ANALYSIS OF STORMWATER RUNOFF FOR A SELECTED CATCHMENT OF EASTERN DHAKA USING HYDROLOGIC MODEL

M.Sc. Engineering Thesis

by

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CERTIFICATION OF APPROVAL

The thesis titled "Analysis of Stormwater Runoff for a Selected Catchment of Eastern Dhaka using Hydrologic Model", submitted by Muneer Ahammad, Roll No. 0413162004 P, Session April/2013, has been accepted as satisfactory in partial fulfillment of the requirement for the degree of **Master of Science in Water Resources Engineering** on 21 March, 2018.

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DECLARATION

It is hereby declared that this thesis work or any part of it has not been submitted elsewhere for the award of any degree or diploma.

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List of Abbreviations

A1FI	IPPC scenario with future based on fossil fuel–intensive development
BMD	Bangladesh Meteorological Department
BUET	Bangladesh University of Engineering and Technology
BWDB	Bangladesh Water Development Board
CC	Climate Change
CEGIS	Center for Environmental and Geographic Information Services
DAP	Detailed Area Plan
DCC	Dhaka City Corporation
DEM	Digital Elevation Model
DMDP	Dhaka Metropolitan Development Plan
DNCC	Dhaka North City Corporation
DND	Dhaka-Narayanganj-Demra
DSCC	Dhaka South City Corporation
DWASA	Dhaka Water Supply and Sewerage Authority
FVI	Flood Vulnerability Index
GBM	Ganges-Brahmaputra-Meghna (river system)
GCM	General Circulation Model
GHG	Greenhouse Gas
GIS	Geographic Information System
IPCC	Intergovernmental Panel on Climate Change
IWFM	Institute of Water and Flood Management
IWM	Institute of Water Modelling
JICA	Japan International Cooperation Agency
LGED	Local Government Engineering Department
PWD	Public Works Department
RDP	Regional Development Plan
RAJUK	Rajdhani Unnayan Kartripakkha (Capital Development Authority of Bangladesh)
SoB	Survey of Bangladesh

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ABSTARCT

Dhaka, the capital of Bangladesh and one of the world‘s rapidly growing megacities, faces the recurring phenomena of urban flooding and waterlogging following intense rainfall nearly every year. The water level of the Brahmaputra and rainfall and runoff from the entire Brahmaputra Basin mainly influence the process of river floods in and around the city. Dhaka‘s average annual rainfall is about 2,000 mm, most of it occurring during the summer monsoon. As a low-elevation city with a tropical monsoon climate, Dhaka has a long history of river flooding as a natural hazard. A growing concern is that, in a changing climate, characterized by heavier and more erratic rainfall in the Ganges-Brahmaputra-Meghna (GBM) Basin during the monsoon season, the situation may worsen. Thus, stormwater management is a challenging task in such densely populated urban area having weak water governance and a lack in the institutional arrangement. It is even more challenging as the natural drainage system experiences relatively high water levels in the peripheral rivers during the rainy season, and the system capacity is compromised due to encroachment of the runoff detention areas.

At present, only the western part of Dhaka is protected against river floods of Turag, Buriganga, and Balu rivers. The problem of eastern side of the city covers both the external river flood and internal flood from stormwater. An embankment along the right bank of Balu River can protect the Eastern Dhaka from river flood, and then the internal stormwater can be discharged into the river with a number of drainage pumps. Against this backdrop, this study focuses on the stormwater management of Begunabari canal catchment, the largest catchment of Eastern Dhaka. The general physical setting for the study is 47.67 km^2 area, which has been modeled in GeoSWMM. The specific objectives of this study include proper calibration of the model, and then its application for future drainage improvements and adaptation measures considering future landuse and climate change impact.

In this study, the model is calibrated with observed water level data at Begunbai canal, Banasree and then drainage facilities have been designed for 2 day 10 year return period. The design rainfall value of 286 mm is increased by 16% considering the climate effects of A1FI 2050 scenario. The study‘s key findings highlight the required pumping capacities and retention facilities after implementation of Eastern Embankment. While finding this requirement, future landuse suggested by RDP, 2035 is considered along with stated climate change effect. The suggested pumping station at Begunbari canal outlet would drain out 50

cumec stormwater to maintain internal water level not to exceed 4.5m PWD at times of design storm evacuating the excess flooding within 2 days. To do such it will require 1.5 sq. km of retention pond at the pumping station. The bottom level of retention pond is considered at -0.5 mPWD while pump start level at 3.3m PWD in the monsoon. Regulators would be functional during pre-monsoon and post monsoon period when Balu is at lower level allowing gravity drainage. Here, the designed pump has been applied for 2day rainfall of 5, 10 and 25 years return period. It was observed that the inundated area with depth greater than 1.5 m can be reduced by 15-30%, while the total inundated area can be reduced by 18-25% by pumping.

The study also explored the scope of flood reduction by storage tanks of rainwater harvesting system. There are 13 subcatchments identified where future development would occur. 30% of this future development area is considered here as rooftop area and the storage tanks are designed accordingly. The design values of storage volumes are taken from an implemented project in Gujarat, India. Total 6475 m^2 storage area is considered taking the depth of tank as 10 m. The inundation depth and area both were found reduced in this case by taking measures of pumping with storage tank for rainwater harvesting. For 2 days rainfall, the area with depth greater than 1.5 m can be reduced by around 87%, while the total inundated area can be reduced by around 60% by storage tank along with the pumping. Again, the stored water can be used for household purpose and even for groundwater recharge. So, this option should be considered during future development in this area.

CHAPTER ONE
INTRODUCTION

1.1 Background

Stormwater is water generated on the land surface, originating from rainfall or melting snow or ice (Durrans, 2003). The concept of stormwater is strongly related to urban areas where conveyance systems exist. Typically stormwater drains into the nearby natural channels. River flooding refers to the inundation that occurs when excessive runoff from upstream catchments leads to the river channel‘s capacity being exceeded, resulting in water spilling over the bank; thus, the floodplain is flooded by water spilling over from the river channel. While urban flooding refers to the inundation that occurs after heavy-rainfall events, which causes runoff to exceed the capacity of the local drainage system. As a result, water remains stagnant in the built-up area for extended periods. This type of flooding is also known as rain flooding or waterlogging; however, these terms can also apply to a rural setting.

Bangladesh, being located at the downstream of the Ganges-Brahmaputra-Meghna basin is one of the most vulnerable countries to climate change. While the capital Dhaka, world‘s one of the top rapidly growing megacities, is also an urban hotspot for climate risks. Located in central Bangladesh on the lower reaches of the Ganges-Brahmaputra Delta, the city faces the recurring phenomena of urban flooding and waterlogging following intense rainfall nearly every year. As a low-elevation city with a tropical monsoon climate, Dhaka has a long history of river flooding as a natural hazard. Recent major floods have been worse in terms of depth and extent of inundation and duration. Rapid, unplanned urbanization and the gradual filling up of low-lying flood plains, rivers, canals, and other water bodies traditionally used to drain or retain water during rainfall have exacerbated the problem. A growing concern is that, in a changing climate, characterized by heavier and more erratic rainfall in the monsoon, the situation may worsen.

Stormwater management is a challenging task in urban areas, particularly in relatively densely populated cities having weak water governance and a lack in the institutional arrangement. It is even more challenging when the natural drainage system experiences relatively high water levels in the peripheral rivers during the rainy season, or if the system capacity is compromised due to encroachment of the runoff detention areas. Having these problems, flooding during the monsoon season is a regular phenomenon in Dhaka. Here, monsoon causes huge rainfall during a concentrated part of the year. This rain water is naturally drained by mostly a gravity drainage system while wetlands and low lands work as natural retention storage. Urbanization, particularly unplanned urbanization, has hampered this natural state of drainage, and hence causes sudden inundation and water-logging. The wetlands of Dhaka City are changing very rapidly. In 1960, the total area of water bodies and lowlands were 2952.02 and 13527.58 ha, respectively whereas in 1988, the total area of the same decreased to 2103.62 and 12717.73 ha, respectively. The lowland area further declined to 1990.71 ha in 2008. Thus the water bodies and lowlands decreased by 32.57% and 52.58%, respectively, from 1960 to 2008. This changing trend of wetlands makes the drainage system of Dhaka City vulnerable, creating water logging problems and their consequences. During monsoon rains, many areas of Dhaka go under water because canals, the primary drainage system of the city, are blocked and cannot carry the huge volume of stormwater. The situation has turned very severe in the recent years (Islam, 2010). In the past few decades, both river and urban flooding have seriously impacted Dhaka in terms of floodwater inundation and duration, especially in fringe areas, mainly affecting slum dwellers. Residents suffered from health-related problems as a result of consuming contaminated drinking water. In addition, communications, livelihoods, and service facilities were severely affected for days.

The 1988 flood inundated about 85 percent of Dhaka (water depths were in a range of 0.3–4.5 m), isolating the city from the rest of the country for about two weeks. Water supplies were severely affected; and road, rail, air transport, and telecommunications were disrupted. A decade later, about 56 percent of Dhaka was inundated by the 1998 flood, which was unprecedented in terms of duration and damage. Most of the city‘s eastern portion suffered from river flooding, and 23 percent of its western part was affected by urban flooding (Hasnat 2006). Nearly 80 percent of the city was inundated at water depths of 0.3–3.0 m, which lasted about 10 weeks. Some twothirds of city residents suffered damage to assets, and daily wage earners (e.g., rickshaw pullers and vendors) suffered direct loss of income. In all, some three-fifths of residents were affected (Hasnat 2006). Rising water levels of the Buriganga, Turag, Balu, and Lakhya rivers affected the eastern and southeastern portions of the city (Hossain, Azam, and Serajuddin 2006). In central areas, poor discharge capacity of drainage *khals* (open channels) increased the extent and duration of flooding. Some 66 km of khals and 80 km of stormwater drains were damaged (Huq and Alam 2003). At Narinda, the retention tank overflowed, affecting its only pumping station. The estimated cost of rehabilitating the water supply system was about Tk. 127 million (Huq and Alam 2003).

In terms of river levels, the 2004 flood was of shorter duration than the 1988 and 1998 events; however, water took longer to drain from Dhaka city areas and extreme rainfall in September made the situation worse. The flood wreaked havoc on major portions of the city‘s population. More than 5 million people— two-fifths of city residents—were affected. Out of the city‘s 22 *thanas* (subdistricts), 18 were inundated. The eastern portion of the city was hardest hit. Gulshan, Banani, Baridhara, and Nikunja experienced prolonged inundation, while the Motijheel commercial area, including Arambagh and Gopibagh, was underwater for a few days as a result of drainage congestion. Water pipelines stretching over a few hundred kilometers and many ponds were submerged. The sewerage system broke down, which resulted in contaminated drinking water, in turn, posing a serious threat to public health (World Bank, 2015).

The August 2007 flood exceeded the duration of the 1988 and 2004 events; however, the magnitude of peak flow above the danger mark for all rivers surrounding Greater Dhaka was less than 1988, 1998, and 2004 levels. Flooding of the Buriganga River receded within 24 hours, but lasted more than 20 days for other major rivers and canals (World Bank, 2015). Eastern Dhaka was especially hard hit due to the absence of a flood-protection embankment, with boats serving as the only means of transport. Sections of major roads and bridges were washed away, and numerous drowning deaths were reported. Thousands of stranded residents lacked access to safe drinking water. Hospitals were overwhelmed with patients suffering from water-borne illnesses, including more than 90,000 cases of diarrhea within a single week. Most patients were residents of the more severely inundated, low-lying areas.

In July 2009, heavy monsoon rains battered the capital city. The national weather office reported more than 333 mm of precipitation within a 12-hour period; more than 290 mm fell within six hours, setting a 60-year record. Downed power lines resulted in at least nine deaths and numerous injuries. Rickshaw pullers struggled to carry passengers through knee-to-waist-deep water. Bus and ferry services were disrupted and major thoroughfares were submerged for hours, leaving thousands of stranded residents. In more vulnerable, low-lying areas, many families were trapped in their flooded homes, and slum dwellers suffered the most (World Bank, 2015). Production was suspended at most industrial units in the southeastern area of Dhaka-Narayanganj-Demra (DND). Businesses and schools were forced to close, and trading on the stock market was delayed.

The stormwater flooding situation in Dhaka has deteriorated in the recent years because of poor drainage system. Dhaka‘s average annual rainfall is about 2,000 mm, most of it occurring during the summer monsoon (Zaman, 2014). The rivers surrounding the city are distributaries of the Brahmaputra River. Thus, the water level of the Brahmaputra and rainfall and runoff from the entire Brahmaputra Basin influence the process of river floods in and around the city. The entire drainage system is also influenced by water levels of the Ganges and Meghna rivers. At present, the western part of Dhaka is protected against river floods that formerly resulted from overflow of the Turag, Buriganga, and Balu rivers. However, the western part of the city is now vulnerable to urban flooding resulting from the combined effects of poor drainage of rainfall inside the city‘s western (polder) area, inadequate drainage capacity of pump stations, and high water levels of the outfall rivers. The eastern side of the city suffers from river floods, and plans have been developed to build an embankment along the Balu and Lakhya rivers and develop the eastern polder with a number of drainage pumps.

Following 1987 and 1988 floods and its subsequent damages, development partners felt the urgency to support Bangladesh in managing floods on a coordinated way. Flood Action Plan (FAP) comprising regional studies and pilot projects were taken up. Dhaka being the capital city of Bangladesh got the priority; Dhaka City is divided into 12 drainage zones for stromwater drainage planning in the master plan for flood protection, which was prepared after the severe floods of 1987 and 1988 (JICA,1992). These zones are determined according to topographic characteristics, natural khal system and drainage outlet to the peripheral rivers. The master plan (FAP 8A and FAP 8B) proposed several flood protection and drainage improvement measures such as construction of embankment, improvement of khals, and construction of pumping stations. This master plan for flood protection and drainage improvement for greater Dhaka (JICA, 1991) has been partially implemented, mostly in the western half of the city, leaving integrated stromwater management in Dhaka as a pending task. A portion of the storm runoff generated in western Dhaka passes through the Hatirjheel detention area toward the Balu river through the Begunbari khal (BUET, 2005). Thus, Begunbari canal serves as an important outlet for Dhaka. At present, the gates of the Rampura regulator located at the outlet of Hatirjheel are closed when water level rises to a critical level. FAP 8A submitted its recommendations to place embankments from Tongi to Demra along the rivers at 100 years flood level with four pumps to drain out the internal run-off and household wastewater using retention reservoirs at different locations. Time passed and more studies such as Flood Control and Drainage to Eastern Bypass Area by Halcrow (2006), Drainage Master Plan by DWASA (2006), Detailed Area Plan by RAJUK (2010)

Strom Water Drainage Master Plan (2016) by DWASA, Urban Flooding of Greater Dhaka in a Changing Climate by World Bank (2015), Flood Hydrology Study (2016) by RAJUK were conducted.

The Eastern part of Dhaka is currently undergoing rapid change in land use from rural to urban landscapes inter-laced with wetlands and open channels (khals). According to proposed land use plan of RAJUK's Detailed Area Plan (DAP), most of the area will become urban residential and commercial in nature. DAP has also delineated retention areas following suggestions of Halcrow 2006 (reduced retention area is seen at RDP of Rajuk) (Saman, 2015). However, some of these areas have already been encroached by private land developers, which can be detrimental to the future drainage system of the study area. Thus an urgency is felt for immediate implementation of flood control and drainage improvement in an integrated way. While proper planning and implementation of development projects in East Dhaka would turn it into a vibrant hub of activity, unplanned and rapid urbanization would only worsen the overall condition and expose more people to risk from flooding, earthquakes and other disasters.

According to the World Bank experts, construction of a flood barrier in Eastern Dhaka would have a direct and substantial impact on the economy, even without further simultaneous strategies. However, in this case it would only lead to informal residential and manufacturing development. On the other hand, if a flood barrier is constructed simultaneously with development of roads and bridges to ease connectivity to East Dhaka, then it would not only ease traffic congestion in the city but would also turn the current city centre into a tradable services hub. Bangladesh is wanted to be a middle-income country by 2021 and it mostly depends on Dhaka‘s urbanization. Being largely rural, the development potential of Eastern Dhaka is massive. For this, the East Dhaka needs to be designed and built sustainably. At this stage Government of Bangladesh considered the importance of the Eastern Bypass as of high priority and attempts are made to make best use of existing khals to minimize land acquisition for retention storage.

Despite flooding, stormwater is also important regarding the urban water balance. The expansion of impervious land-cover implies both larger stormwater runoff volumes and peak flows and consequently reduces other components of the hydrologic cycle, e.g. infiltration and evapotranspiration. Moreover, stormwater directly transports harmful substances from urban surfaces into downstream water systems, thus degrading the water quality. (e.g.

Scalenghe and Marsan, 2009; Göbel et al., 2007) The importance of urban stormwater modeling is constantly increasing due to three global trends: urbanization, population growth, and climate change. The first two trends induce a rapid growth of cities, making stormwater management ever more challenging while at the same time a rising number of people is affected by the harmful effects of stormwater on the environment. In Dhaka city, these effects are expected to be amplified in the future due to climate change and associated higher frequencies of extreme weather events. On this background, here stormwater analysis is carried out for one of the important catchments of Eastern Dhaka taking consideration of future landuse and climate change effect. In this study, Begunbari canal catchment (47.67 square km) area of Eastern Dhaka has been modeled in GeoSWMM.

1.2 Study Objectives

Improvement of stormwater drainage management is important for Dhaka City because of the deteriorating urban flooding and drainage congestion situations, and the consequent impacts on the economy, environment, health and quality of life in the city. Stormwater drainage management would be more efficient in a detention-based gravity drainage approach aided by pumping. For the Eastern Dhaka, river flooding is the most critical and proposed Eastern Bypass is likely to be completed by 2025 to solve this external flooding. However, mathematical modeling by GeoSWMM is capable to take into systematic consideration of external river water level, rainfall-runoff generated in the urban catchments and storage and pumping capacities of the detention basin at different rainfall events. This study‘s overall aim is to provide an effective planning approach for minimizing the damage risk of rainfall-induced urban flooding in Begunbari canal catchment for future landuse change and in a changing climate.

Specific objectives of this research are as follows:

1. To set up and calibrate the hydrologic model of the study area using GeoSWMM
2. To simulate the flood depths in the catchment and compare the simulated flood depths with MIKE 11 output
3. To consider climate change scenario and estimate inundation depth caused by precipitation events with various return period and intensity for existing drainage system
4. To analyze and simulate mathematical model considering Eastern Dhaka Bypass to propose future drainage improvements and adaptation measure

1.3 Organization of the Report

First chapter: The first chapter of this report gives background information to provide the context of the study. Scope and specific objectives of the work are also stated here.

Second chapter: The second chapter reviews previous studies available in connection with the present study. This chapter gives a clear view of different types of works done on the study area and it also provides information about the related works using urban drainage models.

Third chapter: This chapter describes about the GeoSWMM model. The conceptual model is presented here with its capabilities and applications. The various objects and their operations are included here.

Fourth chapter: This chapter describes the methodology of the entire work. It includes a brief description about the selection of study area, data source, data collection, data processing and analysis of data along with model setup and calibration.

Fifth chapter: This chapter deals with the result and analysis of the calibrated model. Inundation depths in various scenarios are presented here and further analysis has been done to observe the impact of adaptation measures.

Sixth chapter: In this last chapter, conclusion of the whole study is drawn. The suggestions and recommendations are also incorporated in this chapter.

CHAPTER TWO
LITERATURE REVIEW

2.1 Review of studies on drainage modeling

2.1.1 The effect of spatial resolution on rainfall-runoff modeling

Chen and Tucker (2003) compared different approaches for catchment delineation, e.g. to burn a stream network into a digital elevation model (DEM) and to use the *Watershed* tool in ArcGIS. They concluded that GIS is a powerful tool for catchment delineation if there are comprehensive and accurate spatial data sources available and the details of the sewer system are correctly accounted for. They also add that manual fine-tuning is often necessary after the automated delineation process.

Based on the spatial resolution, GIS-based catchment modeling can be divided into distributed (or high-resolution) and aggregated (or low-resolution) approaches. A distributed model accounts for all minor spatial variations within the study area while an aggregated model excludes and generalizes details of the input data. Highly distributed models are typically used e.g. for modeling event peak-flows, whereas the more aggregated approaches are mainly suitable for studying large-scale processes like climate change as their limitations are less relevant in long-time-scale modeling. The difference between these two is the level at which features within the catchments are aggregated, i.e. the size of the subcatchments modeled. Aggregation results in loss of complexity but, simultaneously, makes data collection easier. Even the most distributed models usually incorporate some aggregated parameters due to the lack of fully-detailed data on spatial variations of parameters. The key question remains, which level of aggregation is still acceptable in order to maintain a certain degree of accuracy in modeling (Jacobson, 2011).

Park et al. (2008) performed SWMM simulations with varying levels of subcatchment aggregation. They concluded that the simulated surface runoff was not affected by the spatial resolution of the model. On the other hand, accumulated pollution loads were reduced with an increasing level of aggregation. The peak flows appeared at slightly different time instants but otherwise the hydrograph was not affected by the model aggregation. Similarly, Ghosh and Hellweger (2011) did SWMM runs for 50 storm events. Their results indicated that the annual runoff is not dependent on the spatial resolution of the model. The effect of spatial resolution on simulated peak flows was altered. While for small storms the peak flows increased with an increasing level of aggregation a decrease was found for large storms.

On the contrary, Smith et al. (2005) found that the aggregation level of the catchment affected the difference between measured and simulated runoff volumes. They recommended minimizing those differences through calibration of model parameters such as the flow width.

For natural catchments, Jang et al. (2007) found out that non-calibrated SWMM models were most accurate if the whole catchment was modeled as one single runoff unit. Those models provided good simulation results without calibration for three undeveloped Korean watersheds with areas ranging from 8.5 km^2 to 55.9 km^2.

There have also been several studies regarding the aggregation level of other rainfall-runoff models similar to SWMM. Thompson and Cleveland (2009) concluded that fully-distributed modeling with HEC-HMS is not feasible in cases where no calibration data is available. The possibilities of scaling the input data of the distributed KINEROS model to match different aggregation levels were studied by Thieken et al. (1999). The outcome was that the flow length could be used as a scaling factor for adapting the same model to catchment delineations of different scales. Zhang et al. (2013) summarized several previous studies and concluded that the flood volume is insensitive to the degree of catchment subdivision. They also noted that only a few studies have concentrated on the effect of catchment subdivision on water balance components like evapotranspiration and infiltration. Their results with a HEC-HMS model showed that (i) overland flow length increases with increasing subcatchment area, (ii) infiltration parameters are independent of the model aggregation level, (iii) the quality of results decreases if the number of subcatchments is too large or too small, and (iv) unlike in previous studies, the aggregation level affects the components of the water balance. Zhang et al. (2013) also conclude that catchment subdivision is useful if detailed data on parameter variations between the subcatchments is available.

Besides the surface runoff processes, also the conveyance system is subject to generalization. The sewer network may be ‗skeletonized' by disregarding minor pipes. The combined sewer modeling protocol of Department of Water Management for the City of Chicago defines, for example, that no conduits with a diameter smaller than 1070 mm are modeled, unless they have substantial hydraulic importance in the sewer system (Cantone and Schmidt, 2013). This results in only one tenth of the conduits being modeled.

2.1.2 Estimating and calibrating SWMM parameters

A SWMM application includes numerous different parameters, of which several vary from subcatchment to subcatchment. These parameters can be classified to measured parameters (e.g. subcatchment area; pipe lengths, pipe shapes, bed slopes, and diameters; manhole type; soil types; land-use types; and rainfall depth) and inferred parameters (e.g. flow width; infiltration parameters; Manning's *n* for pervious and impervious areas; depression storage for pervious and impervious areas; imperviousness; and Manning's *n* for conduits) (Choi and Ball, 2002). The measured parameters are typically easier to obtain, while the inferred parameters usually need tobe calibrated. Nevertheless, also the first mentioned may sometimes involve large uncertainties arising from e.g. inaccuracies in subcatchment delineation.

However, if no runoff measurements required for calibration exist, literature values can be found for several of these parameters. Many parameter values are suggested e.g. in the SWMM User's Manual (Rossman, 2010). In contrast, other parameters such as flow width, hydrological slope, and imperviousness have to be obtained from spatial data.

Detailed spatial data on land cover types etc. is important for calibrating SWMM applications for urban catchments (Jacobson, 2011). Because such data is not always available, there have been attempts to reduce the complexity of the calibration process by concentrating only on the parameters the model is most sensitive to. The problem is, however, that SWMM is sensitive to different parameters in different catchments (e.g. Beling et al., 2011). This highlights the importance of always performing a sensitivity analysis before model calibration. In addition, the method of sensitivity analysis affects the obtained model sensitivity to different parameters (Jacobson, 2011).

Whether calibration could be left undone in some cases is an interesting question. According to Jang et al. (2007), even a non-calibrated SWMM model performs better than an ordinary hydrograph when modeling urbanizing (or urbanized) areas. Amaguchi et al. (2012) developed a vector-based distributed model similar to SWMM which was able to achieve a good reproduction of observed stream flows. However, they recognized the demand of detailed spatial data, and recommended calibration of some parameters (e.g. imperviousness) if only possible.

Overall, it seems there have been experiments with calibrated SWMM models of a low spatial resolution, as well as with non-calibrated high-resolution SWMM models. Again, Henri Tikkanen (2013) did hydrological modeling of a large urban catchment using SWMM. This study proposes a methodology for rapid catchment delineation and stormwater management model (SWMM) parameterization in a large urban area, without calibration. The effect of spatial resolution on the accuracy of modeling results is also being discussed. A catchment delineation and SWMM parameterization is conducted for an urban area in the city of Lahti in southern Finland. GIS methodology is utilized for simultaneous processing of data representing large areas. This study suggests that, literature values are also of importance where no spatial data is available. To evaluate the parameterization results, the SWMM application is run using an hourly data series of meteorological observations covering a period of four years.

2.1.3 Studies with Application of GeoSWMM

Maliha (2017) worked on design of stormwater network with limited data. The area of the study was Bogra city where urbanization leads to increase of impervious surfaces thereby increasing runoff and provision for adequately planned drainage management system has become mandatory. Several types of data are required to design a drainage system but in Bogra city, all the data are not available. This study focused on designing an adequate drainage system where there is scarcity of data. In this study, peak runoff and the maximum flows in the design canals are estimated using the model GeoSWMM. The total runoff volume obtained from the model simulation for high intensity of rainfall is used to design the minimum cross section of the drainage canal. Around 9 canals are included here and required area was calculated for 5 year and 10 year return period. The maximum calculated cross-sectional area of canal section was found as 89.33 m^2.

Ablullah (2017) worked with GeoSWMM in the Tangail Sadar Upazilla, where there is a lack of planned and adequate drainage network system. The main objectives were to evaluate present drainage circumstances, recognize the future necessities, design of cross sectional areas of conduits. He found the runoff coefficients of delineated subcatchments ranges from 0.5 to 0.6. The requirement of minimum drainage area was calculated for 5 and 10 year return period considering 0.8 m/s velocity. The maximum calculated cross-sectional area of canal section was found as 372.38 m^2.

Mohammad (2017) applied GeoSWMM in the Eastern part of Chattogram City, where rapid urbanization is going on. The analysis of stormwater runoff results that found the runoff coefficients of delineated subcatchments ranges from 0.6 to 0.7. The natural canal network was included in the model and their adequate requirement was calculated. It was calculated from the model output of peak runoff considering 0.8 m/s velocity.

Islam (2017) designed drainage conduits for the Western part of Chattogram City. He found the runoff coefficients of delineated subcatchments ranges from 0.6 to 0.7. The requirement of minimum drainage area was calculated for 5 and 10 year return period considering 0.8 m/s velocity. The maximum calculated cross-sectional area of canal section was found as 9.98 m^2.

All the mentioned studies above are done with limited data since there is requirement of cross-sectional data, observed boundary data and also observed data for proper calibration. Hence the scopes of these studies were limited to find the required drainage area for peak runoff. However, the present study focuses on calibration and validation of the model with observed data and then the model will be applied for simulating different scenarios.

2.2 Previous Studies on Drainage and Flooding Problem of Dhaka City

Murshed et al. (2012) studied on the adaptation strategies of drainage System of Dhaka City. Observed rainfall data from 1953 to 2010 of Dhaka station has been analyzed. Increasing trend of three important indices for urban drainage, monthly maximum consecutive 5 day precipitation (0.277mm per year) , 99th percentile rainfall (0.4mm per year), number of heavy precipitation days when precipitation ≥20 mm (0.063 days per year) and decreasing trend of consecutive wet days (0.04 mm per year) indicate high intensity rainfall is occurring more frequently within a short period. This leads to overflow of urban drainage. Last ten years data with 341mm rainfall occurred in 8 hours (September 2004), 333 mm rainfall of which 290 mm occurred in six hours (28th July, 2009) proves the erratic tendency of recent rainfall events. The design rainfall of present drainage system of Dhaka city is only 245 mm of 2 day cumulative rainfalls and a 5 year return period. This study recommends some adaptation strategies, giving priority on institutional reform, infrastructural design, rehabilitation and extension of drainage system for Dhaka city in response to climate change.

Khan et al. (2015) studied the application of Urban Growth Model (UDM) to determine the future condition of Dhaka City. In 2004 an extreme flood event occurred in the country, which caused major damage to Dhaka City. If the same event occurs in 2050, it can be expected that the damage would increase significantly. Through application of urban growth model, urban flood model and damage model the damage that can be expected to happen in 2050 was determined here. For 3 types (Business As usual , Business As Usual Constrained, Planned) of scenarios, different growth rates are applied: LOW, MEDIUM and HIGH. While the BAU scenario is related the medium scenario, the low and high scenarios are translations of the population growth scenarios into actual rates of land cover changes. This means that a total of 3x3=9 different scenarios were developed. The outputs from the UGM represent land cover instead of land use. In the approach presented here, residential and non-residential property damage have merged as building content damage, assuming that the depth-damage curves (DDCs) are available for both types of properties. The flood information can be obtained from hydraulic modelling. The outputs from the urban growth model for different scenarios were applied to urban flood model developed for Dhaka City under WP2 of CORFU project. The flood models were able to provide the details of the flood impact due to changes in urban conditions. These outputs from the flood models were then used to determine the damage due to flooding. The change in flood depth is not significant. This is because the flooding scenario is controlled by the downstream Balu River water level for Eastern Dhaka. The damage for the 2004 event using UGM and building data were 6.07M and 4.7M USD, respectively for the Dhaka Case Study area. 29% overestimation was found using the UGM data. If the 2004 and 2050 events occur in 2050 UG BAU high scenario, the damage are 28.2M and 68.4M USD, respectively. It was mentioned that the derived Depth-Damage-Curve (DDC) for the UGM scenario may need further refinement to produce better results.

Yahya et al. (2010) looked into the vulnerability of flooding in Dhaka city by analysing the existing flood protection infrastructures and recommend a number of steps to reduce the risk to flooding. Heavy rainfall, waste water from households and industries and septic waste accumulated in wetlands and canals causes urban flooding particularly in low laying areas in the western part of Dhaka city. Natural open drainage system is continuously encroached upon or sometimes destroyed for constructionbuildings; roads thereby violating the law. This study recommended for law enforcement, natural drainage system, storm water pump station near Rampura bridge an urban management system , deepening of Begunbarikhal, Gulshan lake, dredging in the Buriganga, Turag, Balu Rivers and overall coordination in an integrated manner. A mathematical model has to be developed to assimilatethe present and future phenomenon of rainfall-run-off and ground water table fluctuations in order to properly plan in respect of water related issues.

Taleb (2012) did a comparative study of urban area extension and flood Risk in Dhaka City. This study suggested comprehensive measures including structural and non-structural measures that will be included a number of flood control and mitigation programs as well as alternative valuable resources for urban planners and decision makers to devise sustainable land use and environmental planning. This study find out that, recent unplanned urban development in Dhaka city of Bangladesh have directly impact on drainage system of the city. The low-lying areas of the City of Dhaka, especially its eastern and western buildup fringes are greatly vulnerable to seasonal inundations. The study further reveals that the spree of recent urban growth and the associated structural development have significantly been affecting the urban-inundations and drainage failure in the City. Some appropriate integrated environment-friendly adjustment-oriented measures may bring some success in this respect. At the same time special attention need to be given to the facts that the normal natural settings i.e. morphological and hydrological situation of the City should not be disturbed in the process of urban-development. With this respect the endangered and extinct khals (channels) and, rivers and other existent important wetlands of the city should be preserved and saved for the sake of the smooth functioning of the City's choked and disturbed drainage system.

Huq and Alam (2003) discussed flood management and vulnerability of Dhaka City. After implementation of the flood-control project in Dhaka West, unplanned and uncontrolled expansion of urban areas stretched rapidly toward the low-lying areas and floodplains adjacent to the flood-protection embankment and river. Because of the rapid population increase and scarcity of land in Dhaka West, unplanned expansion is also taking place in Dhaka East at the same pace. Evaluation of Phase I work revealed that the existing earthen embankment is unstable and cracking in large sections. Construction of embankments through low-lying areas without providing adequate drainage facilities has caused internal flooding, adversely affecting the residents in those areas. While designing and implementing the plan, environmental impacts should be kept in mind and an adequate number of drainage facilities should be provided for the proposed embankment. The hydrological data should be analyzed during the design phase of the flood protection infrastructure. Dredging work in the Buriganga, Turag, and Balu Rivers should be regularly carried out to maintain navigability and reduce drainage congestion. In addition to structural measures, non structural measures should also be considered to reduce flood damage.

2.3 Previous Dhaka Drainage Plans

2.3.1 JICA Studies (1987 and 1990)

The ―Study on Stormwater Drainage System Improvement Project in Dhaka City" (JICA, 1987) was one of the first major studies focusing on the drainage problems of the city. After the 1988 flood, the study was reviewed and updated in 1990. The aim of both studies was to prepare a phased drainage improvement program and identify priority projects with preliminary designs. The study area covered the main city area (about 138 sqkm) which was then adjusted to 135sqkm in the 1990 study. The study area consisted of 10 drainage zones. 109km of pipe drains, 53km of khals, one pump station with 9.6 m3/s capacity (at Narinda, Old Dhaka) and several culverts and bridges. The recommended urgent works included increasing pumping capacity to 10 m3/s, improvements to the sluice gate at the pump station, khal improvements with sodding (vegetation) protection and laying of box culverts in urbanized areas. The study also recommended initial dredging of khals and cleaning of drainage pipes followed by annual cleaning of the drainage network. The study also recommended land use restrictions, such as maintaining area of 208 ha (approx 2 sqkm) for Kallyanpur retention pond, preservation of khal areas, etc. The 1990 updated study also highlighted the need for timely land acquisitions, strengthening of DWASA Drainage Division and financing of urgent projects through foreign aid. As climate change was not a major issue at the time, the impact of climate change on flooding was not considered in this study.

2.3.2 FAP 8A (1990 to 1992)

Flood Action Plan (FAP) 8A was part of a larger 26-component study initiated by the GoB after the devastating 1988 floods. FAP 8A (Greater Dhaka Protection Project - Study in Dhaka Metropolitan Area) was conducted by JICA. FAP 8A consisted of three phases from October 1990 to April 1992. Main activities carried out were:

- A master plan study on comprehensive flood control and storm water drainage in the Dhaka metropolitan area.
- Identification of priority works for feasibility study.

The study area was 850 sqkm, which included Dhaka, Tongi, Savar, Keraniganj and Narayanganj. FAP 8A not only incorporated the recommendations from the previous JICA studies but also highlighted the need for flood forecasting (early warning system), flood evacuation system, land regulation and zoning of drainage areas. FAP 8A also undertook frequency analyses of rainfall and river water levels in and around Dhaka. These analyses formed the basis of the basic design criteria for the drainage system:

- Gravity drainage outfall water level would be the 2-yr frequency river water level
- Pump and regulating pond capacity would be based on 2-day consecutive rainfall with 5-yr return period

Main Phase 1 (priority) activities included construction of the western embankment along Turag River (from Tongi Railway Bridge to Sirnirtek); raising road and flood wall along left bank of Buriganga River (above 1988 flood level); and flood protection of the airport. These works were completed by 1990. Phase 2 activities included construction of an embankment along right bank of Balu River (from Demra to Tongi Railway Bridge); construction of 5 pump stations; and restoration of 12 khals. As yet, only two of the pumps stations have been completed.

The study also included hydrodynamic modeling of the 1988 flood with different flood control options. However, FAP 8A did not investigate the impact of climate change on flooding.

2.3.3 FAP 8B

The ‗Dhaka City Integrated Flood Protection Project' (FAP 8B) was conducted by Louis Berger International (1991) and financed by the ADB. This study focused on protection of western part of Dhaka (approx. 136 sqkm) which is the most built up area. The study had four parts: A – Flood Protection (BWDB), B – Drainage Improvement (DWASA), C – Environmental Improvement (DCC) and D – Implementation Assistance. The Flood Protection part mainly involved strengthening of existing embankments and flood walls; construction of sluice gates under existing embankments; raising and flood proofing of the main north-south road connecting Tongi Railway Bridge and the Friendship Bridge over the Buriganga River; provision of temporary pumping facilities and construction of a pump station at Goranchatbari in the north-west part of the city. Part B mainly involved rehabilitation of khals and construction of pipe drains. Part C included solid waste management and sanitation improvement activities. FAP 8B also highlighted the lack of proper land use management and development controls. The study recommended the formation of a development committee with representatives from RAJUK, DWAS, DCC, DoE and other utilities/agencies. As with the previous studies, potential impacts of climate change were not considered.

2.3.4 IWM Study

DWASA engaged IWM to undertake the ―Study on Drainage Master Plan for Dhaka City" (IWM 2006). The aim of the study was to evaluate previous drainage studies in light of changed city situation and provide recommendations to improve the drainage system. The study area was about 316sqkm, covering western and eastern Dhaka (259sqkm) and the DND area (57sqkm). The study undertook frequency analyses of rainfall and river level data from various stations in and around Dhaka. The analyses found that there were no significant changes in return period rainfalls for the periods up to 1990 and up to 1999. Similarly, return period of river water levels were similar for the periods up to 1990 and up to 2000. As a result, the IWM study adopted the design rainfall and design flood water levels identified in the FAP 8A study. The IWM study also adopted the drainage criteria specified in the FAP 8A. Although the IWM study looked at possible changes in rainfall and river water level frequencies up to 2009; it did not consider climate change impacts in the future.

