DEVELOPMENT OF AN URBAN DRAINAGE SYSTEM FOR A SMALL TOWNSHIP – A CASE STUDY

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DECLARATION

I do hereby declare that the project work reported herein, has been performed by me and this work has neither been submitted not being concurrently submitted in candidature for any degree at any other University.

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DEDICATED TO MY PARENTS

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ABSTRACT

Due to urbanization and increase in population, urban regions of Bangladesh require immediate improvement of drainage system. Nowadays, climate change has become a global issue and Bangladesh is in a high probability of being heavily affected by climate change. The effect will be severe in Bangladesh's urban areas, where drainage is already a serious problem. Realizing the importance of this issue, Government of Bangladesh (GOB) has emphasized on this matter; and has already taken initiative to develop the drainage system of the small Township/Municipal (Urban regions of a District) areas. Dhamrai Municipality, located in the North Central Region of Dhamrai upzila of Dhaka District under Dhaka Division is the object of this study. This study provides a framework to water management policy within Municipal areas.

Despite of its location in the North Central Region, significant part of its area is affected by external flood. The parts of north-west and south-east are flood affected. Two earthen borrow pit exists along both sides of the Dhaka- Aricha highway. But these borrow pits are encroached at reaches by local inhabitants which at present scenario is hampering the natural drainage route to the desired outfall. Moreover, many of the drains fall into relatively low lying areas in a haphazard ways, thus causing drainage congestion and water logging problems in some places after heavy rainfall. The proposed drainage system within the study area is an open drainage system. Total seven major drains have been proposed with storm drains. Among them, five major drains has been recommended as priority needs while two major drains are proposed in view of future needs for the study area. Rests of the zones are planned with their outfalls for future drainage details.

Three different scenarios for two different case studies were formulated and simulated to test the drainage capacity. The results of the simulations for each scenario were analyzed individually in order to evaluate the possible risks of future inundations in the study area. Two cases were conducted by considering a proposed drainage system without re-excavated borrow pits and another considering proposed drains with re-excavated borrow pits. The scenarios are based upon the analyses such as design year rainfall events and various outfall water level stages. In order to simulate the scenarios, the DHI computer program MIKE11 has been used to create a rainfall-runoff model which consists of a hydrological and a hydrodynamic model. The necessary information to create the model was taken from IWM, BMD, BWDB, SRDI and several literature reviews. The calibration optimizes the hydrodynamic model so when the historical rainfall data is put into the model and is simulated the out coming graph of the water level is made as equal or nearly equal to the graph of the recorded water level as possible. The results are presented in the form of flood map and damage map, so the area of inundations under different rainfall events and water level stages along with its pattern of damages could easily be assessed out.

The historical rainfall was utilized in the simulations which showed, Case Study 2 provided better results and improvements in flooded areas compared to Case Study 1. Around 99% and 80% of land will be above flood level under scenario S1 for Case Study 2, which means a

small impact for the design year rainfall events on the drainage system. Backwater effects from the river Bangshi, has large impacts on the water level in the low-lying parts of the drainage network. Only 50% and 46% land will be above flood levels under scenario S3 for Case Study 2, which shows 5% and 6% improvements in flooded areas compared to Case Study 1. Due to flooding, most of the damages will occur in homesteads/ residential areas but majority of the damage remains within first category of damages. Moreover, industrial sectors and commercial enterprises also undergo low to moderate type of damages. None of the scenarios indicate any additional areas in risk of flooding and damages in the future compared with today's situation.

It is recommended that, construction of some tertiary drains connecting the proposed secondary drains or raising of the land levels, will improve rainfall flooding conditions within the places which are facing water logging problems at present scenarios, due to the presence of pocket depressions.

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ACRONYMS & ABBREVIATIONS

ADB	Asian Development Bank
AVG	Average
BM	Bench Mark
BMD	Bangladesh Meteorological Department
BWDB	Bangladesh Water Development Board
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute
DPHE	Department of Public Health Engineering
DTW	Deep Tube Well
FAP	Flood Action Plan
FF	Flood Free
GIS	Geographic Information System
GOB	Government of Bangladesh
GPS	Global Positioning System
GSB	Geological Survey of Bangladesh
HD	Hydrodynamic
HTW	Hand Tube Well
IDF	Intensity Duration Frequency
IWM	Institute of Water Modelling
MIKE 11	One-dimensional river modelling system of DHI
MPO	Master Plan Organisation
NCRM	North Central Region Model
PTW	Production Tube Well
PWD	Public Works Datum
RF	Rainfall
RR	Rainfall Runoff
RWH	Rain Water Harvesting
SOB	Survey of Bangladesh
SRDI	Soil Research Development Institute
STP	Sewage Treatment Plant
SW	Surface Water
SWM	Solid Waste Management
SWMC	Surface Water Modelling Centre
TBM	Temporary Bench Mark
UDM	Urban Drainage Model
WL	Water Level
WRP	Water Resources Planning
WT	Water Table
WTP	Water Treatment Plant

Chapter 1
INTRODUCTION

1.1 General

Flooding in urban drainage systems may occur depending on the drainage system i.e. cross connections in the system (storm, waste water and drainage pipes), damage, roots and sediments in the system, infiltration and exfiltration, pollutants and nutrients transported in the system, general drainage design characteristics, drainage outlet not reaching the desired outfall etc (Schmitt *et al*., 2004). Thus failure or lack of planned and adequate drainage system in urban areas cause large infrastructural damages, loss of business and spreading of diseases. Changes of land use from rural to urban increases runoff as a result of growth and spread of impervious surfaces thus increasing hydrological and economical stresses on the environment (Reuterwall and Thoren, 2009). This picture is quite common within the small Townships of Bangladesh.

Due to urbanization and increase in population, urban regions of Bangladesh require immediate improvement of drainage system. Whereas, climate change is a global issue and Bangladesh being in a high probability of being heavily affected by climate change because of its vast low-lying areas along the Ganges- Brahmaputra- Meghna Delta, the effect will be severe in Bangladesh's urban areas, where drainage is already a serious problem. Realizing the importance of this issue, Government of Bangladesh (GOB) has emphasized on this matter; and has already taken initiative to develop the drainage system of the small Township/Municipal (Urban regions of a District) areas. In this regard, Department of Public Health Engineering (DPHE) has been conducting a feasibility study for drainage improvement of the small townships/municipalities for the next 30 years. No doubt, this study will change the socio-economic life of people; more importantly give a framework that will help in making water management policy for these areas.

Dhamrai, one of a Municipality among the GOB declared 148 municipalities, with a population of about 56,777; has been selected as a study area as it is situated in an important economic hub in the North Central Region of Bangladesh. Despite of its location in the North Central Region significant part of its area is affected by external flood. The parts of north-west and south-east are flood affected. Two earthen borrow pit exists along both sides of the Dhaka- Aricha highway. Large part of its area drains out storm water into the borrow pit through different bridges and culverts. But these borrow pits are encroached by local inhabitants which at present scenario is hampering the natural drainage route to the desired outfall. Moreover, many of the drains fall into relatively low lying areas in a haphazard ways, thus causing drainage congestion and water logging problems in some places after heavy rainfall.

1.2 Objectives

The major objectives of this study is conducted through

i) Analyzing the topography, landuse pattern and existing drainage system within the study area.

ii) Planning of a drainage system and storm-runoff assessment for the proposed drainage system using modified rational formula.

iii) Design of the proposed drainage system for the selected study area to reduce inundations caused by seasonal rainfall.

iv) Application of Urban Drainage Module (UDM); model B concept of MIKE11 (developed by DHI), to verify the adequacy of the proposed drainage system under different rainfall scenarios to provide a viable drainage network for the study area.

v) Incorporation of the rainfall-runoff obtained from the UDM into the North Central Regional Model (NCRM) developed by Institute of Water Modelling (IWM), Bangladesh, to reflect the regional hydrological influences on that area.

1.3 Method Overview

The approach of assessing the study consists of the following steps: (i) frequency analysis of historical rainfall and water level data to obtain the design year characteristics, (ii) planning of the proposed drainage system with identification of possible routes and outfalls, (iii) surface runoff assessment for the proposed drainage system using modified rational formula, (iv) design of the proposed drainage sections against the flows estimated from the rainfall-runoff model to check its performance against extreme event phenomenon, (v) setting up the hydrological model, (vi) setting up the hydrodynamic model and (vii) flood and damage map analysis. Figure 1.1 gives a schematic understanding of the strategy of the work procedure of this study to accomplish the objectives.

Rainfall-runoff model was created through the application of Urban Drainage Module (UDM); model B concept of MIKE 11 Software (1-D Modeling) for hydrological modeling. Hydrological model is a surface model that uses the kinematic wave computation considering the gravitational and frictional forces only. The necessary information to set up the hydrological model was taken from topographic survey data, several literature reviews, satellite images/ aerial photograph, physical process data and data obtained during the analytical analysis carried out regarding the surface runoff assessments. More details about this procedure can be found in Section 5.2. Hydrodynamic model was set up to identify the regional hydrological influences on that area. This task has been carried out through the incorporation of data including the network, cross sections, boundary conditions and hydrodynamic parameters for the concerned area. Quantitative data such as water level was used to perform the calibration of the hydrodynamic model. The hydrograph of the simulated water level was compared to the recorded water level hydrograph. In order to make the curve fit the recorded water level curve, some parameters were adjusted. More details about this procedure can be found in Section 5.3.

Three different scenarios for two different case studies were formulated and simulated to test the drainage system's capacity and future function. The results of the simulations for each scenario were analyzed individually in order to evaluate the possible risks of future inundations in the study area.

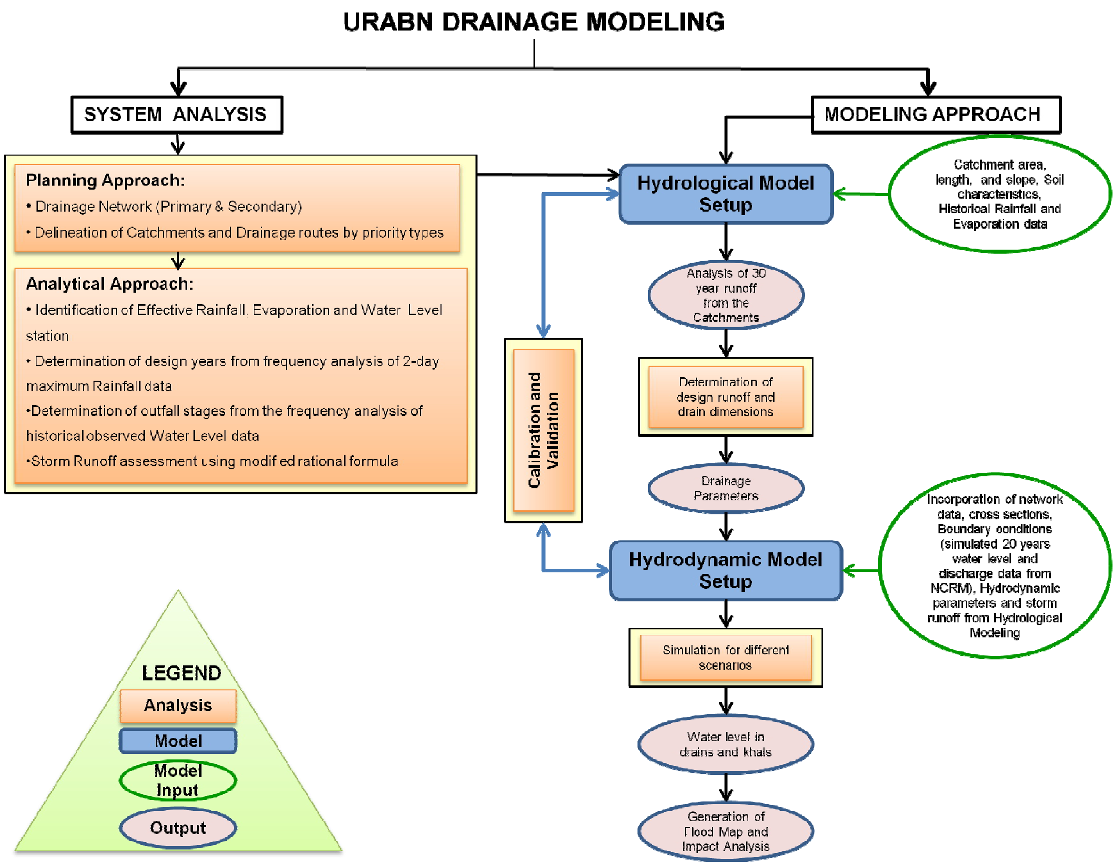


Figure 1.1: Activity Flow Diagram for the Study

1.4 Limitations of the Study

Following are some of the limitations of this study

i) Due to limited rainfall stations of Bangladesh Meteorological Department (BMD) and Bangladesh Water Development Board (BWDB), no station was found within the study area. For this reason, rainfall gauging station Savar (R031) of BWDB, which is a neighboring station of the study area was taken under consideration to assess the storm intensity of that area.

ii) As Intensity Duration Frequency, (IDF) curve of the study area is not available, so to assess the storm intensity, known design storm intensity of Dhaka was taken as a base to apply a conversion factor so that it can relate rainfall events between Dhaka and the reference station for the concerned area.

iii) There is no measuring hydrometric station in and around the study area which is necessary to calibrate the model. Hence, the calibration and validation stations are available from BWDM, IWM etc which are far away from the study area.

iv) Due to the lack of a planned and adequate drainage system it was not possible to set up model for the existing drainage system.

v) The amount of impervious surfaces has a big impact on the result in the model. To have access to such accurate data makes the model more reliable. However such information was not available and an interpretation from an aerial photo was made. This results in less accurate determination of the amount of impervious surfaces.

vi) Spot levels along the existing roads have been used to prepare the Digital Elevation Model (DEM) for the study area for visualization of the flood plain topography, which may provide some higher values in respect to the actual value of the land surface. In addition, detailed spot elevations in the range of 10-40 cm are required to obtain a DEM of good resolution, which was not followed in this study as the core areas were covered and obstructed by physical features.

vii) No socio-economic survey data regarding the damages incurred by the households, economic enterprises, urban infrastructures, agricultural and industrial was collected during past events related to flood and water logging. So, a damage function matching the scenarios of our study area could not be developed. Hence, the grid cells affected by floods from the rasterised landuse shape were analyzed to develop damage map.

1.5 Assumptions

Some of the assumptions made for this study are

i) Rainfall data obtained from the neighbor station of the study area is assumed to be applicable for Dhamrai Municipality.

ii) Location of roads, residential areas, commercial areas and all the features were based on existing data in form of GIS-layers conducted by the topographic survey team of Institute of Water Modelling (IWM), which are assumed to have the accurate coordinates.

iii) The runoff and storage co-efficient for each of the planned catchments and zones has been taken as 0.4 and 0.7 respectively considering the fact that the agricultural lands at present may built up in near future to residential areas with detached houses as no data was found regarding the urbanization and future extensions.

iv) The scenarios of the future situation (i.e. design year rainfall events and water level stages) are based on HYMOS analysis, assuming the analysis to be appropriate for this study.

v) All proposed drains within the area are assumed to be secondary and has been designed against 2 year return period. The task was conducted following the "Urban Drainage Manual", May 1998 of Local Government Engineering Department and prepared by DHV and AQUA Consultants and was assumed to be accurate.

vi) The storm runoff assessment using the modified rational formula was assumed to be correct and the rainfall runoff model was adjusted against the storm runoff obtained from the analytical analysis of the drains.

vii) While setting up the rainfall- runoff model, physical process data inputs i.e. wetting loss, depression storage, infiltration capacity was incorporated through available literature reviews as it is difficult to find the actual variation of the parameters with soil characteristics and antecedent moisture conditions. Moreover, no accurate data was found regarding the soil parameters for the study area.

viii) The impact of garbage, litter and sediments in the drains are assumed to have a small effect of the out coming result and were neglected in the model.

ix) As no socio-economic survey data regarding the damages in the past events are available, the damage map has been developed assuming flood depth in the range of (30-90)cm as low damage, (91 to 180)cm as moderate damage and (181 to above 360)cm as severe damage.

1.6 Structure of the Report

i) Chapter 1**…..**(Introduction): Briefly discusses the present and future status of the drainage system, objectives, method overview, limitations, assumptions and structure of the study.

ii) Chapter 2**…..**(Literature review): Describes in detail about the previous study carried out on urban drainage modeling in different countries with different methodology.

iii) Chapter 3**…..**(Profile of the study area): Describes the location, topography, climate, hydrology, existing water supply, drainage and sanitation situation within the study area.

iv) Chapter 4**…..**(Data Collection and Analysis): Describes the type of data collected in this study and the methods used to analyze and processing of the data for the study.

v) Chapter 5**…..**(Urban Drainage Model): Describes the setting up of the hydrological and hydrodynamic model.

vi) Chapter 6**…..**(Results and Discussion): Presents the analysis of the data and discusses the major findings.

vii) Chapter 7**…..**(Conclusions and recommendations): Major conclusions and recommendations are included in this final chapter.

Chapter 2
LITERATURE REVIEW

2.1 Introduction

Computer models for urban drainage networks are created to replicate rainfall events. This is useful when evaluating the function and capacity of the network. Computer models can also be utilized in order to simulate urban flooding. The models give an overview of the drainage network when evaluating flood risks in urban areas. Different scenarios regarding for example increased precipitation and urbanization can be simulated in the model to give a forecast of the future situation. As infrastructure is expensive to construct and maintain, this type of simulation and analysis can emphasize the focus on the most critical points of the drainage network.

Many studies have been made on the subject of urban flooding and a few specific studies have been chosen to demonstrate different procedures when performing computer modeling of urban flooding. There are several approaches to assess urban flooding and one method is one-dimensional modeling. This approach uses rainfall-runoff models which consist of an artificial hydrological model and a hydraulic model. Computer software that can be utilized to construct a rainfall-runoff model is for example Urban Drainage Module (UDM); model B concept of MIKE11 (developed by DHI), MIKE MOUSE (Model of Urban Sewers) or MIKE URBAN by DHI and SWMM (Storm-Water Management Model) by EPA. Three studies made on one-dimensional modeling can be found in Section 2.1. Another approach is to use two-dimensional surface modeling, where flooding can be simulated by overland flow. These types of models can give a clear overview of the locations of flooding as the flooding can be presented as a two-dimensional model. Two studies made on two-dimensional modeling can be found in Section 2.2. It is also possible to combine one-dimensional models and two-dimensional models to get a better understanding of the connection between the surcharged drainage network and overland urban flooding. Two studies made on coupled 1-D and 2-D modeling can be found in Section 2.3.

