A MATHEMATICAL MODELLING OF STORMWATER DRAINAGE OF THE CATCHMENT OF BOALIA KHAL INCLUDING LOW IMPACT DEVELOPMENT MEASURES

by

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CERTIFICATION OF APPROVAL

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ABSTRACT

Dhaka city is one of the fastest growing megacities in the world and is facing a massive challenge to deal with rainfall-induced flooding. Severe storm events due to climate change and landuse changes are playing a key role in the acceleration of urban flooding. The main drivers of rainfall-induced flooding are fast & unplanned urbanization and incapacity of stormwater drainage systems to manage the peak runoff volume. The catchment of Boalia khal in the northeastern part of Dhaka city is one of those areas which face frequent flooding induced by intense rainfall during the pre-monsoon and monsoon periods. This study aims to obtain an optimum combination of Low Impact Development (LID) measures for stormwater management in the Boalia khal catchment area.

This research focuses on investigating the influence of different individual LID (Rainwater Harvesting (RWH), Soak-Away Pits (SAPs), Porous Pavement (PP), Wetland Basin (WB)) and a combination of LID measures for the proposed drainage system to reduce the runoff volume, flood extent and pumping duration in the Boalia khal catchment area. A 1D-2D coupled drainage model was developed using the DHI MIKE URBAN interface.

The design rainfalls were distributed using the Alternating Block Method. The drainage system of Dhaka city has been designed to handle a 1-day, 5-year rainfall event, while the pumps have been designed to cope with a 2-day, 5-year rainfall event. Consequently, the current systems are ill-equipped to deal with rainfall events that lead to urban floods, such as 1-day 10-year and 25-year occurrences. To assess the effectiveness of LID (Low Impact Development), this study conducted an analysis of various scenarios considering these different rainfall events.

The peak water level is reduced by 8.4 cm at upstream of Boalia Khal and by 19.2 cm at downstream for a 25-year rainfall event after the application of low impact development measures. The combined (RWH-SAPs, PP and WB together) application of LID measures decreased the flood-affected area by 14% for a 10-year storm event and 15% for a 25-year storm event compared to without LID. Integrating RWH-SAPs and PP in the future drainage system may reduce pumping duration by up to 14%, equivalent to 3.0 hours. The addition of Wetland Basins alongside these LID measures can lead to a more substantial reduction of pumping duration up to 53%, i.e. 10 hours

for a 10-year storm event. Thus, the application of porous pavement is effective for the reduction of flood extent and WBs are more effective for the reduction of pumping duration along with flood extent. To get optimum performance of wetland basins the invert level of the inlet control structure was set below the pump start level.

It was found that climate change could lead to increased flood extents and longer pumping durations. However, the application of LID strategies mitigated these impacts, reducing flood extents and pumping durations under climate change scenarios. LID measures reduced the flooded area by 20% and average pumping duration by 43% for a 100-year storm event.

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List of Acronyms and Abbreviations

A1FI	IPPC scenario with future based on fossil fuel intensive development
ABM	Alternating Block Method
ASCE	American Society of Civil Engineers
Avg.WL	Average Water Level
BMD	Bangladesh Meteorological Department
BTM	Bangladesh Transverse Mercator projection
BMP	Best Management Practice
BUET	Bangladesh University of Engineering and Technology
BWDB	Bangladesh Water Development Board
CC	Climate Change
DAP	Detailed Area Plan
DC	Drainage Compartment
DCC	Dhaka City Corporation
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute
DNCC	Dhaka North City Corporation
DND	Dhaka Narayanganj Demra
DSA	Demand Side Approach
DWASA	Dhaka Water Supply and Sewerage Authority
FAP	Flood Action Plan
EV	Extreme Value
GPS	Global Positioning System
GIS	Geographic information system
IL	Invert Level
HD	Hydrodynamic
HWL	High Water Level
IDF	Intensity duration frequency
IMD	Indian Meteorological Department
IWM	Institute of Water Modelling
JICA	Japan International Cooperation Agency
LID	Low Impact Development
LN	Log-Normal

LWL	Low Water Level
MGC	Maximum Ground Coverage
NCRM	North Central Regional Model
NAM	Nedbør-Afstrømnings-Model (precipitation-runoff-model)
NSE	Nash-Sutcliffe efficiency
NWRD	National Water Resources Database
O&M	Operation and Maintenance
PAM	Permeable Asphalt Mixture
PP	Porous Pavement
PWD	Public Works Department
Q	Discharge
RAJUK	Rajdhani Unnayan Kartripakkha
RWH	Rainwater Harvesting
SC	Scenario
SPAP	Surface-drainage Permeable Asphalt Pavement
SAP	Soak-Away Pit
TA	Time Area
TBM	Temporary Bench Mark
T_c	Time of Concentration
TIV	Target Infiltration Volume
UTM	Universal Transverse Mercator
WB	Wetland Basin
WRE	Water Resources Engineering
WL	Water Level
WARPO	Water Resources Planning Organization

Measurement Units & Symbols

%	Percent
^{o}C	Degrees Celsius
cumec	Cubic Meters Per Second
K	Kelvin
km	Kilometer
Km^2	Square Kilometer
knots	One Nautical Mile (1.852 Km) Per Hour
lpcd	Litters Per Capita Per Day
m	Meter
m^3	Cubic Meter
m^3/s	Cubic Meter Per Second
mPWD	Meter PWD
ML	Mega Liter, Million Liters
mm	Millimeter
Mm^3	Million Cubic Meter
MLD	Million Liters per Day Equivalent to 1000 Cubic Meters Per Day

Chapter 1

Introduction

1.1 Background and Research Motivation

Urban flooding is a serious problem to deal with in rapidly growing cities. Due to the rapid growth in demography and urbanization and the frequent occurrence of severe storm events due to climate change, the impacts of urban flooding are accelerating(Jha et. al., 2012). No exception, Dhaka city, one of the fastest growing megacities in the world is facing a massive challenge to deal with rainfall-induced flooding. Dhaka city has been experiencing such flooding problems for the past few years (Ahmed et. al., 2018). It is a regular phenomenon in Dhaka preferably during monsoon (June to September). Fast and unplanned urbanization coupled with the incapacity of the storm water drainage system to manage the peak runoff volume, are the main drivers to occur rainfall-induced flooding in built-up areas on various scales (Mark et. al., 2001; Kabir and Parolin, 2012) This problem becomes severe when there is a higher water stage in the surrounding rivers (Huq and Alam, 2003). Furthermore, the ever-increasing population (expected to be around 23 million by 2025) along with the decrease of wetland at an alarming rate (about 32.57% decrease of water bodies and 52.58% decrease of lowlands from 1960 to 2008) have made the situation more complex (Islam et. al., 2010)

The catchment area of Boalia khal in the north-eastern part of Dhaka city is one of those areas which face frequent flooding induced by intense rainfall during the pre-monsoon and monsoon periods. Part of the catchment area is still undeveloped, however, recent trends of rapid unplanned development activities by filling the wetlands and canals are alarming. Such unplanned development activities have decreased the natural wet detention areas significantly. This makes the study area highly vulnerable towards extreme flooding in future.

A future landuse plan for the Boalia khal catchment and surrounding areas has been prepared by RAJUK and DNCC. To protect the areas from River floods, an embankment will be built along the right bank of the Balu River. As a result, pumping facilities have been proposed for mitigating rainfall-induced flooding, particularly during monsoon. These infrastructures have primarily been designed through 1D hydrodynamic modelling (Bird et. al., 2018). A recent study on this area consisted of an outline drainage design without considering any future development or embankment and pumping facilities (JICA, 1992; Halcrow, 2006). Moreover, recently RAJUK has conducted a re-design of the Boalia khal and other khals considering a free flow condition at the outlet without assessing pumping requirements.

None of those studies has made any attempt to include Low Impact Development (LID) measures such as wetland basins, green rooftops, permeable or porous pavements, rain gardening, rooftop rainwater harvesting, Soak-Away Pits etc. to introduce sustainable management of stormwater (Chapman et. al., 2008; Salleh and Taherm 2012; Bhandari et. al., 2018). Also, most of the previous research on the drainage system of Dhaka was focused mainly on flood extent for existing drainage conditions with a limited study on Low Impact Development (LID) measures to manage urban stormwater (Bari and Efroymson, 2009; Mowla and Islam, 2013; Feroz Islam, 2014) There is a clear research gap regarding the applicability of LID measures in Dhaka city as well as in the Boalia khal catchment area for efficient management of stormwater runoff.

This research, therefore, focuses on investigating the influence of different individual and combined LID measures to reduce the runoff volume and thereby the extent of urban flooding in the Boalia khal catchment area. A 1D-2D coupled drainage model built on the interface of DHI MIKE URBAN has been used in this research. The research also provides an assessment of the pumping requirement in the study area during monsoon to mitigate rainfall-induced urban flooding.

1.2 Fundamental Idea

The term "urban flooding" refers to inundation caused by heavy precipitation events that cause runoff to exceed the capacity of the local drainage system. As a result, water stagnates in the built-up area for extended periods. This type of flooding is also referred to as rain flooding.

A hydrological event's return period (for example, rainfall or river flow) is an estimate of the time interval between two events of the same size or intensity. It is the inverse of the probability that the event will be exceeded in any given year and is a statistical measure of the average recurrence interval over a long period. As a result, 10-year, 20-year, and 30-year rainfall amounts have a 10%, 5%, and 3.3 percent chance of occurring in any given year respectively. However, this does not imply that if an event with a 10-

year return period occurs, the next one will happen in about 10 years; rather, it means that there is a 10% chance that it will happen in any given year, regardless of when the last similar event occurred. Or, to put it another way, it is ten times more likely to occur than a flood with a 100-year return period (or a probability of 1 percent). The return period chosen for design purposes is determined by several factors, including the size of the drainage area, the risk of failure, the importance of structures at risk, and the desired level of conservatism by decision-makers and stakeholders. Shorter return periods are typically based on observed data (where available), whereas longer periods are estimated using statistical distribution.

For areas protected from river flooding, the return period of an urban flooding event is similar to the return period of the rainfall event that caused the inundation. However, in river flood-prone urban areas, the return period of urban flooding is determined not only by the return period of the rainfall event but also by the return period of the river stage (water level). In other words, the return period of urban flooding in unprotected areas is a probability function that is related to the frequency of rainfall events and high river stages. Other factors that influence the return period of urban flooding events include land cover changes, the condition of drainage channels, and so on. However, assigning probability distributions to these factors is difficult, and in hydrologic analyses (modelling), these factors are typically kept constant for a given scenario.

In an urban drainage study with the application LID, the LID design guidance is developed to provide the engineering solution for the reduction of stormwater runoff with additional guidance for the design of wetlands and pervious pavements. LID is a stormwater management strategy that focuses on preserving or restoring a site's natural hydraulic functions. In contrast to collecting stormwater in a piped or channelized network and managing it at a large–scale “end of pipe” location, LID employs a decentralized approach that disperses flows and manages runoff closer to where it originates. This management practice focuses on simulating the natural retention, filtration, and infiltration mechanisms encountered by storm water runoff in an undeveloped area. As a result, the preservation of native vegetation and natural drainage features is the most important factor to consider when applying LID to site design.

1.3 Objective of Study

The aim of this study is to obtain an optimum combination of LID measures for stormwater management in the study area i.e. Boalia khal catchment area. To achieve this goal, the specific objectives of the study are as follows:

i. To develop, calibrate, and validate a 1D-2D coupled drainage model by using MIKE URBAN with measured water level and discharge at Boalia khal.

ii. To generate flood maps for present and future landuse and drainage networks under different rainfall and river flood conditions and investigate the impact of the embankment and pumping facility on the present and future drainage conditions.

iii. To simulate drainage conditions for future landuse including different LID measures special emphasis on inlet-outlet design of wetland basin and compare their effectiveness to formulate an optimum stormwater drainage management plan of the study area.

1.4 Outcome of Study

The following outcomes are expected after this study:

1. A 1D-2D coupled MIKE URBAN model of the study area, calibrated and validated by measured runoff.

2. A drainage management plan for the study area including different LID measures.

3. A systematic understanding of the suitability of different LID measures for the reduction of drainage congestion for densely populated areas like Dhaka city.

Chapter 2

Literature and Past Study Review

2.1 General

Before initiating any research work for a particular study area, a past study review is considered essential to know the different plans in that area and generate an idea of how to assimilate the research with those plans. Without a past study review, it is not possible to determine what work has already been accomplished regarding the particular research question.

2.2 Review of Relevant Past studies

In addition, there are several technical study reports and master plans prepared since the 1990s which focus on stormwater management and mitigation of rainfall-induced flooding in the entire eastern Dhaka area. Most of the studies estimated flood extent only under existing drainage conditions. The adequacy of the proposed internal drainage network was not analyzed. No investigation of alternative solutions e.g how to reduce the runoff and manage urban stormwater. A review of those studies is provided in the following sub-sections.

2.2.1 Feasibility Study on Greater Dhaka Protection Project (Study in Dhaka Metropolitan Area) of Bangladesh Flood Action Plan-FAP 8A, (1992):

After the devastating flood of 1988, a comprehensive study in the name of the Flood Action Plan (FAP) was launched. The objectives of FAP 8A were to formulate a strategy for flood mitigation and stormwater drainage improvement for priority areas (194.04 km^2) of Greater Dhaka East, Narayanganj DND and Narayanganj West. Under FAP 8A, the Greater Dhaka East was divided into four drainage compartments (DC) (**Figure 2-1**), namely DC1 (Uttarkhan and Dakshinkhan), DC2, DC3 and DC4. The study area i.e. areas of the Boalia khal catchment is under compartment DC1. An embankment along the Balu River was designed based on the highest flood levels at Tongi (8.60 mPWD) and Demra (7.40 mPWD). In order to minimize the flood damage of the DC1, a pumping station (P5) was proposed combined with retarding ponds. The required pump capacity was calculated 25.2 m^3/s and the retarding pond area is 265 summarized in **Table 2:**1 the pump drainage and associated retarding pond capacities were determined using a 2-day consecutive rainfall with a 5-year return period. The design water levels of retarding areas were set at 3.0 m PWD (LWL) and 4.0 m PWD (HWL).

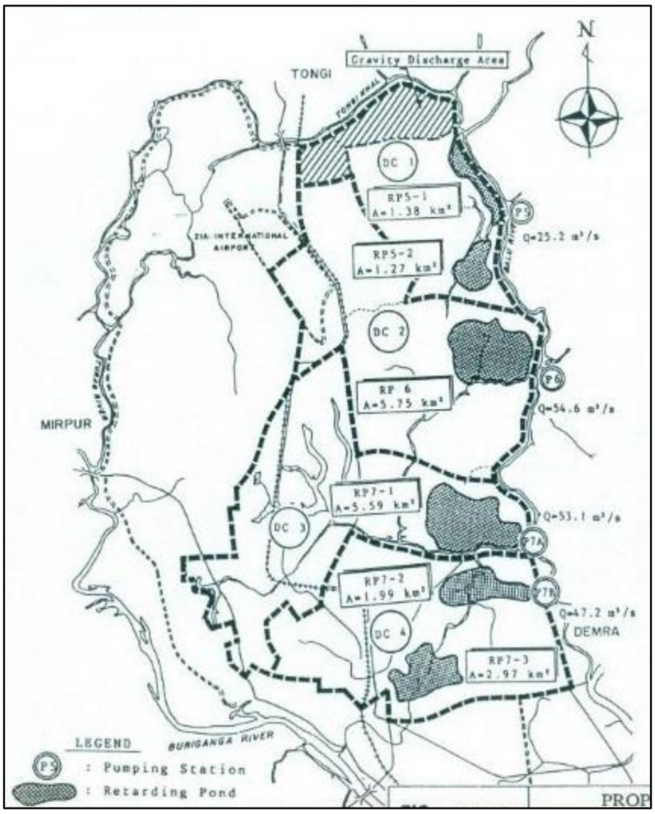


Figure 2-1: Compartment of Greater Dhaka East of FAP 8A

Table 2:1: Recommended Retarding Area and Pump Capacity by FAP 8A

Zone	**Drainage Area (Km^2)**	**Retarding Area (ha)**	**Pump Capacity (m^3/s)**
DC-1	30.56 (22.11 pump drain+ 8.45 gravity drain)	265	25.2

2.2.2 Dhaka Flood Protection Project, Bangladesh [2]

This study was conducted from January to August 1995 and covered the western fringe areas of Dhaka City which stretches from Kellar Morh to ShirnirTek The whole study area was divided into three units viz. (1) Hazanbagh (Kellar Morh to Hazaribagh) Unit, (2) Mohammadpur (Satmasjid Road to Kallyanpur Canal) Unit, and (3) Mirpur (Mirpur Bridge to Shirnir Tek) Unit (**Figure 2-2**). Each unit was again divided into two sides i.e., inside and outside of the embankment. Thus, a total of six areas were included in this study (three from inside and three from outside). The reason behind selecting these areas is that all of them were highly flood prone before construction of DFPP. Based on the displacement of the people by the construction of DFPP and its effects on various socio-economic and environmental aspects, the settlers were classified into four groups, i.e., (1) settlers who have been living inside the embankment since before its construction, (2) the people who migrated to the protected inside area after the project, (3) people who have been living outside since before the construction of the embankment, and (4) the new migrants to the outside area after the project. All the settlers of the above mentioned groups were considered as target groups of the study as they all are affected by the construction of the embankment. Total number of respondents was 90 of which 54 were selected from the inner part and the rest 36 were from the outer part of DFPP. The techniques used for this research were a reconnaissance survey, field observation, questionnaire survey and laboratory test. A total of 15 hypotheses were set to detect the effects. The data were analysed and the hypotheses were tested through various statistical tools such as T-test, Z-test, Chi-square test and simple statistics like frequencies, percentage, mean etc.

Effects on Some Environmental Aspects

In the inner part of the embankment, both point and non-point sources of domestic wastes and industrial wastes are generated. Point source domestic and industrial wastes are disposed of into the sewer lines whereas the non-point source wastes generated from enormous number of slums are disposed of into the open low-lying water bodies. Eventually, these wastes are carried out by surface run-off to the low-lying areas just inside the embankment. Before the construction of the embankment, these wastes could be disposed of directly to the nearby rivers through many disposal points. In this case, the river flow could use its assimilative capacity to purify the wastes and could carry the wastes to the downstream location. But, after the construction of DFPP, there are 10 sluices starting from Kellar Morh to Tongi Railway Bridge for disposing the wastes to the rivers. It takes two to four days to dispose the wastes (sohd waste mixed with liquid run water accumulated in the low-lying ponds) to the rivers. This character of waste disposal system has made the stagnant low-lying water bodies as 'low-lying mixed (waste and water) waste bodies' which has been worsening by the disposal of human excreta of the slum dwellers who are approximately 25 % to 30% (Centre for Urban Studies, 1988) of the city population.

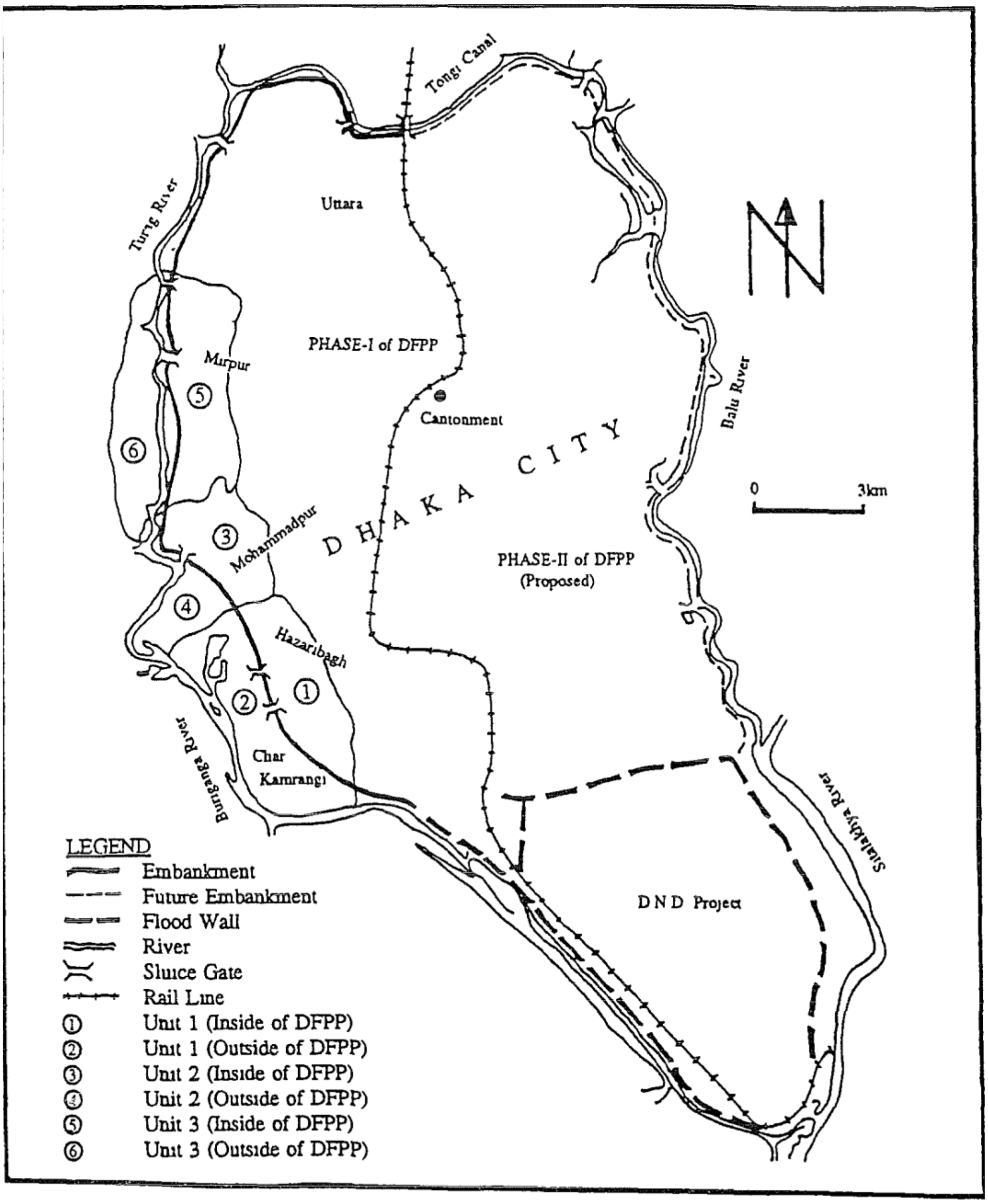


Figure 2-2: Study Area under Dhaka Flood Protection Project, Bangladesh

The human excreta comes in contact with the stagnant water and remains there for a couple of days until being pumped out. This is causing surface water pollution of ponds and lagoons. Then through infiltration, it is polluting the ground water. People in the adjoining areas are using water from the stagnant water bodies for their daily use as they are not served by the water supply of Dhaka Water and Sewerage Authority. In all of the three study areas of the outer part of DFPP, no waste disposal system is provided by Dhaka City Corporation or any other organization. People in these places use service latrines as well as septic tanks from which the wastes are accumulated in low-lying areas. People residing beside the rivers dispose off their solid wastes directly to the rivers. The investigation reveals that about one-fourth insiders assured that they were getting waste disposal service while three-fourths answered negatively. For the outsiders, 100 per cent claimed that they did not get any waste disposal service from the government. For the total respondents, only 14.4 per cent answered positively while 85.6 per cent respondents answered negatively. In brief, the construction of DFPP has caused severe adverse effect on waste disposal system in its fringe areas leading to create other related environmental problems.

2.2.2 Updating/Upgrading the Feasibility Study of Dhaka Integrated Flood Control Embankment cum Eastern Bypass Road Multipurpose Project, Halcrow (2006):

A feasibility study of Eastern Bypass was launched by Halcrow (under BWDB) in 2006. The study also adopted 2-day of consecutive rainfall with a 5-year return period for hydrological analysis, however with the presumption that **80%** area would be urbanized in future. The study divided the eastern Dhaka area into three compartments instead four. Out of the three compartments, Compartment -1 covers the study area.

The main objective of study was to offer flood protection at the Eastern Bypass area from the Balu River and improve internal drainage congestion. The key outputs of the study were i) Embankments crest level; ii) Flood maps for different scenarios; iii) Flood control and drainage structure size and sill level and iv) The capacity of pump stations and their locations. Two separate options were examined for the determination of pump capacity and these are:

Table 2:2: Determination of pump capacity for Compartment-1

Option	Pump Capacity (m^3/s)	Retention area (ha)	Max WL in Retention pond (mPWD)	Operation time of pump for design rainfall	Pump start level (mPWD)	Pump stop level (mPWD)
Option 1	40	230	3.54	93	2.5	2.0
Option 2	70	230	2.85	36	2.5	2.0

2.2.3 Detailed Area Plan (2010):

A detailed area plan prepared by RAJUK in 2010 presented a landuse plan in the Eastern Bypass project area considering the flood control and drainage facilities as proposed by the Halcrow study in 2006. The plan presented a map of the proposed water retention areas, however, some of those water bodies are already been filled up shown in **Figure 2-3**.

Figure 2-3: Map Showing the Filling of Water Retention Area

The study area falls within location-6 and location-9 as per the DAP, 2010. These two locations include parts of Bhatara, Badda, Sutibhoila mouzas and Bar Kathaldia mouza and small parts of DCC wards No. 17, 18 and 21. As evident from **Figure 2-4**, the existing landuse is dominated by residential areas (about 39%), cultivated land/open space/forest (about 33.5%) and swamp/marsh/char/island/water bodies (22%). However, the amount of open space/forest and swamp/water bodies have been reduced significantly while delineating the future landuse (**Figure 2-5**). The proposed landuse is mostly dominated by residential area (about 58%), proposed road network (about 9%), mixed-use zone (about 6%). The amount of open space has been proposed to reduce by only 1.4% of the area whereas water bodies have been proposed to reduce to about 11%.

Even though water bodies in the future landuse have been proposed to reduce, however, the most important element of the DAP is delineating a zone as water retention area. These proposed water retention areas will be critical in the future for the efficient management of stormwater drainage in the study area.

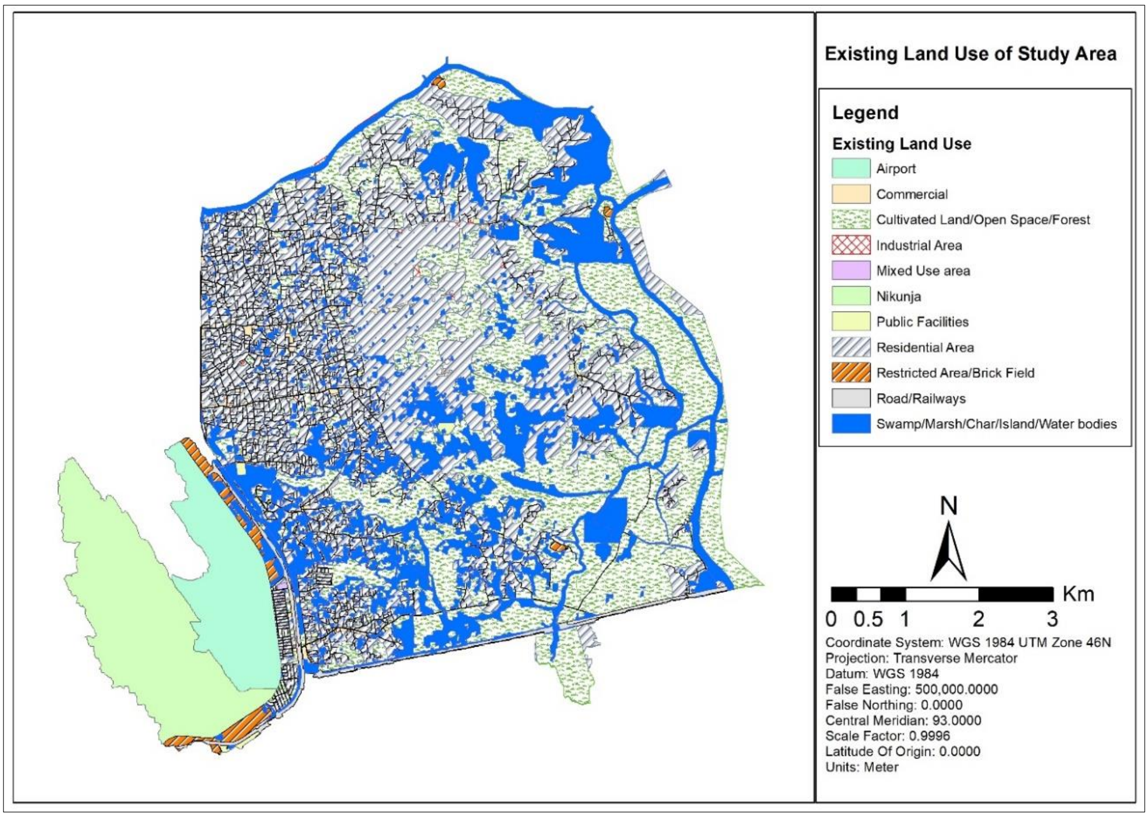


Figure 2-4: Existing landuse of the study area

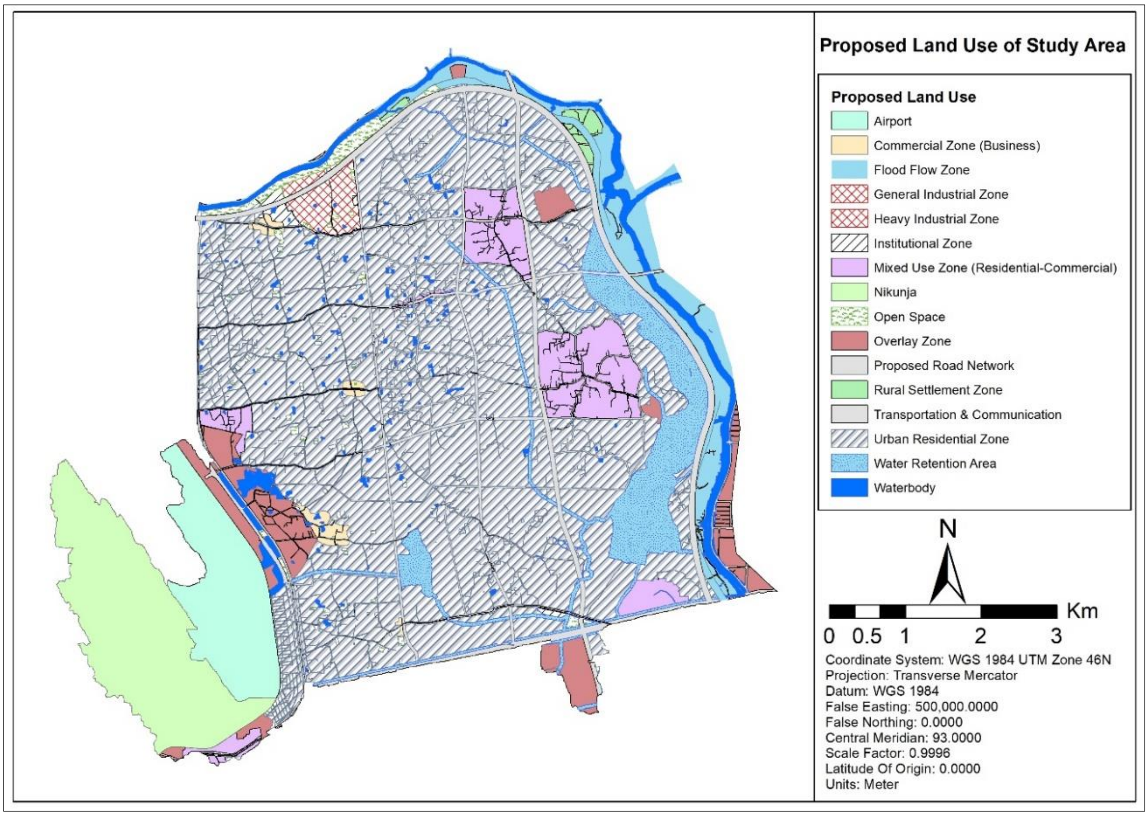


Figure 2-5: Future Landuse of the study area (DAP, 2010)

2.2.4 Urban Flooding of Greater Dhaka in a Changing Climate: Vulnerability, Adaptation and Potential Cost by World Bank (2014) with support from IWM

The detailed study covered 345 km^2 of Dhaka metropolitan area including 121 km^2 area of Eastern Dhaka which also covers the catchment area of Boalia khal. This study was conducted considering both existing and DAP future landuse. The overall aim of this hydrologic study was to assess the impact of climate change on location-specific waterlogging for various scenarios.

The drainage model of Eastern Dhaka area was simulated for different storm events with and without climate change impacts. For future scenarios (2025 and 2050), the future land of DAP was used for computation of impervious area and runoff volume. The proposed water retention areas as delineated by DAP were included in model as well for simulating future scenarios. Also, as planned improvements, all the improvement work (khal, retention pond, regulator and pump stations) proposed under the Eastern Dhaka Bypass project were considered. Of particular importance with respect to the Boalia khal catchment, the retention area adjacent to Babur khal proposed by DAP was considered along with a pump station of capacity 70 m^3/sec.

As dry weather flow, 100% generated wastewater was provided as an inflow in the base scenario, however, for future scenarios this was not included. DWASA's plan to implement a sewerage system all over Dhaka city by 2035 provided the basis for this assumption.

By analyzing the flood extent and flood duration for future scenarios, it was found that the planned drainage system could cope with a 100-year return period event. It was seen that event of climate change impacts, no additional measures will be required (**Figure 2-6**). This study proved that preserving the DAP proposed water retention areas in eastern Dhaka are particularly important to minimize rainfall-induced flooding in the future and tackle climate change impacts.

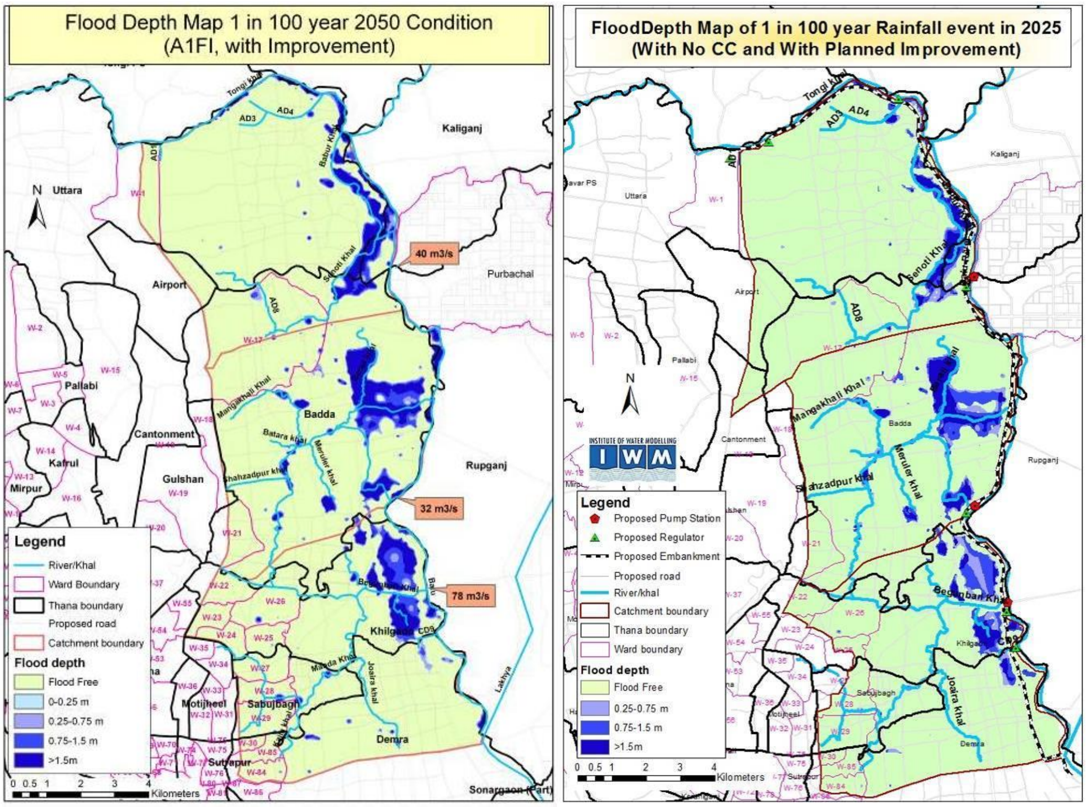


Figure 2-6: Flood Depth Map with and without Climate Change

2.2.5 Stormwater Drainage Master Plan for Dhaka City- Final Report (2016):

The study covered the total Dhaka Metropolitan area (about 1500 km^2) and presented its development plan dividing the study area into 13 zones. Among these zones, the zone UTK (Uttarkhan) covers the Boalia khal catchment area. The study utilized the findings of Halcrow, 2006 in respect of drainage and flood control provisions. However, the study proposed the construction of an elevated road using the embankment crest or its berm. Some of the recommendations on structural measures are:

- Along the Kuril- Purbachal connecting road (on both sides of the road), 100 feet wide-open channel or canal need to divert a huge amount of flow to the Balu River.
- Set up minimum plinth level (9.0 mPWD) and Platform Level (8.0 mPWD) for any structural development.
- Instead of the construction of the Eastern Bypass Road, the designated authority (BWDB) may consider an elevated transport route option to assure undisturbed natural conveyance of stormwater and inundation.

The drainage master plan also provided a guideline for the design of various drainage items. The recommended return period for the design of different drainage infrastructures is presented in **Table 2:3**.

Table 2:3: Design Average Recurrence Interval (ARI) for Storm Runoff Flow Estimation

Sl. No.	Drainage Item	Return Period (Year)
1	Property Drainage (for individual buildings), Inlets and Roadside Drains, Tertiary Drains, Saucer Drains, etc.	2
2	On-site Detention, Secondary Drains, Pump Stations	5
3	Primary Drains	10
4	Khals (Canals), Urban Rivers, Pond, Lake and Water Detention/Retention Areas.	20
5	Flood Flow Zones, Embankments, Emergency Spillways and Bridges	50

2.2.6 Technical Study of Flood Control and Drainage Development at Dhaka Circular Road Project, January 2017

This study covered a 121 km^2 area of Eastern Dhaka including 44 km^2 of Boalia khal catchment. The main objective of the study was to suggest appropriate flood control and drainage facilities for the project area. The option of alternate arrangements/options of reviving the drainage channels to increase retention capacity within the existing drainage channels in conjunction with smaller pumping facilities at the bridge-cum-regulators was reviewed under the study, so that huge land acquisition for retention ponds can be avoided. To achieve the objective, a hydrologic model (NAM) and a hydraulic model were set up using MIKE 11. The drainage improvement plan considered for model simulation is shown in **Figure 2-7**.

Different scenarios were simulated to estimate the minimum retention pond and pump capacity required to drain out the excess runoff generated by a 2-day 5-year return period rainfall event. The optimized pump capacity, pump operation level and pumping duration are provided in **Table 2:4**.

Table 2:4: Pump capacity, maximum water level and duration of pumping for optimizing Scenario

Khal Name	Pump discharge (m^3/s)	Maximum WL (mPWD)	Duration (hr)
Babur Khal (P5)	34	4.49	16+6
Boalia khal (P4)	42	4.49	16+6

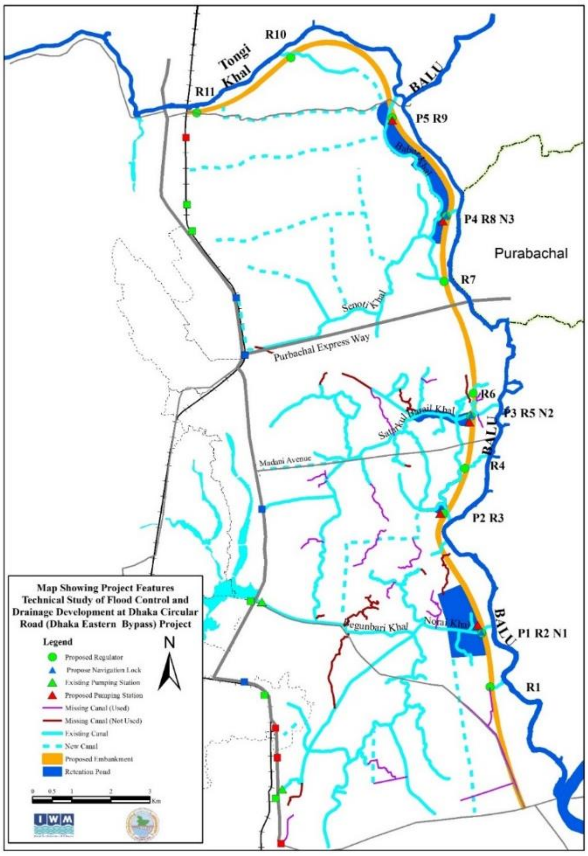


Figure 2-7: Embankment, Pump House and Regulator Location Map

2.2.7 Low Impact Development Design Guidance Manual" by USKH Inc., Water Resources Group (2008), Alaska, USA.

As urbanization continues to increase, the management of stormwater runoff becomes a critical issue. Low Impact Development (LID) techniques offer sustainable solutions to mitigate the adverse impacts of stormwater runoff on the environment and traditional drainage systems. The "Low Impact Development Design Guidance Manual" by USKH Inc., Water Resources Group (2008), Alaska, USA has presented the key LID techniques: Soak–Away Pits, Pervious Pavements and Constructed Wetlands.

Soak–Away Pits:

The manual discusses Soak–Away Pits as an essential LID technique for stormwater management. These pits, also known as infiltration pits or dry wells, facilitate the infiltration of stormwater runoff into the underlying soil. By simulating natural hydrological processes, Soak–Away Pits reduce surface runoff, recharge groundwater, and alleviate pressure on traditional stormwater infrastructure. The manual provides guidance on selecting appropriate locations, conducting soil testing, sizing the pits, and ensuring regular maintenance for optimal performance.

Pervious Pavements:

USKH Inc.'s, 2008 manual emphasizes Pervious Pavements as another effective LID technique. Pervious pavements are designed to allow water to pass through their surface, promoting infiltration and reducing runoff. The manual explores various types of pervious pavements, such as porous asphalt and permeable concrete, and their applications in different land uses. It discusses design considerations, including structural support, aggregate size, and proper maintenance to ensure the longevity and effectiveness of pervious pavements.

Constructed Wetlands:

The "Low Impact Development Design Guidance Manual" introduces Constructed Wetlands as a nature-based LID technique for stormwater management. These wetlands are designed to treat and retain stormwater runoff, enhancing water quality and providing habitat for wildlife. The manual outlines the key design principles for constructed wetlands, including sizing, vegetation selection, and hydraulic retention time. It also highlights their potential role in flood control and groundwater recharge.

Throughout the manual, case studies showcase successful implementations of Soak–Away Pits, Pervious Pavements and Constructed Wetlands in Alaska. These real-world examples demonstrate the practical application and positive outcomes of using LID techniques to address stormwater runoff challenges in urban areas. The case studies also offer insights into potential challenges and lessons learned from these LID projects.

The "Low Impact Development Design Guidance Manual" by USKH Inc., Water Resources Group (2008), provides a comprehensive overview of three essential LID techniques: Soak–Away Pits, Pervious Pavements, and Constructed Wetlands. These techniques offer valuable solutions for managing stormwater runoff, reducing surface runoff, improving water quality, and promoting groundwater recharge. The manual's detailed guidance, case studies, and comparative analysis empower professionals, planners, and policymakers to adopt sustainable stormwater management practices and foster environmental conservation in urban areas.

2.2.8 Rooftop Rainwater Harvesting related study

Rainwater Harvesting in the Context of BNBC, 2021:

The Bangladesh National Building Code (BNBC) 2021 distinguishes the importance of sustainable water management practices, especially in the face of increasing water scarcity and climate change impacts. BNBC suggested that every building proposed for construction on plots having an extent of 300 m^2 or above shall have facilities for conserving and harvesting rainwater. BNBC, 2021 lays down specific guidelines and requirements for implementing RRH systems in both residential and commercial buildings. This section examines how BNBC, 2021 addresses rainwater harvesting, its objectives, and its potential impact on water resources and building design.

In chapter of rooftop Rainwater Harvesting BNCC, 2021 has provided the guideline subjected to the following aspects:

(i) Storing Rainwater: The capacity of water tank should be enough for storing water required for consumption between 2 dry spells.
(ii) Precautions in Rainwater Harvesting and Rainwater Storage.
(iii) Qualifying Rainwater for Harvesting
(iv) Catchments area for Collecting Rainwater: For flat surface, the catchment area is its plan area plus 50% of the adjoining vertical wall contributing rainwater accumulation on the concerned catchment.
(v) Flushing out First Rainwater: 20 minutes for Dhaka metropolitan area.
(vi) Determining Volume of Rainwater Storage:

Rainwater storage volume in m^3 = $D \times N \times D_p / 1000 + flooting$

Where, D = Rainwater demand in liter per capita per day.