The study found that the drainage system in western Dhaka continued to be inadequate despite the previous studies and efforts to improve the situation. Particular problems included combined sewer and stormwater drainage; disposal of wastes in the drainage system; right of way of drainage khals not secured; and lack of proper maintenance. Some key recommendations for the western part included: two new permanent pump stations at Maniknagar and Rampura (which currently has temporary pumps); removal of all encroachments of khals; and proper cleaning of DCC and DWASA drainage network. For eastern Dhaka, the recommendations included: proper implementation of development controls by RAJUK; should the eastern embankment (along right bank of Balu River) be constructed then pump stations at outfalls of Boalia Khal, Shahjadpur Khal and Norai Khal would be required; immediate measures to preserve land for retention ponds; and protection of public lands and khals from land grabbers.

For the DND area, the study recommended increasing of pump capacity at Sharulia; new pump stations at Adamjee and Pagla; protection of khals from encroachment; and re-sectioning of some khals to increase conveyance capacity.

From an institutional perspective, the IWM study recommended that a ―central high-powered coordination committee" engaging all concerned agencies working with utility services of the city. Furthermore, it was recommended that all flood and drainage infrastructure to be handed over to DWASA and that the Drainage Circle of DWASA be re-organized and strengthened.

2.3.5 DAP

The Detailed Area Plan (DAP) was undertaken by RAJUK in order to implement the structure plan and urban area plans of the 1995 Dhaka Metroplitan Development Plan (RAJUK 2010a). The purpose of DAP was to:

- Provide basic infrastructure and services in the study are through systematic planning.
- Create congenial environment to promote economic activities.
- Improve drainage system of the area and protect flood flow from encroachment.
- Create service centers to enable urban growth.

The DAP area is the same as the DMDP area (1528sqkm) and was divided into 5 groups. Each group was to a different consulting firm to develop the detailed area plan. The consultants undertook extensive survey and data collection work. Based on this information a detailed assessment of the area around 2004-6 was provided in the DAP reports. This included analyses of the drainage system in different areas. Some of the consultants provided details of the drainage system (in terms of length and type of drains, culverts, khals, etc.).The DAP reports highlighted the poor condition of the drainage system in urban areas and urban fringe areas. The main reasons identified include: lack of connection between drains to outfall channels (khals) due to wastes and siltation; illegal encroachment of khals; and filling up of lowlands (floodplains). The DAP study also highlighted the need to protect ponds, lakes and wetlands as these serve multiple beneficial purposes, including stormwater retention.

The DAP studies provide proposed land use zones for the whole study area. This includes areas designated as water bodies, retention ponds and flood flow zones. Approximately, 23% of the DAP (DMDP) area is designated as flood flow areas. Other recommendations include: reclamation of public lands; establishing of right of way (ROW); resection, re-excavation and widening of khals; and flood protection measures identified in FAP 8A and 8B. It is expected that the proper implementation of DAP recommendations will improve the drainage system and thus water logging conditions of the Greater Dhaka area. However, the DAP studies did not consider potential climate change impacts in the future.

2.3.6 Preparation/Updating of Stormwater Drainage Master Plan for Dhaka City

This is an ongoing World Bank funded DWASA project undertaken by Jurutera Perunding Zaaba Sdn. Bhd (of Malaysia), Farhat Consulting Engineers & Architects Ltd. (Bangladesh and SARM Associates Limited (SARM, Bangladesh). The Master Plan area covers the DAP area and the planning horizon of the project is to 2040. The consultants will undertake detailed surveys of the drainage system, land levels and other relevant data. Based on this information hydrologic and hydrodynamic models will be developed to represent the current drainage system. After that, the models will be used to analyse future development scenarios and effectiveness of different structure and non-structural measures. The consultants are also expected to identify priority projects for drainage improvements; make recommendations for land acquisition; develop a design manual with relevant design standards for drainage infrastructures; develop strategies for solid waste and water quality management; and recommend necessary legislative and regulatory reforms required for the successful implementation of the Stormwater Drainage Master Plan. However, the consultants do not plan to look specifically into the potential impacts of climate change on the drainage system.

2.3.7 BUET Study

The study area for this work includes all the catchments from which direct runoff may eventually drain out to the proposed stormwater channels located on both sides of Kuril-Purbachal Link Road. The study (2015) of DWRE, BUET is particularly interesting here as it drains the stormwater from an Eastern Dhaka Catchment.

The Purbachal Township will be connected to Kuril with a 300-ft wide 8 lane link road. The distance of the Link Road from Kuril to Balu River is approximately 5.8 km. On both sides of the 300-ft wide Kuril-Purbachal Link Road from Kuril to Balu River, there are provisions for 100-ft wide canals to be used as stormwater channels under Detailed Area Plan (DAP). This study includes detailed hydrological and hydraulic study to assess direct runoff from the surrounding catchments.

This study further identifies the segments on both sides of Kuril-Purbachal Link Road where stormwater channels are necessary for two conditions (i.e. natural and alternate conditions). In natural condition, the existing AD8 Khal-Boalia Khal and Kathaldia Khal systems located to the North and South of Kuril-Purbachal Link Road respectively mostly carry the stromwater from the catchments within the study area. Only a channel section, for relatively smaller direct runoff from Nikunja-1, East and West Joar Shahara and a subcatchment, is required on the North side of Link Road from Kuril to Boalia Khal. For alternate condition, as the AD8 Khal is diverted to the North of Kuril-Purbachal Link Road through a bypass channel; a channel section is also required on the North side of Link Road from Kuril to Boalia Khal but to carry a relatively larger direct runoff specifically adjacent to Police Officers Housing Society (POHS). The bypass channel has been approved by the Ministerial Committee to review DAP. Bypass channels are not uncommon in planned township development and often used to optimize land use and can be a valid option for POHS. In addition, design parameters are evaluated for various channel sections (i.e. Rectangular RCC Channel, Sand-Cement Lined Trapezoidal Section and Earthen Trapezoidal Section) for the proposed stormwater channels on the North side of Link Road from Kuril to Boalia Khal.

2.3.8 Urban Flooding of Greater Dhaka in a Changing Climate by World Bank

The comprehensive study of World Bank (2015) covered 345 km^2 including Dhaka metropolitan area, DND area and Narayanganj township. This study covered 121 km^2 area of Eastern Dhaka. This study recommends a prudent strategy that begins by addressing the current adaption deficit that Dhaka‘s already face in coping with rainfall extremes. Closing the current adaption gap is an attractive, no-regrets option that can provide policymakers a solid foundation on which to prioritize and sequence additional measures to reduce potential future damage from climate risk. The following table shows a significant decrease in flooded areas for all inundation depths in 2050, including more than a threefold increase in flood-area; however, the time required for flooding to recede decreased only slightly compared to the 2004 event.

Table 2.1: Comparison of Flooding Results, Eastern Dhaka Model (WB, 2015)

Scenario	Maximum Flooded Area (km^2)					Average duration depth above 0.25m depth (hours to recede)
	Flood free (0-0.25 m)	0.1-0.25 m	0.25-0.75 m	0.75-1.5 m	>1.5 m	
2004 (actual storm, 341 mm/ 24 hour)	34.40	2.87	7.89	10.81	64.73	>240
2050 100-year event (SE+PI, 341 mm/24 hour)	104.8	1.1	2.2	4.0	8.6	234

SE: Socioeconomic Changes, PI: Planned Improvements

2.3.9 Technical Study of Flood Control and Drainage Development at Dhaka Circular Road Project

The study (IWM, 2016) has the objective to explore minimum land acquisition for all infrastructures especially for retention pond necessary for optimizing the pumping capacity. Hydrometric, hydrologic and topographic data were collected and a mathematical hydraulic model study was conducted to explore options for minimum land requirement. The model was calibrated with the observed data and option studies were carried out for drainage study for keeping the flooding level in the project area below 4.5m PWD under design storm of 2day 1:5 year return period draining within 2 days. The study has special interest in connection to this current work as it considers Easter Dhaka along with the Bypass. However, this study suggests 51 cumec capacity pumps at Begunbari outlet.

2.4 Review of Climate Studies

Estimates of changes in monsoon precipitation and for extreme events (Climate Change in 2025 and 2050 for A1FI and A1B emission scenarios) have been provided by Dr. Masahiro Sugiyama (2012). A method known as pattern scaling was used to convert low resolution (at least 100km x 100km) GCM outputs to regional or city scale (at most 50km x 50km). This method is a form of statistical downscaling, which combines observed (historical) data with simulated data to produce estimates of future changes in climate. In this way, biases in precipitation estimated from climate models are minimized. First, the change in local temperature in relation to the mean global temperature was estimated. Then, the change in local rainfall is estimated based on the relationship between local rainfall change and local temperature change. This change is then converted to a per cent change by taking into account of the local rainfall at the present time.

Table 2.2: Summary of Climate Change Estimates

Parameter Description	Symbol	2025	2050
Mean global temperature change (K)	ΔT_{global} (A1FI)	0.85	2.0
	ΔT_{global} (B1)	0.70	1.3
Ratio of local temp. change to global temp. change (unitless)	$\Delta T_{local} / \Delta T_{global}$	0.90	0.92
Local temp. change in for two extreme emission scenarios (K)	ΔT_{local} (A1FI)	0.79	1.8
	ΔT_{local} (B1)	0.65	1.2
Change in mean local rainfall per mean global temp. change (%/K)	$\Delta P_{mean} / P_{mean}^{present} / \Delta T_{global}$	3.65	4.9
Change in mean monsoon rainfall for two extreme climate scenarios (%)	$\Delta P_{mean} / P_{mean}^{present}$ **(A1FI) (June-July-August)**	**4.2**	**9.9**
	$\Delta P_{mean} / P_{mean}^{present}$ **(B1) (June-July-August)**	**3.5**	**6.4**
Change in extreme local rainfall per mean global temp. change (%/K)	$\Delta P_{extreme} / P_{extreme}^{present} / \Delta T_{local}$	8.5	9
Change in extreme monsoon rainfall for two extreme climate scenarios (%)	$\Delta P_{extreme} / P_{extreme}^{present}$ **(A1FI)**	**7.00**	**16.00**
	$\Delta P_{extreme} / P_{extreme}^{present}$ **(B1)**	**5.75**	**11.00**

Source: Sugiyama (2012)

As shown in table 2.2, the estimate change in extreme rainfall by 2025 is 5.75% to 7.0%, irrespective of return period. The increase is more marked by 2050, ranging from 11.0 to 16.0%. The estimated changes in mean rainfall in the monsoon season (June, July and August) were used for regional and basin level modeling.

2.5 Other Studies

IWFM undertook a study of the 2004 flood which heavily affected the Greater Dhaka area: ―Investigation of Hydrological Aspects of 2004 Flood - with Special Emphasis on Dhaka City" (IWFM 2005). The study focused on identifying the main features of the flooding in terms of extent (area) affected, timing and duration of inundation, etc. Some recommendations in line with the main studies discussed above were also reiterated.

A modeling study for the Eastern Dhaka area was undertaken in the following project: ―Updating/Upgrading the Feasibility Study of Dhaka Integrated Flood Control Embankment cum Eastern Bypass Road Multipurpose Project" (Halcrow 2006). The scenarios modeled included embankments on both sides of Balu River and also just on the right bank. Different pump options were investigated. The study recommended three pump stations with associated retention ponds along the proposed Eastern Embankment. Some potential impacts on RAJUK‘s Purbachal project were identified in this study.

IWFM also undertook a similar study for the 2007 flood: ―Hydrological Aspects of 2007 Flood" (IWFM 2008). Again, the focus of the study was an ex-post analysis of the key features of the flood. The study also looked into the performance of the drainage system and provided some recommendations, such as fixing of sluice gates and installation of permanent pumps at Rampura.

It can be observed that various studies have been done for Dhaka drainage but none of them highlighted on any single catchment. This study has been carried out for the catchment under Begunbari canal outlet of Eastern Dhaka. Here, the modelling has been done by GeoSWMM which is built upon open source SWMM model. This is the largest catchment of Eastern Dhaka as Begunbari canal is the largest drainage channel that drains water into the Balu river. The study area is in the Dhaka city but mostly rural, so there is potential of large scale development and urbanization and consequent landuse change. So, there are many interesting and challenging aspects to highlight on this selected catchment. This current study is going to focus on this catchment to try different adaptation measures considering the future landuse and climate change impact.

CHAPTER THREE
MODEL DESCIPTION

3.1 General

Model is basically a representation of reality. Modeling of phenomenon and real happening is always a ‗miniature‘ or the functions of a group of mathematical equations. These equations are not sufficient to represent the total complexity of real world rather than the simplification (Karssenberg, 2002). Though models give more generalized view of basin hydraulics of the real world, (a) these are more advanced, capable, functional scope, multi activities, efficient models are coming continuously and are improving their work status as per requirement; (b) these can handle huge amount of data sets for large drainage networks with a limited time and cost; (c) easy user interface and most of the recent models are with GIS platform. Visualization of input and output with different dimension is possible here and the input error can be corrected and updated continuously. The effective modeling depends upon experience; selection of appropriate model.

3.2 Choice of Model

The available tools are described here.

MOUSE

The MOUSE program developed by Danish Institute of Water and Environment (DHI) is an advanced and comprehensive software package, consisting the sub models of Rainfall-Runoff, Hydrodynamic (HD), Water Quality process (WQ) and Sedimentation Transportation (ST) etc. It provides powerful facilities to solve water quantity and quality problems in urban catchments by running one model, or combing several sub models.

SOBEK-Urban

The Water Flow module of SOBEK-Urban allows to analyse the hydrodynamic behaviour of a sewer network under different hydrological conditions and different management operation strategies. An example of an application is the analysis of the management of a sewer network discharging the inflow of runoff under high rainfall conditions. Such an application focuses on preventing high water levels at specific locations by taking measures such as creating additional storage, increasing the discharge capacity of the network, or adaptation of the operation rules.

StormCAD

StormCAD provides comprehensive modeling for the design and analysis of storm sewer systems. StormCAD is FEMA-approved and provides calculations for catchment runoff, gutters, inlets, junctions, pipe networks, and outfalls, and its intuitive interface makes the design and analysis of storm sewer systems easier than ever.

Flood Simulation by Two Dimensional Tank Modelling (Japan)

Kinoshita, Sato and Terayama (1996) presented a model, called Flood Simulation by Two-Dimensional Tank Model, which was composed of runoff, conduit, network, pump station and ponding model. Among them, the Runoff Model was used to produce inflow hydrograph to the sewer systems; the Conduit Model was constructed by the same principle as the Mouse pipe model; the Network Model was used to simulate confluence and divergence of the flow. The flow diversion at a diversion manhole was calculated by using the overflow weir formula and orifice formula; the Ponding model was a fictitious tank used to store excess water located on the top of the overflow manhole

Dynamic Water logging Modelling (DHI, Dhaka)

Dhaka, the capital of Bangladesh, suffers from a severe waterlogging problem recurring several times each year due to even moderate rainstorms, because of the unplanned or poorly planned urban development along with manmade intervention in the natural drainage systems. A pilot study was carried out in order to understand the flooding problem (Kamal and Rabbi, 1998), where, the MOUSE program was used in combination with GIS. In addition to the rainfall runoff and pipe flow models, a surface flow model was developed considering the interaction of underground pipes and street drainage networks, so called Dynamic Water logging modeling. GIS technique was applied for producing flood maps and visualizing simulation results.

Urban Flood Dynamic Simulation Modelling (UFDSM, China)

China is a country frequently suffering from flood disasters. Particularly in last twenty years, as a consequence of rapid urbanization, urban flooding has become a severe problem. In order to study the risk of urban flooding and implement effective measures for flooding control, a project sponsored by the Chinese National Natural Science Fund was implemented during 1995-1997 (Liu, 1998). A flood model, called An Urban Flood Dynamic Simulation Model (UFDSM) was developed by this project team (Hu et al., 1999), where five submodules: infiltration, rainfall-runoff, pipe flow, surface flow and linkage model were integrated into the modeling package. That modelling can simulate urban flooding in sewers and on the surface.

SWMM

SWMM was developed by a consortium of American engineers for the US Environmental Protection Agency (EPA). It is used in the United State for the design of storm water drainage systems and has been incorporated in regional water quality management planning. In addition, it has been applied globally for storm water planning, design and rehabilitation purposes.

GeoSWMM

The GeoSWMM model, a geospatial stormwater and wastewater management model, has been developed as an add-in to ESRI's ArcGIS® 10.x and above. It allows users to simulate flow, velocity of water, depth of water and water quality for projects ranging from a simple site development to a complex urban watershed involving open channels, sewers, pumps, hydraulic structures and Best Management Practices (BMPs). GeoSWMM is built upon EPA's SWMM 5 model which requires users to manually enter data through its interface.

This study has been done using the GeoSWMM. The main reason for using this dynamic rainfall-runoff simulation modeling tool is to account for the influences of the watershed properties over the watershed to simulate runoff, along with its capability to route runoff and external inflows through the drainage system network. The runoff component of GeoSWMM operates on a collection of sub catchment areas that receive precipitation and generates runoff and pollutant loads. The routing portion of GeoSWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. This tool tracks the quantity and quality of runoff generated within each sub catchment and the flow rate, flow depth, and quality of water in each pipe and channel at each simulation time step.

By allowing users to prepare model input and run the model within ArcGIS®, GeoSWMM provides users access to all ArcGIS® functions and tools to create and edit data. Users can take advantage of GeoSWMM and any other tools in the Geographic Information System (GIS) environment for planning, analysis and design of stormwater runoff, storm sewer, combined sewers, sanitary sewers, and other drainage systems in urban areas, with many applications in non-urban settings as well. It also provides an integrated environment for editing study area input data, running hydrologic, hydraulic and water quality simulations and viewing the results in a variety of formats. These include color-coded drainage area and conveyance system maps, time series graphs and tables and profile plots. All these features and functions can be accessed within the GIS environment.

GeoSWMM accounts for various hydrologic processes that produce runoff from urban areas. These include:

- Time-varying rainfall
- Evaporation of standing surface water
- Snow accumulation and melting
- Rainfall interception from depression storage
- Infiltration of rainfall into unsaturated soil layers
- Percolation of infiltrated water into groundwater layers
- Interflow between groundwater and the drainage system
- Nonlinear reservoir routing of overland flow
- Capture and retention of rainfall/runoff with various types of low impact development (LID) practices

GeoSWMM also contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units and diversion structures. These abilities include:

- Handling networks of unlimited size
- Using a wide variety of standard closed and open conduit shapes as well as natural channels
- Modeling special elements such as storage/treatment units, flow dividers, pumps, weirs, and orifices
- Applying external flows and water quality inputs from surface runoff, groundwater interflow, rainfall-dependent infiltration/inflow, dry weather sanitary flow, and user-defined inflows
- Utilizing either kinematic wave or full dynamic wave flow routing methods
- Modeling various flow regimes, such as backwater; surcharging, reverse flow, and surface ponding
- Applying user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weirs crest levels.

In addition to modeling the generation and transport of runoff flows, GeoSWMM can also estimate the production of pollutant loads associated with this runoff. The following processes can be modeled for any number of user-defined water quality constituents:

- Dry-weather pollutant buildup over different land uses
- Pollutants wash-off from specific land uses during storm events
- Direct contribution of rainfall deposition
- Reduction in dry-weather buildup due to street cleaning
- Reduction in wash off load due to BMPs
- Entry of dry-weather sanitary flows and user-specified external inflows at any point in the drainage system
- Routing of water quality constituents through the drainage system
- Reduction in constituent concentration through treatment in storage units or by natural processes in pipes and channels.

Applications of GeoSWMM

GeoSWMM can be used in thousands of sewer and stormwater models within the GIS environment. Typical applications include:

- Designing and sizing of drainage system components for flood control
- Sizing of detention facilities and their appurtenances for flood control and water quality protection
- Flood plain mapping of natural channel systems
- Designing control strategies for minimizing combined sewer overflows
- Evaluating the impact of inflow and infiltration on sanitary sewer overflows
- Generating non-point source pollutant loadings for waste load allocation studies
- Evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings

3.3 GeoSWMM Conceptual Model

3.3.1 Introduction

GeoSWMM conceptualizes a drainage system as a series of water and material flows between several major environmental compartments. These compartments also contain GeoSWMM objects. The Atmosphere compartment, from which precipitation falls and pollutants are deposited onto the land surface compartment. GeoSWMM uses Rain Gage objects to represent rainfall inputs to the system. The Land Surface Compartment, which is represented through one or more Subcatchment objects. It receives precipitation from the Atmospheric compartment in the form of rain or snow; it sends outflow in the form of infiltration to the Groundwater compartment and also as surface runoff and pollutant loadings to the Transport compartment. The Groundwater compartment receives infiltration from the Land Surface compartment and transfers a portion of this inflow to the Transport compartment. This compartment is modeled using Aquifer objects. The Transport compartment contains a network of conveyance elements (channels, pipes, pumps, and regulators) and storage/treatment units that transport water to outfalls or to treatment facilities. Inflows to this compartment can come from surface runoff, groundwater interflow, sanitary dry weather flow, or from user-defined hydrographs. The components of the Transport compartment are modeled with Node and Link objects.

Not all compartments need appear in a particular GeoSWMM model. For example, one could model just the transport compartment, using pre-defined hydrographs as inputs. All the units of the equations here are in SI unit system if otherwise not specified.

3.3.2 Visual Objects

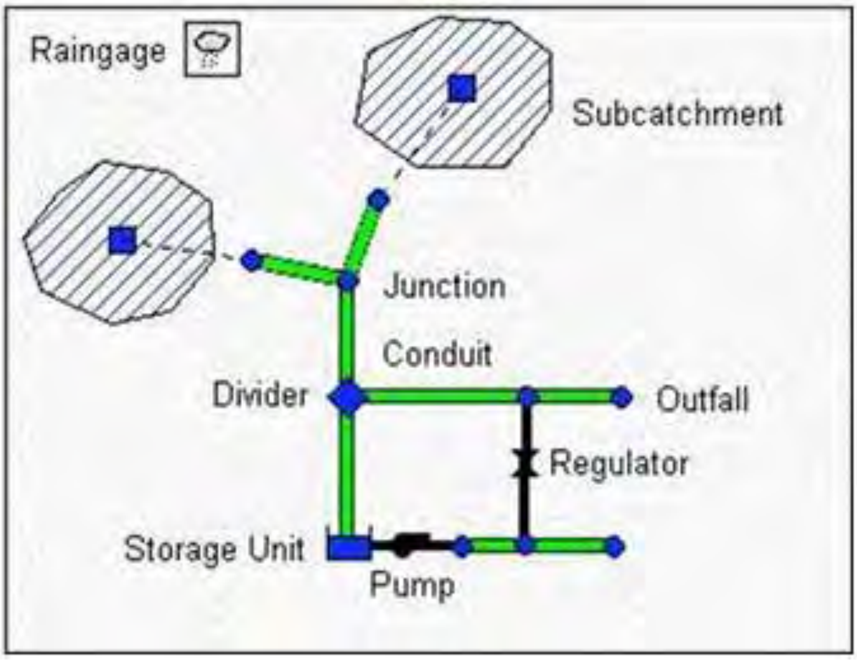


Figure 3.1: Example of Physical Objects used to model a Drainage System (Source: User Manual for EPA-SWMM 5.0)

Fig. 3.1 depicts how a collection of GeoSWMM‘s visual objects might be arranged together to represent a stormwater drainage system. These objects can be displayed on a map in the GeoSWMM workspace. The following sections describe each of these objects.

Rain Gages

Rain Gages supply precipitation data for one or more subcatchment areas in a study region. The rainfall data can be either a user-defined time series or come from an external file. Several different popular rainfall file formats currently in use are supported, as well as a standard user-defined format.

The principal input properties of rain gages include:

1. Rainfall data type (e.g., intensity, volume, or cumulative volume)

2. Recording time interval (e.g., hourly, 15-minute, etc.)

3. Source of rainfall data (input time series or external file)

4. Name of rainfall data source

Subcatchments

Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point. The user is responsible for dividing a study area into an appropriate number of subcatchments, and for identifying the outlet point of each subcatchment. Discharge outlet points can be either nodes of the drainage system or other subcatchments.

Subcatchments can be divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of thepervious subarea, but not through theimpervious subarea. Impervious areas are themselves divided into two subareas - one that contains depression storage and another that does not. Runoff flow from one subarea in a subcatchment can be routed to the other subarea, or both subareas can drain to the subcatchment outlet.

Infiltration of rainfall from the pervious area of a subcatchment into the unsaturated upper soil zone can be described using three different models:

- Horton infiltration
- Green-Ampt infiltration
- Curve Number infiltration

To model the accumulation, re-distribution, and melting of precipitation that falls as snow on a subcatchment, it must be assigned a Snow Pack object. To model groundwater flow between an aquifer underneath the subcatchment and a node of the drainage system, the subcatchment must be assigned a set of Groundwater parameters. Pollutant buildup and washoff from subcatchments are associated with the Land Uses assigned to the subcatchment. Capture and retention of rainfall/runoff using different types of low impact development practices (such as bio-retention cells, infiltration trenches, porous pavement, vegetative swales, and rain barrels) can be modeled by assigning a set of predesigned LID controls to the subcatchment.

The other principal input parameters for subcatchments include:

- Assigned rain gage
- Outlet node ID
- Assigned land uses
- Tributary surface area
- Imperviousness
- Slope
- Characteristic width
- Manning's n for overland flow on both pervious and impervious areas
- Depression storage in both pervious and impervious areas
- Percent of impervious area with no depression storage

Junction Nodes

Junctions are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings. External inflows can enter the system at junctions. Excess water at a junction can become partially pressurized while connecting conduits are surcharged and can either be lost from the system or be allowed to pond atop the junction and subsequently drain back into the junction.

The principal input parameters for a junction are:

- Invert elevation
- Height to ground surface
- Ponded surface area when flooded (optional)

- External inflow data (optional)

Outfall Nodes

Outfalls are terminal nodes of the drainage system used to define final downstream boundaries under Dynamic Wave flow routing. For other types of flow routing they behave as a junction. Only a single link can be connected to an outfall node.

The boundary conditions at an outfall can be described by any one of the following stage relationships:

- The critical or normal flow depth in the connecting conduit
- A fixed stage elevation
- A tidal stage described in a table of tide height versus hour of the day
- A user-defined time series of stage versus time

The principal input parameters for outfalls include:

- Invert elevation
- Boundary condition type and stage description
- Presence of a flap gate to prevent backflow through the outfall

Flow Divider Nodes

Flow Dividers are drainage system nodes that divert inflows to a specific conduit in a prescribed manner. A flow divider can have no more than two conduit links on its discharge side. Flow dividers are only active under Kinematic Wave routing and are treated as simple junctions under Dynamic Wave routing.

There are four types of flow dividers, defined by the manner in which inflows are diverted:

- Cutoff Divider: Diverts all inflow above a defined cutoff value
- Overflow Divider: Diverts all inflow above the flow capacity of the non-diverted conduit
- Tabular Divider: Uses a table that expresses diverted flow as a function of total inflow
- Weir Divider: Uses a weir equation to compute diverted flow

The flow diverted through a weir divider is computed by the following equation:

$$Q_{div}=C_w(fH_w)^{1.5} \quad (3.1)$$

Where Q_{div} is the diverted flow, C_w is the weir coefficient, H_w is the weir heght and f is computed as:

$$f = \frac{Qin - Qmin}{Qmax - Qmin} \quad (3.2)$$

Where Q_{in} is the inflow to the divider, Q_{min} is the flow at which diversion begins, and

$$Q_{max} = C_w(fH_w)^{1.5} \quad (3.3)$$

The user-specified parameters for the weir divider are Q_{min}, C_w and H_w

The principal input parameters for a flow divider are:

- Junction parameters (see above)
- Name of the link receiving the diverted flow
- Method used for computing the amount of diverted flow

Storage Units

Storage Units are drainage system nodes that provide storage volume. Physically they could represent storage facilities as small as a catch basin or as large as a lake. The volumetric properties of a storage unit are described by a function or table of surface area versus height.

The principal input parameters for storage units include:

- Invert elevation
- Maximum depth
- Depth-surface area data
- Evaporation potential
- Ponded surface area when flooded (optional)
- External inflow data (optional)

Conduits

Conduits are pipes or channels in which water flows from one node to another in the conveyance system. They have a wide variety of cross-sectional shapes following some standard open and closed geometries.

Most open channels can be represented with a rectangular, trapezoidal, or user-defined irregular cross-section shape. For the latter, a Transect object is used to define how the channel's depth varies with distance across the cross-section. The most common shapes for new drainage and sewer pipes are circular, elliptical, and arch pipes. They come in standard sizes that are published by the American Iron and Steel Institute in Modern Sewer Design and by the American Concrete Pipe Association in the Concrete Pipe Design Manual. The Filled Circular shape allows the bottom of a circular pipe to be filled with sediment and thus limit its flow capacity. The Custom Closed Shape allows any closed geometrical shape that is symmetrical about the centerline to be defined by supplying a Shape Curve for the cross section.

GeoSWMM uses the Manning's equation to express the relationship among the flow rate (*Q*), cross-sectional area (*A*), hydraulic radius (*R*), and slope (*S*) in all conduits. For standard U.S. units:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \tag{3.4}$$

Here *n* is the Manning roughness coefficient. The slope *S* is interpreted as either the conduit slope or the friction slope (i.e., head loss per unit length), depending on the flow routing method used.

For pipes with Circular Force Main cross-sections, either the Hazen-Williams or Darcy-Weisbach formula is used in place of the Manning equation for fully pressurized flow. For U.S. units the Hazen-Williams formula is:

$$Q = 1.318 CAR^{0.63} S^{0.54}\ Ax \tag{3.5}$$

Where *C* is the Hazen-Williams friction factor that varies inversely with surface roughness and is supplied as one of the cross-section's parameters. The Darcy-Weisbach formula is:

$$Q = \sqrt{\frac{8g}{f}} AR^{1/2} S^{1/2} \tag{3.6}$$

Here *g is* the acceleration of gravity and *f* is the Darcy-Weisbach friction factor. For turbulent flow, the latter is determined from the height of the roughness elements on the walls of the pipe (supplied as an input parameter) and the flow's Reynolds Number using the Colebrook-White equation. The choice of which equation to use is a user-supplied option.

A conduit does not have to be assigned a Force Main shape for it to pressurize. Any of the closed cross- section shapes can potentially pressurize and thus function as force mains that use the Manning equation to compute friction losses.

A conduit can also be designated to act as a culvert if a Culvert Inlet Geometry code number is assigned to it. Culvert conduits are checked continuously during dynamic wave flow routing to see if they operate under Inlet Control as defined in the Federal Highway Administration‘s publication Hydraulic Design of Highway Culverts. Under inlet control a culvert obeys a particular flow versus inlet depth rating curve whose shape depends on the culvert‘s shape, size, slope, and inlet geometry.

The principal input parameters for conduits are:

- Names of the inlet and outlet nodes
- Offset height or elevation above the inlet and outlet node inverts
- Conduit length
- Manning's roughness
- Cross-sectional geometry
- Entrance/exit losses (optional)
- Presence of a flap gate to prevent reverse flow (optional)

Pumps

Pumps are links used to lift water to higher elevations. A pump curve describes the relation between a pump's flow rate and conditions at its inlet and outlet nodes. Four different types of pump curves are supported:

- *Type1*: An off-line pump with a wet well where flow increases incrementally with available wet well volume
- *Type2*: An in-line pump where flow increases incrementally with inlet node depth
- *Type3*: An in-line pump where flow varies continuously with head difference between the inlet and outlet nodes
- *Type4*: A variable speed in-line pump where flow varies continuously with inlet node depth

- Ideal: An "ideal" transfer pump whose flow rate equals the inflow rate at its inlet node. No curve is required. The pump must be the only outflow link from its inlet node. Used mainly for preliminary design.

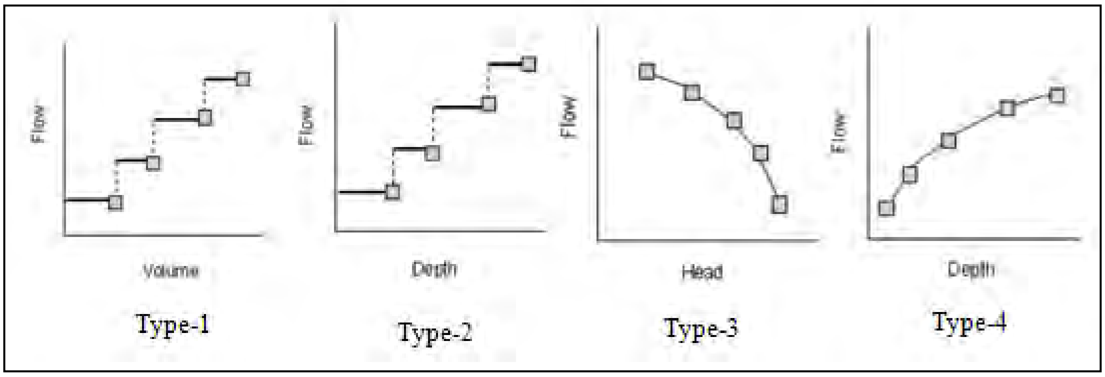


Figure 3.2: Different Types of Pump Curves (Source: User Manual for EPA-SWMM 5.0)

The on/off status of pumps can be controlled dynamically by specifying startup and shutoff water depths at the inlet node or through user-defined Control Rules. Rules can also be used to simulate variable speed drives that modulate pump flow.

The principal input parameters for a pump include:

- Names of its inlet and outlet nodes
- Name of its pump curve
- Initial on/off status
- Startup and shutoff depths

Flow Regulators

Flow Regulators are structures or devices used to control and divert flows within a conveyance system. They are typically used to:

- Control releases from storage facilities
- Prevent unacceptable surcharging
- Divert flow to treatment facilities and interceptors

GeoSWMM can model the following types of flow regulators:

- Orifices
- Weirs
- Outlets

Orifices

Orifices are used to model outlet and diversion structures in drainage systems, which are typically openings in the wall of a manhole, storage facility, or control gate. They are internally represented in GeoSWMM as a link connecting two nodes. An orifice can have either a circular or rectangular shape, be located either at the bottom or along the side of the upstream node, and have a flap gate to prevent backflow.

Orifices can be used as storage unit outlets under all types of flow routing. If not attached to a storage unit node, they can only be used in drainage networks that are analyzed with Dynamic Wave flow routing.

The flow through a fully submerged orifice is computed as:

$$Q = CA\sqrt{2gh} \tag{3.7}$$

Where *Q*= flow rate, *C*= discharge coefficient, *A*= area of orifice opening, *g*= acceleration of gravity, and *h*= head difference across the orifice. The height of an orifice's opening can be controlled dynamically through user-defined Control Rules. This feature can be used to model gate openings and closings.

The principal input parameters for an orifice include:

- Names of its inlet and outlet nodes
- Configuration (bottom or side)
- Shape (circular or rectangular)
- Height or elevation above the inlet node invert
- Discharge coefficient
- Time to open or close

Weirs

Weirs, like orifices, are used to model outlet and diversion structures in a drainage system. Weirs are typically located in a manhole, along the side of a channel, or within a storage unit. They are internally represented in GeoSWMM as a link connecting two nodes, where the weir itself is placed at the upstream node. A flap gate can be included to prevent backflow.

Four varieties of weirs are available, each incorporating a different formula for computing flow across the weir as listed in Table 3.1.

Table 3.1: Available Types of Weir

Weir Type	Cross Section Shape	Flow Formula
Transverse	Rectangular	$C_wLH^{3/2}$
Side Flow	Rectangular	$C_wLH^{5/3}$
V-Notch	Triangular	$C_wSH^{5/2}$
Trapezoidal	Trapezoidal	$C_wLH^{3/2}+C_{ws}SH^{5/2}$
C_w = Weir discharge coefficient, L = Weir length, S = Side slope of V-notch or trapezoidal weir, H= Head difference across the weir, C_{ws}= Discharge coefficient through sides of trapezoidal weir		

Weirs can be used as storage unit outlets under all types of flow routing. If not attached to a storage unit, they can only be used in drainage networks that are analyzed with Dynamic Wave flow routing.

The height of the weir crest above the inlet node invert can be controlled dynamically through user-defined Control Rules. This feature can be used to model inflatable dams.

The principal input parameters for a weir include:

- Names of its inlet and outlet nodes
- Shape and geometry
- Crest height or elevation above the inlet node invert
- Discharge coefficient

Outlets

Outlets are flow control devices that are typically used to control outflows from storage units. They are used to model special head-discharge relationships that cannot be characterized by pumps, orifices, or weirs. Outlets are internally represented in GeoSWMM as a link connecting two nodes. An outlet can also have a flap gate that restricts flow to only one direction.

Outlets attached to storage units are active under all types of flow routing. If not attached to a storage unit, they can only be used in drainage networks analyzed with Dynamic Wave flow routing.

A user-defined rating curve determines an outlet's discharge flow as a function of either the freeboard depth above the outlet's opening or the head difference across it. Control Rules can be used to dynamically adjust this flow when certain conditions exist.

The principal input parameters for an outlet include:

- Names of its inlet and outlet nodes
- Height or elevation above the inlet node invert
- Function or table containing its head (or depth) - discharge relationship

3.3.3 Non-Visual Objects

In addition to physical objects that can be displayed visually on a map, GeoSWMM utilizes several classes of non-visual data objects to describe additional characteristics and processes within a study area.

Climatology

Temperature

Air temperature data are used when simulating snowfall and snowmelt processes during runoff calculations. They can also be used to compute daily evaporation rates. If these processes are not being simulated then temperature data are not required. Air temperature data can be supplied to GeoSWMM from one of the following sources:

- A user-defined time series of point values (values at intermediate times are interpolated)
- An external climate file containing daily minimum and maximum values (GeoSWMM fits a sinusoidal curve through these values depending on the day of the year)

For user-defined time series, temperatures are in degrees F for US units and degrees C for metric units. The external climate file can also be used to directly supply evaporation and wind speed as well.

Evaporation

Evaporation can occur for standing water on subcatchment surfaces, for subsurface water in groundwater aquifers, and for water held in storage units. Evaporation rates can be stated as:

- A single constant value
- A set of monthly average values
- A user-defined time series of daily values

Values computed from the daily temperatures contained in an external climate file daily values read directly from an external climate file.

If rates are read directly from a climate file, then a set of monthly pan coefficients should also be supplied to convert the pan evaporation data to free water-surface values. An option is also available to allow evaporation only during periods with no precipitation.