After reading these studies, the intention of this study was to perform a one-dimensional rainfall-runoff model combined with a two-dimensional surface model. This would make it possible to identify locations in risk of flooding and assess the spreading of any flooding. There would also have been a possibility to present the results on a two-dimensional surface, including houses and streets. This would have made the presentation of the results easy to visualize and comprehend. Detailed information about the elevations is important when creating a two-dimensional elevation model (Mark *et al.* 2004). It is also important to choose the appropriate method to implement the houses in the elevation models (Schubert *et al.*, 2008). Elevation data and information about the slope in the area is also important in order to simulate the spreading of flooding (Schmitt *et al.*, 2004). However, the necessary information such as detailed elevation data and existing drainage system was not available and thereby a creation of a two-dimensional model was not possible. Therefore, a one-dimensional model was utilized to simulate the urban flooding in the study area.

The literature studies show that one-dimensional modeling to simulate urban flooding is a conventional approach which can give promising simulation results (Mark *et al.*, 1998). A one-dimensional model can also be extended in the future to include a two-dimensional model or a statistical tool to assess failure probabilities of different part of the drainage network (Thorndahl and Willems, 2008). No appropriate study was found about simulating open drainage network with a one-dimensional model, but several studies where found made on closed underground drainage networks. So simplifications that can be made in the model, when simulating an open drainage network with different types of cross sections, were not found in any studies.

2.2 One Dimensional Modeling on Urban Flooding

Dhaka, Bangladesh

A MOUSE model was created in order to simulate flow and pollutant transport in the city sewer system in Dhaka (Mark *et al.*, 1998). There were big problems with flooding in the city and the flooded water depth could in some places be 30 – 50 cm. The flooding occurs even at low rainfall depth and this creates large infrastructural problems when roads are flooded (Mark *et al*., 1998). Flooding causes long-term economical and environmental damages to the infrastructure, such as basement flooding which is a common problem during flooding. The model simulates the flow inside the sewer system and also flow on the streets (Mark *et al*., 1998).

The model was verified against prior flood records in the city (Mark *et al.*, 1998). After the verification the model could be used to test suggested improvements to the system so the best and most cost efficient solution could be chosen. The result of the simulations was presented using geographical information system (GIS) in the computer software ArcView.

The results of the modeling in Dhaka showed that the model performed a good reproduction of the flooding in the city according to the flood records (Mark *et al*., 1998). The model will in the future be used to optimize the sewer system in Dhaka.

Frejlev, Denmark

In the Danish town Frejlev, a MOUSE model was created and was with a statistical tool called “first-order reliability method (FORM)” (Thorndahl and Willems, 2008). The aim of combining a MOUSE model with a statistical tool was to find probability of failure of specific component in the sewer system. Such failure can be overflow to receiving water, surcharge or flooding. In Frejlev the sewer system is an underground system that has the possibility to overflow to a nearby stream. In the sewer system, detention storage has been built in order to prevent overflowing of the system (Thorndahl and Willems, 2008). The catchment area was 87 ha and there are approximately 2000 inhabitants in the city.

The conclusion from this study was that the implementation of FORM was applicable when trying to estimate the probability of failure in the sewer system. An advantage compared to traditional methods was that the simulation time can be reduced to 1% of the simulation time in the traditional method (Thorndahl and Willems, 2008). But the simulation with FORM only presents results from one manhole at a time whereas the traditional method presents results from all manholes in the model.

The implementation is only verified against a catchment where the transport of water is done by gravitational forces and not with a catchment with many pumps (Thorndahl and Willems, 2008).

Senai Town, Malaysia

The study area was the Senai Town which is situated in an important economic centre in the mid-southern region of Johor State in South-East Malaysia close to the border of Singapore. Flash floods, water pollution and ecological damage are associated with storm water in Malaysia. To solve future problems with flooding in the region, the Government of Johor carried out a Drainage Master Plan (DMP) for Bandar Senai (DID, 2005a). The purpose of the DMP was to indentify existing drainage problems and propose long-term improvements with a projected year of 2020. The objective of this study was to identify areas with risk of flooding today and in the future.

A rainfall-runoff model was created in the computer program MIKE URBAN in order to forecast the future situation in the drainage system. The necessary information to create the model was taken from the DMP and collected from onsite observations. Quantitative data such as rainfall and water level was recorded in order to perform a calibration of the model during the 24th of October to the 18th of November 2008. The tributary Cabang Sungai Senai Fasa was the object of this study which has a catchment area of about 33 hectares and a length of almost 1 kilometre. The drainage network was almost entirely an open drainage system and consists of concrete lined channels and culverts of different dimensions. At low-lying areas in the Sungai Senai catchment, flash floods have occurred in the past due to insufficient capacity in the drainage system. Additional cause was the backwater effect from the river Sungai Skudai (Reuterwall and Thoren, 2009).

Four scenarios were defined to evaluate the drainage system’s capacity and future function. These scenarios were based on future changes such as projected increased precipitation, backwater effects from connected rivers, some suggested improvements of the drainage network found in the DMP and effects of exploitation around the study area. In order to simulate the four scenarios, the DHI computer program MIKE URBAN was used to create a rainfall-runoff model which consists of a hydrological and a hydraulic model. A calibration was performed using the collected rainfall- and water-level data (Reuterwall and Thoren, 2009).

The recorded rainfall was utilized in the simulations which showed that two sections had a large risk of flooding with today’s situation. The results of the simulations of the future scenarios indicated a small impact of increased precipitation on the drainage system. Backwater effects from the rivers had a large impact on the water level in the low-lying parts of the drainage network. The suggested changes by the DMP resulted in a lowering of the water level in the overall system. Future exploitation was simulated by increasing the catchment, which resulted in severe flooding in the upstream part of the drainage network. None of the scenarios indicate any additional areas in risk of flooding in the future compared with today's situation (Reuterwall and Thoren, 2009).

2.2 Two Dimensional Modeling on Urban Flooding

Potential and limitations

The problems with urban flooding are from minor to large ones, ranging from water entering the basements of some houses to major cities being flooded for days. In the industrialized part of the world these problems are mainly due to insufficient capacity in their sewer system during heavy rainstorms (Mark *et al.*, 2004). Regions in South/South-East Asia suffer more often of much heavier local rainfall and lower drainage standards. Together with the fact that cities in these regions are growing rapidly without the funds to adapt their existing drainage system, these problems are becoming more urgent (Mark *et al.*, 2004).

In history there are several examples of urban flood problems. For example, In Mumbai in India in 2000, 15 lives were lost when the water depth reached 1.5 m, 17.000 telephone lines ceased to function after flooding occurred and electricity was cut off. Bangkok was flooded for 6 months in 1983 which caused both the loss of lives as well as infrastructural damages of about $146 million (AIT, 1985). In 2002, Jakarta in Indonesia suffered from heavy rainfall which extended floods to the city centre, forcing 200,000 people from their homes and killing 50 people nationwide (Bangkok Post, 2002).

The view on damage when water flows on urban surface varies. (Konig *et al*., 2002) divides damages into categories:

i) Direct categories – typically material damage caused by water or flowing water.
ii) Indirect damage – e.g. traffic disruptions, administrative and labour costs, production losses, spreading of diseases, etc.
iii) Social consequences – negative long term effects of a more psychological character, such as decrease of property value in frequently flooded areas.

As well as damage on properties and goods, urban flooding can cause massive infrastructural problems and enormous economic losses regarding production. (Mark *et al.*, 2004)

In the strive for understanding and reducing urban flooding many cities in the developed part of the world use computer-based solutions to manage local and minor flooding problems. Using software such as MOUSE, Info Works and SWMM it is possible to create computer models of the drainage or sewer system in order to understand the complex relation between rainfall and flooding (Mark *et al*., 2004). As the existing conditions have been analyzed it is possible to evaluate a mitigation scheme and implement the optimal scheme. Few studies have been made on urban flooding with a combination of conditions in the surcharged pipe network and the flooding on the surface of the catchment (Mark *et al*., 2004). The ones made have dealt with urban flooding as a one-dimensional (1D) problem and a 2D model can be considered as a benchmark for the 1D model (Schmitt *et al*., 2002). Currently, a model that combines 1D pipe flow model with a 2D hydrodynamic surface flood is being developed (Alam, 2003).

Physical processes such as the hydrological process, the hydraulics of the drainage system, the digital elevation model (DEM), the flow exchange between the streets and the pipe system are all involved in urban flooding (Mark *et al.*, 2004). The digital elevation model gives information about the land elevation and requires detailed spot elevations. It is recommended for the intervals of the spot elevations to be in the range of 10-40 cm in order to obtain a good resolution and cover important details in the area (Mark *et al.*, 2004). Other technical requirements that are necessary can be summarized as:

i) Dynamic flow description: by using a dynamic wave model, the model includes backwater effects and surcharge from manhole including rapid change of water level.

ii) Parallel flow routing: when surface flooding takes place, it is not necessary that the flow direction on the streets have to be the same as the flow direction in the pipe system.

iii) GIS interface: GIS is an important tool in order to provide input data and to display the results of simulation of urban flooding. The application of GIS together with the DEM of the study area, it is possible to calculate the surface storage. The results of the simulation can be readable in flood inundation maps which are produced by overlaying of water surface and DEM, giving the flood depth map. (Mark *et al.*, 2004)

These facts have also been pointed out by (Maksimovic and Prodanovic, 2001). Other physical processes like evaporation and infiltration are important to consider if they influence the conditions of the urban flooding (Mark *et al.*, 2004). A comparison of accumulated evaporation to accumulated rainfall during the period of rain and flooding is necessary in order to know whether evaporation should be included in the model simulation. Evaporation does not affect the simulated maximum flooding if there is only a small evaporation compared to the accumulated rainfall. When it comes to drawbacks and limitations the most important inaccuracy is the dealing with street channels as prismatic and of the flow as one-dimensional (Mark *et al*., 2004).

Some simplification is always involved when engineering predictions are made. Urban flooding is a complex phenomenon and it is impossible to include all details in the modeling (Mark *et al*., 2004). However, this should not hinder from make attempts in using a 1D approach, especially when internal floods are caused by heavy rainfall. Accurate simulations of local conditions on a small scale are difficult to perform. On the contrary, promising results are likely to be achieved when simulation of urban flooding on a larger scale (Mark *et al*., 2004). The combination of 1D hydrodynamic modeling and GIS is believed to be a cost efficient system for drainage systems suffering for urban flooding when it comes to planning and managing (Mark *et al*., 2004).

As 1D modeling approach is sometimes insufficient, future approaches may use a hydrodynamic pipe flow model beneath ground in combination with a full 2D hydrodynamic model in order to be able to describe the surface flow (Mark *et al*., 2004).

Erzhutten, Germany

A dual drainage model called RisUrSim was created to be able to simulate interaction between flow in the underground sewer system and overland flow when the sewer system is surcharged (Schmitt *et al*., 2004). Flooding can occur even if there is no overland flow. Backwater effects from the sewer system cause these types of flooding in the basement of the nearby houses. Wastewater from the sewer system goes into the basements via the outgoing wastewater pipe that is connected in the bottom of the basement. Produced wastewater in the building cannot exit the house, which will increase the flooding of the basement. These types of flooding mostly occur when the sewer system is combined, meaning that the drainage water and wastewater is lead in the same pipe.

Surface flooding depends on local constraints in the sewer system. The spreading of these flooding depends on ground slopes and walkway curb heights. These properties are harder to simulate because it requires a large amount of data in the model, which is often not available (Schmitt *et al*., 2004).

The conclusion of this study was that to simulate urban overland flooding in an underground drainage or sewer system, an underground hydraulic structure must be directly linked to an overland flow routing model (Schmitt *et al*., 2004). This allows the hydraulic structure to flood via the manholes in the hydraulic structure. When the water has exceeded the hydraulic structure the water is routed in the overland model. The water that is routed overland can enter the drainage or sewer system again via the manholes when they are not surcharged (Schmitt *et al*., 2004). If possible, the water can also flow overland to other manholes in the drainage or sewer system. These manholes can be upstream or downstream the original manhole that was flooded. To get an accurate description of this flooding routing to other manholes, the overland model must be detailed.

Glasgow, Scotland

An overland flow model was constructed in order to simulate the flow of water in an urban catchment in Glasgow (Schubert *et al*., 2008). Using remote sensing technology the different types of surfaces were identified. A finite element method was used to generate a mesh structure, where each mesh triangle represents a specific surface (Schubert *et al*., 2008). Two different methods were used to simulate the properties of buildings in the catchment area. The “building-hole method” deletes the mesh where the buildings are represented after the mesh is created (Schubert *et al*., 2008). This means that the flood is not calculated in these meshes because they are deleted. The second method is called “building-block method” and instead of deleting parts of the mesh it increases the elevation of the mesh where the buildings are represented (Schubert *et al.*, 2008). This way the flooding is simulated around the building but the flooding must be severe in order to flood the whole house. These two methods were combined with three different sizes of the mesh structure.

The conclusion was that both methods were equally good at reproducing flooding in Glasgow (Schubert *et al*., 2008). The difference was that building-hole method was 30% faster to compute than the building-block method (Schubert *et al.*, 2008). However, when the mesh was coarser the building-hole method presented the best result. A "non-building method" was also evaluated together with the building-block method and the non-building block method showed a good result at coarser mesh (Schubert *et al.*, 2008). The building-block method was therefore better to used when the mesh was smaller.

2.3 Coupled 1-D and 2-D Modeling on Urban Flooding

One-dimensional models can describe flows in channels efficiently while two-dimensional models can make good results for overflow. The integration of a two-dimensional model with a one-dimensional model can give a full play to their own characteristics and advantages and resolve the problem in spatial resolution and calculation accuracy, frequently encountered while two models were used separately.

Jinan City, China

In this study, a 1D-2D hydrodynamic model using Mike Flood was constructed and applied in Jinan city, Shandong province to simulate the flood scenarios for different flood events. Initially, the Mike 11 model was calibrated and validated for the rivers of Jinan urban areas and subsequently, the flood inundation was simulated using Mike Flood. The model simulated flood inundation is validated using observed historical flood data.

Mike Flood is a tool that integrates the 1D model and the 2D Mike 21 model into a single coupled model. There are three typical links in Mike Flood model, namely, standard link, lateral link and structure link, which were suitable for different applications. Lateral links were the mainly type used for simulating flood flowing through the two-dimensional domain. Lateral linkage explicitly couples Mike11 to Mike21 by modeling water entering the floodplain from the stream channel laterally. The flow from the stream channel to the floodplain is modeled using a simple weir equation. Momentum is not conserved using lateral links, due to the inability of 1D models to simulate cross-channel flow (Chuanqi and Wang, 2012).

A lateral link allows a string of Mike 21 cells to be laterally linked to a given reach in Mike 11, either a section of a branch or an entire branch. Flow through the lateral link is calculated using a structure equation or a QH table. This type of link is particularly useful for simulating overflow from a river channel onto a floodplain (Chuanqi and Wang, 2012).

Jinan city is frequently affected by floods. In the recent past years there have been major rainstorm events in 1963, 1964, 1987 and 2007. On August 26th, 1987, average rainfall in

Jinan city was 294mm, the city was in flood and the inundation area was 33 km2. On July 18th, 2007, Jinan experienced severe flooding with a rainfall more than 151 mm in about one hour in urban areas, causing extensive damages to property and human life.

The calibrated Mike Flood model was used to simulate the 5, 10, 20, 50 and 100 year rainfall events. The Mike Flood model was used to define flood levels water depths flood extents and flow velocities. The whole inundation and regression process was continuously, dynamically presented using Mike Flood. For the 100-year storm, the most serious inundation occurred at the 4th hour. At that time, an area of 65 % was inundated with different water depth. The deepest inundation was 4.1 m near the railway bridge in the middle and northern area. After the 8th hour, the flooding became regressed. After the 10th hour, all inundated flow was drained. At the flood peak, high velocities (3-5.3m/s) and supercritical flow occurred along the main South-North slope directions especially in the Erhuan East Road (Chuanqi and Wang, 2012).

The largest problem with this model is that a long time is required for the calculation. It is not feasible to run the Mike Flood model in real time. Therefore, Mike Flood model is used to run a wide range of pre-formulated flood scenarios for urban areas (Chuanqi and Wang, 2012).

City of Odense, Denmark

In the last couple of years several damaging floods have occurred in the city of Odense, Denmark. In order to better understand the reasons behind this and find adaptive solutions to reduce the impact of urban loading, an integrated urban flood study was initiated. In the study numerical coupled 1D-2D models were developed for two catchments. The models simulate the drainage conditions in the sewer network and overland flow and proved useful in urban flood management.

A fully integrated software system, MIKE FLOOD, was used to model the pipe flow, the surface flow and the interaction between them (DHI, 2008). MIKE FLOOD consists of a two dimensional hydrodynamic surface model (MIKE 21) with a dynamic link to a fully dynamic one-dimensional collection model (MOUSE). MIKE FLOOD couple the two models with a link flow equation describing the bidirectional flow through the manholes.

To describe the exchange of water between surface and the manholes three equations are implemented in the model; orifice equation, weir equation and exponential function. In general, it requires an unrealistic amount of detailed data on every manhole in order to apply physical dimensions for the orifice and the weir equation, so from an engineering point of view the physic of the equation is unimportant. It was found that the exponential function was easy to use and produced reliable results in respect to differences between water levels in the two models (Nielsen *et al*., 2008).

The method was applied on two areas in the city of Odense, Denmark. The simulations were validated using photos taken during an extreme rain event. The results obtained shows that the method is capable of successfully describing behavior across an urban flood plain. The model was used to identify and optimize different solutions to reduce flooding of a gymnasium. Water will be routed towards an athletic stadium where the post recovery cost of the flood event is expected to be much lower (Nielsen *et al*., 2008).

To obtain plausible results, detailed knowledge of data from the collection system and high quality digital terrain model is needed. However, if this data is available the method is useful for engineering purposes. To decrease the computational time a grid size of 1-4 meters was found to be sufficient. Furthermore a one-dimensional collection model can be used as a first step to describe the interaction between the pipes and overland flow via open channel flow. Rainfall was not applied to the surface model, but to the manholes in the collection model. This simplified method proved sufficient in this study. The model complex will be a good tool when assessing urban flooding or making emergency plans, i.e. regulating the terrain. The approach can also be used to generate flood risk maps visualized in a GIS-environment (Nielsen *et al*., 2008).

Chapter 3
PROFILE OF THE STUDY AREA

3.1 Location

The study area Dhamrai, was established in the year 1999 as a GOB declared small Municipality and classified as a 'B' class Municipality. It is situated at 45 km north-west from Dhaka district head quarters, in Dhamrai Upazila of Dhaka District under Dhaka Division. It is bounded by Bhararia union at the north, Bangshi River at the east, Kullah union at the south and Shambhag union at the west. It has an area of about 7 sq km with population coverage of about 56,777. It is comprised of 9 wards. It is a trading area, with good business activities. There are 500 commercial buildings inside the Municipal area along with 49 Industries. About 30% of the area in Dhamrai is low lying agricultural land. The location of the study area is shown in Figure 3.1.