N = Population number.

D_p = Number of days for which water will be stored. Consider 90 days for drinking, cooking, utensils cleansing, bathing and ablution purposes; 210 days for other purposes.

Teston et. al., (2018) conducted a study Impact of Rainwater Harvesting (RWH) on the Drainage System to quantify the reduction in stormwater runoff volume and evaluate how the implementation of rainwater harvesting systems could alleviate the burden on the existing drainage infrastructure.

The primary objectives of the study were to investigate the effects of rainwater harvesting on the drainage system of a residential condominium in Curitiba, Southern Brazil. Additionally, the study may have explored the efficiency, feasibility, and overall sustainability of rainwater harvesting as a water management solution in the context of this specific.

The research likely employed a comprehensive methodology, combining field surveys, data collection, monitoring, and numerical simulations. Field surveys might have been conducted to assess the condominium's existing drainage system, rainfall patterns, and hydrological characteristics. Data from rainwater harvesting systems within the condominium were likely collected and analyzed to measure the system's performance in capturing and reusing rainwater. Numerical modeling, such as SWMM (Storm Water Management Model), may have been utilized to simulate various scenarios, including the impact of rainwater harvesting on stormwater runoff volume and the drainage system's overall hydraulic performance.

Based on the research, the study likely reported a substantial reduction in stormwater runoff volume through the implementation of rainwater harvesting systems within the condominium. The collected data and simulations would have demonstrated the effectiveness of these systems in mitigating the impact of rainfall events on the drainage system. The findings may have also highlighted the potential benefits of rainwater harvesting, such as reduced flooding risk and improved resilience of the urban water infrastructure in Curitiba.

2.2.9 Simulation of surface runoff control effect by permeable pavement (2021)

Chen et. al., (2021) were conducted a study to evaluate the effectiveness of permeable pavement in controlling surface runoff and reducing stormwater runoff volume. The researchers sought to quantify the potential benefits of using permeable pavement in mitigating urban runoff and explore its applicability in different environmental settings.

By conducting simulations, the study aimed to provide insights into the performance of permeable pavement under various design and climatic conditions.

The study employed computer simulations and modelling techniques to assess the performance of permeable pavement in controlling surface runoff. The researchers likely used numerical modelling software such as SWMM (Storm Water Management Model) or similar hydrological models to simulate stormwater runoff scenarios with and without permeable pavement. Various parameters, such as rainfall intensity, pavement permeability, slope, and land use, were considered in the simulations to represent different real-world scenarios accurately. the study area was a two-way, two-lane branch road in Xi'an, China; the length and width of the road are 146 m and 20 m, respectively, and the cross slope of the road is 1.5% as shown in **Figure 2-8**.

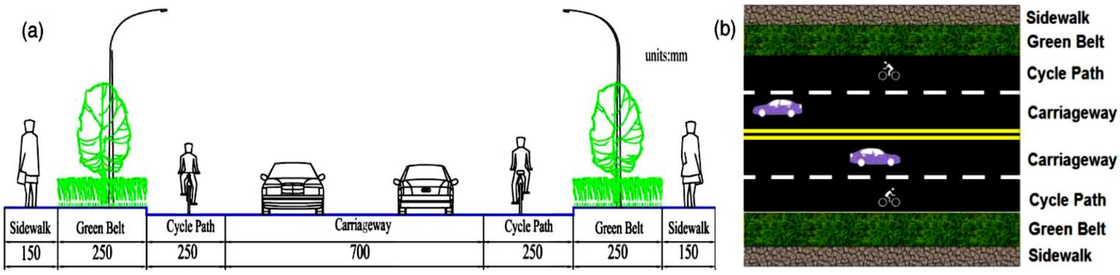


Figure 2-8: (a) cross-sectional of a branch road, (b) the specific distribution.

Based on the simulations, the study demonstrated that permeable pavement is effective in controlling surface runoff. It was observed that the implementation of permeable pavement reduced the volume of stormwater runoff, particularly during intense rainfall events. The results also indicated that the performance of permeable pavement is influenced by various factors, including pavement permeability, slope, and rainfall characteristics. Furthermore, the study likely compared the runoff reduction efficiency of permeable pavement to conventional impervious surfaces, highlighting the advantages of using this sustainable stormwater management practice.

2.2.10 Wetland Basin Related study

Smith et. al., (2018) investigated the role of wetland basins in flood control by conducting a case study in an urban area prone to severe flooding. The study found that well-designed wetland basins effectively attenuated peak flows during storm events, helping to alleviate flood risks in downstream areas. Additionally, the research highlights the importance of considering wetland basin size, location, and connectivity within the stormwater drainage network for maximum flood mitigation benefits.

Jones et. al., (2019) examined the hydraulic performance of constructed wetland basins using numerical simulations and field measurements. The study assessed various design configurations and demonstrated that the incorporation of wetland basins in stormwater drainage systems can improve flow regulation, reducing the risk of localized flooding and erosion. The findings underscore the significance of proper design and maintenance of wetland basins to ensure their long-term hydraulic effectiveness.

Chen et. al., (2020) employed a hydrological model to assess the impact of wetland basins on the performance of urban drainage systems. The study demonstrated that integrating wetland basins into the stormwater management strategy can lead to a substantial reduction in peak runoff volumes and attenuate flow rates during storm events. Additionally, the research discusses the need for proper land-use planning to identify suitable locations for wetland basin implementation.

2.2.11 Other Related Study

Mark et. al., (2001) conducted a study titled "Modelling of Urban Flooding in Dhaka City." This research employed a dual approach, combining physically based modeling with GIS (Geographic Information System). The urban drainage system was structured using the MOUSE model, which consisted of two networks: one for simulating the free-surface flow over the streets and another for the sewer pipe system. The study demonstrated the effectiveness of this modeling system, in conjunction with GIS, as a potent analytical tool for addressing urban flooding issues. It was found that the application of GIS and the model's results allowed for clear and straightforward presentation of the flood inundation map. In conclusion, the researchers advocate for the further integration of mathematical modeling and GIS, as they believe it will lead to more cost-efficient and versatile studies in the future.

Feroz Islam (2014) assess the impacts of climate change on Central Dhaka's water logging for various scenarios. The study involved the development of both hydrologic and hydrodynamic models using MIKE URBAN (Mouse Engine). To calibrate the model, the flood extent data from a previous study on the 2004 flood conducted by IWFM was utilized. Various scenarios were then formulated to explore the potential impacts of climate change and adaptation measures. The findings indicate that during intense rainfall events, the drainage system becomes reliant on pumps. The capacity of the drainage system to efficiently transport excess water from rainfall runoff to retention ponds becomes crucial in such situations. Consequently, regular maintenance of the drainage network is essential to reduce sediment build-up and ensure its effective functioning.

Rimi (2016) research on the analysis of rainfall data and the generation of intensity-duration-frequency relationships for selected urban areas in Dhaka offers valuable insights into the understanding of extreme rainfall events in the region. Under the study Gumbel and Log Pearson Type III (LPTIII) distribution were used to develop frequency analysis of rainfall data. Chi-square goodness of fit test was used and found that the Gumbel Distribution Method gave a better fit than Log Pearson Type-III method. The study like produced a series of IDF curves, depicting the rainfall intensity for specific return periods (e.g., 2, 5, 10, 25, 50, 100 years) across varying durations (e.g., 5 minutes, 10 minutes, 30 minutes, 1 hour, 24 hours). For estimating short duration rainfall intensity two method were evaluated such as- i) U.S. depth duration ratios and Hershfield and Wilson diagram and ii) empirical formula developed by Indian Meteorological Department (IMD). These relationships are vital in estimating the frequency of extreme rainfall events and determining the design standards for urban drainage systems and flood control measures.

Ahammad (2018) analyzes stormwater runoff for a selected catchment of eastern Dhaka by using hydrologic model. The study area covered approximately 47.67 km^2 and was simulated using GeoSWMM. Calibration of the model was achieved by comparing it with observed water level data at Begunbai canal, Banasree, and designing drainage facilities for a 2-day 10-year return period. For the climate effects of the A1FI 2050 scenario, a design rainfall value of 286 mm was taken into account. The research findings revealed the necessary pumping capacities and retention facilities required after the implementation of the Eastern Embankment. Additionally, the study explored the potential for reducing flooding through the use of storage tanks in a rainwater harvesting system.

Chapter 3

Theory

3.1 Frequency Analysis

Frequency of a hydrologic event, such as the annual peak flow is the probability that a value will be equaled or exceeded in any year. This is more appropriately called the exceedance probability, *P(F)*. The reciprocal of the exceedance probability is the return period T in years, i.e., $T = {}^{1}\!/_{P(F)}$

To justify extrapolating the frequency relationship, the length of the record should be sufficient. For example, it might be reasonable to estimate a 50-year flood based on a 30-year record, but to estimate a 100-year flood on the basis of a 10-year record would normally be absurd (Neill, 1973). Viessman and Lewis, 1996 noted that as a general rule, frequency analysis is cautioned when working with shorter records and estimating frequencies of hydrologic events greater than twice the record length.

Frequency analysis can be conducted in two ways: one is the analytical approach and the other is the graphical technique in which magnitudes of an event are plotted against the probability of exceedance. The analytical approach was used in this study for frequency analysis of rainfall, discharge and water level data.

Analytical frequency analysis is based on fitting theoretical probability distributions to given data. Numerous distributions have been suggested in different studies on the basis of their ability to “fit” the plotted data. The Gumbel extreme value (EV1), Normal and Lognormal distributions methods were used in this study for frequency analysis.

(i) Gumbel Theory of Distribution

Distributions of the extreme values selected from sets of samples of any probability distribution converge to any one of three forms of Extreme Value Distributions, called Type I, II, and III, respectively, when the number of selected extreme values is large. The three limiting forms are special cases of a single distribution called Generalized Extreme Value (GEV) Distribution. (Chow et. al., 1988). The cumulative distribution function for the GEV is

$$F(x) = \exp\left[-\left(1 - \kappa \frac{x-u}{\alpha}\right)^{1/\kappa} \right] \qquad (1)$$

where κ, u, and are parameters to be determined. For EVI Distribution x is unbounded, and Distribution is also known as the Gumbel Distribution. The Extreme Value Type I (EVI) cumulative distribution function is

$$F(x) = \exp\left[-\exp\left(-\frac{x-u}{\alpha}\right)\right] \qquad - \propto \leq x \leq \propto \qquad (2)$$

The parameters are estimated by

$$\alpha = \frac{\sqrt{6}}{\pi} s \qquad (3)$$

$$u = \bar{x} - 0.5772\alpha \qquad (4)$$

Eq (2) expressed as

$$F(x) = e^{-e^{-y}} \qquad (5)$$

where y is the reduced variate defined as

$$y = \frac{x-u}{\alpha} \qquad (6)$$

And,

$$y = -\ln\left[\ln\left(\frac{1}{F(x)}\right)\right] \qquad (7)$$

Noting that the probability of occurrence of an event $x \geq x_T$ is the inverse of its return period T, it can be written as

$$\frac{1}{T} = P(x \geq x_T) = 1 - P(x \leq x_T) = 1 - F(x_T)$$

So, $\quad F(x_T) = 1 - \frac{1}{T}$

By substituting for *F(* x_T *)* value into Eq (7)

$$y_T = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right] \qquad (8)$$

For a given return period x_T is related to y_T by the following equation

$$x_T = u + \alpha y_T$$

(ii) Normal Distribution

An alternative method based on frequency factor is used for calculating the magnitude of extreme events. It has shown that most frequency functions can be generalized to following equation.

$$x_T = \bar{x} + K_T s \quad (9)$$

where x_T is a flood of specified probability or return period T, $\bar{x}$ is the mean of the flood series, s is the standard deviation of the series; and K_T is the frequency factor and is a function of return period and type of probability distribution, as well as coefficient of skewness for skewed distribution.

The frequency factor can be expressed as

$$K_T = \frac{x_T - \bar{x}}{s} = z \quad (10)$$

Thus, for Normal Distribution K_T is the same as the standard normal variable z. The value of z and hence K_T can be obtained from **Table 1 in Appendix**.

(iii) Lognormal Distribution

The recommended procedure for use of the Lognormal Distribution is to convert the data series to logarithms and compute:

1) $y_i = \log x_i$

2) Compute the mean, $\bar{y}$ and standard deviation s_y

3) Compute $y_T = \bar{y} + K_T s_y$

$$K_T = \frac{y_T - \bar{y}}{s_y} = z$$

Value of K_T, can be taken from **Table 2 in Appendix**.

4) Finally compute $x_T = anti\log y_T$

<u>Goodness-of-fit Tests</u>

The goodness of fit of a probability distribution can be tested by comparing the theoretical and sample values of the relative frequency or the cumulative frequency function. In the case of the relative frequency function, the $\chi 2$ – test is used and with cumulative frequency function the Kolmogorov-Smirnov test is used.

Chi-Square Test: The test statistic is given by

$$\chi^2 = \sum_{i=1}^{k} \frac{n\left[f_s(x_i) - p(x_i)\right]^2}{p(x_i)} \tag{11}$$

where k is the number of intervals; the sample value of the relative frequency of interval i is, fs(xi) = ni/n; the theoretical value of the relative frequency function (also called incremental probability function) is p(xi) = F(xi) - F(xi-1). It may be noted that nfs(xi) = ni, the observed number of occurrences in interval i, and np(xi) is the corresponding expected number of occurrences in interval i.

To describe the test, the probability distribution must be defined. A distribution with υ = k-l-1 degrees of freedom (l is the number of parameters used in fitting the proposed distribution) is the distribution for the sum of squares of υ independent standard normal random variables zi. The critical distribution function is tabulated **in Table 3** in **Appendix** from Haan, 1977. A confidence level is chosen for the test; it is often expressed as 1-α, where α is termed the significance level.

3.2 Intensity Duration Frequency (IDF) Curves

The rainfall intensity-duration-frequency (IDF) curve allows to the computation of the design storm depth h*(P, d), which is the expected rainfall depth for a given duration, d, of the storm and a given probability of occurrence, P (Di Baldassarre et. al., 2006). It is current practice in applied engineering to express the probability of occurrence of an event by specifying its return period, T, which is the average length of time separating the event itself from the closest one having equal or greater magnitude. Estimation of the design storm depth is required when designing artificial drainage systems or engineering works that interfere with natural river networks. When observed rainfall extremes are available for the storm duration d_i of interest, the design storm depth can be estimated by fitting on such data a suitable extreme value probability distribution, that allows computing the rainfall quantities $Q_i(T, d_i)$ corresponding to the return period T. In order to estimate the design storm depth for durations of the event that are different with respect to the observed data refer to, the IDF parametric curve is used to interpolate the above quantities, therefore allowing to compute h*(T, d) for any storm duration. Increasing the number of parameters allows one to obtain a more flexible fitting of the rainfall quantities $Q_i(T, d_i)$, but the effects of parameter uncertainty are amplified and therefore the uncertainty of the estimated h*(T, d) is amplified as well.

Another approach for limiting the parameterization requirements of IDF curves by assuming that the rainfall data, once suitably rescaled, follow the same probability distribution for any storm duration. This technique presents the considerable advantage of allowing the expression of the IDF curve as an analytical function of the return period, therefore not requiring the a priori selection of T itself. This type of approach was not considered in this study as the scale invariance assumption is often not consistent with the statistical behaviours of short-duration extreme rainfalls(Burlando and Rosso, 1996)

Design storm depth estimation is complicated when referring to short storm durations, as typically happens when designing urban drainage systems. In fact, in many countries, short-duration rainfall data are rarely available, and therefore the IDF curves are typically extrapolated below the range of durations that were used for calibrating their parameters. It is well known that such a procedure amplifies the estimation error, the magnitude of which is expected to depend on the type of IDF equation that is used (Chen, 1983).

3.2.1 Estimation of Short-Duration Rainfall

The Indian Meteorological Department (IMD) employs an empirical reduction formula to estimate rainfall values for various durations such as one hour, two hours, three hours, five hours, and eight hours based on annual maximum values. Chowdhury et. al. (2007) used an empirical reduction formula developed by the Indian Meteorological Department (IMD) to estimate short-duration rainfall from daily rainfall data and discovered that this formula provides the best estimate of short-duration rainfall (Rasel and Hossain, 2015). To estimate short-duration rainfall, the IMD empirical formula $P_t = P_{24}\sqrt[3]{\frac{t}{24}}$ has been used in this study. Where, P_t is the required rainfall depth in mm at t hour duration, P_{24} is the daily rainfall in mm and t is the rainfall duration in hours for which the rainfall depth is required.

3.2.2 Design Storm Hyetographs from IDF Relationships

In the conventional hydrologic design method, such as the rational method, only the peak discharge was used. There was no consideration of the time distribution of discharge (the discharge hydrograph) or the time distribution of precipitation (the precipitation hyetograph). However, more recently developed design methods also focuses on temporally varied storm pattern and discharge. In this respect, the design of hyetographs has drawn the attention of scientists.

The **Alternating Block Method** (ABM) is a simple and widely acknowledged method for developing a design hyetograph from an intensity-duration-frequency (IDF) curve. The design hyetograph produced by this method specifies the precipitation depth occurring in n successive time intervals of duration Δt over a total duration $T_d = n\Delta t$, After selecting the design return period, the intensity is read from the IDF curve for each of the durations Δt, $2\Delta t$, $3\Delta t$, ... , and the corresponding precipitation depth is found as the product of intensity and duration. By taking differences between successive precipitation depth values, the amount of precipitation to be added for each additional unit of time Δt is found. These increments, or blocks, are reordered into a time sequence with the maximum intensity occurring at the centre of the required duration T_d and the remaining blocks arranged in descending order alternately to the right and left of the central block to form the design hyetograph (Chow et. al., 1988).

3.3 Rainfall-Runoff Model

3.3.1 Runoff Modelling with MOUSE in MIKE URBAN

The Mouse engine of MIKE URBAN includes a series of surface runoff models. The surface runoff models available in MIKE URBAN (MOUSE) are:

- Time-Area Method (A)
- Kinematic Wave (B)
- Linear Reservoir (C1 and C2)
- Unit Hydrograph Method (UHM)

Surface runoff computations in MOUSE can be based on any of the four concepts, as long as the necessary data for each catchment is specified in the set-up. However, it is not possible to combine different runoff computation concepts for different model areas in a single simulation run. Except for UHM, runoff computed by any of the surface runoff models can be supplemented by a continuous runoff component, i.e. rainfall-induced infiltration can be added to the computed surface runoff hydrographs as the catchment "base flow" (DHI, 2017).

3.3.2 Data Organization for Runoff Models

The simulated flow of water in a particular catchment can vary significantly depending on the runoff model used. Each model has its own unique input data, model parameters, and method of calculating runoff. It is crucial to understand the context and meaning of the runoff computations employed in each model, along with the interpretation of different data and model parameters.

The data and model parameters for the runoff model are organized into three categories:

- **(i)** **General catchment data:** These are common to all runoff models and include fundamental information such as catchment size, network connection point, geographical coordinates, and specifications of constant additional inflow.
- **(ii)** **Model-specific catchment data:** Each model has its own set of data for catchment geometry and land use description, which may vary in detail. These data also refer to specific parameter sets associated with each model.
- **(iii)** **Model parameters:** These parameters are grouped into sets, and each set contains all the editable parameters required to perform a particular type of runoff computation. The surface runoff computation for a given catchment relies on the parameters included in the corresponding set linked to that catchment.

For the purposes of this study, "Model A" is the chosen model used to compute surface runoff.

3.3.3 Surface Runoff Model "A"-Time Area Method

3.3.3.1 Concept

The concept of surface runoff computation of MOUSE Runoff Model A is founded on the so- called "Time-Area" method. The runoff amount is controlled by the initial loss, the size of the contributing area and a continuous hydrological loss. The shape of the runoff hydrograph is controlled by the concentration time and by the time-area (T-A) curve. These two parameters represent a conceptual description of the catchment reaction speed and the catchment shape (Kidmose et. al., 2015).

Input Data

Surface Area Model “A)’ Time Area Method required the following (**Table 3:1**) input data.

Table 3:1: Input Data for Surface Runoff Model “A”

General Catchment Data	Model Specific Data	Hydrologic Parameters
a. Catchment ID b. Catchment Connection Point c. Catchment Co-Ordinates d. Catchment Area [ha] e. number of inhabitants associated with the catchment f. Additional flow [m^3/s]	a. Impervious area - a fraction of the catchment area, [%], considered to contribute to the runoff.	a. Initial loss (m) b. Runoff reduction factor accounts for water losses caused by e.g. evapotranspiration, imperfect imperviousness c. Time Area Curve: • TACurve1-rectangular catchment • TACurve2-divergent catchment • TACurve3-convergent catchment

The user-defined TA curve can be specified, thus allowing the correct description of irregular catchments. The default TA type is a pre-computed curve type 1. Time Area Coefficient formula has been introduced covering curve shapes “in-between” the three standard ones (and outside). The formula has been suggested by Nittaya Wangwongwiroj at AIT. The curves “in-between” are specified by giving the Time Area Coefficient directly in MOUSE instead of specifying a Time/Area Curve (DHI, 2017).

$$y = 1 - (1 - x)^{\frac{1}{a}} \qquad for\ 0 < a < 1 \tag{12}$$

$$y = x^{a}\ for\ 1 \leq a \tag{13}$$

Where

y = Accumulated dimensionless area

x = Dimensionless concentration time

a = Time area curve coefficient

A plot of the curves is presented in **Figure 3-1.**

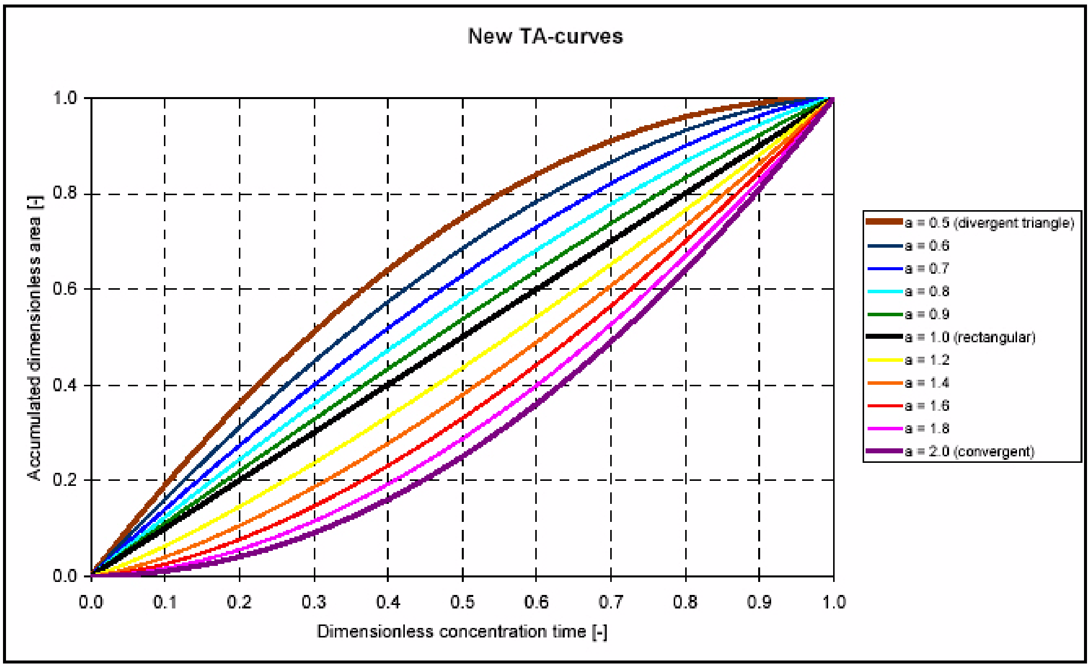


Figure 3-1: Time-area coefficient variations (DHI, 2017)

Runoff Computation

The continuous runoff process is discretised in time by the computational time step Δt. The assumption of the constant runoff velocity implies the spatial discretisation of the catchment surface to a number of cells in a form of concentric circles with a centre point at the point of outflow. The number of cells equals:

$$n = \frac{t_c}{\Delta t} \qquad (14)$$

Where:

$t_c = concentration\ Time$

$\Delta t = simulation\ time$

MOUSE calculates the area of each cell on the basis of the specified time area curve. The total area of all cells is equal to the specified impervious area. A time-area curve characterises the shape of the catchment, relating the flow time i.e. concentric distance from the outflow point and the corresponding catchment sub-area. There are three pre-defined time/area curves available in MOUSE. Irregularly shaped catchments can be more precisely described by the user-specified TA curves (**see Figure 3-2)(** DHI, 2017**)**.

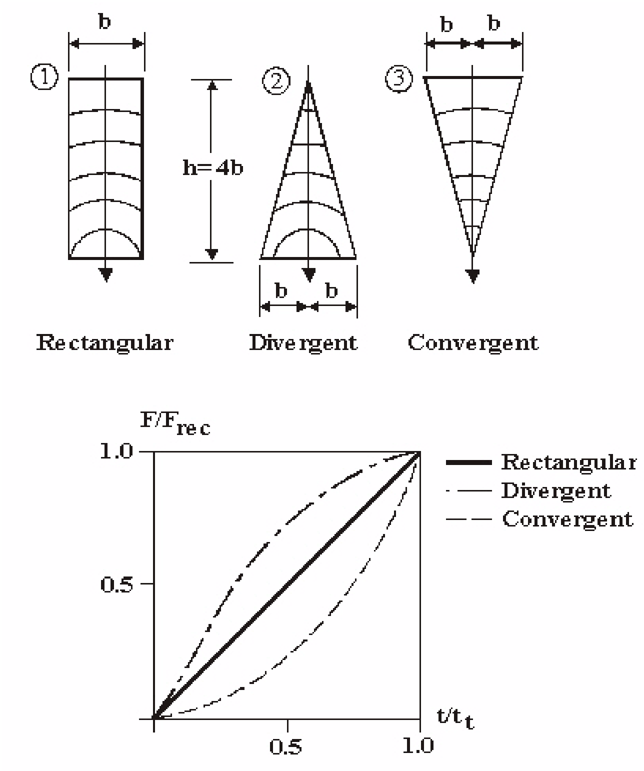


Figure 3-2: Three pre-defined time/area curves available in MOUSE (DHI, 2017)

The runoff starts after the rain depth has exceeded the specified initial loss for the catchment. The runoff stops when the accumulated rain depth on the whole catchment surface regresses below the specified initial loss for the catchment. At every time step after the start of the runoff, the accumulated volume from a certain cell is moved to the downstream direction. Thus, the actual volume in the cell is calculated as a continuity balance between the inflow from the upstream cell, the current rainfall (multiplied by the cell area) and the outflow to the downstream cell. The outflow from the most downstream cell is actually the resulting surface runoff hydrograph (Kidmose et. al. 2015).

3.3.4 Time of Concentration

Time of concentration (t_c) is the travel time of runoff flows from the most hydraulically remote point upstream in the contributing catchment area to the point under consideration. The concept of time of concentration is important in all methods of peak flow estimation as it is assumed that the rainfall occurring during the time of concentration is directly related to the peak flow rate. The design storm duration was taken as equal to or greater than the time of concentration (t_c). The t_c was calculated as the sum of the overland flow time (to) and the time of travel in street gutters (t_g), or roadside swales, drains, canals and khals (t_d).

The relevant equations used to calculate the t_c are given in **Table 3:2,** although there are other equations available recommended by different scholars. These equations have been preferred based on the recommendation of the Drainage Master Plan of DWASA, 2015. Calculation of t_c is subject to the catchment properties, particularly length, slope and roughness of the drainage path. For the estimation of overland flow travel time, Friend's equation has been used. The overland flow time (t_0) was estimated with the proper judgment of the land surface condition since the length of sheet flow is short for steep slopes and long for mild slopes. This equation was applied only for distances (L) recommended in **Table 3:2**. Catchment roughness, length and slope affect the flow velocity and subsequently the overland flow time. Typical values of Horton's roughness n* for various land surfaces are given in **Table 3:2**. The drain flow time (t_d) Manning's equation has been used for the remaining length of the flow paths downstream. Care has been given to obtain the values of hydraulic radius and friction slope which are key inputs to the drain flow time equation. The recommended minimum time of concentration for a catchment has been taken as 5 minutes, which applies to very small sub-catchments only.

Table 3:2: Equations to Estimate Time of Concentration (QUDM, 2007)

Travel Path	Travel Time	Remarks
Overland Flow	$t_o = \frac{107n^*L^{1/3}}{S^{1/5}}$	t_0= Overland sheet flow travel time (minutes) L= Overland sheet flow path length(m) *for Steep Slope (>10%), L ≤ 50 m* *for Moderate Slope (<5%), L ≤ 100 m* *for Moderate Slope (<1%), L ≤ 200 m* $n^* =$ Horton's roughness value for the surface (**Table 3:3**) *S*= Slope of overland surface (%)
Drain Flow	$t_d = \frac{nL^{1/3}}{60R^{2/3}S^{1/5}}$	$n =$ Manning's roughness coefficient (**Table 3:4**) R= Hydraulic Radius (m) S= Friction Slope (m/m) L= Length of reach (m) t_d= Travel time in the drain (minutes)

Table 3:3: Values of Horton's Roughness n* for various land surfaces (QUDM, 2007)

Land Surface	**Horton's Roughness n***
Paved	0.015
Bare Soil	0.0275
Poorly Grassed	0.035
Densely Grassed	0.060

Table 3:4: Values of Horton's Roughness n* for various drains and pipes (QUDM, 2007)

Drain/Pipe	**Manning Roughness n**
Grassed Drain	
Short Grass Cover (<150 mm)	0.035
Tall Grass Cover (≥ 150 mm)	0.050
Lined Drain	
Concrete	
Smoot Finish	0.015
Rough Finish	0.018
Brick/ Stone	
Brickwork with plaster	0.020
Dressed Stone in Mortar	0.020
Random Stones in Mortar or Rubble Masonry	0.035
Pipe Material	
Vitrified Clay	0.012
Precast Concrete	0.013
uPVC	0.011

3.4 Hydraulic Model

The 1D computational engine for modelling unsteady flows in pipe networks, river networks and estuaries is based on an implicit, finite difference numerical solution of the one-dimensional shallow water equations (Saint Venant equations). The numerical algorithms provide efficient and accurate solutions in branched and looped pipe and river networks. The computational scheme is applicable to vertically homogeneous flow conditions. These conditions are found in pipes ranging from small profile collectors for detailed urban drainage and pressurised sewer mains affected by the varying water level at the outlet, to open rivers, channels and flood plainextending from steep rivers to tidal-influenced estuaries. Both subcritical and supercritical flows are treated by means of the same numerical scheme that adapts to local flow conditions. Flow features such as backwater effects and surcharges are simulated precisely. Pressurized flow computations are modelled using a narrow vertical extension of closed pipes. Free surface and pressurized flows are thus described with the same numerical scheme, which ensures a smooth and stable transition between the two flow types. Advanced models describing flow over hydraulic structures including structure control are also included. The features implemented in the conceptualization of physical systems enable realistic and reliable simulations of the performance of both existing sewer and river networks and those under design (DHI, 2017).

The 1D engine solves the problem of water flowing through a network of reaches, nodes, and structures. The problem is fully specified by boundary conditions at the network boundaries and initial conditions (DHI, 2017).

3.4.1 Saint Venant equations

Solving the problem of an arbitrary flow of water through three-dimensional gravity drain is computationally impractical. Fortunately, the geometry of pipes and canals allows for a set of approximations, which naturally fit the problem space. The approximations rely on the following assumptions:

- Water is incompressible and homogeneous, i.e., it has negligible variations in density
- The bottom slope of pipes and canals is small, so that the cosine of the angle it makes with the horizontal may be taken as 1.
- Wave lengths are large compared to the water depth. This ensures that the flow everywhere can be regarded as having a direction parallel to the bottom, i.e. vertical accelerations can be neglected and a hydrostatic pressure variation along the vertical can be assumed.
- There is no acceleration of water perpendicular to the flow direction of a reach.

With these assumptions, two simplified equations can be derived from the Navier-Stokes equations, which are usually called the Saint Venant equations, or Shallow Water equations:

Continuity Equation (Conserves Mass)

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial X} = 0 \tag{15}$$

Momentum Equation (Conserves Energy)

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial y}{\partial x} + gA\left(S_e + S_c + S_f - S_0\right) = 0 \tag{16}$$

Where:

$\frac{\partial Q}{\partial t}$ = Local Inertia (Hyd)

$\frac{\partial\left(\frac{Q^2}{A}\right)}{\partial x}$ = Convective Inertia (Hyd of Diffusive Wave)

$\frac{\partial y}{\partial x}$ = Pressure Slope (Hyd and San)

S_f = Friction Slope (Hyd, San and Rnf)

S_0 = Bed Slope (Hyd and San)

S_c = Entrance/Exit Losses (Hyd and San)

S_e = Expansion/Contraction Losses (Hyd and San)

Saint Venant equations are only valid for free surface flows. Since the pressurized flow in closed reaches is an important part of the 1D problem space, an approximate treatment of pressurized flow has been adopted. To solve the Saint Venant equations, a finite difference scheme is implemented. For super-critical flows (Froude number larger than 1), a simpler approximation where non-linear terms are suppressed provides an accurate flow description (DHI, 2017).

3.5 1D-2D Couple Model

Surface flooding can be simulated with MIKE URBAN using the following approach: Combined 1D and 2D model: The flow in the subsurface pipes is simulated using the MOUSE 1D engine and the overland flow is simulated using the MIKE 21 2D overland flow model. This combination of models is called MIKE FLOOD. The main input required for the

The 2D overland flow model is a digital elevation model. Based on this, the 2D model is capable of simulating overland flow paths and velocities. The main advantages of using this approach are:

– Leads to more reliable and more accurate modelling

– Requires less engineering judgment

– Requires less engineering hours

– Provides better result visualization

The main disadvantage of this approach is the longer simulation run times (DHI, 2017).

3.6 Concept of Low Impact Developments

Low Impact Development (LID) is a storm water management method that focuses on drainage systems by maintaining or restoring a site's natural hydraulic processes. Rather than collecting storm water in a piped or channelized network and regulating it at a large–scale "end of pipe" location, LID uses a decentralized strategy that disperses flows and manages runoff closer to where it originates. Storm water runoff would encounter natural retention, filtration, and infiltration mechanisms on an undeveloped site, therefore this management strategy concentrates on simulating these. As a result, the preservation of native vegetation and natural drainage features is the most critical issue to consider when applying LID to site design. A brief description of the LID elements is provided in this section:

3.6.1 Rooftop Rainwater Harvesting (RWH):

In recent years, rainwater harvesting (RWH) has gained popularity. It has positive effects on urban water infrastructure even in developed countries with functioning water supply systems as well as drainage systems. The two advantages of RWH systems are volumetric runoff reduction and peak flow reduction as part of sustainable drainage networks. Volumetric runoff reduction is significant in cities with combined sewer systems because it helps to reduce the detrimental effects of releasing a mix of urban drainage water. The volumetric technique compares runoff volumes generated in a watershed before and after considerable RWH system adoption.

The Volume of rain water to be preserved depends upon the purpose of use, rainfall intensity at the locality and the available catchments from where rainwater shall be collected. The usable rainwater volume was calculated using Equation (Teston et. al., 2018),

$$\mathrm{V_{use}} = \mathrm{P} * \mathrm{A} * \mathrm{C} * \mathrm{n} \qquad (17)$$

where V_{use} is the daily volume of usable rainfall (L); P is the daily rainfall (mm); A is the roof area (m^2); C is the runoff coefficient (dimensionless); and n is the coefficient to represent rainwater losses (dimensionless); it depends on the solids disposal device and the surface flow deviation; it was taken as 0.85 to represent rainwater losses of 15%.

3.6.2 Soak-Away Pits

Soak-Away Pits are relatively small excavated chambers that serve as an underground infiltration reservoir. Soak-Away Pit depth generally ranges between 3 and 10 feet, and widths generally range from 4 to 8 feet. The maximum allowable contributing area is also relatively small and should not exceed 1,900 $feet^2$. The volume of runoff that the soak–away pit will need to accommodate is one of the most important concerns in its construction. The target infiltration volume is a function of the contributing area, runoff coefficient, and target precipitation. The equation relating the three variables is:

$$TIV = \frac{A*P*C}{12} \qquad (18)$$

Where,

TIV= Target Infiltration Volume ($feet^3$)

A= Contributing Area ($feet^2$)

P= Target precipitation (inches), for 1-year, 24 Hour Storm

C= Runoff Coefficient, usually 0.85

Source: USKH Inc., 2008

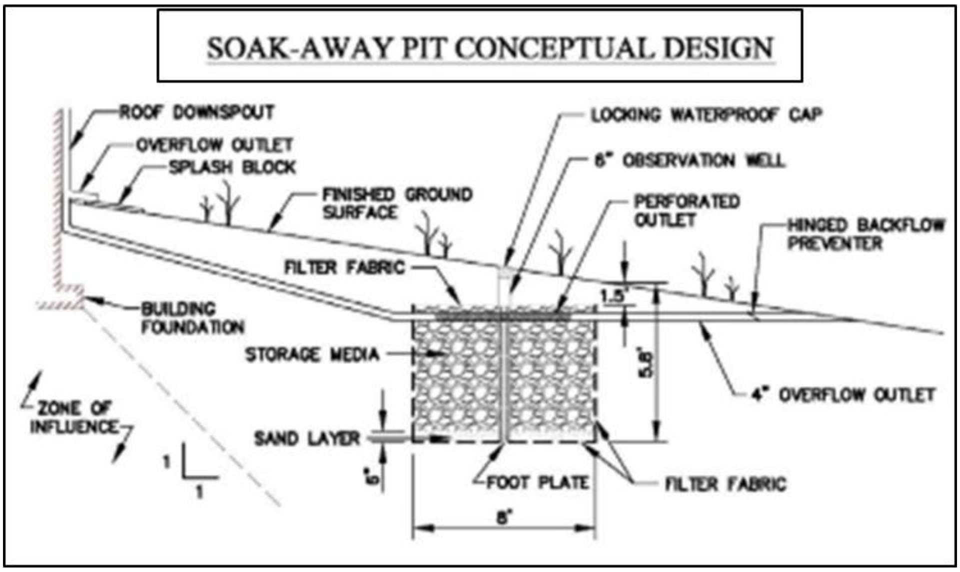


Figure 3-3: Conceptual Design of Soak-Away Pit (USKH Inc., 2008)

Pit depth is the depth of the pit from the surface to the bottom of the excavation. Pit depth is a function of the design infiltration rate, the storage media void ratio, and the retention time. The equation for determining pit depth is provided below:

$$D_s = \frac{I*t}{n_s*12} + 2 \qquad (19)$$

D_s = Soak–Away Pit Depth (feet), from 4 to 10 feet

I = Design Infiltration Rate (inches/hour), between 0.3 and 1 inches/hour

t = Retention Time (hours), from 24 to 72 hours

n_s= Storage Media Void Ratio, 0.4 typical for 1.5 to 3–inch stones

The pit footprint is the plan view area of the pit and is a function of the design infiltration rate, the retention time, and the target infiltration volume. The equation for determining the pit footprint is provided below:

$$A_s = \frac{TIV * 0.66}{n_s * (D_s - 2)} \tag{20}$$

Source: USKH Inc., 2008

3.6.3 Pervious pavements

Pervious pavement is one of the important approaches to lowering the overall imperviousness of an area while retaining necessary surfaces for bicycle lanes, shoulders, sidewalks, etc., and is the use of porous pavement technologies. Some porous pavement technologies are not applicable in areas where sanding is common. However, other types of porous pavement can be used when adequate drainage, such as a sand or gravel bed, is provided. The suitable porous pavement types are:

1. **Open–Graded Aggregate:** This is unbound aggregate, single–sized, angular, durable, and clean of fine particles so that dust is not generated.
2. **Open–Jointed Paving Blocks or Interlocking Concrete Pavements:** These are modular paving units that allow infiltration between individual units. They are typically built over an open-graded or rapid-draining crushed stone base. Perforated drainage pipes can provide drainage in heavy overflow conditions, or provide secondary drainage if the base loses some of its capacity over time. For installations where slow–draining subgrade soils are present, perforated pipes can drain excess runoff and alleviate the potential for frost heaving.
3. **Concrete Grids**: These are perforated concrete units installed over a compacted soil subgrade, which overlies a dense–graded base of compacted crushed stone, which in turn overlies a 1 to 1–1/2 inch thick bedding sand. The openings in the grids are filled with either topsoil & grass or aggregate (DHI, 2017).
4. **Surface drainage permeable pavement:** This contains two layer of entire surface with permeable asphalt pavement structures or two permeable cement concrete structures, replacing the entire surface layer with permeable materials to form another permeable asphalt pavement structure. The edge drainage system is adopted in this structure. Rainfall permeated the PP through the road surface, then penetrated the drainage system through the crown slope, and finally flowed into the urban drainage pipe network. Permeable material is only used on the upper surface layer or the entire surface layer, and dense-graded material is applied for other layers, so the parameter value of the aquifer is zero (Chen et. al., 2021).

3.6.4 Wetland basin

Wetland basin is an urban stormwater best management practice (BMPs). Wetlands can be natural or constructed. A constructed wetland basin is a shallow retention pond that is constructed to temporarily store stormwater runoff. A constructed wetland requires a perennial base flow to permit the growth of rushes, willows, cattails, and reeds to slow runoff and allow time for sedimentation, filtering, and biological uptake. Flood control storage can be provided in addition to the design storm volume, and the system can be designed to meet both flood control and retention requirements. A conceptually constructed wetland basin has been shown in **Figure 3-4.**

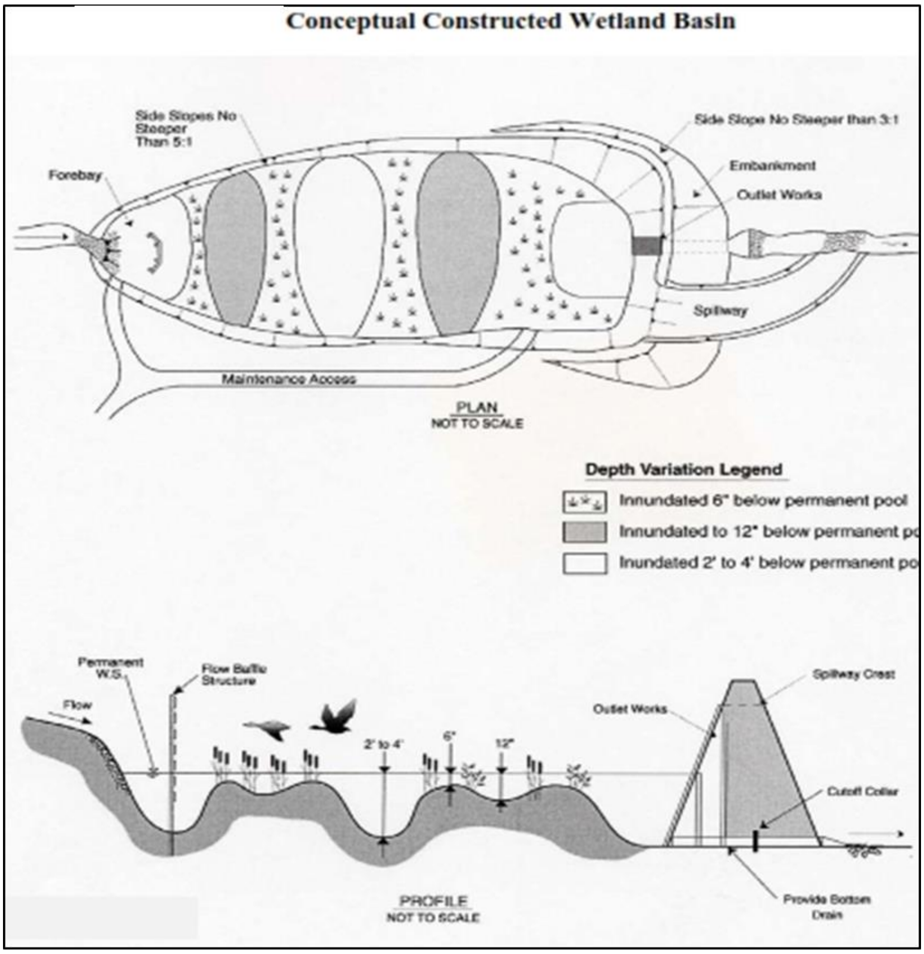


Figure 3-4: Conceptual Design of Constructed Wetland Basin (USKH Inc., 2008)

Basin surcharge storage volume equal to the post-development runoff excess for the 1-year, 24-hour storm. Level pool routing may be applied to reduce volume requirements provided that retention times remain between 12 and 24 hours. The drainage area of an Ideal wetland basin is 25 acres or greater.