Wind Speed

Wind speed is an optional climatic variable that is only used for snowmelt calculations. GeoSWMM can use either a set of monthly average speeds or wind speed data contained in the same climate file used for daily minimum/maximum temperatures.

Aquifers

Aquifers are sub-surface groundwater areas used to model the vertical movement of water infiltrating from the subcatchments that lie above them. They also permit the infiltration of groundwater into the drainage system, or exfiltration of surface water from the drainage system, depending on the hydraulic gradient that exists. The same aquifer object can be shared by several subcatchments. Aquifers are only required in models that need to explicitly account for the exchange of groundwater with the drainage system or to establish baseflow and recession curves in natural channels and non-urban systems.

Aquifers are represented using two zones – an un-saturated zone and a saturated zone. Their behavior is characterized using such parameters as soil porosity, hydraulic conductivity, evapotranspiration depth, bottom elevation, and loss rate to deep groundwater. In addition, the initial water table elevation and initial moisture content of the unsaturated zone must be supplied.

Unit Hydrographs

Unit Hydrographs (UHs) estimate rainfall-dependent infiltration/inflow (RDII) into a sewer system. A UH set contains up to three such hydrographs, one for a short-term response, one for an intermediate-term response, and one for a long-term response. A UH group can have up to 12 UH sets, one for each month of the year. Each UH group is considered as a separate object by GeoSWMM, and is assigned its own unique name along with the name of the rain gage that supplies rainfall data to it.

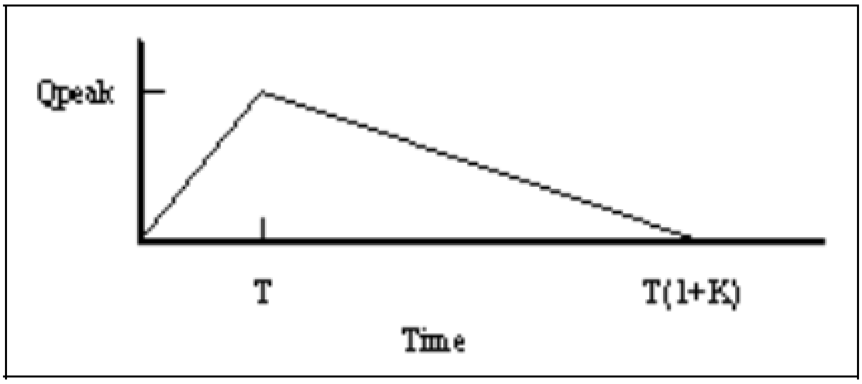


Figure 3.3: An RDII Unit Hydrograph

Each unit hydrograph is defined by three parameters:

- R: the fraction of rainfall volume that enters the sewer system
- T: the time from the onset of rainfall to the peak of the UH in hours
- K: the ratio of time to recession of the UH to the time to peak
- Each unit hydrograph can also have a set of Initial Abstraction (IA) parameters associated with it. These determine how much rainfall is lost to interception and depression storage before any excess rainfall is generated and transformed into RDII flow by the hydrograph. The IA parameters consist of:
    - A maximum possible depth of IA (inches or mm)
    - A recovery rate (inches/day or mm/day) at which stored IA is depleted during dry periods
    - An initial depth of stored IA (inches or mm)

To generate RDII into a drainage system node, the node must identify (through its Inflows property) the UH group and the area of the surrounding sewershed that contributes RDII flow. An alternative to using unit hydrographs to define RDII flow is to create an external RDII interface file, which contains RDII time series data.

Transects

Transects refer to the geometric data that describe how bottom elevation varies with horizontal distance over the cross section of a natural channel or irregular-shaped conduit. Fig. 3.4 displays an example transect for a natural channel.

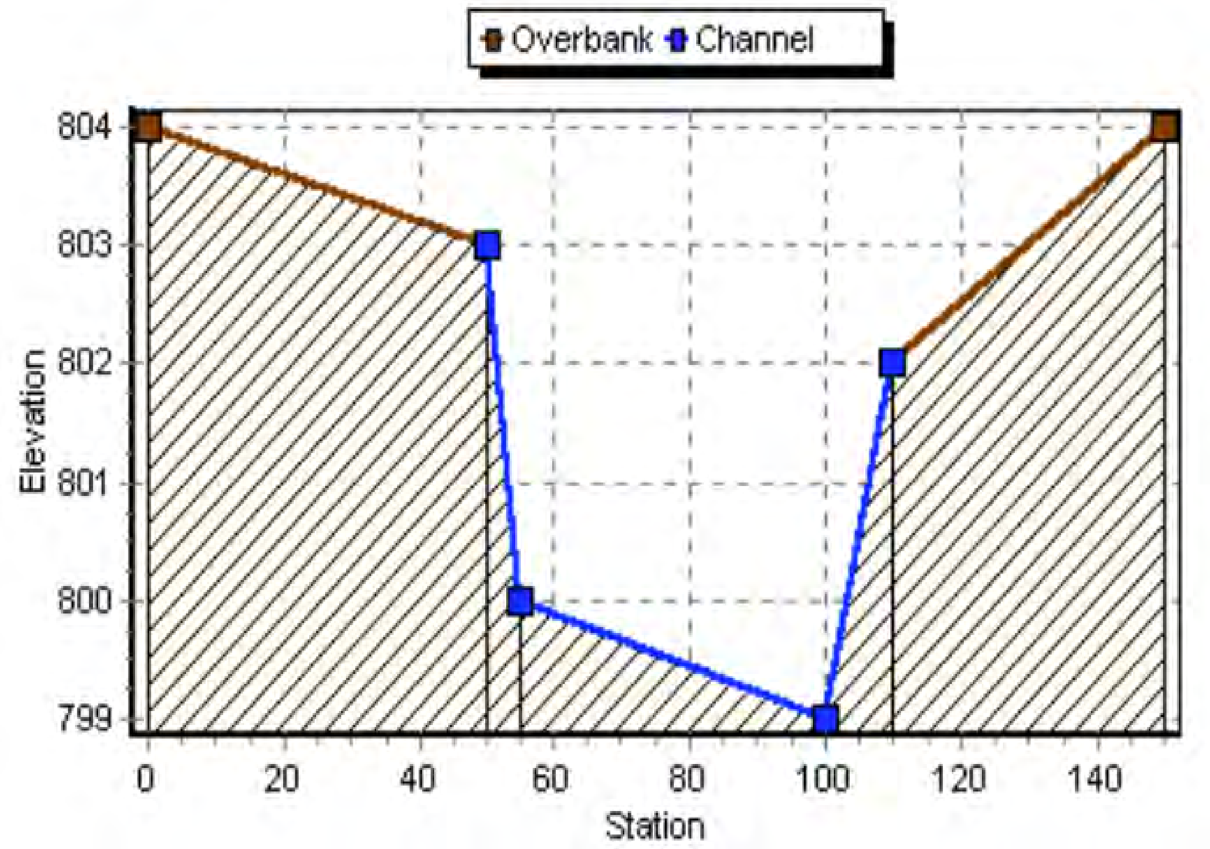


Figure 3.4: Example of a Natural Channel Transect

Each transect must be given a unique name. Conduits refer to that name to represent their shape. A special Transect Editor is available for editing the station-elevation data of transect. GeoSWMM internally converts these data into tables of area, top width, and hydraulic radius versus channel depth. In addition, as shown in the diagram above, each transect can have a left and right overbank section whose Manning's roughness can be different from that of the main channel. This feature can provide more realistic estimates of channel conveyance under high flow conditions.

External Inflows

In addition to inflows originating from subcatchment runoff and groundwater, drainage system nodes can receive three other types of external inflows:

Direct Inflows - These are user-defined time series of inflows added directly into a node. They can be used to perform flow and water quality routing in the absence of any runoff computations (as in a study area where no subcatchments are defined).

Dry Weather Inflows - These are continuous inflows that typically reflect the contribution from sanitary sewage in sewer systems or base flows in pipes and stream channels. They are represented by an average inflow rate that can be periodically adjusted on a monthly, daily, and hourly basis by applying Time Pattern multipliers to this average value.

Rainfall-Dependent Infiltration/Inflow (RDII) - These are Stormwater flows that enter sanitary or combined sewers due to "inflow" from direct connections of downspouts, sump pumps, foundation drains, etc. as well as "infiltration" of subsurface water through cracked pipes, leaky joints, poor manhole connections, etc. RDII can be computed for a given rainfall record based on set of triangular unit hydrographs (UH) that determine a short-term, intermediate-term, and long-term inflow response for each time period of rainfall. Any number of UH sets can be supplied for different sewershed areas and different months of the year. RDII flows can also be specified in an external RDII interface file.

Direct, Dry Weather, and RDII inflows are properties associated with each type of drainage system node (junctions, outfalls, flow dividers, and storage units) and can be specified when nodes are edited. It is also possible to make the outflows generated from an upstream drainage system be the inflows to a downstream system by using interface files.

Control Rules

Control Rules determine how pumps and regulators in the drainage system will be adjusted over the course of a simulation. Some examples of these rules are:

Simple time-based pump control

RULE R1

IF SIMULATION TIME > 8

THEN PUMP 12 STATUS = ON

ELSE PUMP 12 STATUS = OFF

Pump station operation

RULE R3A

IF NODE N1 DEPTH > 5

THEN PUMP N1A STATUS = ON

RULE R3B

IF NODE N1 DEPTH > 7

THEN PUMP N1B STATUS = ON

RULE R3C

IF NODE N1 DEPTH <= 3

THEN PUMP N1A STATUS = OFF

AND PUMP N1B STATUS = OFF

Curves

Curve objects are used to describe a functional relationship between two quantities. The following types of curves are available in GeoSWMM:

- Storage - describes how the surface area of a Storage Unit node varies with water depth
- Shape - describes how the width of a customized cross-sectional shape varies with height for a Conduit link.
- Diversion - relates diverted outflow to total inflow for a Flow Divider node
- Tidal - describes how the stage at an Outfall node changes by hour of the day
- Pump - relates flow through a Pump link to the depth or volume at the upstream node or to the head delivered by the pump
- Rating - relates flow through an Outlet link to the head difference across the outlet
- Control - determines how the control setting of a pump or flow regulator varies as a function of some control variable (such as water level at a particular node) as specified in a Modulated Control rule.

Each curve must be given a unique name and can be assigned any number of data pairs.

Time Series

Time Series objects are used to describe how certain object properties vary with time. Time series can be used to describe:

- Temperature data
- Evaporation data

- Rainfall data
- Water stage at outfall nodes
- External inflow hydrographs at drainage system nodes
- External inflow pollute graphs at drainage system nodes
- Control settings for pumps and flow regulators

Each time series must be given a unique name and can be assigned any number of time-value data pairs. Time can be specified either as hours from the start of a simulation or as an absolute date and time-of-day. Time series data can either be entered directly into the program or be accessed from a user-supplied Time Series file.

For rainfall time series, it is only necessary to enter periods with non-zero rainfall amounts. GeoSWMM interprets the rainfall value as a constant value lasting over the recording interval specified for the rain gage that utilizes the time series. For all other types of time series, GeoSWMM uses interpolation to estimate values at times that fall in between the recorded values.

For times that fall outside the range of the time series, GeoSWMM will use a value of 0 for rainfall and external inflow time series, and either the first or last series value for temperature, evaporation, and water stage time series.

Time Patterns

Time Patterns allow external Dry Weather Flow (DWF) to vary in a periodic fashion. They consist of a set of adjustment factors applied as multipliers to a baseline DWF flow rate or pollutant concentration. The different types of time patterns include:

- Monthly: one multiplier for each month of the year
- Daily: one multiplier for each day of the week
- Hourly: one multiplier for each hour from 12 AM to 11 PM
- Weekend : hourly multipliers for weekend days

Each Time Pattern must have a unique name and there is no limit on the number of patterns that can be created. Each dry weather inflow (either flow or quality) can have up to four patterns associated with it, one for each type listed above.

3.4 Computational Methods

GeoSWMM is a physically based, discrete-time simulation model. It employs principles of conservation of mass, energy and momentum wherever appropriate. This section briefly describes the methods GeoSWMM uses to model Stormwater runoff quantity and quality through the following physical processes:

- Surface Runoff
- Infiltration
- Groundwater
- Snowmelt
- Flow Routing
- Surface Ponding
- Water Quality Routing

3.4.1 Surface Runoff

The conceptual view of surface runoff used by GeoSWMM is illustrated in Fig. 3.5 below. Each subcatchment surface is treated as a nonlinear reservoir. Inflow comes from precipitation and any designated upstream subcatchments. There are several outflows, including infiltration, evaporation, and surface runoff. The capacity of this "reservoir" is the maximum depression storage, which is the maximum surface storage provided by ponding, surface wetting, and interception. Surface runoff per unit area, Q, occurs only when the depth of water in the "reservoir" exceeds the maximum depression storage, d_p, in which case the outflow is given by Manning's equation. Depth of water over the subcatchment (d in feet) is continuously updated with time (t in seconds) by solving numerically a water balance equation over the subcatchment.

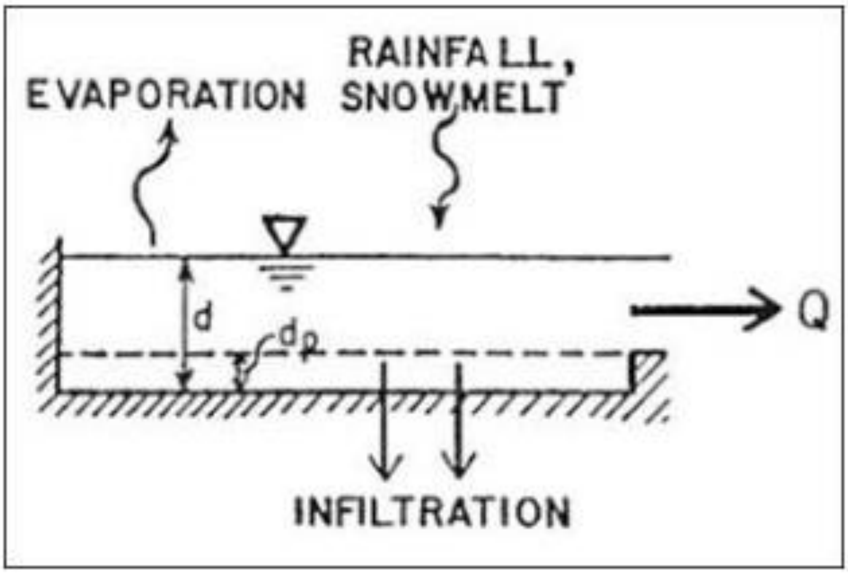


Figure 3.5: Surface runoff (Illustrative Diagram)

3.4.2 Infiltration

Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious subcatchments areas. GeoSWMM offers three choices for modeling infiltration:

Horton's Equation

This method is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. Input parameters required by this method include the maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and a time it takes a fully saturated soil to completely dry.

Green-Ampt Method

This method for modeling infiltration assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The input parameters required are the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front.

Curve Number Method

This approach is adopted from the NRCS (SCS) Curve Number method for estimating runoff. It assumes that the total infiltration capacity of a soil can be found from the soil's tabulated Curve Number. During a rain event this capacity is depleted as a function of cumulative rainfall and remaining capacity. The input parameters for this method are the curve number and the time it takes a fully saturated soil to completely dry.

GeoSWMM also allows the infiltration recovery rate to be adjusted by a fixed amount on a monthly basis to account for seasonal variation in such factors as evaporation rates and groundwater levels. This optional monthly soil recovery pattern is specified as part of a project's Evaporation data.

3.4.3 Groundwater

Fig. 3.6 is a definitional sketch of the two-zone groundwater model that is used in GeoSWMM. The upper zone is unsaturated with variable moisture content. The lower zone is fully saturated and therefore its moisture content is fixed at the soil porosity. The fluxes shown in the Fig., expressed as volume per unit area per unit time, consist of the following:

f_I: Infiltration from the surface

f_{EU}: Evapotranspiration from the upper zone which is a fixed fraction of the un-used surface evaporation

f_U: Percolation from the upper to lower zone which depends on the upper zone moisture content & depth d_U

f_{EL}: Evapotranspiration from the lower zone, which is a function of the depth of the upper zone d_U

f_L: Percolation from the lower zone to deep groundwater which depends on the lower zone depth d_L

f_G: Lateral groundwater interflow to the drainage system, which depends on the lower zone depth d_L as well as the depth in the receiving channel or node

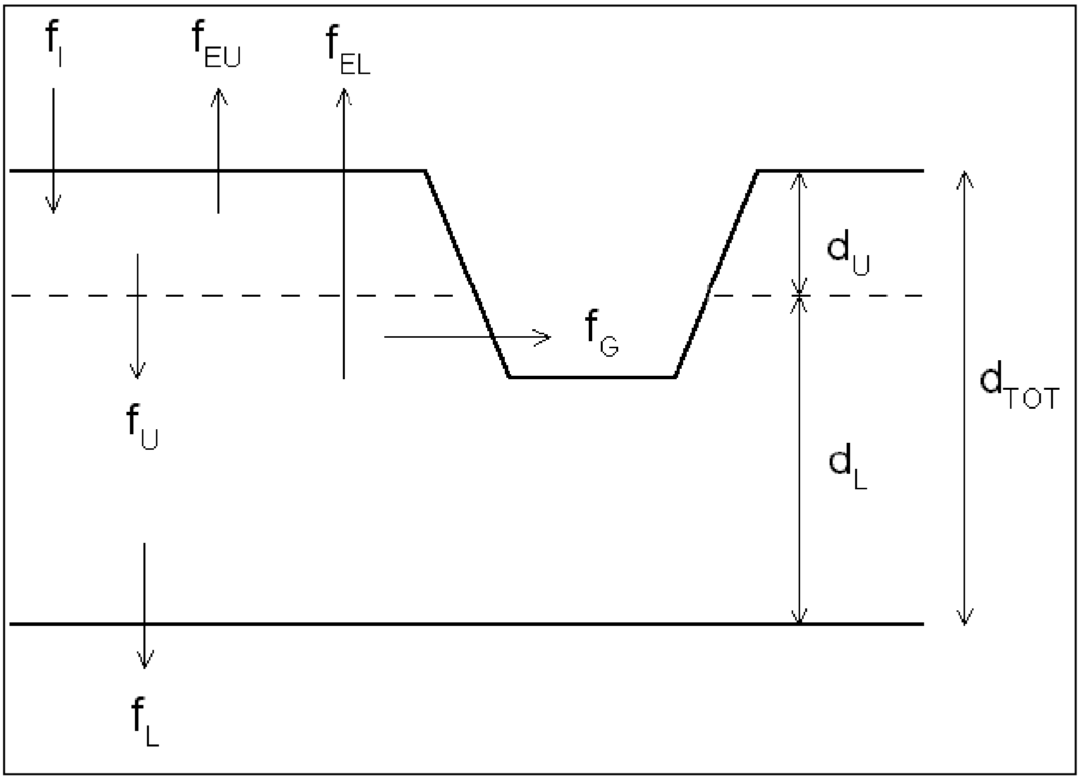


Figure 3.6: Two Zone Groundwater Model

After computing the water fluxes that exist during a given time step, a mass balance is written for the change in water volume stored in each zone so that a new water table depth and unsaturated zone moisture content can be computed for the next time step.

3.4.4 Flow Routing

Flow routing within a conduit link in GeoSWMM is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e., the Saint Venant flow equations). The GeoSWMM user has a choice on the level of sophistication used to solve these equations:

- Steady Flow Routing
- Kinematic Wave Routing
- Dynamic Wave Routing

Steady Flow Routing

Steady Flow routing represents the simplest type of routing possible (actually no routing) by assuming that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The normal flow equation is used to relate flow rate to flow area (or depth).

This type of routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal or pressurized flow. It can only be used with dendritic conveyance networks, where each node has only a single outflow link (unless the node is a divider in which case two outflow links are required). This form of routing is insensitive to the time step employed and is really only appropriate for preliminary analysis using long-term continuous simulations.

Kinematic Wave Routing

This routing method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equal the slope of the conduit.

The maximum flow that can be conveyed through a conduit is the full normal flow value. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available.

Kinematic wave routing allows flow and area to vary both spatially and temporally within a conduit. This can result in attenuated and delayed outflow hydrographs as inflow is routed through the channel. However this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow, and is also restricted to dendritic network layouts. It can usually maintain numerical stability with moderately large time steps, on the order of 5 to 15 minutes. If the aforementioned effects are not expected to be significant then this alternative can be an accurate and efficient routing method, especially for long-term simulations.

Dynamic Wave Routing

Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations (finite difference form) and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes.

With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flows can exceed the full normal flow value. Flooding occurs when the water depth at a node exceeds the maximum available depth, and the excess flow is either lost from the system or can pond atop the node and re-enter the drainage system.

Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flow in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices. This generality comes at a price of having to use much smaller time steps, on the order of a minute or less (GeoSWMM will automatically reduce the user-defined maximum time step as needed to maintain numerical stability).

Each of these routing methods employs the Manning equation to relate flow rate to flow depth and bed (or friction) slope. The one exception is for circular Force Main shapes under pressurized flow, where either the Hazen-Williams or Darcy-Weisbach equation is used instead.

3.4.5 Surface Ponding

Normally in flow routing, when the flow into a junction exceeds the capacity of the system to transport it further downstream, the excess volume overflows the system and is lost. An option exists to have instead the excess volume be stored atop the junction, in a ponded fashion, and be reintroduced into the system as capacity permits. Under Steady and Kinematic Wave flow routing, the ponded water is stored simply as an excess volume. For Dynamic Wave routing, which is influenced by the water depths maintained at nodes, the excess volume is assumed to pond over the node with a constant surface area. This amount of surface area is an input parameter supplied for the junction.

Alternatively, the user may wish to represent the surface overflow system explicitly. In open channel systems this can include road overflows at bridges or culvert crossings as well as additional floodplain storage areas. In closed conduit systems, surface overflows may be conveyed down streets, alleys, or other surface routes to the next available Stormwater inlet or open channel. Overflows may also be impounded in surface depressions such as parking lots, back yards or other areas.

CHAPTER FOUR

METHODOLOGY

4.1 General

In order to develop mathematical hydrologic and hydrodynamic model, various kinds of data of recent and previous years have been collected and compiled. These data also form the basis for further analysis and interpretation of model results leading to accurate assessment of hydrologic and hydrodynamic condition of the study area.

According to the modeling requirements, data includes satellite images, digital elevation model, landuse data, rainfall data, water level and cross-sections. This chapter describes a brief description about the selection of study area, data source, data collection, data processing and analysis of data along with model setup and calibration.

4.2 Study Area

Dhaka is a city of tropical monsoon climate with four seasons: pre-monsoon (March to May), monsoon (June to September), post-monsoon (October to November) and dry season (December to February). Average monthly rainfall during monsoon period varies between 300mm to 450mm. This high intensity rainfall demands for efficient stormwater management which is currently absent.

Figure 4.1: View of the Balu River from Balu Bridge

Dhaka, the rapidly growing capital of Bangladesh went through considerable change of physical setting during last few decades. It can be assumed that this process will continue in future, and the low lying Eastern part of the city might face the challenge of urbanization the most. Dhaka is surrounded by a number of rivers that include Buriganga, Tongi, Turag, Balu and Sitlakhya. Eastern part of Dhaka features a mix of urban and rural landscapes interspersed with numerous wetlands and canals. Canals are the main component of the drainage system. Outfalls of the khals are mainly to the Balu River; however, some also discharge into Tongi Khal. A road bypass likely to be completed by the year 2025 will function as embankment along the right bank of the Balu River and protect Eastern Dhaka from river flooding.

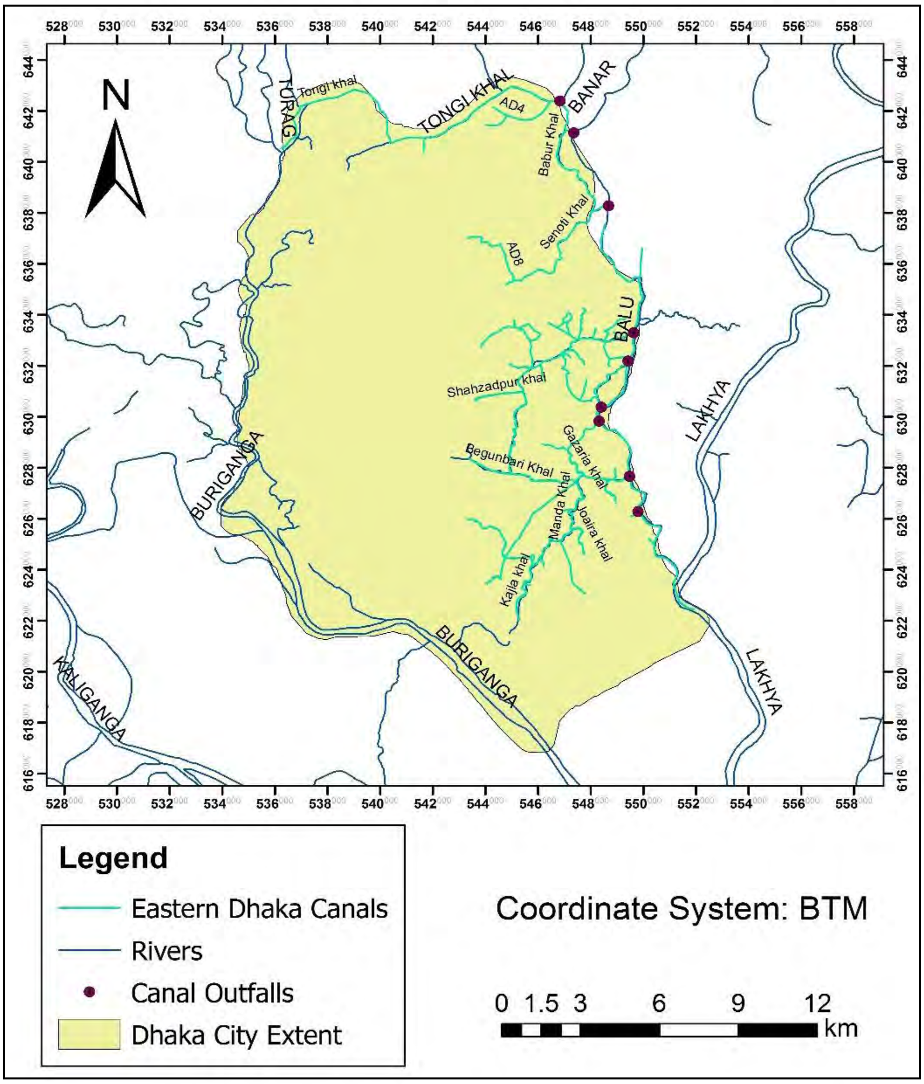


Figure 4.2: Drainage Network of Eastern Dhaka

Several outfalls are observed from Figure 4.2 that drain water from catchments into the Balu River. One of the important canals of the network is the Begunbari Canal. In this study, Bagunbari canal catchment area of Eastern Dhaka is selected to model in GeoSWMM. This catchment covers most part of the Eastern Dhaka with an area of 47.67 square km. It is bounded to the north by the Madani Avenue; the Balu River to the east; Demra Road to the south, which forms a boundary with the DND area; and Pragati Sharani to the west, which protects Western Dhaka from river flooding.

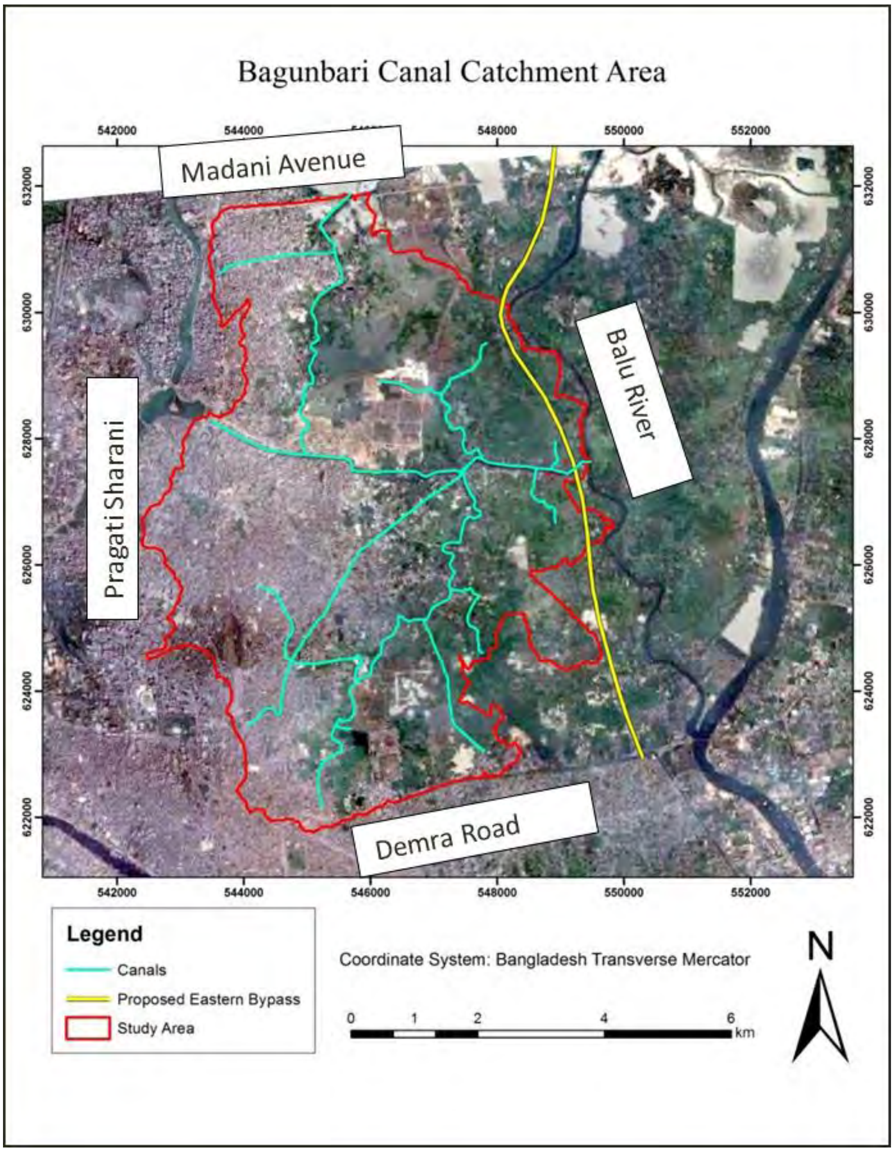


Figure 4.3: Study Area Map

Figure 4.3 is showing the boundary of the study area. Here, almost all the stormwater is drained by natural canal system. From figure, it is seen that the Balu River flows at the east of the catchment. During high flood monsoon, the external water from the river enters into the city. Hence, the stormwater cannot be discharged efficiently and the low lying eastern part of the city suffers from inundation. It can be observed that, western part of the study area is built up, the other parts of the city is undeveloped. The river water can be prevented to enter into the city area by building an embankment along the Balu River. In that case, the internal stormwater have to be drained by adequate pumping and retention facilities.

Figure 4.3 also shows the location of proposed Eastern Bypass Road. While in the figure 4.4 we can observe the current landuse pattern of the study area. The presented landuse map is derived from IWM report which originally followed DAP, 2005. Land use in Eastern Dhaka is dedicated mainly to agriculture and residential use. The study area boundary is predominantly governed by agricultural lands, open spaces and wetlands with some extent of residential area at its west. However, after the Eastern Embankment, it is expected the city to be developed at the east. Most of the lands of the study area will then be built up area.

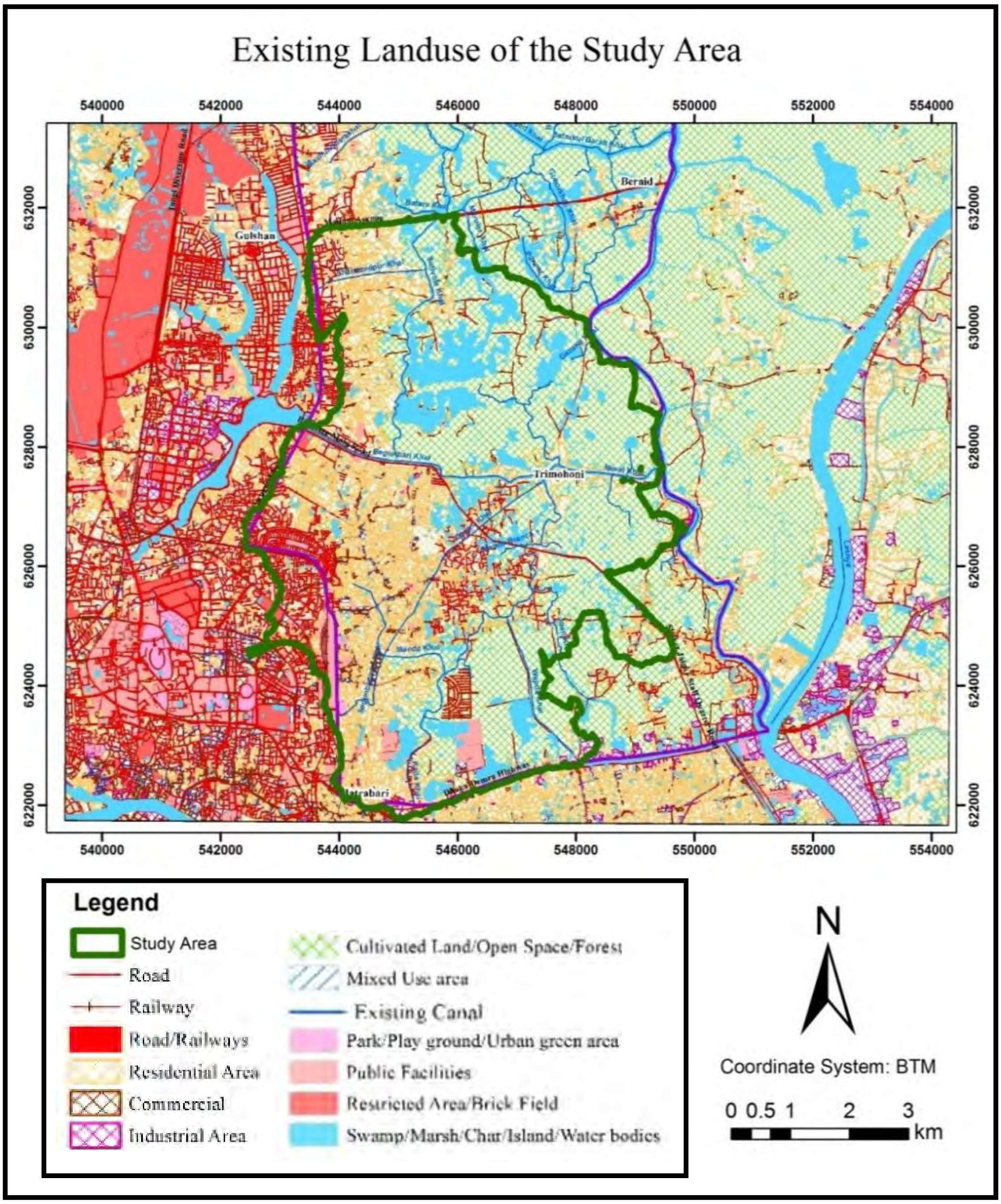


Figure 4.4: Existing Landuse of the Study Area (Source: IWM)

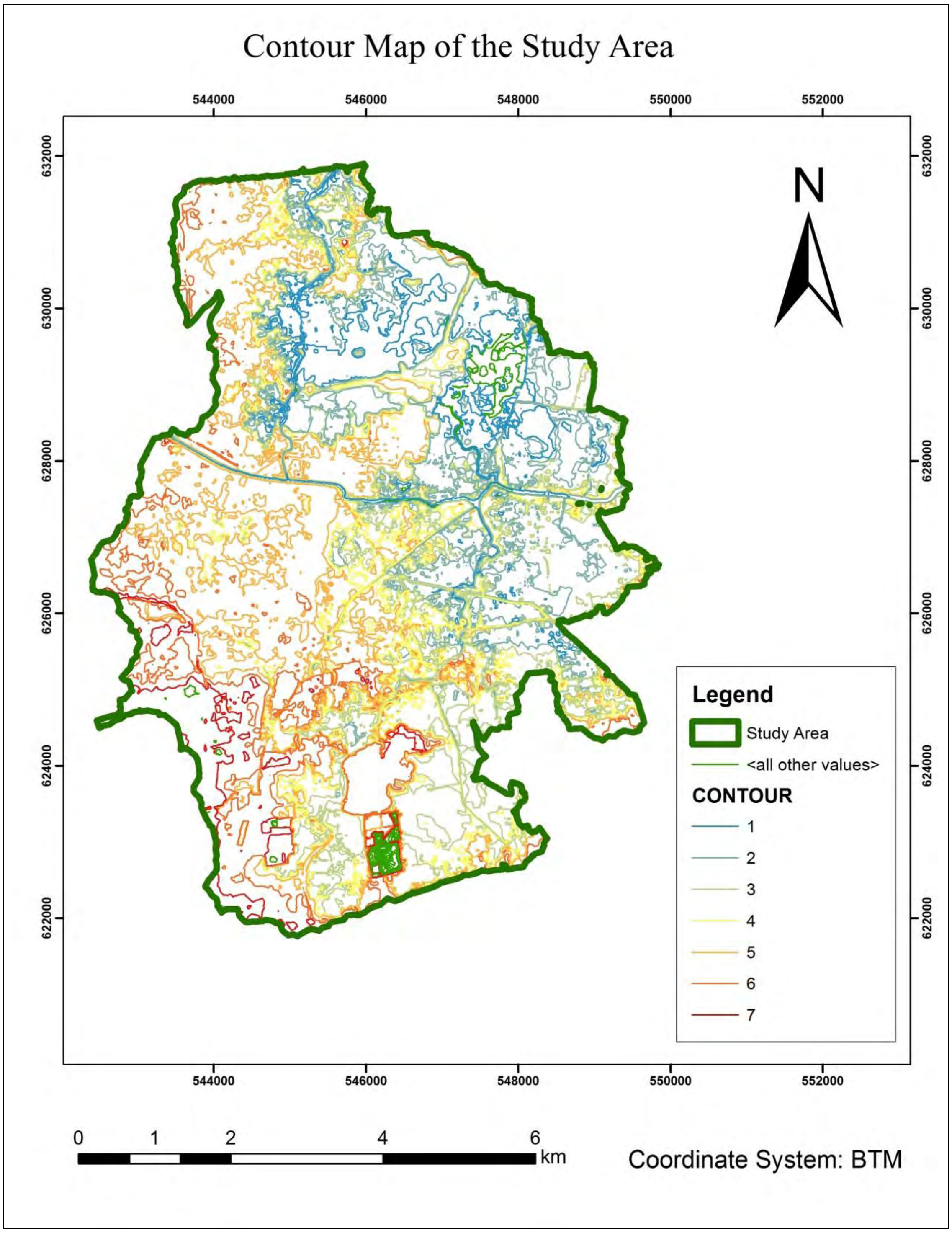


Figure 4.5: Contour Map of the Study Area

Figure 4.5 presents the contour map of the area. Most of the area in this catchment is low lying. Around half of the area is below 4 mPWD level. The built up western part is comparatively higher in elevation. It is observed that, where road infrastructures are present, the area there is more populated and hence, the elevation is higher.