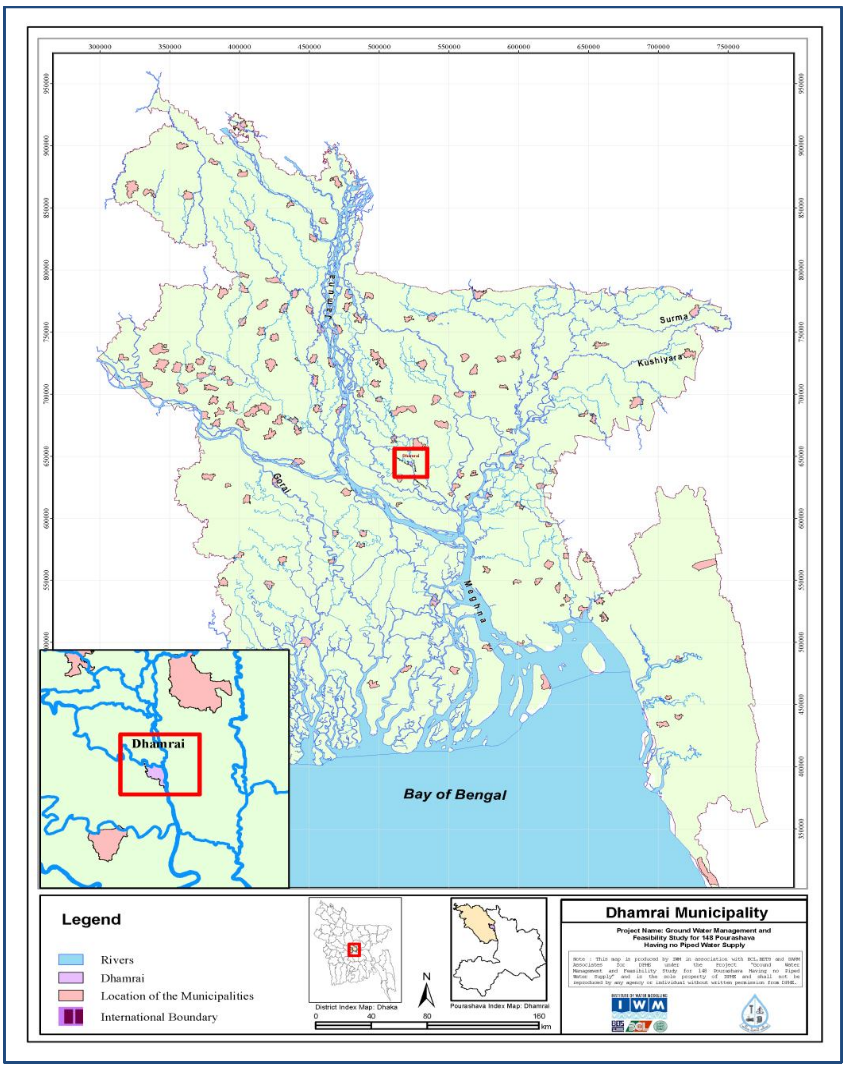


Figure 3.1: Location of the Study Area

3.2 Topograpy

The land elevation of the study area effectively ranges between 5.93 mPWD and 10.73 mPWD. It is assessed that only 3% land of the area is below 4.63 mPWD while 13%, 45%, 64%, 78%, 89%, 96% and 100% of the land are below 5.93 mPWD, 7.23 mPWD, 7.88 mPWD, 8.53 mPWD, 9.18 mPWD, 9.83 mPWD and 11.13 mPWD respectively. The use of present Municipality area can be broadly divided into lands for agricultural (30%) and non-agricultural (70%). Major settlements are in the areas of Wards No. 1, 2, 4, 5, 6, 7 & 8 with some scattered settlements in Wards No. 3 & 9. The area-elevation curve and DEM of the study area is shown in Figure 3.2 and Figure 3.3 respectively.

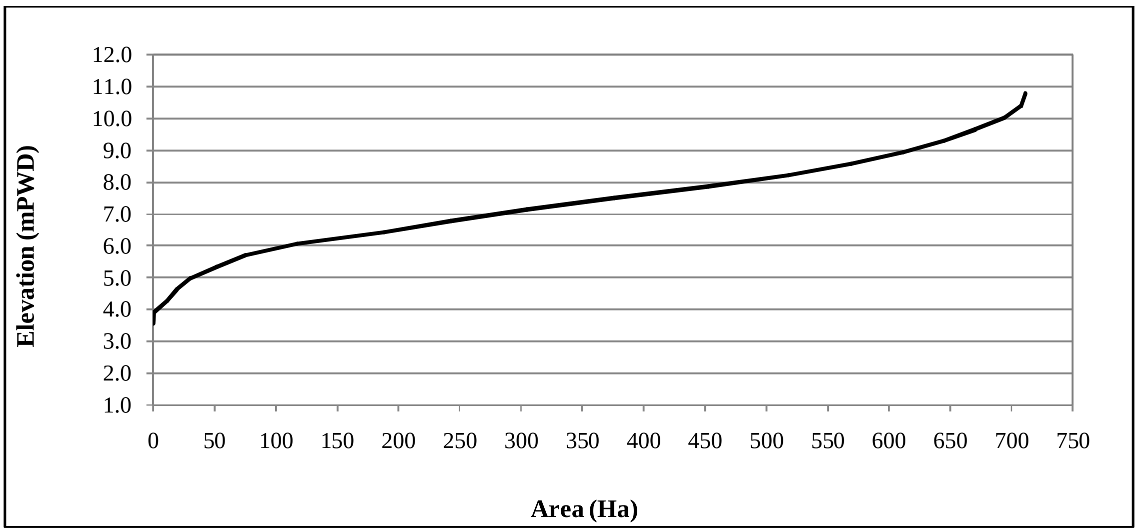


Figure 3.2: Area-Elevation curve of Dhamrai

3.3 Climate

The area experiences the Indian Ocean Monsoon climate. The area experiences four meteorological seasons: Pre-monsoon (March to May), Monsoon (June to September), Post-monsoon (October to November) and Dry (December to February). Average annual rainfall is in the range of 1700 to 2200 mm. About 70% rainfall occurs during the period from June to September. Average temperature range is between 25^0C to 31^0C. Maximum temperature may rise up to 40^0C and may go down to 6^0C. Average humidity remains at 80% to 90%.

3.4 Hydrology

The study area lies in the Bangshi River basin. Kekla, a contributory river of Bangshi, flows at the upper northern part of the study area. A khal named Baligao, flows towards south east of the study area which is connected to Bangshi river. Bangshi is a schematized river of the North Central Region Model (NCRM) developed earlier by IWM. The Bangshi is one of the secondary rivers in the North Central Region which carries perennial flows. North Central Regional Model (NCRM) covers the region bounded by the Jamuna, the Ganges, the Meghna

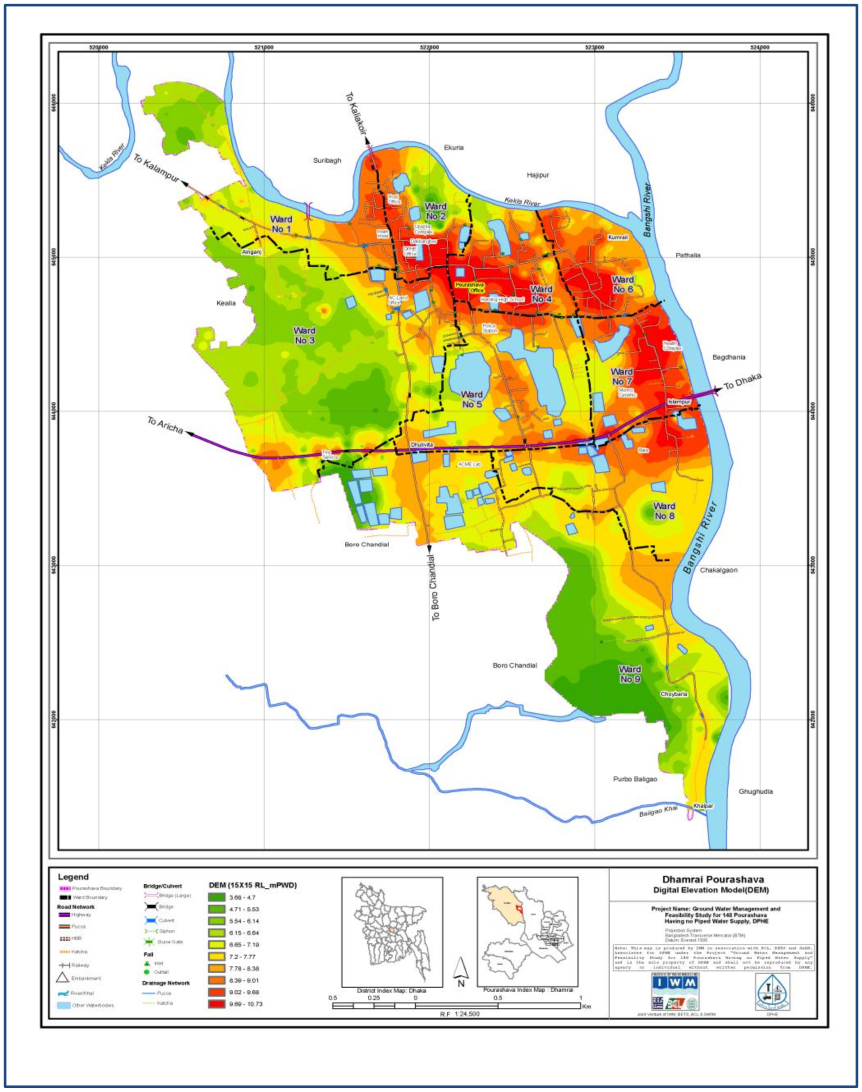


Figure 3.3: Digital Elevation Model (DEM) of Dhamrai

and the Old Brahmaputra; it includes the national capital Dhaka. Figure 3.4 shows the river system around the study area.

Significant part of its area is affected by external flood. The parts of north-west and south-east are flood affected. The area has its significant lands flood free while the rest of the lands are in the order of moderate-deep-shallow-very deeply flooded areas. It is assessed that 58% of the land of study area is above the average flood level while 7 % land is subjected to shallow depth (less than 30 cm) of flooding. The rest of the land ranges from moderate to very deep flooding. It is assessed that 14%, 14% and 7% of land is subjected to moderate (30-90 cm flood depth), deep (90-180 cm flood depth) and very deep (more than 180 cm flood depth) flooding in reference to average year flood. Figure 3.5 represents land classification for the study area.

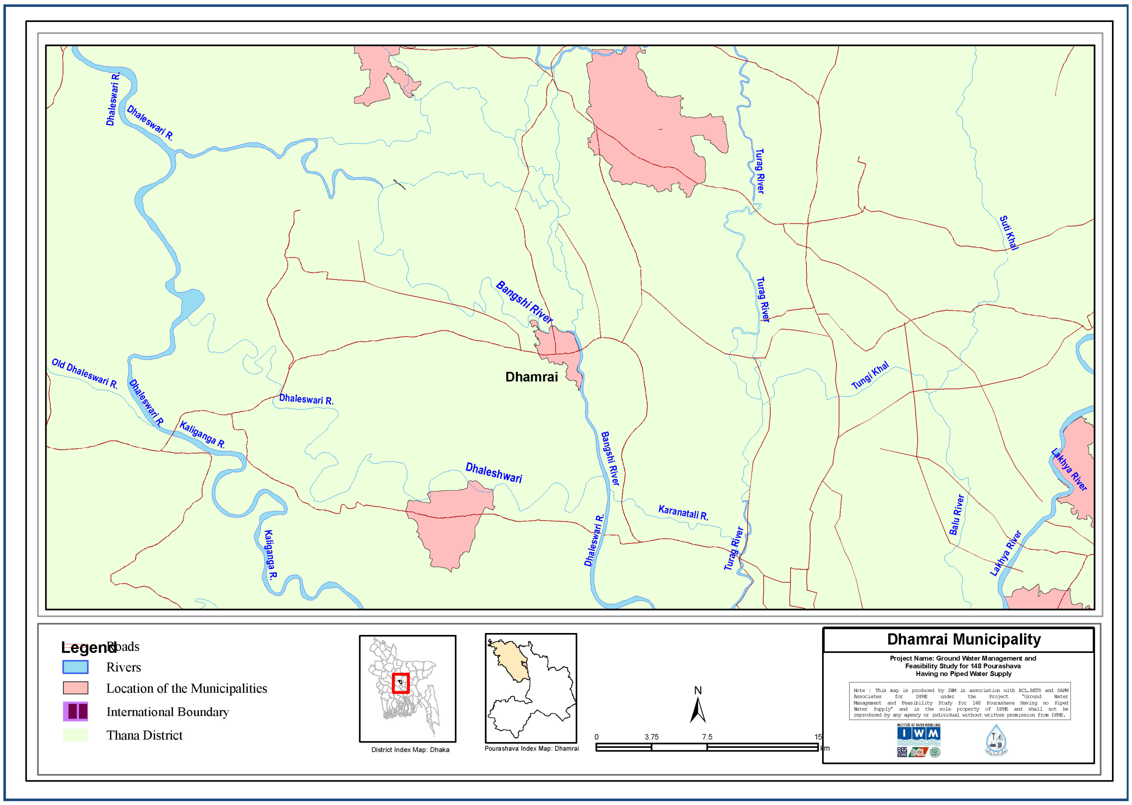


Figure 3.4: River system around the Study Area

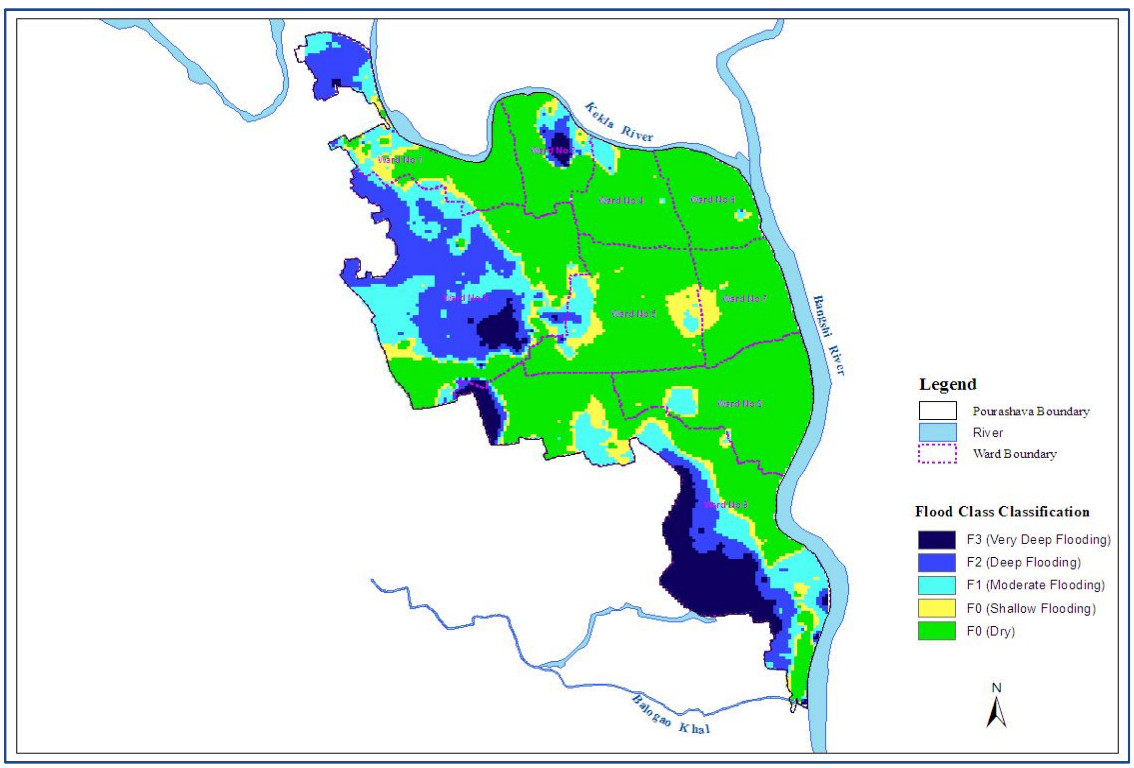


Figure 3.5 : Land Classification of Dhamrai

Kekla drains and routes the storm water from upper northern parts of the study area. On the other side, the existing Baligao Khal routes storm water from south-eastern part and finally drains and routes to Bangshi River. Two earthen borrow pits exist along both sides of the Dhaka- Aricha highway. These borrow pits, in respect of present scenario, are the convenient major drains which the Municipality needs to consider them effectively alive and route to the Bangshi River. But the borrow pits are encroached at reaches reducing their original section geometry as well as hydraulic connectivity.

3.5 Water Supply Situation

Inside the Municipality, there is no piped water supply network except that of Upazila Complex, Bata Shoe Factory complex and Health complex where Production Pump is installed. Shallow Hand Tube Well (HTW) and Production Tube Well (PTW) are being used by the general population to abstract ground water. There are about 4,002 Hand Tube Wells (HTW) in the area, out of which 2 Nos. has been installed by the Municipality. HTWs are arsenic free. There is no Rain Water Harvesting (RWH) facility in practice. It has 53 deep Production Tube Well (PTW), out of which 50 is private. The river water has been identified as source of drinking water to the municipal population along with the ground water source.

3.6 Existing Drainage Network

There exist few lined drains within the study area. The total length of pucca drain inside the study area is only about 3.5 km which is not sufficient for a Municipality having an area of about 7 sq.km. These can drain some local areas of the Municipality. As there is no planned and systematic drainage system within the area, many of the drains fall into relatively low lying areas in a haphazard way thus causing drainage congestion and water logging problems after heavy rainfall. The urban area is increasing and the degree of drainage concern is also increasing. The runoff resulting from rainfall fails to reach the outfall due to lack of planned and systematic drainage network system and also most of the drains are in partial flowing condition. There are a number of cross drainage structures within the area found during the topographic survey conducted by IWM. The existing drains and structures are shown in Figure 3.6.

Water logging occurs at some places due to unplanned construction of homestead and absence of a systematic drainage system. Gandhikul of ward-1, Bandimara & Bijornogor of ward-3, Taltola of ward-8 & 9 and Choto Chandial of ward-9 suffers from water logging during monsoon period for a couple of days.

3.7 Existing Sanitation Situation

The sanitary condition in Dhamrai is not satisfactory. Currently 75% of the household is under sanitation coverage. No treatment facility, disposal and sewerage system are available inside the area. There are about 1200 households with septic tanks. But in most houses, they use only sanitary pit latrines, and in pits, unhygienic overflow of sludge is observed.