Chapter 4

Methodology

4.1 Overview of the Study Area

The Boalia khal catchment area is surrounded by Tongi Khal on the north, Balu River on the east, Purbchal Expressway (300 ft Road) on the south, and Dhaka- Mymensingh Highway on the west. Additionally, runoff is received from Airport & Nikukja sub-catchment through a flood control structure & 2 nos. GRP Pipe, and from a small part of the Bashundhara Residential Area (**Figure 4-2**). The net study area is about 44.0 Km^2. The population density in the study area was 8,956 per km^2 in 2011 (BBS, 2011), which is lower than other developed areas in Dhaka city but is expected to rise as high as 79,075 per km^2 by 2050. Population increasing trend of Dhaka Metropolitan is shown in **Figure 4-1** . A significant portion of the study area is still undeveloped. More than 50% of the area belongs to Cultivated Land /Open Space (34%) and Swamp/Water Bodies (22%) (DAP, 2010).

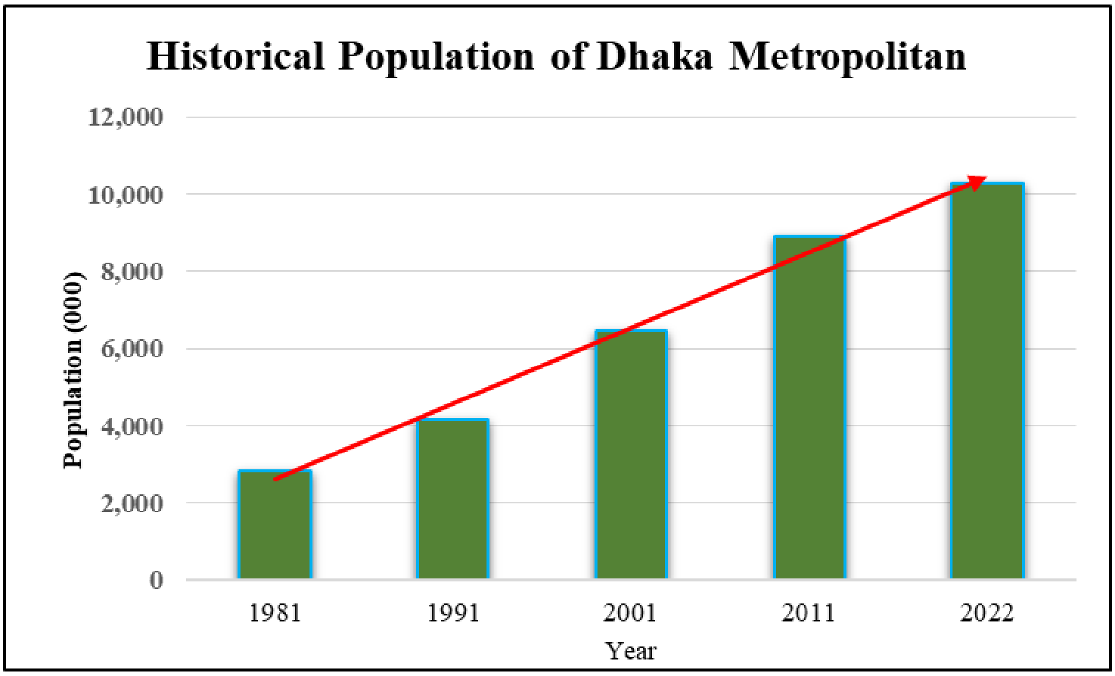


Figure 4-1**: Historical Population of Dhaka Metropolitan**

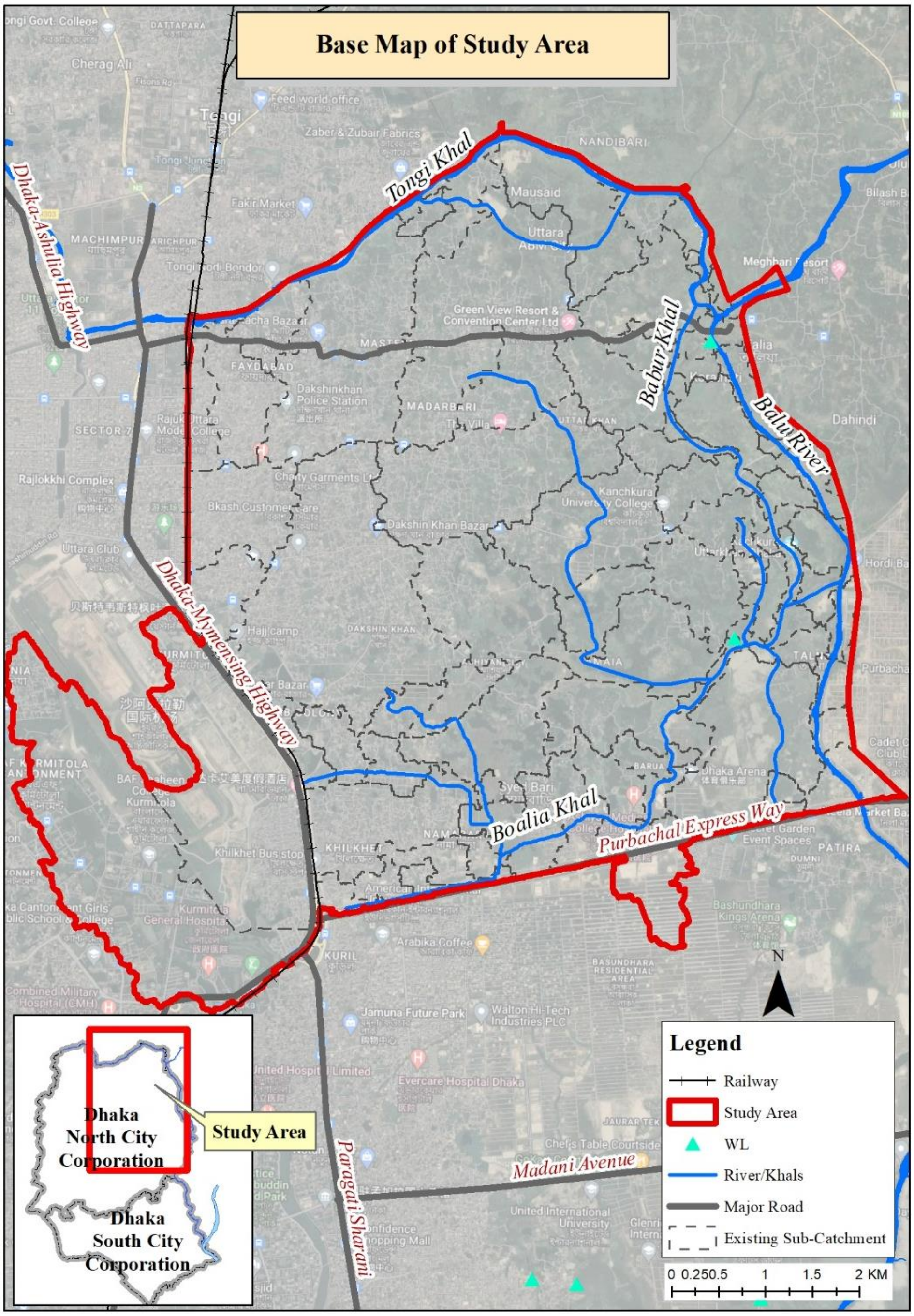


Figure 4-2**: Base Map of the Study Area**

The existing drainage system discharges to the nearest low-lying areas. Also, household wastewater openly discharges to the drainage system. The existing stormwater drainage system is not adequate for such large areas and is often unable to manage usual rainfall. Serious drainage congestion occurs in monsoons, even during the dry season. During such events, most of the areas get flooded by surface runoff mixed with wastewater which leads to health hazards for the inhabitants. Uncontrolled and unplanned urbanization is intensifying over the years. The situation will likely deteriorate further in the future if no action is taken.

The study area's climate is characterized as tropical monsoon. The tropical monsoon climate comprises four hydrological seasons: pre-monsoon (April-May), monsoon (June-September), post-monsoon (October-December), and dry season (January-March) (Dasgupta et. al., 2015).In the pre-monsoon, some rainfall usually in the form of thunderstorms occurs, due to which short-term urban flooding occurs. The average annual rainfall is around 2025 mm and about 70% of the annual rainfall occurs during the monsoon season, resulting in catastrophic urban flooding. The average temperature ranges from around 20^0 degrees Celsius in December and January to around 30^0 degrees Celsius from April to September. In March and April, the maximum temperature might reach 40^0 degrees Celsius. The daily evaporation rate fluctuates between 2.0 and 3.8 mm on average. It reaches its lowest point in July and its maximum point in March (**Table 4:1**).

Table 4:1: Average Climatic Conditions in the Study Area

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Rainfall (mm)	6.6	21.0	52.2	130.3	275.8	355.4	387.4	310.6	281.8	167.4	28.2	9.7
Mean Temp (°C)	18.3	21.9	26.6	27.6	29.2	29.3	28.6	30.0	29.5	27.5	23.8	20.0
Max Temp (°C)	30.0	35.9	39.6	39.6	39.4	37.0	35.7	37.5	37.5	37.0	34.2	30.6
Min Temp (°C)	6.5	7.8	13.5	15.5	18.9	21.5	21.5	22.5	21.4	18.0	13.3	9.4
Humidity (%)	70.7	64.7	64.1	71.4	77.1	82.6	83.9	82.7	82.9	79.0	73.0	72.4
Avg. Wind speed(Km/hr)	5.8	6.4	7.3	8.2	7.6	7.4	7.0	6.6	6.4	5.8	5.2	5.2
Max Wind Speed(Km/hr)	10.1	11.8	13.7	15.3	15.5	14.5	13.6	13.0	13.5	13.9	11.9	10.6
Sunshine (Hours)	6.6	7.6	7.9	7.8	6.9	4.4	4.1	4.7	4.7	6.4	7.3	6.7
Evaporation (mm/D)	2.6	3.0	3.8	3.5	3.3	2.3	2.0	2.2	2.1	2.8	3.1	2.7
Solar Radiation (MJ/m2day)	13.9	17.0	19.6	21.1	20.3	16.6	16.1	16.5	15.4	16.0	15.1	13.4

Source: BMD

4.2 Data Collection and Analysis

To conducting this study various type data was needed and these data analysis and processing extent is given in **Table 4:2.** The location of these data measurement station are shown in **Figure 4-3.**

Table 4:2: Data Collection and Processing extent

Data Type	Source	Processing Extent/ Procedure
Topography	RAJUK & IWM	• Generation of Area-Elevation curve for existing pond • Sub-catchment slope using Arc Hydro tools • Create future DEM considering proposed landuse
Landuse	RAJUK	• Imperviousness Calculation
Population	BBS	• Projection of Population • Wastewater Calculation
Rainfall	BMD	• Frequency analysis and IDF curve generation • Generation of design rainfall hyetograph- ✓ Alternating Block Method
Water Level and Discharge	BWDB	• Frequency analysis
Drainage network	IWM and Relevant agencies	• Flow direction, location of outfall, khal cross sections, pipe diameter, manhole depths, slope

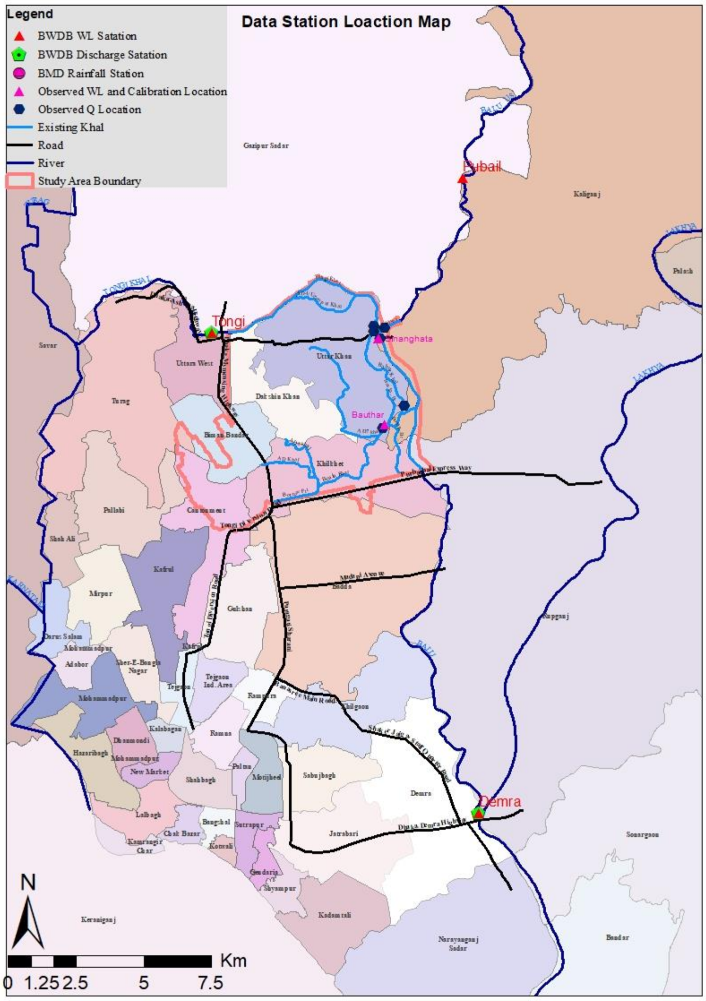


Figure 4-3**: Location of Data Measurement Station**

4.2.1 Topography

The Digital Elevation Model (DEM) data of Dhaka city (*Source: RAJUK*) for the year 2010 available at IWM database were collected. For this study, 2016 has been considered as the base year. In order to represent the 2016 land elevation for the study area, The DEM of 2010 has been updated based on the surveyed topographic data collected from IWM, satellite images, and landfill information based on field visits. The DEM of 2016 **(Figure 4-4)** has been used for simulation scenarios of existing conditions.

As per the DEM of 2016 (**Figure 4-4),** ground elevation in the study area varies from -1.18 mPWD to +10 mPWD. Although elevation varies within a great range, most of the area is almost flat and slightly sloping towards the east. Elevation of the western part of the area is slightly higher and average elevation varies from 6.0 mPWD to 8.0 mPWD. The Eastern side of the area is comparatively lower in elevation. Elevation of the developed portion of this area varies from 5.5 mPWD - 8.0 mPWD. Elevation below 4.5 mPWD is mostly agricultural land and water retention area. The area-elevation curve (**Figure 4-5)** for the study area indicates that at present more than 60% of the study area is lower than +6.0 mPWD elevations which is the average flood level for the Eastern Dhaka area.

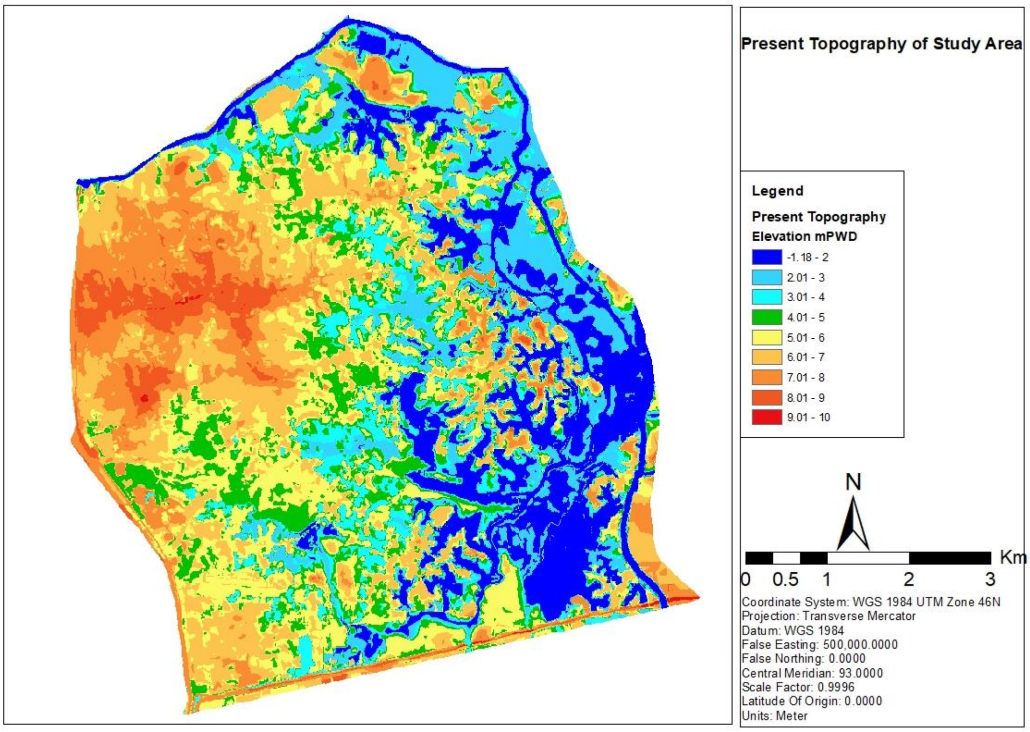


Figure 4-4**: Present Topography of the study area**

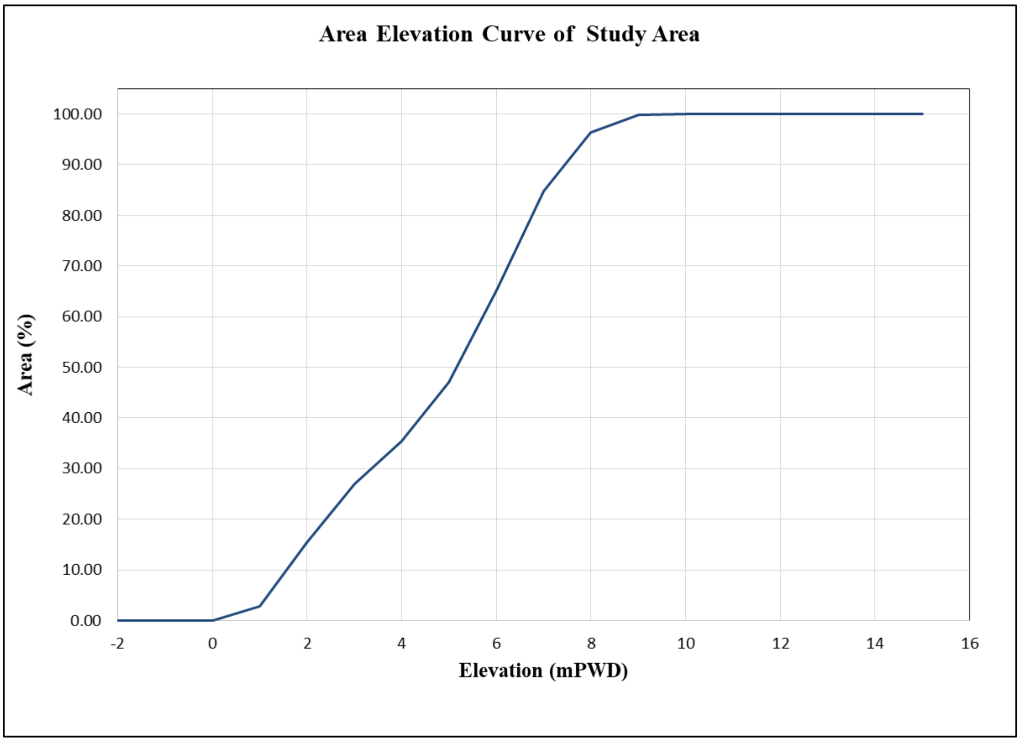


Figure 4-5**: Area- Elevation Curve**

Based on the future landuse plan of RAJUK, an estimated ground elevation map **(Figure 4-6)** has been prepared to simulate future scenarios (2050). A number of things have been considered while preparing this elevation map which are:

- Proposed roads will be constructed at 6.5m elevation based on the road level survey data of existing road under Study of Options for Road and Drainage Network Improvement in Extended DNCC Area, 2017.
- The proposed eastern embankment will be at 9.3m elevation based on proposed of crest level under Dhaka Eastern Bypass Project, 2017.
- Proposed urban areas will be above average flood level based on the recommendation for level of land raising not below 5mPWD in the Eastern Bypass area under Dhaka Eastern Bypass Project, 2017.
- Other areas like water bodies, retention areas, and agricultural areas will be at the same elevation as of existing.

Applying those considerations, most of the low land areas would become elevated and about only 34% area would remain below the average flood level. The land developing companies and landowners are developing the low-lying areas by earth filling on a level similar to the average flood level. If the trend continues about 50% of the present water retention and water body areas will be elevated in future and water body/water retention areas will reduce to about 11% from 22%. The difference of elevation between future and present topography is shown in **Figure 4-7.**

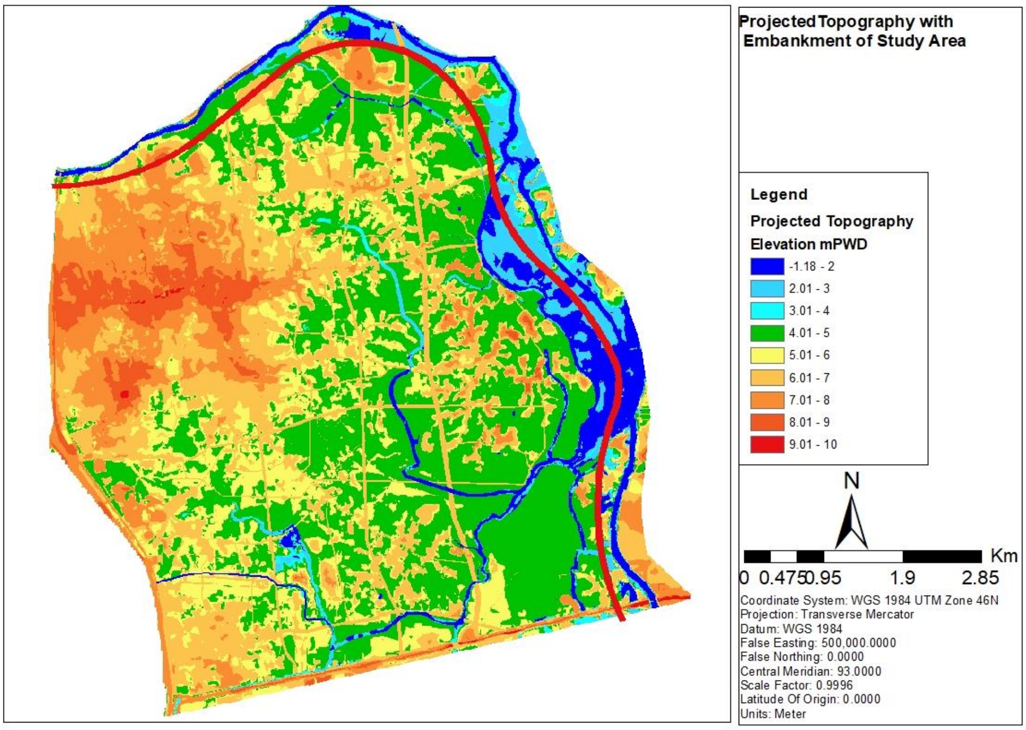


Figure 4-6**: Future topography of study area**

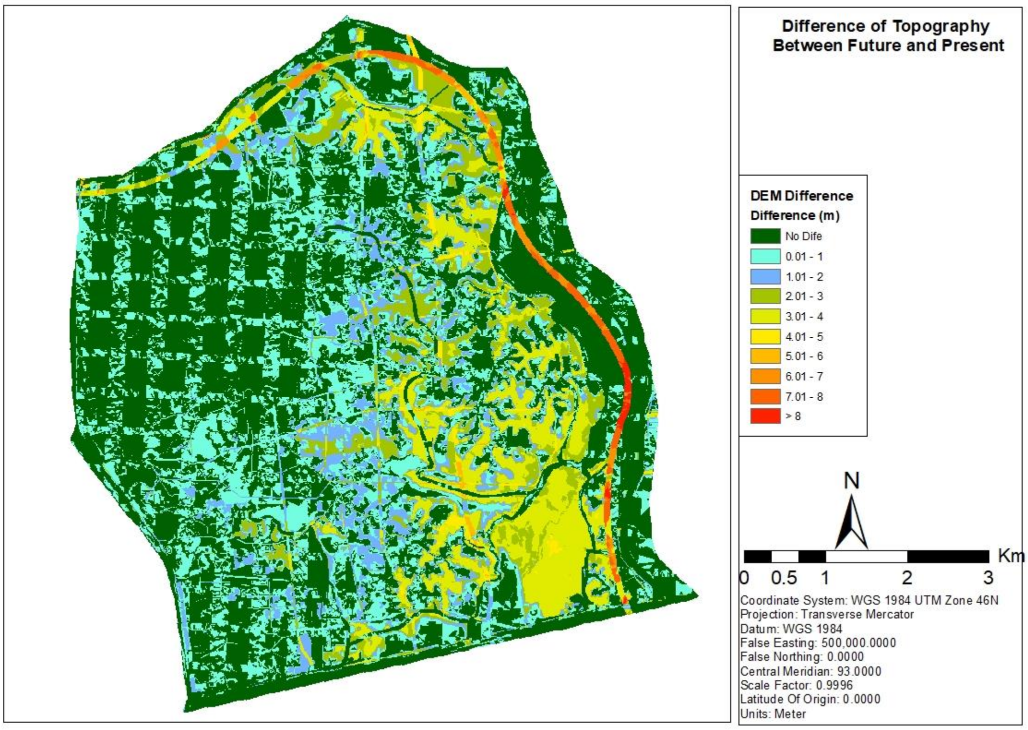


Figure 4-7**: Elevation Difference between Future and Present of study area**

4.2.2 Landuse

The existing and future landuse data from the Detailed Area Plan (DAP) of RAJUK has been collected and analyzed. The existing and planned landuse as per DAP are shown in **Figure 4-8** and **Figure 4-9.** According to RAJUK (DAP, 2010) most of the study area (about 94%) is mainly dedicated to three categories, cultivated land/open space (33.52%), residential (38.95%), and swamp/water body (21.65%). The remaining areas were functioning as transportation network, commercial and industrial activity, and public facilities areas. A small portion of the area was designated as mixed-use zone.

Under the planned landuse, the residential area has been proposed to increase significantly occupying more than 58% of the study area. Other prominent planned landuse categories are road networks (9%), water retention areas (7.4%), mixed-use zone (6%) and flood flow zone (5%) (Figure 4-9). Vacant lands and agricultural areas have been proposed to decrease due to significant increase in the residential area and mixed-use zone. Overall the impervious area is expected to increase significantly in the future.

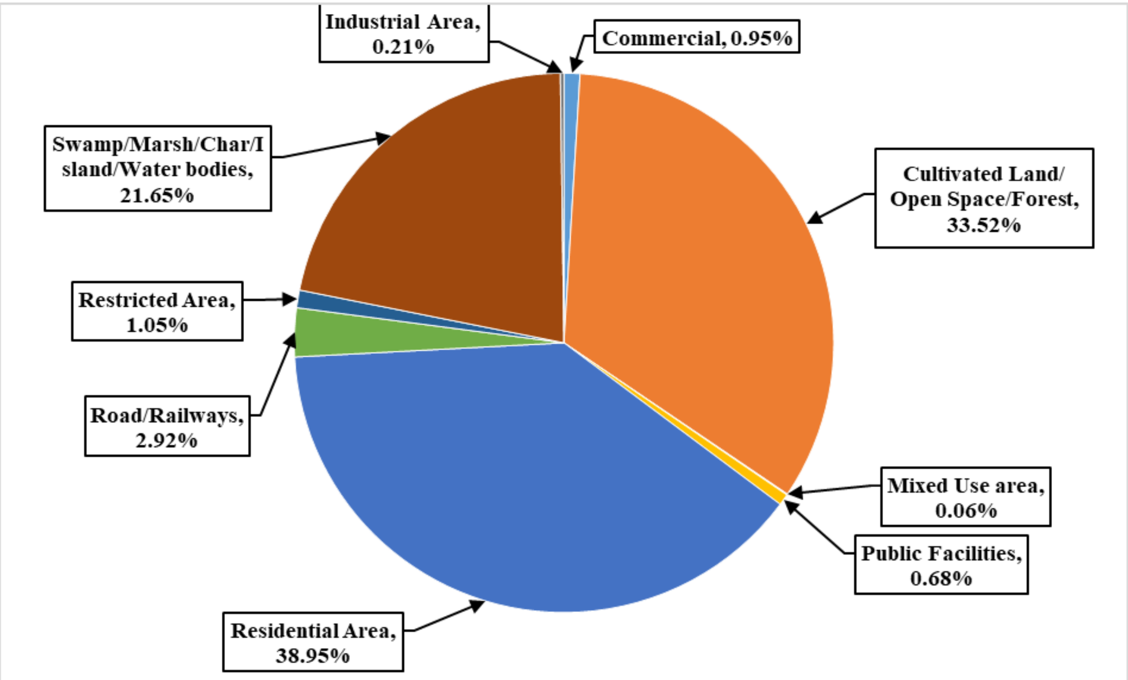


Figure 4-8 **: Existing Landuse Category of the Study Area (DAP, 2010)**

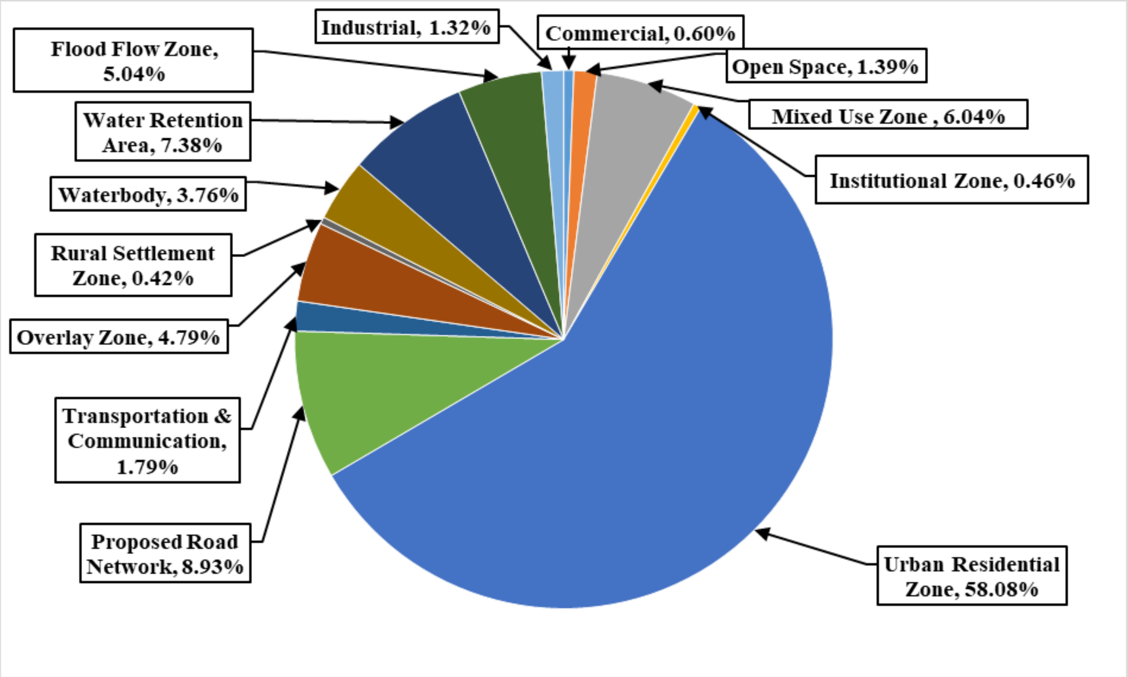


Figure 4-9: Proposed Landuse Category of the Study Area (DAP, 2010)

4.2.3 Rainfall

Rainfall data of Dhaka station at Banani (Station ID: 41923) under the Bangladesh Metrological Department (BMD) were used for the development of design storm and simulation of model scenarios. Historical Rainfall data from 1953 to 2020 have been used for various analyses. From 1953 to 2002 rainfall data frequency is daily whereas from 2003 to afterwards the data frequency is 3 hourly. For this study, 1-day 10-year and 1-day 25-year rainfall events have been selected for assessment of the existing network capacity and adequacy of the proposed internal drainage network. The historical maximum rainfall data has been shown in **Table 4:3.**

Table 4:3: Annual Maximum Rainfall for Dhaka (1953-2020)

Year	1-day Max	2-day Max	Year	1-day Max	2-day Max
1953	90	127	1988	135	175
1954	147	255	1989	118	151
1955	115	120	1990	94	133
1956	326	346	1991	123	167
1957	73	98	1992	90	91
1958	137	140	1993	140	183
1959	125	179	1994	74	126
1960	141	223	1995	83	153

Year	1-day Max	2-day Max	Year	1-day Max	2-day Max
1961	185	185	1996	150	177
1962	116	141	1997	121	147
1963	189	257	1998	122	223
1964	114	206	1999	141	169
1965	177	181	2000	158	259
1966	257	270	2001	71	123
1967	125	147	2002	88	115
1968	145	240	2003	93	111
1969	86	104	2004	341	497
1970	152	262	2005	128	246
1971	251	328	2006	185	224
1972	231	251	2007	152	216
1973	168	177	2008	106	146
1975	143	212	2009	333	422
1976	163	263	2010	87	114
1977	100	133	2011	94	173
1978	128	191	2012	62	74
1979	108	166	2013	122	157
1980	91	125	2014	75	92
1981	81	148	2015	90	150
1982	146	167	2016	75	94
1983	133	199	2017	149	235
1984	151	201	2018	82	116
1985	92	132	2019	81	147
1986	176	321	2020	89	130
1987	138	172			

The drainage system in Dhaka city is designed for a 1-day 5-year event, while the pumps are geared for a 2-day 5-year event. As a result, the system in most cases is unable to cope with the storms, resulting in urban flooding. Because extreme rainfall events are occurring more frequently, this study focuses on analyzing the performance of the proposed drainage system under 1-day rainfall with 10-year, 25-year, and 100-year return periods. To reduce and manage storm runoff in the future, various LID measures have also been examined and the drainage system was simulated and analyzed for 1-day 10-year, 25-year, and 100-year events.

4.2.3.1 Estimation of Design Rainfall for Different Return Periods

For estimation of design rainfall with different return periods the historical daily maximum rainfall data series Chi-Square goodness of fit test is performed to evaluate the accuracy of the fitting of different probability distributions. To identify the best fitted probability distribution Chi-Square test was performed using the Normal, Gumbel and Lognormal distributions method. The calculated and critical Chi-Square value is tabulated in **Table 4:4**.

Table 4:4: χ2 Value of Daily Rainfall Data for Different Methods

Distribution Method	χ2 Calculated	χ2 Critical DOF=2, α%=5%	Remarks	Distribution adopted for frequency analysis
Normal	20.06	5.99	Not accepted	Gumbel
Gumbel	4.04	5.99	Accepted	
Lognormal	4.83	5.99	Accepted	

Based on χ^2 test, Gumbel and Lognormal distributions were found within the critical value. Since the calculated value of χ^2 using the Gumbel is less than the Lognormal, therefore the Gumbel distribution is adopted for frequency analysis. **Table 4:5** shows the calculated rainfall depths with different return periods i.e., 2, 2.33, 5, 10, 25, 50 and 100 years are estimated by using the Gumbel distribution.

Table 4:5: Rainfall amount at a different return period

Return period	**2**	**2.33**	**5**	**10**	**25**	**50**	**100**
Rainfall	125.30	135.18	178.11	213.08	257.26	290.03	322.57

4.2.3.2 Development of IDF and Design Rainfall

Intensity duration frequency (IDF) curves (**Figure 4-10**) were developed for the rainfall station utilizing 67 years of data, and utilizing the IDF curves design 1-day rainfall patterns for different return periods (10-year, 25-year) were prepared. The design rainfalls were distributed using the Alternating Block Method.

While developing the IDF curves, to estimate rainfall intensities with respect to various duration like 10 min, 20 min, 30 min, 1-hr, 2-hr, 3-hr, 5-hr, 8-hr etc., the 1-day annual maximum values, were segregated using the empirical reduction formula recommended by the Indian Meteorological Department (IMD). The formula recommended by IMD is as follows:

$$P_t = P_{24}\sqrt[3]{\frac{t}{24}}$$

Where P_t is the required rainfall depth in mm at t-hr duration, P24 is the daily rainfall in mm and t is the duration of rainfall for which the rainfall depth is required in hour. Rasel et. al , 2015 followed this empirical formula to estimate short-duration rainfall in all seven divisions of Bangladesh. Gumbel's Extreme Probability Method was utilized to determine rainfall (P_T) values and intensities (I_T) by using these estimated different duration data. **Table 4:6** and **Table 4:7** show rainfall frequency (P_T) and intensity (I_T) values for various durations and return intervals.

Table 4:6: Computed frequency rainfall (PT) values for different durations and return periods using Gumbel Method

Duration, T_d (min)	T, Return Period (Years)						
	2	2.33	5	10	25	50	100
10	23.91	25.79	33.98	40.65	49.08	55.33	61.54
20	30.12	32.49	42.81	51.22	61.84	69.72	77.54
30	34.48	37.20	49.01	58.63	70.79	79.81	88.76
60	43.44	46.86	61.75	73.87	89.19	100.55	111.83
120	54.73	59.05	77.80	93.07	112.37	126.68	140.89
180	62.65	67.59	89.06	106.54	128.63	145.02	161.28
360	78.93	85.16	112.20	134.23	162.06	182.71	203.20
720	99.45	107.29	141.37	169.12	204.19	230.20	256.02
1440	125.30	135.18	178.11	213.08	257.26	290.03	322.57

Table 4:7: Computed intensities (IT) values for different durations and return periods using Gumbel Method

Duration, T_d (min)	T, Return Period (Years)						
	2	2.33	5	10	25	50	100
10	143.4	154.7	203.9	243.9	294.5	332.0	369.2
20	90.36	97.48	128.44	153.66	185.51	209.15	232.61
30	68.95	74.39	98.02	117.26	141.57	159.61	177.52
60	43.44	46.86	61.75	73.87	89.19	100.55	111.83
120	27.36	29.52	38.90	46.54	56.18	63.34	70.45
180	20.88	22.53	29.69	35.51	42.88	48.34	53.76

Duration, T_d (min)	T, Return Period (Years)						
	2	2.33	5	10	25	50	100
360	13.16	14.19	18.70	22.37	27.01	30.45	33.87
720	8.29	8.94	11.78	14.09	17.02	19.18	21.34
1440	5.22	5.63	7.42	8.88	10.72	12.08	13.44

After Determining the rainfall (PT) values and intensities (IT), IDF curves are developed **(Figure 4-10)** for different return periods. Finally, a general equation has been established which is mostly identical to generated IDF curve equation for 24 hours duration by Rimi, 2016.

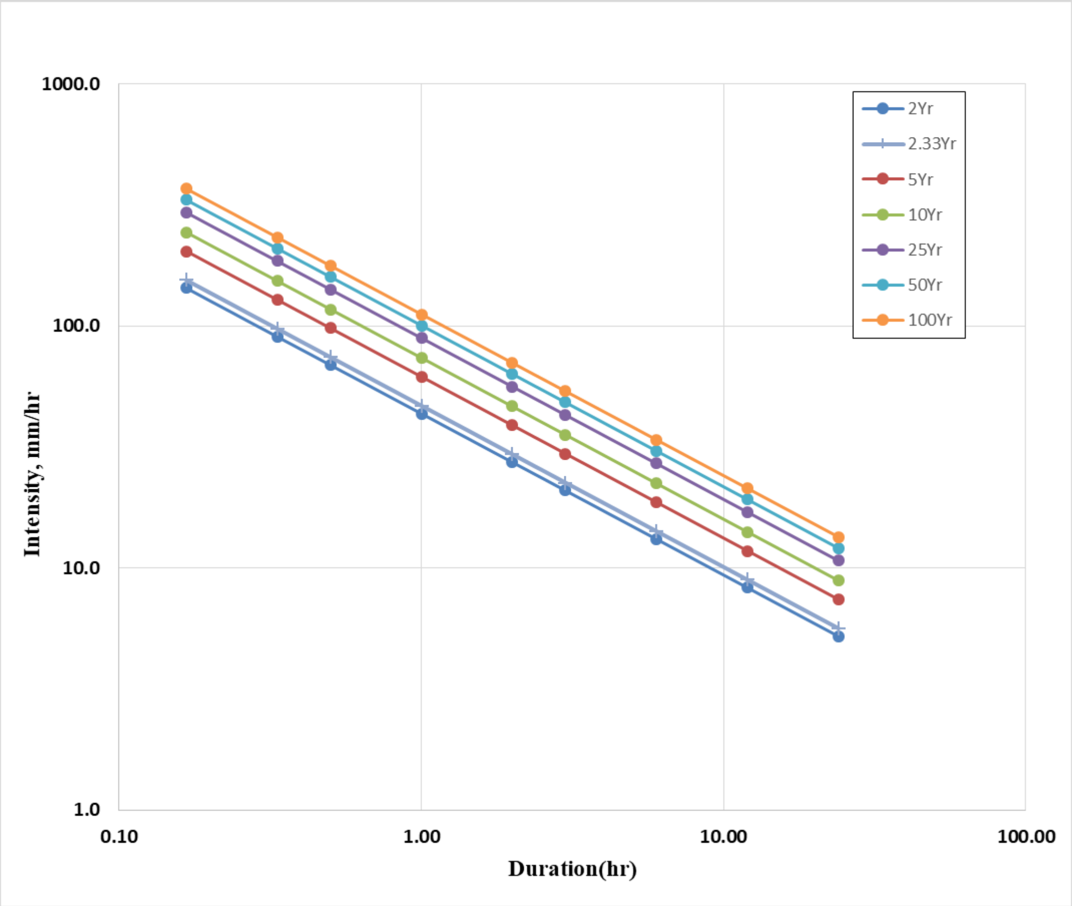


Figure 4-10**: Intensity Duration Frequency (IDF) curves for Dhaka**

The equation of IDF curve is found $I = \frac{39.50\ T^{0.2401}}{T_d^{\ 0.6667}}$

Where, I= Rainfall Intensity (mm/hr)

T= Design Return Period (Year)

Td= Rainfall Duration (Hour)

4.2.3.3 Analyzing Design Rainfall Pattern

The design rainfall pattern has been generated using the Alternating Block Method (ABM). The design rainfall pattern generated by this method specifies the precipitation depth occurring in 24 successive time intervals of duration 60 minutes over a total duration of 24 hours. For 1-day 10-year return period rainfall, the intensity is calculated from the IDF curve equation (Shown in **Section 4.2.3.2**) for every 60 minutes, and the corresponding precipitation depth is found as the product of intensity and duration. By taking differences between successive precipitation depth values, the amount of precipitation for each of the 60 minutes blocks has been calculated.