The climate of the study area can be classified as tropical monsoon. It consists of four hydrological seasons: pre-monsoon (April to May), monsoon (June to September), post-monsoon (October to December) and dry season (January to March). Annual average rainfall in this area is about 2000 mm (Table 4.1). Extraordinary rainfall events have been observed in the years 2004 and 2009 which caused serious drainage congestions. On 13 September, 2004, 341mm of rainfall occurred within a span of 8 hours (Ahmed, 2008). And, on July 28, 2009, 290mm of rainfall occurred within a span of 6 hours (Uddin, 2009).

Average temperature varies from about 20^0 C in December and January to about 30^0 C in April to September. Maximum temperature sometimes exceeds 40^0 C in March and April. The average monthly evaporation rate varies from 75 to 130mm. It is lowest in November and highest in August.

The climatic conditions in the study area are shown in Table 4.1.

Table 4.1: Climatic Conditions in the Study Area

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Temp. (^{0}C)												
-Max	34.2	36.6	40.6	42.3	40.6	38.4	35.2	35.9	35.3	38.8	33.3	31.2
-Min	5.6	4.5	10.4	15.6	18.4	20.4	21.7	21.0	22.0	10.4	10.6	6.7
-Average	18.8	21.5	26.1	28.7	28.9	28.7	28.7	28.7	28.7	27.4	23.6	19.8
Rel. Humidity (%)	70	66	63	71	79	86	87	86	86	81	75	74
Evaporation (mm)	104	79	81	77	78	83	87	130	118	106	75	105
Wind Velocities (Knots)	2	2	3	5	5	4	4	4	3	2	1	1
Average Rainfall(mm)	6.5	20.2	52.3	124.0	283.0	398.2	391.4	328.0	264.0	160.0	25.3	7.4

Source: BMD

4.3 Field Visit

A visit to Maniknagar pump station had been made on the 2 August, 2016. Stormwater comes into its reservoir area from Motijhil and surrounding area. The total pump capacity is 15 cumec and pumping starts when intake level is around 4.0 mPWD. Thus water is discharged into the catchment area from here. Figure 4.6 shows different components of the pump house.

Figure 4.6: Maniknagar Pump house Area

Figure 4.7: Rampura Pump house Area

Rampura pump house was visited on the 12 January, 2016. It is the largest operating pump station with a capacity of 25 cumec. It discharges water from upstream reservoir Hatirjhil. A large amount of stormwater of Central Dhaka is discharged into the Begunbari canal in monsoon from here.

The Water Level calibration location at Banasree was visited on the same day. The local people gave an idea about the rise of water level there. The information suggested that banks are not overtopped here. The banks were found well demarcated but the quality of water was not good. Water and waste were simultaneously dumped into the canal and thus the water was very contaminated.

Figure 4.8: Pictorial View of Banasree Area

Trimohoni is the meeting place of three canals- Begunbari, Gerani and Manda Canal. The place was visited on the 11 May, 2017. Both engine driven boats and country boats ply from Balu river upto Banasree market via Trimohoni even in the dry season. A conversation with commercial boatman reveals that they use this route to market their vegetable products. The boat came from Comilla using the Mehgna-Lakhya-Balu route. Though the navigability is good here but the water has a thick black colour with stinky smell. This artery drains both storm water and untreated sewage and waste water.

There are settlement and land areas near Balu on way to Demra from Norai outfall. The green areas are fast getting dredge filled by the real estate companies. Small pockets of agricultural areas are still available. The canals are mostly maintaining un-encroached sections except at areas near market or homesteads or at bridge crossings.

Figure 4.9: Pictorial View of Trimohoni Area

The road beside Manda Canal is locally known as Trimohoni Road. It continues beside the canal at its left bank till Trimohoni area. The surrounding area was rural in near past and now the urban effects are well visible. Though the canal section is narrow, construction of bridge over the canal was observed in frequent intervals. Both side of the canal is under development and the land levels are getting elevated. Near Nandipara Bridge, the locality is comparatively densely populated. Large amount of waste was found dumped into the canal and there was also illegal encroachment over the canal. The large amount of solid waste almost fully blocked the flow of water. So, the management of solid waste of the locality has to be brought under certain authority. Otherwise it will seriously impact the stormwater flow in monsoon by changing the canal section.

Figure 4.10: Pictorial View of Manda Canal at Nandipara

The field visits have provided the following understanding to contribute to the study:

- Many rural areas yet are not under City Corporation and proper solid waste management should be arranged there in quick time. Otherwise the use of canal as dumping zone will seriously harm the conveyance capacity of canals;
- The general level of land around road infrastructures is higher (around 4-6 m PWD);
- In case of delay, the remaining low lying land suitable for retention pond/ reservoir may be lost;
- There are areas distributed at different places may be considered for retention pond;
- Major drainage channels are still active though encroachments are going on;
- In case of increasing of drainage channel width, homesteads, market area may be seen as obstacles at places;
- The stinky smell from water of drainage channels and Balu river indicates bad quality of water;
- There are distinct channel that offer navigational facilities transporting various cargo comprising agricultural produce to construction materials, passenger movement from different places to far inside the project area . Some of these channels offer round the year movement, others either seasonal or navigable during high tide. The vessels vary in sizes, in general, one such cargo vessel can be of 20m in length, 3.5m wide and 3m draft. One

meter depth of water is needed for these vessels. Thus, provision for navigation lock may be needed.

4.4 Data Collection

The necessary data are collected from various sources. These data include: satellite images, landuse data, digital elevation model, rainfall data, canal cross-sections, and river and canal water level.

The 10m X 10m resolution Digital Elevation Model data has been collected from RAJUK. This data sets the base of catchment delineation and different catchment parameters. Since the study area is in the city region and comparatively small, high resolution data is required and this resolution is sufficient to provide satisfactory results.

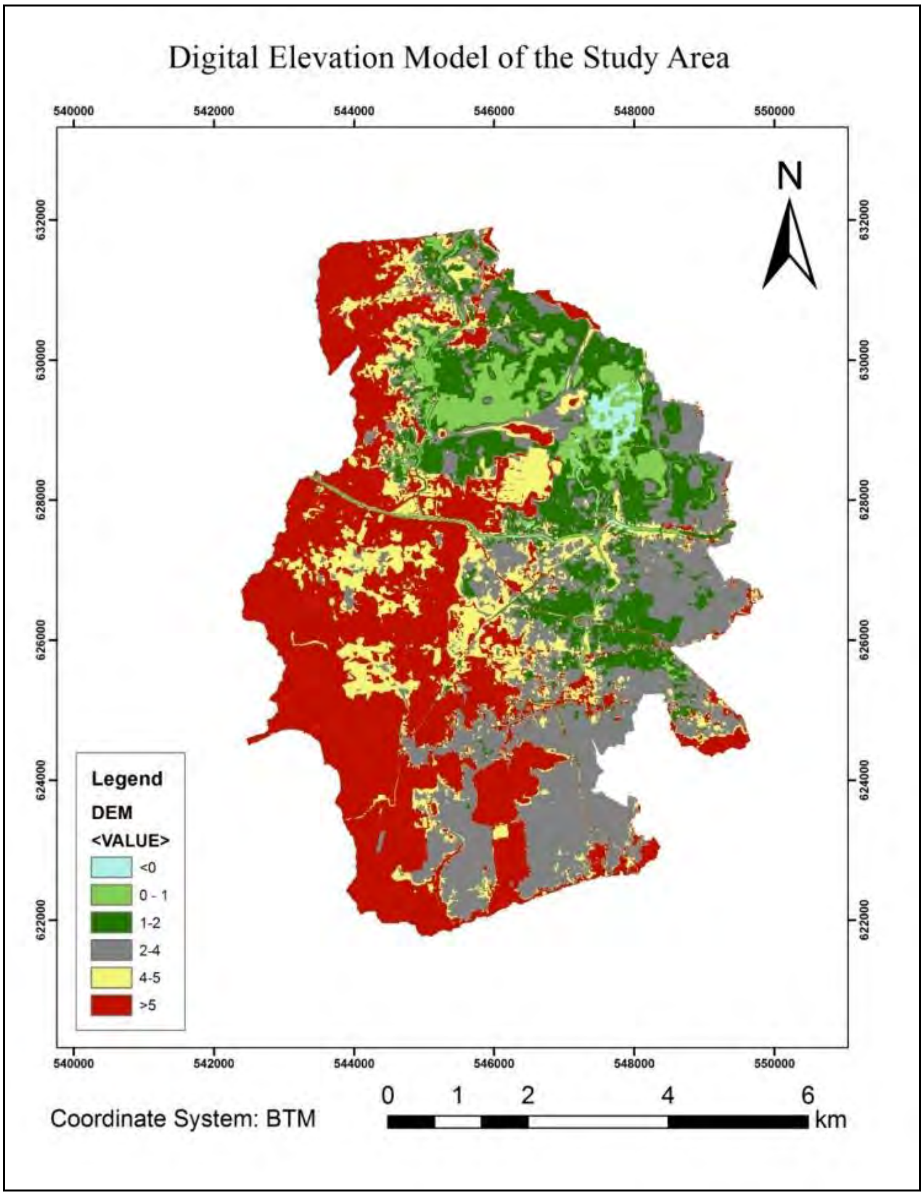


Figure 4.11: Elevations of the Study Area

Rainfall data has been collected from BMD while the Water level data is collected from BWDB. The locations of the stations are shown in the map (Figure 4.12).

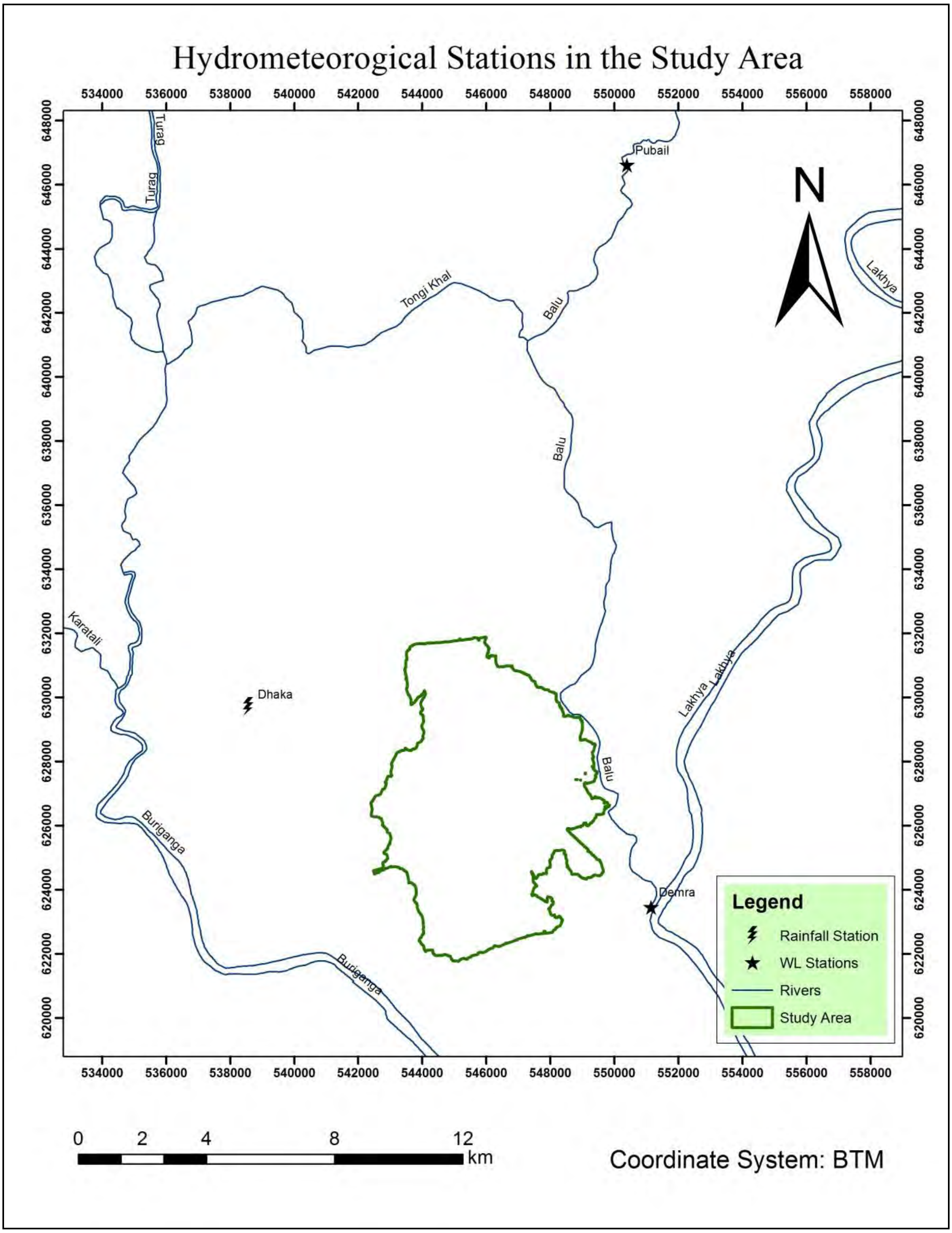


Figure 4.12: BMD and BWDB Stations in the Study Area

The drainage infrastructure data has been collected from IWM. The stormwater of the area is mainly conveyed by the natural canals. The canal network and the sample cross section are shown here in Figure 4.13 and 4.14.

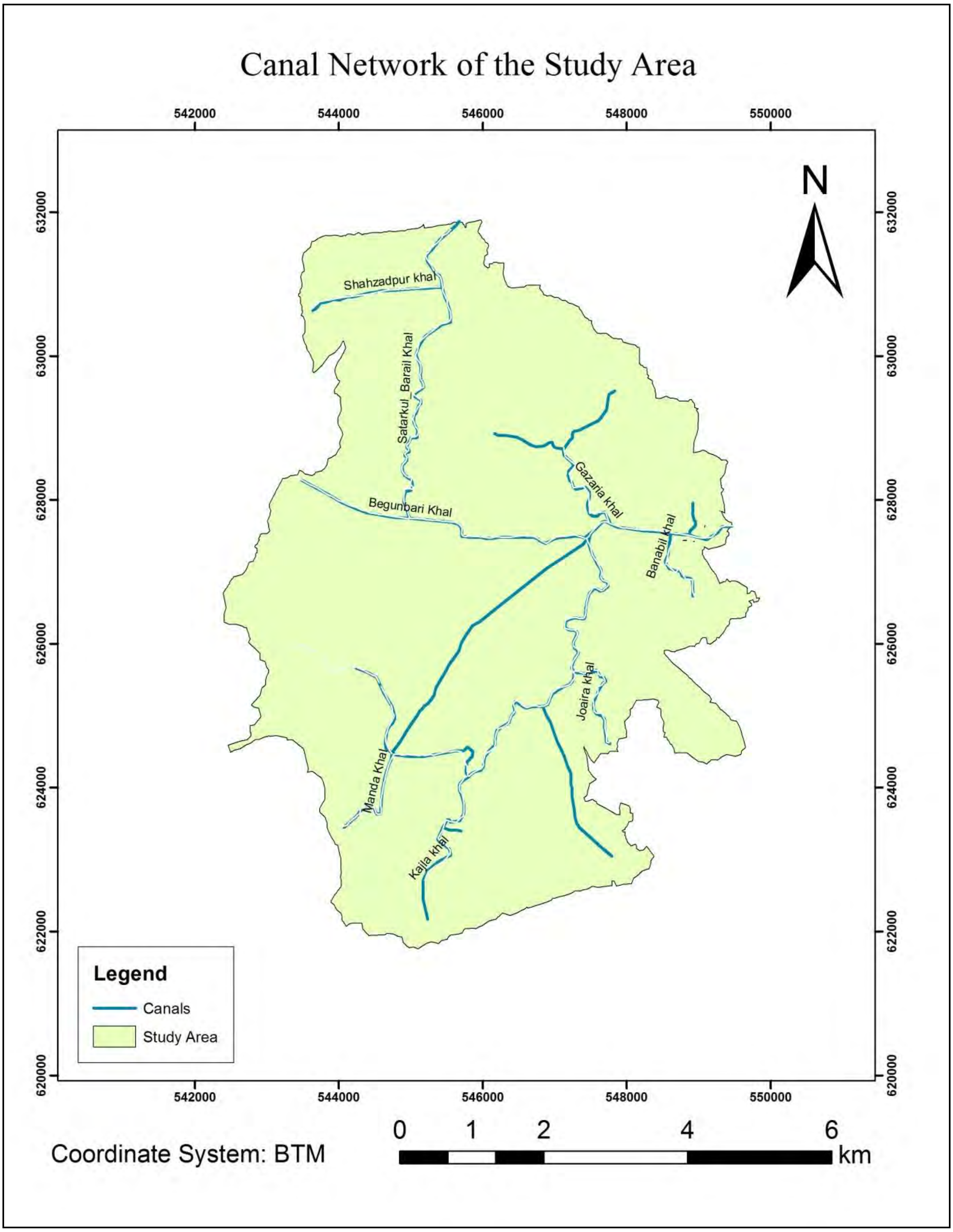
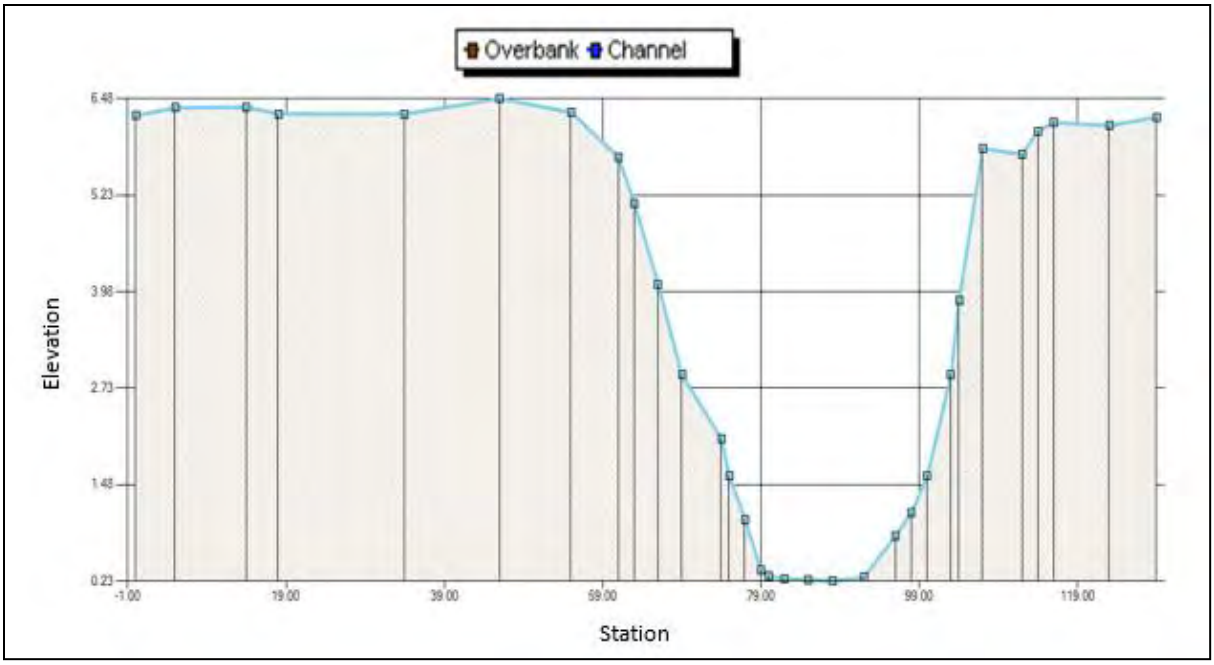


Figure 4.13: Canal Network in the Study Area



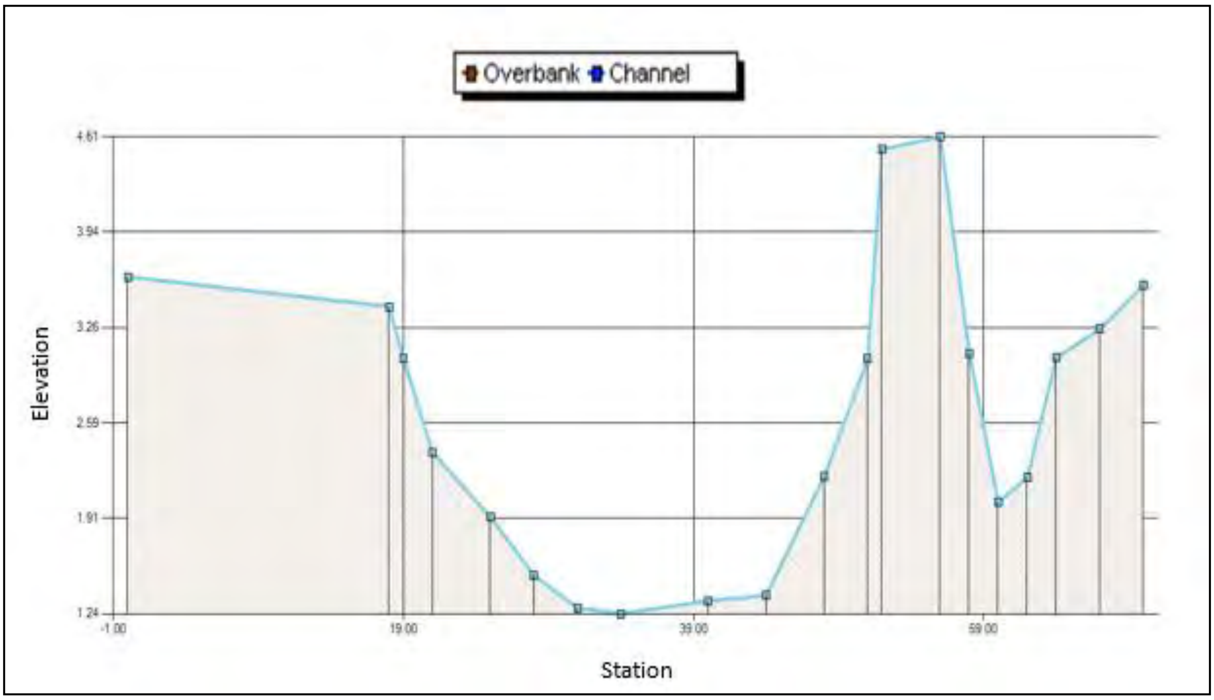


Figure 4.14: Canal Cross-section of Begunbari (upper) and Manda (lower) Canal

The summary of data collection is presented in Table 4.2.

Table 4.2: Summary of Collected Data

Data Type	Station/Location	Source	Time/Duration
Topography (DEM)	Dhaka	RAJUK/SoB	2015
Meteorological (Daily Rainfall)	Station: Dhaka	BMD	1953-2016
Meteorological (3-Hourly Rainfall)			2003-2016
Land use	Eastern Dhaka	RAJUK	2005, 2035
		Google Earth	2017
River stage (Balu River)	Station: Demra and Pubail	BWDB	1945-2014
Drainage channel sections	Eastern Dhaka	IWM/BWDB	2015

4.5 Data Analysis

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 exceedence probability, *P(F)*. The reciprocal of the exceedence probability is the return period T in years, i.e., $T = 1/P(F)$. The length of record should be sufficient to justify extrapolating the frequency relationship. For example, it might be reasonable to estimate a 50-year flood on the basis of a 30-year record, but to estimate a 100-year flood on the basis of a 10-year record would normally be absurd (Neill, 1992). 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.

Analytical Frequency Analysis

Analytical frequency analysis is based on fitting theoretical probability distributions to given data. Numerous distributions have been suggested on the basis of their ability to –fit" the plotted data from streams (Lindsey, 1992). The Log Normal and Log-Pearson Type III (LP3) has been adopted for use in the United States federal agencies for flood analysis. The first asymptotic distribution of extreme values (EV1), commonly called Gumbel Distribution has been widely used and is recommended in the United Kingdom. EV1 Distribution was found to fit peak flow data for several rivers in Bangladesh (Saleque, 1991).

The three distribution functions mentioned above can be used for water level data as well, though EVI distribution is most frequently used in Bangladesh; hence used in this present study. In this present study, Gumbel, LP3 and log normal distributions are analyzed for rainfall data to find out the best-fit frequency distribution function.

Probability Distribution Functions

Extreme Value Distributions: 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. The cumulative distribution function for the GEV is

$$F(x) = \exp\left[-\left(1-\kappa\frac{x-u}{\alpha}\right)^{1/\kappa}\right] \qquad (4.1)$$

where κ, u, and α are parameters to be determined. For EVI Distribution x is unbounded, while for EVII, *x* is bounded from below, and for EVIII, x is bounded from above. The EVI and EVII Distributions are also known as the Gumbel and Frechet Distributions, respectively.

The Extreme Value Type I (EVI) cumulative distribution function is

$$F(x) = \exp\left[-\exp\left(-\frac{x-u}{\alpha}\right)\right] \qquad -\infty \le x \le \infty \qquad (4.2)$$

The parameters are estimated by

$$\alpha = \frac{\sqrt{6}}{\pi}s \quad \text{and} \quad u = \bar{x} - 0.5772\alpha \qquad (4.3)$$

Eq (4.2) can be expressed as

$$F(x) = e^{-e^{-y}} \qquad (4.4)$$

where y is the reduced variate defined as

$$y = \frac{x-u}{\alpha} \qquad (4.5)$$

Solving Eq (4.4) for y:

$$y = -\ln\left[\ln\left(\frac{1}{F(x)}\right)\right] \tag{4.6}$$

Noting that the probability of occurrence of an event $x \geq x_T$ is the inverse of its return period T, we can write

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

so $$F(x_T) = 1 - \frac{1}{T}$$

and substituting for $F(x_T)$ into Eq (4.6)

$$y_T = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right] \tag{4.7}$$

For a given return period x_T is related to y_T by Eq (4.5), or

$$x_T = u + \alpha y_T \tag{4.8}$$

Frequency Analysis using Frequency Factors

Calculating the magnitudes of extreme events by the method outlined in the above example requires that the probability distribution function be invertible, that is, given a value of T or $F(x_T) = 1 - \frac{1}{T}$, the corresponding value of x_T can be determined. Some probability distribution functions are not readily invertible, like the Normal and Pearson Type III Distributions. Thus an alternative method based on frequency factor is used for calculating the magnitudes of extreme events. Chow (1951) has shown that most frequency functions can be generalized to

$$x_T = \bar{x} + K_T s \tag{4.9}$$

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 distributions, such as LP3.

In the event that the variable analyzed is $y = \log x$, for example as in Lognormal and LP3 Distributions, the same method is applied to the statistics for the logarithms of data using $y_T = \bar{y} + K_T s_y$, and the required value of x_T is found taking antilog of y_T.

Chow (1951) proposed the frequency factor as in Eq (4.9), and it is applicable to many probability distributions used in hydrologic frequency analysis. The K-T relationship can be expressed in mathematical terms or by a table.

Alternate Form of EVI Distribution

Chow (1951) derived the following expression for frequency factor for the EVI Distribution

$$K_T = \frac{\sqrt{6}}{\pi}\left[0.5772 + \ln\left\{\ln\left(\frac{T}{T-1}\right)\right\}\right] \quad (4.10)$$

Log-Pearson Type III (LP3) Distribution: The recommended procedure for use of the LP3 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 coefficient of skewness

$$C_s = \frac{n\sum(y_i - \bar{y})^3}{(n-1)(n-2){s_y}^3}$$

4) Compute $y_T = \bar{y} + K_T s_y$ (4.11)

where K_T is taken from defined table.

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

Table gives values of the frequency factors for the LP3 Distribution for various values of return period and coefficient of skewness, C_s. When C_s =0, the frequency factor is equal to the standard normal variable z.

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 $y_T = \bar{y} + K_T s_y$

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

So, K_T can be taken from Table.

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

Best-fit Probability Distribution

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-Square Test (χ^2 – test) is used.

Chi-Square Test: The test statistic is given by

$$\chi^2 = \sum_{i=1}^{k} \frac{n[f_s(x_i) - p(x_i)]^2}{p(x_i)} \qquad (4.12)$$

where k is the number of intervals; the sample value of the relative frequency of interval i is, $f_s(x_i) = n_i/n$; the theoretical value of the relative frequency function (also called incremental probability function) is $p(x_i) = F(x_i) - F(x_{i-1})$. It may be noted that $nf_s(x_i) = n_i$, the observed number of occurrences in interval i, and $np(x_i)$ is the corresponding expected number of occurrences in interval i.

Analysis of Rainfall Data

The three probability distribution function used in the present study (i.e. EVI, LP3 and log normal), EVI and log normal distributions are two parameter distributions; whereas, LP3 is a three parameter distribution. For a certain dataset, when the computed χ^2 value is lowest for any among probability distribution functions, the data series can be said to fit that particular distribution. Here, the frequency analysis has been done for annual maximum value of consecutive 2-days rainfall. The analysis has been done for the period 1953-2016.

Table 4.3: Annual Maximum Value of Daily Rainfall and Total Annual Rainfall for Dhaka Station (1953-2016)

Year	Annual Maximum Value of Daily Rainfall (mm)	Total Annual Rainfall (mm)	Year	Annual Maximum Value of Daily Rainfall (mm)	Total Annual Rainfall (mm)
1953	90	1832	1985	92	2053
1954	147	2102	1986	176	2500
1955	115	1376	1987	138	2187
1956	326	2495	1988	135	2482
1957	73	1554	1989	118	1627
1958	137	1194	1990	94	2103
1959	125	2085	1991	123	2850
1960	141	1834	1992	90	1169
1961	185	2169	1993	140	2819
1962	116	1786	1994	74	1540
1963	189	1971	1995	83	1751
1964	114	2291	1996	150	2044
1965	177	2089	1997	121	1896
1966	257	1814	1998	122	2310
1967	125	2053	1999	141	2374
1968	145	1873	2000	158	2124
1969	86	1540	2001	71	1679
1970	152	1995	2002	88	1875
1971	251	1922	2003	93	1693
1972	231	1698	2004	341	2347
1973	168	2083	2005	128	2637
1975	143	2145	2006	185	1919
1976	163	2238	2007	152	2885
1977	100	1861	2008	190	2385
1978	128	2251	2009	333	1931
1979	108	1837	2010	87	1523
1980	91	2218	2011	94	1776
1981	81	1865	2012	62	1329
1982	146	1805	2013	122	1590
1983	133	2374	2014	70	1472
1984	151	3028	2015	90	2175
			2016	75	1366

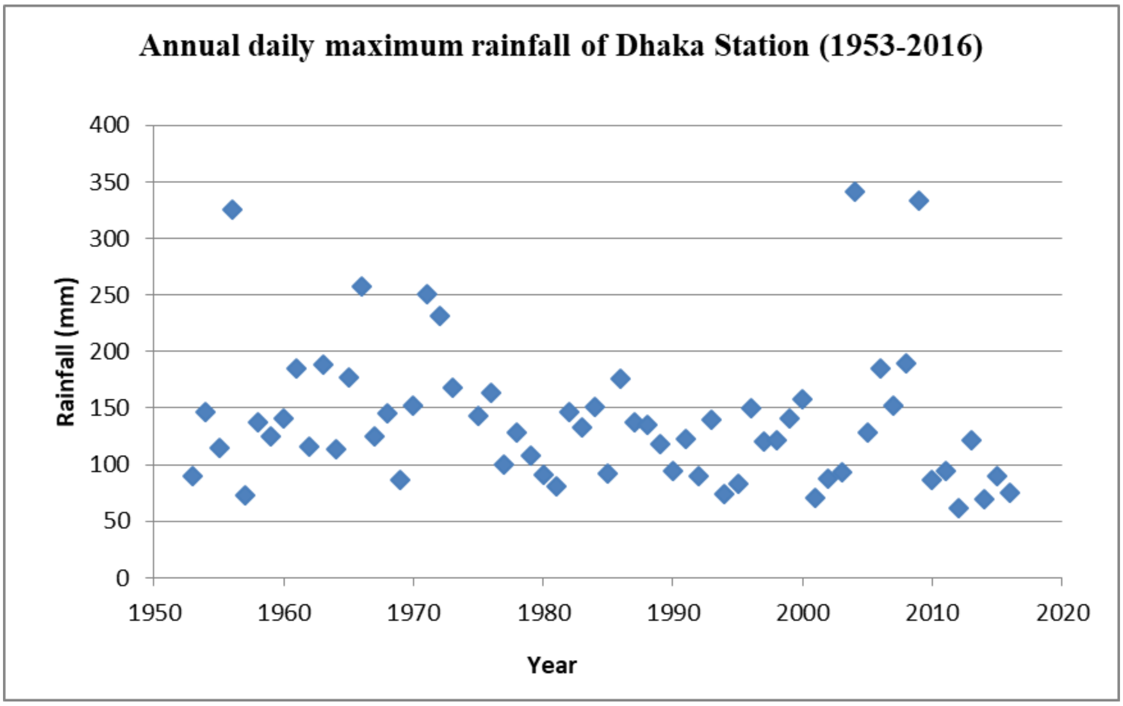


Figure 4.15: Annual Daily Maximum Rainfall at Dhaka

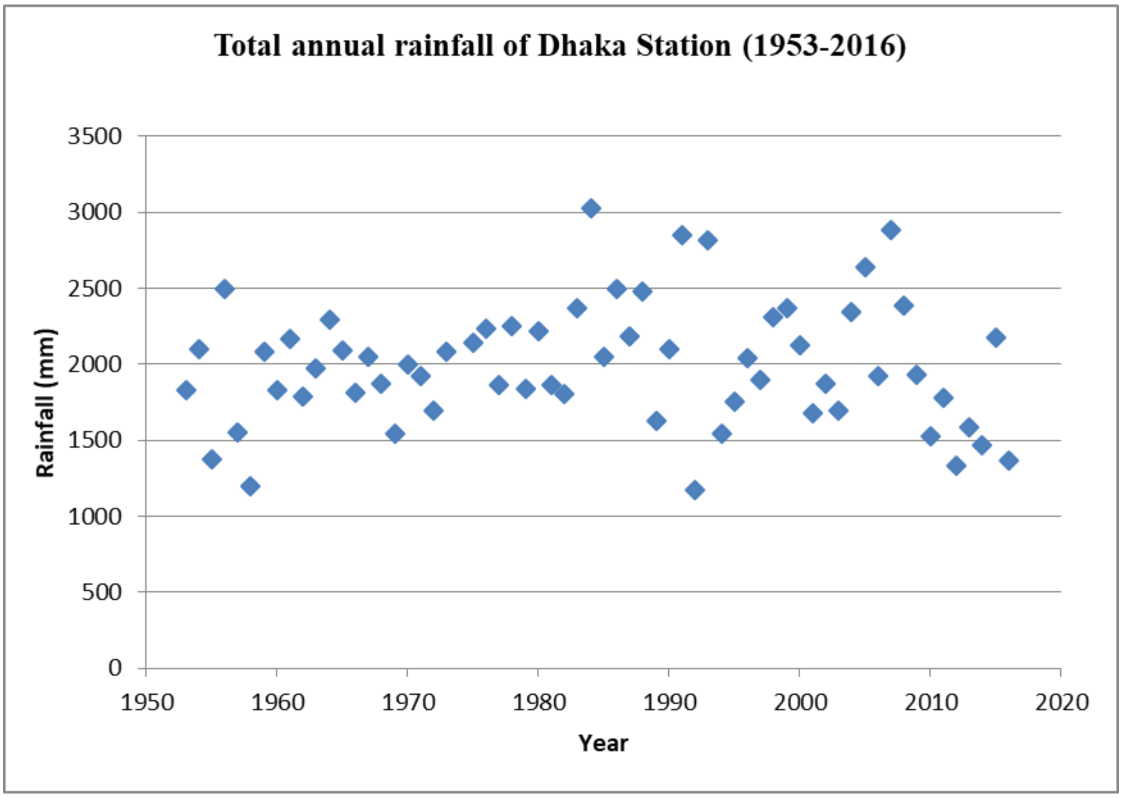


Figure 4.16: Total Annual Rainfall at Dhaka

Table 4.4: Chi Sq. Value from Annual Series of Maximum Value of 2 day rainfall (mm) for EVI Distribution

Lower Limit	Upper Limit	Obs. No.	Observed Frequency	Red. Var. yi	F(xi)	Expected Frequency	Chi Sq.
70	130	13	0.21	-0.38	0.23	0.23	0.17
130	195	29	0.46	0.70	0.61	0.38	1.20
195	260	13	0.21	1.77	0.84	0.24	0.24
260	325	4	0.06	2.85	0.94	0.10	0.84
325	390	2	0.03	3.92	0.98	0.04	0.04
390	455	1	0.02	5.00	0.99	0.01	0.04
455	520	1	0.02	6.08	1.00	0.00	1.87
		63					4.41

Table 4.5: Chi Sq. Value from Annual Series of Maximum Value of 2 day rainfall (mm) for Log Pearson Type III (LP3) Distribution

Lower Limit	Upper Limit	Obs. No.	Observed Frequency	K_T	F(xi)	Expected Frequency	Chi Sq.
1.85	2.11	13	0.21	-0.78	0.23	0.24	0.32
2.11	2.29	29	0.46	0.29	0.61	0.37	1.32
2.29	2.41	13	0.21	1.05	0.84	0.23	0.20
2.41	2.51	4	0.06	1.64	0.94	0.09	0.56
2.51	2.59	2	0.03	2.12	0.98	0.04	0.07
2.59	2.66	1	0.02	2.53	0.99	0.01	0.03
2.66	2.72	1	0.02	2.89	1.00	0.01	1.02
		63					3.52

Table 4.6: Chi Sq. Value from Annual Series of Maximum Value of 2 day rainfall (mm) for Log Normal Distribution

Lower Limit	Upper Limit	Obs. No.	Observed Frequency	Z	F(xi)	Expected Frequency	Chi Sq.
1.85	2.11	13	0.21	-0.78	0.22	0.22	0.05
2.11	2.29	29	0.46	0.29	0.61	0.39	0.79
2.29	2.41	13	0.21	1.05	0.85	0.24	0.27
2.41	2.51	4	0.06	1.64	0.95	0.10	0.76
2.51	2.59	2	0.03	2.12	0.98	0.03	0.02
2.59	2.66	1	0.02	2.53	0.99	0.01	0.07
2.66	2.72	1	0.02	2.89	1.00	0.00	2.13
		63					4.08

From the analysis, lowest χ^2 value is found for Log Pearson Type III (LP3) Distribution. As annual maximum value of 2-day rainfall data series fits the LP3 distribution, LP3 distribution is used in the present study for computing various rainfall values for different return periods.