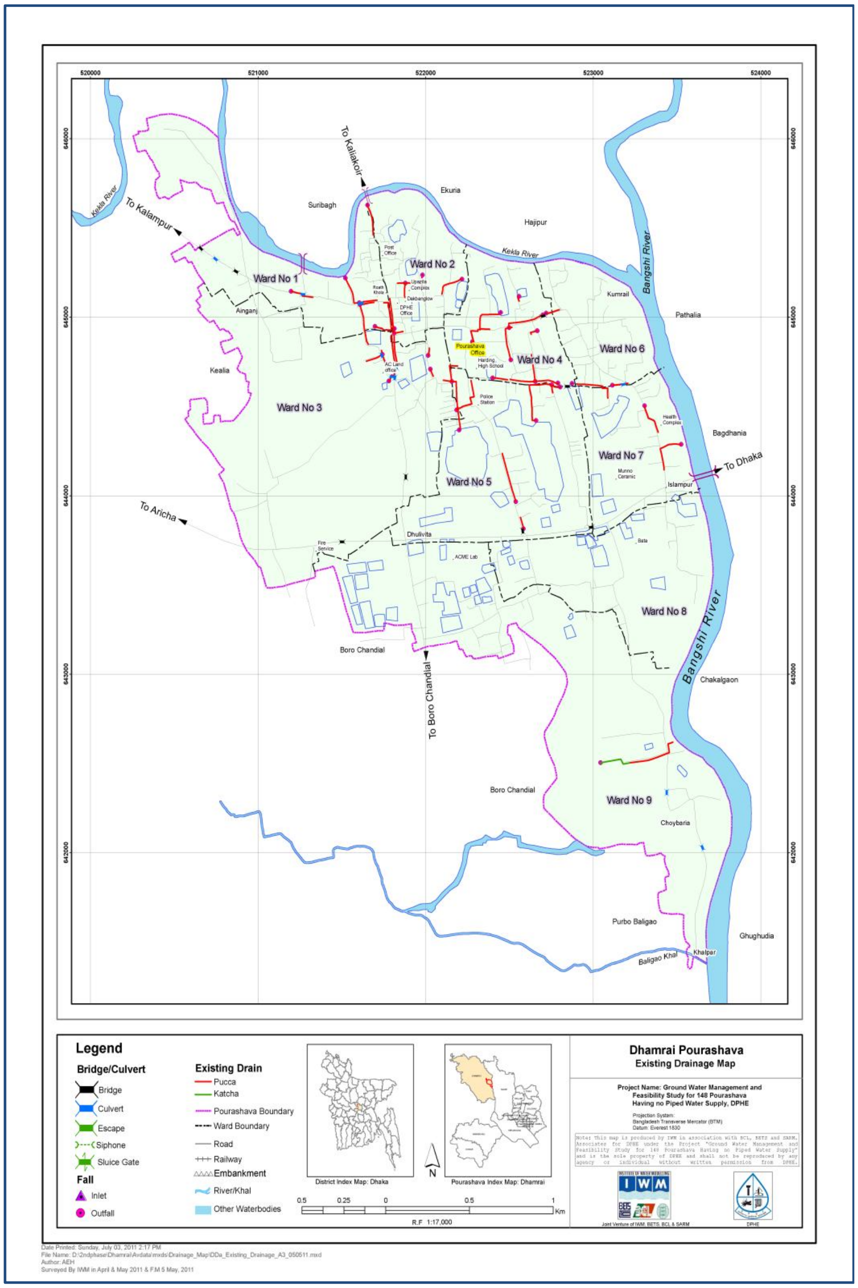


Figure 3.6: Existing Drainage Network of Dhamrai

Chapter 4
DATA COLLECTION, ANALYSIS AND PROCESSING

4.1 Data Collection

The study is primarily based on various types of data, which are utilized to stimulate the hydrological and hydrodynamic situation of the study area. Topographic, hydro-meteorological and hydrometric data are principal inputs of this study. This chapter explains data collection for carrying out the study, along with its available sources.

4.1.1 Topographic Data

Topographic survey was conducted regarding the-

i) Bench Mark (BM) and Temporary Bench Mark (TBM) establishments.
ii) Alignment survey (i.e. survey of existing roads, drains and surrounding rivers).
iii) Physical Feature survey (i.e. projection of Schools, Colleges, Mosques, Madrasha, Mandir, Hospital, Bus Terminal, Market, Electric Tower, offices, Police station, Post office etc).
iv) Land Level survey (i.e. geo-referenced position/co-ordinates in open lands).
v) Landuse survey (i.e. agricultural and non-agricultural) and
vi) Cross Sectional survey (i.e. river cross-sections and flood plain topography).

Source: All these data in GIS format (shape files and maps) has been collected from IWM.

4.1.2 Hydro-Meteorological Data

Two types of hydro-meteorological data were collected for the study. They are the Rainfall and Evaporation data.

i) Rainfall Data (Data Period April 1962 to April 2009): Savar (R031) rainfall gauging station with reasonable length of records is located in a close vicinity of the study area. It is selected as the reference station for assessment of storm intensity and development of design year event for the study area. It is also used for the set up of hydrological modeling for rainfall runoff model simulation.

ii) Evaporation Data (Data Period April 1964 to November 2009): Due to limited number of evaporation station, Dhaka station has been selected as the evaporation station for the study area, for the development of hydrological modeling.

Source: Rainfall and evaporation data has been collected from BWDB.

Location of rainfall, evaporation and water level stations in Bangladesh is shown in Figure 4.1.

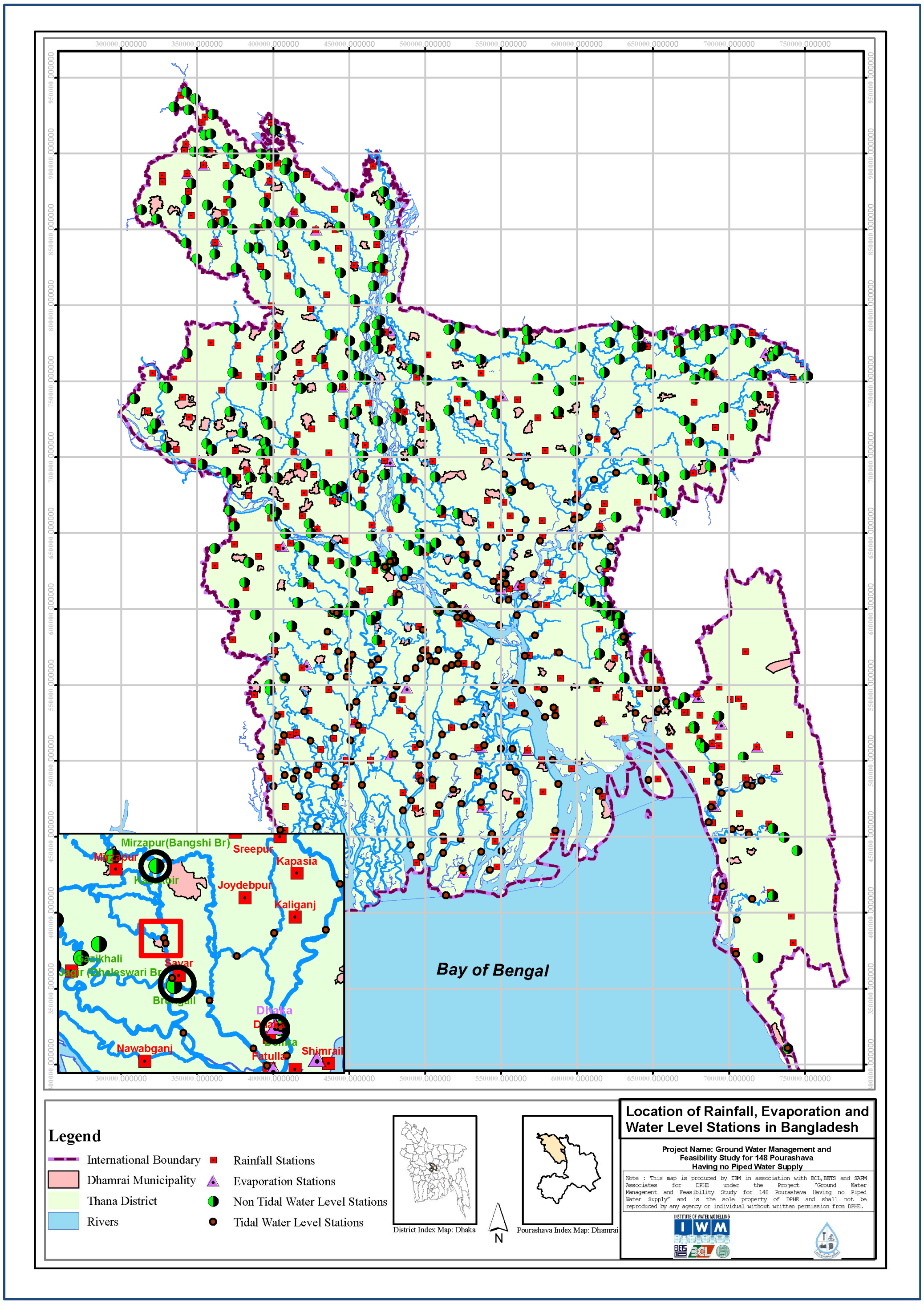


Figure 4.1: Location of Rainfall, Evaporation and Water Level Stations in Bangladesh

4.1.3 Hydrometric Data

Two types of hydrometric data were collected for the study. They are the Water Level and Discharge data.

i) Water Level Data: Kaliakoir (301), a non tidal gauging is in a close vicinity of the study area near the confluence of Bangshi and Turag River. Another station named Brungail Khal (303) is situated on the river Dhaleshwari. The Kaliakoir station is at a distance of 20 km upstream and the Brungail station is at a distance of about 10 km downstream from the study area. Both of the stations were used for slope analysis to obtain the average year and 1 in 5 yr water level at the study areas outfall. Observed water level data of Savar station was also used for the calibration of the hydrodynamic model.

Source: Water level data has been collected from BWDB. 20 year simulated water level data of the NCRM of IWM has been used for the set up of hydrodynamic modeling.

ii) Discharge Data: Due to very limited and poor databases of discharge data it was not possible to use observed discharge data for the set up of hydrodynamic model. For this reason, 20 year simulated data of the NCRM of IWM has been used for the set up of hydrodynamic modeling.

Source: Discharge data has been collected from 20 year simulated NCRM data of IWM.

4.2 Data Analysis and Processing

4.2.1 Rainfall

4.2.1(i) Rainfall Conversion Factor Calculation

Design rainfall storm intensity for the study area is assessed from the known design storm intensity of Dhaka applying a conversion factor which relates the rainfall events between Dhaka and reference station for the study area. This implies that the storm event of Dhaka and the study area is correlated by the conversion factor. The conversion of storms at Dhaka to storms at Dhamrai follows the “Urban Drainage Manual”, May 1998 of Local Government Engineering Department and prepared by DHV and AQUA Consultants.

Savar (R301) gauging station was taken as the rainfall station for the study area. Observed records of short duration (1988-07) of yearly 1-day maximum rainfall at Dhamrai is less than the base station Dhaka. It is found that the average 1-day maximum rainfall event at Dhamrai is 0.97 times than that of Dhaka and conversion factor for Dhamrai is estimated to 1.07. Consequently rainfall intensity at Dhamrai is assessed higher than Dhaka for same storm duration and frequency. Details on the calculation can be found at Annex A-1 and Annex A-2.

4.2.1(ii) Development of Design Years

In order to select the simulation year, frequency analysis have been carried out for the rainfall data of Savar station, for the drainage model study. In this context, 2-day maximum rainfall data for the period of 47 years (1962-2008) of the station is used. The analysis was carried out using HYMOS software of Delft Hydraulics. The analysis was done in Gumbel Distribution methods using statistical package HYMOS developed by Delft Hydraulics for hydrological analysis. The current study adopted the guidelines set out in Flood Hydrology Study (FAP 25, 1992).

4.2.2 Water Level

The nearest water level gauging is available at Kaliakoir (301) which is fairly calibrated by the regional model. In consideration of reliability at the location of calibration, the average year flood level at the study area is estimated to 7.14 mPWD by the adjustment of length and slope, in reference to Kaliakoir (301) and another station Brungail Khal (303) located about 20 km upstream and 10 km downstream from the study area. Water level data at this station is available for the period of 1978-95. Statistical analysis of maximum yearly water level is carried out to determine outfall stages of various return periods. The analysis was carried out using HYMOS software of Delft Hydraulics. The analysis was done in Gumbel Distribution methods. The current study adopted the guidelines set out in Flood Hydrology Study (FAP 25, 1992).

4.2.3 Analytical Analysis of Drains

The analytical analysis of the drain follows three parts: (i) Planning of the drainage routes with identification of catchments, zones and outfalls, (ii) Estimation of storm runoff for the proposed drainage system and (iii) Design of the drain sections. GIS has been applied to delineate catchments and drainage routes of Dhamrai. A number of drainage areas have been delineated as catchments for the study area based on the area of interest. Existing roads, Digital Elevation Model (DEM), infrastructure, homestead, contour maps, natural canals and rivers in and around the study area and the outfalls have been considered in delineating the drainage routes and catchments.

4.2.3(i) Drainage Improvement Plan

- **Identification of outfalls**

The Eventual outfalls for the present and expanding core area of the study area are in the Bangshi River, Kekla River and Baligao Khal. Re-sectioning of existing borrow-pits will be a major outfall for substantial parts of northern side and some parts of southern side of the Municipality. The outfalls of the proposed drains and drainage zones have been identified and exhibited in the following section.

- **Identification of proposed drainage system with catchments and zones**

About 58% land of the municipal area is above the average flood level. Proposed drainage system has been planned for the core area of the Municipality as well as for the extended area in near future in consideration of priority needs. The extension area has been considered by observing the satellite image and field surveys and interaction with the local inhabitants as no study or literatures were found regarding the future expansion of the study area.

The area of Dhamrai has been planned for improvement under gravity drainage system. The whole area has been divided into 22 zones for drainage improvement plan shown in Figure 4.2. Zones 1 through 7 are planned with proposed storm drains as they are in the core area or will be characterized as core area in near future. Rest of the zones is not planned with proposed storm drains while they are planned with their outfalls for future drainage details. Zones 8, 9 & 10 will eventually drain in the re-section of existing borrow pit (B1), right side of the Dhaka-Aricha highway and finally route and drain to the Bangshi River. On the other side zones 16, 17, 18 & 19 will drain in the re-section of existing borrow-pit (B2, B3, B4 & B5), left side of the Dhaka-Aricha highway and finally route and drain to Bangshi River through existing bridges and culverts and the proposed re-excavated borrow-pit (B1). Zones 12, 13, 14 & 15 will eventually drain and route to Kekla River. Zone 11 & 20 will drain in the Bangshi River. Zones 21 & 22 will drain into Baligao Khal and finally route and drain to Bangshi River.

S1 through S7 are the 7 major drains which are planned for the storm drainage of most of the study area. These drains have been proposed for the drainage of the study area in consideration of topography, existing infrastructure, present runoff and drainage pattern, and experiences and views of the local people.

S1 drainage system will drain storm water from some parts of Ward #3 & Ward #4 and almost whole parts of Ward # 2 to Kekla River.

S2 drainage system will drain storm water from major parts of Ward # 4 and some parts of Ward #6 to Kekla River.

S3 drainage system will drain storm water from major parts of Ward #1 to Kekla River.

S4 drainage system will drain storm water from some parts of Ward #8 & Ward # 9 to Bangshi River.

S5 & S6 drainage systems will drain storm water from some parts of Ward #3 & 5 to borrow-pit (B1) proposed for re-section along right side of the Dhaka-Aricha highway.

S7 drainage system will drain storm water from parts of Ward #8 to Bangshi River.

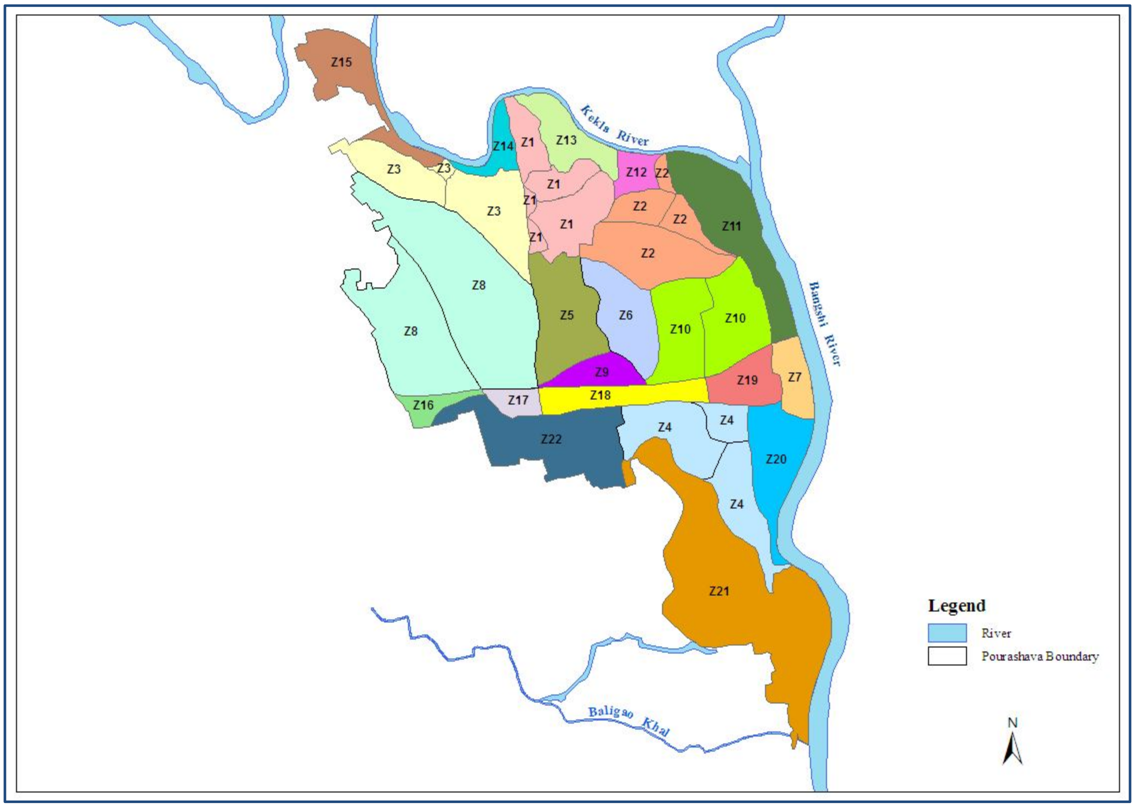


Figure 4.2: Proposed Drainage Zones for Dhamrai

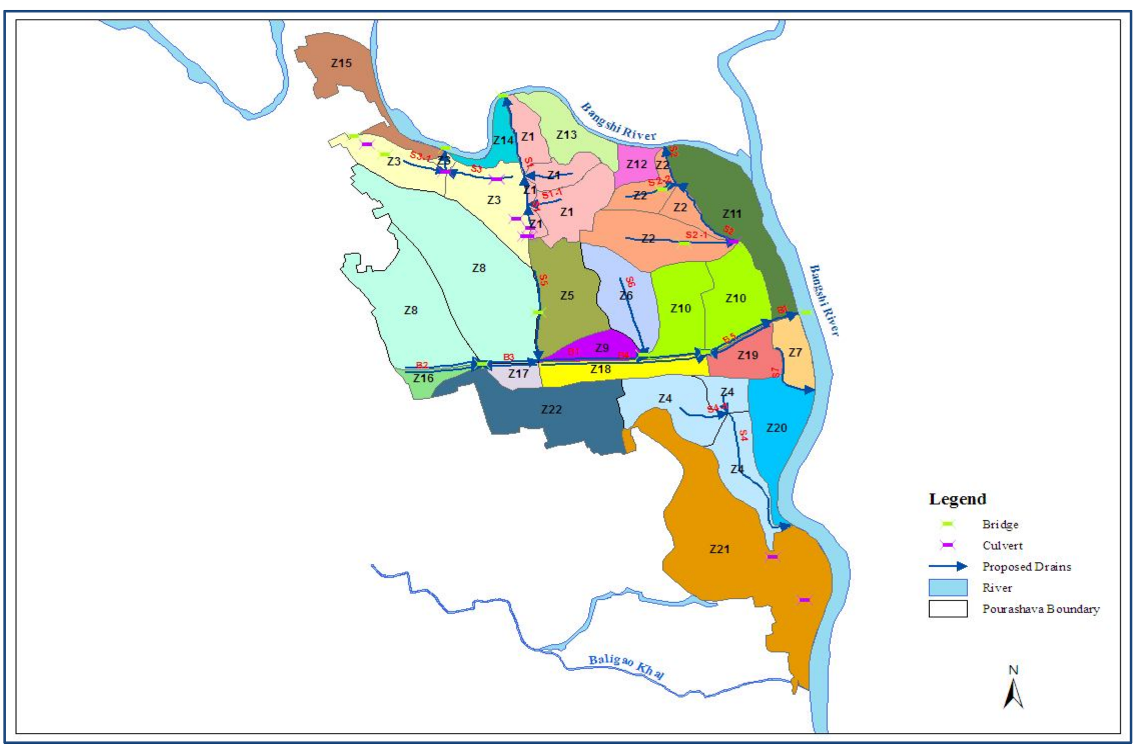


Figure 4.3: Proposed Drainage routes for the Planned Zones

S1, S2, S3, S4 and S7 drainage systems have priority needs while S5 and S6 drainage systems are proposed in view of future needs for the study area. The list of proposed drainage channels which has been considered for storm drain is given in Table 4.1.