These increments, or blocks, are reordered into a time sequence with the maximum intensity occurring at the centre of the required duration of 12 hours and the remaining blocks arranged in descending order alternately to the right and left of the central block to form the design rainfall pattern. The calculation for the 1-day 10-year design rainfall distribution pattern has been given in **Table 4:8** and the hyetograph is shown in **Figure 4-11.**

Table 4:8: Alternating Block Method Calculation for 1-day 10-year design Rainfall

n	Δt (min)	Duration Td(hr)= n*Δt	Intensity mm/hr	Cumulative Depth, mm	Incremental Depth, mm	Rank	Time (hr)	Precipitation (mm)
1	60	1.00	68.67	68.67	68.67	1	0-1	2.87
2	60	2.00	43.26	86.52	17.85	2	1-2	3.06
3	60	3.00	33.01	99.04	12.52	3	2-3	3.27
4	60	4.00	27.25	109.00	9.97	4	3-4	3.53
5	60	5.00	23.48	117.42	8.42	5	4-5	3.85
6	60	6.00	20.80	124.77	7.36	6	5-6	4.25
7	60	7.00	18.76	131.35	6.58	7	6-7	4.77
8	60	8.00	17.17	137.33	5.98	8	7-8	5.50
9	60	9.00	15.87	142.83	5.50	9	8-9	6.58
10	60	10.00	14.79	147.93	5.10	10	9-10	8.42
11	60	11.00	13.88	152.71	4.77	11	10-11	12.52
12	60	12.00	13.10	157.20	4.49	12	11-12	68.67
13	60	13.00	12.42	161.45	4.25	13	12-13	17.85
14	60	14.00	11.82	165.49	4.04	14	13-14	9.97
15	60	15.00	11.29	169.34	3.85	15	14-15	7.36

n	Δt (min)	Duration Td(hr)= n*Δt	Intensity mm/hr	Cumulative Depth, mm	Incremental Depth, mm	Rank	Time (hr)	Precipitation (mm)
16	60	16.00	10.81	173.02	3.68	16	15-16	5.98
17	60	17.00	10.39	176.55	3.53	17	16-17	5.10
18	60	18.00	10.00	179.95	3.40	18	17-18	4.49
19	60	19.00	9.64	183.22	3.27	19	18-19	4.04
20	60	20.00	9.32	186.38	3.16	20	19-20	3.68
21	60	21.00	9.02	189.44	3.06	21	20-21	3.40
22	60	22.00	8.75	192.40	2.96	22	21-22	3.16
23	60	23.00	8.49	195.27	2.87	23	22-23	2.96
24	60	24.00	8.25	198.06	2.79	24	23-24	2.79

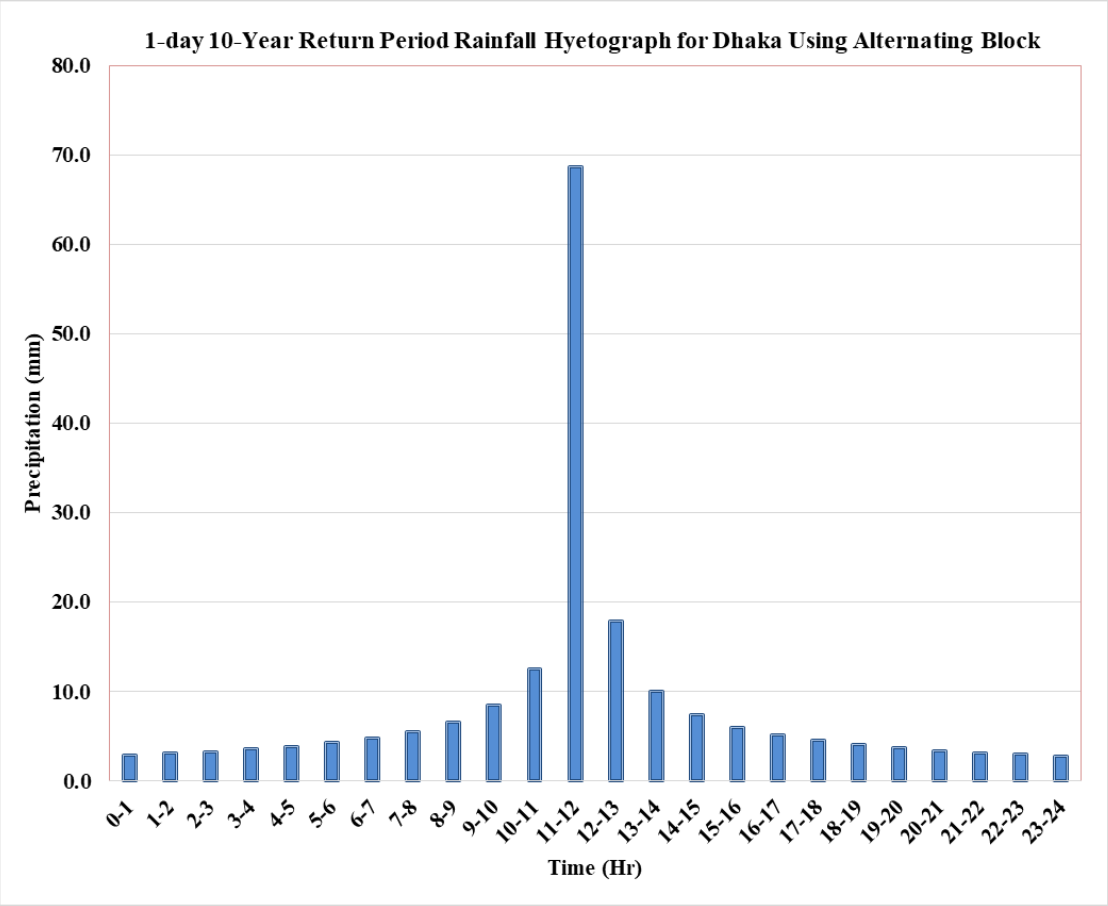


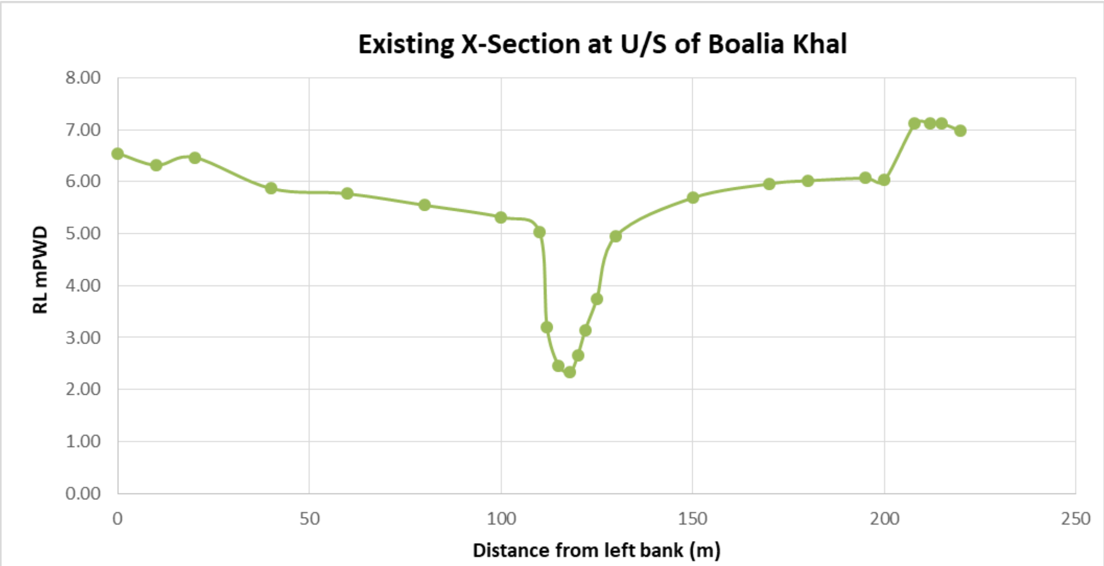
Figure 4-11**: 1-day 10-year design rainfall hyetograph using Alternating Block Method**

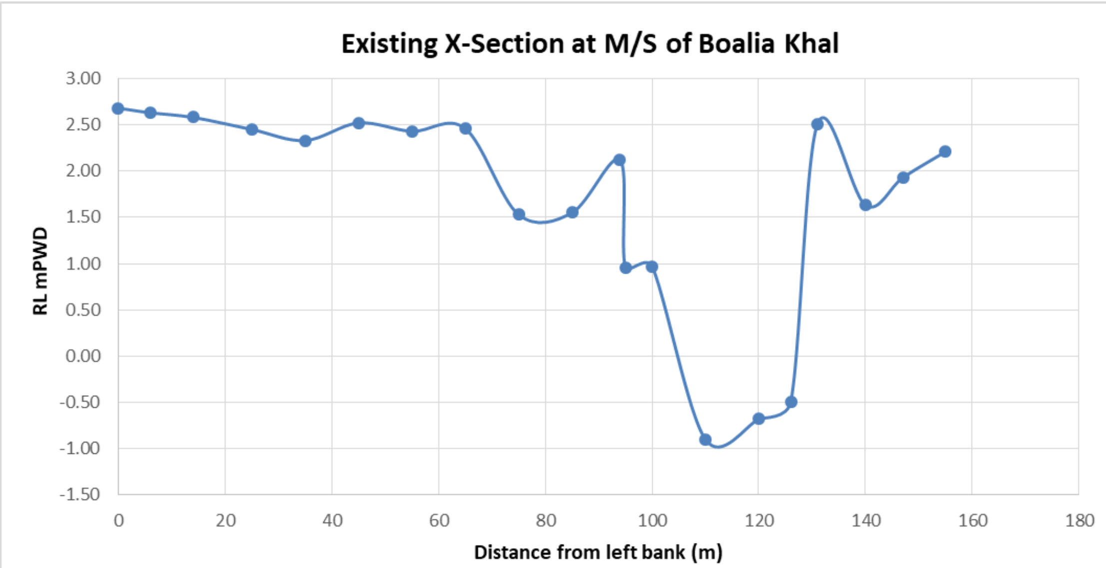
4.2.4 Evapotranspiration

The "Reduction factor" parameter in the MIKE URBAN model represents evapotranspiration loss in the computation of surface runoff. The evaporation data of the Dhaka station has been used as the proxy of evapotranspiration data while estimating the reduction factor. It was found that the annual average evaporation at Dhaka station accounts for about 9% of the annual average precipitation. Considering this, the reduction factor was assumed as 0.9 in this study.

4.2.5 Drainage Network Data

Drainage network data such as alignment & cross-section of khal, existing culvert, etc. have been collected from IWM. A Cross-section survey of all khals and the peripheral rivers (Balu River and Tongi khal) was conducted by IWM in the year 2016 which were utilized. Some of the same cross section of Boalia Khal are shown in **Figure 4-12**. For the Airport and Nikunja catchment, drainage network data was not available. Therefore, the estimated runoff of these catchments has been used as inflow to the system.





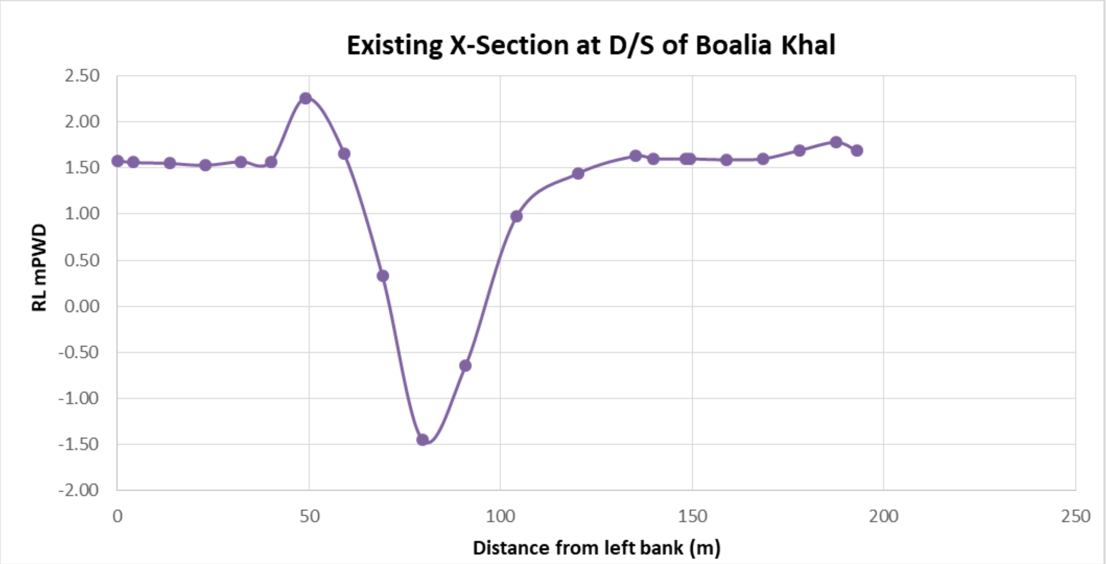


Figure 4-12: Existing Cross Section of Boalia Khal

4.2.6 Water Level

The water level data of the peripheral water level stations Demra (SW 7.5), Pubail (SW7), and Tongi (SW299) in the Balu River and Tongi khal have been collected from BWDB (1962- 2020). For different stations the available data range is different. From the available observed data, it is found that the highest flood level occurred in 1988 near Tongi Bridge and Demra in Balu River. Using data from the peripheral stations, frequency analysis was performed using the log Normal and Gumbel distribution methods as shown in **Table 4:6** to estimate water level for different return period events. These estimated water level values were used as boundary conditions under different scenarios.

Table 4:9: Frequency distribution of historical water level data (1962-2020)

River	Station	Method	Return Period in Years						
			2	2.33	5	10	25	50	100
Balu	Demra	3PLogNormal	5.81	5.90	6.22	6.44	6.66	6.83	6.97
		Gumbel EV1	5.75	5.83	6.17	6.45	6.78	7.07	7.33
Balu	Pubail	3PLogNormal	5.81	5.92	6.37	6.69	7.02	7.29	7.51
		Gumbel EV1	5.74	5.85	6.30	6.68	7.11	7.50	7.85
Tongi Khal	Tongi	3PLogNormal	5.98	6.09	6.54	6.86	7.19	7.46	7.68
		Gumbel EV1	5.91	6.01	6.48	6.86	7.30	7.69	8.05

4.2.7 Discharge

The Historical (1977- 2017) mean daily discharge data of Demra station (SW7.5) has been collected from the National Water Resources Database (NWRD), WARPO. Frequency analysis was conducted using those data by the log Normal and Gumbel distribution methods. The results of the frequency analysis are shown in **Table 4:10.** In addition, On the other hand, observed discharge data (1994-2013) from Demra station and Tongi station (2016-2020) has been collected from BWDB.

Table 4:10: Frequency distribution of mean daily discharge data of Demra Station

Method	**Return Period in Years**						
	2	**2.33**	**5**	**10**	**25**	**50**	**100**
3PLogNormal	398	422	525	604	691	766	832
Gumbel EV1	392	416	523	610	711	801	881

4.3 Hydrological Model Setup

For urban catchments, there are two distinct groups of hydrological models. One is the surface runoff Model: This is the most typical type of model of urban runoff analyses. Another is continuous hydrological models: these models handle the precipitation volume balance without any truncation using more or less complicated principles,

The so-called "Time-Area" method is the foundation of the MOUSE Runoff Model A is the surface runoff computation concept. The Time-Area Method (MOUSE model A) is a simple surface runoff model with minimum data requirements. The initial loss, the size of the contributing area, and a continual hydrological loss all are controlled by the runoff amount. The time-area (T-A) curve and the time of concentration both are controlled by the runoff hydrograph shape. The conceptual descriptions of the catchment reaction speed and catchment shape are represented by these two parameters.

4.3.1 Catchments Delineation

4.3.1.1 Existing Drainage Catchments

In hydrological modelling and watershed management, catchment delineation plays an important role. For the existing scenarios of the hydrological model, catchment delineation was done with the help of GIS analysis using the 2016 DEM. The delineation process includes filling depressions, calculating flow direction, flow accumulation, delineating streams with an accumulation threshold, defining streams, segmenting streams, delineating watersheds, processing watershed polygons, processing streams, and aggregating watersheds/ catchments. After accomplishing all processes the flow direction and 89 catchments have been delineated accordingly. The flow direction and existing catchments with major streamlines are shown in **Figure 4-13** and **Figure 4-14** respectively.

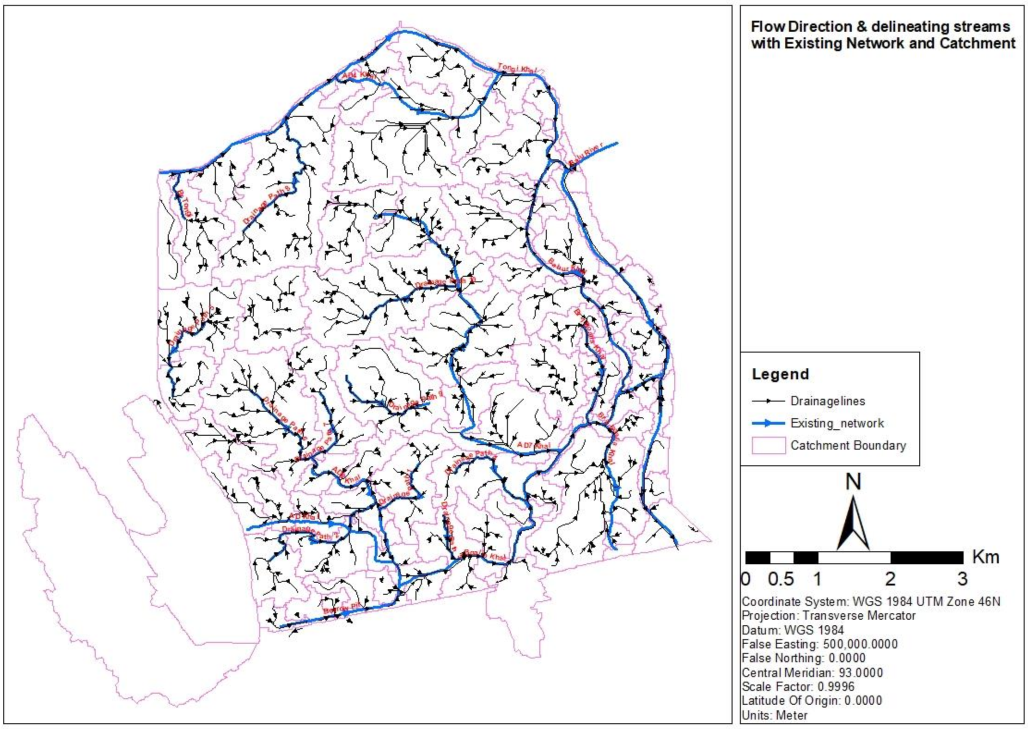


Figure 4-13: Flow Direction of the Existing catchments

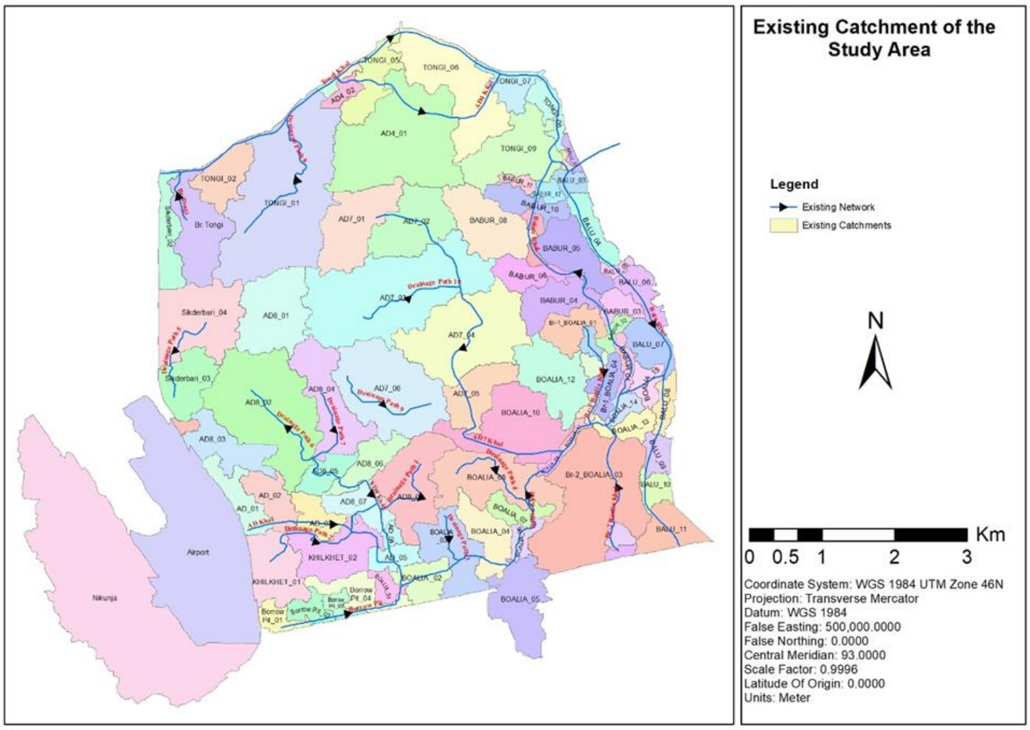


Figure 4-14: Existing catchment areas

4.3.1.2 Future Drainage Catchments

It was assumed that, except for the major drainage channels, the natural drainage system and catchments will no longer exist in the future. Many smaller natural drainage systems are likely to be converted to flow through pipes or roadside drainage systems due to landfilling, while the primary system will remain unchanged. In consideration of the aforementioned situations, a drainage system has been proposed by the DNCC, which has resulted in the re-delineation of catchments for future scenarios. In the DAP future landuse and DNCC proposed road alignment, the roadside drains were considered to transfer stormwater to the main drainage system. Considering the anticipated socio-economic development, a total of 108 future catchments were delineated. The anticipated drainage system and the catchment areas are shown in **Figure 4-15**.

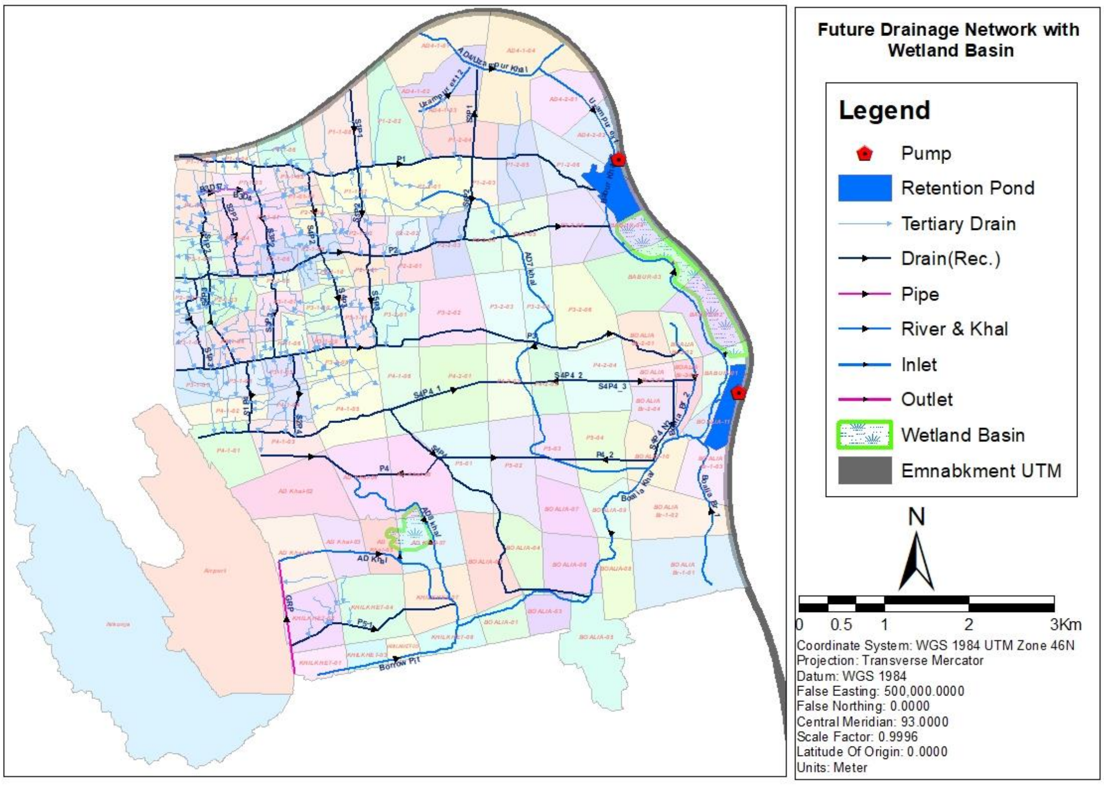


Figure 4-15: Future Catchment areas with an improved drainage system

4.3.2 Catchment Parameters

4.3.2.1 Imperviousness and Initial Loss

Imperviousness is one of the major factors influencing the amount of runoff generated from any storm. The imperviousness of each existing and future catchment has been calculated based on landuse data of DAP and projected population density for the years

2016 and 2050 respectively. This method was developed under the World Bank, 2015 research project (Climate and Disaster Resilience of Greater Dhaka Area: A Micro Level Analysis) for Dhaka. Based on the various types of land cover identified in each catchment, the percentages of the pervious and impervious area were estimated for the study area, using typical values in the field (i.e., the actual percent values corresponding to a particular land cover) as shown in **Table 4:11**. For example, in the residential-use category, the percentage of impervious areas in low-density areas varies from 18 percent to 34 percent as the population density increases from 10,000 people per km^2 to 20,000 per km^2. In high-density areas, the percentage of impervious areas varies from 60 percent to 70 percent as the population density increases from 40,000 people per km^2 to 50,000 per km (**Table 4:11**). In this way, the relationship captures the time dimension for future scenarios as well.

Table 4:11: Impervious Percentage for Various Landuse

DAP land Type	Density of Population (in thousand per km^2)								
	10	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
	Low Density			Moderate Density			High Density		
	Impervious Area by Land Type (in percent)								
Agriculture	0	1	1	1	1	1	1	1	1
Circulation Network	2	3	4	4	5	6	6	7	7
Commercial Activity	19	27	35	44	50	56	63	68	73
Community Service	21	30	39	48	55	62	69	74	80
Diplomatic	9	14	18	22	25	28	31	34	36
Education & Research	11	16	21	26	30	34	38	41	44
Governmental Services	9	14	18	22	25	28	31	34	36
Manufacturing and Processing Activity	15	22	28	35	40	45	50	54	58
Mixed Use	21	30	39	48	55	62	69	74	80
Recreational Facilities	4	5	7	9	10	11	13	14	15
Residential	18	26	34	42	48	54	60	65	70
Restricted Area	2	3	4	4	5	6	6	7	7
Service Activity	9	14	18	22	25	28	31	34	36
Transport & Communication	60	65	70	75	80	85	90	95	100
Vacant Land	2	3	4	4	5	6	6	7	7
Water Body not connected to drains	0	1	1	1	1	1	1	1	1
Water Body connected to drains	100	100	100	100	100	100	100	100	100
Historical	9	14	18	22	25	28	31	34	36

Source: World Bank, 2015. Landuse: DAP, 2010, Detailed Area Plan.

The percentage of imperviousness for the existing catchment has been given in **Table 4:12** and for future catchment in **Table 4:13.** The minimum and maximum impervious for existing catchment are 12.8% and 83.1% respectively and the weighted average imperviousness is 40.5%. The percentage of imperviousness for the future catchments varies from 58 to 98 and the weighted average imperviousness is 79.50 were calculated.

Table 4:12: Impervious Area for existing land-use patterns

Sl. No.	Catchment Name	% Impervious	Sl. No.	Catchment Name	% Impervious
1	AD_01	70.3	46	BOALIA_01	41.8
2	AD_02	50.4	47	BOALIA_02	55.0
3	AD_03	39.5	48	BOALIA_03	41.3
4	AD_04	22.2	49	BOALIA_04	38.1
5	AD_05	40.3	50	BOALIA_05	23.2
6	AD4_01	43.0	51	BOALIA_06	29.3
7	AD4_02	23.6	52	BOALIA_07	31.7
8	AD7_01	34.2	53	BOALIA_08	39.3
9	AD7_02	23.7	54	BOALIA_09	52.8
10	AD7_03	31.3	55	BOALIA_10	29.1
11	AD7_04	38.4	56	BOALIA_11	51.9
12	AD7_05	65.4	57	BOALIA_12	46.6
13	AD7_06	53.8	58	BOALIA_13	16.8
14	AD8_01	39.4	59	BOALIA_14	29.7
15	AD8_02	41.1	60	BOALIA_15	38.3
16	AD8_03	63.5	61	Borrow Pit_01	54.1
17	AD8_04	43.4	62	Borrow Pit_02	36.3
18	AD8_05	38.3	63	Borrow Pit_03	40.6
19	AD8_06	28.4	64	Borrow Pit_04	58.6
20	AD8_07	53.1	65	Br. Tongi	39.1
21	AD8_08	49.7	66	Br-1_BOALIA_01	27.3
22	Airport	59.0	67	Br-1_BOALIA_02	48.2
23	BABUR_01	58.6	68	Br-1_BOALIA_03	68.3
24	BABUR_02	28.5	69	Br-1_BOALIA_04	44.0
25	BABUR_03	12.8	70	Br-1_BOALIA_05	25.5
26	BABUR_04	27.9	71	Br-2_BOALIA_01	35.3
27	BABUR_05	14.6	72	Br-2_BOALIA_02	18.6
28	BABUR_06	31.0	73	Br-2_BOALIA_03	22.0
29	BABUR_07	42.9	74	KHILKHET_01	47.8
30	BABUR_08	36.3	75	KHILKHET_02	52.2
31	BABUR_09	71.9	76	Nikunja	46.8
32	BABUR_10	83.1	77	Sikderbari_01	73.1
33	BABUR_11	56.5	78	Sikderbari_02	43.2

Sl. No.	Catchment Name	% Impervious	Sl. No.	Catchment Name	% Impervious
34	BABUR_12	79.9	79	Sikderbari_03	35.8
35	BALU_01	19.6	80	Sikderbari_04	38.5
36	BALU_02	46.1	81	TONGI_01	36.1
37	BALU_03	26.7	82	TONGI_02	36.5
38	BALU_04	43.1	83	TONGI_03	46.6
39	BALU_05	21.9	84	TONGI_04	51.3
40	BALU_06	24.1	85	TONGI_05	34.0
41	BALU_07	18.8	86	TONGI_06	35.3
42	BALU_08	41.6	87	TONGI_07	69.0
43	BALU_09	31.2	88	TONGI_08	79.5
44	BALU_10	33.7	89	TONGI_09	54.4
45	BALU_11	45.1			

Table 4:13: Impervious Area for Future landuse patterns (Future catchments)

Sl. No.	Catchment Name	% Impervious	Sl. No.	Catchment Name	% Impervious
1	AD Khal-02	76.81	55	P2-1-04	76.48
2	AD Khal-06	77.14	56	P2-1-09	74.06
3	AD Khal-04	76.48	57	P1-1-02	83.87
4	Airport	74.90	58	P1-1-04	79.88
5	BOALIA Br-1-01	82.84	59	P1-1-03	68.94
6	BOALIA Br-1-03	83.74	60	P2-1-06	77.02
7	BOALIA Br-1-02	94.84	61	P2-1-02	78.07
8	BOALIA Br-2-05	87.02	62	P1-1-05	80.21
9	BOALIA Br-2-02	80.11	63	P1-01-01	76.67
10	BOALIA Br-2-01	86.98	64	P1-2-02	77.59
11	BOALIA Br-2-03	86.47	65	P1-1-07	77.65
12	BOALIA Br-2-04	75.80	66	P1-2-01	78.37
13	BABUR-04	93.84	67	P1-2-03	88.73
14	BABUR-01	95.34	68	P1-1-08	68.47
15	BABUR-02	93.64	69	P1-2-04	82.05
16	BABUR-03	88.26	70	P1-2-06	81.20
17	KHILKHET-04	75.55	71	P1-2-05	87.88
18	BOALIA-03	82.46	72	AD4-2-01	80.69
19	BOALIA-01	82.19	73	AD4-2-02	83.72
20	AD Khal-07	87.69	74	P3-1-03	80.82
21	KHILKHET-06	80.60	75	P3-1-10	73.12
22	BOALIA-04	78.31	76	P3-1-02	78.47
23	BOALIA-06	77.34	77	P3-1-04	74.77
24	KHILKHET-05	78.00	78	P2-1-08	77.22
25	KHILKHET-03	77.95	79	P3-1-07	75.28
26	KHILKHET-02	77.44	80	P3-1-06	73.37

Sl. No.	Catchment Name	% Impervious	Sl. No.	Catchment Name	% Impervious
27	KHILKHET-01	80.37	81	P2-1-03	75.01
28	BOALIA-02	77.43	82	P2-1-01	79.40
29	AD Khal-05	79.16	83	P2-1-10	74.17
30	BOALIA-05	86.14	84	P2-1-05	75.63
31	AD Khal-01	82.29	85	P3-1-09	76.82
32	AD Khal-03	77.53	86	P2-1-12	74.25
33	KHILKHET-07	78.80	87	P2-2-03	79.41
34	Nikunja	76.09	88	P2-2-04	79.99
35	P3-1-01	87.89	89	P2-1-11	76.64
36	P4-1-05	74.86	90	P2-2-01	80.35
37	P3-2-01	79.68	91	P2-2-02	78.82
38	P4-1-02	80.73	92	P2-2-05	81.43
39	P3-1-05	75.43	93	P2-2-06	79.66
40	P4-2-02	77.44	94	P3-2-06	85.94
41	P4-1-03	78.76	95	P4-2-04	83.10
42	P4-1-01	58.24	96	P3-2-05	82.08
43	P3-1-11	73.59	97	P4-2-03	80.36
44	P4-1-06	78.61	98	BOALIA-10	88.50
45	P3-1-08	76.98	99	P5-04	78.57
46	P4-1-04	67.31	100	P5-03	80.31
47	P3-2-02	77.67	101	BOALIA-07	77.05
48	P3-2-03	77.38	102	BOALIA-09	79.83
49	P4-2-01	78.23	103	BOALIA-08	81.96
50	P5-02	76.32	104	BOALIA-11	97.92
51	P5-01	76.84	105	AD4-1-02	79.42
52	P2-1-07	75.09	106	AD4-1-04	79.29
53	P1-1-06	62.62	107	AD4-1-03	78.74
54	P1-1-01	76.88	108	AD4-1-01	78.74

For the parameter "initial loss", default values (10 mm) in the model were considered for each catchment and were later optimized during the model calibration.

Table 4:14: Typical Parameters and Values for MIKE Urban Model A

Parameter	Value Used
Impervious Area (%)	70
Time of concentration (minutes)	120
Initial loss (mm)	10
Reduction factor	0.9
Time area curve no.	Varies with the catchment shape

4.3.2.2 Time of Concentration (T_c)

The study area was divided into 89 sub-catchments according to existing landuse patterns and drainage networks. The minimum and maximum time of concentration

(T_c) for the existing catchment are 12 minutes and 106 minutes respectively and the weighted average T_c is 59.7 minutes. During the calculation of existing catchment T_c, Horton's and Manning's roughness coefficients were considered as 0.035.

On the other hand, future catchments were delineated according to the proposed drainage networks and landuse. The T_c for the future catchments varies from 11 to 48 minutes and the average T_c is 18 minutes were calculated. During the calculation of future catchment T_c, Horton's and Manning's roughness coefficients were considered as 0.015.

4.4 Hydraulics Model Setup

4.4.1 Physical Properties of Drainage Network

In existing scenarios, about 44 km of natural drainage network has been considered as existing drainage network, these include 6.71 km Balu River, 7.66 km Tongi khal, and 29.7 km of internal natural khal. In addition, about 15 km of major streamlines were also considered during the model simulation. The existing drainage network has been shown in **Figure 4-16**. In 29.7 km natural drainage khal the Boalia khal and its branches are about 9.2 km. For the cross-section of khals, survey data from the eastern bypass project of BWDB have been used in existing scenarios. The cross-sections have been selected in such a way that the correct alignment and description of the khal bathymetry are represented in the model. The cross-section of streamlines is created using the DEM 2016.

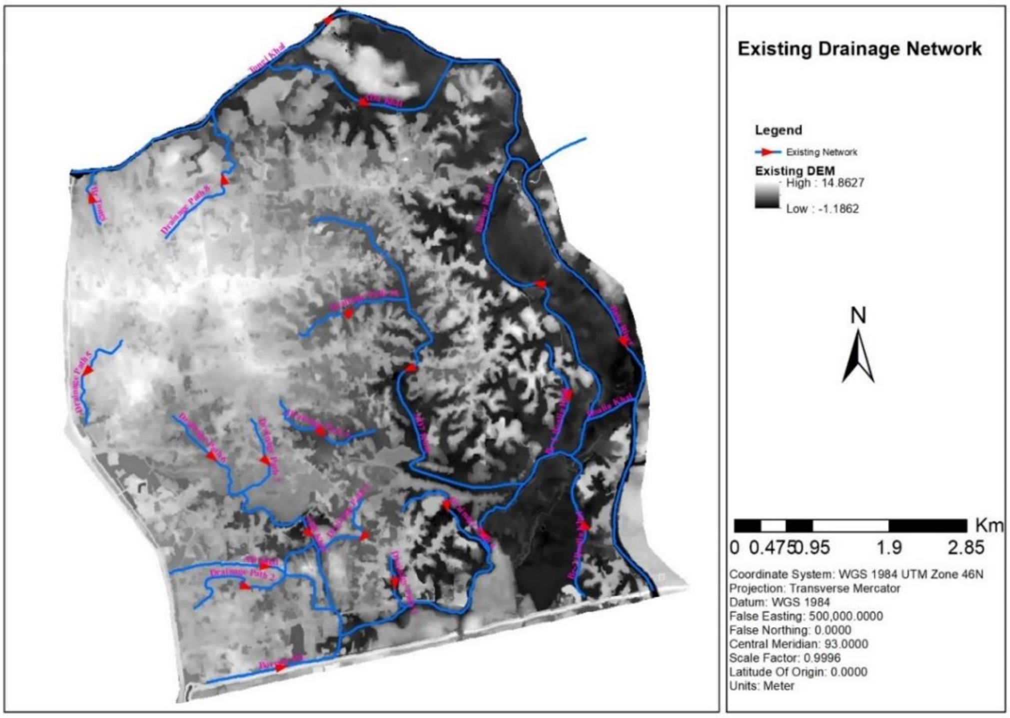


Figure 4-16: Existing Drainage System

In future drainage networks, along with internal khal, DNCC planned primary and secondary roadside drains have been included. The Future drainage network has been shown in **Figure 4-17**. In the future model, about 0.72 km circular pipe, 2 nos GRP pipe from Nikunja-1 to upstream of AD khal, about 50 km RCC rectangular roadside drain, and about 29.7 km drainage khals have been used. For the internal drainage, khal cross-sections have been changed as per the proposed DNCC plan.

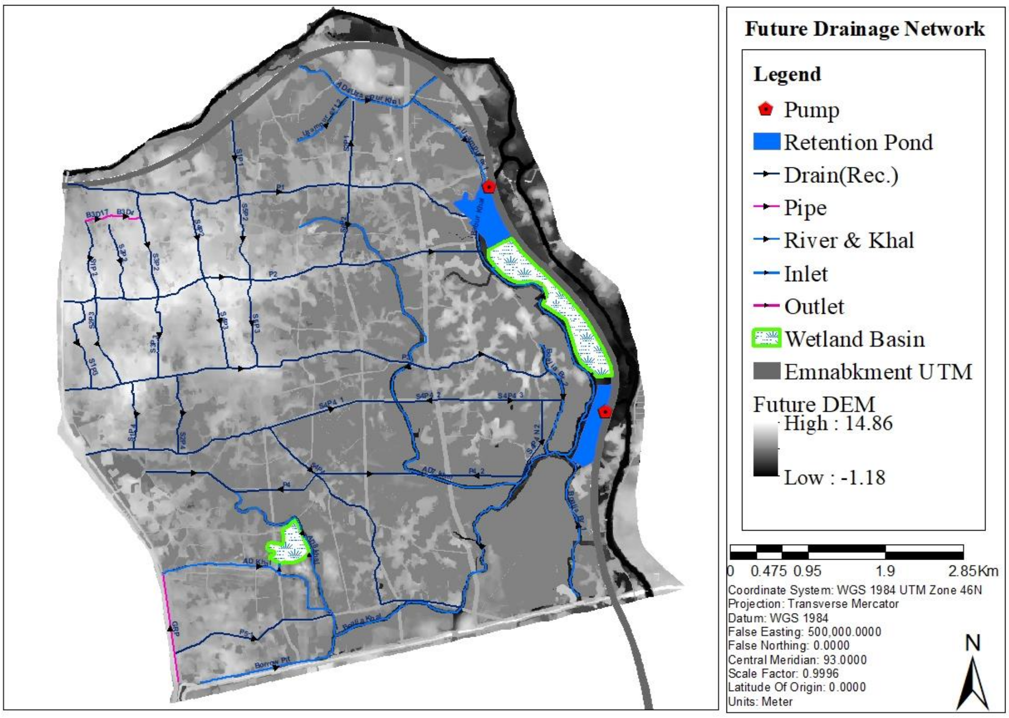


Figure 4-17: Future Drainage System (IWM, 2018 for DNCC)

4.4.2 Reservoirs and Retention Pond Properties

Four (04) nos existing reservoirs/ponds have been considered in the one-dimensional (1D) calibration and validation model. These ponds are located upper portion of the Boalia khal catchment (shown in **Figure 4-18**). In the 1D model these were represented by a storage basin and connected to the natural khal using a dummy link and overflow wire. During the 1D-2D coupling simulation of existing scenarios, these ponds acted as a local depression in the DEM. According to the DAP future landuse, three out of four reservoirs/ponds were not declared as water bodies (see **Figure 2-5**), so during the preparation of future DEM these ponds have been filled up.

Figure 4-18: Existing reservoir/pond

4.4.3 Embankment Properties

A flood embankment (**Figure 4-17**) starting from the Tongi to Demra has been considered following the alignment suggested in the both Halcrow, 2006 and the Eastern Bypass study IWM, 2017. The crest level of the embankment was designed at 9.3 mPWD. In fixing the crest level of the embankment 100-year flood level has been considered along with 1m for freeboard including confinement effect and an additional allowance of 0.25m for climate change.

4.4.4 Pump Properties

In order to drain out stormwater after applying of Eastern embankment, two pumps station have been considered in this study. The location and capacity of pumps have been applied as per the recommendation of the study of "Flood Control and Drainage Development at Dhaka Circular Road (Eastern Bypass) Project" conducted by IWM, 2017 under BWDB. One pump is located at the outfall of Boalia khal with a capacity of 42 m^3/s. Another One is located at the outfall of Babur khal and capacity of 34 m^3/s (**Figure 4-17**). The pump's efficiency of 80 % has been used for all scenarios. When the water level inside the embankment reaches 4.0 mPWD pump will start running and would stop when it goes down to 3.8 mPWD. So in the retention areas, there would be 1 to 1.5m storage level if the maximum allowable level is considered 5.0 mPWD or 5.5 mPWD respectively.

4.4.5 LID Properties

4.4.5.1 Rooftop Rainwater Harvesting

Rainwater harvesting has been designed for considering the water demand for outdoor usages such as gardening, car washing, and garage/front space washing. RWH has been applied in this study considering dual benefits, one is potable water savings and the other is runoff reduction. For RWH, fifty (50) percent (according to RAJUK Maximum Ground Coverage (MCG) 50% - 67.5%) area of the urban residential zone and the industrial zone have been considered as rooftop areas. 50% of the residential and industrial zone which are areas greater than 300 m^2 have been assumed under rainwater harvesting. It is mentioned here that, according to BNBC, 2021, every building proposed for construction on plots having an extent of 300 m^2 or above shall have facilities for conserving and harvesting rainwater. For calculating collection volume, a flat roof has been considered with a runoff coefficient of 0.8, which means that 70% of the rain can be harvested.

To determine the volume of rainwater storage based on consumption rates and building occupancy the Demand Side Approach (DSA) method has been used.

Rainwater storage volume in m^3 = D x N x D_p + Floating

Where,

D = Rainwater demand in litres per capita per day = 26 liters/C/Day

N = Population number =Number of Household x Household Size

For one 300 m^2 roof area 5 number of the household were considered and as per BBS household size is 4.5.

D_p= Number of days for which water will be stored. Consider 90 days for drinking, cooking, utensils cleansing, bathing and ablution purposes; 210 days for other purposes. Considering the 210 days, rainwater storage volume= 122.85 m^3.

The estimation of rainwater harvest has been calculated based on the average rainfall (mm) of the period of 1953- 2020 (**Table 4:15**).

Table 4:15: Estimation of Rainwater Harvest for a building (300 m^2)

Month	Average Rainfall(mm) for the years 1953- 2020	Effective rainfall (70%)	Rainfall harvest (m³)	Cumulative rainfall harvest (m³)	Demand (based on total use (m³/month)	Difference between columns (4) & (6)
1	*2*	*3*	*4*	*5*	*6*	*7*
Jan	6.82	4.78	1.15	1.15	17.55	-16.40
Feb	21.15	14.81	3.55	4.70	17.55	-14.00
Mar	55.39	38.77	9.31	14.01	17.55	-8.24
Apr	142.17	99.52	23.88	37.89	17.55	6.33
May	287.90	201.53	48.37	86.26	17.55	30.82
Jun	329.34	230.54	55.33	141.59	17.55	37.78
Jul	393.62	275.53	66.13	207.71	17.55	48.58
Aug	300.37	210.26	50.46	258.18	17.55	32.91
Sep	304.39	213.07	51.14	309.31	17.55	33.59
Oct	171.04	119.73	28.73	338.05	17.55	11.18
Nov	28.56	19.99	4.80	342.85	17.55	-12.75
Dec	10.56	7.39	1.77	344.62	17.55	-15.78
Total	**2051.3**	**1435.9**	**344.62**		**210.60**	

It is observed that May to September rainwater can be stored. The sum of harvested rainwater in 5 months (May-Oct) is 271.42 m^3. Rainwater demand for one 300 m^2 roof area is 0.59 m^3/day.

Volume to be harvested per day=0.59+122.85/150=1.40 m^3/day.

4.4.5.2 Soak-Away Pit

A soak–away pit is a good choice to treat and infiltrate runoff from rooftop downspouts. The urban residential zone and industrial zone in the future landuse have been considered for soak–away pits and it has been designed to combine with rooftop rainwater harvesting. A soak–away pit area can be occupied 4% of the total contributing area. The calculation of the targeted infiltration volume of a typical soak–away pit is given in **Table 4:16**.