Annual maximum values of rainfall (mm) for various return periods (i.e. T = 5, 10, 25, 50 and 100) are computed by LP3 distribution and presented in the following tables.

Table 4.7: Annual Maximum Value of 2-Day Rainfall (mm) for Various Return Periods Computed by LP3 Distribution

T	y	s_y	K_T	y_t	X_T	X_T (mm), rounded
5	2.24	0.16	0.83	2.38	239.02	240
10	2.24	0.16	1.30	2.46	285.72	286
25	2.24	0.16	1.82	2.54	348.12	349
50	2.24	0.16	2.17	2.60	396.53	397
100	2.24	0.16	2.49	2.65	447.01	448

Table 4.8: Annual Maximum Value of Daily Rainfall (mm) for Various Return Periods Computed by LP3 Distribution

T	y	s_y	K_T	y_t	X_T	X_T (mm), rounded
5	2.11	0.17	0.81	2.24	174.91	178
10	2.11	0.17	1.32	2.33	213.65	214
25	2.11	0.17	1.92	2.43	268.43	269
50	2.11	0.17	2.32	2.50	314.25	315
100	2.11	0.17	2.55	2.54	342.78	343

Table 4.9: Annual Maximum Value of 3-hr Rainfall (mm) for Various Return Periods Computed by LP3 Distribution

T	y	s_y	K_T	y_t	X_T	X_T (mm), rounded
5	1.82	0.10	0.85	1.91	81.06	82
10	1.82	0.10	1.25	1.95	88.77	89
25	1.82	0.10	1.63	1.99	97.08	98
50	1.82	0.10	1.87	2.01	102.60	103
100	1.82	0.10	2.10	2.03	108.18	109

Table 4.10: Annual Maximum Value of 6-hr Rainfall (mm) for Various Return Periods Computed by LP3 Distribution

T	y	s_y	K_T	y_t	X_T	X_T (mm), rounded
5	1.95	0.15	0.85	2.08	121.01	122
10	1.95	0.15	1.27	2.15	140.55	141
25	1.95	0.15	1.72	2.22	164.59	165
50	1.95	0.15	2.01	2.26	182.03	183
100	1.95	0.15	2.26	2.30	199.09	200

Table 4.11: Annual Maximum Value of 9-hr Rainfall (mm) for Various Return Periods Computed by LP3 Distribution

T	y	s_y	K_T	y_t	X_T	X_T (mm), rounded
5	2.00	0.18	0.85	2.16	142.89	143
10	2.00	0.18	1.27	2.23	170.84	171
25	2.00	0.18	1.71	2.31	206.44	207
50	2.00	0.18	1.99	2.37	232.19	233
100	2.00	0.18	2.25	2.41	258.62	259

Table 4.12: Annual Maximum Value of 12-hr Rainfall (mm) for Various Return Periods Computed by LP3 Distribution

T	y	s_y	K_T	y_t	X_T	X_T (mm), rounded
5	2.03	0.19	0.84	2.19	153.89	154
10	2.03	0.19	1.28	2.27	186.43	187
25	2.03	0.19	1.76	2.36	228.60	229
50	2.03	0.19	2.06	2.42	260.75	261
100	2.03	0.19	2.33	2.47	293.23	294

Analysis of Water Level Data

A map of hydrologic network of surface water (i.e. river) in Bangladesh was jointly formulated by Bangladesh Water Development Board (BWDB), United Nations Development Programme (UNDP) and United Nations Department for Development Support and Management Services 56 (UN-DDSMS). In the map, the study area is located in the Dhaka Hydrologic Sub-Division under North-Eastern Measurement Division of BWDB. Direct runoff from the study area eventually flows out to Balu River. There are one tidal water level station (i.e. Pubail, SW7), and one tidal water level and discharge station (i.e. Demra, SW7.5). Balu is a tidal river though the tidal range is not significant. Annual average and maximum values of daily tidal range along with maximum and minimum water levels for Pubail and Demra stations are presented in Table 4.13 and Table 4.14.

Table 4.13: Annual Maximum WL for High Tide & Minimum WL for Low Tide (Pubail, SW7)

Year	Date of occurrence	Max water level for High Tide, mPWD	Date of occurrence	Min water level for Low Tide, mPWD	Annual Average of Daily Tidal Range, mPWD	Annual Maximum tidal range, mPWD
1945	29-Aug-45	5.82	-	-	-	-
1947	13-Aug-47	5.53	16-Feb-47	0.55	0.03	0.21
1948	12-Aug-48	5.82	06-Mar-48	0.49	0.02	0.18
1950	06-Sep-50	5.43	13-Feb-50	0.61	0.05	0.28
1952	31-Dec-52	5.41	21-Mar-52	0.27	0.04	0.24
1953	19-Jan-53	1.37	10-Feb-53	0.61	0.11	0.37
1954	07-Sep-54	6.65	23-Dec-54	1.10	0.02	0.19
1956	14-Jul-56	6.31	09-Feb-56	1.42	0.06	0.34
1957	31-Dec-57	1.23	26-Mar-57	0.99	0.08	0.33
1958	07-Sep-58	6.54	01-Feb-58	0.84	0.07	0.45
1959	27-Aug-59	6.08	20-Mar-59	1.02	0.05	0.30
1960	29-Sep-60	6.17	06-Apr-60	0.93	0.06	0.46
1961	07-Sep-61	5.70	12-Feb-61	0.99	0.07	0.28
1962	07-Sep-62	6.92	14-Apr-62	1.52	0.05	0.42
1963	10-Sep-63	6.28	20-Feb-63	1.10	0.09	0.43
1964	02-Jan-64	2.04	10-Mar-64	1.01	0.16	0.21
1968	06-Aug-68	6.45	06-Aug-68	6.45	0.05	0.41
1969	06-Sep-69	6.19	28-Feb-69	1.04	0.08	0.34
1970	13-Aug-70	6.74	05-Mar-70	1.08	0.08	0.44
1971	08-Sep-71	6.42	09-Mar-71	0.99	0.06	0.31
1972	16-Aug-72	5.64	11-Feb-72	0.99	0.09	0.45
1973	22-Aug-73	6.25	29-Jan-73	1.01	0.08	0.40
1974	19-Aug-74	6.95	18-Feb-74	1.09	0.07	0.31
1975	13-Aug-75	5.73	20-Feb-75	1.09	0.09	0.15
1976	17-Jul-76	5.64	11-Feb-76	1.08	0.09	0.40

Year	Date of occurrence	Max water level for High Tide, mPWD	Date of occurrence	Min water level for Low Tide, mPWD	Annual Average of Daily Tidal Range, mPWD	Annual Maximum tidal range, mPWD
1977	05-Sep-77	6.03	30-Jan-77	0.99	0.07	0.34
1978	01-Jan-78	1.90	19-Feb-78	0.93	0.16	0.32
1980	01-Sep-80	6.66	02-Apr-80	1.37	0.04	0.25
1981	09-Jan-81	1.83	30-Jan-81	1.22	0.09	0.22
1983	28-Sep-83	6.05	07-Apr-83	1.64	0.04	0.42
1984	07-Aug-84	6.35	27-Feb-84	0.75	0.07	0.31
1985	08-Aug-85	5.83	16-Feb-85	1.07	0.06	0.25
1986	09-Oct-86	5.70	21-Feb-86	0.84	0.07	0.27
1987	27-Aug-87	6.90	10-Feb-87	1.05	0.06	0.24
1988	10-Sep-88	7.29	31-Jan-88	1.13	0.06	0.27
1989	30-Jul-89	5.47	30-Mar-89	1.08	0.06	0.24
1990	29-Jul-90	5.60	20-Feb-90	1.03	0.05	0.21
1991	16-Sep-91	6.07	11-Mar-91	1.10	0.07	0.51
1992	22-Jul-92	4.81	27-Feb-92	1.10	0.06	0.26
1994	25-Aug-94	5.22	23-Jan-94	1.09	0.06	0.38
1995	20-Jul-95	6.27	11-Mar-95	0.89	0.07	0.27
1996	29-Jul-96	6.15	01-Feb-96	1.04	0.06	0.32
1997	23-Jul-97	5.74	05-Mar-97	1.17	0.06	0.32
1998	12-Sep-98	7.32	09-Mar-98	1.02	0.07	0.28
1999	31-May-99	2.95	31-Dec-01	1.71	0.10	0.28
2002	07-Aug-02	6.05	23-Feb-02	1.16	0.06	0.25
2003	21-Jul-03	5.81	09-Mar-03	5.18	0.05	0.24
2004	17-Oct-04	5.16	03-Feb-04	0.75	0.09	0.26
2005	01-Aug-05	5.45	05-Feb-05	1.07	0.07	0.30
2006	24-Sep-06	4.86	26-Jan-06	0.86	0.10	0.39
2007	31-Jul-07	5.70	14-Mar-07	0.76	0.18	0.43
2008	06-Jul-08	8.12	31-Dec-08	1.76	0.05	1.06
2009	28-Aug-09	5.43	07-Feb-09	1.05	0.10	0.32
2010	28-Sep-10	5.38	08-Feb-10	1.00	0.09	0.45
2011	19-Aug-11	5.78	01-Mar-11	1.18	0.10	0.35
2012	27-Jul-12	4.70	18-Mar-12	0.77	0.10	0.75
2013	13-Sep-13	4.58	23-Feb-13	0.45	0.12	0.43
2014	31-Jul-14	4.11	10-Mar-14	0.52	0.90	0.49

Table 4.14: Annual Maximum WL for High Tide & Minimum WL for Low Tide (Demra, SW7.5)

Year	Annual High Water Level, m PWD	Date of occurrence	Annual Low Water Level, m PWD	Date of Occurrence	Annual Average of Daily Tidal Range, mPWD	Annual Maximum tidal range, mPWD
1962	6.28	02-Sep-62	1.25	14-Apr-62	0.20	0.58
1963	5.91	06-Sep-63	1.16	06-Apr-63	0.19	0.61
1964	6.39	12-Aug-64	0.85	10-Mar-64	0.12	0.67
1965	5.80	25-Aug-65	1.04	03-Mar-65	0.00	0.00
1966	1.49	01-Jan-66	0.92	21-Feb-66	0.00	0.00
1967	5.42	28-Jul-67	1.14	08-Apr-67	0.00	0.00
1968	6.07	01-Aug-68	1.01	18-Jan-68	0.14	0.61
1969	5.84	30-Aug-69	0.73	27-Feb-69	0.28	0.71
1970	6.23	08-Aug-70	0.81	16-Feb-70	0.23	0.70
1971	6.02	29-Aug-71	1.40	07-Apr-71	0.17	0.70
1972	5.39	08-Aug-72	0.79	11-Feb-72	0.27	0.61
1973	5.87	19-Aug-73	0.82	29-Jan-73	0.25	0.67
1974	6.57	11-Aug-74	0.88	18-Feb-74	0.24	0.64
1975	5.59	10-Aug-75	0.97	07-Feb-75	0.28	0.82
1976	5.46	14-Jul-76	0.83	25-Feb-76	0.29	0.67
1977	5.91	02-Sep-77	0.81	31-Jan-77	0.26	0.68
1978	2.06	11-Jan-78	0.79	19-Mar-78	0.50	0.70
1979	5.58	08-Aug-79	1.04	08-Apr-79	0.26	0.64
1980	6.22	26-Aug-80	0.82	12-Feb-80	0.29	0.76
1981	5.73	09-Aug-81	0.95	16-Mar-81	0.32	0.71
1982	5.99	11-Sep-82	0.76	19-Feb-82	0.30	1.02
1983	5.89	26-Sep-83	0.34	13-Mar-83	0.22	1.45
1984	6.32	27-Sep-84	0.93	27-Feb-84	0.21	0.66
1985	5.69	04-Aug-85	1.00	16-Feb-85	0.31	0.80
1986	5.24	09-Aug-86	0.67	18-Feb-86	0.32	0.78
1987	6.45	22-Aug-87	0.85	10-Mar-87	0.30	0.68
1988	7.09	06-Sep-88	0.89	29-Jan-88	0.30	0.74
1989	5.43	24-Jul-89	0.99	04-Mar-89	0.32	0.75
1990	5.44	09-Oct-90	0.89	20-Feb-90	0.31	0.68
1991	5.90	15-Sep-91	0.84	11-Mar-91	0.29	0.89
1992	4.69	20-Jul-92	0.92	27-Feb-92	0.34	0.70
1993	5.85	03-Sep-93	0.92	03-Feb-93	0.32	0.79
1994	5.14	24-Aug-94	0.90	23-Jan-94	0.36	0.72
1995	6.17	16-Jul-95	0.72	27-Jan-95	0.32	0.74
1996	5.98	26-Jul-96	0.81	01-Feb-96	0.30	0.80
1997	5.71	22-Jul-97	0.96	19-Feb-97	0.32	0.73
1998	6.97	15-Sep-98	0.90	10-Mar-98	0.28	1.15
1999	2.49	27-Dec-99	1.06	12-Jan-99	0.44	0.62
2000	5.85	14-Aug-00	1.07	15-Feb-00	0.22	0.62

Year	Annual High Water Level, m PWD	Date of occurrence	Annual Low Water Level, m PWD	Date of Occurrence	Annual Average of Daily Tidal Range, mPWD	Annual Maximum tidal range, mPWD
2001	5.52	09-Sep-01	1.06	05-Mar-01	0.28	0.68
2002	6.00	03-Aug-02	1.02	08-Mar-02	0.27	0.70
2003	5.99	18-Jul-03	0.95	25-Feb-03	0.26	0.73
2004	6.98	27-Jul-04	1.00	01-Feb-04	0.23	0.63
2005	5.98	25-Jul-05	1.17	05-Feb-05	0.28	0.67
2006	5.02	23-Sep-06	0.85	09-Feb-06	0.31	0.86
2007	6.27	06-Aug-07	0.84	14-Mar-07	0.28	0.89
2008	5.93	04-Sep-08	1.10	03-Feb-08	0.27	0.75
2009	5.50	27-Aug-09	1.00	19-Feb-09	0.28	0.60
2010	5.60	14-Sep-10	0.83	11-Feb-10	0.27	0.82
2011	5.74	20-Aug-11	1.07	01-Mar-11	0.31	0.72
2012	5.30	24-Jul-12	1.27	02-Feb-12	0.28	0.57
2013	4.58	13-Sep-13	0.45	23-Feb-13	0.33	0.73
2014	4.11	16-Jul-14	0.52	10-Mar-14	0.38	0.73

Analytical Frequency Analysis

Frequency analysis of annual maximum values of high tide water level (i.e. annual high water levels) at Pubail and Demra Stations are done by using Gumbel distribution (i.e. Extreme Value Type I distribution) to determine the high waters levels for various return periods as shown in Table 4.15.

Table 4.15: Estimated High Water Levels for Different Return Periods at Pubail and Demra Stations

Return Period, T (Years)	High Water Level (m PWD)	
	Pubail Station (SW7)	Demra Station (SW7.5)
5	6.40	6.21
10	6.85	6.51
25	7.41	6.88
50	7.82	7.16
100	8.24	7.43

In addition, frequency analysis of 80 and 90 percentile values of annual high water levels during rainy season (June – October) at Pubail and Demra stations are also done by using Gumbel distribution for various return periods. The idea is a more realistic assessment of design stormwater runoff from the study area for 80 and 90 percentile values of high water levels on Balu River. Results of frequency analysis of 80 and 90 percentile values of annual high water levels at Pubail and Demra stations are as shown in Tables 4.16 and 4.17.

Table 4.16: Estimated 80 Percentile values of High Water Levels for Different Return Periods at Pubail and Demra Stations

Return Period, T (Years)	80 Percentile Values of High Water Level (m PWD)	
	Pubail Station (SW7)	Demra Station (SW7.5)
5	5.92	5.63
10	6.29	5.87
25	6.75	6.18
50	7.09	6.40
100	7.43	6.62

Table 4.17: Estimated 90 Percentile values of High Water Levels for Different Return Periods at Pubail and Demra Stations

Return Period, T (Years)	90 Percentile Values of High Water Level (m PWD)	
	Pubail Station (SW7)	Demra Station (SW7.5)
5	6.25	5.92
10	6.56	6.19
25	6.95	6.54
50	7.23	6.79
100	7.52	7.05

4.6 Model Development

The stream network of the study area is given in the model as Conduit layer. This is a polyline feature class layer that represents the model conduits. The conduits here are open channels, for which the cross-sections are given as irregular transects.

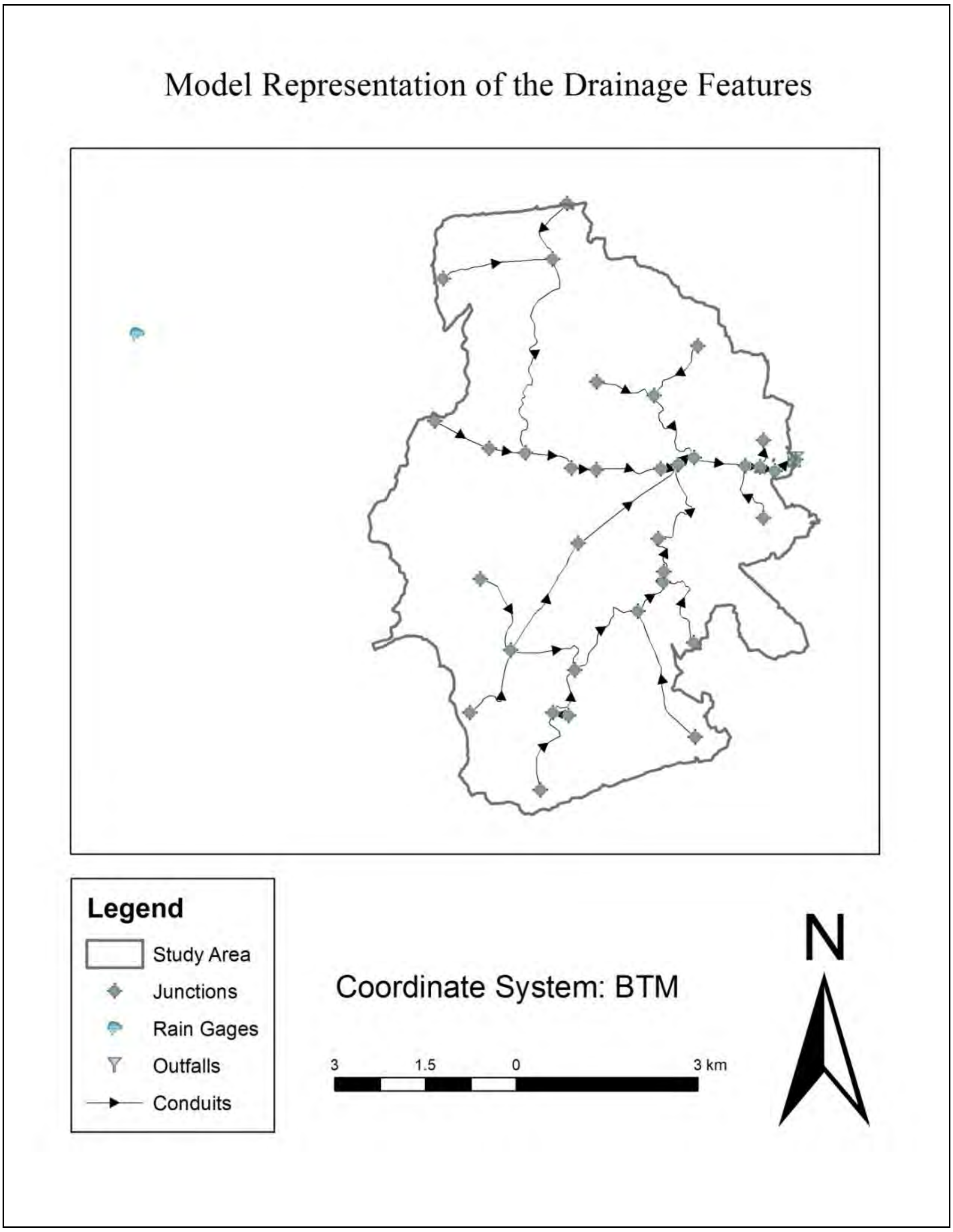


Figure 4.17: Drainage Features in the Model

Catchment Delineation

The "Watershed Delineation" tool in GeoSWMM performs GIS based topographic analysis to define the subcatchment boundaries in a SWMM project. Using DEM information of the project area, the tool enables to analyze the model topography and identify the stream layout and to delineate subcatchment boundaries at given locations (points of interest). Subcatchment properties (e.g. area, shape, surface slope, imperviousness, outlet location etc.) play quite significant roles during any hydrologic event and therefore their accurate representation is essential to the SWMM model. The Watershed Delineation tool helps to efficiently create and manage the model subcatchments and associated streams and outlets. The tool provides the utmost flexibility in assigning desired outlets (or pour points) and generating their contributory watershed areas in a GeoSWMM project.

In hydrology, a watershed is an upslope drainage area that contributes water flow to a common outlet during a hydrologic event. The contributory drainage area is also referred as the catchment or the drainage basin. The outlet (or the water pour point) is the point on surface at which water flows out of a catchment area. It is the point with lowest elevation along the catchment boundary.

A catchment may contain smaller watersheds, called subcatchments or subwatersheds. The outlet's points define the shape and spatial extent of the subcatchment boundaries. The number of the subcatchments is equal to the number of outlets. The stream network that conveys water from a number of subcatchments to a common outlet looks like the veins of a leaf, with the position of the outlet at the end of the midrib (Fig. 4.18).

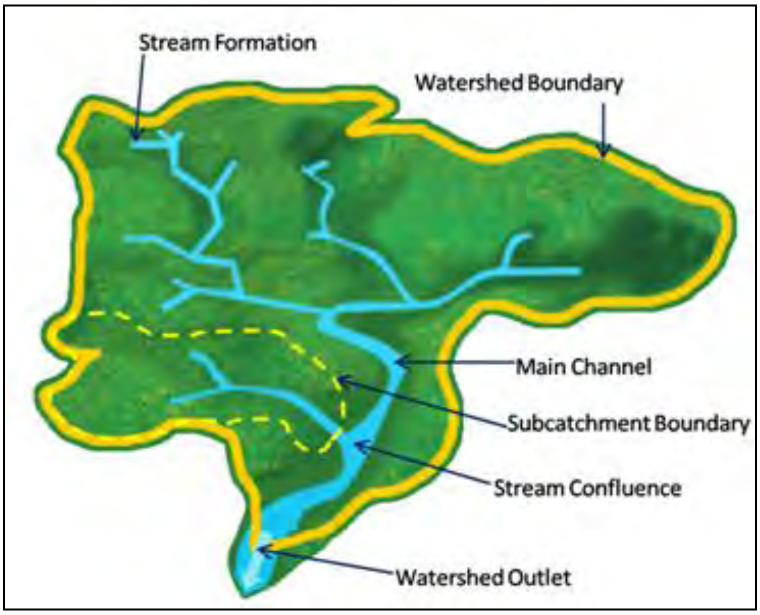


Figure 4.18: A Schematic Diagram of Watershed Components

A watershed segmentation process, commonly referred as the watershed delineation, generates the subcatchments for a set of outlets within a drainage basin. It is an essential task for an accurate representation of the hydrologic and hydraulic components in a SWMM model. The Watershed Delineation Tool is simple and interactive tool to aid users automatically delineate subcatchment boundaries. The Watershed Delineation Tool in GeoSWMM primarily models the flow of water across an area of interest. The terrain of the area determines the extent of drainage area and the direction of flow during a hydrologic event. In GIS, the terrain data are widely represented by a Digital Elevation Model (DEM).

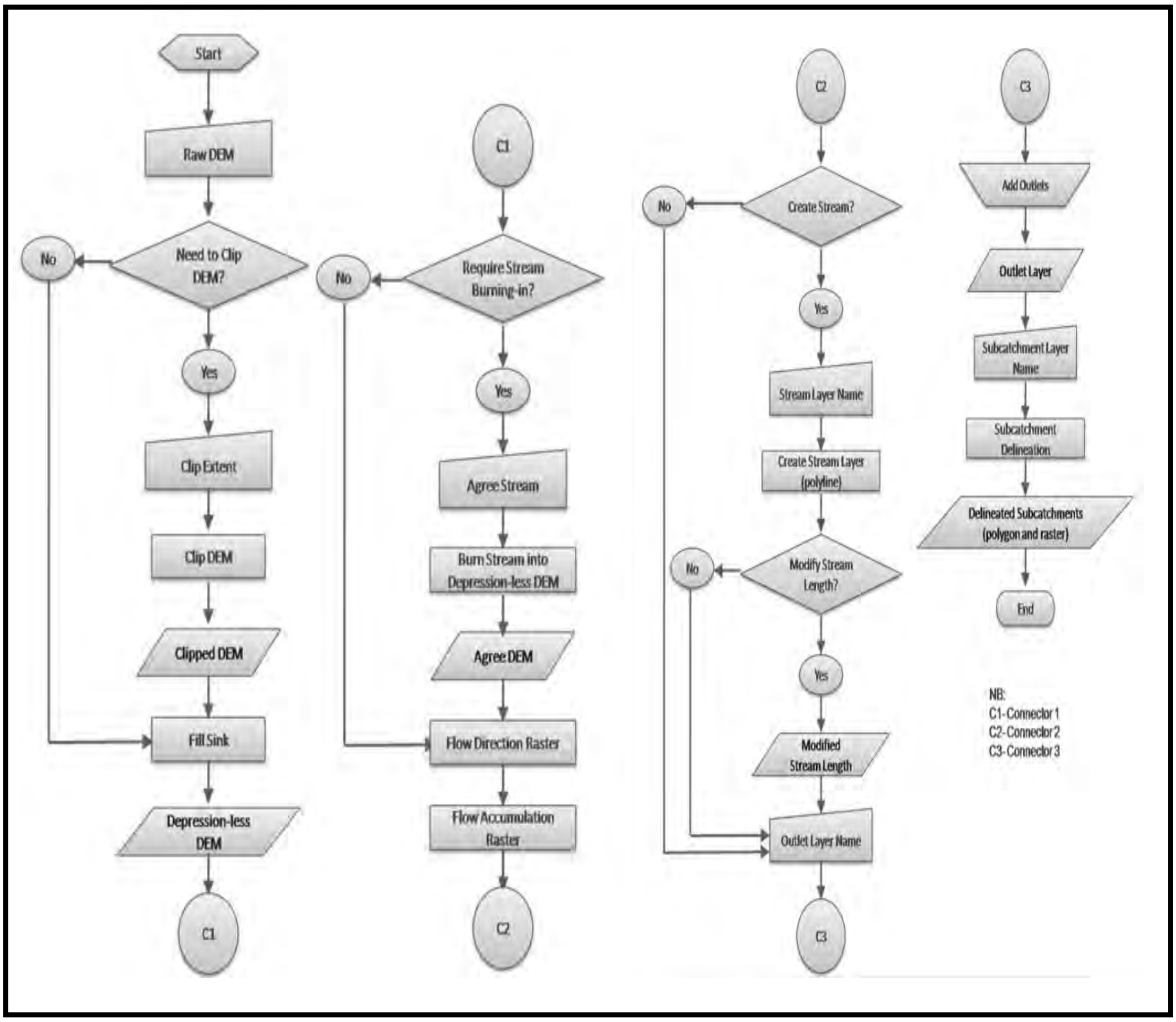


Figure 4.19: Watershed Delineation Process Flow Chart (Source: GeoSWMM user manual)

The Watershed Delineation Tool works with user supplied DEM data (in raster or grid format) to segment the model catchment boundary into multiple subcatchments. This process is known as automatic delineation. The accuracy of catchment delineation, therefore, depends on the resolution (e.g. grid cell size) of the DEM data. The catchment delineation process involves a set of successive spatial operations in ArcGIS that require a clear understanding. The computation techniques applied by the Watershed Delineation Tool are similar to that of the Hydrology toolset of ArcGIS 10.2. The basic operations undertaken by the tool are illustrated in a flow chart as shown in the following Fig. 4.19.

There are some hydrologic and GIS terms used in the above flow chart such as sink, stream burning-in, AGREE stream, AGREE DEM, flow direction raster, and flow accumulation raster. The raw DEM input supplied by the user represents the continuous surface of the earth in the study area. These raw DEMs often contain sinks, the cells (or sets of cells) surrounded by higher elevation values, which may cause imperfections in delineation results. Sinks are also referred as depressions or pits and need to be handled in a proper way before performing subcatchment delineation. The Fill Sink operation eliminates the existing anomalies in a DEM dataset.

The tool analyzes and processes DEM data applying an 8-pour point approach in order to determine the direction of flow. The Flow Direction Raster generated in the interim period contains the values that represent the direction of water movement. However, in some terrain data users may need to define flow paths exactly along the existing stream layout. In such case, the "burning-in" technique is applied with the available stream layer in the DEM data. The user specified vector stream layer is called AGREE stream and the resulting DEM is called AGREE DEM. AGREE is a surface reconditioning system for DEMs (Hellweger, 1997). The system adjusts the surface elevation of the DEM to be consistent with the vector stream (AGREE stream). The burning-in technique allows the user to abruptly lower the cell elevations by a fixed amount along the stream line in the target DEM. This process is also known as DEM reconditioning. If the flow direction raster is produced using the AGREE DEM then the flow path automatically becomes aligned with the AGREE stream layer.

The flow accumulation raster is processed from the flow direction raster, which is subsequently used to define the stream network and subcatchment boundaries. The term ―flow accumulation" for a given cell is defined as the number of upstream cells draining to that cell. This tool allows users to develop a stream network satisfying a minimum threshold of the flow accumulation value and to adjust the lengths of different segments of the network.

Outlets are the basis of watershed segmentation and they are usually specified at the stream confluences, gauge locations, hydraulic structure locations, and at any other points of interest. However, the stream formation is not essential for catchment delineation unless the user wants to specify the stream confluences as the subcatchment outlets.

In this work, the canal network is previously known. So, stream formation is not a necessary step. The stream is burnt in the DEM so that all the canals are included in the operation. The generated outlets are minimized to optimize the best use of available data. These are incorporated in the model as ‗junction', which is showed in Fig. 4.17. Finally the catchments are delineated on the basis of these junctions. The delineated subcatchments are presented in Fig. 4.20.

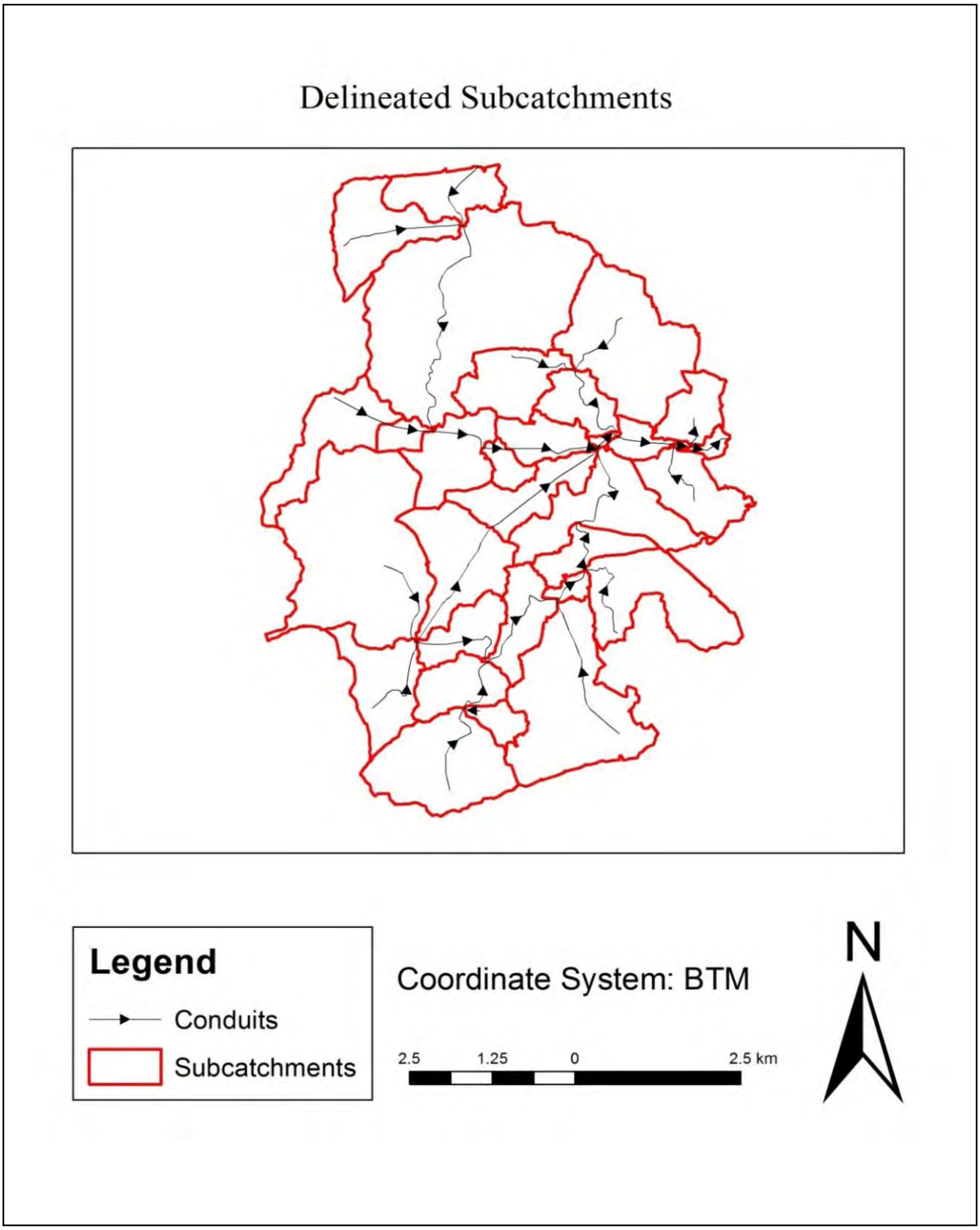


Figure 4.20: Delineated Subcatchments of the Study Area

Subcatchment parameterization

To varying degrees it is possible to have the results of GeoSWMM's runoff computations approximate those obtained from other well known methods such as *Runoff Coefficient Method.* This method is sometimes used in preliminary screening-level models to generate runoff flows from long-term rainfall records. It computes runoff Q after all depression storage has been filled as:

$$Q = CiA \tag{4.13}$$

where C is a runoff coefficient, i is the rainfall rate, and A is the subcatchment area. GeoSWMM returns runoff coefficient for each subcatchment after simulation using the Manning Equation. So, desired value of runoff coefficient can be achieved by adjusting the subcatchment parameters.

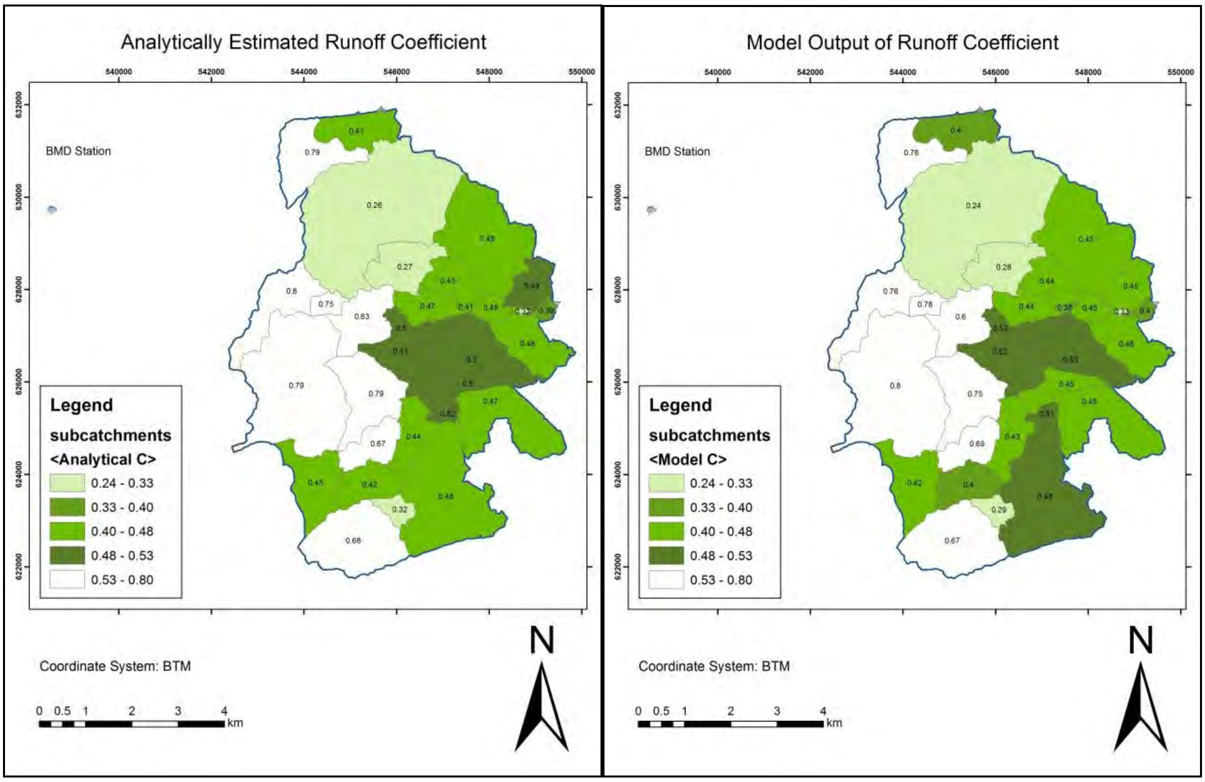


Figure 4.21: Analytical estimation and model output of runoff coefficient of Subcatchments

In this work, the model has been simulated from March to September of 2016. The model returns a C-value for each Subcatchment. The initial subcatchment parameterization has been done by comparing the runoff coefficient of the model output with the analytically estimated runoff coefficient value of each catchment according to landuse (Figure 4.21). The parameters are described here.