Table 4.1 : List of proposed drainage system in Dhamrai

SL No	ID of Drain and Catchment	Length (m)	Area (ha)	Drain Type
1	S1-1	227	18	Secondary
2	S1-2	332	8	Secondary
3	S1	989	38	Secondary
4	S2-1	743	28	Secondary
5	S2-2	352	8	Secondary
6	S2	899	43	Secondary
7	S3-1	300	17	Secondary
8	S3	597	42	Secondary
9	S4-1	355	22	Secondary
10	S4	1217	48	Secondary
11	S5	661	31	Secondary
12	S6	602	23	Secondary
13	S7	494	11	Secondary

4.2.3(ii) Storm Runoff Assessment

The storm runoff assessment for the catchments and zones are based upon some selected design criteria. Those are briefly discussed below.

The Modified Rational Method has been used for estimation of peak flows for the proposed drains. It gives reasonably accurate result and is a widely used method for calculation of runoff for last few decades. The runoff by Modified Rational Method is:

Peak runoff, $Q_P = C_s C_r IA/360$

Where; Q = Peak runoff flow rate (m^3/s)

I = rainfall intensity (mm/hr)

C_s = storage coefficient

C_r = runoff coefficient

A = catchment area (hectares)

- **Rainfall intensity**

Rainfall intensity with a 5 year return period is employed for design of canal improvement and primary drains. Rainfall intensity with a 2 year return period is employed for design of secondary drains. The design storm duration is equal to the time of concentration for the drainage area under consideration. In this study, all drains were considered to be secondary that’s why all proposed drains have been design against 2 year return period. Conversion factor for Dhamrai rainfall intensities was found to be 1.07.

- **Time of concentration**

Time of concentration (T_c) is generally defined as the longest runoff travel time for contributing flow to reach the outlet or design point, or other point of interest. It is frequently calculated along the longest flow path physically. The time of concentration is the sum of time of entry (T_e) and travel time (T_t). Time of entry is the time taken for runoff from the farthest point in the contributing area to flow over the ground and enter into the drain. Travel time is the time taken for runoff to flow through the drain.

The time of entry (T_e) is estimated using Kirpitch Equation with the minimum time of entry set as 4 minutes. The Kirpitch equation is:

$T_e = 0.019621L^{0.77}/S^{0.385}$

Where T_e = time of entry in minutes

L = maximum length of overland flow in meter

S = average ground slope

Travel time (T_t) is calculated by dividing the length of drain by the water velocity. This has been calculated for individual catchments and reaches.

- **Storage coefficient**

The rainfall after evaporation and infiltration accumulates first in the depressions, until these have been reached their capacity and then runoff. To take these effects a storage coefficient is used. The value of the storage coefficient is based on average ground slope and the nature of the ground surface. For estimating the Storage Coefficient Table 4.2 is used. In this study, storage coefficient was assumed to be 0.7 for all catchments considering all catchments within residential areas with detached houses as no data regarding the urbanization was found.

Table 4.2 : Storage Coefficients, C_s

Characteristics of surface	Storage Coefficient		
	Slope < 1:1000	Slope < 1:500	Slope > 1:500
Paved areas-road and market	0.8	0.9	1
Densely built up areas	0.8	0.9	1
Central areas mixed commercial and housing	0.7	0.8	1
Residential areas with detached houses	0.7	0.8	0.9
Walled areas and garden	0.6	0.7	0.8
Large permeable areas (dry agriculture)	0.5	0.6	0.8
Paddy field (flooded)	0.3	0.4	0.5

- **Runoff coefficient**

The runoff coefficient represents the ratio between the volume of runoff and the volume of rainfall. The runoff coefficient is selected from list of C_r values given in Table 4.3 below. In this study, runoff coefficient was assumed to be 0.4 for all catchments considering all catchments within residential areas with detached houses.

Table 4.3 : Runoff coefficients, C_r

Land use designation	Runoff coefficient C_r
Paved areas-road and market	0.9
Densely built up areas	0.7
Central areas mixed commercial and housing	0.6
Residential areas with detached houses	0.4
Walled areas and garden	0.3
Large permeable areas (dry agriculture)	0.3
Paddy field (flooded)	0.8

Following the design criteria storm runoff assessment has been done for the proposed drainage system/zones and sub basins/ sub zones which are shown details in Annex B-1.

4.2.3(iii) Design of Drain Sections

Once the peak runoff flow rate of a watershed is computed the next step is to calculate the geometric section in designing drain. For a given discharge the geometric section of a drain depends mainly on bed slope and the frictional resistance of the contact surface to flowing water. Manning's Equation is here used for the calculation of flow velocity and determining drain section. The equation can be used for all shapes of open drains as only the area and wetted perimeter of the drain needs to be calculated to assess the drain capacity as shown in Annex B-2.

- **Manning's equation**

The Manning's Equation used for the calculation of flow velocity is given below. In determining the shape of drains the criteria of the Design discharge (Q_d) should be greater than the Peak discharge (Q_P) and is only satisfied on iteration process.

Design Velocity, $V = [1/n][R^{2/3}][S^{1/2}]$

Design discharge, $Q_d = AV = A[1/n][R^{2/3}][S^{1/2}]$

Where, V = velocity of flow, m^3/s

n = Manning's roughness coefficient value

S = Hydraulic gradient, m/m

R = hydraulic radius=A/P, m

A= flow area, m^2

The value of Manning's roughness coefficient 'n' used in the equation is given in Table 4.4. For this study, concrete drain type with a value of n 0.014 has been assumed.

Table 4.4 : Manning's 'n' values for Channel Flow

SL No.	Type of Drain	Manning's "n"
1	Concrete drains	0.014
2	Brick drains (plastered)	0.014
3	Brick drains (unplastered)	0.016
4	Unlined(kutcha) drains (earthen)	0.025
5	Unlined(kutcha) drains (grass)	0.030

- **Lined and unlined channels**

The outfalls i.e., the regional rivers, natural canals and khals can be defined as unlined drains. They are carved or shaped by nature before urbanization occurs. They often have mild slopes and are reasonably stable. It is possible to have manmade unlined drains. Man made and re-sectioned unlined drains are normally trapezoidal in shape. In this study all proposed drains has been considered to be rectangular only the borrow pits which has been encroached by the local inhabitants are considered to be trapezoidal section for its re-sectioning.

- **Discharge velocity**

The velocity of storm water flows in drains should be maintained within acceptable limits to ensure self cleaning of the section as well as scouring and erosion of the conduit, (particularly the invert) does not occur. The maximum velocity is governed by the type of drain lining and bottom slope of drain. For lined primary drains and outfalls velocity should not exceed 3 m/s and for unlined velocity should not exceed 1.5 m/s.

- **Free board**

The term drain freeboard generally refers to the vertical distance between the design water surface elevation and the level of the top of the drain. Its function is to prevent overtopping of the channel caused by a number of factors including discrepancies in calculations, construction tolerances, wave action (caused by wind, flow turbulence and lateral inflows to the channel), hydraulic jumps, floating debris, channel sedimentation, increases in channel roughness associated with seasonal change, and the occurrence of flows in excess of the design capacity. Channel freeboard is incorporated in order to provide protection from flooding for adjacent land and buildings. For primary drain free board is calculated according to the USBR and Lacy's formulae, respectively

$$F = (CD)^{1/2}$$

$$F = 0.20 + 6.15\ Q^{1/3}$$

Where, $C = 0.46$ to 0.76, depending on discharge

D = depth of flow in meter

Q = discharge in m^3/sec

The designed drains should have minimum allowance for freeboard of 200 mm for primary drains, 150 mm for secondary drains and 100 mm for tertiary drains.

- **Longitudinal slope and side slope**

The velocity of flow is a function of longitudinal slope, shape and frictional resistance of bed and bank materials of the drain. The longitudinal and side slopes of a drain should be so selected that it carries the desired runoff discharge as well it should not erode the bank and scour the bed of unlined primary drain and outfall. High discharge velocity also not encouraged as it may carry sediment and debris and other pollutant which may causes the deterioration of water quality at outfall. The recommended longitudinal slopes are 1:500 for tertiary drain, 1:1000 for secondary drain and 1:2000 for primary drain and outfall. The preferred side slope for unlined trapezoidal section for primary drain and outfall is 1:1.5

although in some areas side slopes of 1:2 may be required due to poor ground conditions. For lined channel the longitudinal and side slopes may be steeper than that mentioned.

4.2.4 Damage Analysis

Model outputs were used to estimate the damages in homestead, commercial enterprises and in Industrial sectors. The land use type- agriculture and roads were excluded for the damage analysis as the study aims in urban drainage and also the roads in our study area has width less than 20 m. So it was not possible to rasterised the road network for damage analysis.

The techniques of geo-informatics are deployed to complete the analysis using a set of raster and vector data such as Flood depth maps and land use data. Inundation maps are used as input for the damage estimation due to excessive flooding. In this case the damages are directly related to the depth of flood water in each pixel with a range indicating, (31 to 90)cm as low damage, (91 to 180)cm moderate damage and (181 to above 360)cm as severe damage. With the results of model, it was possible to visualize the type of damages for different rainfall event with different outfall stages. The result of the analysis is presented for the urban areas as a whole in homestead, commercial and industrial sectors.

Chapter 5
URBAN DRAINAGE MODELING

5.1 Introduction

Dhamrai lies within the North Central Region (NCR). The NCRM developed in IWM has been used to address the drainage system of Dhamrai. The Urban Drainage Model for Dhamrai has been developed by truncating NCRM and emphasizing the study area. The objective of developing the Urban Drainage Model is to create a tool for Dhamrai Municipality, which would enable the simulation of the performance of the complex drainage system.

Hydraulic and hydrologic features of Dhamrai have been nested to update the existing NCRM. The truncated model of NCRM is updated with the incorporation of drainage features to act as Urban Drainage Model for Dhamrai Municipality which comprises Rainfall-runoff model for hydrological analysis and Hydrodynamic model for hydraulic analysis. Runoffs generated from the Rainfall-runoff model have been included in Hydrodynamic model. Details regarding the two models- hydrological and hydrodynamic model are described in Section 5.2 and Section 5.3.

5.2 Hydrological Model

Two different urban runoff computation concepts are available in the Rainfall Runoff Module as two different runoff models:

- Model A: Time/area Method
- Model B: Non-linear Reservoir (kinematic wave) Method

Model study of hydrological analysis of Dhamrai drainage system has been carried out using URBAN module; Model B concept. It generates catchment runoffs from the proposed drainage area.

5.2.1 Introduction to Urban Model B (Non-Linear Reservoir Method)

The concept of surface runoff computation of Urban Runoff Model B is founded on the kinematic wave computation. This means that the surface runoff is computed as flow in an open channel, taking the gravitational and friction forces only. The runoff amount is controlled by the various hydrological losses and the size of the actually contributing area. The shape of the runoff hydrograph is controlled by the catchment parameters length, slope and roughness of the catchment surface. These parameters form a base for the kinematic wave computation (Manning equation).

Catchment Parameters

- Length: Usually defines the flow channel. The model assumes a prismatic flow channel with rectangular cross section. The channel bottom width is computed from catchment area and length.

- Slope: Average slope of the catchment surface, used for the runoff computation according to Manning.

- Area: The area distribution percentages divide the catchment area into five sub-catchments with identical geometrical, but distinct hydrological properties. The five sub catchment types are: impervious steep, impervious flat, pervious -small impermeability, pervious - medium impermeability, pervious - large impermeability.

Hydrological Parameters

- Wetting Loss: One-off loss, accounts for wetting of the catchment surface.

- Storage Loss: One-off loss, defines the precipitation depth required for filling the depressions on the catchment surface prior to occurrence of runoff.

- Start Infiltration: Defines the maximum rate of infiltration (Horton) for the specific surface type.

- End Infiltration: Defines the minimum rate of infiltration (Horton) for the specific surface type.

- Horton’s Exponent: Time factor "characteristic soil parameter". Determines the dynamics of the infiltration capacity rate reduction over time during rainfall. The actual infiltration capacity is made dependent of time since the rainfall start only.

- Inverse Horton’s Equation: Time factor used in the "inverse Horton's equation", defining the rate of the soil infiltration capacity recovery after a rainfall, i.e. in a drying period.

- Manning’s Number: Describes roughness of the catchment surface, used in hydraulic routing of the runoff (Manning's formula).

Runoff Computation

The model computations are based on the volume continuity and the kinematic wave equations. The first step is the calculation of effective precipitation intensity. The effective precipitation intensity is the precipitation which contributes to the surface runoff. Next, the hydraulic routing, based on the kinematic wave formula (Manning) and volume continuity is applied. The sketch with schematics of the model computation is shown in Figure 5.1.

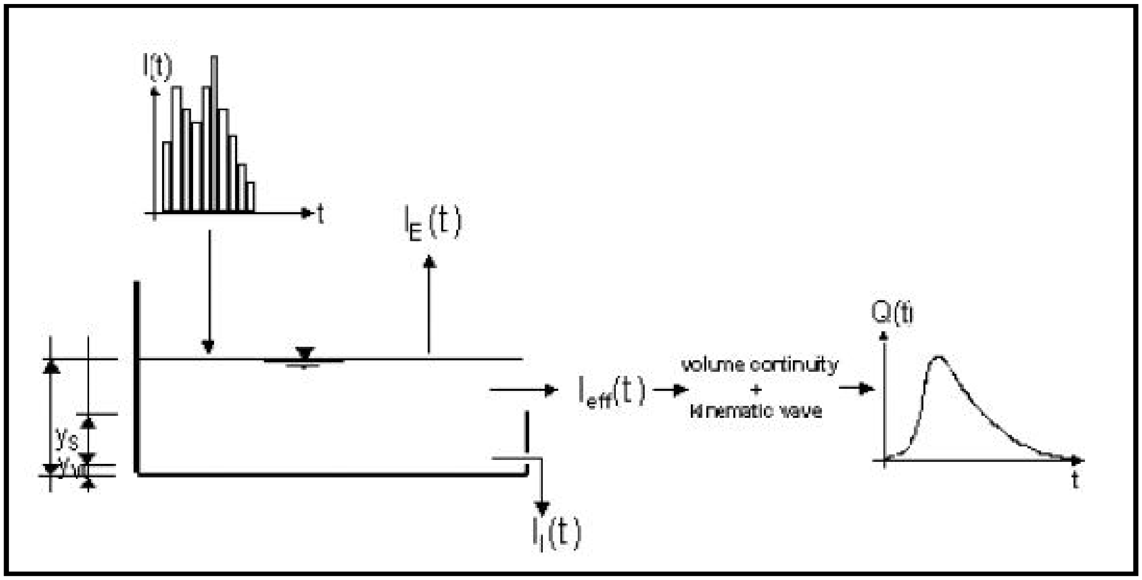


Figure 5.1: The simulation processes in the Surface Runoff Model B

Computing Effective Precipitation

The simulated hydrologic processes account for various losses calculated - evaporation, wetting, infiltration and surface storage -according to the conventions and equations presented below. The remaining precipitation is called effective precipitation, defined generally as:

$$I\ (t) = I(t) - IE\ (t) - IW\ (t) - II\ (t) - IS\ (t)$$

where,

I(t) = Actual precipitation at time t

IE(t) = Evaporation loss at time t. It should be noted that the Evaporation loss for the catchment is accounted only if it has been specified as input time series

IW(t) = Wetting loss at time t

II(t) = Infiltration loss at time t

IS(t) = Surface Storage loss at time t

The individual terms in the loss equation are fundamentally different, as some terms are continuous where others are discontinuous. If the calculated loss is negative, it is set to zero.

The actual precipitation, I(t), is assumed to be uniformly distributed over the individual catchments. Otherwise, it may vary as a random time function. The evaporation, IE(t), is a continuous loss that is normally of less significance for single event simulations. If included in the computation, the evaporation is the first part subtracted from the actual precipitation.

The wetting, IW(t), is a discontinuous loss. When precipitation starts, a part of the precipitation is used for wetting of the surface if the surface is initially dry. The model assumes that the precipitation remaining after subtraction of the evaporation loss is used for wetting of the catchment surface. When the surface is wet, the wetting loss, IW, is set to zero.

The infiltration, II(t), is the water loss to the lower storage caused by the porosity of the catchment surface. It is assumed that the infiltration starts when the wetting of the surface has been completed.

The infiltration is a complex phenomenon, dependent on the soil porosity, moisture content, groundwater level, surface conditions, storage capacity, etc.