Table 4:16: Estimation of targeted infiltration volume of a typical soak–away pit for a rooftop (1900 ft2)

Items	Unit	Calculation	Notes
Step 1: Calculate the Target Infiltration Volume			
Contributing Area, A	(ft^2)	1900	Should be less than 1,900 feet^2
Target Infiltration Rainfall, P	(in)	1.72	1–Year, 24–Hour Rainfall Depth (62.2 mm & 70% effective rainfall)

Items	Unit	Calculation	Notes
Runoff Coefficient, C		0.7	
TIV = A*P*C/12	(ft^3)	190.08	
	(m^3)	5.39	
Step 2: Calculate the Depth of the Pit (Must be between 4 and 10 feet)			
Void Ratio, ns		0.4	0.4 is typical of 1.5 to 3 in stone
Design Infiltration Rate, I	(in/hr)	0.675	Based on the site investigation
Retention Time, t	(hr)	48	Must be between 24 to 72 hours
Ds = (I*t)/(ns*12) + 2	(ft)	8.75	
Step 3: Calculate the Soak–Away Pit Footprint (Must be less than 64 $feet^2$)			
TIV (from Step 1)	(ft^3)	190.08	
ns (from Step 2)		0.4	
Ds (from Step 2)	(ft)	8.75	
As = (TIV*0.66)/(ns*(Ds–2))	(ft^2)	46.48	

4.4.5.3 Porous pavements

Porous pavement technology is an approach to lower the overall imperviousness of an area. Under this study, 25% area of the Transport & Communication and Proposed Road Network of the DAP future landuse has been assumed as porous pavements. Surface drainage permeable asphalt pavement (SPAP) road type (**Figure 4-19**) was considered in this study. The plan of road by IWM (2018) for DNCC, edge drainage system is adopted. According to Chen et. al., (2021), SPAP can effectively decrease total runoff depth and performed a considerable delaying effect on runoff time. SPAP performed a better effect on runoff time under weak rainfall intensity because of its excellent water storage capacity. Chen et. al. (2021) also refer reduction percentage of runoff coefficient on different permeable pavements against recurrence periods shown in **Table 4:17**. Rainfall permeated the PP through the road surface, then penetrated the drainage system through the crown slope, and finally flowed into the urban drainage pipe network. Permeable material was only used on the upper surface layer or the entire surface layer (**Figure 4-20**), and dense-graded material was applied for other layer. In this study, the runoff coefficient of Transport & Communication and Proposed Road Network landuse were reduced by 49.12% and 42.08% for 10 and 25-year return period rainfall respectively. Therefore, the weighted average imperviousness of future drainage catchments was decreased by about 5.8%.

Table 4:17: Reduction percentage of runoff coefficient on different permeable pavements

Type of surface drainage permeable pavement	Rain recurrence period (year)					
	1	**2**	**5**	**10**	**20**	**50**
The Control Group						
SPAP-I	46.67%	32.10%	23.62%	19.65%	16.87%	14.14%
SPCC	96.29%	69.69%	51.20%	43.61%	36.52%	30.74%
SPAP-II	100.00%	80.25%	59.00%	49.12%	42.08%	35.35%
SPCC-AP	100.00%	100.00%	74.82%	62.25%	53.29%	44.77%

Source: Chen et. al. (2021)

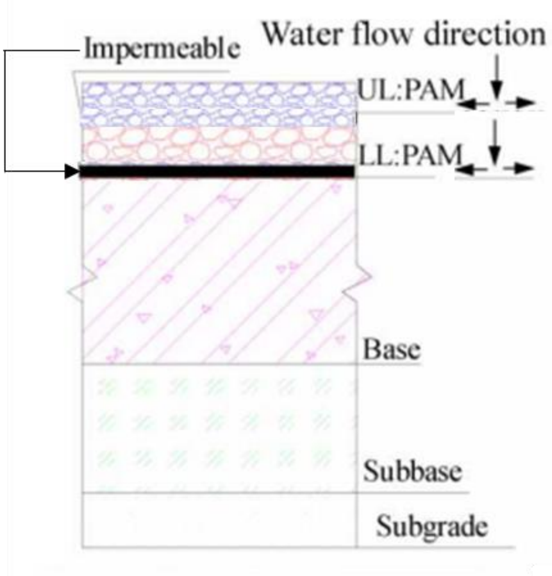


Figure 4-19: Typical Surface Drainage Permeable Asphalt Pavement Structures (Chen et. al., 2021)

Figure 4-20: Permeable Asphalt Paving Systems (TRUEGRID Paver, USA)

4.4.5.4 Wetland basin

Constructed wetlands are designed and built to temporarily store stormwater runoff with a permanent pool up to 2.0 mPWD. It can provide value in terms of natural aesthetics, recreational facilities, flood control, and pollutant removal as well.

Constructed wetland design requires a thorough understanding of basic hydrologic and hydraulic principles, and wetland mechanics. Forebay provides the opportunity for larger particles to settle out. Under this study, two locations have been selected as wetland basins. One upstream of Boalia khal has an area of 15.34 Hectares and another downstream of area 56.0 hectares. These have been done by considering an embankment with a spillway around the boundary. This structure around the basin can serve the purpose of retaining excess water. The basins were connected with a drainage canal by controlled inlet and outlet channels of size 5mx5m. The invert level of the inlet and outlet are fixed by the model simulation and analysis of flood depth as well as pumping duration.

4.4.5.5 Simulation Setup

MOUSE simulations editor is split into the following five tab sheets: General, Runoff parameters, Network parameters, Network Summary and 2D overland parameters. On the General page, the following is specified: Name of simulation (equal to the name of the result file), Simulation period and Simulation type.

For the Runoff Simulation T-A Curve (A) model has been selected among available runoff models types such as Kinematic Wave (B), Model C1, Model C2, RDI (solo), RDI +B, UHM and fixed time step has been considered as 60 sec. The time step during the wet and dry periods is considered as 60 and 300 sec respectively. On the other hand, for Network and Network+ 2D overland simulation Dynamic Wave model has been applied. The minimum and maximum time steps for network parameters are considered 10 and 60 sec respectively and the time step for 2D overland parameters is considered 0.5 to 1.0 sec.

4.4.6 Boundary Conditions

4.4.6.1 Rainfall

"Rainfall" is a boundary condition for the type of catchment boundary. This type of boundary is used for precipitation-runoff hydrological calculations. To simulate the calibration and validation scenarios BMD observed rainfall from May to September 2016 was used as a catchment boundary. In the other scenarios, 1-Day 10-year and 25-year design rainfall have been used as catchment boundaries.

4.4.6.2 Water level and Discharge

Types of variables associated with MOUSE network (i.e node/grid point) boundary conditions are: Water level and Flow (discharge). The water level boundary conditions are termed as "External Water Level" and the discharge boundary conditions are termed "Network Loads".

The observed water level of Balu River at Demra and Sinanghata was used as an “External Water Level” to simulate the calibration and validation model. The rated discharge of Tongi khal at Tongi and the discharge at Pubail of Balu River taken from the North Central Regional Model (NCRM) were used as an upstream boundary. It is mentioned here that the rated discharge of Tongi is found by creating a rating curve using observed discharge and water level.

During the future scenarios simulation considering the embankment cum eastern bypass, Flood-controlling structures i.e. sluice gates remained closed for all scenarios. Thus, the river water level will have no influence on the flooding and was not used as an external boundary. The water body and retention pond of all pump stations water level of 3.0 mPWD were used as initial boundary conditions for all scenarios.

4.5 Simulation Case or Scenarios

For scenarios SC1 and SC2 existing landuse pattern has been used. For future scenarios, the planned landuse pattern has been used.

Table 4:18: Model Simulation Scenarios of this Study

Scenario	Model Setup							Outcomes
	Network	Landuse & Topography	Rainfall	Embankment	Pumping	River WL/Q	LID	
SC1	Existing	Existing	BMD 3-H, 2016	-	-	Observed, 2016	-	A MIKE URBAN 1D-2D coupled calibrated and validated model
SC2	Existing	Existing	1D-10yrs & 25 Yrs	-	-	High, Average, and Low river WL & Discharge	-	Assessment of the existing network for a design storm event & river flood
SC3	Future (Planed by DNCC)	Future (Planed by RAJUK)	1D-10yrs, 25 & 100 Yrs	Include	(i)As per BWDB plan ii)without Pump	-	-	Analyzed the adequacy of the future internal drainage network. Identifies future max overflow locations and Development flood inundation maps.
SC3-CC	Future (Planed by DNCC)	Future (Planed by RAJUK)	1D-10yrs, 25 Yr. with Climate Change	Include	As per BWDB plan	-	-	Assessment of the effect of climate change impact on the future planned drainage system.
SC4(i)	Future	Future (Planed by RAJUK)	1D-10yrs & 25 Yrs	Include	(i)As per BWDB plan	-	RWH with Soak–Away Pits	Assessment of individual performance of RWH with Soak–Away Pits, Porous Pavement

Scenario	Model Setup							Outcomes
	Network	Landuse & Topography	Rainfall	Embankment	Pumping	River WL/Q	LID	
SC4(ii)	(Planed by DNCC)				ii)without Pump		Porous Pavement	and Wetland Basin in reducing runoff and pump duration
SC4(iii)							Wetland Basin	
SC5	Future (Planed by DNCC)	Future (Planed by DNCC)	D-10yrs & 25 Yrs	Include	As per BWDB plan	-	RWH with Soak–Away Pits & Porous Pavement	Assessment effectiveness RHW & SAPs and Porous Pavement in reducing runoff and pump duration
SC6	Future	Future	1D-10yrs, 25 & 100 Yrs	Include	As per BWDB plan	-	RWH with Soak–Away Pits, Porous Pavement & Wetland Basin	Investigate the size and invert of the **inlets-outlets** of the wetland basin to optimize its use and performance assessment for **combined LID measures**. Assessment of the performance of LID in case of a 100-year storm event.
SC6-CC	Future	Future	1D-10yrs, 25 Yr. with Climate Change	Include	Optimized pump	-	Above LID	Evaluate the performance of LID for the effect of climate change impact on the prepared optimum drainage plan.

4.6 Analysis of Results

Analyzing the model's results is very important for this drainage study. A precise model will give a virtuous result, but it may be achieved from proper analysis. The results were analyzed using Arc GIS for determining the flooded area for different inundation depths and the pumping duration. In this study, a GIS-based automation tool (**Figure 4-21**) was formed to analyse the model result. Step by step procedure for the analysis of the model result and development of the maximum inundation map is as follows:

Step-1: Extract DFS2 max depth file from 2D max depth simulation file (DFS2) e.g. "SC-6_WB_PP&RWH_SKPit_10Yr38CCBase_9_Depth.dfs2" by MIKE zero toolbox—Statistics---Txstat

Step-2: Make the water body depth zero (0) by the DHI topography adjustment tool.

Step-3: Make ASCI file from max depth adjusted DFS2 file using MIKE zero toolbox: GIS-Mike2Grid.

Step-4: Use a GIS-based automation tool to calculate sub-catchment-wise flood depth (dbf file).

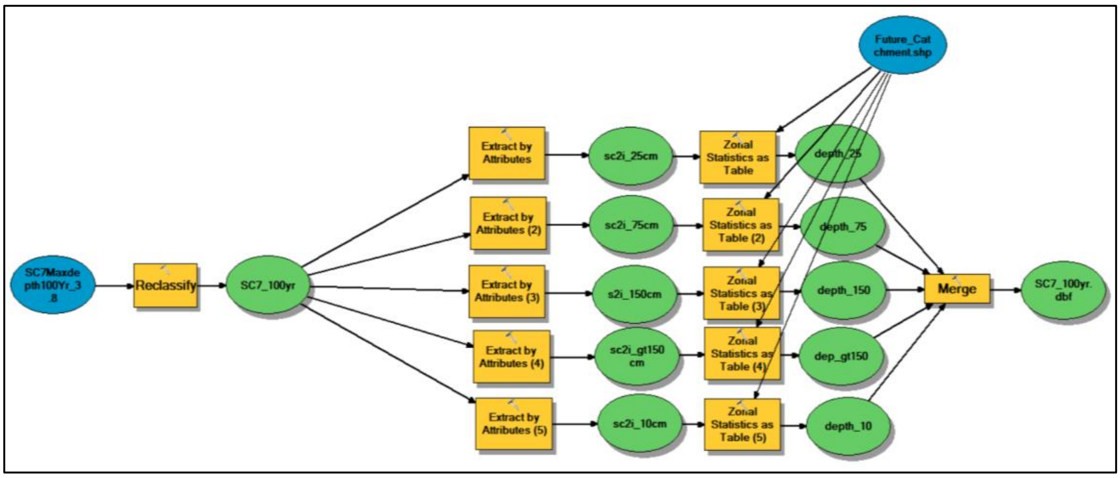


Figure 4-21: GIS-Based Automation Tool for Analysis of Model Result

It is mentioned here that, during the analysis of flood depth 0.1m or more depth has been considered of inundation area as flooded area.

Chapter 5

Results and Discussions

5.1 Model Calibration

Calibration is the process of adjusting model parameters and forcing within their reasonable range to best fit the result of a model with the observed data. For the present study, the hydraulic model was calibrated by optimizing mainly the manning's roughness coefficient, time of concentration and initial loss. The hydraulic model was calibrated against the observed water level at two locations of the drainage network as shown in **Figure 5-1**.Those two locations are Bauther on Boalia khal and Sinanghata on Balu River. Calibration was done for the period May-June of 2016. Besides, the simulated discharge data of the Balu River has been compared with the observed discharge data of the Balu River at Sinanghata.

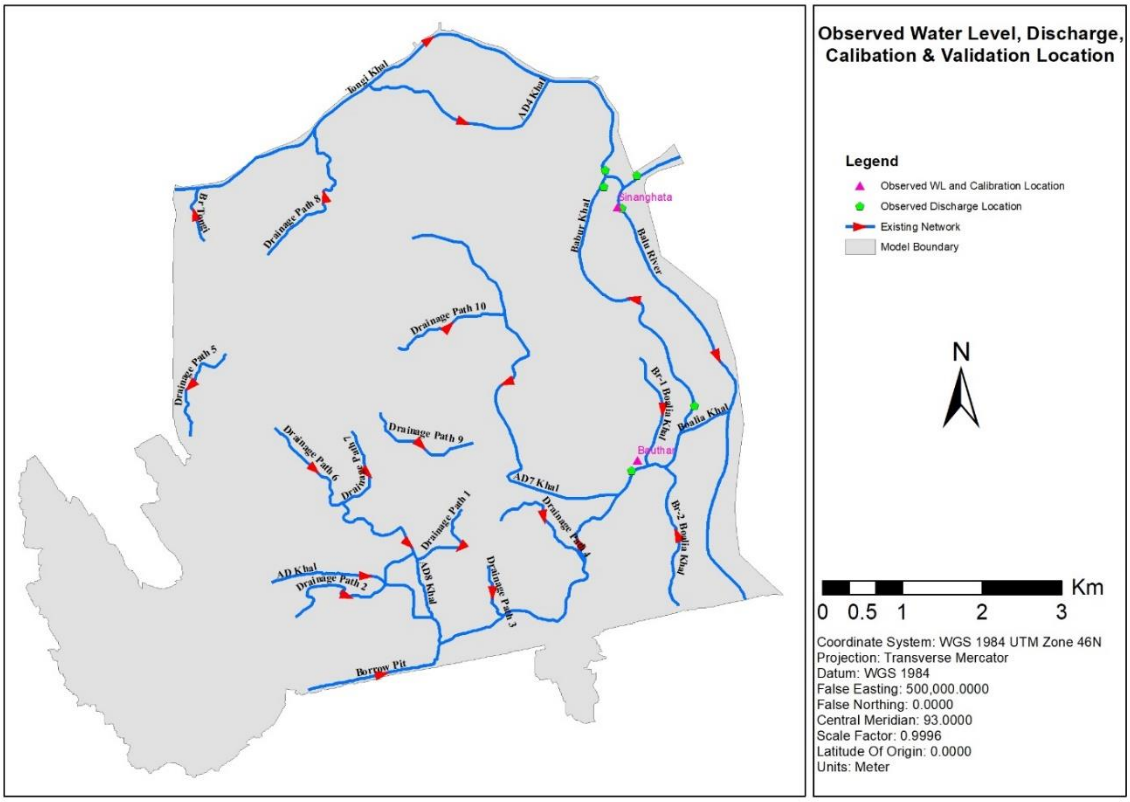


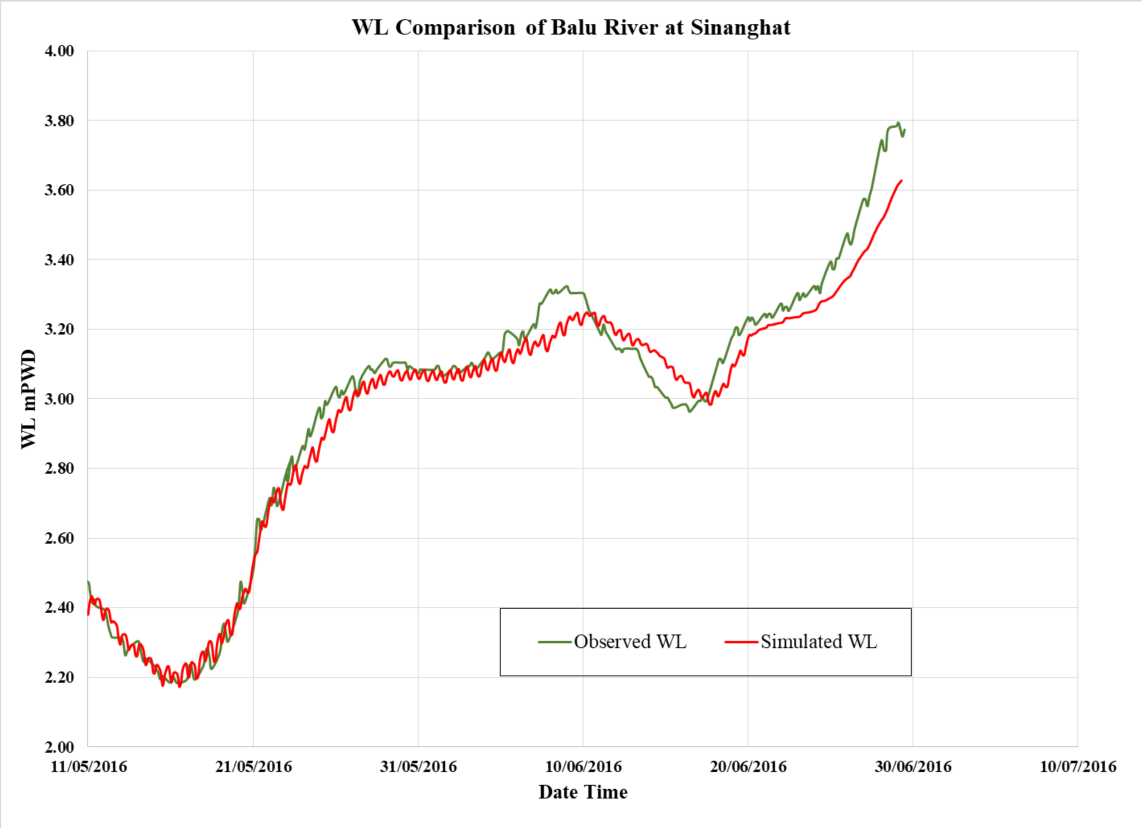
Figure 5-1: Calibration and Validation Location

Both the graphical and statistical approaches were used for the evaluation of calibration and validation. Two commonly used statistical indicators, the coefficient of determination (R^2) and the Nash-Sutcliffe efficiency (NSE), were utilized in this work to assess model calibration and validation.

The coefficient of determination (R^2) describes how closely observed and simulated data coincide. Its value is between 0 and 1, with values near 1 indicating higher degrees of calibration. The relative amount of the residual variance in comparison to the variance of the measured data is determined by Nash-Sutcliffe efficiency (NSE). Its value can range from ∞ to 1, with 1 being the ideal number. Values between 0 and 1 are typically accepted as acceptable. American Society of Civil Engineers (ASCE) and several scientists (e.g. Sevat and Dezetter, 1991) have recognized it as the best objective function for fitting of hydrological models (Moriasi et. al., 2007). Moriasi et. al., (2007) also proposed values of NSE for performance ratings of hydrologic models. NSE value from 0.65 to 0.75 has been recognized as an indicator of “Good” performance. The performance of a model has been recognized as “Very good” if the NSE value falls in the range of 0.75 to 1.0.

5.1.1 Comparison of Simulated & Observed Water Level

A comparison of simulated and observed water levels at the two above-mentioned points for the calibration period has been shown in **Figure 5-2** and **Figure 5-3.** The simulated water level at both Sinanghata (Balu River) and Bauther (Boalia khal) highly coincide with the observed water level as can be seen graphically and from the statistical parameters. The value of NSE at both calibration points is above 0.9 which is indicative of a “very good” model performance as suggested by Moriasi et. al., (2007).



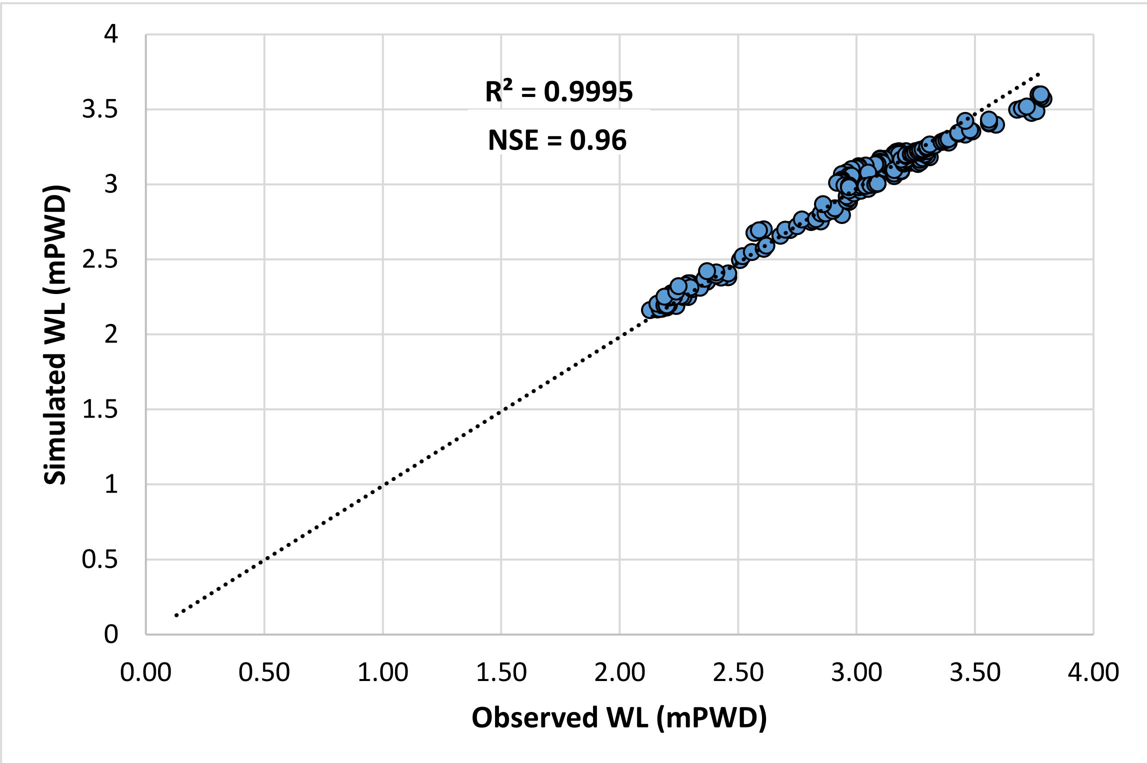
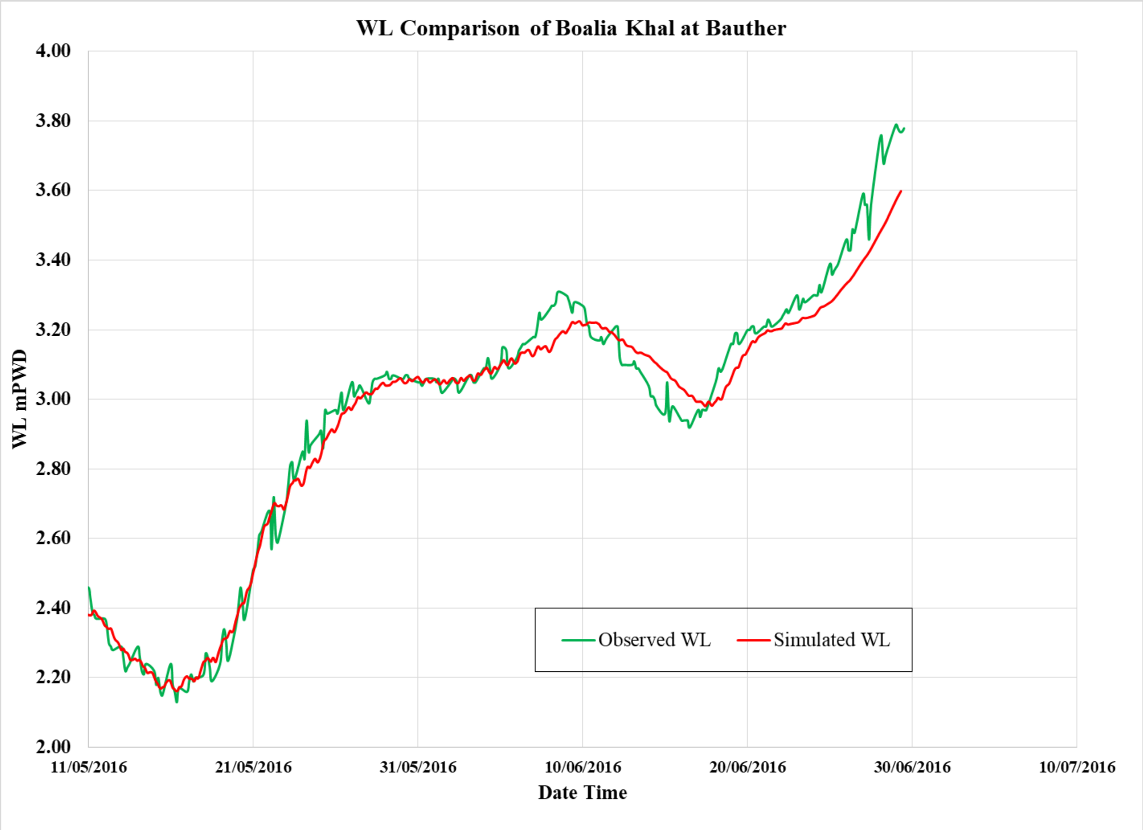


Figure 5-2: Comparison of Observed and Simulated Water Level at Sinanghata, Balu River (May-Jun, 2016)



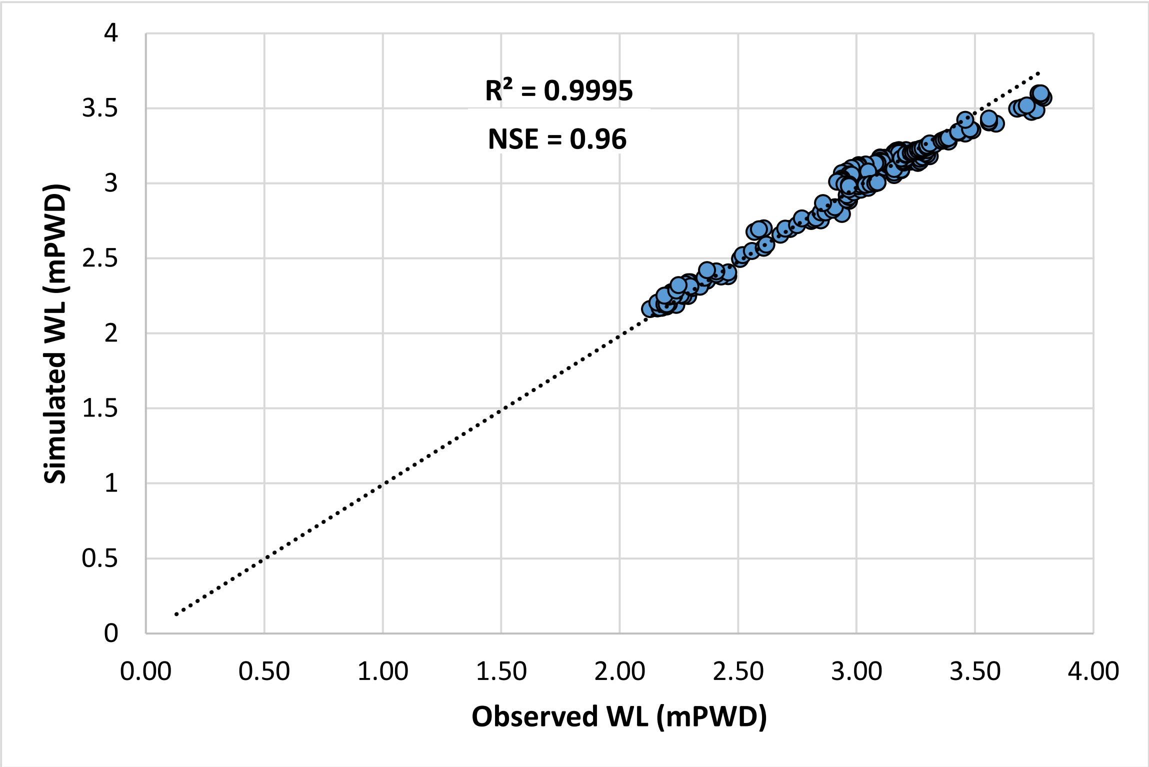


Figure 5-3: Comparison of Observed and Simulated Water Level at Bauther, Boalia Khal (May-Jun, 2016)

5.1.2 Comparison of Simulated & Observed Discharge

A comparison of simulated discharge and observed discharge at Sinanghata point Balu River for the calibration period has been shown in **Figure 5-4**. Very few observed data were available. Comparing the observed data with time series of simulated data does not reflect any significance. Nevertheless, a good correlation between observed discharge data and simulated discharge data was found while comparing the observed discharge data with the moving average of the simulated data (**Figure 5-4**). Again, this reflects that the developed model is well-calibrated and can reasonably simulate a hydrological event.

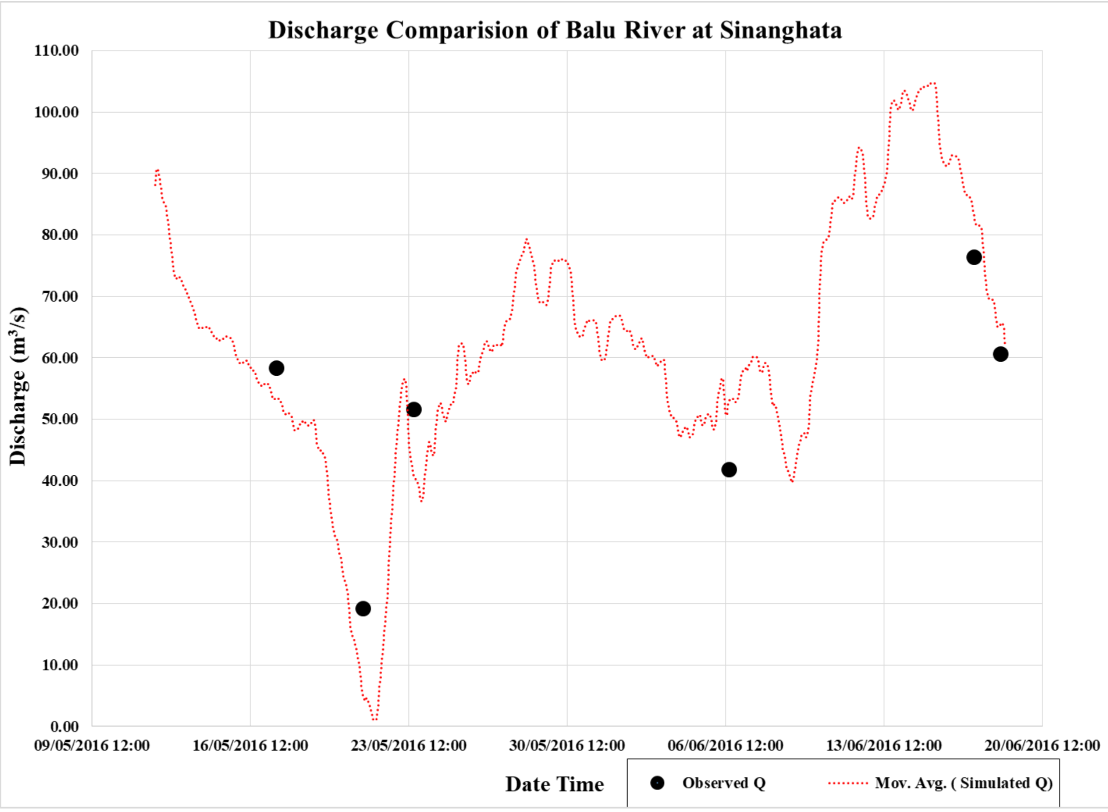


Figure 5-4: Comparison of Observed and Simulated Discharge at Sinanghata, Balu River

5.2 Model Validation

Validation of a hydraulic model is the process of evaluating whether the model accurately represents the behaviour of the system under other storm events apart from the storm event that has been used for model calibration. For this study, the hydraulic model was validated for the period July-September of 2016 using the observed water level at Bauther on Boalia khal and Sinanghata on Balu River.

Like model calibration, the coefficient of determination (R^2) and the Nash-Sutcliffe efficiency (NSE) was utilized to assess the validity of the model.

A comparison of simulated and observed water level for the validation period has been shown in **Figure 5-5** and **Figure 5-6.** The simulated water level at both Sinanghata (Balu River) and Bauther (Boalia khal) coincide quite well with the observed water level data at respective locations as can be seen graphically which indicates that the model is reasonably validated.

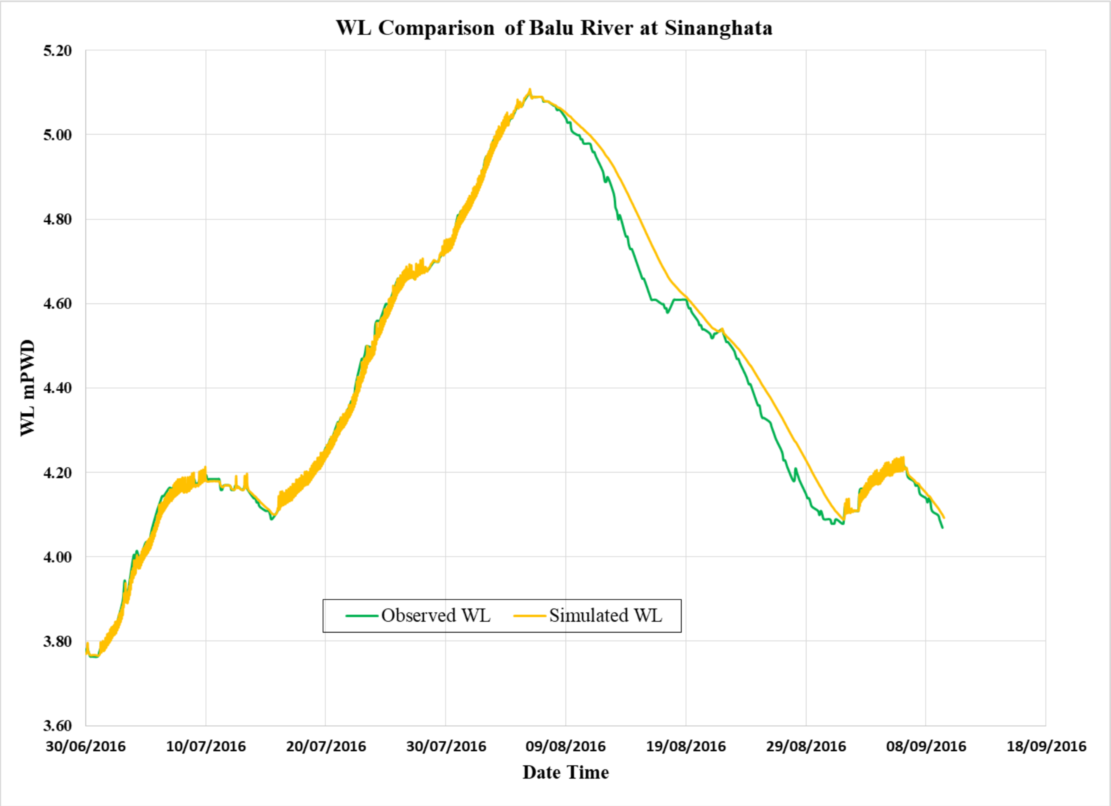


Figure 5-5: Comparison of Observed and Simulated Water Level at Sinanghata, Balu River (Jul-Sep, 2016)

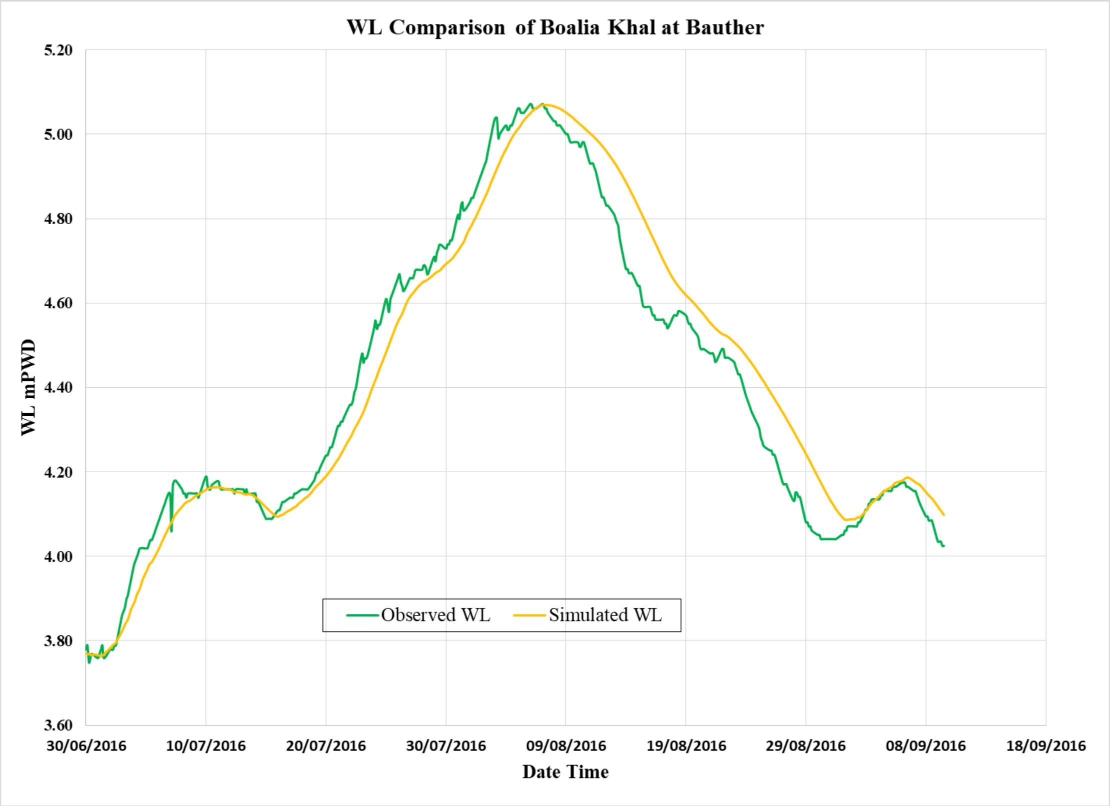


Figure 5-6: Comparison of Observed and Simulated Water Level at Bauther, Boalia Khal (Jul-Sep, 2016)

5.3 Scenario Analysis

5.3.1 Performance of Existing Network (SC2)

The performance of the existing network was evaluated based on model simulations under 10-year and 25-year return period design rainfall events (developed following Alternating Block Method) with high, average and low river water level conditions. Under high water level conditions, the extent of the flooded area is 65%-70% higher as compared to the average water level condition in Balu River. The flood extent is 200%-230% more as compared to the low water level condition. The increase is more prominent for flood depths higher than 0.75 m. as the flood extent for a 25-year return period storm event is very similar to the flood extent of a storm event of a 10-year return period under high, average and low water level condition (**Figure 5-7**).

Comparing the flood maps of 10-year and 25-year return period event under different water level conditions (**Figure 5-8** to **Figure 5-13)**, it was observed that the flooded area extends from Balu River towards the northwest direction. During high river water level conditions, Boali khal and its surrounding khal catchments were affected by flood depths higher than 1.5 m. Only parts of Br. Tongi and Sikderbari sub-catchments remains flood free under high water level condition in the Balu River (**Figure 5-8** to **Figure 5-9**). All this indicates that flooding in the existing situation is basically driven by the water level condition in the Balu River.