Slope Calculation

The "Slope Calculation" tool works with a Digital Elevation Model (DEM) dataset to find the average surface slope of the model subcatchments. The values are calculated in percentage rise. The tool requires a DEM raster and the subject subcatchment layer as input. In addition, an attribute column name needs to be specified where the tool will preserve the calculated slope value for each subcatchment. Users may assign a valid field name from the available columns in a drop-down list or insert a new field name. The characteristics of the required input features in the Slope Calculation tool are described below.

DEM Raster: This is a raster dataset that contains elevation values of the model subcatchment area. Subcatchment Layer: This is a polygon feature layer that represents the subcatchments.Subcatchment Slope Field: This is an attribute field available in the subcatchment layer in which the slope calculation tool will store the calculated slope values. If a new field name is assigned, then it will be added in the attribute table of the subcatchment layer. Thus the average slope is calculated (Figure 4.22) using GIS tool based on the DEM.

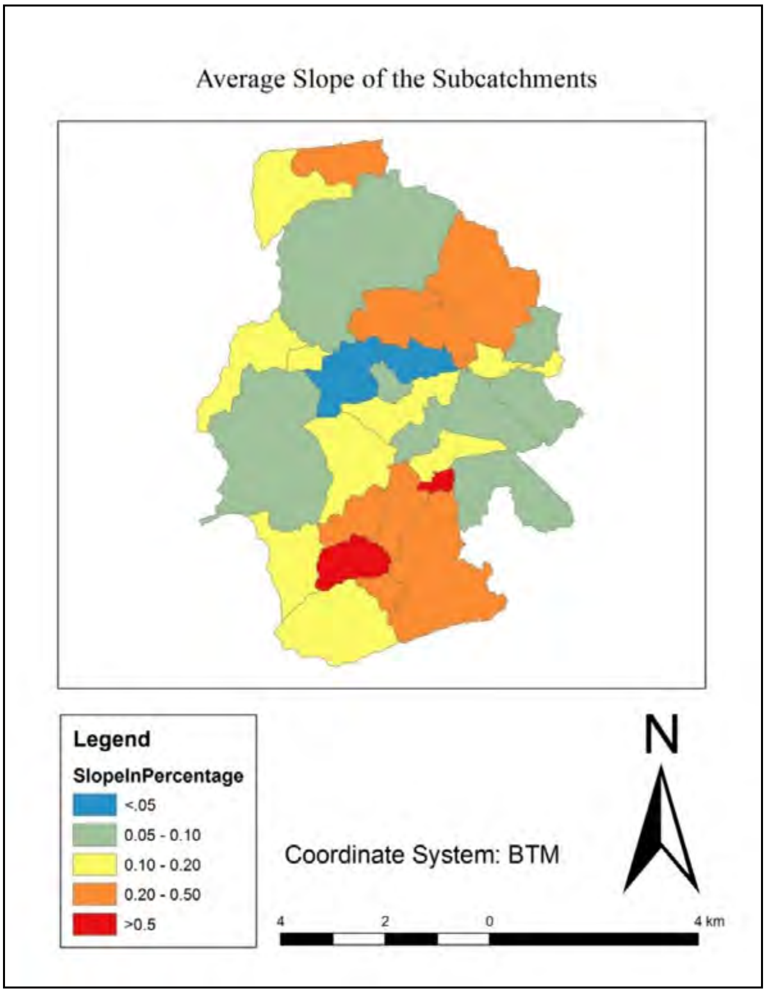


Figure 4.22: Average Slope of the Subcatchments

Imperviousness Calculation

The imperviousness parameter describes the percentage of impervious surfaces in relation to the total area of a subcatchment. It is often used as a calibration parameter (Choi and Ball, 2002) as it is not quite straightforward to physically define, due to e.g. the fact that many surfaces are in reality partially impervious. In this study, landuse map and google earth images were used for subcatchment parameterization.

The "Imperviousness Calculation" tool works with a grid or a polygon feature layer datasetthat represents surface imperviousness in order to find the area-weighted average surface imperviousness of the model subcatchments. In an imperviousness raster, each cell value represents the fraction (or the percentage) of impervious area within that cell. The tool requires a dataset that represents imperviousness (raster or polygon feature layer) and the subject subcatchment layer as inputs. If the imperviousness dataset is a polygon feature layer, then the attribute field containing the imperviousness value needs to be specified by the user during calculation. In addition, an attribute field name needs to be specified where the tool will preserve the calculated imperviousness value for each subcatchment. Users may assign a valid field name from the available columns in a drop-down list or insert a new field name.

The characteristics of the required input features in the Imperviousness Calculation tool are:

Imperviousness Layer: This is either a raster or a polygon feature dataset which contains imperviousness values of the model subcatchment area.

Imperviousness Field Name: This is an attribute field available in the imperviousness feature layer which contains the imperviousness values of the model subcatchment areas. It is not applicable if the imperviousness dataset is a raster.

Subcatchment Layer: This is a polygon feature layer that represents the model subcatchments.

Subcatchment Imperviousness Field: This is an attribute field available in the subcatchment layer in which the slope calculation tool will store the calculated imperviousness values. If a new field name is assigned, then it will be added in the attribute table of the subcatchment layer.

Figure 4.23 presents the imperviousness values of the Subcatchments. It has been observed that imperviousness is one of the most sensitive parameters that control the discharge and water level. So, the values are checked with google earth images in background. We have come to know that, in the study area the west part is developed while the eastern side is mostly rural. The figure is correctly representing higher imperviousness in the west and lower imperviousness values in the east side.

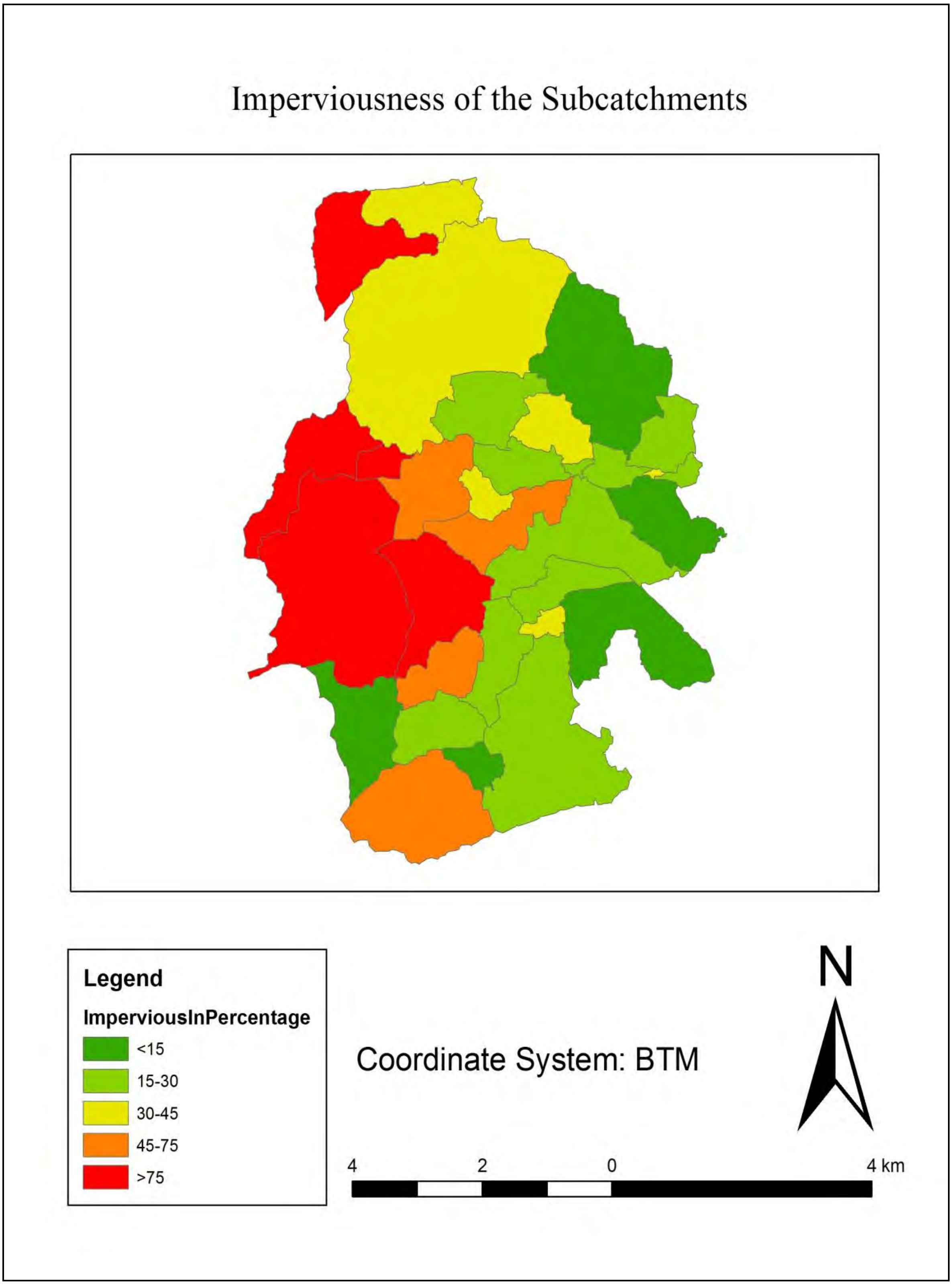


Figure 4.23: Subcatchment imperviousness values used in model

Flow Width Calculation

The "Subcatchment Characteristic Width Calculation" tool helps the GeoSWMM users estimate the characteristic width property for the model subcatchments. It primarily works with a subcatchment layer and a conduit layer (or stream layer) to measure the characteristic width. However, this is a calibration parameter and the tool only provides the best approximation of width value based on the user inputs. The tool stores the calculated widths in an attribute field of the subcatchment layer.

The characteristic width is a special hydrologic parameter for a subcatchment that affects the time of concentration and determines the shape of the runoff hydrograph. It depends on the characteristics of the overland flow length and the shape of the subcatchment. If the overland flow line is represented by the physical width of an ideal rectangular shaped subcatchment, then the characteristic width will be the physical length of that rectangle (Fig. 4.24). However, in reality, subcatchments are shaped like non-uniform polygons and therefore the width cannot be estimated in a straightforward manner (Fig. 4.25). It is one of the key calibration parameters in a SWMM model. Very typically it is used as a calibration parameter (Park et al., 2008; Gironás et al., 2009), although there are ways to deduce an initial estimate even without calibration. According to Rossman (2010) and Gironás et al. (2009), the width parameter can be calculated by dividing the subcatchment area by the length of the longest overland flow path in the area. In case several flow paths exist, their maximum lengths should be averaged. Channelized flow should never be included in this flow length, which typically reduces the maximum flow length to less than 200 meters; on urban areas even below that (Gironás et al., 2009).

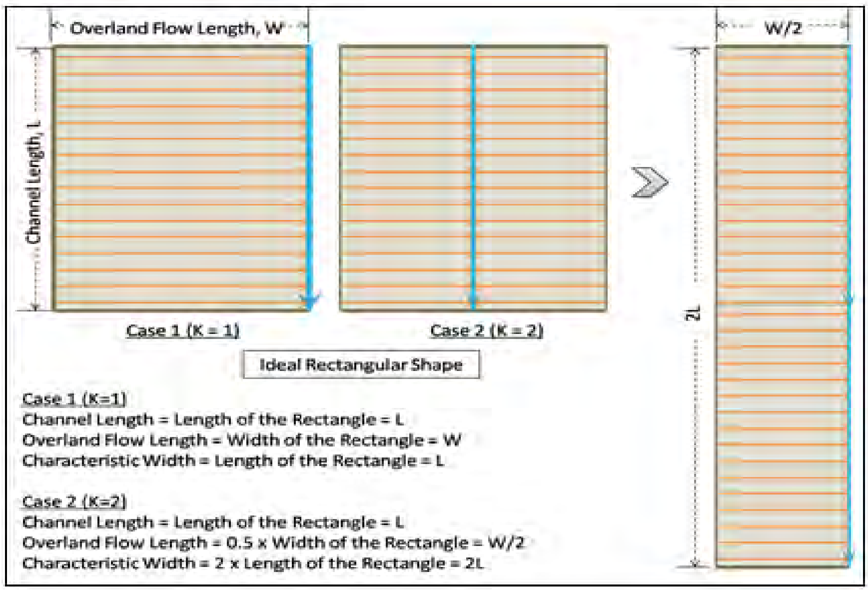


Figure 4.24: Concept of the Characteristic Width of an Ideal Rectangular Shaped Subcatchment

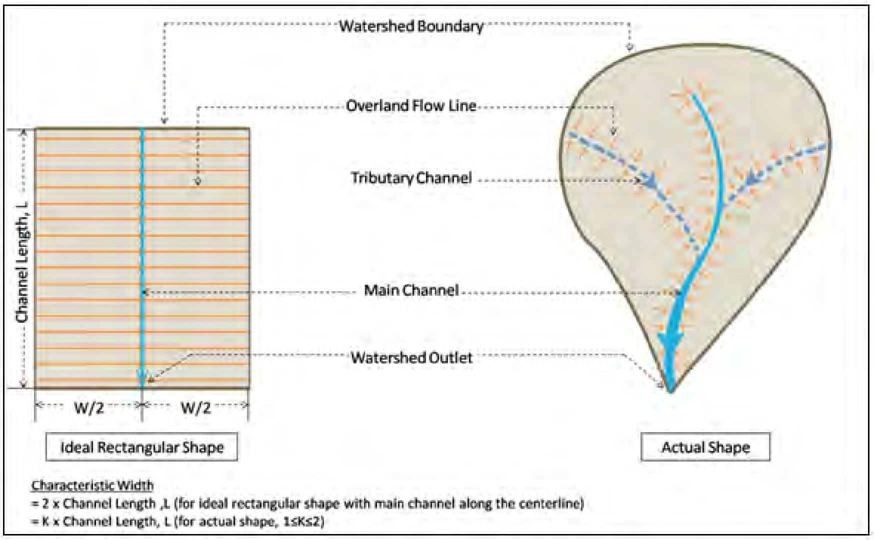


Figure 4.25: Representation of the Characteristic Width for a Non-uniform Subcatchment

So, it is a calibration parameter that theoretically depends on the combination of Subcatchment and its conduit length. In this study this value approximately varies between 300 to 3500 m for different Subcatchments.

Depression storage

SWMM treats the pervious and impervious parts of a subcatchment separately, and thus both may be given independent values of depth of depression storage. One can also define a portion of the impervious area to have no depression storage at all. This could be realistic on steep roofs, for example. The SWMM User‛s Manual (Rossman, 2010) suggests some literature values for the depression storage. For impervious areas, the values range from 1.3 to 2.5 mm, and for lawns from 2.5 to 5.1 mm. The highest value is given for forest litter (7.6 mm). These values are not very exact, and depression storage actually is one of the common calibration parameters used for SWMM parameterization (Choi and Ball, 2002).

Based on the values above, the depression storage for subcatchments were set to around 4-7 mm for impervious subcatchment fraction and to 4-8 mm for the pervious fraction. The percent of impervious area without depression storage was set to zero.

Infiltration

Horton infiltration model is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. Input parameters required by this method include the maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and a time it takes a fully saturated soil to completely dry. Representative values are as follows:

1. DRY soils (with little or no vegetation): Sandy soils: 5 in/hr Loam soils: 3 in/hr Clay soils: 1 in/hr
2. DRY soils (with dense vegetation): Multiply values in A. by 2
3. MOIST soils: Soils which have drained but not dried out (i.e., field capacity): Divide values from A and B by 3. Soils close to saturation: Choose value close to min. infiltration rate. Soils which have partially dried out: Divide values from A and B by 1.5 - 2.5.

Minimum infiltration rate on the Horton curve (in/hr or mm/hr) is equivalent to the soil‘s saturated hydraulic conductivity. Infiltration rate decay constant for the Horton curve (1/hours) has typical values range between 2 and 7. Drying Time is the time in days for a fully saturated soil to dry completely. Typical values range from 2 to 14 days. Maximum infiltration volume can be estimated as the difference between a soil's porosity and its wilting point times the depth of the infiltration zone. The values were set for different subcatchments in this range also with consideration of the landuse pattern in the study area.

Manning's roughness coefficient *n* for overland flow

or impervious areas the roughness coefficient *n* was set around the value of 0.011 to 0.012, which is close to the literature value for smooth asphalt (Rossman, 2010). For non-asphalt surfaces, this was considered a good compromise between smoother materials (e.g. rooftops), and slightly rougher materials (e.g. concrete).

For pervious areas an *n* value was used around 10 to 12 times the values of corresponding impervious area. This was a compromise between the values for short grass (0.15), dense grass (0.24) and woods with light underbrush (0.40) (Rossman, 2010).

4.7 Model Calibration and Validation

Rainfall and Boundary Condition

The boundary location and calibration location are shown in the figure 4.26.

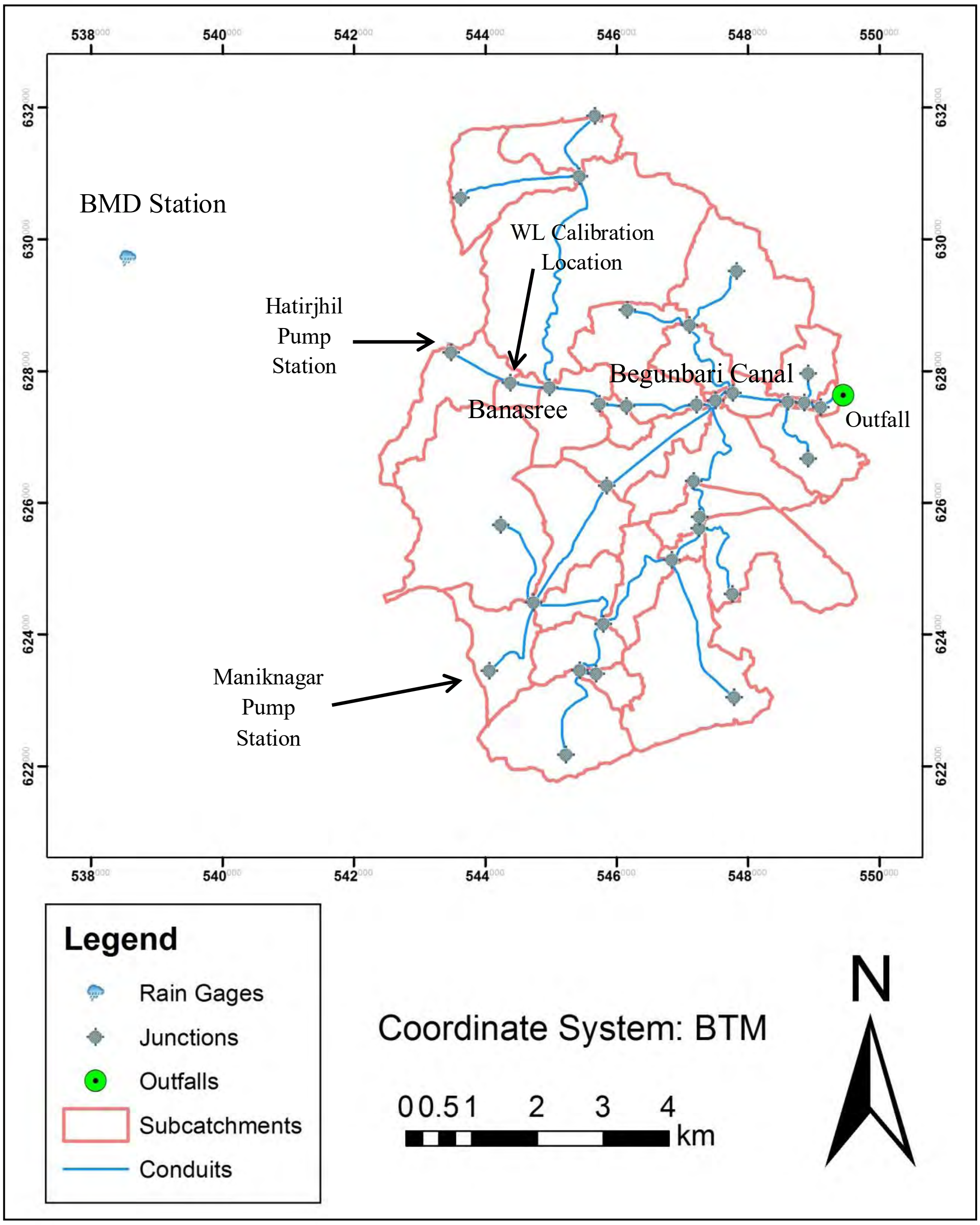


Figure 4.26: Boundary and Calibration Locations

After subcatchment parameterization in the above discussed range, the 3-hourly precipitation data (Figure 4.27) of rain gauge station of Bangladesh Meteorological Department (BMD) was used in this model.

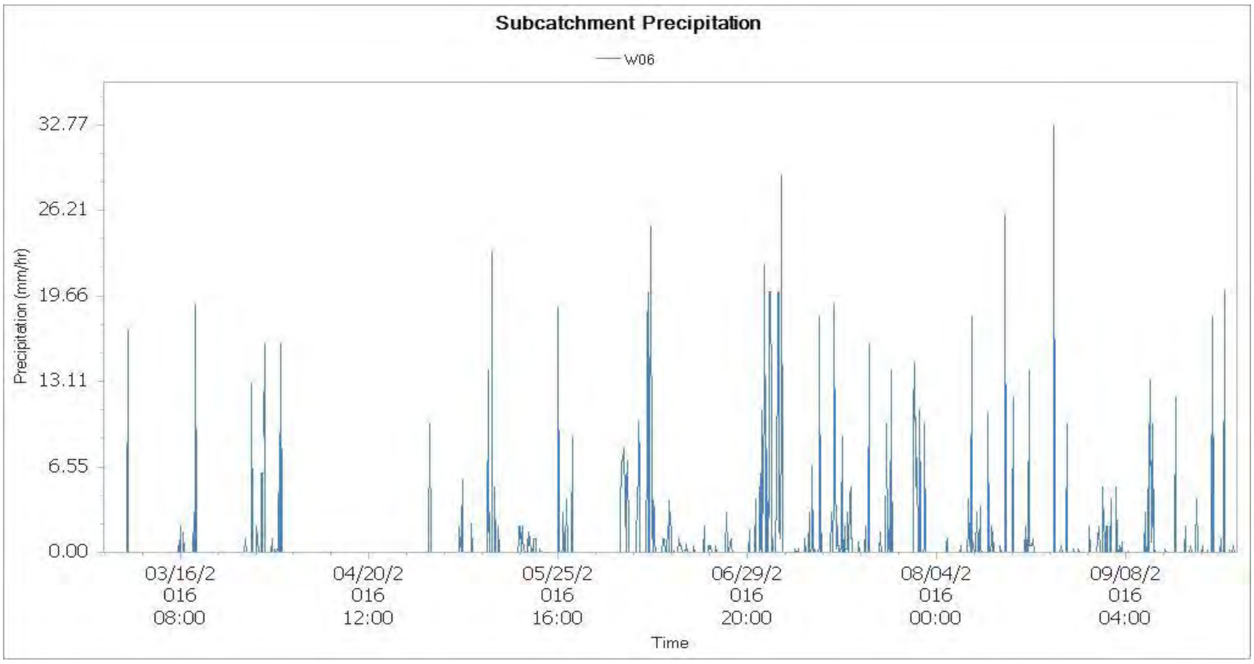


Figure 4.27: Rainfall Data

The time series water level is used as boundary condition (Figure 4.28) for the ends at the outfall of the canal network. There are two pump stations (locations are shown in figure 4.26) at Hatirjhil and Maniknagar from where, the catchment receives pump discharge. From field visits and discussion with concerned officials, it was found as a reasonable idea to give constant discharge of 25 m^3/s and 15 m^3/s respectively in the monsoon.

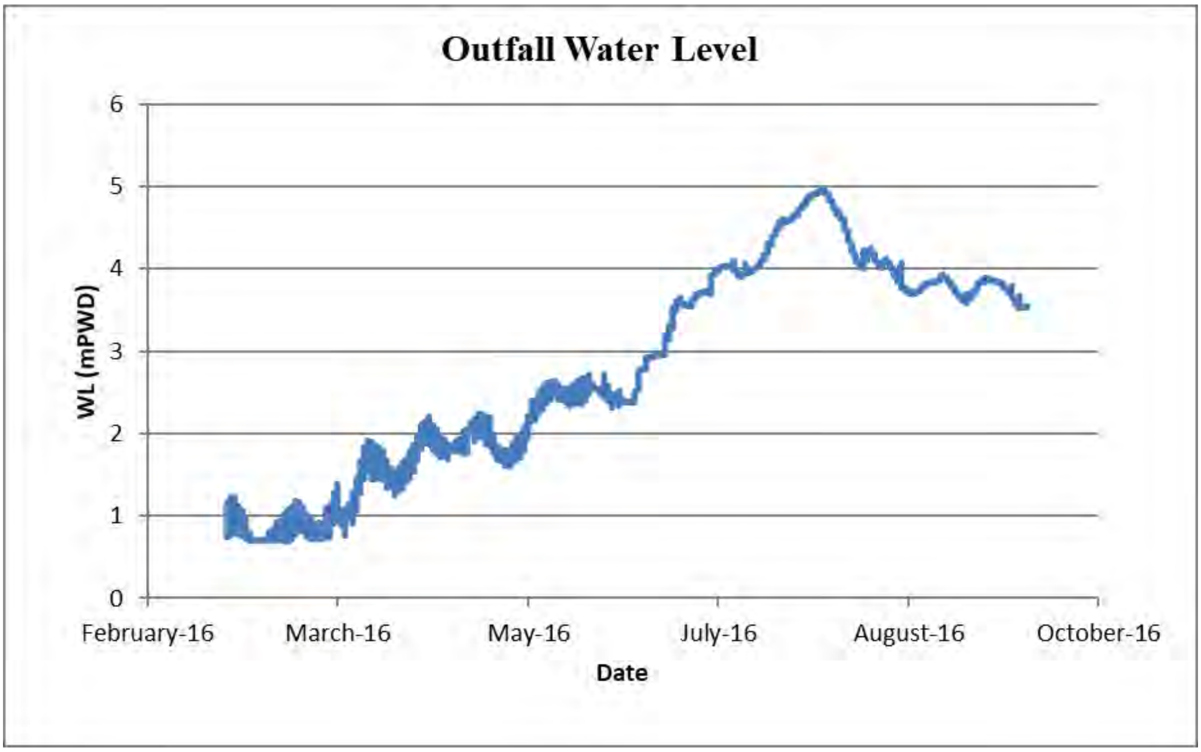


Figure 4.28: WL Boundary Condition at node Outfall/Outlet

Calibration

The precipitation and boundary data was used for the same time period (May-September, 2016) as discussed above for calibration. After initial parameterization, simulated water depths have been adjusted at this stage by Manning's *n* values.

The model was calibrated with observed water levels at Banasree (location is shown in figure 4.26). Table 4.18 compares important statistics of the modeled and observed data at the Banasree gage. The statistics of the modeled and observed data matched well, except for the minimum depth. It is possible that the actual rainfall in the area was smaller than the rainfall recorded at the weather stations, which was away from the watershed.

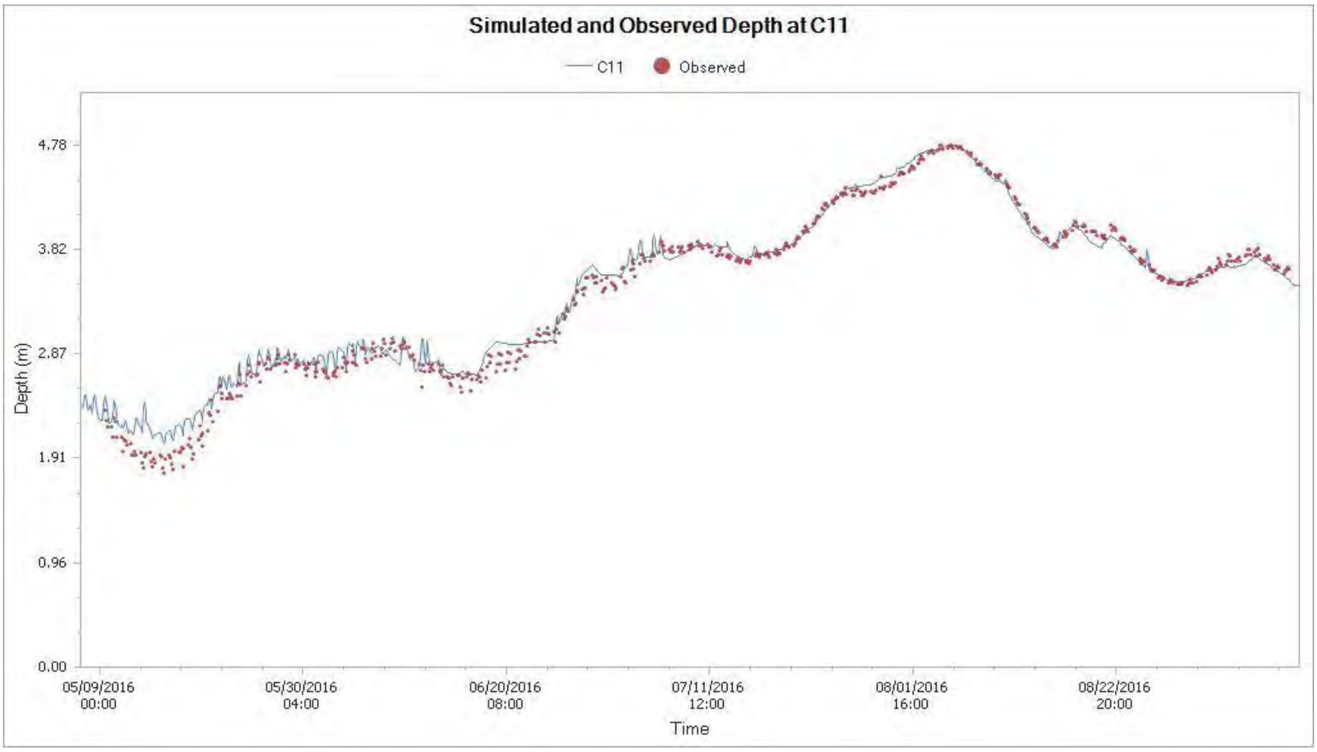
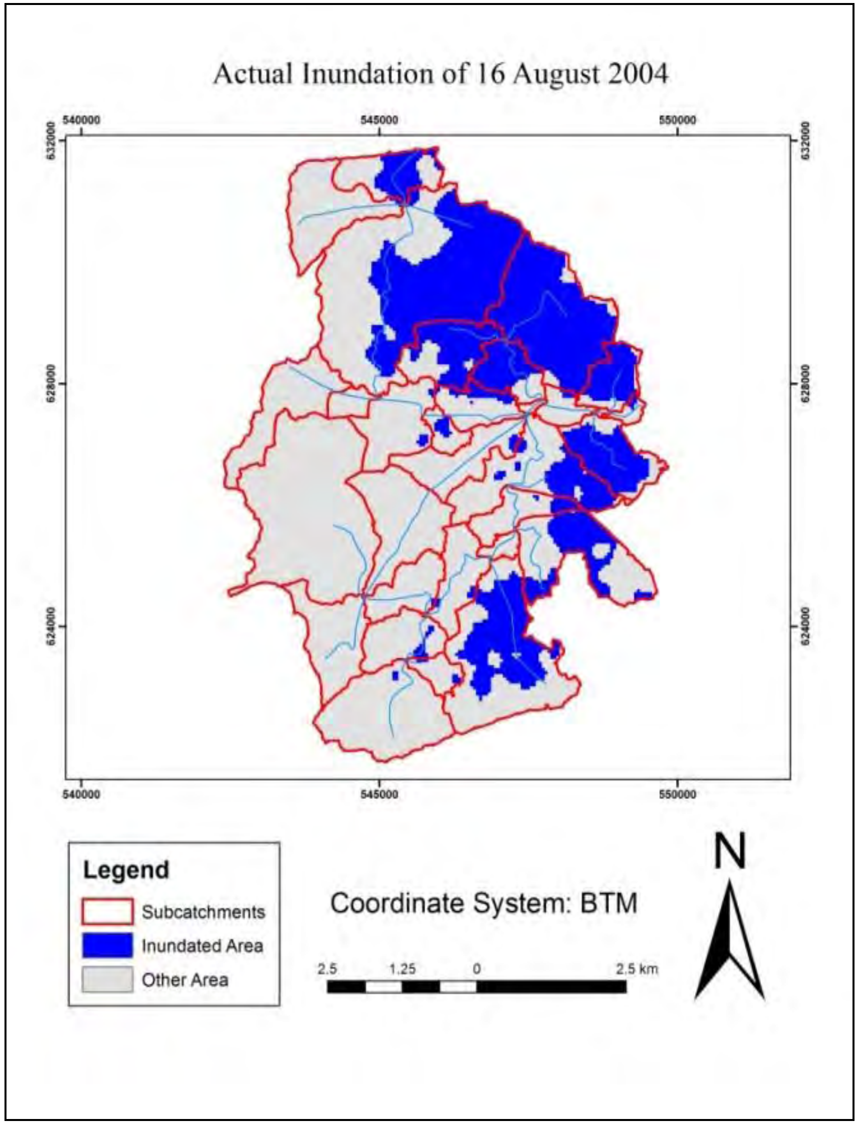


Figure 4.29: Water Depth calibration at Banasree

Table 4.18: Hydrologic calibration statistics at Banasree

Description	Simulated Value	Observed Value
Average Water Depth (m)	3.47	3.43
Maximum Water Depth (m)	4.78	4.77
Minimum Water Depth (m)	2.05	1.77
10% lowest water depth (m)	2.54	2.46
20% lowest water depth (m)	2.79	2.72
50% lowest water depth (m)	3.63	3.63
5% highest water depth (m)	4.65	4.61

Validation



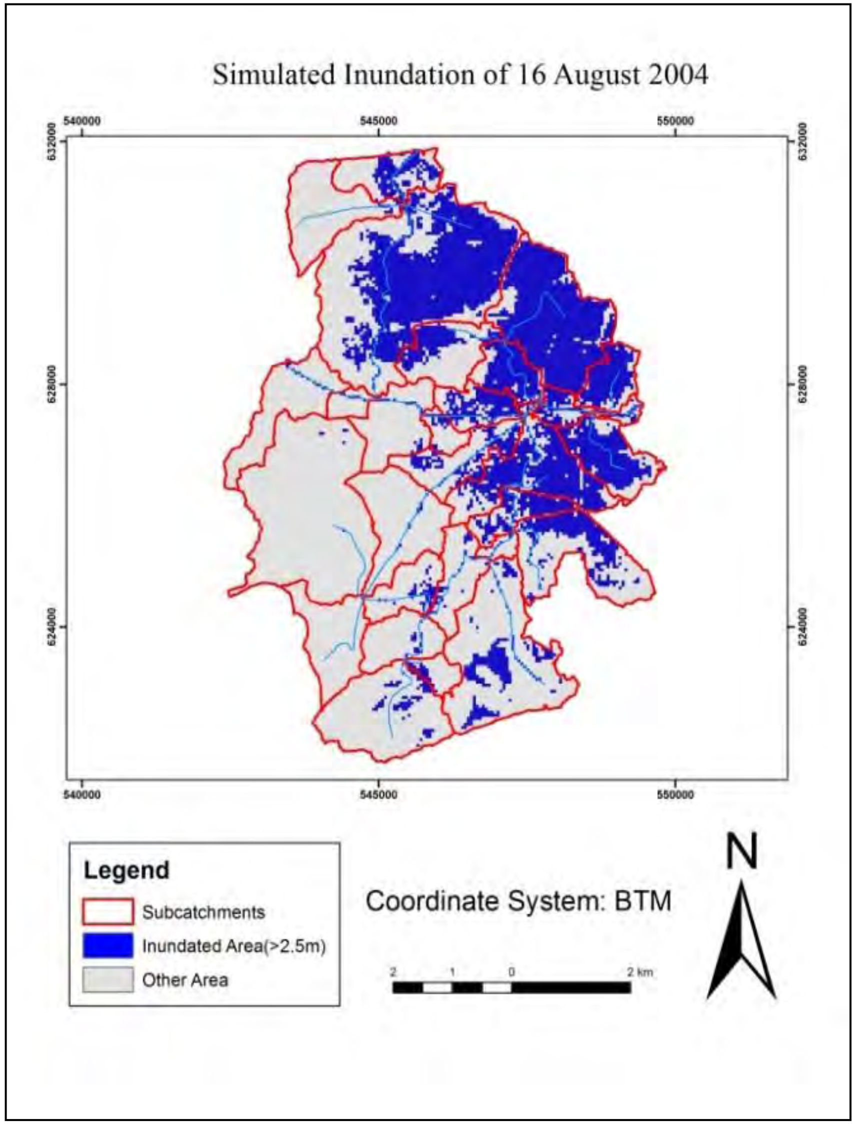


Figure 4.30: Actual (Source: CEGIS) and Simulated Inundation of 16 August

After calibration, the model was validated for two rainfall events (16 August and 13 September) of 2004, and the inundation depths showed close match with available results. Firstly, the current model was applied to find the flood depths and to match that with actual inundation. The actual inundation map of 16 August, 2004 was collected from CEGIS which was derived from 50 m X 50 m resolution Radarsat image. The flood depths from model output showed the similar extent (Figure 4.30) if we consider area with 2.5 m or greater flood depth as inundated area. It can be considered because the image processing and extraction of water body might ignore the smaller depths. Also the image is of coarser resolution than the DEM used in model. DEM used in this model is based on 2015 data.

Again, for the World Bank Report of 2015, IWM applied MIKE11 model to simulate the flood depths (Figure 4.31) in the Dhaka City for 13 September, 2004 (Zaman, 2014). The base condition flood map was extracted from this figure. The current model was also applied for the 100 year return period rainfall of 13 September, 2004 to find the flood depths of the comparable range. Figure 4.32 is showing the inundation depths of MIKE11 model and the current model respectively.