The surface storage IS(t), is the loss due to filling the depressions and holes in the terrain. The model begins with the surface storage calculation after the wetting process is completed. The surface storage is filled only if the current infiltration rate is smaller than the actual precipitation intensity reduced by evaporation.

Surface Runoff Routing

The runoff starts when the effective precipitation intensity is larger than zero. The hydraulic process is described with the kinematic wave equations for the entire surface at once. This description assumes uniform flow conditions on the catchment surface, i.e. equal water depth over the entire surface of certain category. This type of runoff model is also called a non-linear reservoir model. The surface runoff at time t is calculated as:

$$Q(t) = M \,.B \,.I^{(1/2)}\, yR\,(t\,)^{(5/3)}$$

where:

M = Manning's number

B = Flow channel width, computed as:

B = A / L

I = Surface slope

yR(t) = Runoff depth at time t

The depth yR(t) is determined from the continuity equation:

$$I_{eff}(t) \cdot A - Q(t) - \frac{dy_R}{dt} \cdot A$$

where:

Ieff = Effective precipitation

A = Contributing catchment surface area

dt = Timestep

dyR = Change in runoff depth

5.2.2 Set Up of Urban Model B (Hydrological Modeling)

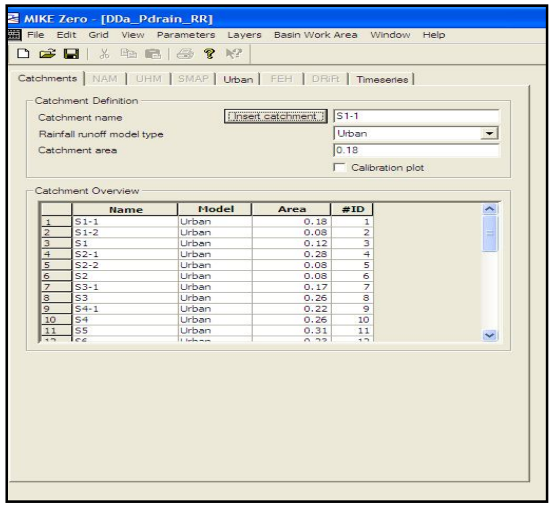
At first, Planning of zones and catchments with identification of routes and outfalls has been carried out. The surface runoff assessment was then carried out for each of the catchment and zones (details have been described in Section 4.2.3). An independent drainage model for Dhamrai has been developed using hydrologic features in the area. The model acts as Urban Drainage Model for Dhamrai which comprises only rainfall-runoff model for hydrological analysis. Historical simulation of the model has been carried out. Runoffs generated from the model have been analyzed to determine design flow of drains and also have been included in the Hydrodynamic model.

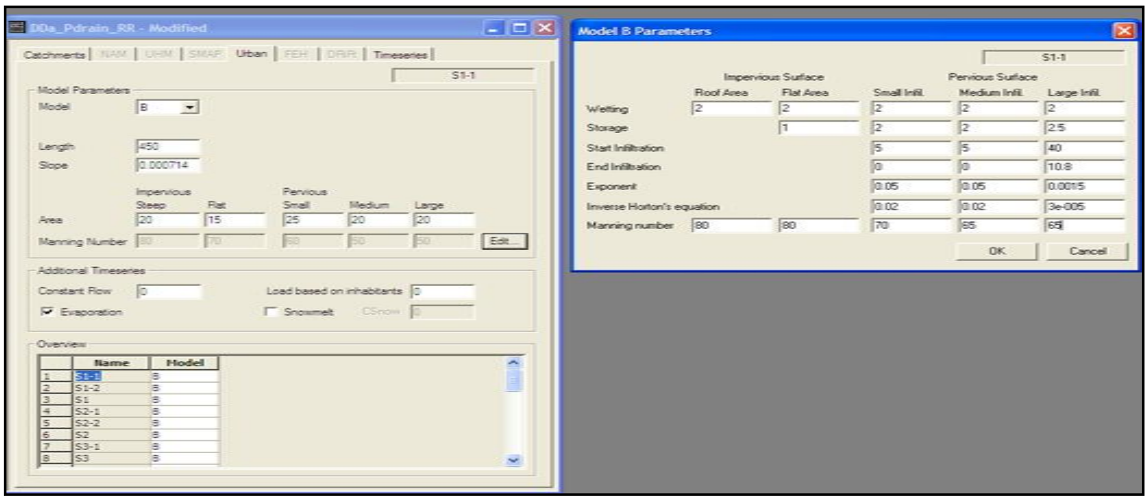
The detail information regarding each of the catchment and zones were incorporated in the Rainfall-Runoff model. This includes individual catchment and zone area, length and slope, percentage of impervious and pervious area within the catchment, wetting loss, storage loss, infiltration capacity and manning's roughness. The catchment area, length and slope have been calculated from GIS. MIKE URBAN can accommodate five different sub-catchments descriptions. The run-off and surface routing parameters for the sub-catchments were finalized during calibration following the methodology used for the recently IWM-studied Khulna Drainage Model under an ADB-financed project in 2009-10. Values regarding the storage loss, start and end infiltration has been incorporated consulting the Soil Research Development Institute (SRDI) report on the Guidelines for the use of Land and Soil Resources of Dhamrai Upazila, Dhaka Zila and Engineering Hydrology by Subramanya. Table 5.1 shows the parameter values that were adopted in the final model. Historical rainfall and evaporation data were an input to develop the Rainfall-Runoff model.

Table 5.1: Parameters used for the Description of the Rainfall-Runoff Process

	Impervious Area		Pervious Area		
	Roof Area	**Flat Area**	**Small Infiltration**	**Medium Infiltration**	**Large Infiltration**
Wetting	2	2	2	2	2
Storage		1	2	2	2.5
Start Infiltration			5	5	40
End Infiltration			0	0	10.8
Exponent			0.05	0.05	0.0015
Inverse Horton's Equation			0.02	0.02	3.00E-05
Manning's Number	80	80	70	65	65

The model has been calibrated for secondary drains against flows estimated using empirical formula. Secondary drains are designed for 2 year return period. For the present study calibration, monsoon period has been emphasized rather than that of dry period, because dry period calibration is not significant for the drainage study. The main parameter to carry out calibration procedure is Manning's Roughness, M (inverse of roughness coefficients n). The value of M for drainage channels ranges from 65 to 80 as adopted in rainfall-runoff model Urban B module. It is observed that higher values of M are significant in changing runoff volume. A screen print regarding the model setup is shown in Figure 5.2.





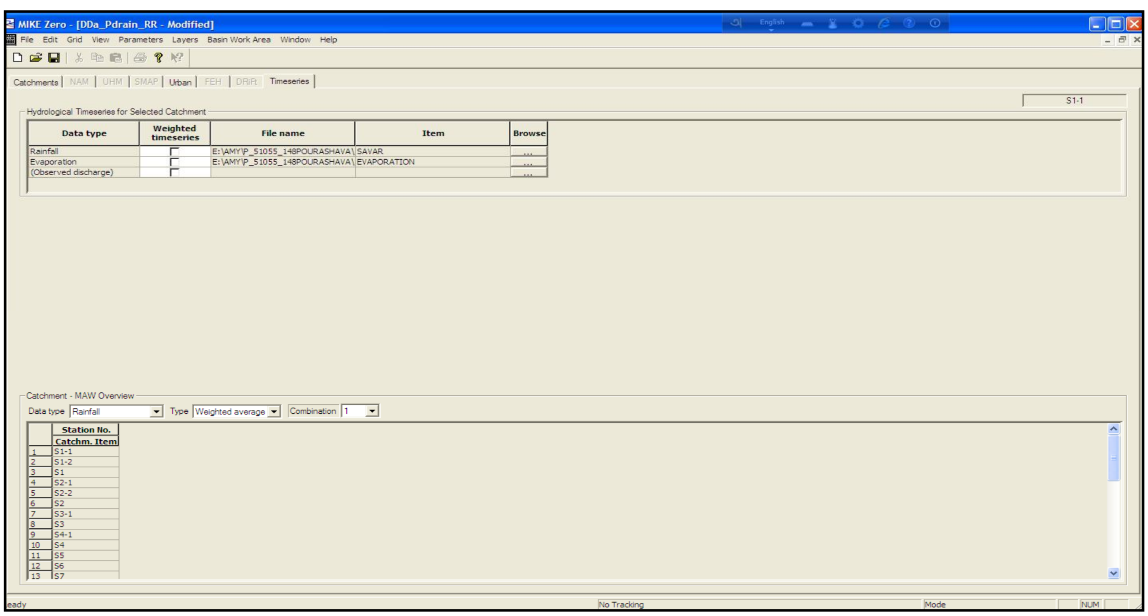


Figure 5.2: Set up of the Rainfall-Runoff Model (Hydrological Model)

5.3 Hydrodynamic Model

5.3.1 Introduction to Hydrodynamic Model

The MIKE 11 hydrodynamic module (HD) uses an implicit, finite difference scheme for the computation of unsteady flows in rivers and estuaries. The module can describe sub-critical as well as supercritical flow conditions through a numerical scheme which adapts according to the local flow conditions (in time and space). MIKE 11 HD applied with the dynamic wave description solves the vertically integrated equations of conservation of continuity and momentum (the 'Saint Venant´ equations). The equations of continuity and momentum, as used by MIKE 11, is given in equation 1 and equation2. The resulting equations are:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_L$$

Equation 1: Conservation of Mass

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(\alpha \frac{Q^2}{A}) + gA\frac{\partial h}{\partial x} + \frac{gQ|Q|}{M^2 AR^{3/4}} = 0$$

Equation 2: Conservation of Momentum

where,

Q = Discharge (m^3/sec)
A = Cross Sectional Area (m^2)
q_L = Lateral Inflow
h = Water Level (m)
M = Manning's Number ($m^{(1/3)}$/s)
R= Resistance Radius (m)
α = Momentum Distribution coefficient

The transformation of Equations 1 and 2, to a set of implicit finite difference equations is performed in a computational grid consisting of alternating *Q*- and *h*-points, i.e. points where the discharge, *Q* and water level *h*, respectively, are computed at each time step. The computational grid is generated automatically by the model on the basis of the user requirements. *Q*-points are always placed midway between neighbouring *h* points, while the distance between *h*-points may differ. The discharge will, as a rule, be defined as positive in the positive *x*-direction (increasing chainage).

5.3.2 Set up of Hydrodynamic Model

The Hydrodynamic modeling requires five components: network data, cross sectional data, boundary conditions, HD parameters and rainfall-runoff from the contributing catchments and zones. For this reason, four components were needed for the set up of hydrodynamic model. The rainfall runoff was incorporated from the results obtained from hydrological modeling.

5.3.2(a) Network Data

A truncated model was developed from the existing NCRM of IWM. The only additional input was the proposed drainage system in the model. Areal extent of Dhamrai in the existing North central Regional Model (NCRM) is shown in Figure 5.3.

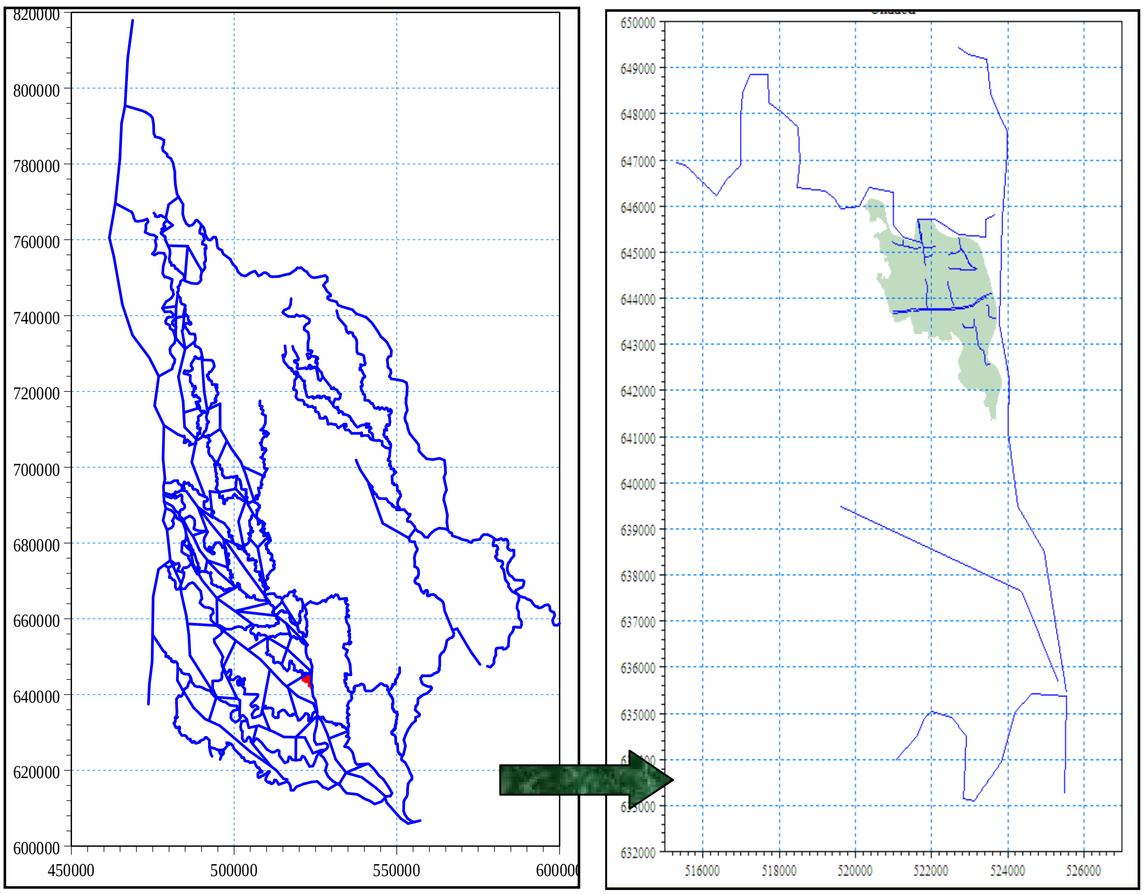


Figure 5.3: Domain of Dhamrai Drainage Model in the existing NCRM

The drainage network of Dhamrai contains No. 13 drains with No. 5 re-sectioning borrow pits. The Bangshi River is the only outfall channel as identified on the basis of engineering survey and drainage network mapping. It was decided to schematize the model for the secondary channels plus a part of the network important to the system. Figure 5.4 shows a schematic diagram of rivers and drainage channels. Table 5.2 contains the list of drainage channels included in the model. In the NCRM, Kekla one of the outfall for Dhamrai is named as BANSI_SOUTH. The NCRM has been truncated at its downstream were an observed water level station is available.

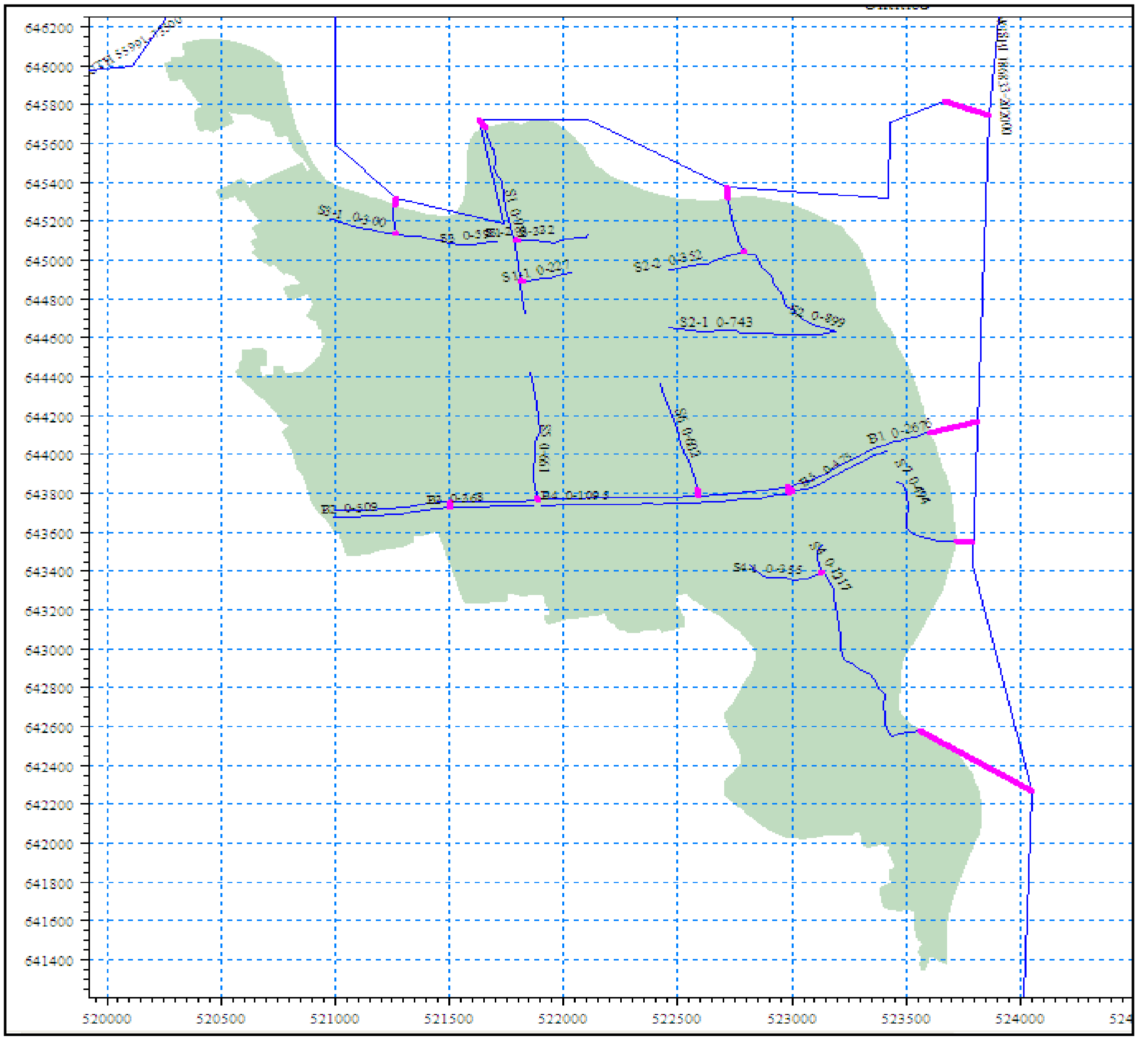


Figure 5.4: Schematic Drainage Network for Dhamrai Urban Drainage Model

Table 5.2: Length of Rivers and Drainage Channels for Dhamrai Drainage Model

Sl No	Name	Upstr. Ch.	Downstr. Ch.	Upstr. Conn. Name	Upstr. Conn. Ch.	Downstr. Conn. Name	Downstr. Conn. Ch.
1	BANGSHI	186833	202000			DHALESWARI	120000
2	BANSI_SOUTH	55991	73500			BANGSHI	190924
3	BANSI_S_RB	30000	33000			BANSI_S_RB	33000
4	DHALESWARI	111602	122462				
5	DHALES_L120	0	1000	DHALESWARI	120000	BANSI_S_RB	33000
6	S1	0	989			BANSI_SOUTH	70360
7	S1-1	0	227			S1	169
8	S1-2	0	332			S1	384
9	S2	0	899			BANSI_SOUTH	71629
10	S2-1	0	743			S2	0
11	S2-2	0	352			S2	610
12	S3	0	598			BANSI_SOUTH	69380
13	S3-1	0	300			S3	453
14	S4	0	1217			BANGSHI	197485
15	S4-1	0	355			S4	157

Sl No	Name	Upstr. Ch.	Downstr. Ch.	Upstr. Conn. Name	Upstr. Conn. Ch.	Downstr. Conn. Name	Downstr. Conn. Ch.
16	S5	0	661			B1	884
17	S6	0	602			B1	1602
18	S7	0	494			BANGSHI	196700
19	B1	0	2676			BANGSHI	195050
20	B2	0	509			B1	511
21	B3	0	368			B1	511
22	B4	0	1095			B1	2002
23	B5	0	475			B1	2002

5.3.2(b) Cross Sectional Data

Cross sections for the drainage routes have been determined using analytical methods as described in Section 4.2.3. Most of these cross sections are rectangular except the borrow pits, which are considered as trapezoidal sections. The updated and proposed cross sections along with the cross sections for the rivers available in the NCRM have been incorporated to develop a hydrodynamic (HD) component for Dhamrai Urban Drainage Model. Figure 5.5 represents the screen print for the cross sectional data setup for the model.

