

Simulation results of scenario development showed that applying WSUD technology would result in reducing flooding volume. Performance of WSUD tends to be lower for the entire system as compared to the local area. Therefore, the focus on individual "high impact" catchments may be important in prioritizing the spatial implementation of WSUD features to maximum flood reduction benefits.

The design of bioretention cells could reduce surface (pluvial) flooding volume up to 3.2 % while the total runoff reduction was up to 12.7% systemwide (with a range from 6-78% reduction for individual catchments). Meanwhile, flooding volume for the individual junctions improved for most of the area with bioretention cell. For example, the Nava Nakon main road (section A-A'), Nava Nakorn light and heavy transportation roads (section B-B') and (section D-D') would have volume reduction up to 100 % and flood duration reduction by about 0.25 hours in average for all scenarios.

All design scenarios under bioswale reduced runoff volume by between 0.7 – 3% for the entire Nava Nakorn with the maximum reduction ratio of 11.4 – 80 % for individual catchments. Performance of bioswale on flood reduction benefit however was not so clear for a major part of the area with this control such as Nava Nakorn section A-A' and section C-C' factory and industry area. The maximum flood volume reduction was 2.9 – 11 % at individual junctions for all scenarios and those located mostly at the residential complex, section B-B' while the maximum reduction of flood duration at individual junctions was between 0.5 – 0.8 hour.

With proper design, raingardens may be an effective approach to control the flooding problem. From the result, a small proportion of raingarden (0.2 % of the study area) reduced flooding volume up to 1.5% systemwide with peak flooding control of 1.67 m^3/s and flood duration control of 0.02 hour in average. On the other hand, runoff volume reduced from the entire study area was in a range of 1.4 – 1.7 % for scenario 1 – 3, however the control was much greater for the high impacted catchments for example raingarden design in scenario 2 for which the volume reduction was from 79 – 93 %.

Under the detention pond control, flood associated problems such as volume, duration and peak flooding rate were fully managed at the critical upstream node (represented as junction OF4 in the model) from the overall flooding volume of 458 m^3 with the highest peak flow rate of 0.3 m^3/s and flood duration of 0.97 hour. Sizing the pond however should be carefully evaluated to prevent flooding to be more severe for the upstream connected junctions; scenario 1, for example, created worse backwater effect for the upper junction (double the flooding volume).

Combining all WSUD features to be implemented on the study area would leverage flood reduction benefit to a better level. From the simulation result, with all features being placed at once, flood volume was reduced by up to 7.5% for an optimal design scenario while flooding associated with individual junctions was reduced by up to 5.7 hour. Flooding by individual junctions improved significantly with the WSUD connectivity, specifically, a majority of flooded junctions that were not controlled by the individual design, for example bioswale.

5.2 Limitation and Recommendation

Result from the simulation under the baseline scenario indicated runoff and flooding level varied between junctions, especially for some junctions flooding had a greater duration despite the smaller catchment area being connected to. Sensitivity analysis with respect to the size of the subcatchment, % impervious area, and width of subcatchment should be further addressed in future work as they would be the major factors contributing to this issue. In addition, performance of all scenarios produced under WSUD technology such as bioretention cell and bioswale were not so clear with respect to surface (pluvial) flooding control, in particular, peak flooding rate as with the 2-year storm (72 mm) runoff was abundantly exceeded the maximum storage capacity of the system. It is possible that scaling the system up would help to manage this problem in the future (as this could be resulting from the undersized design) or we could focus on controlling peak flooding for a smaller storm events, for example 30 mm rainfall event. Further study on detention pond sizing or decentralised set of ponds would enhance performance of the design to improve flooding in a higher level.

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