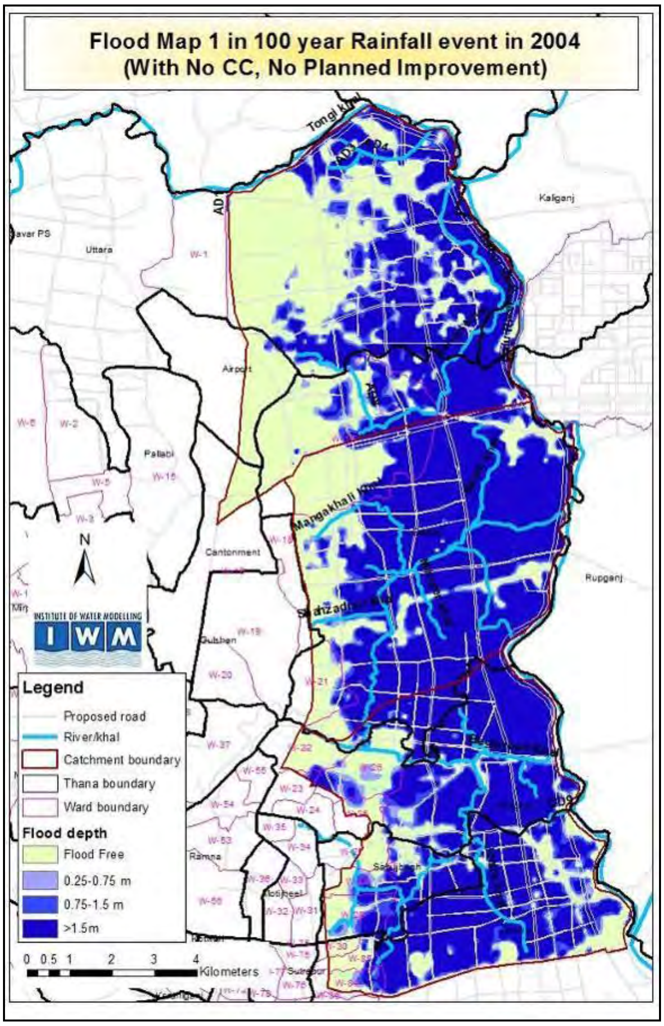
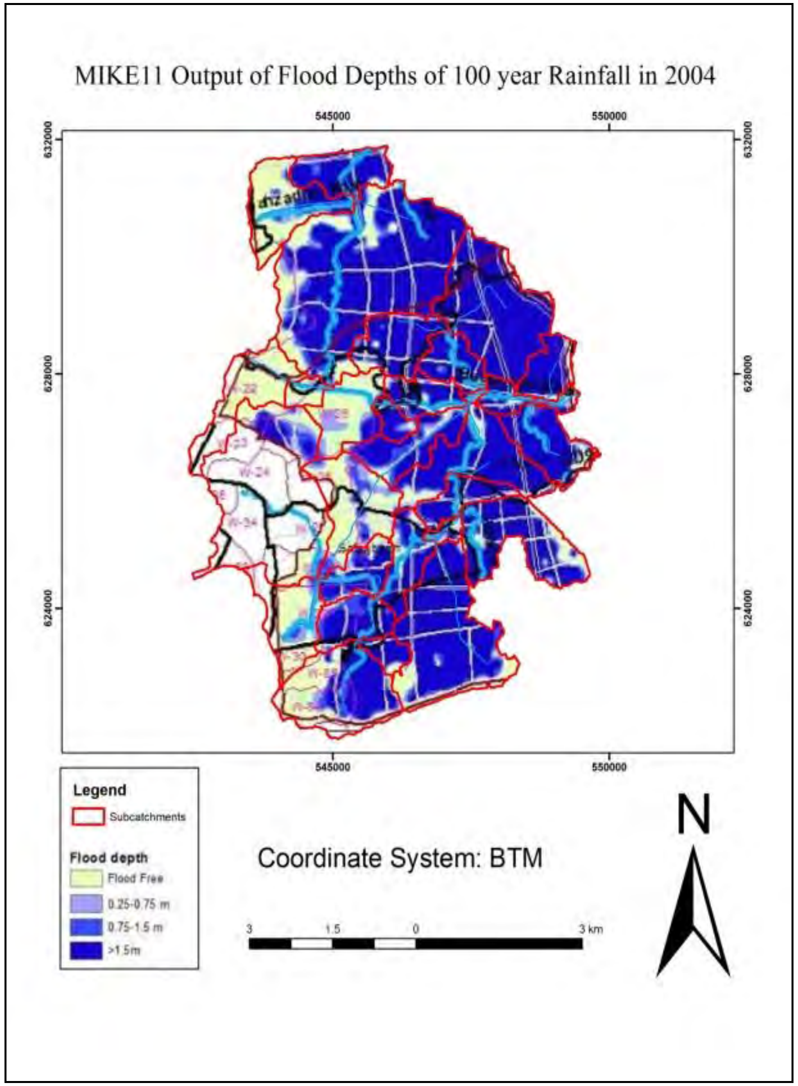


Figure 4.31: MIKE 11 output of Flood depths of Eastern Dhaka (Source: IWM)



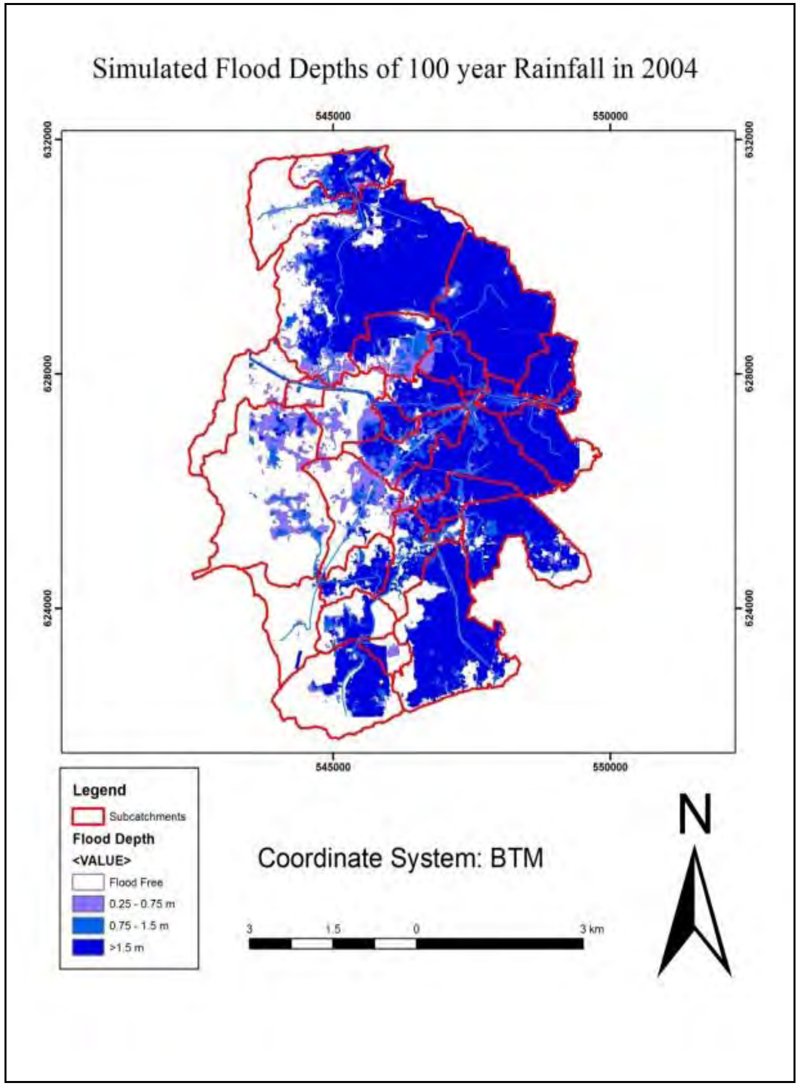


Figure 4.32: Flood depth outputs from MIKE11 (Source: IWM) and Current Model

The above comparisons for validation have showed good agreement with discrepancy in some areas. However, it mainly results from the use of DEM of different time phase. The following Google Earth images (Figure 4.33 and 4.34) reveal that the land elevation and landuse have changed from 2004-2015 on those marked locations.

Figure 4.33: Google Earth image of Study Area of 2004

Figure 4.34: Google Earth image of Study Area of 2015

This calibrated model will be now applied for designing adaptation measures, and impact assessment of different scenarios in following chapter.

CHAPTER FIVE

RESULTS AND DISCUSSION

5.1 General

In existing condition, the river flood in monsoon inundates the study area. To prevent this, embankment with gated pumping facility is the primary solution for this area. Pumping capacity is designed here for 2 consecutive days of rainfall of 10 year return period. The climate change impact and future landuse are taken into consideration while determining the pumping requirement.

The eastern part of the city is still low-lying, but it is expected to be developed with increasing urbanization. With the implementation of eastern embankment, the land filling will increase. At the same time, it will be challenging to find enough area for retention pond. Here, the required pumping capacity is examined with the change in retention area. Additionally, an approach of rainwater harvesting is also applied in the calibrated model. It was found that, rainwater harvesting can play an efficient role in stormwater management for this study area.

5.2 Selection of Design Features

Design Rainfall

A return period of rainfall, also known as a recurrence interval is an estimate of the likelihood of rainfall to occur. It is a statistical measurement typically based on historic data denoting the average recurrence interval over an extended period of time, and is usually used for risk analysis. It is a common practice to specify the return period for analysis and design of stormwater drainage by experience or through some form of guidelines.

In small watershed hydrology, the peak discharge is related to the rainfall intensity, which in turn is related to the time of concentration (through the applicable intensity-duration-frequency curves). Small drainage areas will have a short time of concentration and this will produce a high intensity and high peak discharge per unit of area. However, since the area is small, the peak discharge is correspondingly small. Thus, for small areas, with time of concentration measured in minutes, it is usually not economical to design for long return periods such as 50 years or 100 years.

In Master Plan for Greater Dhaka Protection Project of Bangladesh Flood Action Plan no. 8A (FAP 8A), design rainfall, for Khal Improvement and Trunk Drain, is applied with 5 years return period. Various studies were conducted to protect Eastern part of Dhaka City against flooding and drainage congestion due to stormwater runoff and to improve its transport facilities since JICA FAP 8A study (1992). Then Halcrow Fox undertook the World Bank financed Dhaka Eastern Bypass Study in 1996. After the devastating flood of 1998, Halcrow Fox submitted a report on options for combined flood protection and eastern bypass in 1999. After the severe flood of 2004, BWDB acted as the lead agency to upgrade previous studies on combined flood protection embankment and bypass project for Eastern Dhaka. In that study, design return period is taken as 5 year for internal drainage (i.e. stormwater runoff).

According to the research work of San Diego State University, for urban drainage of small catchment, design rainfall return period is 5 to 10 years. The recommended rainfall return period by experts for design of different components of urban drainage system in Australia is 5 to 10 years (BUET, 2015). It is recommended for Intensely Developed Residential Areas by ARR 1958, 1987. In India, 2 to 5 years return period for rainfall is taken into consideration for design of drainage system in the Residential Areas. If there is any industrial and commercial area, the design return period is considered as 5 to 10 years. This is referred in Rajasthan Urban Infrastructure Development Project. A study (Fortunato, 2014) had been done to define a methodology to identify the optimal rainfall return period for the design of urban drainage systems. The choice of the optimal return period is made minimizing the total costs of the system: the sewer network is dimensioned for a set of possible design rainfall return periods, and the corresponding construction, maintenance and operation costs are evaluated. For each scenario, the total expected damage from flooding caused by rainfall events with return period greater than the design one is then estimated by hydraulic simulation. This methodology has been applied to a small urban catchment in Palermo (Italy) and the optimal solution for the case study is the adoption of a design rainfall period of 10 years.

2-days consecutive rainfall with a 5-year frequency was applied as a design rainfall for pumping station and retarding pond in FAP 8A (1991). While in the 1987 JICA Study, the rainfall pattern of heavy rainfall was studied. The duration of heavy rainfall was found as six hours and accordingly the heavy rainfall pattern was determined. This distribution ratio is applied here for other rainfall intensity as well.

Table 5.1: Percentages of Rainfall for Various Time Steps

Time Step	%
1	9
2	15
3	44
4	16
5	9
6	7

Considering the above factors, the 2-days consecutive rainfall with 10 year return period is selected for this current study as a conservative approach and the value is found as 286 mm. The design distribution has been done following table 5.1.

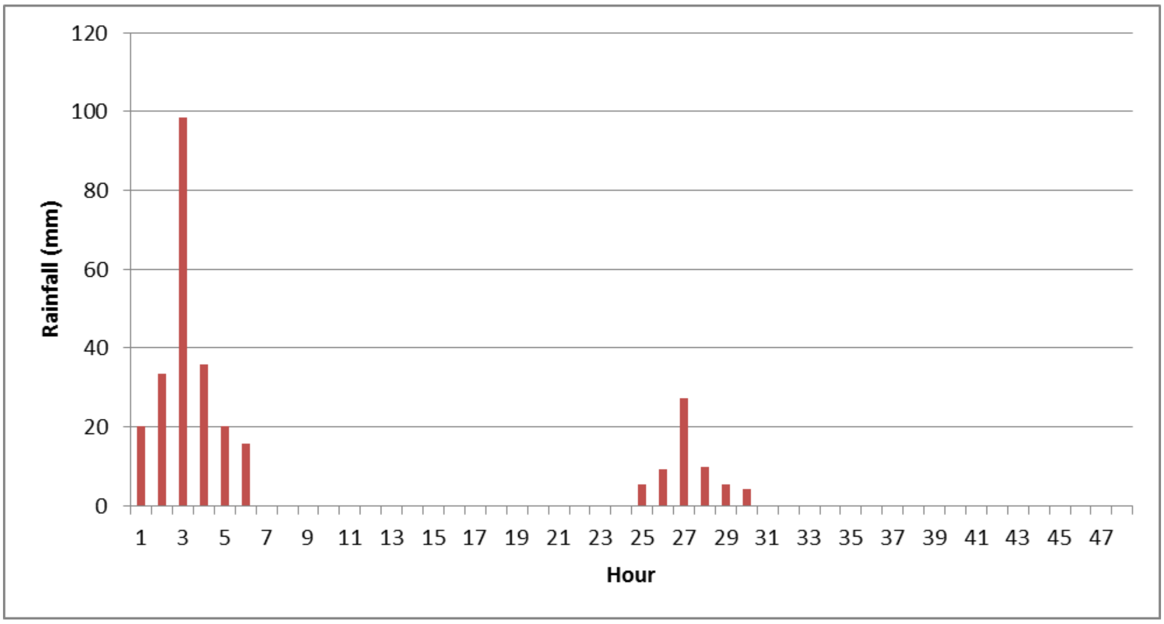


Figure 5.1: Distribution of 2 day 10 year rainfall

Climate Change Impact

Here the climate effects of A1FI 2050 scenario has been applied (Dasgupta, 2015). The climate change effects consider the rainfall to be increased by 16%. Hence, the design rainfall is increased and has become 331.76 mm. Again, climate change will increase the rainfall intensity along with the frequency. Thus, it is more justified to consider 10 year return period instead of 5 year for internal drainage design. So, the design rainfall is distributed according to the Table 5.1.

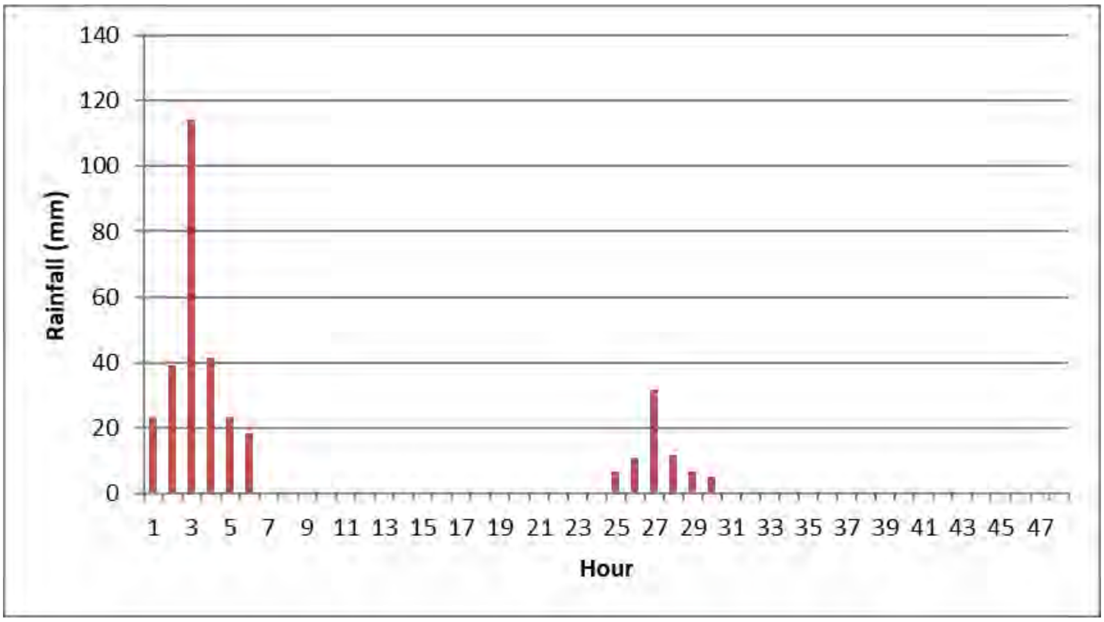


Figure 5.2: Distribution of 2 day 10 year rainfall (considering A1FI 2050 scenario)

Future Landuse

Eastern Dhaka is very potential for urban development. At present a significant part of the land is low and used for agricultural purpose. Most of the residential areas are built up on raised lands mostly on the western part of this area. Now-a-days many land development companies and the land owners are developing this area up to the average flood level. In this way the land use pattern is changing very rapidly. This will result more impervious area in the catchment and hence, more runoff.

The Dhaka Regional Development Plan (RDP, 2035) was published in 2015 by RAJUK with the projected land use in 2035 in which the Eastern Dhaka area is divided into three types of land categories: Growth management area (74.95%), conservation area (6.67%) and central urban area (15.38%). For designing pumping capacity this future landuse is applied in this study.

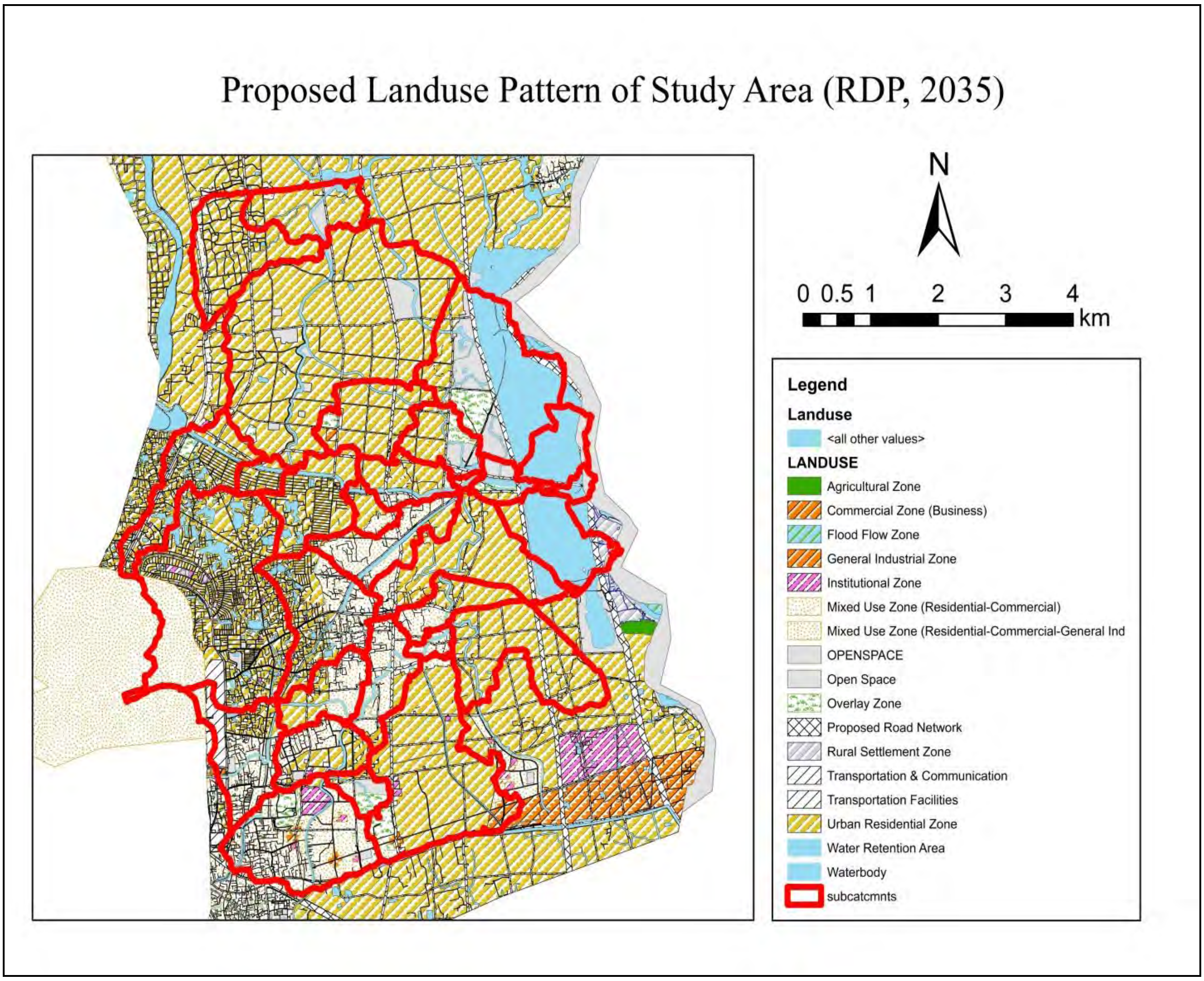


Figure 5.3: Proposed landuse of the study area according to RDP, 2035

In future the natural drainage system is expected to be preserved and future development will follow the existing slope. So, although the general land elevation will be higher than now but the catchment shape will remain close to existing delineation. The main changed parameter will be imperviousness and the new imperviousness is presented in following figure.

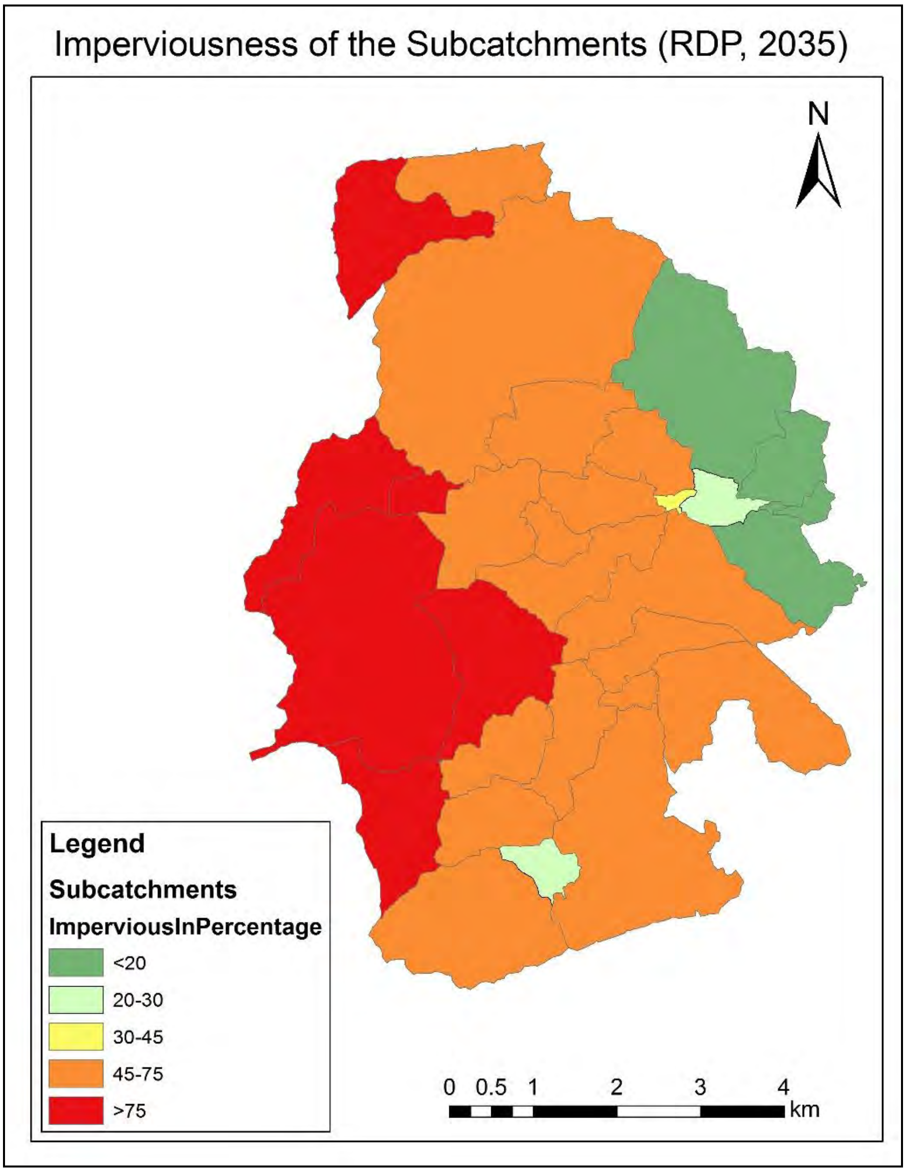


Figure 5.4: Increased imperviousness of subcatchments

5.3 Adaptation Measures

Pumping

Currently, most of the area is under 4 mPWD level. But as the future landuse suggest that the land levels at the eastern side will increase, the maximum allowable water level is selected accordingly. Two very important criterions are the maximum water level that would be allowed in the retention ponds during monsoon and the pumping periods. In the Halcrow study (2006) the maximum allowable level was limited to 4mPWD. In this study, the level is considered from 4mPWD to 4.5mPWD in pump design. As land filling in the catchment area is happening at a very faster rate the low lying areas are moving to higher ground level. It is reported that in the Eastern Dhaka area all land developments at current time are happening at elevation >5mPWD which also supports ultimate selection of maximum allowable water level as 4.5mPWD.

Retention pond is required to store storm water for a certain period and facilitate pumping options. In RDP land use planning about 6.15km^2 is kept for retention pond to facilitate the pumping stations of Eastern Dhaka. To reduce the land requirement and thereby land acquisition cost, other options are simulated with reduced retention pond of about 0.7 km^2, 1.5 km^2 and 2.6 km^2 and accordingly the pump capacity is estimated through modeling.

It is considered that pump would be operational when the water level in the retention pond area rises to 3.3mPWD and would stop when it goes down to 2mPWD. So in the retention areas there would be 1 to 1.5m storage level if maximum allowable level is considered 4 mPWD or 4.5 mPWD respectively. The pump capacity and retention area would be considered in such a way that maximum 2day is required to pump out the runoff generated from 2day 10year rainfall. For the selected design rainfall (2-day 10-year return period with CC impact), the model was run within maximum storage water level of 4.5 mPWD. The objective is to drain the stromwater out within 48 hours.

For maximum 4.35 mPWD water level at retention pond, the necessary retention area and pumping capacity were found as follows.

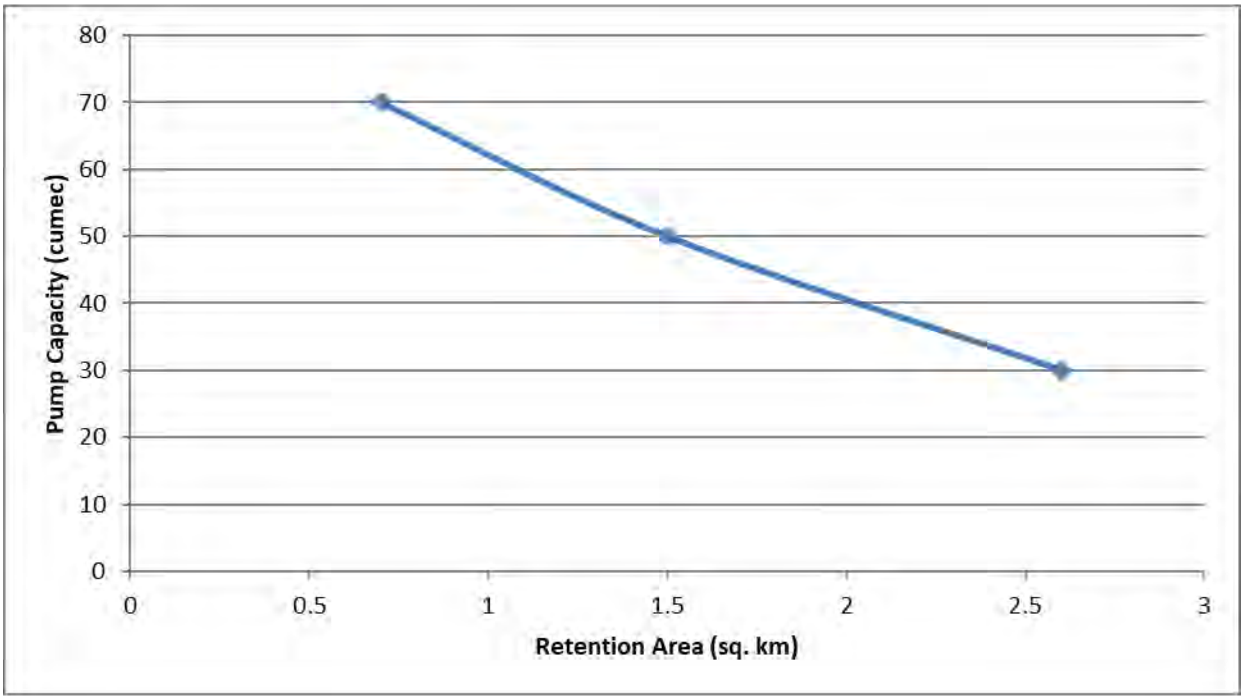


Figure 5.5: Required pumping capacity for various retention areas

The figure above shows the sensitivity between pump capacity and retention area too. But large retention area will not be most likely available while high capacity pumping is also not always manageable considering the fact that existing maximum capacity in Dhaka is 25 cumec (at Hatirjhil). Regulator at Begunbari canal outlet and Eastern Embankment are supposed to protect the countryside from river flood. However, current practical scenario suggests that 1.5 square km area retention pond and 50 cumec pumping is the best combined option to discharge the stormwater of this catchment.

Rainwater Harvesting

There has been growing interest in the use of rainwater harvesting systems in recent years. Rainwater can be collected and stored to supply a range of non potable domestic uses. Until recently, rainwater tanks were primarily considered as a solution to reduce potable water consumption. The impact of rainwater harvesting practices on drainage systems, mainly during extreme rainfall events, has been a secondary consideration, one reason being that these two functions, namely supplying water and managing stormwater runoff, appear to be contradictory. This study uses the calibrated model to assess the benefits achieved in runoff reduction during heavy rainfall events.

The present study area offers a good potential for harvesting rainwater. As most of the area is not developed and prescribed landuse suggests future development in most of the open spaces, rainwater harvesting plan can also be incorporated for future urban area. However, most of the rainwater harvesting system is designed from the water use perspective rather than stormwater management. In this work, an implemented case study of India is followed to design storage tank for rainwater harvesting. The Bimanagar Cooperative Society comprises of 324 residential units. A rainwater harvesting system was put in place in 2002, when the Ahmedabad (Gujarat) Urban Development Authoritu (AUDA) made it compulsory for all buildings of over 1500 sq. m to install these systems. The total rooftop area was 16635 sq. m while the storage volume was 296.73 m^3. In this current study, storage tanks are installed following this ratio. It is considered that one central storage tank is used for each catchment which will be dedicatedly used for such colony-based rainwater harvesting.

In the Begunbari canal catchment, around 12.1 sq. km area is identified in 13 subcatchments for future urban development. 30% of this area is considered as rooftop, from where rainwater will be harvested. The total rooftop area is found as 3.63 sq. km and hence, total storage volume is calculated as 64750 m^3. The catchment-wise underground storage tank area is calculated considering 10m depth.

Table 5.2: Storage area calculation for subcatchments

Catchment	Future Development Area (km^2)	Potential Rooftop Area (m^2)	Storage Volume (m^3)	Storage Area (m^2)
W49	2.68	804000	14341.50	1434.15
W62	1.197	359100	6405.52	640.55
W35	0.61	183000	3264.30	326.43
W60	0.4	120000	2140.52	214.05
W13	0.624	187200	3339.22	333.92
W14	0.16	48000	856.21	85.62
W42	0.975	292500	5217.52	521.75
W4	0.22	66000	1177.29	117.73
W5	1.16	348000	6207.52	620.75
W44	0.185	55500	989.99	99.00
W47	0.16	48000	856.21	85.62
W06	3.33	999000	17819.85	1781.99
W9	0.4	120000	2140.52	214.05

5.4 Result Analysis

Climate Change Impact

The designed pumping system is run here first to observe the climate change impact. The RDP landuse is kept while the rainfall has been changed according to figure 5.1 and 5.2.

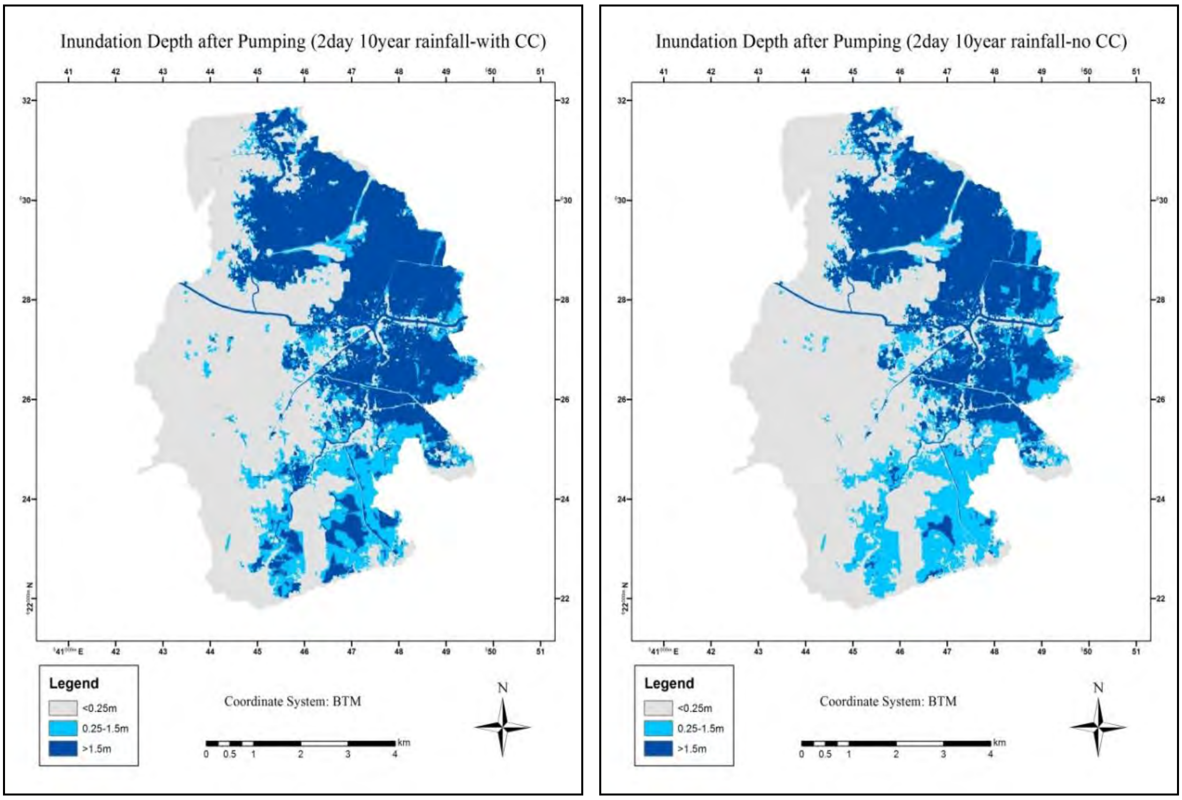


Figure 5.6: Inundation depth after pumping (with and without CC)

Table 5.3: Inundated area: with and without Climate Change impact

	Max Flooded Area (sq.km)			**Total Flooded Area (sq.km)**
	depth: 0-0.25 m	depth: 0.25-1.5 m	depth: >1.5 m	
With CC	27.42	7.83	18.29	27.32
Without CC	28.89	7.16	17.48	25.76

So, it is observed that after pumping, the increased rainfall due to climate change impact cause higher inundated area. Due to 16% increase in rainfall, the total inundated area also increases. The amount of increase here is 6.06%.

Catchment Response

The hydrological response of the catchment is studied for the different conditions. All the boundaries are kept free to check the flow hydrograph at outfall for the design rainfall. The 331.76 mm rainfall of 2 days is applied in existing condition, and also for proposed RDP, 2035 landuse pattern. Again, storage tanks for harvesting rainwater are also used to examine its impact. The invert levels of storage tanks are kept 7 m below the nearest stream bottom. The upstream end of connecting pipe between storage tank and the stream is kept at 8 m offset, considering the total tank depth of 10 m.