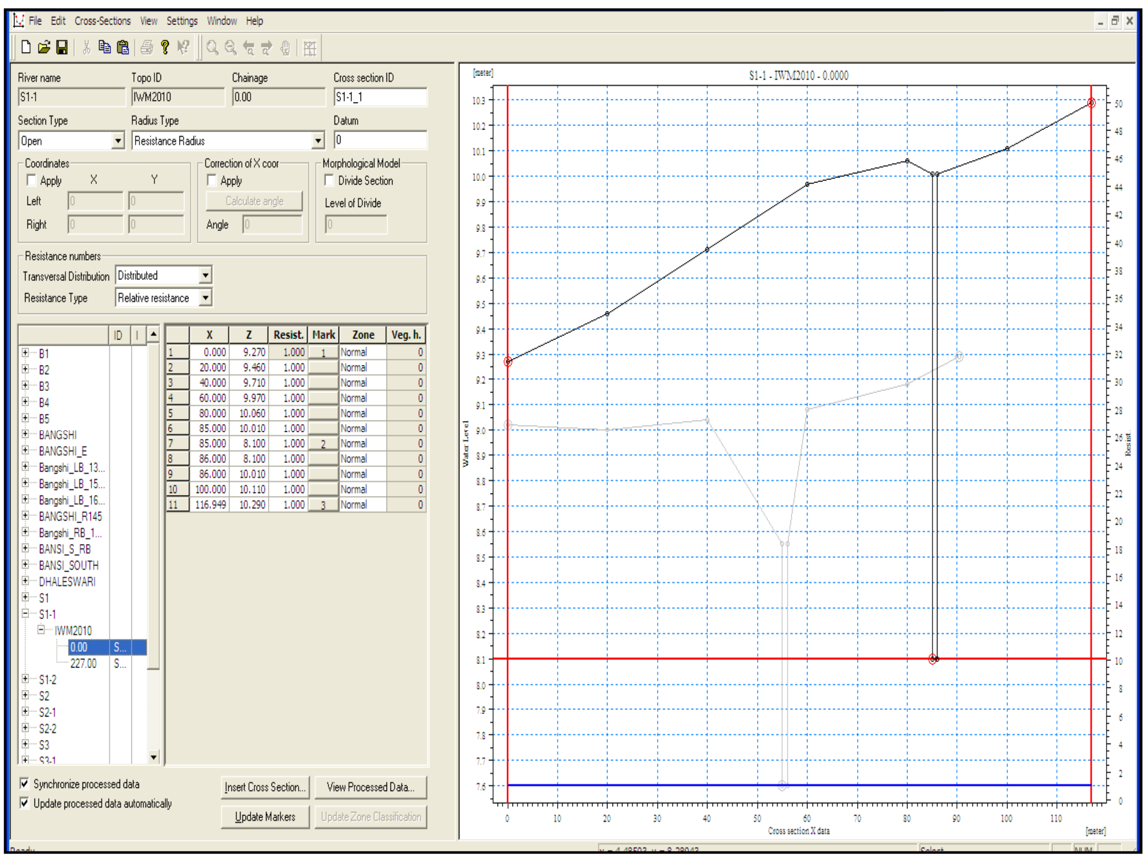


Figure 5.5: Set up of the cross sectional data

5.3.2(c) Boundary Conditions

There are 22 open boundaries in the Dhamrai Urban Drainage Model, 21 of which are upstream boundaries at the upstream ends of the rivers/khals/drains and 1 downstream boundary at the downstream end. The upstream and downstream boundaries are presented in Table 5.3.

Table 5.3: Table showing U/S & D/S Boundaries in the Drainage Model

Sl No	Branch Name	Chainage (m)	Location	Boundary Type	Bounday Value
1	S1	0	U/S	Discharge	Q_c=0
2	S1-1	0	U/S	Discharge	Q_c=0
3	S1-2	0	U/S	Discharge	Q_c=0
4	S2-1	0	U/S	Discharge	Q_c=0
5	S2-2	0	U/S	Discharge	Q_c=0
6	S3	0	U/S	Discharge	Q_c=0
7	S3-1	0	U/S	Discharge	Q_c=0
8	S4	0	U/S	Discharge	Q_c=0
9	S4-1	0	U/S	Discharge	Q_c=0
10	S5	0	U/S	Discharge	Q_c=0
11	S6	0	U/S	Discharge	Q_c=0
12	S7	0	U/S	Discharge	Q_c=0
13	B1	0	U/S	Discharge	Q_c=0
14	B2	0	U/S	Discharge	Q_c=0
15	B3	0	U/S	Discharge	Q_c=0
16	B4	0	U/S	Discharge	Q_c=0
17	B5	0	U/S	Discharge	Q_c=0
18	BANGSHI	1,86,833	U/S	Discharge	20 year simulation of regional model
19	BANSI_SOUTH	55,991	U/S	Discharge	20 year simulation of regional model
20	BANSI_S_RB	30,000	U/S	Discharge	20 year simulation of regional model
21	DHALESWARI	1,11,602	U/S	Discharge	20 year simulation of regional model
22	DHALESWARI	1,22,462	D/S	Water Level	20 year simulation of regional model

5.3.2(d) HD Parameters

The models have been calibrated for the hydrological year 2005 at Savar where observed water level data is available. The calibration plot is shown in Figure 5.6 Validation of the models has been carried out for the year 2006 and validation plot is given in Figure 5.7. The main parameter to carry out calibration procedure is Manning's Roughness, M (inverse of roughness coefficients n). The value of M for main rivers and drainage channels ranges from 28 to 50 and 65 to 85 respectively.

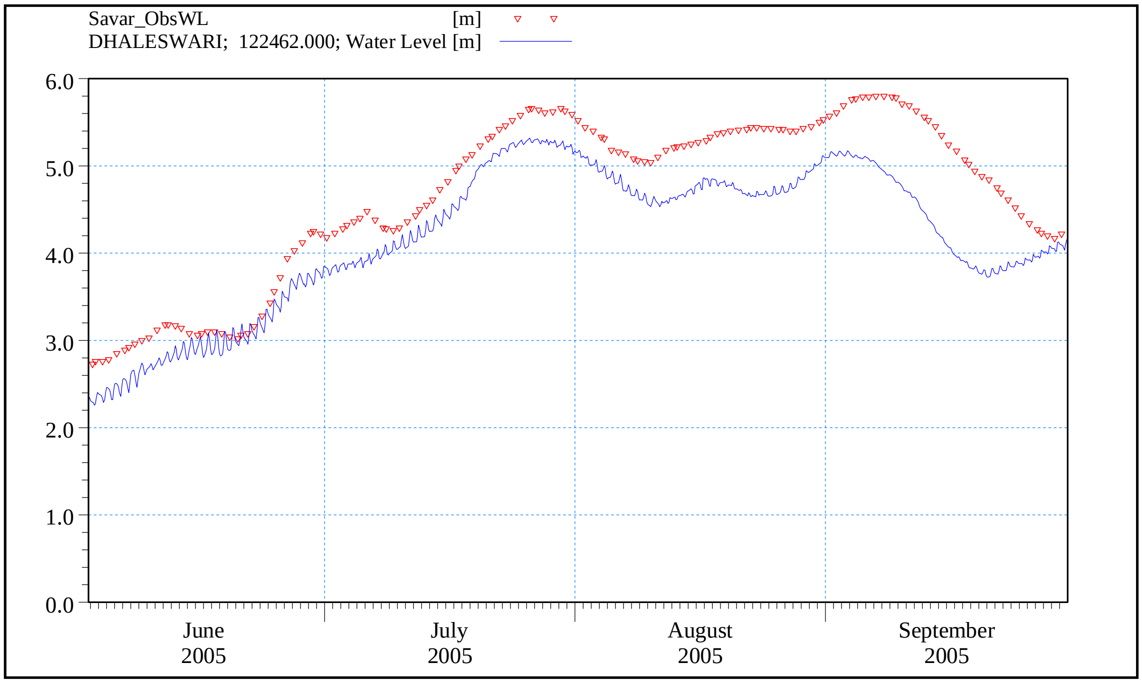


Figure 5.6: Calibration Plot at Savar for the year 2005

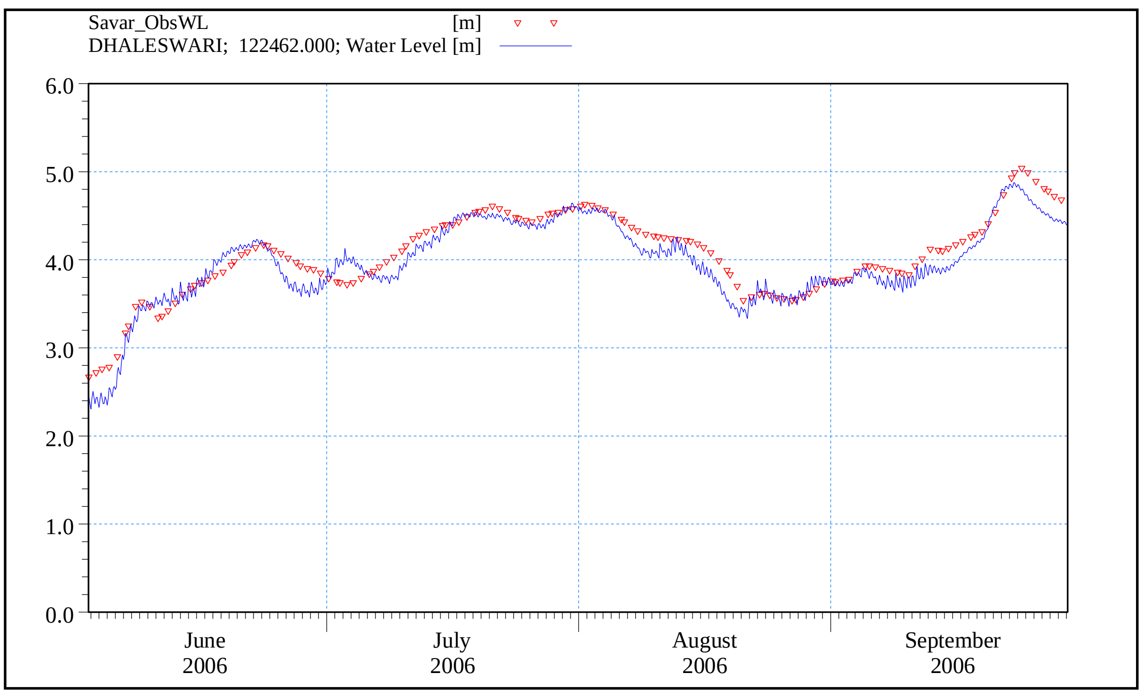


Figure 5.7: Validation Plot at Savar for the year 2006

Chapter 6
RESULTS AND DISCUSSION

6.1 Frequency Analysis of Rainfall Data

The Frequency Analyses of 2-day maximum rainfall data at Savar for Dhamrai 1 in 1.1 year, 1 in 2 year and 1 in 5 year return period and their representing years are given in Table 6.1. The sample plot of frequency analysis on Gumbel Distribution is shown in Figure 6.1 and other details can be found at Annex A-3.

Table 6.1: Results of Frequency Analysis of 2-day maximum rainfall data of Savar station for Dhamrai

Return Period	Rainfall (mm)	Representing Year
1.1 year	117	2001
2 year	174	2000
5 year	248	1990

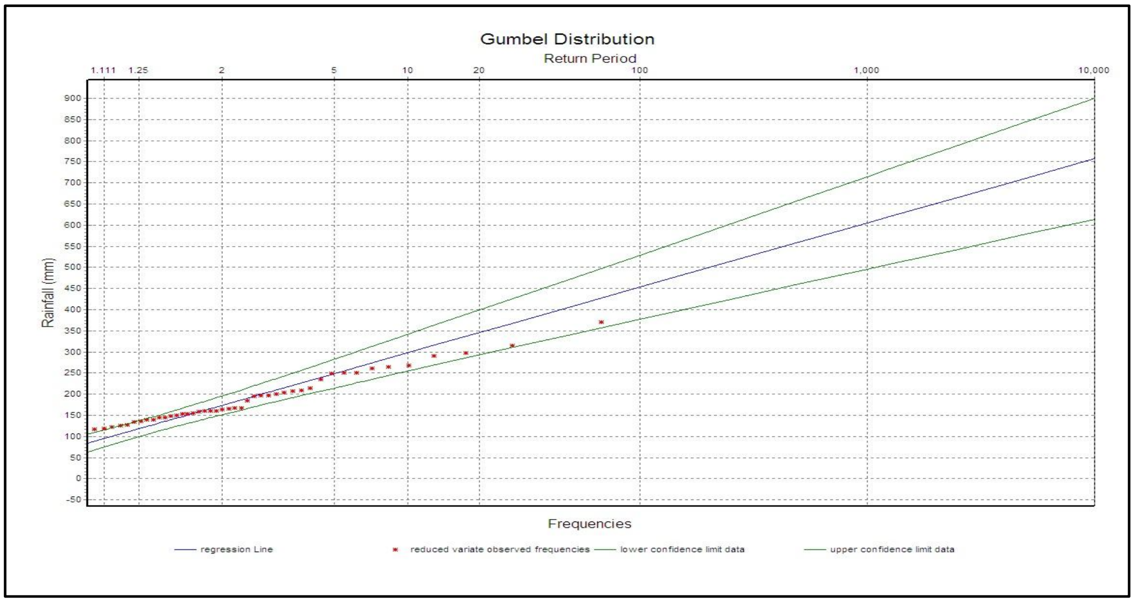


Figure 6.1 : Plot of Gumble Distribution for 2-day maximum rainfall data analysis in Savar Station for Dhamrai

6.2 Frequency Analysis of Water Level Data

The Frequency Analyses of yearly maximum simulated water level data of various return periods and their representing year is given in Table 6.2. Sample plot is represented in Figure 6.2 and details can be found at Annex A-4.

Table 6.2: Frequency Analysis of WL for Outfall Channel at Bangshi

Return Period	Water Level (m)
1.1 year	6.18
2 year	7.09
2.33 year	7.14
5 year	8.07
20 year	9.02

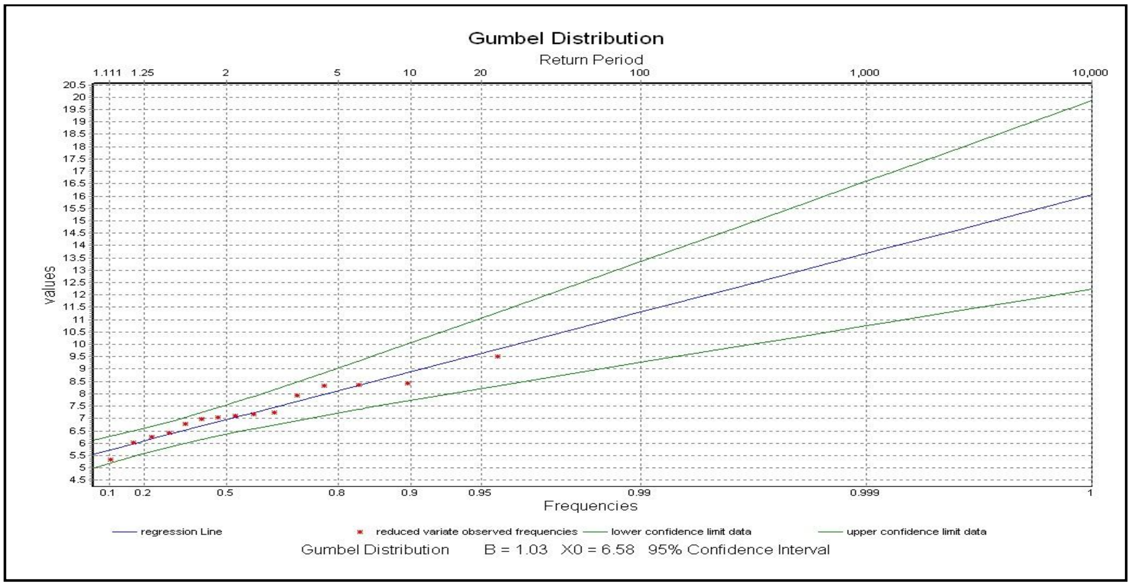


Figure 6.2 : Plot of Gumble Distribution for Annual Maximum Water Levels at Bangshi

6.3 Storm Runoff Assessment (Empirical and Model)

The summary of runoffs of all 22 zones is given in Table 6.3. It is considered for the estimate of discharges, that when storm drains are required in areas of the Municipality, such areas will have the characteristics of urbanization like mostly that of usual residential areas. A graphical representation of runoffs from empirical estimation and model simulations for the zones and individual catchments are shown in Figure 6.3 and Figure 6.4.