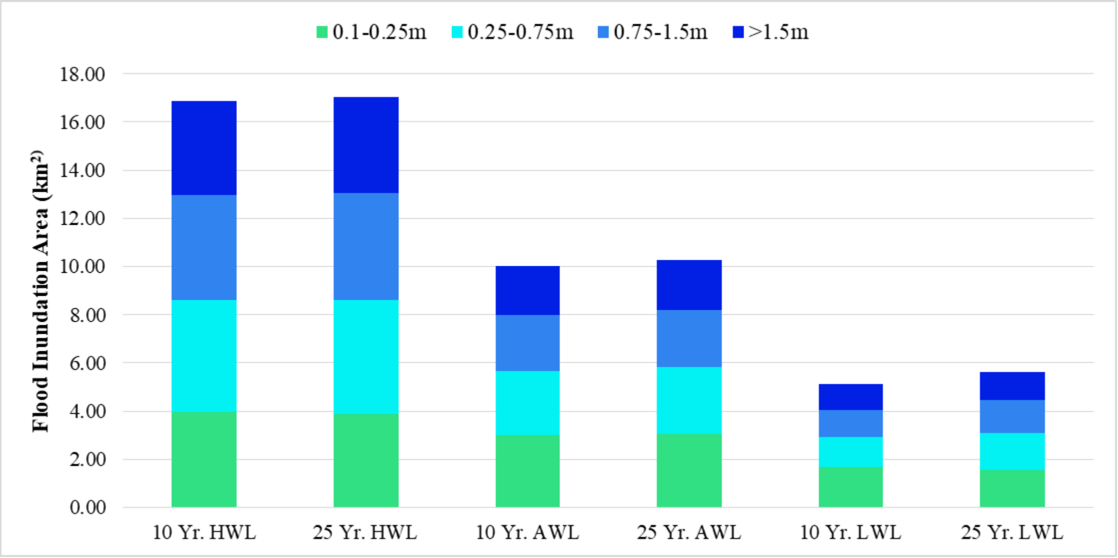


Figure 5-7: Flood Inundation Area for Existing Condition under Different Storm Events

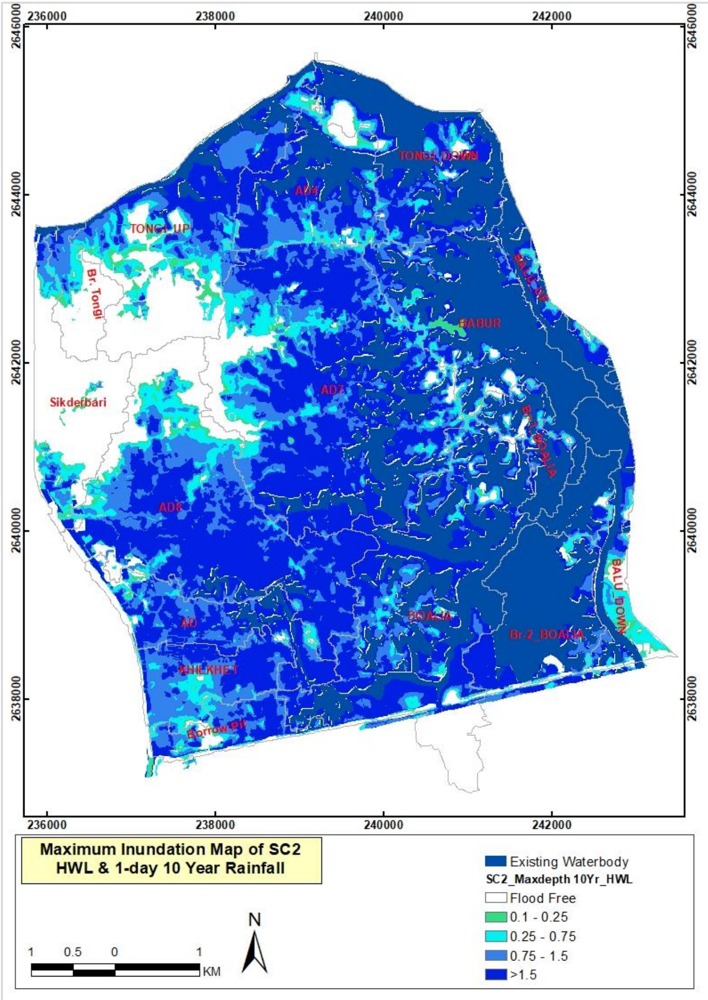


Figure 5-8: Flood Inundation Map in Existing Condition (SC2) under a 10 Yr. Rainfall and High River Water Level

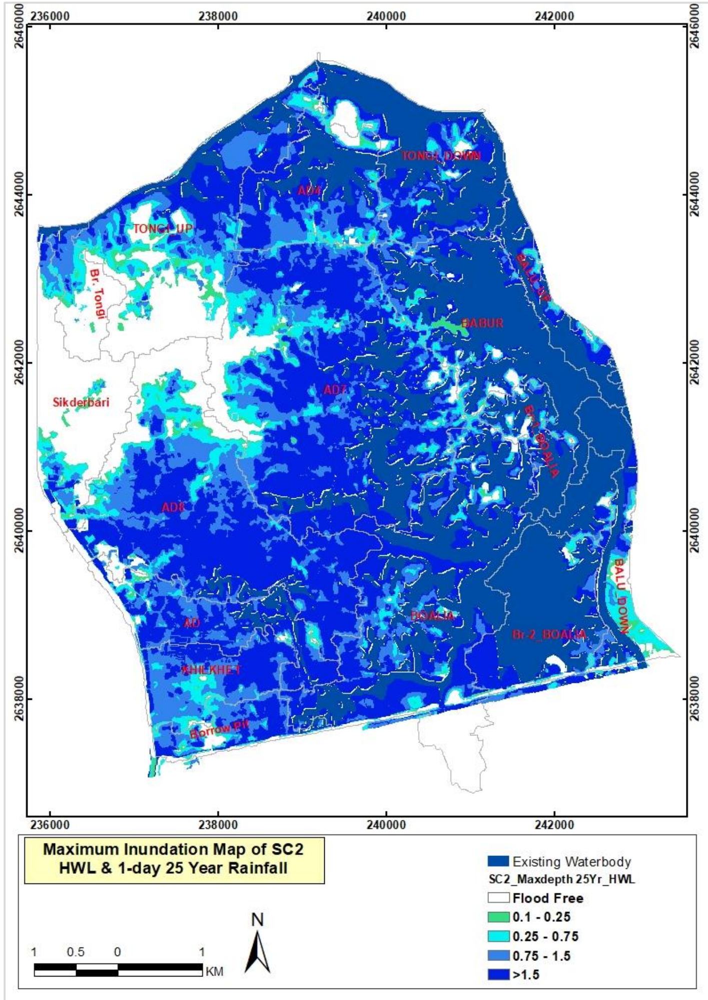


Figure 5-9: Flood Inundation Map in Existing Condition (SC2) under a 25 Yr. Rainfall and High River Water Level

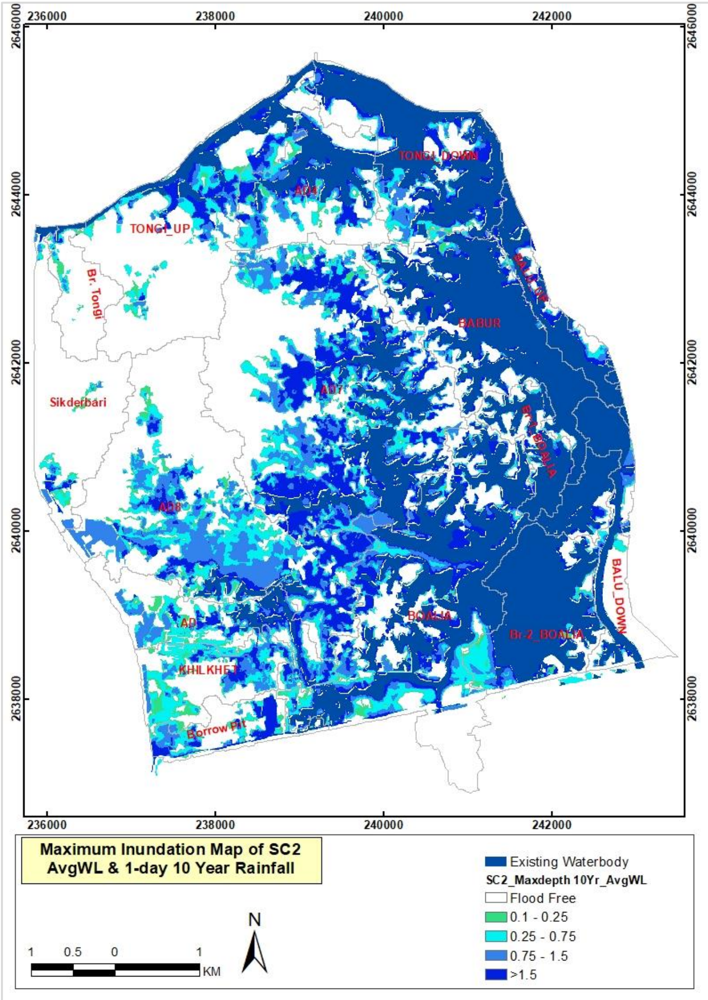


Figure 5-10: Flood Inundation Map in Existing Condition (SC2) under a 10 Yr. Rainfall and Avg. River Water Level

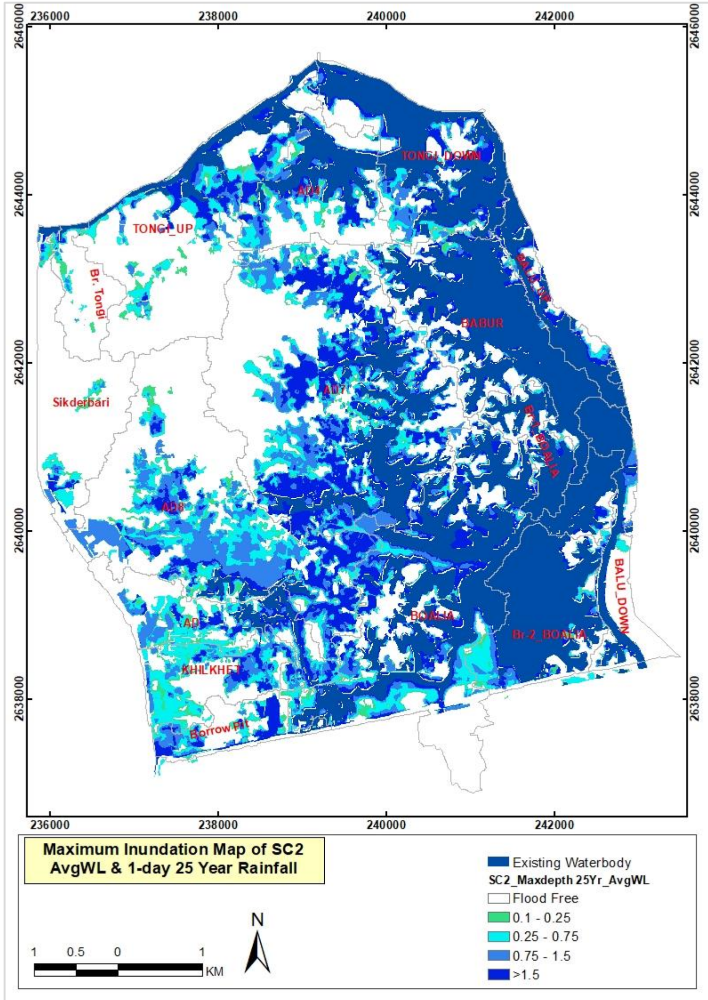


Figure 5-11: Flood Inundation Map in Existing Condition (SC2) under a 25Yr. Rainfall and Avg. River Water Level

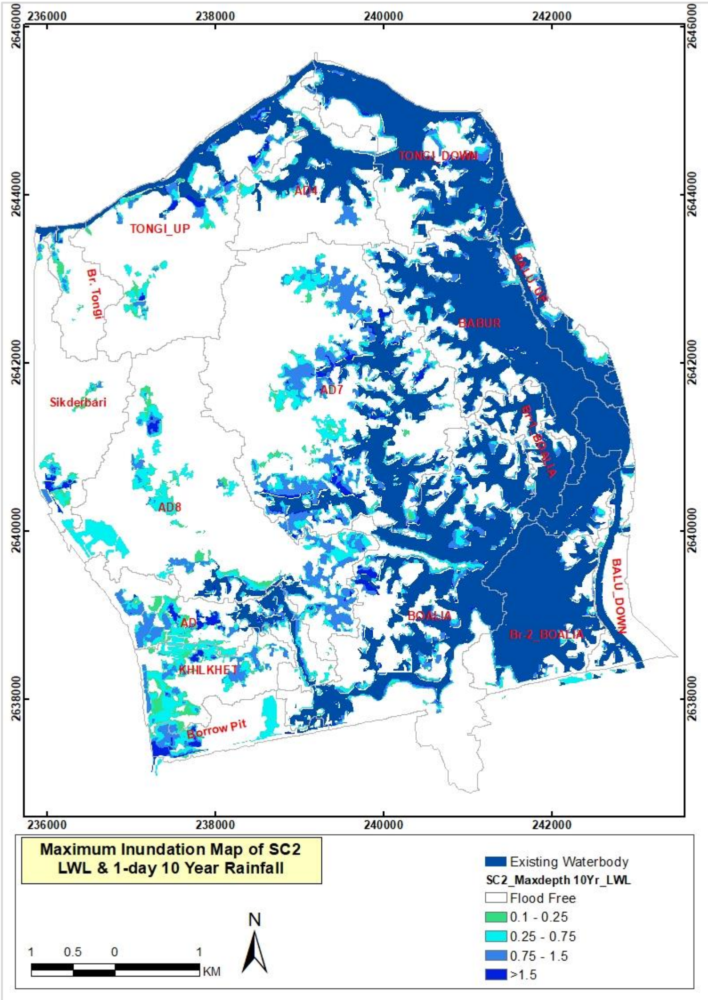


Figure 5-12: Flood Inundation Map in Existing Condition (SC2) under a 10 Yr. Rainfall and Low River Water Level

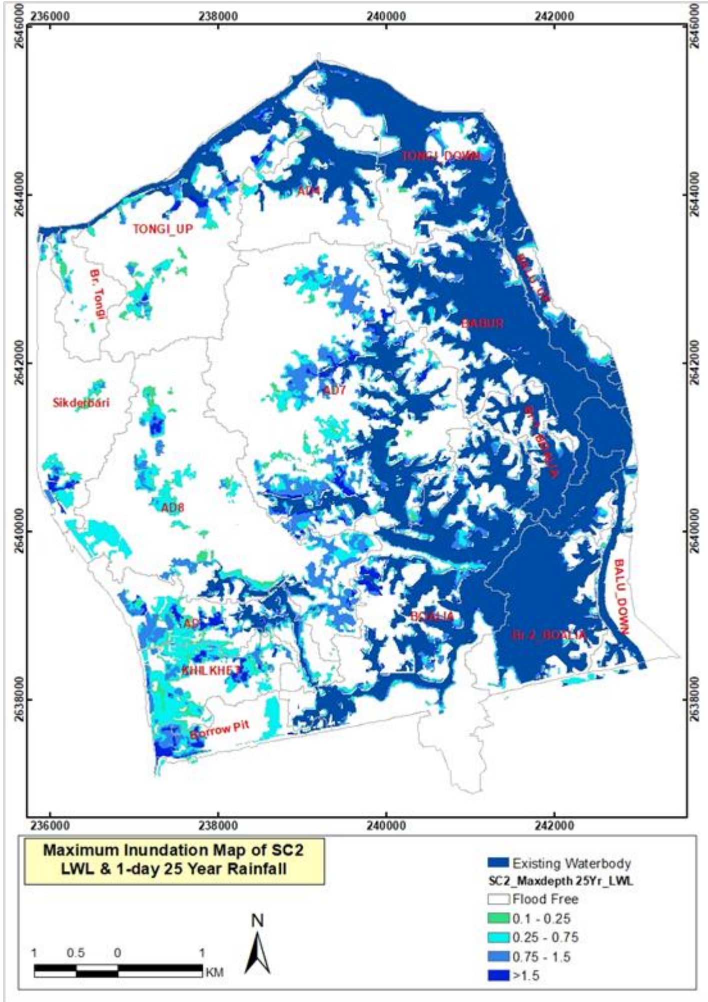


Figure 5-13: Flood Inundation Map in Existing Condition (SC2) under a 25 Yr. Rainfall and Low River Water Level

5.3.2 Performance of the Planned Drainage Network for Future (SC3)

The performance of the planned drainage network for the future was evaluated based on simulations under 10-year and 25-year design rainfall events (developed following Alternating Block Method) with flood protection embankment along the right bank of Balu River and planned retention ponds & pump stations as per DAP and BWDB plans. As compared to the existing situation, the overall flood extent is likely to be reduced in the future if plans for RAJUK, BWDB and DNCC are implemented. This is logical as due to the flood protection embankment along the Balu River, the river water level will no longer influence the flooding situation. Flooding will occur only in areas where the planned drainage network won't have enough capacity to cope with rainfall events. This implies that flood extent will be spatially distributed all across the catchment area depending on the conveyance capacity of the planned drainage system rather than concentrating in areas with relatively lower topography as was seen in the existing situation. In fact, flooding in relatively higher elevation areas like the northwest corner of the study area is also likely to occur under storm events of both 25-year and 10-year return periods (**Figure 5-16 and Figure 5-17**).

Also, in future conditions, the flooding pattern is likely to be influenced by the return period of the storm events. Flood extent is likely to increase by 34% for a 25-year return period event as compared to a 10-year return period event (**Figure 5-14**). From the flood inundation map **Figure 5-18,** it is shows that about 15% area more flooded in future without pumping facilities for 25-year storm as compared to the flood inundation under pumping facilities of same storm.

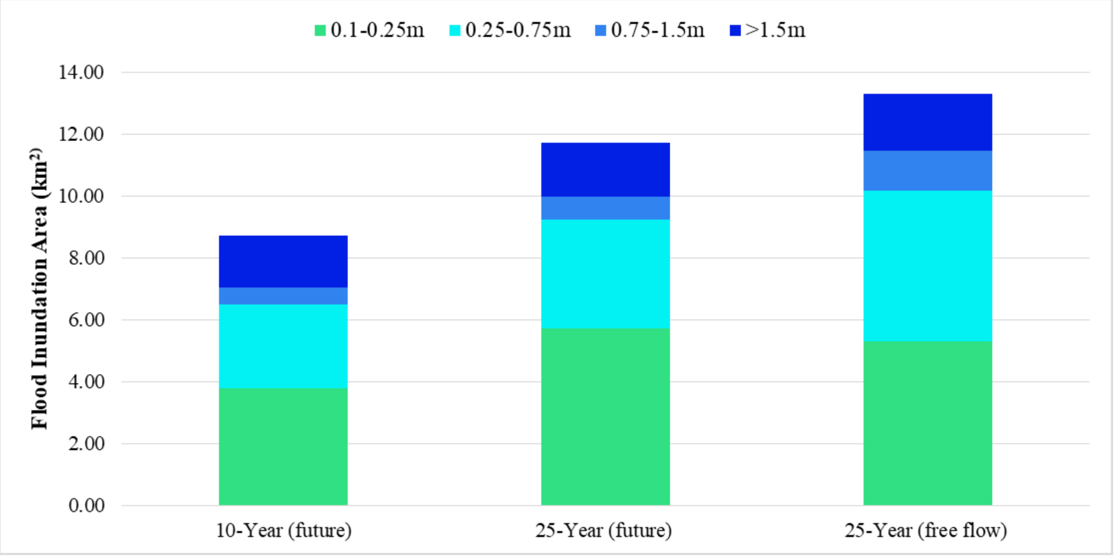


Figure 5-14: Flood Inundation Area for Future Conditions under Different Storm Events

The pumping duration significantly increases (21% of Boalia pump and 5% of Babur pump) for a 25-year return period event compared to a 10-year return period event of rainfall (**Figure 5-15**). This is expected as the cumulative rainfall for a 25-year event is 20% higher than a 10-year return period event.

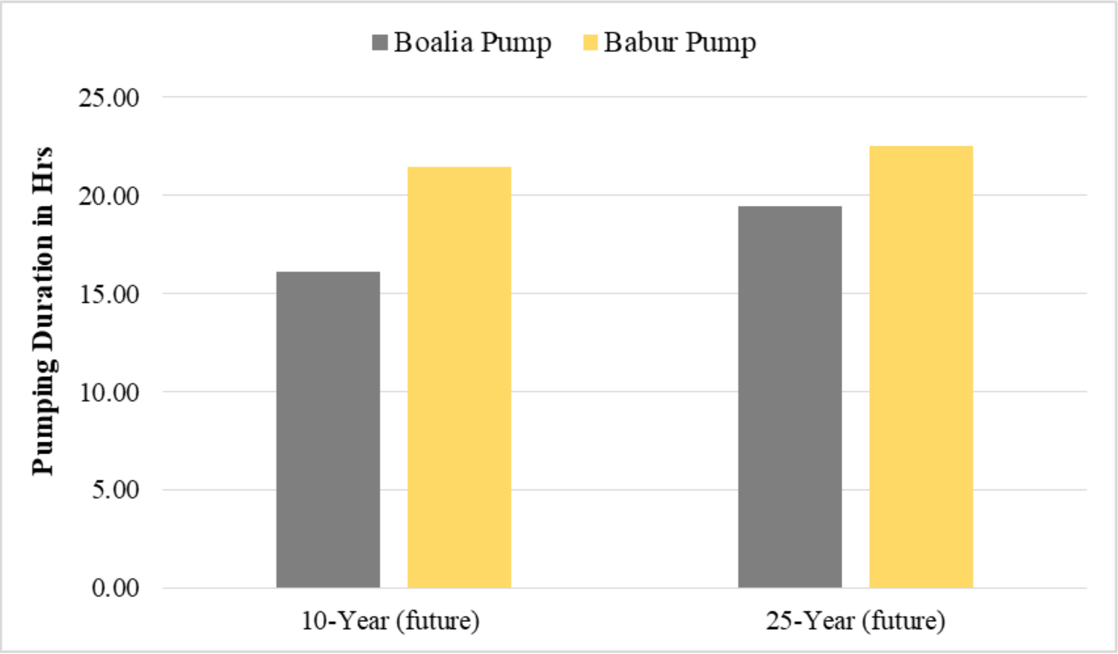


Figure 5-15: Pumping Duration for Future Conditions under Different Storm Events

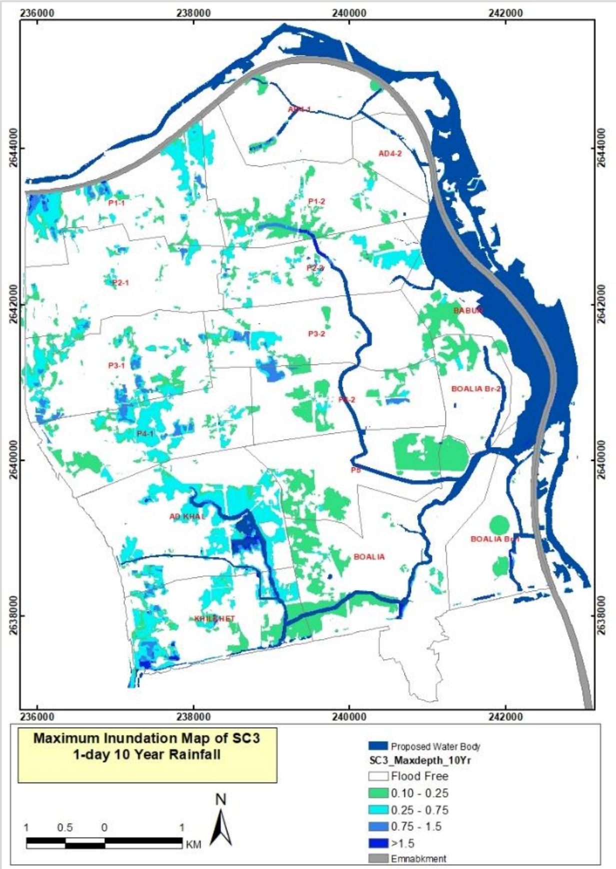


Figure 5-16: Flood Inundation Map in Future Condition (SC3) under a 10 Yr. Rainfall with Pumping Facilities

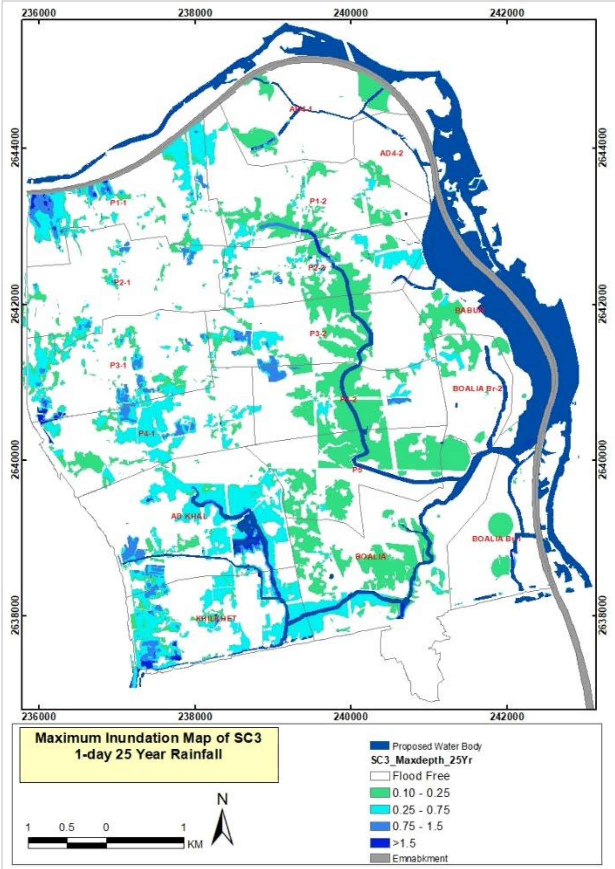


Figure 5-17: Flood Inundation Map in Future Condition (SC3) under a 25 Yr. Rainfall with Pumping Facilities

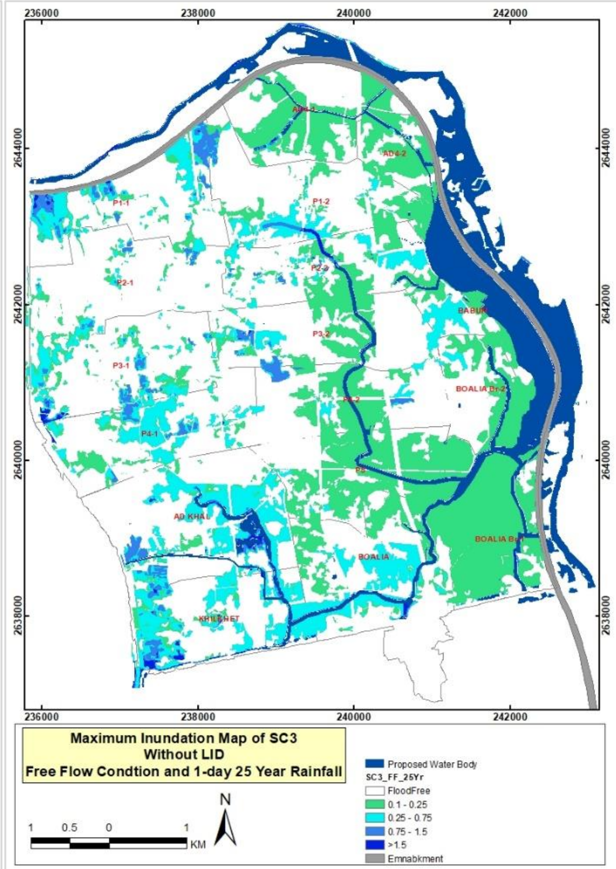


Figure 5-18: Flood Inundation Map in Future Condition (SC3) under a 25 Yr. Rainfall without Pumping Facilities

5.3.3 Performance of Low Impact Development (LID) Strategies (S4, SC5 & SC6)

To examine the individual performance of the application of LID, Rainwater Harvesting (RWH) & Soak-Away Pits (SAPs) **SC4(i)**, Porous Pavement (PP) **SC4(ii)** and Wetland Basin (WB) **SC4(iii)** were applied separately with future drainage network scenario **SC3.** To gather a better understanding of the individual performance of LID, the scenario **SC4(i), SC4(ii)** and **SC4(iii)** have also been simulated under free flow condition considering without pumping and normal river water level. In the scenario, **SC5** Porous Pavement was added with **SC4(i)** and finally, in **SC6** Wetland Basins were added with **SC5.** The performance of the LID was evaluated based on simulations under 10-year and 25-year design rainfall events.

In this section, the simulation results of stormwater drainage including low impact development measures are discussed for the following three aspects: water level hydrograph, flood extent, and pumping duration.

Water Level Hydrograph

Figure 5-19 and **Figure 5-20** show that LID can effectively decrease maximum water level, and this reduction efficiency was positively correlated with the application approach of LID. The reduction of water level of different scenarios was as follows: SC4 (i) < SC4 (ii) <SC5< SC4 (iii) < SC6.

There is no significant reduction of water level after the application of Rainwater Harvesting (RWH) with Soak-Away Pits **SC4(i)**. The water level was found low as a result of the application of porous pavement **SC4(ii**) compared to **SC4(i)**. On the other hand, a remarkable reduction of water level was observed after Wetland Basins **SC4(iii)** were used, even the rate of decrease of water level is more as a result of the application of Wetland Basins **SC4(iii)** compared to together with the application of Rainwater Harvesting (RWH) & Soak-Away Pits and Porous Pavement **(SC5)**. When wetland Basins are included with **SC5**, the overall water level was found minimum. In scenario **SC6**, water level is reduced by 8.4 cm at the upstream (U/S) of Boalia Khal and by 19.2 cm at the downstream (D/S) of Boalia Khal for a 25-year return period event as compared to **SC3**.

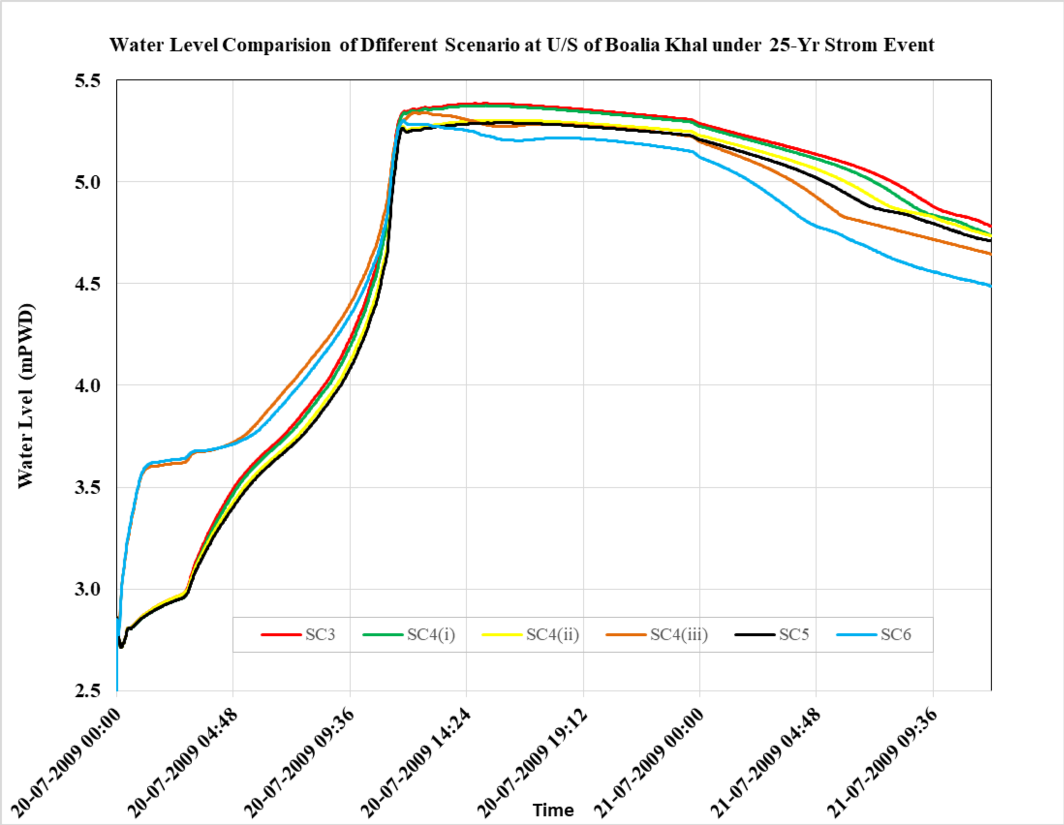


Figure 5-19: Water Level Comparison between with and without LID Scenarios under a 25 Yr. Rainfall Event at Boalia Khal U/S

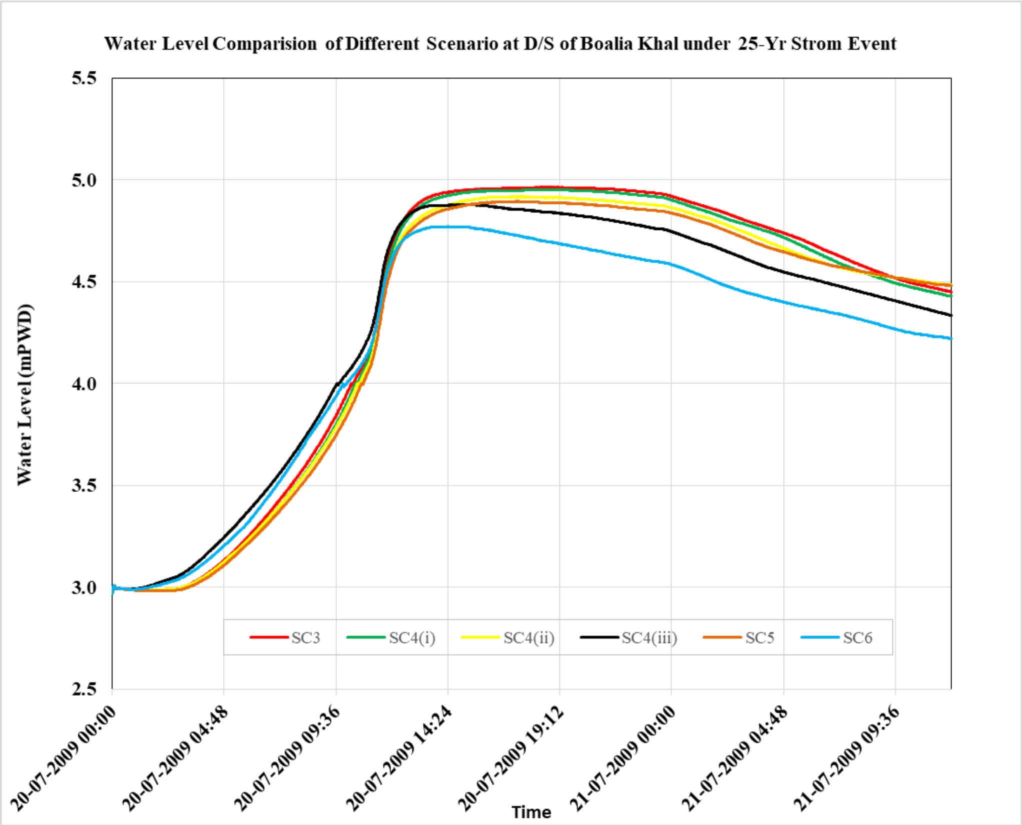


Figure 5-20: Water Level Comparison between with and without LID Scenarios under a 25 Yr. Rainfall Event at Boalia Khal D/S

Flood Extent

To represent the simulation results of the drainage system with LID, flood maps for different return periods of rainfall (**Figure 5-27 to Figure 5-43)** were generated. The results were analyzed using Arc GIS for determining the flooded area for different inundation depths.

The flood extent is likely to be influenced by the return period of the storm events as well as the application of LID measures. There is no significant change in the flooded area in **SC4(i)** compared to **SC3** i.e only Rainwater Harvesting (RWH) & Soak-Away Pits (SAPs) cannot effectively reduce the flooding area (**Figure 5-21** and **Figure 5-23).** The extent of flood is decreased by 10% as a result of use of Porous Pavement **SC4(ii)**. Wetland basins perform comparably less than porous pavements in terms of reducing flood extent. The similar performance of LID were found (**Table 5:1**) from the simulation result under free flow condition.

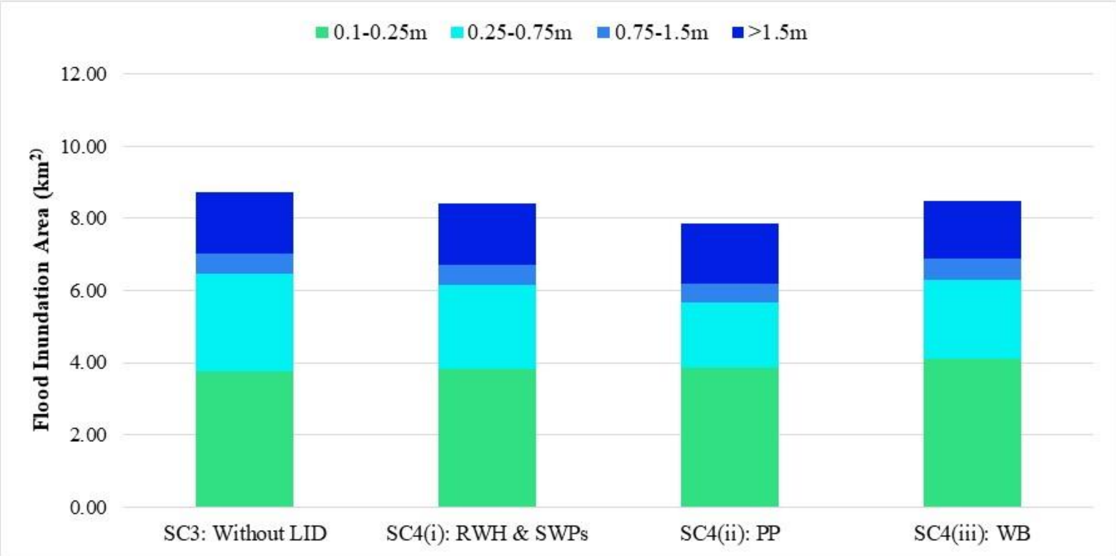


Figure 5-21: Comparison of Flood Inundation Area in Future Condition under a 10-year Strom Event without LID and Different Types of Individual LID.

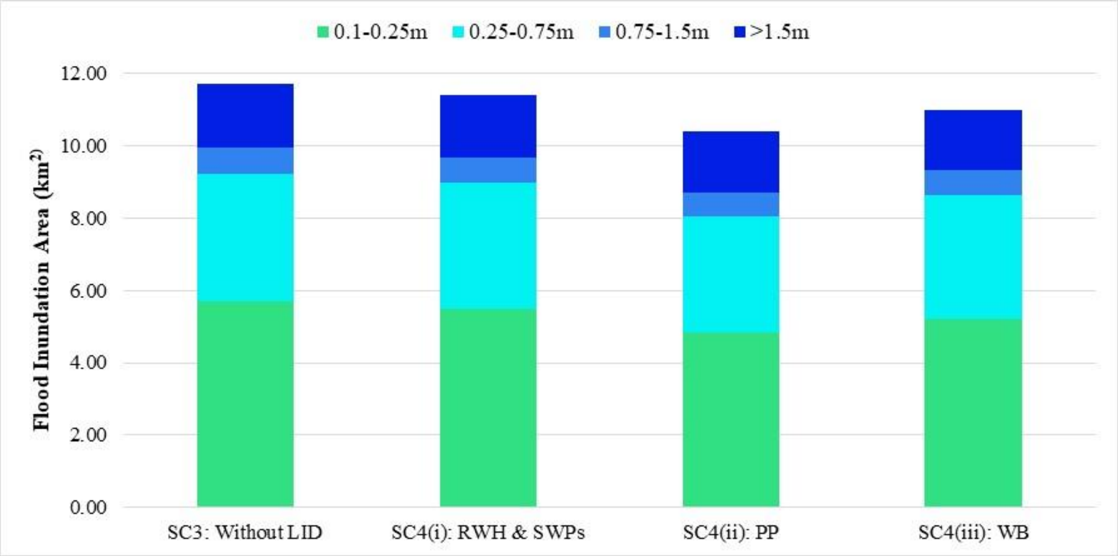


Figure 5-22: Comparison of Flood Inundation Area in Future Condition under a 25-year Strom Event without LID and Different Types of Individual LID.

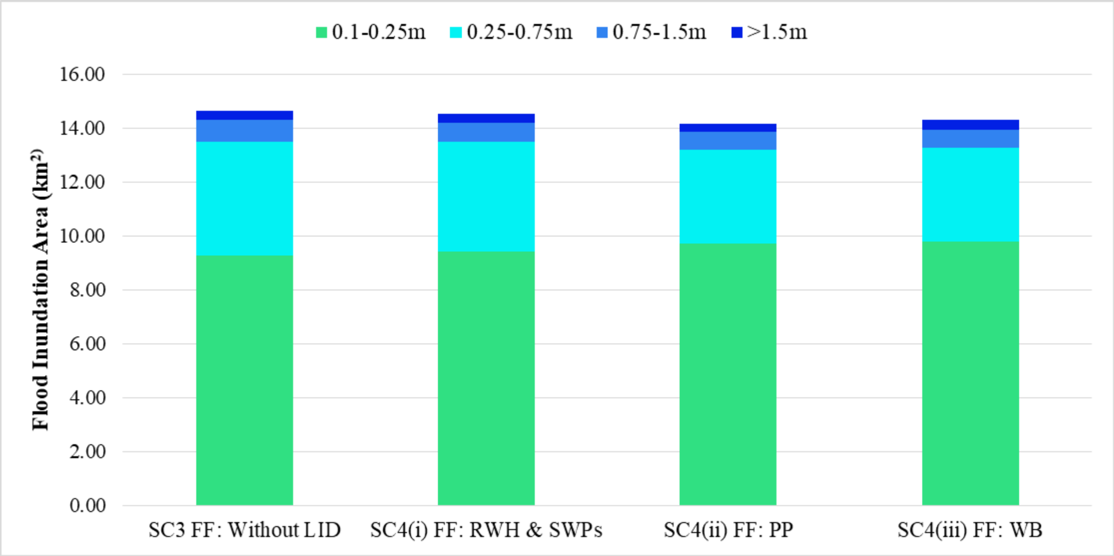


Figure 5-23: Comparison of Flood Inundation Area in Free Flow (FF) Condition under a 25-year Strom Event without LID and Different Types of Individual LID.

Table 5:1: Compression of Flood Extent between SC3 and Different Types of Individual LID SC4(i), SC4(ii), SC4(iii) under with pump and Free Flow Condition

LID Measures	Reduction of Flood Extent as compared to SC3		Reduction of Flood Extent as compared to SC3 FF
	10-year Rainfall	25-year Rainfall	25- year Rainfall
RWH-SWPs, SC4(i)	4%	3%	0.8%
PP, SC4(ii)	10%	11%	3.3%
WB, SC4(iii)	3%	6%	2.8%

As a result of the application of RWH-SAPs, PP and WB together the flood extent is reduced by 14% for a 10-year storm event and 15% for a 25-year storm event in comparison to without LID **(Figure 5-24 & Figure 5-25 and Table 5:2**). On the other hand, the flooded area was reduced by 12% in both rainfall events when two types of LID (RWH-SAPs, PP) were applied together. The flood inundation area under free flow condition **(Figure 5-26** and **Table 5:2**) represents the similar nature of result. Thus the combination of the three types of LID is more effective to reduce the inundation area.

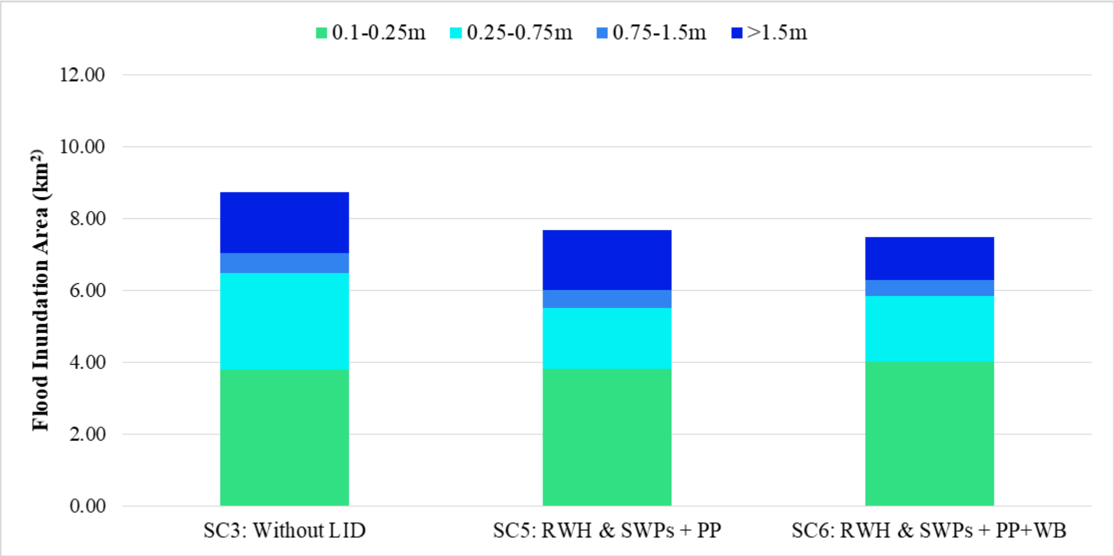


Figure 5-24: Comparison of Flood Inundation Area in Future Condition under a 10-year Strom Event with and without Low Impact Development

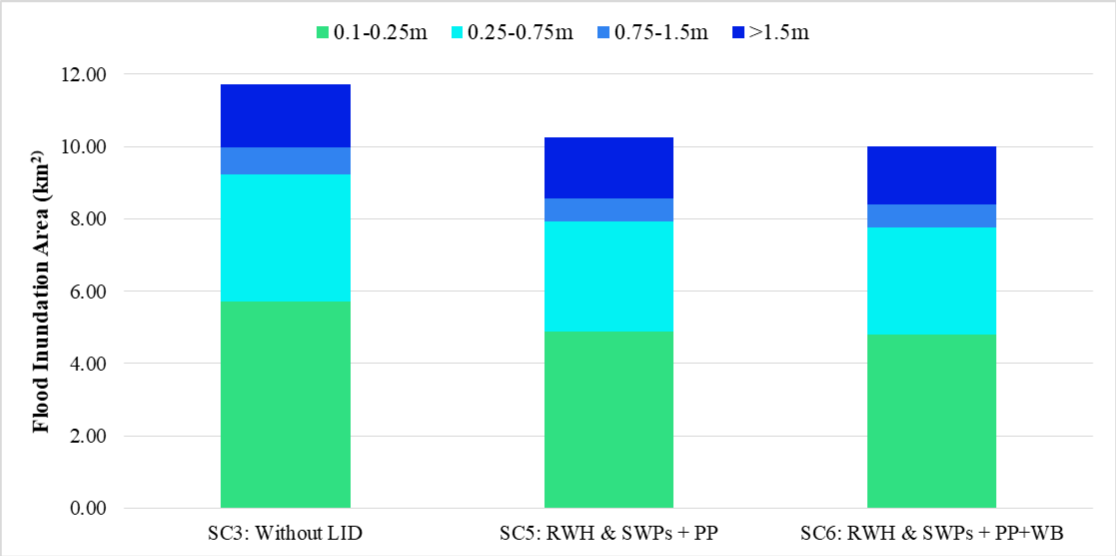


Figure 5-25: Comparison of Flood Inundation Area in Future Condition under a 25-year Strom Event with and without Low Impact Development

Table 5:2: Compression of Flood Extent between SC3 and Combination of RWH-SWPs + PP (SC5) and RWH-SWPs+ PP +WB (SC6) under with pump and Free Flow Condition

Combination of LID Measures	**Reduction of Flood Extent as compared to SC3**		**Reduction of Flood Extent as compared to SC3 FF**
	10-year Rainfall	**25-year Rainfall**	**25- year Rainfall**
RWH-SWPs + PP, SC5	12%	12%	4.8%
RWH-SWPs + PP + WB, SC6	14%	15%	8.9%

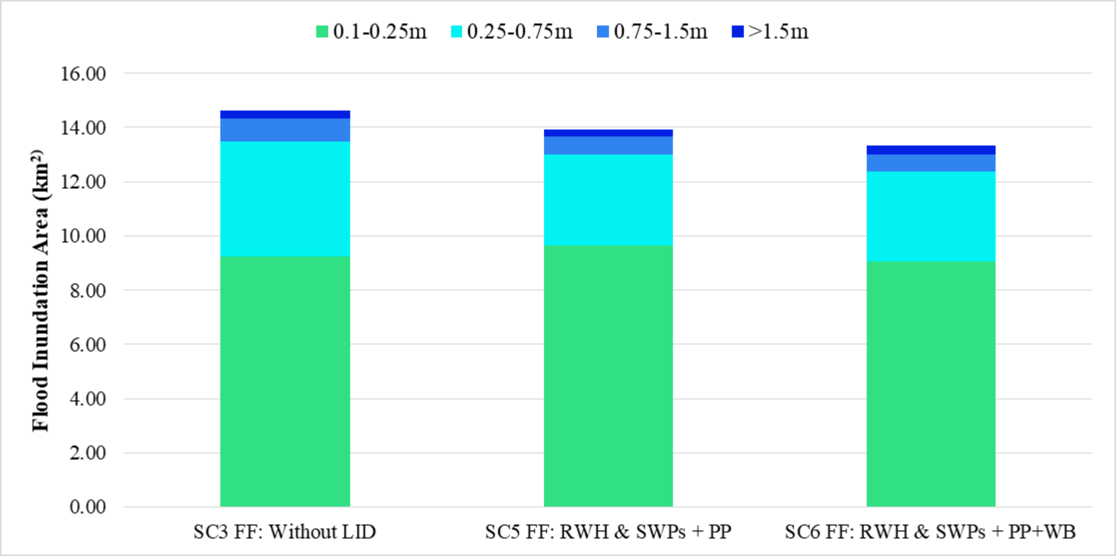


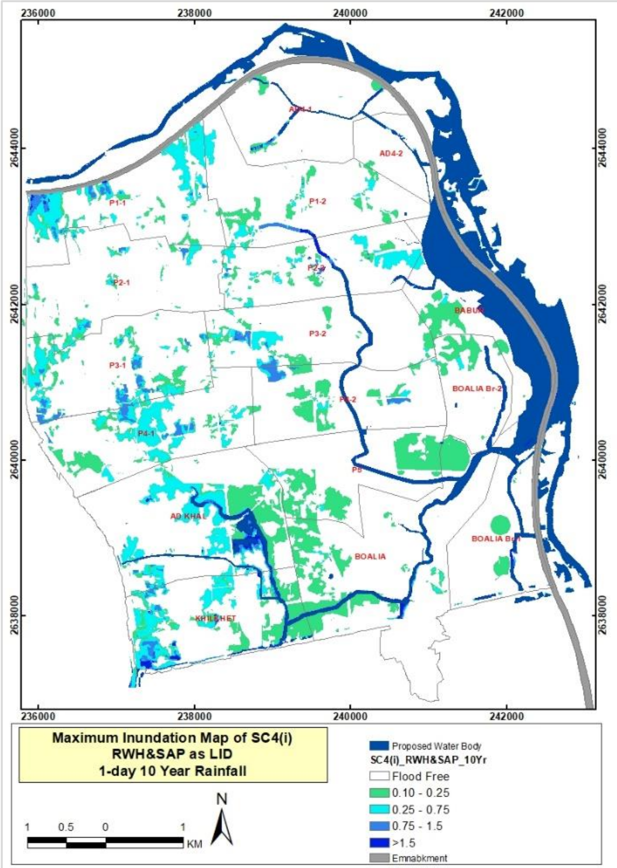
Figure 5-26: Comparison of Flood Inundation Area in Free Flow (FF) Condition under a 25-year Strom Event with and without Low Impact Development

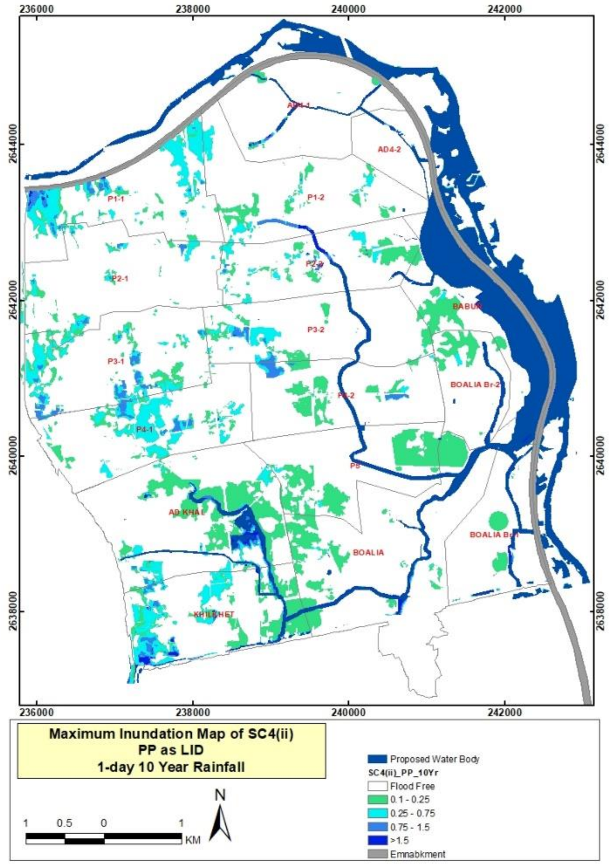
The inlet control structures' Invert Level (IL) of the wetland basin has also influenced the reduction of flood extent. The flood extent will be decreased when the invert level of wetland basins is set below the pump start level compared to the same level of the inlet control structures' invert level and pumping start level. **Table 5:3** demonstrates that when the invert level is set at 3.8 mPWD rather than 4.0 mPWD, flood extent is reduced by 3% for a 10-year storm event and by 4% for a 25-year storm event.