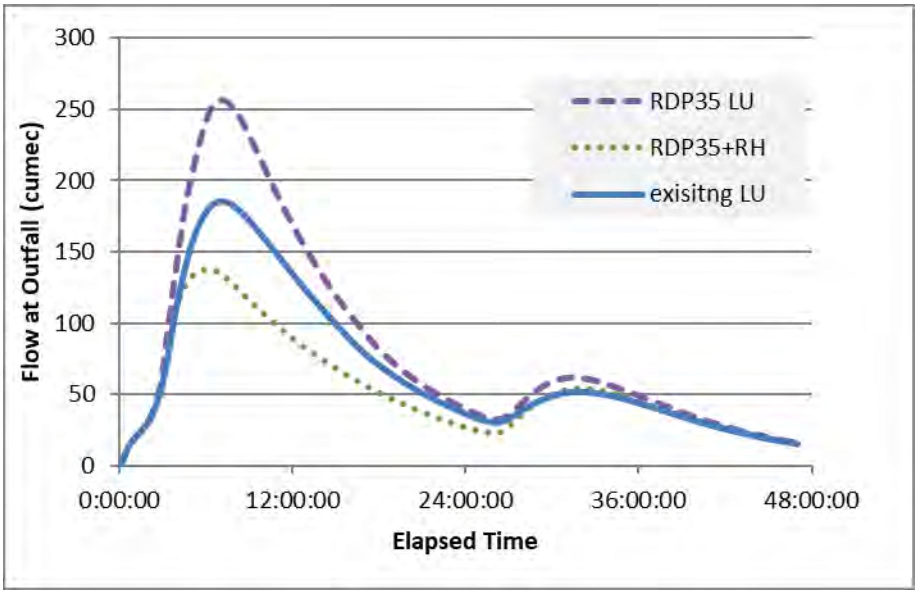


Figure 5.7: Outfall flow at different conditions

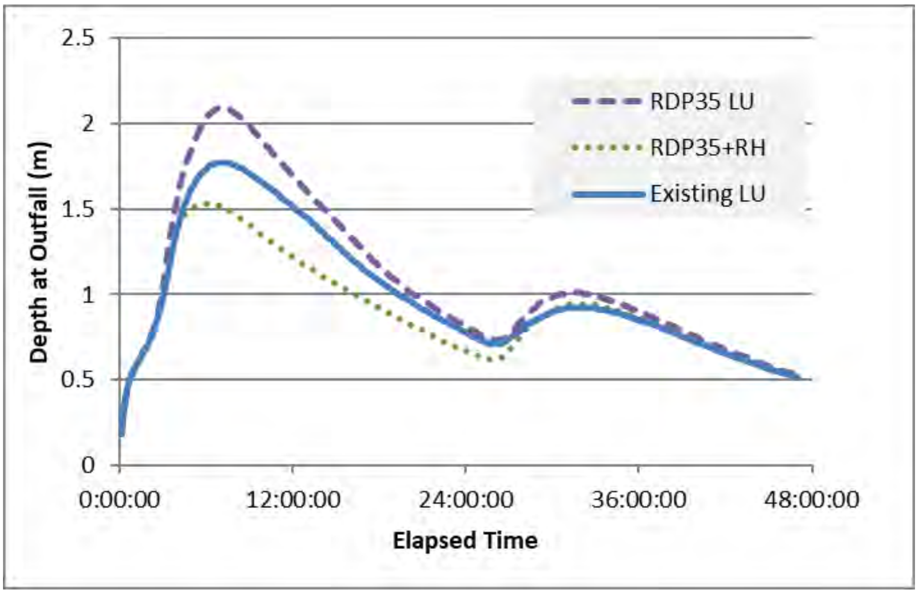


Figure 5.8: Outfall depth at different conditions

The figures 5.7 and 5.8 are showing that runoff depth and flow is increased in future landuse. But application of storage volumes can significantly reduce both the peaks of flow and depth at outfall. Here the peak flow reduces to 138.85 cumec from 258.77 cumec (46.34% reduction) while the maximum depth reduces to 1.54 m from 2.1 m (26.7% reduction). The peaks however arrive slightly earlier, which might be caused by the absence of stored water. The remaining water can arrive quickly at the outfall.

Impact Study

The impacts of adaptation measures are observed here for 2 days rainfall of 5 year and 25 year return period along with the design 10 year return period. The hourly rainfall is calculated using the distribution of table 5.1. These rainfalls are presented in figure 5.2, 5.9 and 5.10.

Analysis has been done for three conditions: without any measure, with pumping and with pumping and rainwater harvesting. For all scenarios, future landuse has been considered and for without pumping scenario, 80 percentile value of river water level have been considered of respective return period.

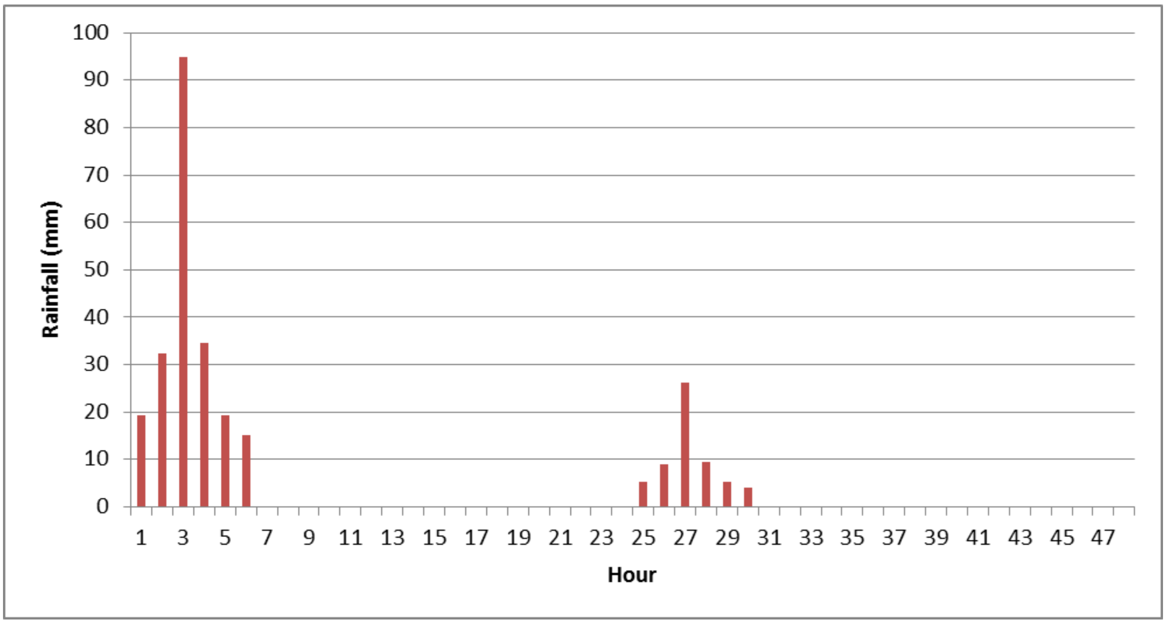


Figure 5.9: Distribution of 2 day 5 year rainfall (considering A1FI 2050 scenario)

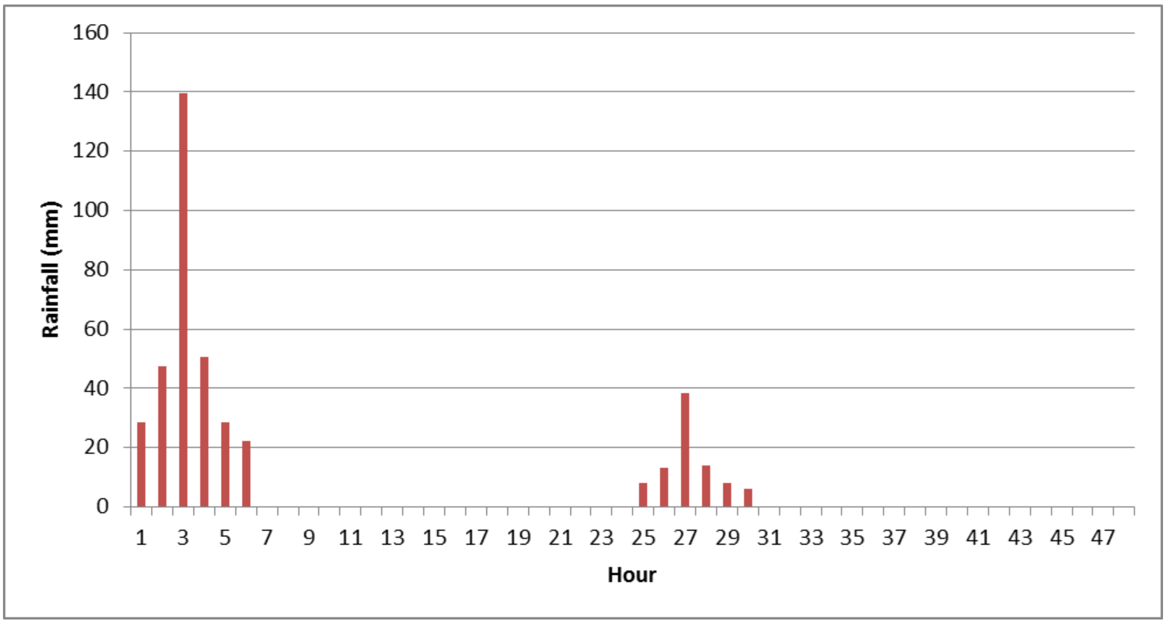


Figure 5.10: Distribution of 2 day 25 year rainfall (considering A1FI 2050 scenario)

2 day 5 year Rainfall

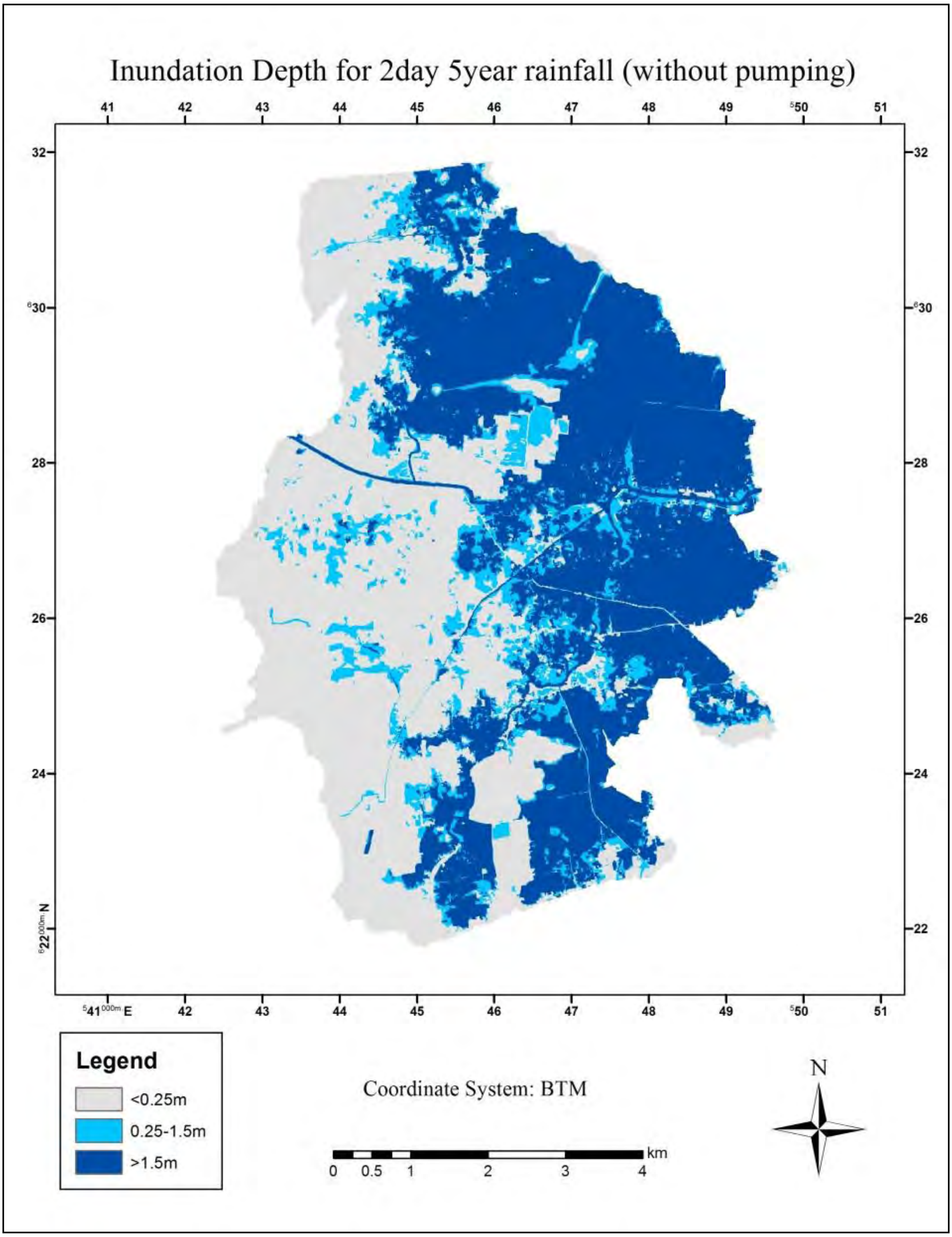


Figure 5.11: Inundation depth for 2day 5year rainfall (without any measure)

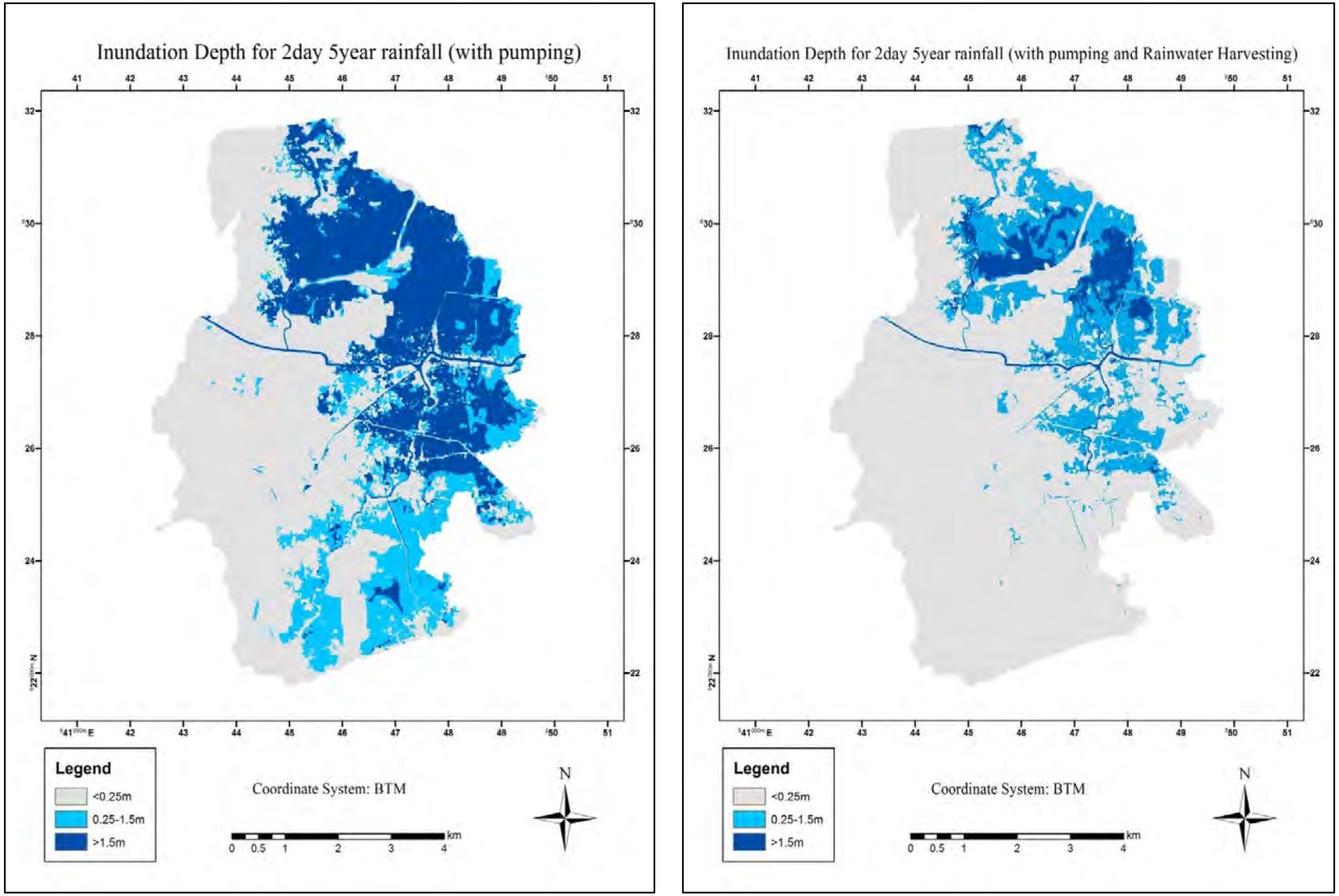


Figure 5.12: Inundation depth for 2day 5year rainfall (with pumping and with pumping+RH)

The 278.4 mm 2 day 5 year rainfall (with climate change impact) of figure 5.9 is applied in the catchment. The above figures (Fig. 5.11 and 5.12) suggest that inundation depth and area both can be reduced in this case by taking measures of pumping and pumping with storage tank for rainwater harvesting. Table 5.4 shows that the area of lower depths increases while the area of higher depth decreases with adaptation measures. The area with depth greater than 1.5 m can be 36.35% reduced by pumping while it can be 87.74% reduced if rainwater can be harvested by storage tank along with the pumping.

Table 5.4: Inundated area for 2day 5year rainfall

	Max Flooded Area (sq.km)			**Total Flooded Area (sq.km)**
	depth: 0-0.25 m	depth: 0.25-1.5 m	depth: >1.5 m	
without any measure	20.36	6.18	21.13	30.11
with pumping	25.85	8.38	13.45	22.84
with pumping+RH	36.96	8.13	2.59	12.56

2 day 10 year Rainfall

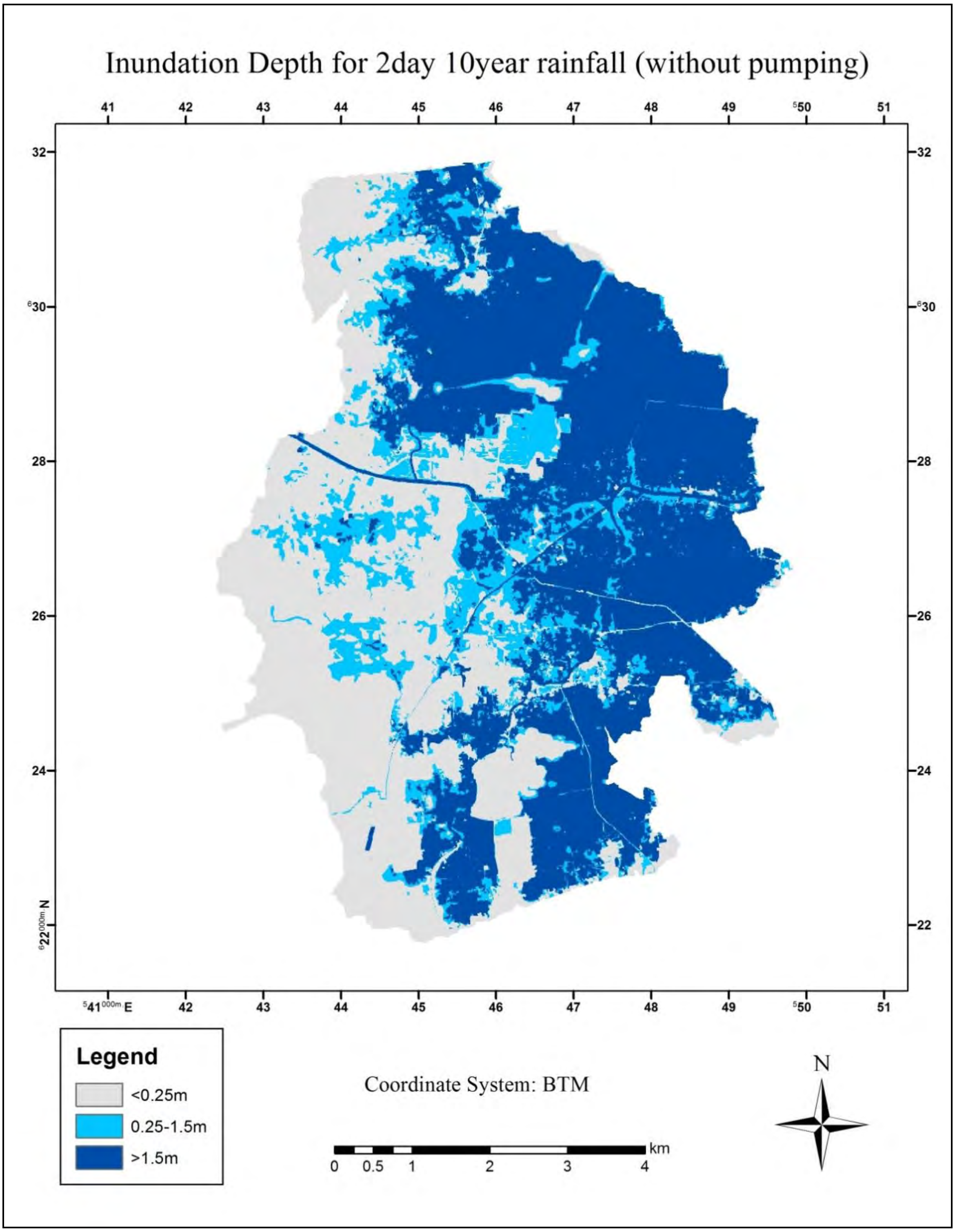


Figure 5.13: Inundation depth for 2day 10year rainfall (without any measure)

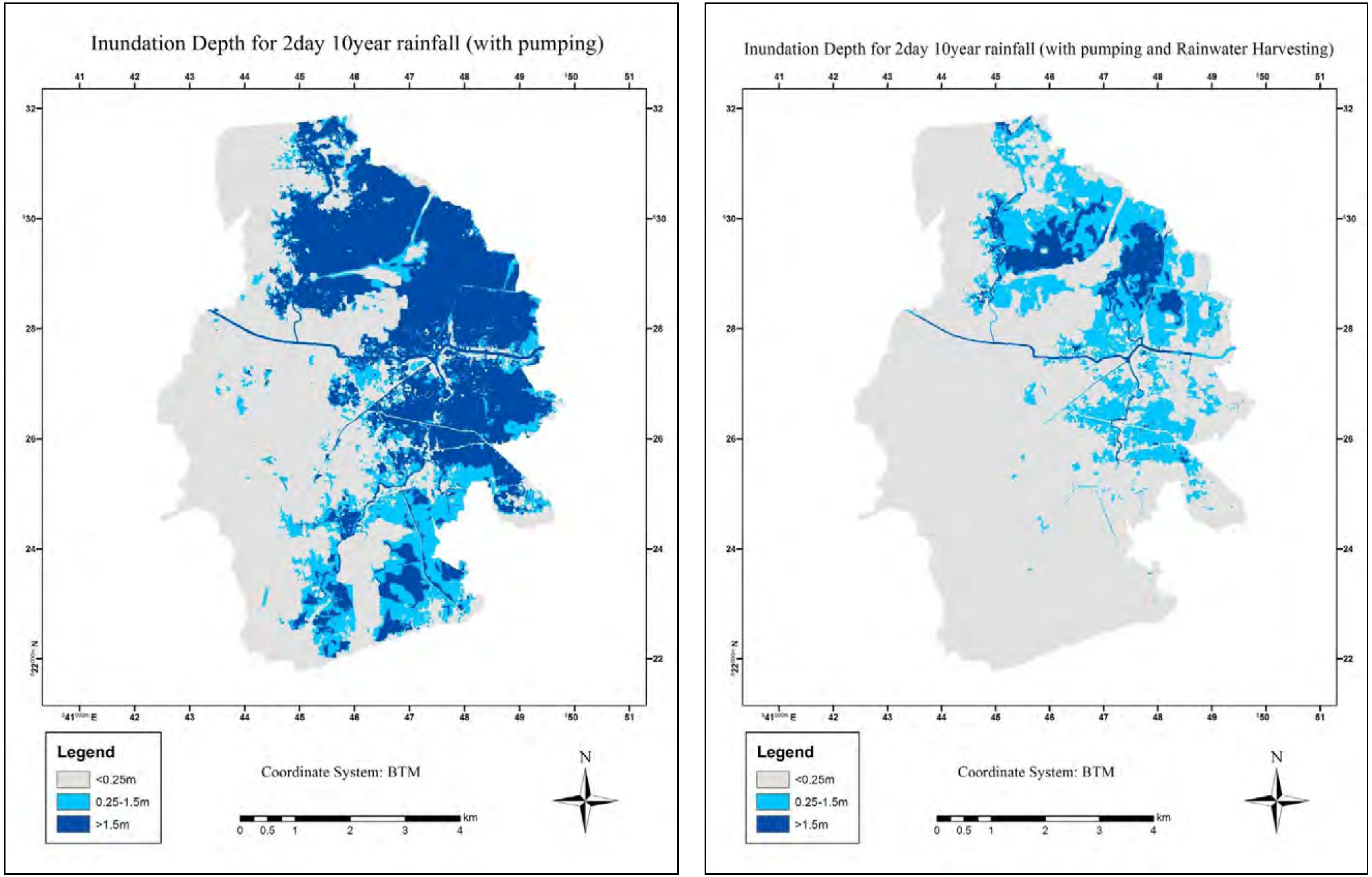


Figure 5.14: Inundation depth for 2day 10year rainfall (with pumping and with pumping+RH)

The 331.76 mm 2 day 10 year rainfall (with climate change impact) of figure 5.2 is applied in the catchment. The above figures (Fig. 5.13 and 5.14) suggest that inundation depth and area both can be reduced in this case by taking measures of pumping and pumping with storage tank for rainwater harvesting. Table 5.5 shows that the area of lower depths increases while the area of higher depth decreases with adaptation measures. The area with depth greater than 1.5 m can be 17.28% reduced by pumping while it can be 87.7% reduced if rainwater can be harvested by storage tank along with the pumping.

Table 5.5: Inundated area for 2day 10year rainfall

	Max Flooded Area (sq.km)			**Total Flooded Area (sq.km)**
	depth: 0-0.25 m	depth: 0.25-1.5 m	depth: >1.5 m	
without any measure	17.81	7.75	22.11	33.4
with pumping	27.42	7.83	18.29	27.32
with pumping+RH	36.72	8.23	2.72	12.79

2 day 25 year Rainfall

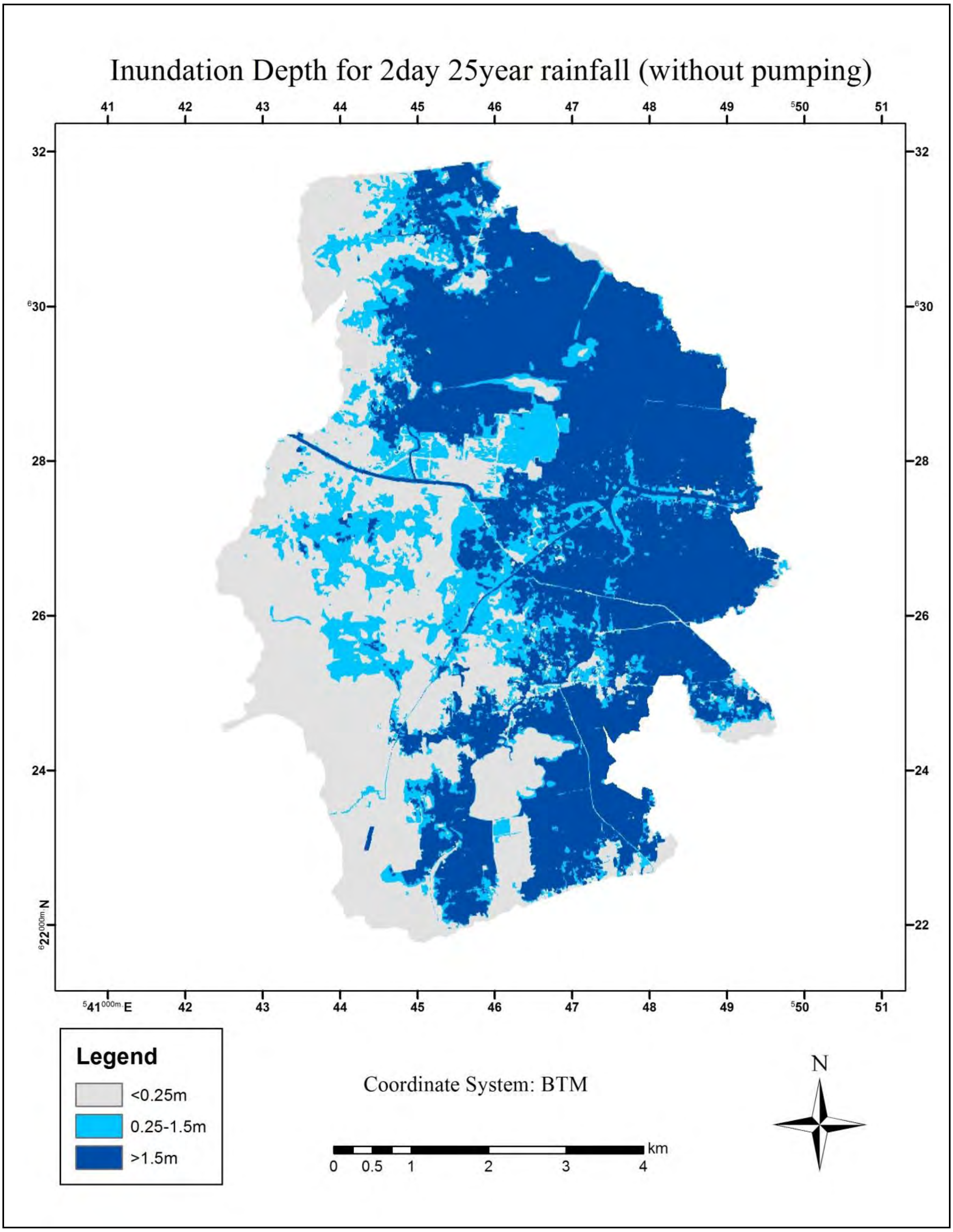


Figure 5.15: Inundation depth for 2day 25year rainfall (without any measure)

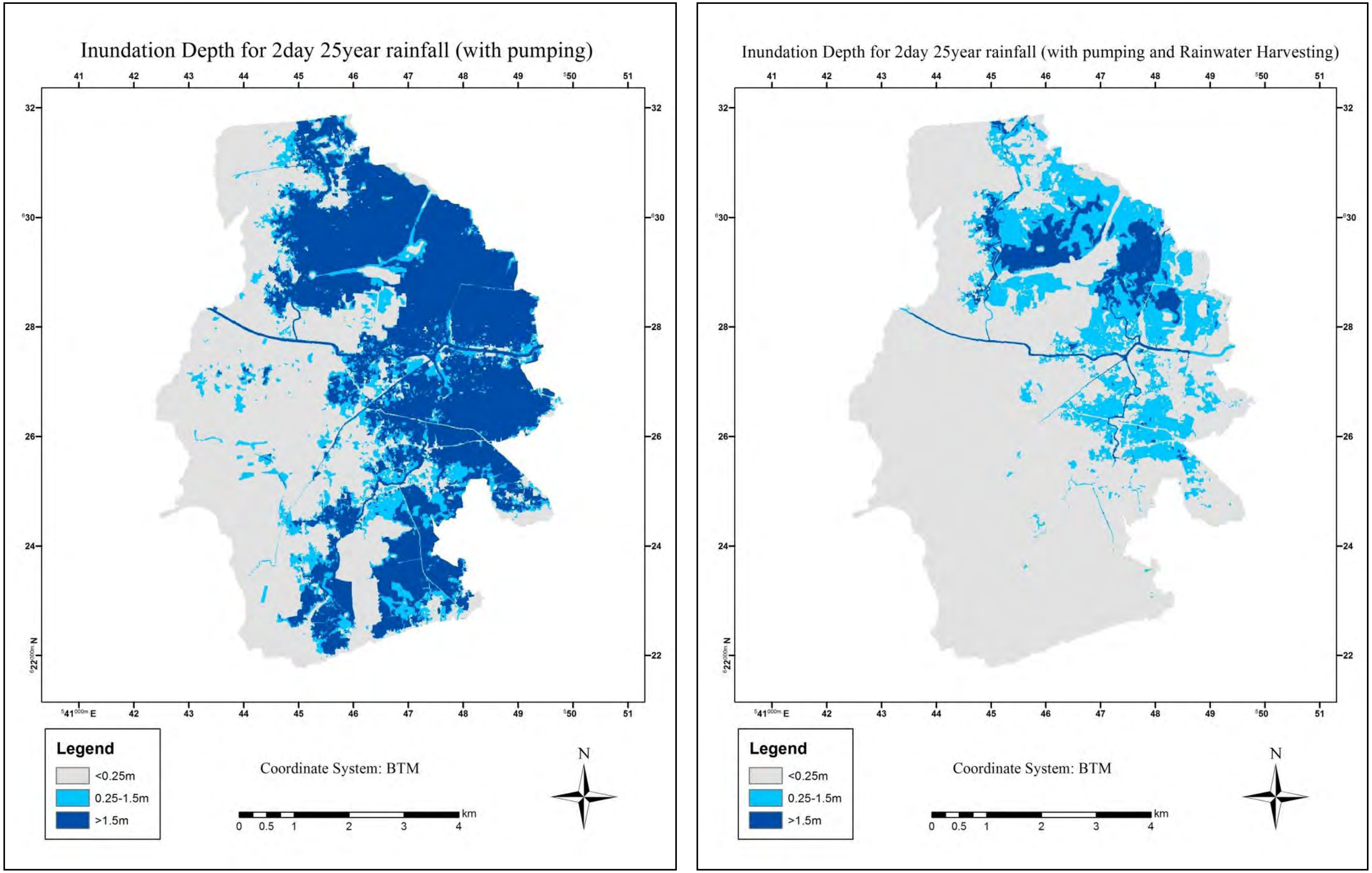


Figure 5.16: Inundation depth for 2day 25year rainfall (with pumping and with pumping+RH)

The 404.84 mm 2 day 25 year rainfall (with climate change impact) of figure 5.10 is applied in the catchment. The above figures (Fig. 5.15 and 5.16) suggest that inundation depth and area both can be reduced in this case by taking measures of pumping and pumping with storage tank for rainwater harvesting. Table 5.6 shows that the area of lower depths increases while the area of higher depth decreases with adaptation measures. The area with depth greater than 1.5 m can be 12.13% reduced by pumping while it can be 87.43% reduced if rainwater can be harvested by storage tank along with the pumping.

Table 5.6: Inundated area for 2day 25year rainfall

	Max Flooded Area (sq.km)			**Total Flooded Area (sq.km)**
	depth: 0-0.25 m	depth: 0.25-1.5 m	depth: >1.5 m	
without any measure	16.57	8.6	22.51	34.69
with pumping	22.08	5.8	19.78	27.39
with pumping+RH	36.51	8.33	2.83	13.01

Thus, the deigned pump capacity of 50 m^3/s and 1.5 km^2 retention pond can effectively reduce the internal flood. And installation of storage tanks for rainwater harvesting can significantly contribute to flood reduction. It is found that the total inundated area can be reduced by 18-25% by pumping. The inundated area can be reduced by around 60% by storage tank along with the pumping for this 2 days consecutive rainfall of 10 year return period.

CHAPTER SIX
CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

In monsoon the river water of Bangladesh rises often over flood level. Due to the flow from upper regions flooding of Balu River is also a common phenomenon. When water level rises over 4 mPWD, the river water enters into the Eastern part of Dhaka. Again, due to huge population stress, Dhaka is expanding and it is also extending to its east. In this scenario, to keep Dhaka from flood free, an embankment on the right side of Balu River is imperative. Then in the monsoon season the internal water will be pumped into the river. In this study, this adaptation measure is applied for the Begunabari catchment of Eastern Dhaka. It is assumed that, future development in this area will be done in such a way so that the natural canals can serve as the main drainage way to the river. The following key information and conclusions are drawn from the study:

- GeoSWMM is used in this work to model the Begunbari canal catchment. The 47.67 sq. km catchment area covers a large area from Rampura to Demra with Balu at other end. This flow through Begunbai canal also covers the flow from Hatir Jheel and Maniknagar pump stations. The model is calibrated using the observed water level at banasree for May-September, 2016.
- Although some of the previous studies work with 2 day 5 year return period, in this study drainage facilities have been designed for 2 day 10 year return period.
- The design rainfall value of 286 mm is increased by 16% considering the climate effects of A1FI 2050 scenario.
- For future landuse scenario, RDP, 2035 landuse has been used.
- The suggested pumping station at Begunbari canal outlet would drain out a 50 cumec flood flow to maintain internal water level not to exceed 4.5m PWD at times of design storm evacuating the excess flooding within 2 days. To do such it will require 1.5 sq. km of retention pond at the pumping station.
- The bottom level of retention pond is considered at -0.5 mPWD. For smooth operation and economy of placement of total number of pumps, pump operation will start at 3.3m PWD in the monsoon. Regulators would be functional during pre-

monsoon and post monsoon period when Balu is at lower level allowing gravity drainage.

- Here, the designed pump has been applied for 2day rainfall of 5, 10 and 25 years return period. It was observed that the inundated area with depth greater than 1.5 m can be reduced by 15-30% by pumping.
- The scope of flood reduction by storage tanks of rainwater harvesting system is also investigated here. There are 13 subcatchments identified where future development would occur. 30% of this future development area is considered here as rooftop area and the storage tanks are designed accordingly. The design values are taken from an implemented project (Kavarana, 2013).
- Total 6475 m^2 storage area is considered taking the depth of tank as 10 m. The inundation depth and area both were found reduced in this case by taking measures of pumping with storage tank for rainwater harvesting. The area with depth greater than 1.5 m can be reduced by around 87% by storage tank along with the pumping for this 2 days consecutive rainfall of 10 year return period. Again, the stored water can be used for household purpose and even for groundwater recharge. So, this option should be considered during future development in this area.
- From the field visit it is observed that, the condition of monitoring and management of drainage pipes is poor. Even the natural channels get blocked in some places. Strong institutional policy and governance is required to ensure smooth flow in pipes and canals. The authority should be responsible to keep the waterbody, open spaces and floodflow zones free from any encroachments.

6.2 Recommendations

The recommendations for future studies are stated as follows:

- This study considers that all the stormwater will be conveyed into the natural canals. So, a planned drainage system should be fixed and implemented. Pipe networks can be incorporated in the model for checking adequacy or design.
- The surveyed cross-sections of the natural canals of this study area are used in this study. Some future improvement scenarios can be done in future such as, canal dredging or canal widening.

- A dedicated colony-based storage tank is considered here for rainwater harvesting. The effect of other ways (such as, single storage unit for each building) of rainwater harvesting can be modeled in future.
- The invert level of the storage tanks were found very sensitive. Since the final level is dependent on the future land level, it should be examined closely based on that. The subcatchments can be divided into smaller parts to study this more precisely. Similarly, the location of the tank, the invert levels of the connecting pipes should be finalized with different trials.
- Since the area under the study will be urbanized in future, the stormwater management and land development strategy should strive to mimic pre-disturbance hydrologic processes by minimizing the impervious area. For such area, application of model for designing and estimating the effectiveness of Best Management Practices (BMPs) and Low Impact Developments (LIDs) can be a scientific approach towards development.
- In this study, future landuse is used but the land levels are kept same as current DEM. It can be expected that the land levels will be raised with future development. So, with the availability of proper data, the model can be updated to better estimate its impact.

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