The inlet control structures' invert level had also a significant impact on the pumping duration. For example, **Table 5:3** shows that the Boalia pump duration is decreased by 1.93 hours for a 10-year storm event and 2.34 hours for a 25-year storm event when the invert level was set at 3.8 mPWD instead of 4.0 mPWD. It is mentioned here that in both cases the pump's start level was set at 4.0 mPWD.

Table 5:3: Compression Inundation Area with various Inlet Invert of Wetland Basin

Scenario	Return Period (Year)	Inlet Invert (mPWD)	Max Flood Extent (Km^2)					Pumping Duration (Hr.)	
			0.1-0.25m	0.25-0.75m	0.75-1.5m	>1.5m	Total	Boalia Pump	Babur Pump
SC6	10	4.00	4.14	1.85	0.46	1.59	8.04	9.57	10.97
		3.80	4.01	1.80	0.38	1.58	7.77	7.64	10.20
	25	4.00	4.94	3.19	0.64	1.62	10.38	10.82	15.55
		3.80	4.81	2.95	0.63	1.61	10.01	8.48	14.70





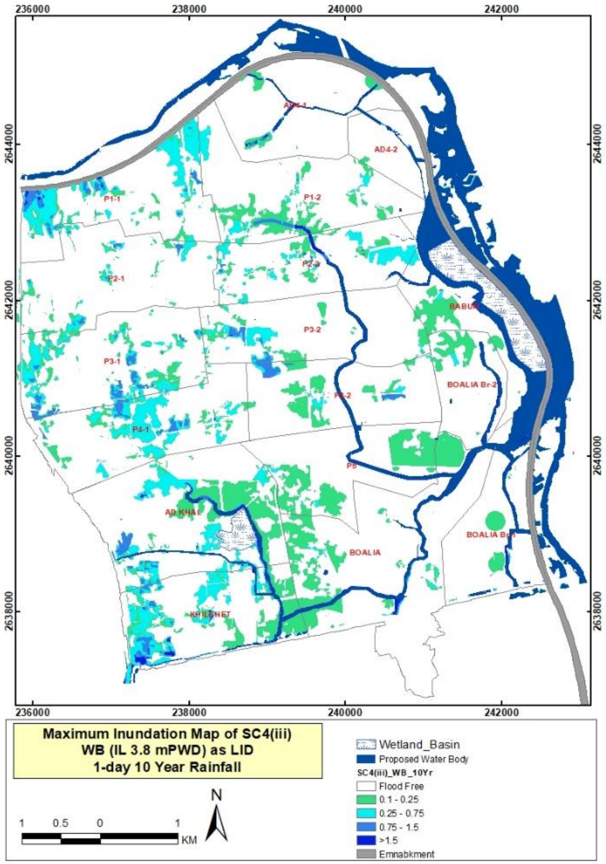


Figure 5-27: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Pumping Facilities and RWH-SWPs as LID SC4(i)

Figure 5-28: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Pumping Facilities and PP as LID SC4(ii)

Figure 5-29: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Pumping Facilities and WB as LID SC4(iii)

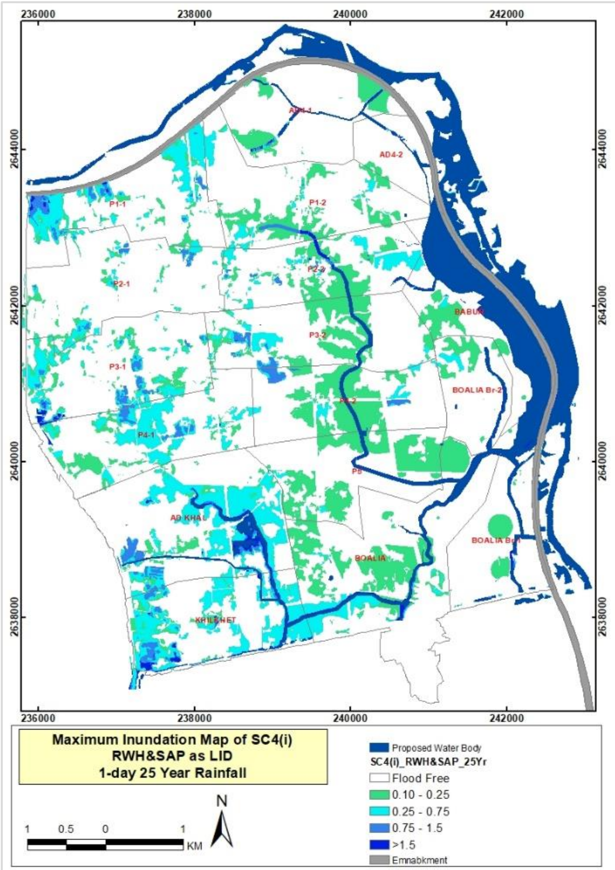


Figure 5-30: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Pumping Facilities and RWH-SWPs as LID SC4(i)

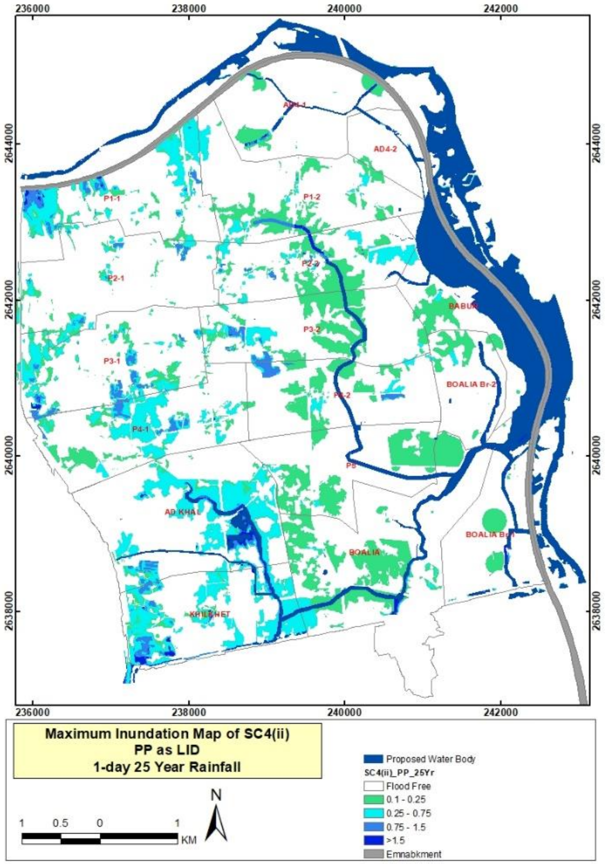


Figure 5-31: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Pumping Facilities and PP as LID SC4(ii)

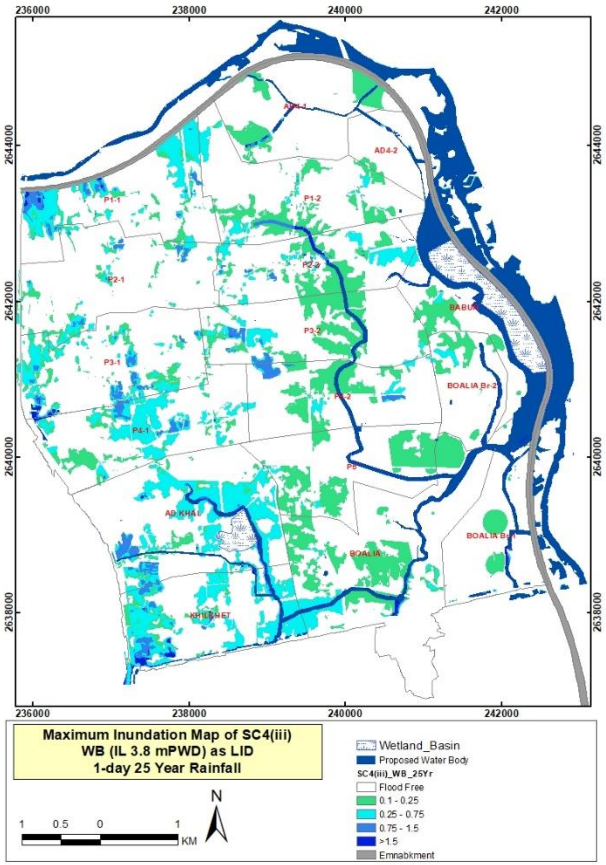
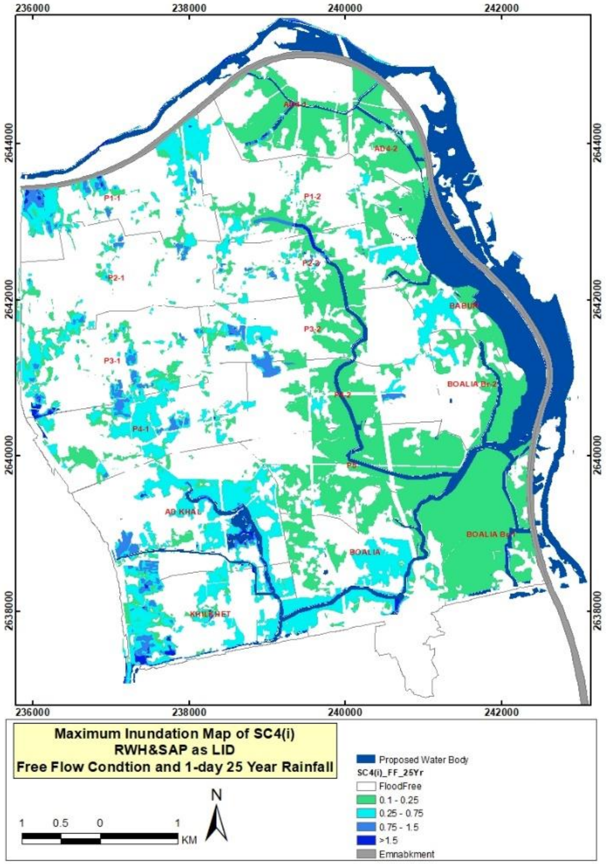
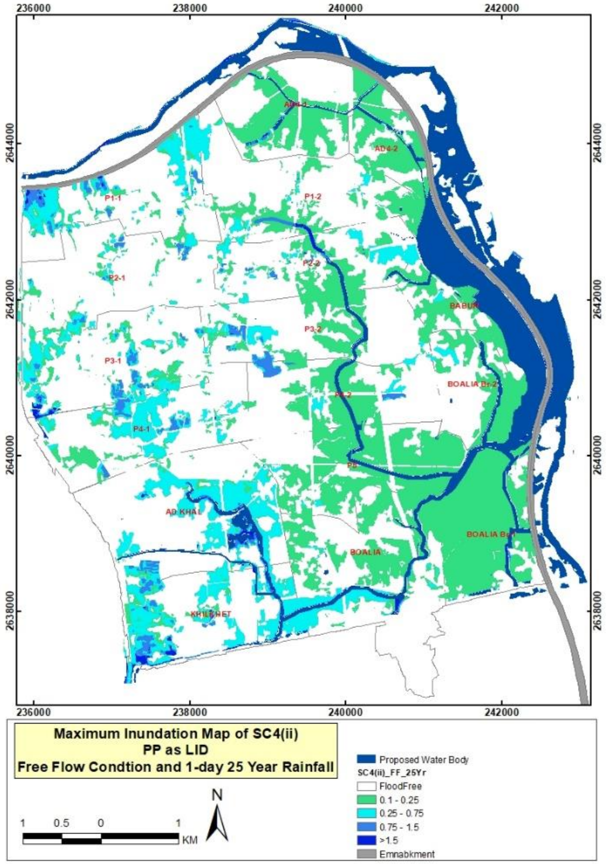


Figure 5-32: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Pumping Facilities and WB as LID SC4(iii)





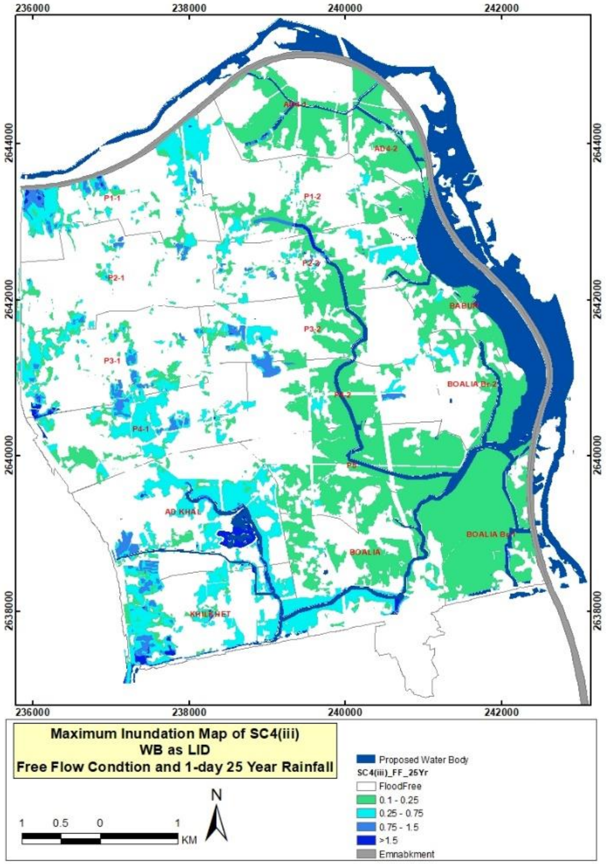


Figure 5-33: Flood Inundation Map in Free Flow Condition under a 25 Yr. Rainfall without Pumping Facilities and RWH-SWPs as LID SC4(i)

Figure 5-34: Flood Inundation Map in Free Flow Condition under a 25 Yr. Rainfall without Pumping Facilities and PP as LID SC4(ii)

Figure 5-35: Flood Inundation Map in Free Flow Condition under a 25 Yr. Rainfall without Pumping Facilities and WB as LID SC4(iii)

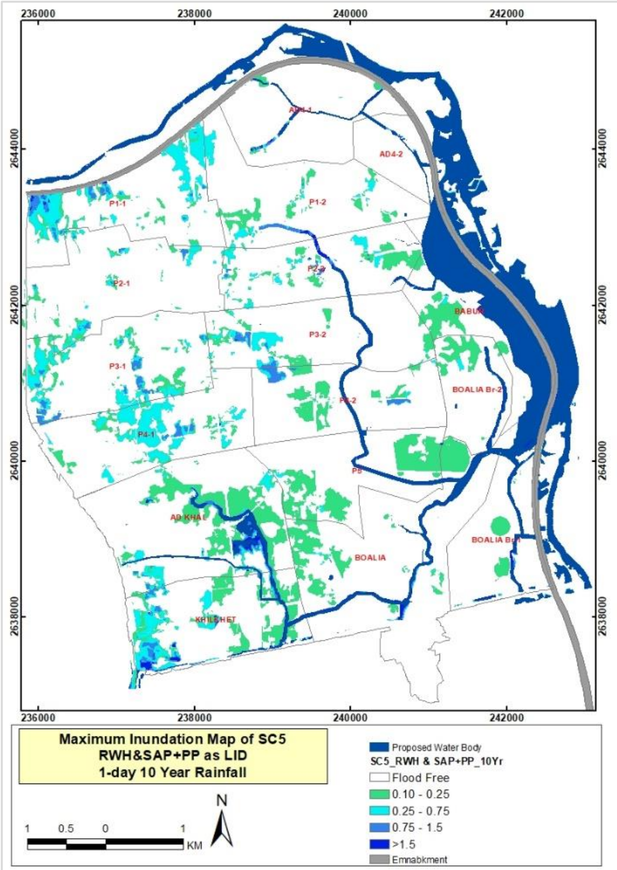


Figure 5-36: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Pumping Facilities and RWH-SAPs+PP as LIDs (SC5)

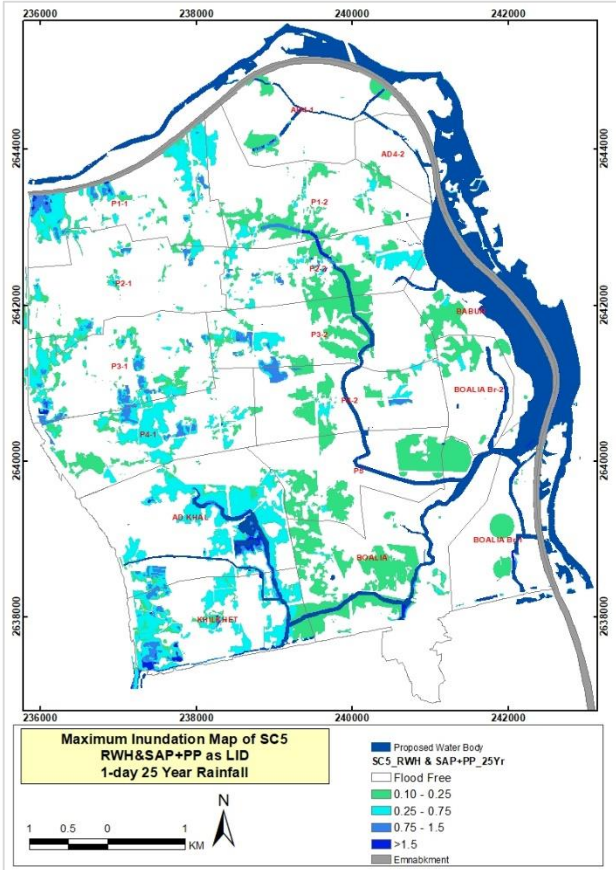


Figure 5-37: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Pumping Facilities and RWH-SAPs+PP as LIDs (SC5)

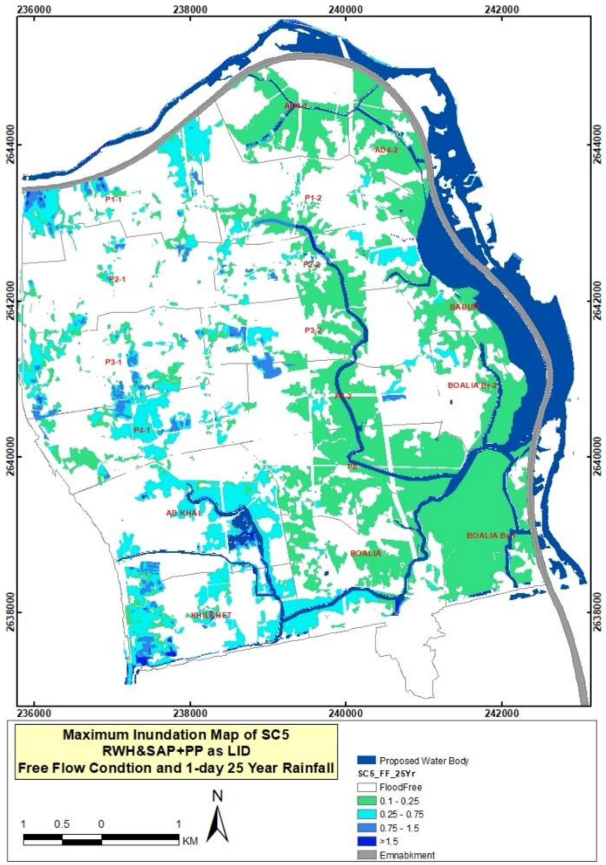


Figure 5-38: Flood Inundation Map in Free Flow Condition under a 25 Yr. Rainfall without Pumping Facilities and RWH-SAPs+PP as LIDs (SC5)

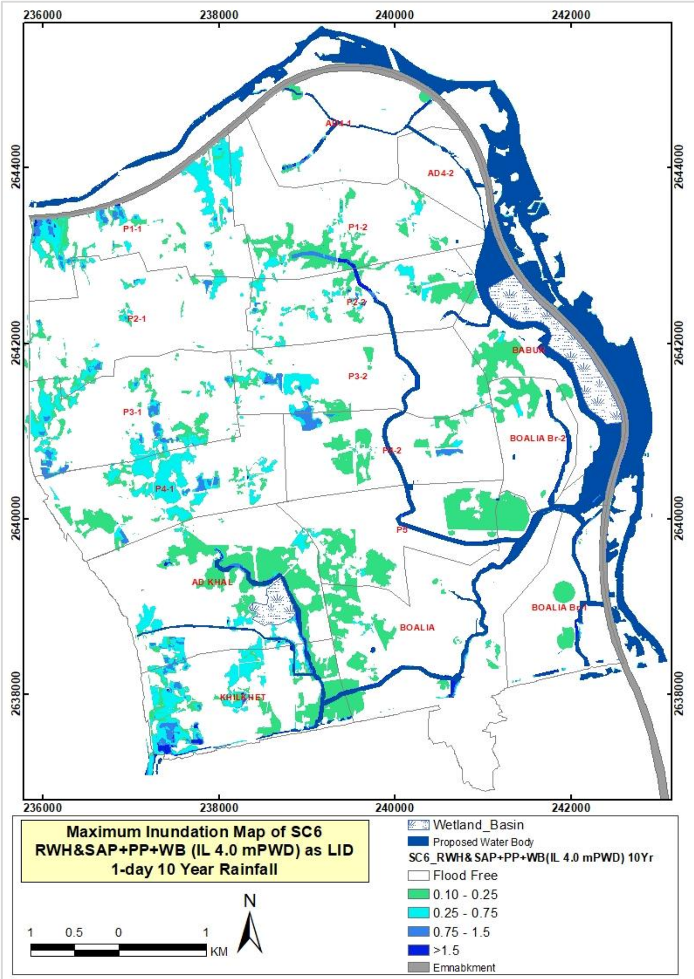


Figure 5-39: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Pumping Facilities and RWH-SWPs+ PP +WB (Inlet Invert 4.0) as LIDs (SC6)

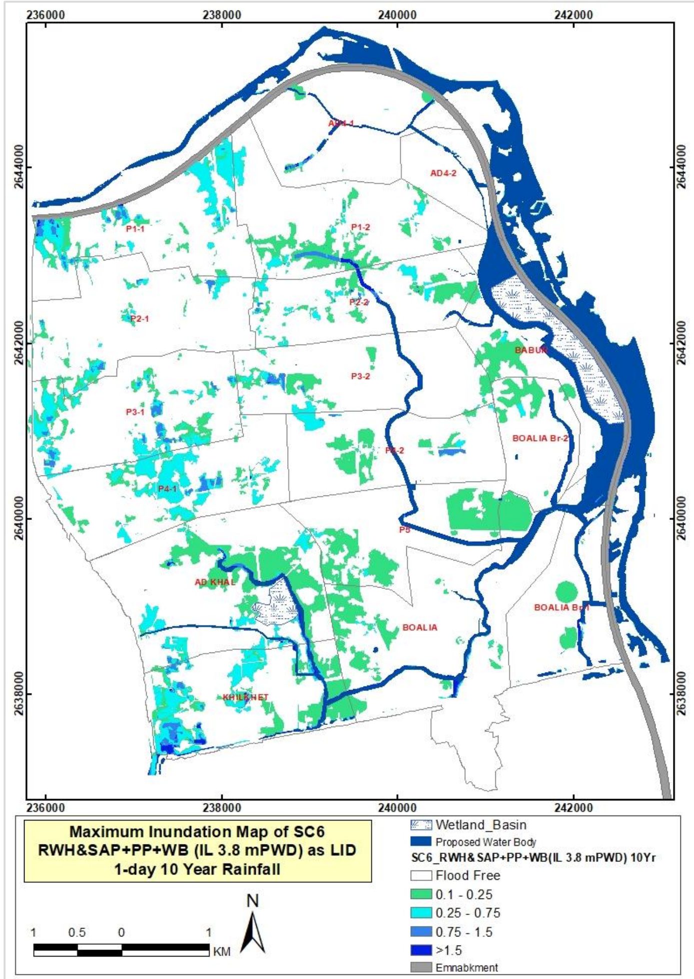


Figure 5-40: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Pumping Facilities and RWH-SWPs+PP+ WB (Inlet Invert 3.8) as LIDs (SC6)

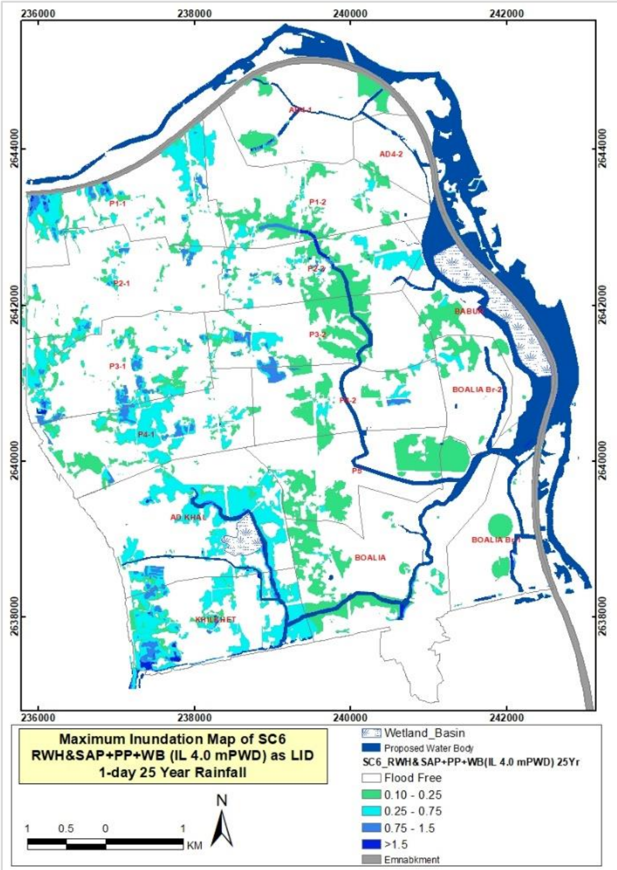


Figure 5-41: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Pumping Facilities and RWH-SWPs+PP+WB (Inlet Invert 4.0) as LIDs (SC6)

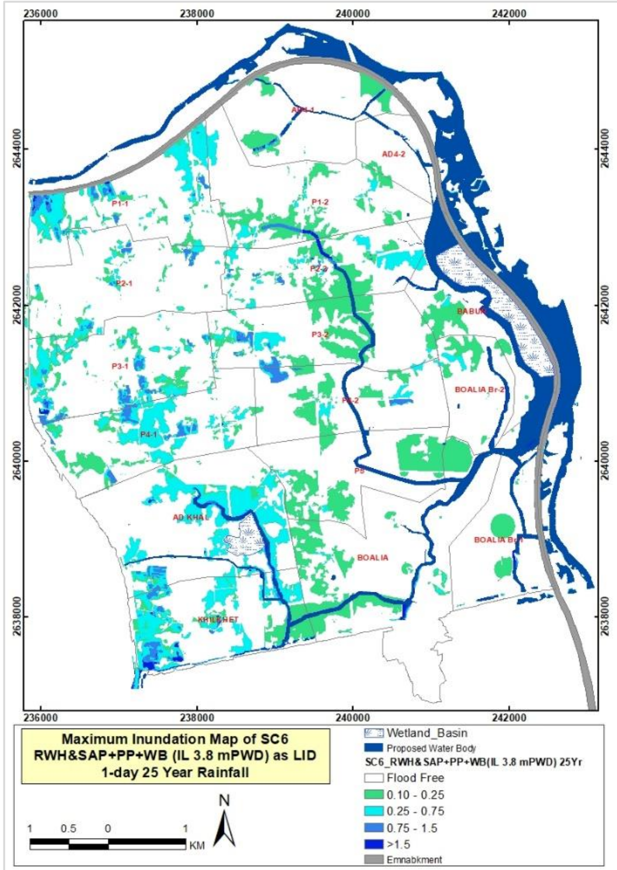


Figure 5-42: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Pumping Facilities and RWH-SWPs+PP+WB (Inlet Invert 3.8) as LIDs (SC6)

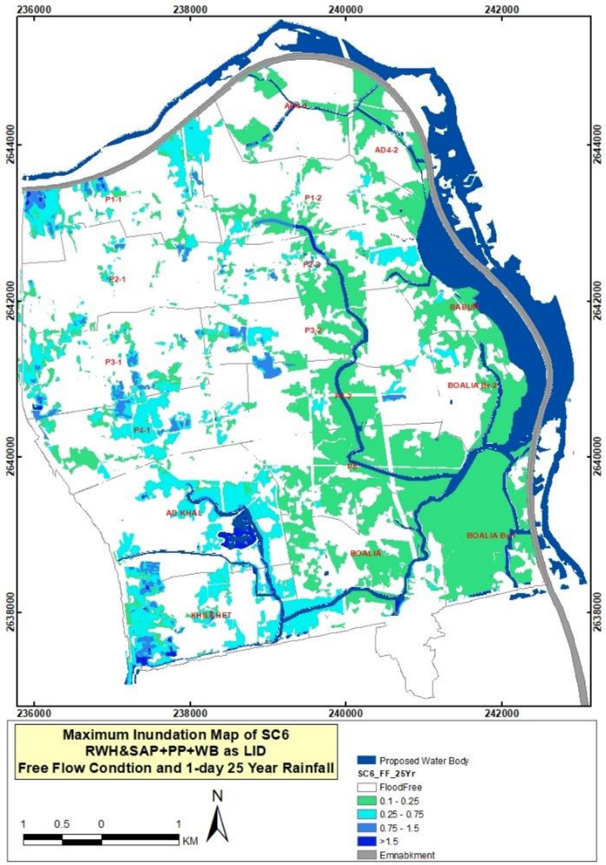


Figure 5-43: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Pumping Facilities and RWH-SWPs+PP+WB (Inlet Invert 3.8) as LIDs (SC6)

Pumping Duration

Pumping duration is higher for 10-year or 25-year storm event simulations without LID as compared to the application of any LID measures. **Figure 5-44** and **Figure 5-45** demonstrated that the individual effect of Rainwater Harvesting (RWH) & Soak-Away Pits (SAPs) and Porous Pavement (PP) cannot remarkably reduce the pumping duration. The duration of the pumping is decreased by 0.79 hours for the Boalia pump and 1.69 hours for the Babur pump for 10-year rainfall after the application of RWH-SAPs **SC4(i)** as compared with **SC3**. Similarly, the pumping duration is decreased by 1.0 hours for both pumps after the application of PP **SC4(ii)**. As a result of the application of Wetland Basins **SC4(iii)**, the pumping duration is decreased by 7.5 hours for the Boalia pump and 5.0 hours for the Babur pump for 1-day 10-year rainfall events compared to **SC3**. On the other hand, for 1-day 25-year rainfall events, the pumping duration is reduced by 4.0 hours for the Boalia pump and 3.5 hours for the Babur pump.

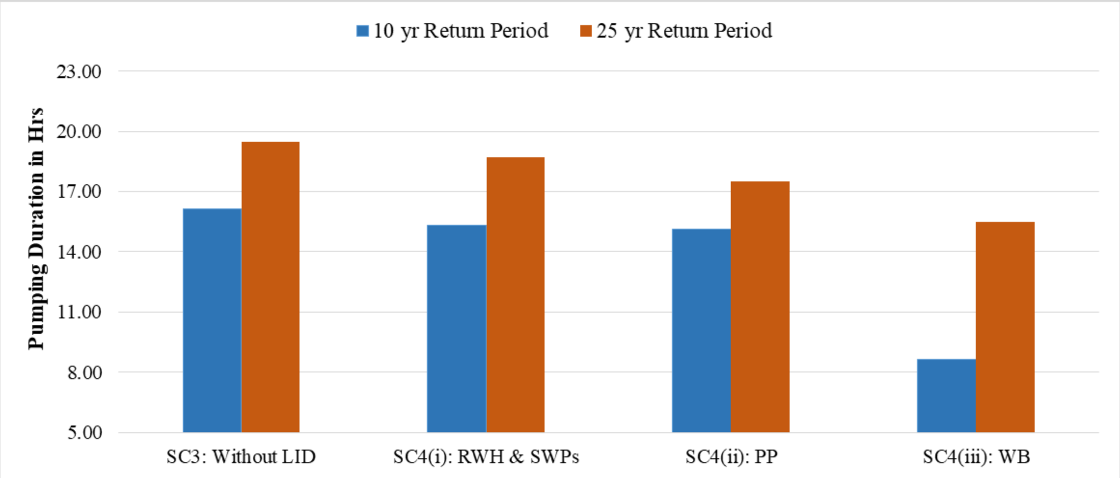


Figure 5-44: Comparison of Pumping Duration of Boalia Pump in Future Condition under a 10-year & 25-year Strom Event without LID and Different Types of Individual LID

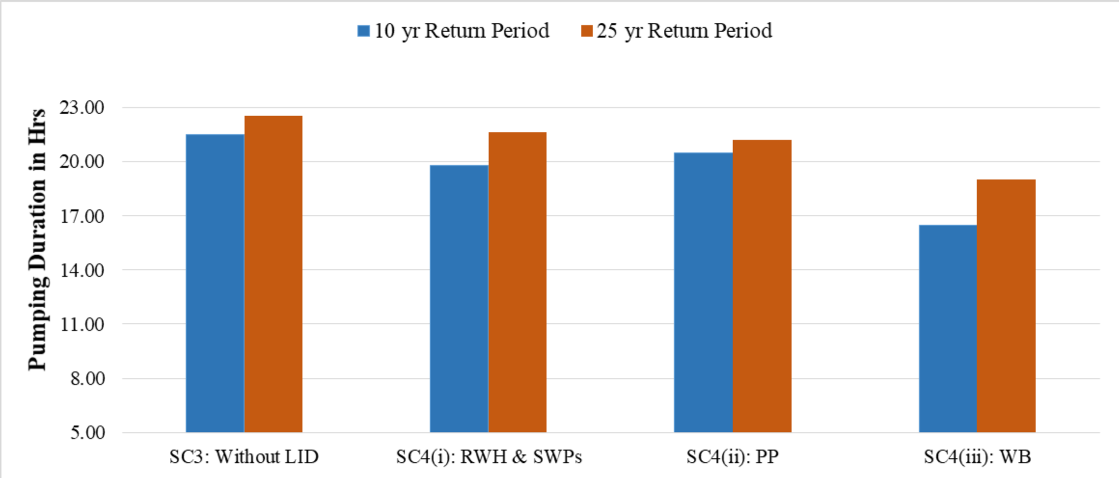


Figure 5-45: Comparison of Pumping Duration of Babur Pump in Future Condition under a 10-year & 25-year Strom Event without LID and Different Types of Individual LID

From **Figure 5-46 & Figure 5-47 and Table 5:4,** it is observed that the pumping duration is decrease more when the wetland basin is applied compared to other LID. This is more effective when the inlet control structures' invert level of the wetland will be set (3.8 mPWD) lower than the pump start level.

The pumping duration is decreased by 14% for both Boalia and Babur pumps for 10-year rainfall events **SC5** compared to **SC3** (**Figure 5-46** and **Table 5:4**). On the other hand, for 25-year rainfall events **SC**5 compared to **SC3**, pumping duration is reduced by 14% for Boalia pump and 7% for the Babur pump (**Figure 5-47** and **Table 5:4**). The application of Rainwater Harvesting & Soak-Away Pits and Porous Pavement may reduce the pumping duration maximum of 14%.

Table 5:4: Reduction of Pumping Duration for Application of LID as Compared to SC3

LID Measures	Boalia Pump		Babur Pump	
	10- year Rainfall	25-year Rainfall	10- year Rainfall	25-year Rainfall
RWH-SWPs, SC4(i)	5%	4%	8%	4%
PP, SC4(ii)	6%	10%	5%	6%
WB, SC4(iii)	46%	21%	23%	16%
RWH-SWPs + PP, SC5	14%	14%	14%	7%
RWH-SWPs + PP + WB, SC6	53%	52%	56%	35%

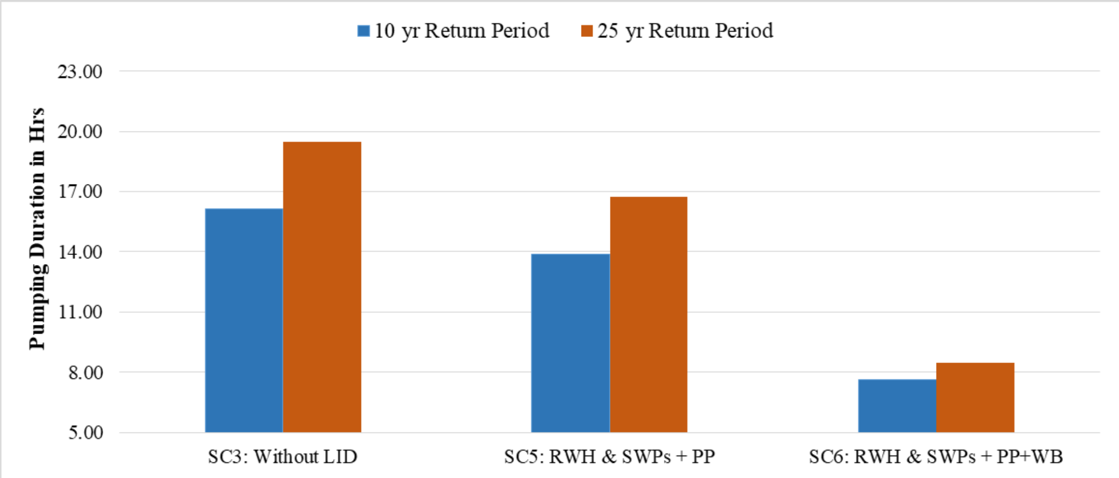


Figure 5-46: Comparison of Pumping Duration from Boalia Khal in Future Condition under a 10-year & 25-year Strom Event with and without Low Impact Development

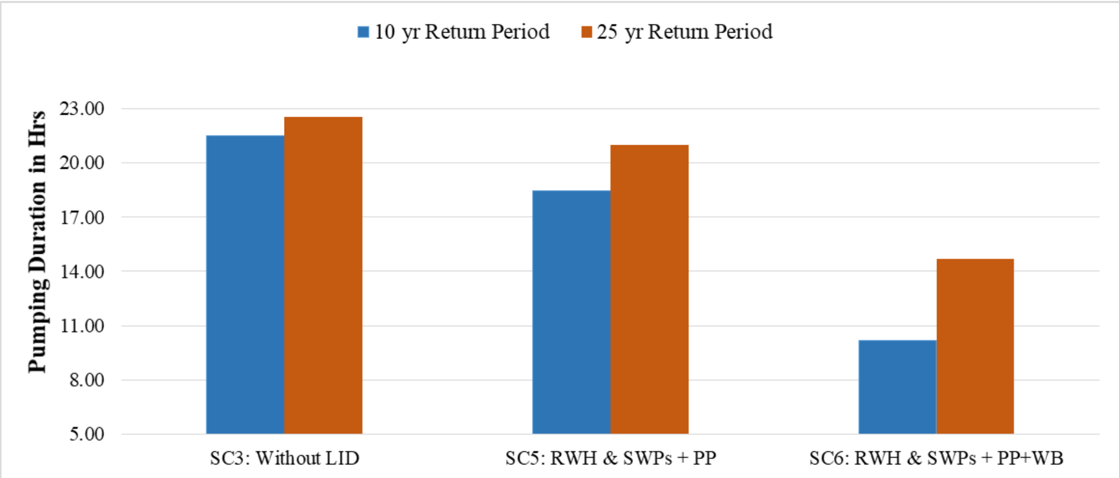


Figure 5-47: Comparison of Pumping Duration from Babur Khal in Future Condition under a 10-year and 25-year Strom Event with and without Low Impact Development

It is interesting to note that, after the application of Wetland Basin along with Rainwater Harvesting (RWH) & Soak-Away Pits and Porous Pavement, the average pumping duration is reduced by 53% i.e about 10 hours for 10-year rainfall and 46% i.e 9.4 hours for 25-year rainfall **(Figure 5-46 & Figure 5-47 and Table 5:4**). So the combination of these three types of LID is more effective to reduce the pumping time for the future drainage system with pumping facilities. Along with reducing pumping duration, the pumping capacity of the Boalia pump could also be reduced to 35 m^3/s from 42 m^3/s for 10-year rainfall.

5.3.4 Performance of Low Impact Development under Climate Change Impact

To assess the effect of climate change impact on the future planned drainage system of the study area and evaluate the performance of LID, the drainage model was simulated for 2050 with future drainage systems and rainfall event of different return periods with climate change. In a changing climate, it is expected that Dhaka will experience a 16 percent increase in extreme 24-hour precipitation by 2050. The statistical downscaling used in Sugiyama (2012) estimated a 16 percent increase factor in extreme 24-hour precipitation for Dhaka in 2050 for the high-emissions, fossil-fuel intensive (A1FI) scenario (Dasgupta, Zaman et al. 2015) [27].

The flood maps generated by the simulation of the drainage model of different return periods of rainfall with climate change are presented in **Figure 5-51 to Figure 5-54**. The results were analyzed using Arc GIS for determining the flooded area for different inundation depths and pumping duration. The results are presented in **Table 5:5** and illustrated in **Figure 5-48**.

Table 5:5: Summary of Flooding with Future drainage system and 2050 climate change

Scenario	Return Period (Year)	Max Flooded Area (km^2)					Pumping Duration	
		0.1-0.25m	0.25-0.75m	0.75-1.5m	>1.5m	Total	Boalia Pump	Babur Pump
SC3	10	3.77	2.71	0.56	1.69	8.73	16.12	21.47
	25	5.70	3.53	0.73	1.75	11.71	19.47	22.52
SC3-CC	10+CC	5.24	3.39	0.69	1.73	11.05	18.78	21.92
	25+CC	6.57	4.33	1.18	1.81	13.90	25.60	26.28
SC6-CC	10+CC	4.52	2.89	0.60	1.60	9.61	11.07	12.78
	25+CC	5.42	3.77	0.83	1.68	11.69	14.88	17.50

As the rainfall increases by 16% due to the effect of climate change, the flood extent of the study area increases by 27 % and 19% for 10-year and 25-year return period rainfall respectively. After the application of LID with climate change rainfall scenario (**SC6-CC**), the inundation area is more as compared to the without LID (**SC3**) scenario. Although the area of shallow flood increases, nevertheless the area of deep flood decreases. The comparison of simulation results between the climate change rainfall without LID (**SC3-CC**) and climate change rainfall with LID (**SC6-CC**) shows that the flooded area decreases by 13 % and 16% for 10-year and 25-year return periods respectively.

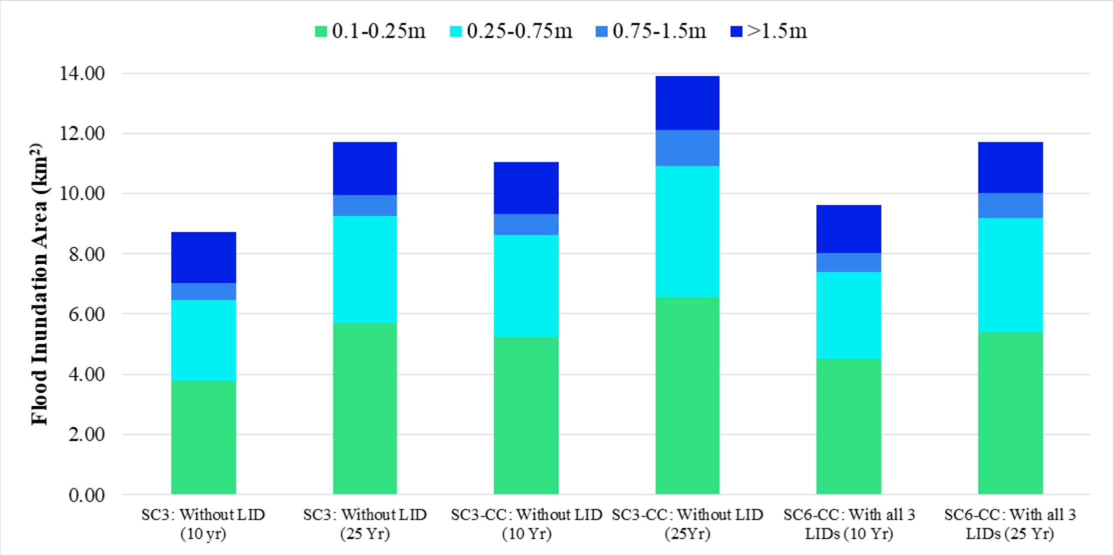


Figure 5-48: Comparison of Flood Inundation Area in Future Conditions under Climate Change Scenario with and without Low Impact Development

After induce of climate change effect on the rainfall, the average pumping duration increases by 9% and 24% for 10-year and 25-year return period rainfall respectively (**Table 5:5**). After the application of LID with climate change rainfall (**SC6-CC**) scenario, though inundation area increases but pumping duration decreases as compared to without LID (**SC3**) scenario. When climate change rainfall with LID (**SC6-CC**) and without LID (**SC3**) were compared in terms of pump duration, it was discovered that the average pumping time decreased by 36% and 23% for 10-year and 25-year return periods respectively.

The pumping duration is decreased by 41 % for both Boalia and Babur pumps for 10-year climate change rainfall events of scenario **SC6-CC** compared to **SC3-CC** **(Figure 5-49** and **Figure 5-50**). On the other hand, for 25-year climate change rainfall events of the scenario, pumping duration is reduced by 42% for the Boalia pump and 33% for the Babur pump.

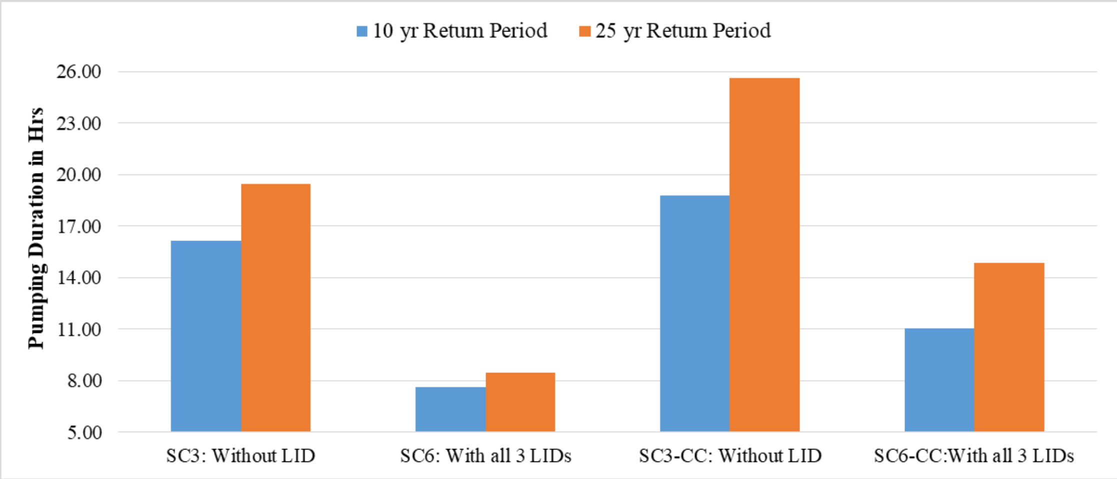


Figure 5-49: Comparison of Pumping Duration from Boalia Khal in Future Conditions under Climate Change Scenario with and without LID

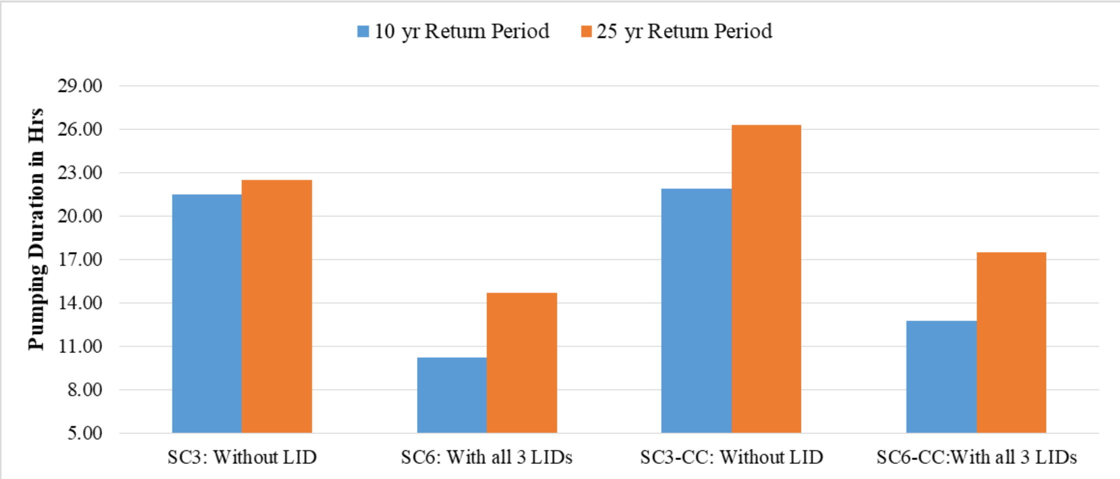


Figure 5-50: Comparison of Pumping Duration from Babur Khal in Future Conditions under Climate Change Scenario with and without LID

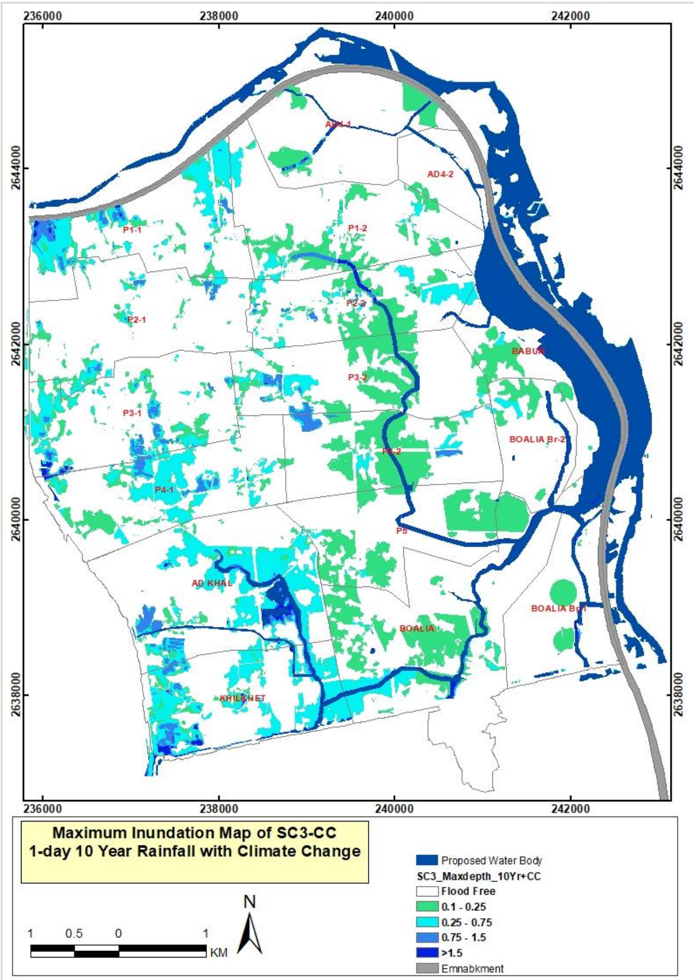


Figure 5-51: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Climate Change Impact and without LIDs (SC3-CC)

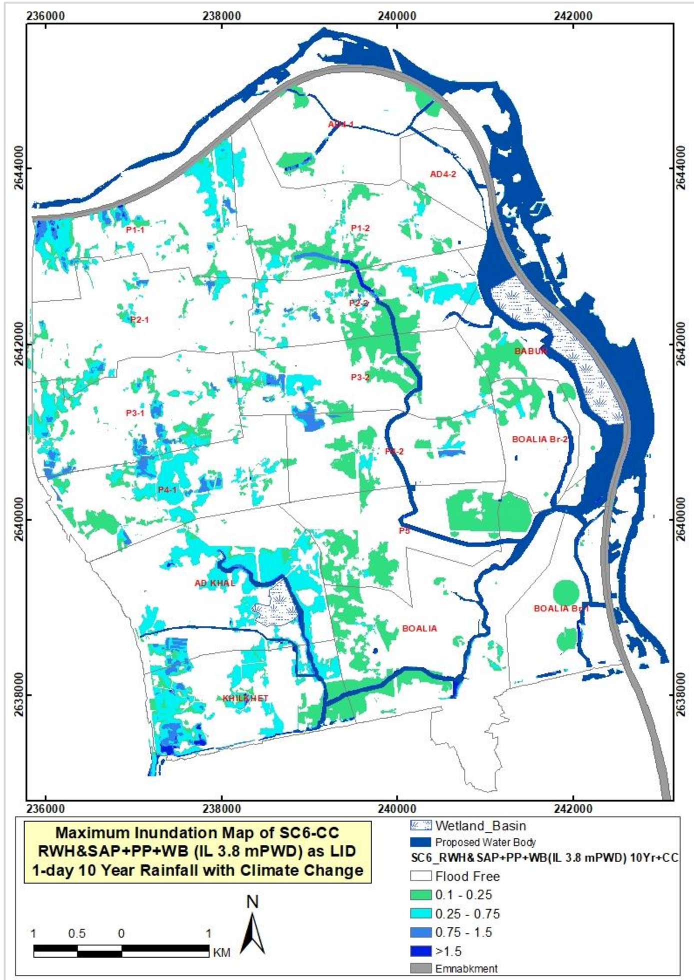


Figure 5-52: Flood Inundation Map in Future Condition under a 10 Yr. Rainfall with Climate Change Impact and with all Three LIDs (SC6-CC)

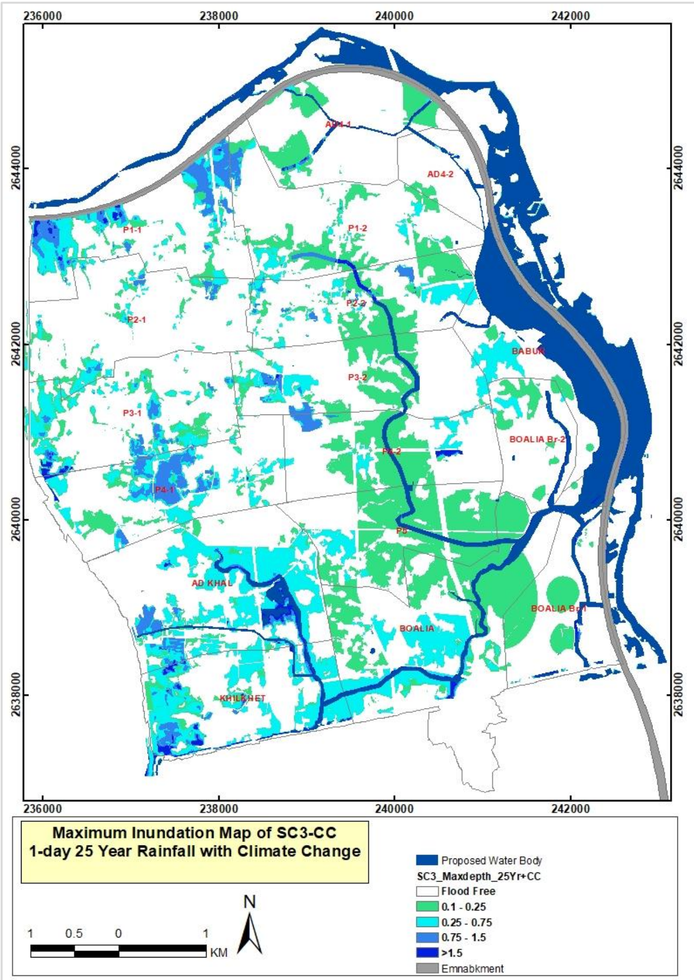


Figure 5-53: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Climate Change Impact and without LIDs (SC3-CC)

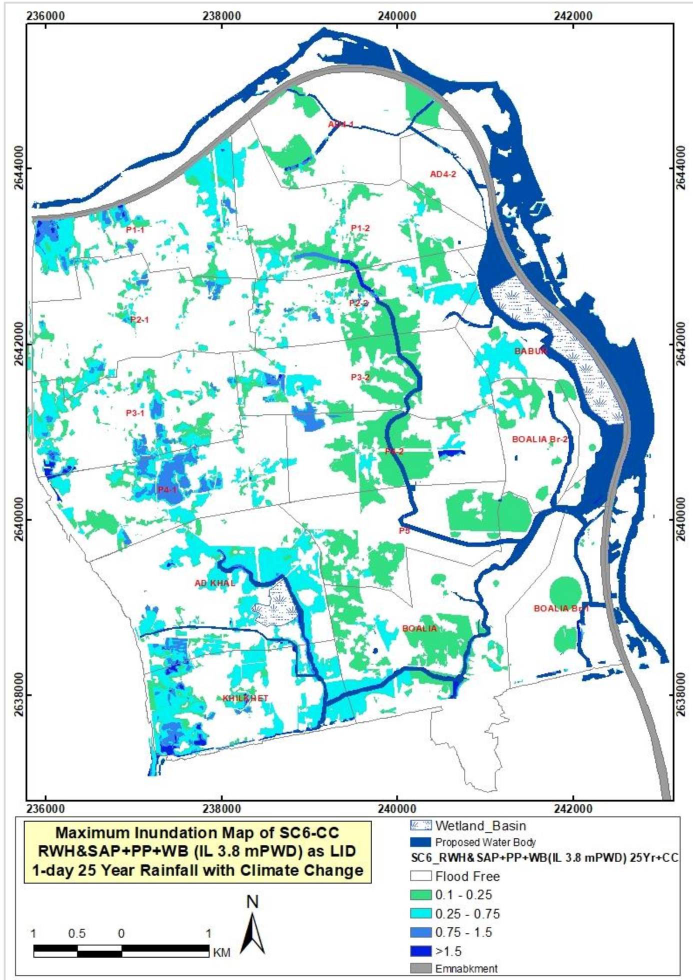


Figure 5-54: Flood Inundation Map in Future Condition under a 25 Yr. Rainfall with Climate Change Impact and with all Three LIDs (SC6-CC)

5.3.5 Performance of Low Impact Development under Extreme Rainfall (100 Yr. Event)

The flood inundation map (**Figure 5-56**) generated from the simulation of extreme rainfall event (100-Year return period) shows that more than 40% of the study area will be flooded if no LID measures are taken during the development. The flooded area for a 100-year storm event is 85% more compared to 10-year rainfall and 38% more compared to 25-year rainfall (**Figure 5-55**). While the cumulative rainfall for a 100-year storm is 50% higher than a 10-year return period event and 25% higher than a 10-year return period event.

To evaluate the performance of Low Impact Development, the extreme event (100-year) rainfall was simulated with an optimum drainage plan after the application of LID. The simulation result shows that the flooded area for a 100-year storm is decreased by 20% after the application of LID as compared to without LID (**Figure 5-57**).

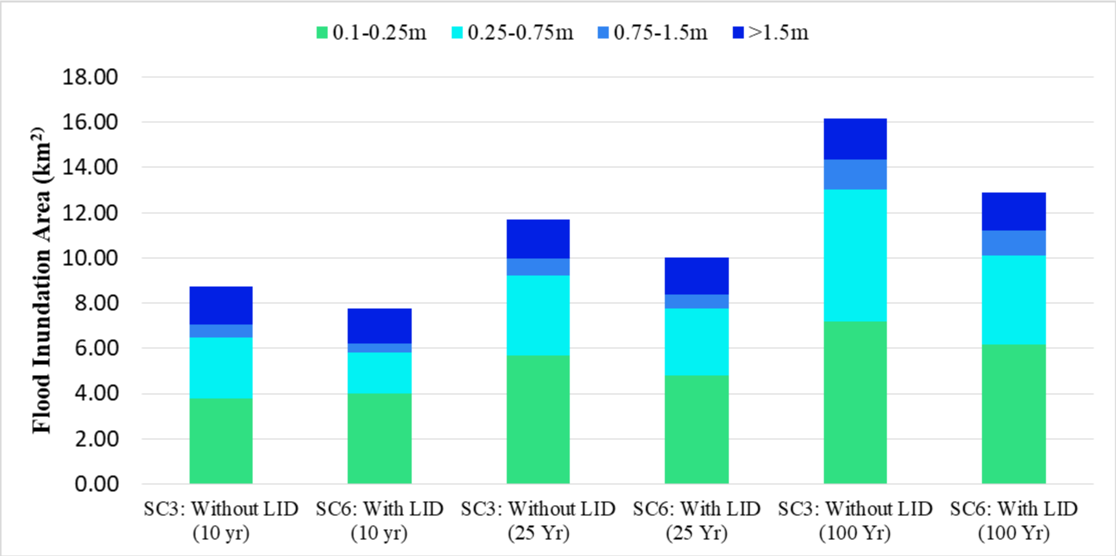
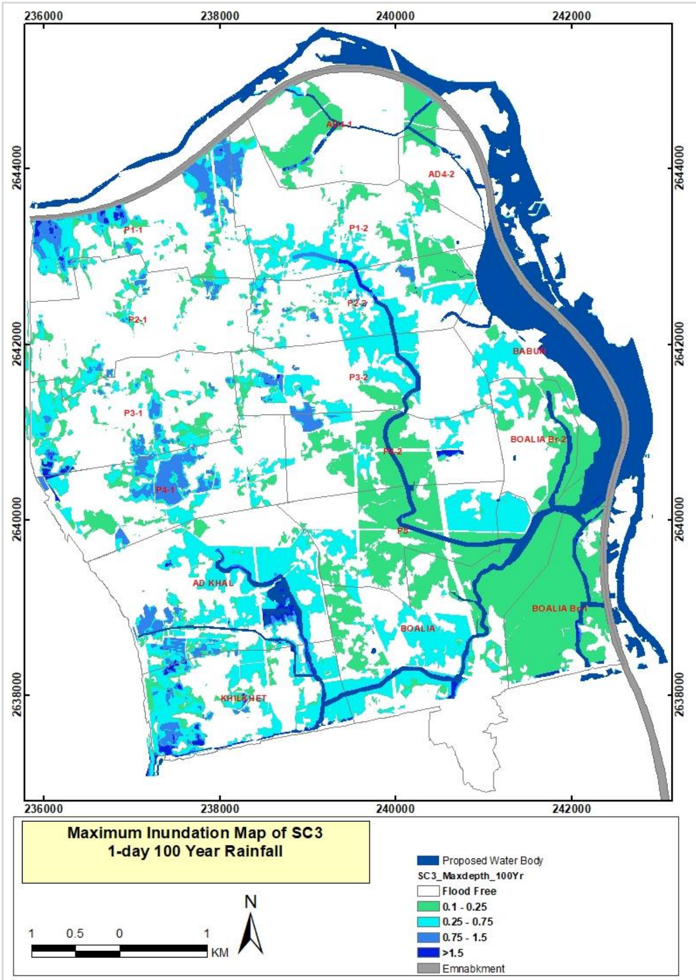


Figure 5-55: Comparison of Flood Inundation Area in Future Conditions under Extreme Rainfall Event with and without Low Impact Development



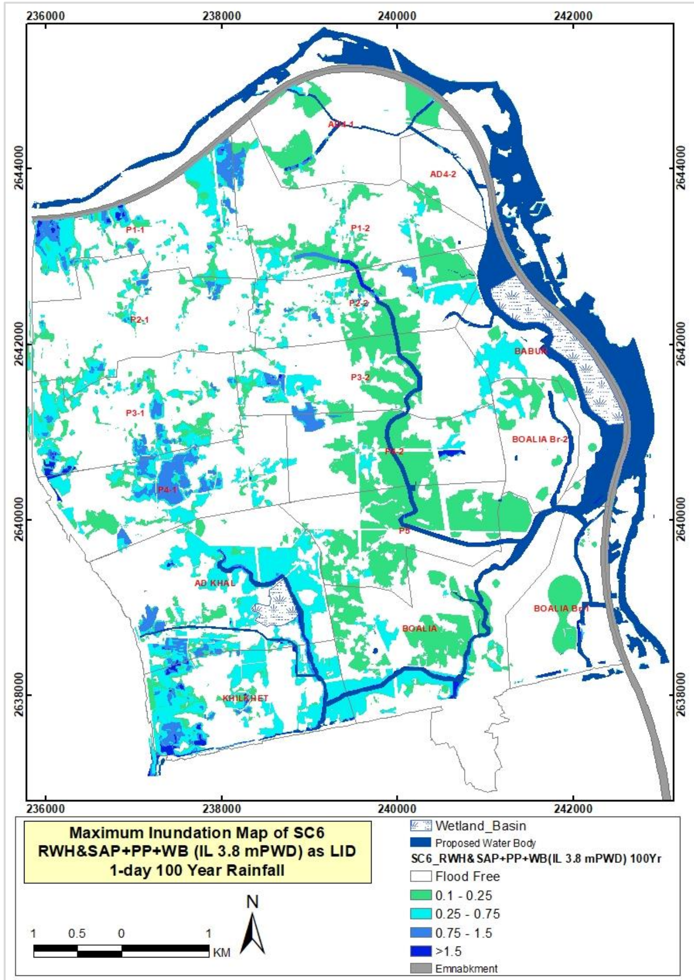


Figure 5-56: Flood Inundation Map in Future Condition under a 100 Yr. Return Period Rainfall Event without LIDs

Figure 5-57: Flood Inundation Map in Future Condition under a 100 Yr. Return Period Rainfall Event with LIDs

As illustrated in **Figure 5-58** and **Figure 5-59**, the average pumping duration for a 100-year storm event without LID increased by 68% compared to 10-year rainfall and 50% compared to 25-year rainfall. While the cumulative rainfall for a 100-year storm is 50% higher than a 10-year return period event and 25% higher than a 10-year return period event. The average pumping time for a 100-year storm decreased by 43% after the use of LID compared to without LID.

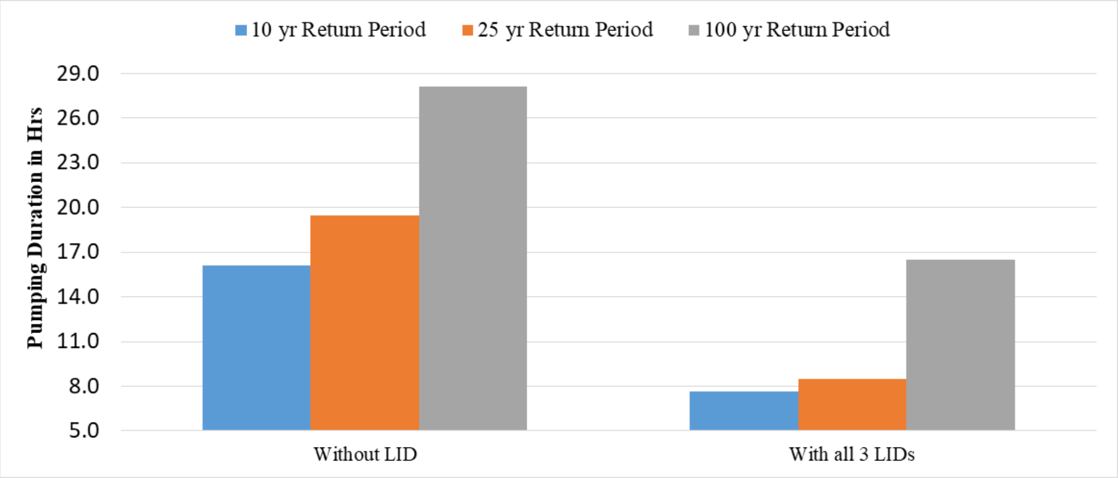


Figure 5-58: Comparison of Pumping Duration from Boalia Khal in Future Conditions under Different Rainfall Events with and without LID

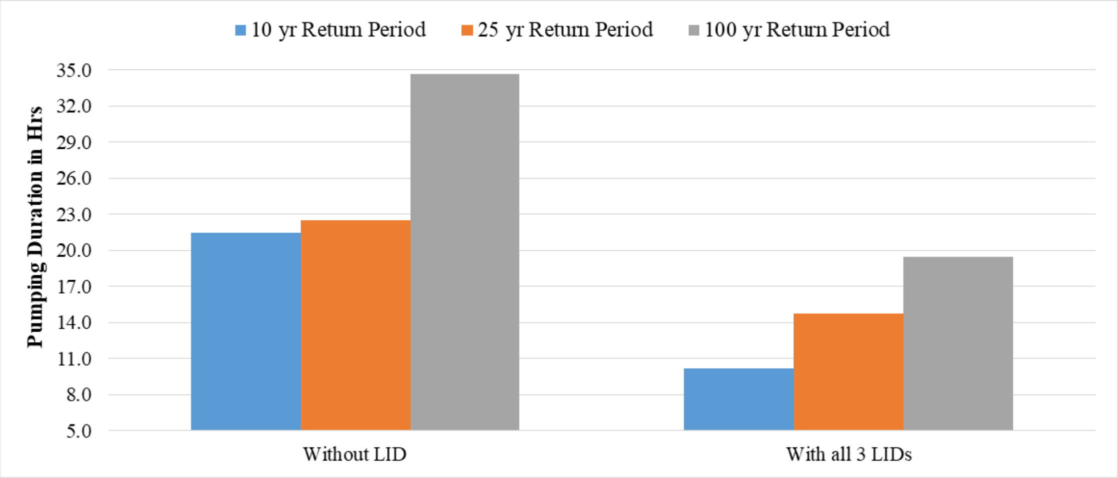


Figure 5-59: Comparison of Pumping Duration from Babur Khal in Future Conditions under Different Rainfall Events with and without LID

5.4 Discussion on Model Results

From the results of scenario analysis, it is concluded that the existing network's performance under different storm events revealed that flood extents were significantly affected by river water levels. The future drainage network's performance showed that the addition of flood protection measures along the Balu River reduced flood extents.

The application of Low Impact Development (LID) strategies, such as rainwater harvesting, porous pavement, and wetland basins, further improved the future drainage system's performance by reducing flood extents and pumping durations.

The application of porous pavement is effective for the reduction of flood extent and WBs are more effective for the reduction of pumping duration along with flood extent.

It was found that climate change could lead to increased flood extents and longer pumping durations. However, the application of LID strategies mitigated these impacts, reducing flood extents and pumping durations under climate change scenarios.

Chapter 6

Conclusion and Recommendations

6.1 Conclusion

This study focused on delineating the drainage sub-catchment and its parameters for both present and future conditions (2050). This delineation was achieved through the analysis of landuse maps, digital elevation maps, and the physical attributes of drainage infrastructures. Frequency analysis was conducted for various return periods using historical rainfall data from 1953 to 2020. The Gumbel method, coupled with Chi-Square test outcomes and IDF curves, was employed for this purpose. Additionally, peak rainfall intensity was determined by applying the Alternating Block Method to distribute design rainfalls.

The research involved the development of hydrological and 1D-2D coupled hydrodynamic drainage models using the DHI MIKE URBAN interface. These models were tailored for both existing and projected drainage scenarios. Multiple scenarios were simulated to cover return periods of 10-year, 25-year, and 100-year. This research mainly focuses on investigating the influence of the application of different types of LID measures such as Rainwater Harvesting (RWH), Soak-Away Pits (SAPs), Porous Pavement (PP), and Wetland Basin (WB) in future drainage conditions

To assess the effectiveness of these LID measures, the simulation results from the models were analyzed in terms of peak water levels, the extent of flooding, and the duration of pumping in the Boalia khal catchment area. Moreover, an optimization investigation was carried out for the size and invert level of the inlets and outlets of the wetland basin. The conclusions drawn from this study are as follows:

- ❖ The models were successfully calibrated and validated by comparing simulated water levels and discharge with observed data.
- ❖ Present flooding in the study area is primarily influenced by the water level conditions of the Balu River. The Boalia khal and surrounding khal catchments experience significant flooding during high river water levels.
- ❖ Implementing flood protection embankments along the Balu River and other plans proposed by RAJUK, BWDB, and DNCC are expected to reduce the overall flood extent in the future.

- For a 25-year return period event, the flood extent increased by 34% compared to a 10-year return period event, while the cumulative rainfall for a 25-year event was 20% higher.
- Only Rainwater Harvesting (RWH)-Soak-Away Pits (SAPs) cannot effectively reduce the peak water level and flooding area. Wetland Basins showed a more pronounced decrease in water levels.
- The peak water level is reduced by 8.4 cm at upstream of Boalia Khal and by 19.2 cm at downstream for a 25-year rainfall event after the application of low impact development measures.
- The simulation results of individual LID techniques indicate that Porous Pavement is effective in reducing flood extent compared to RWH-SAPs, while Wetland Basins are more effective in reducing both pumping duration and flood extent.
- The extent of the flood is decreased by 10.5 % as a result of the use of Porous Pavement.
- The application of Wetland Basins in the future drainage system leads to significant reductions in pumping duration. Specifically, for 1-day 10-year rainfall events, the pumping duration decreases by 7.5 hours (46%) in the Boalia pump and 5.0 hours (23%) in the Babur pump. Similarly, for 1-day 25-year rainfall events, the pumping duration is reduced by 4.0 hours (21%) in the Boalia pump and 3.5 hours (16%) in the Babur pump.
- The combined (RWH-SAPs, PP and WB together) application of LID measures decreased the flood-affected area by 14% for a 10-year storm event and 15% for a 25-year storm event compared to without LID.
- Integrating RWH-SAPs and PP in the future drainage system may reduce pumping duration by up to 14%, equivalent to 3.0 hours. The addition of Wetland Basins alongside these LID measures can lead to a more substantial reduction of pumping duration up to 53%, i.e. 10 hours for a 10-year storm event.
- As the rainfall increases by 16% due to the effect of climate change, the flood extent of the study area increases by 27 % &19% and pumping duration increases by 9% and 24% for 10-year and 25-year return periods of rainfall respectively.
- Simulation results under climate change scenarios indicated that implementing LID measures could significantly decrease flooded areas and pumping durations.

- In case of extreme rainfall event (100-Year return period) occurred, more than 40% of the study area may be flooded in future if no LID measures are taken during the development.
- LID measures reduced the flooded area by 20% and average pumping duration by 43% for a 100-year storm event.

6.2 Recommendations

- To protect the Boalia Khal Catchment from flood damage in the future the flood protection embankment along the Balu River, pumping facilities and other plans of RAJUK, BWDB and DNCC should be implemented appropriately.
- In this study, the rainfall hyetograph has been generated by considering Alternating Block Method (ABM). However, as the system will be pump dependent in future, to get better solutions to alleviate drainage congestion the system may be examined using the design hyetograph proposed by JICA study (1987) for Dhaka and other available methods. Therefore, there is scope to establish an effective design hyetograph by exploring different methods, since the rainfall distribution methods results reveal that outcomes are not identical.
- To get better performance of LID in terms of reduction of flood extent as well as reduction of pumping duration, Rainwater Harvesting & Soak-Away Pits, Porous Pavement and Wetland Basin should be incorporated with the future drainage system combined.
- Physical models of porous pavement and wetland basin can be developed. And evaluated results can be used in adaptive measures simulation.
- To gather a better understanding of the Performance of LID, analysis of flood duration and flood damage can be assessed.

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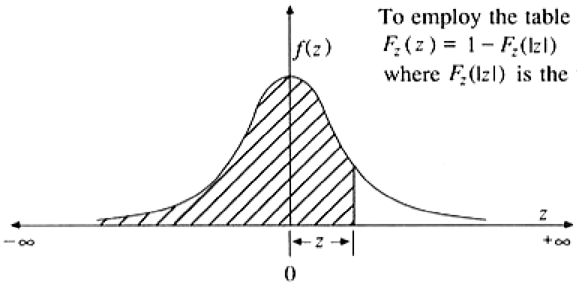
Appendix

Table 1: Cumulative probability of the Standard Normal Distribution

Cumulative probability of the standard normal distribution

z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7517	0.7549
0.7	0.7580	0.7611	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9864	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936
2.5	0.9938	0.9940	0.9941	0.9943	0.9945	0.9946	0.9948	0.9949	0.9951	0.9952
2.6	0.9953	0.9955	0.9956	0.9957	0.9959	0.9960	0.9961	0.9962	0.9963	0.9964
2.7	0.9965	0.9966	0.9967	0.9968	0.9969	0.9970	0.9971	0.9972	0.9973	0.9974
2.8	0.9974	0.9975	0.9976	0.9977	0.9977	0.9978	0.9979	0.9979	0.9980	0.9981
2.9	0.9981	0.9982	0.9982	0.9983	0.9984	0.9984	0.9985	0.9985	0.9986	0.9986
3.0	0.9987	0.9987	0.9987	0.9988	0.9988	0.9989	0.9989	0.9989	0.9990	0.9990
3.1	0.9990	0.9991	0.9991	0.9991	0.9992	0.9992	0.9992	0.9992	0.9993	0.9993
3.2	0.9993	0.9993	0.9994	0.9994	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995
3.3	0.9995	0.9995	0.9995	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9997
3.4	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9998

Source: Grant, E. L., and R. S. Leavenworth, *Statistical Quality and Control*, Table A, p.643, McGraw-Hill, New York, 1972. Used with permission.

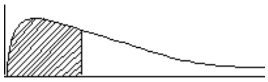


To employ the table for $z < 0$, use
$F_z(z) = 1 - F_z(|z|)$
where $F_z(|z|)$ is the tabulated value.

Table 2: Frequency factors for Extreme Value I Distribution

	Return Period								
Sample	5	10	15	20	25	50	75	100	1000
15	0.967	1.703	2.117	2.410	2.632	3.321	3.721	4.005	6.265
20	0.919	1.625	2.023	2.302	2.517	3.179	3.563	3.836	6.006
25	0.888	1.575	1.963	2.235	2.444	3.088	3.463	3.729	5.842
30	0.866	1.541	1.922	2.188	2.393	3.026	3.393	3.653	5.727
35	0.851	1.516	1.891	2.152	2.354	2.979	3.341	3.598	
40	0.838	1.495	1.866	2.126	2.326	2.943	3.301	3.554	5.576
45	0.829	1.478	1.847	2.104	2.303	2.913	3.268	3.520	
50	0.820	1.466	1.831	2.086	2.283	2.889	3.241	3.491	5.478
55	0.813	1.455	1.818	2.071	2.267	2.869	3.219	3.467	
60	0.807	1.446	1.806	2.059	2.253	2.852	3.200	3.446	
65	0.801	1.437	1.796	2.048	2.241	2.837	3.183	3.429	
70	0.797	1.430	1.788	2.038	2.230	2.824	3.169	3.413	5.359
75	0.972	1.423	1.780	2.029	2.220	2.812	3.155	3.400	
80	0.788	1.417	1.773	2.020	2.212	2.802	3.145	3.387	
85	0.785	1.413	1.767	2.013	2.205	2.793	3.135	3.376	
90	0.782	1.409	1.762	2.007	2.198	2.785	3.125	3.367	
95	0.780	1.405	1.757	2.002	2.193	2.777	3.116	3.357	
100	0.779	1.401	1.752	1.998	2.187	2.770	3.109	3.349	5.261
α	0.719	1.305	1.635	1.866	2.044	2.592	2.911	3.137	4.936

Table 3: χ^2 Distribution



χ^2

DOF ν	$\chi^2_{.995}$	$\chi^2_{.99}$	$\chi^2_{.975}$	$\chi^2_{.95}$	$\chi^2_{.90}$	$\chi^2_{.75}$	$\chi^2_{.50}$	$\chi^2_{.25}$	$\chi^2_{.10}$	$\chi^2_{.05}$	$\chi^2_{.025}$	$\chi^2_{.01}$	$\chi^2_{.005}$
1	7.88	6.63	5.02	3.84	2.71	1.32	0.455	0.102	0.0158	0.0039	0.0010	0.0002	0.0000
2	10.6	9.21	7.38	5.99	4.61	2.77	1.39	0.575	.211	.103	.0506	.0201	.0100
3	12.8	11.3	9.35	7.81	6.25	4.11	2.37	1.21	.584	.352	.216	.115	.072
4	14.9	13.3	11.1	9.49	7.78	5.39	3.36	1.92	1.06	.711	.484	.297	.207
5	16.7	15.1	12.8	11.1	9.24	6.63	4.35	2.67	1.61	1.15	.831	.554	.412
6	18.5	16.8	14.4	12.6	10.6	7.84	5.35	3.45	2.20	1.64	1.24	.872	.676
7	20.3	18.5	16.0	14.1	12.0	9.04	6.35	4.25	2.83	2.17	1.69	1.24	.989
8	22.0	20.1	17.5	15.5	13.4	10.2	7.34	5.07	3.49	2.73	2.18	1.65	1.34
9	23.6	21.7	19.0	16.9	14.7	11.4	8.34	5.90	4.17	3.33	2.70	2.09	1.73
10	25.2	23.2	20.5	18.3	16.0	12.5	9.34	6.74	4.87	3.94	3.25	2.56	2.16
11	26.8	24.7	21.9	19.7	17.3	13.7	10.3	7.58	5.58	4.57	3.82	3.05	2.60
12	28.3	26.2	23.3	21.0	18.5	14.8	11.3	8.44	6.30	5.23	4.40	3.57	3.07
13	29.8	27.7	24.7	22.4	19.8	16.0	12.3	9.30	7.04	5.89	5.01	4.11	3.57
14	31.3	29.1	26.1	23.7	21.1	17.1	13.3	10.2	7.79	6.57	5.63	4.66	4.07
15	32.8	30.6	27.5	25.0	22.3	18.2	14.3	11.0	8.55	7.26	6.26	5.23	4.60
16	34.3	32.0	28.8	26.3	23.5	19.4	15.3	11.9	9.31	7.96	6.91	5.81	5.14
17	35.7	33.4	30.2	27.6	24.8	20.5	16.3	12.8	10.1	8.67	7.56	6.41	5.70
18	37.2	34.8	31.5	28.9	26.0	21.6	17.3	13.7	10.9	9.39	8.23	7.01	6.26
19	38.6	36.2	32.9	30.1	27.2	22.7	18.3	14.6	11.7	10.1	8.91	7.63	6.84

DOF ν	$\chi^2_{.995}$	$\chi^2_{.99}$	$\chi^2_{.975}$	$\chi^2_{.95}$	$\chi^2_{.90}$	$\chi^2_{.75}$	$\chi^2_{.50}$	$\chi^2_{.25}$	$\chi^2_{.10}$	$\chi^2_{.05}$	$\chi^2_{.025}$	$\chi^2_{.01}$	$\chi^2_{.005}$
20	40.0	37.6	34.2	31.4	28.4	23.8	19.3	15.5	12.4	10.9	9.59	8.26	7.43
21	41.4	38.9	35.5	32.7	29.6	24.9	20.3	16.3	13.2	11.6	10.3	8.90	8.03
22	42.8	40.3	36.8	33.9	30.8	26.0	21.3	17.2	14.0	12.3	11.0	9.54	8.64
23	44.2	41.6	38.1	35.2	32.0	27.1	22.3	18.1	14.8	13.1	11.7	10.2	9.26
24	45.6	43.0	39.4	36.4	33.2	28.2	23.3	19.0	15.7	13.8	12.4	10.9	9.89
25	46.9	44.3	40.6	37.7	34.4	29.3	24.3	19.9	16.5	14.6	13.1	11.5	10.5
26	48.3	45.6	41.9	38.9	35.6	30.4	25.3	20.8	17.3	15.4	13.8	12.2	11.2
27	49.6	47.0	43.2	40.1	36.7	31.5	26.3	21.7	18.1	16.2	14.6	12.9	11.8
28	51.0	48.3	44.5	41.3	37.9	32.6	27.3	22.7	18.9	16.9	15.3	13.6	12.5
29	52.3	49.6	45.7	42.6	39.1	33.7	28.3	23.6	19.8	17.7	16.0	14.3	13.1
30	53.7	50.9	47.0	43.8	40.3	34.8	29.3	24.5	20.6	18.5	16.8	15.0	13.8
40	66.8	63.7	59.3	55.8	51.8	45.6	39.3	33.7	29.1	26.5	24.4	22.2	20.7
50	79.5	76.2	71.4	67.5	63.2	56.3	49.3	42.9	37.7	34.8	32.4	29.7	28.0
60	92.0	88.4	83.3	79.1	74.4	67.0	59.3	52.3	46.5	43.2	40.5	37.5	35.5
70	104.2	100.4	95.0	90.5	85.5	77.6	69.3	61.7	55.3	51.7	48.8	45.4	43.3
80	116.3	112.3	106.6	101.9	96.6	88.1	79.3	71.1	64.3	60.4	57.2	53.5	51.2
90	128.3	124.1	118.1	113.1	107.6	98.6	89.3	80.6	73.3	69.1	65.6	61.8	59.2
100	140.2	135.8	129.6	124.3	118.5	109.1	99.3	90.1	82.4	77.9	74.2	70.1	67.3

Source: Catherine M. Thompson, Table of percentage points of the χ^2 distribution, Biometrika, Vol. 32 (1941), by permission of the author and publisher.