Table 6.3: Runoffs from Empirical Estimations and Model

Drainage Zone	Drainage Area (ha)	Discharge (Empirical) (m^3/sec)	Discharge (Model) (m^3/sec)
Zone-1	38	2.47	2.56
Zone-2	44	2.84	2.89
Zone-3	43	2.14	2.24
Zone-4	48	2.71	2.74
Zone-5	31	1.81	1.83
Zone-6	23	1.36	1.40
Zone-7	11	0.73	0.78
Zone-8	120	5.16	5.17
Zone-9	10	0.51	0.55

Drainage Zone	Drainage Area (ha)	Discharge (Empirical) (m^3/sec)	Discharge (Model) (m^3/sec)
Zone-10	47	1.52	2.34
Zone-11	37	2.74	2.75
Zone-12	7	0.47	0.51
Zone-13	16	1.05	1.09
Zone-14	8	0.50	0.56
Zone-15	23	1.46	1.49
Zone-16	6	0.40	0.45
Zone-17	6	0.43	0.46
Zone-18	16	0.99	1.02
Zone-19	14	0.82	0.89
Zone-20	24	1.43	1.48
Zone-21	97	5.45	5.48
Zone-22	41	2.12	2.18

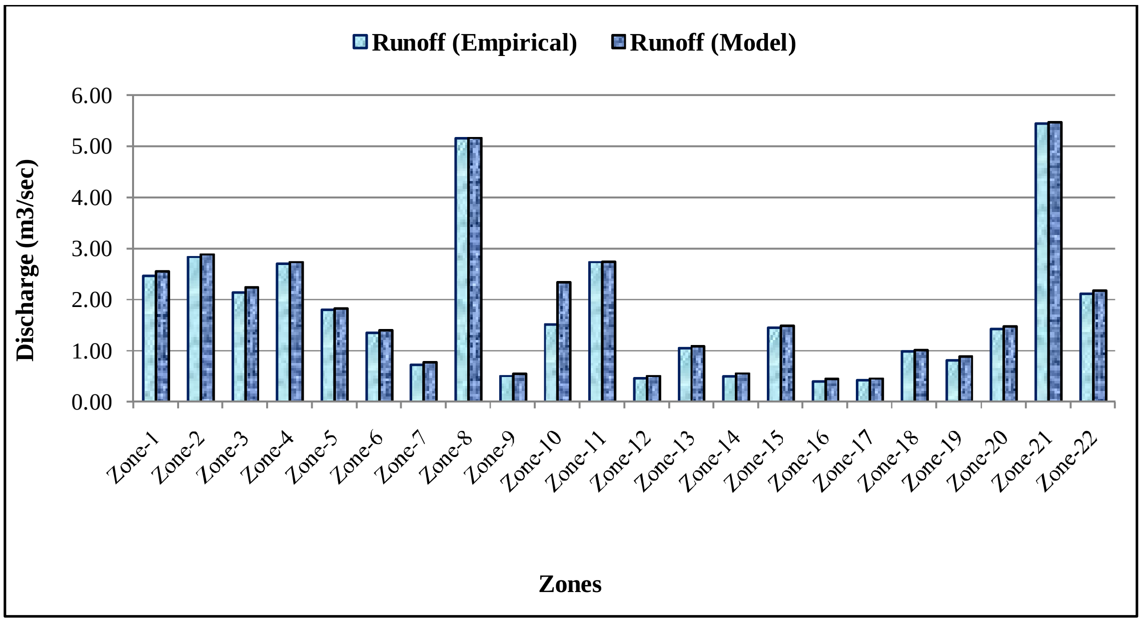


Figure 6.3: Comparison of runoffs between empirical estimates and model simulations for the Proposed Zones

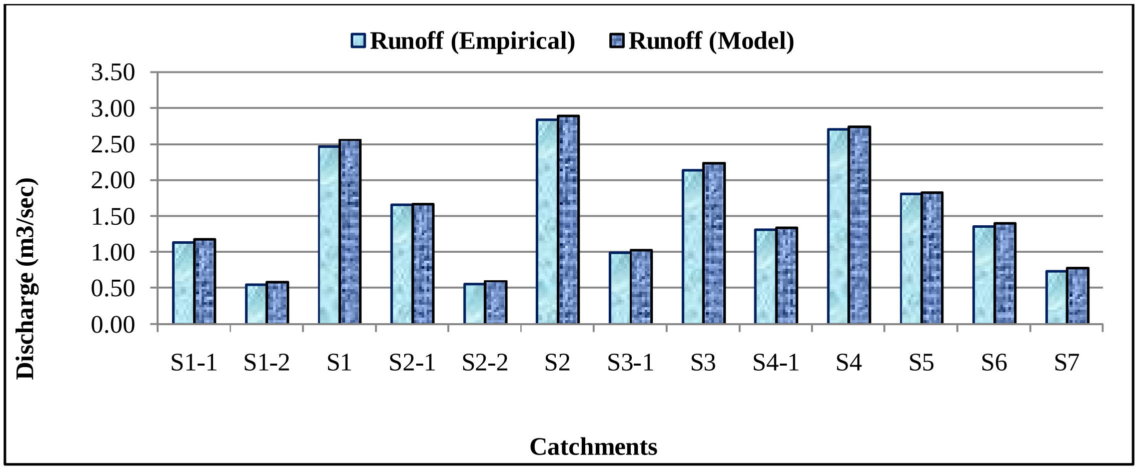


Figure 6.4: Comparison of runoffs between empirical estimates and model simulations for the Proposed Catchments

6.4 Hydraulic Parameter of Drains

Model simulations have been carried out for 2000 based on 2-day yearly maximum rainfall data for secondary drains. Geometrical shapes of cross sections given in Annex B-2 have been updated with land levels of relevant catchments. These cross sections have been finalized after several trials after assessing outfall conditions. Geometric shapes of drains have been determined and given in Table 6.4.

Table 6.4: Drainage Parameters

Drain ID	Ch. (m)	Catch. Cont'd Area (Hac)	Flow (m^3/s)	Rough Co-eff	DrainTop Width (m)	Drain Bottom Width (m)	Side Slope 1:	Drain Depth with FB (m)	Design Bottom level (mPWD)	Drain Type
S1-1	0		1.18	0.014	1.12	1.12	N/A	0.90	8.10	Rectangular.
	227	18	1.18	0.014	1.12	1.12	N/A	0.90	7.60	
S1-2	0		0.59	0.014	0.75	0.75	N/A	1.05	8.00	Rectangular
	332	8	0.59	0.014	0.75	0.75	N/A	1.05	7.65	
S1	0		0.17	0.014	0.65	0.65	N/A	0.55	7.77	Rectangular
	169	2	0.17	0.014	0.65	0.65	N/A	0.55	7.60	
	384	21	1.57	0.014	1.16	1.16	N/A	1.29	7.38	
	989	38	2.56	0.014	1.42	1.42	N/A	1.49	6.76	
S2-1	0		1.67	0.014	1.18	1.18	N/A	1.35	7.95	Rectangular
	743	28	1.67	0.014	1.18	1.18	N/A	1.35	7.20	
S2-2	0		0.60	0.014	0.80	0.80	N/A	1.00	8.00	Rectangular
	352	8	0.60	0.014	0.80	0.80	N/A	1.00	7.64	
S2	0		2.35	0.014	1.36	1.36	N/A	1.47	7.20	Rectangular
	610	34	2.35	0.014	1.36	1.36	N/A	1.47	6.58	
	899	44	2.89	0.014	1.45	1.45	N/A	1.59	6.28	

Drain ID	Ch. (m)	Catch. Cont'd Area (Hac)	Flow (m^3/s)	Rough Co-eff	DrainTop Width (m)	Drain Bottom Width (m)	Side Slope 1:	Drain Depth with FB (m)	Design Bottom level (mPWD)	Drain Type
S3-1	0		1.03	0.014	1.10	1.10	N/A	1.05	6.50	Rectangular
	300	17	1.03	0.014	1.10	1.10	N/A	1.05	6.20	
S3	0		1.29	0.014	1.00	1.00	N/A	1.33	6.70	Rectangular
	453	25	1.29	0.014	1.00	1.00	N/A	1.33	6.20	
	598	43	2.24	0.014	1.30	1.30	N/A	1.49	5.92	
S4-1	0		1.34	0.014	1.30	1.30	N/A	1.00	6.42	Rectangular
	355	22	1.34	0.014	1.30	1.30	N/A	1.00	6.00	
S4	0		0.50	0.014	0.85	0.85	N/A	0.80	6.18	Rectangular
	157	7	0.50	0.014	0.85	0.85	N/A	0.80	6.00	
	1217	48	2.74	0.014	1.44	1.44	N/A	1.55	4.90	
S5	0		1.83	0.014	1.60	1.60	N/A	1.07	6.32	Rectangular
	661	31	1.83	0.014	1.60	1.60	N/A	1.07	5.65	
S6	0		1.40	0.014	1.30	1.30	N/A	1.07	6.84	Rectangular
	602	23	1.40	0.014	1.30	1.30	N/A	1.07	6.22	
S7	0		0.78	0.014	1.10	1.10	N/A	0.70	8.20	Rectangular
	494	11	0.78	0.014	1.10	1.10	N/A	0.70	6.96	
For re-excavation of the Borrow Pits										
B2	0		0.45	0.014	2.30	0.95	1.5	0.65	7.65	Trapezoidal
	509	6	0.45	0.014	2.30	0.95	1.5	0.65	7.38	
B3	0		0.46	0.014	2.30	0.95	1.5	0.65	7.66	Trapezoidal
	368	6	0.46	0.014	2.30	0.95	1.5	0.65	7.46	
B4	0		1.02	0.014	2.90	0.95	1.5	0.85	7.42	Trapezoidal
	1095	16	1.02	0.014	2.90	0.95	1.5	0.85	6.86	
B5	0		0.89	0.014	2.69	0.65	1.5	0.88	7.84	Trapezoidal
	475	14	0.89	0.014	2.69	0.65	1.5	0.88	7.60	
B1	0		4.19	0.014	4.68	1.20	1.5	1.36	6.40	Trapezoidal
	511	87	4.19	0.014	4.68	1.20	1.5	1.36	6.11	
	884	163	7.91	0.014	5.93	1.55	1.5	1.66	5.65	
	1602	196	9.86	0.014	6.46	1.72	1.5	1.78	5.25	
	2002	249	12.96	0.014	7.30	2.35	1.5	1.85	5.02	
	2499	273	14.11	0.014	7.51	2.35	1.5	1.92	4.74	
	2676	273	14.11	0.014	7.51	2.35	1.5	1.92	4.64	

6.5 Model Output

The model simulations have generated water level in the drains and borrow pits draining towards the outfall channel Bangshi. Three scenarios were defined and simulated in the model. They are:

Scenario 1 (S1) - Flooding from RF without backwater effect

In order to check improvement of flooding from rainfall, flood maps have been generated using the maximum water level of model results for 2 and 5 year return period respectively as obtained from the results of rainfall-runoff model simulation. The years 2000 and 1990 are determined as design years for 2 and 5 year return periods as obtained from the 2-day maximum yearly rainfall data detailed in section 6.1. Two simulations in Scenario S1 are:

- Scenario S1(a) - 2 year RF effect only
- Scenario S1(b) - 5 year RF effect only

Scenario 2 (S2) - Flooding from Rainfall considering Average WL at the Outfall

Average water level of 7.14 mPWD has been incorporated in the model setup and then the model has been simulated for monsoon peak, which normally occurs during June-September. Two simulations in Scenario S2 are:

- Scenario S2(a) – Average WL at outfall and 2 year RF
- Scenario S2(b) – Average WL at outfall and 5 year RF

Scenario 3 (S3) - Flooding from Rainfall considering 1 in 5 year WL at the Outfall

1 in 5 year water level of 8.07 mPWD has been incorporated in the model setup and then the model has been simulated for monsoon peak, which normally occurs during June-September. Two simulations in Scenario S3 are:

- Scenario S3(a) – 1 in y year WL at outfall and 2 year RF
- Scenario S3(b) – 1 in 5 year WL at outfall and 5 year RF

Three of the scenarios were carried and analyzed under two different cases studies: (1) Proposed drainage system without re-excavation of the borrow pits and (2) Proposed drainage system with re-excavation of the borrow pits. These two cases were studied to evaluate the improvement of the water logging and inundation scenarios within the study area. As the study is mainly concern with the urban drainage system so, urban areas at present and in near future has been analyzed to identify the inundations with the proposed storm drains.

6.5.1 Case Study 1 (Proposed Drainage System without re-excavation of the Borrow Pits)

i) Flooding from RF without Backwater Effect (S1)

It is clear from Figure 6.5 and Figure 6.6 that, shallow to deep flooding occurs within some parts of Ward No 1, 3 and 5. Flood depth analysis for the urban area as a whole, considering the rainfall effect only is shown in Table 6.5.

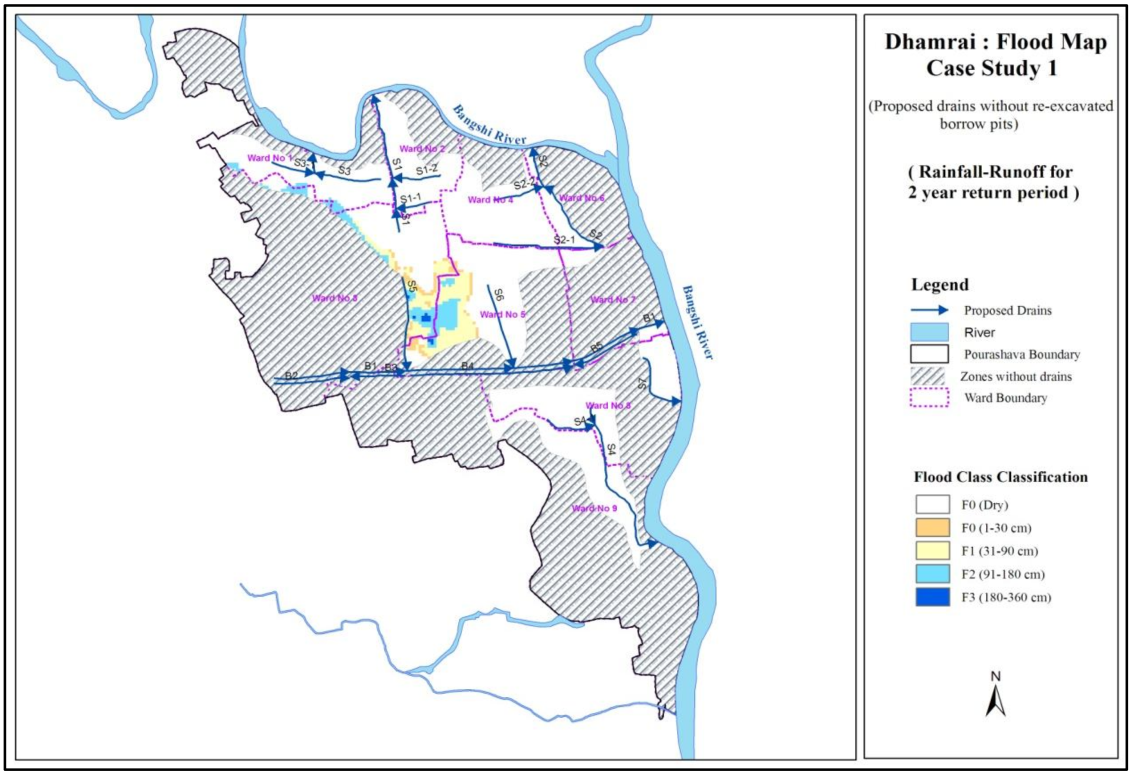


Figure 6.5: Flood Map considering Rainfall-Runoffs for 2 year return period S1(a)

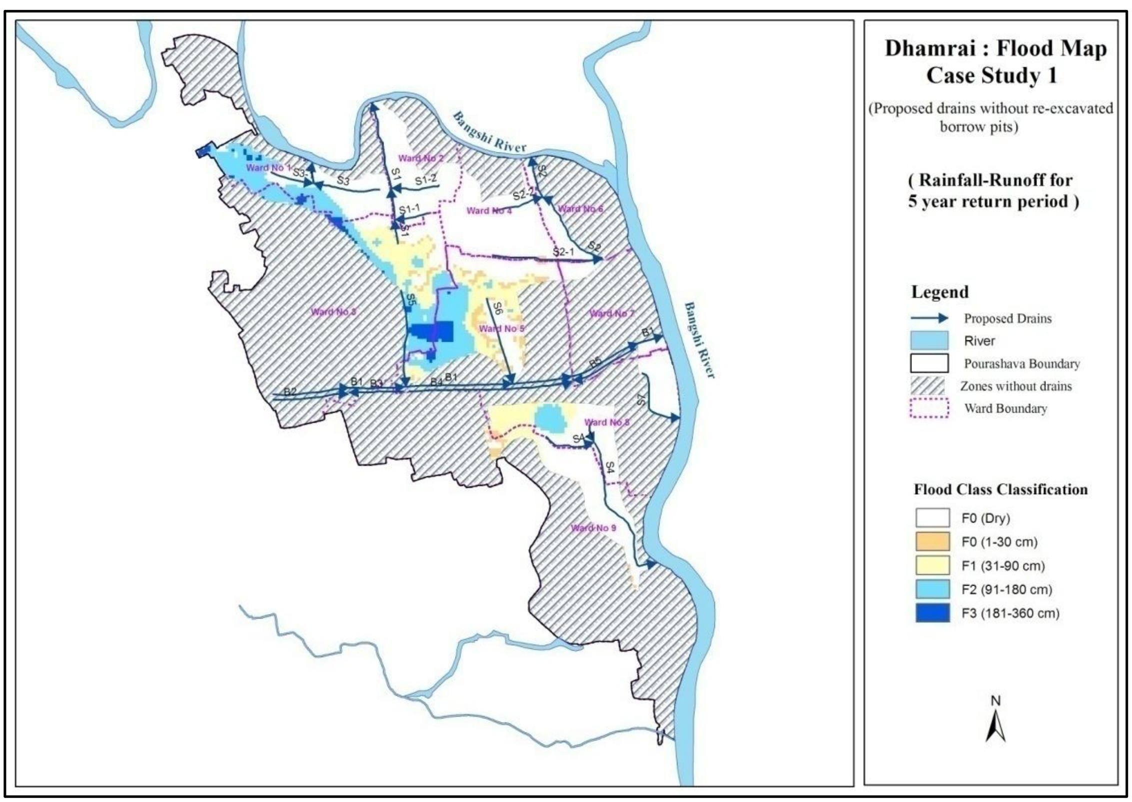


Figure 6.6: Flood Map considering Rainfall-Runoffs for 5 year return period S1(b)

Table 6.5: Flood Depth Analysis for Urban Areas considering RF Effect only

Location	**FF**	**F0**	**F1**	**F2**	**F3**	**Flood Free Area (FF+F0)**	**Eff. Area (FF to F3)** (Area unit in Ha)
Flood depth cm	water free	1-30	31-90	91-180	181-360		
2 yr RF event only	209.44	4.20	14.60	9.88	0.48	213.64	238.60
5 yr RF event only	150.60	8.88	32.64	39.88	6.40	159.48	238.60

ii) Flooding from Rainfall considering Average WL at the Outfall (S2)

Flood maps considering maximum water level of model results are shown in Figures 6.7 and 6.8 representing 2 and 5-year return period respectively with average water level. Shallow to deep flooding occurs within some parts of Ward No 1, 3 and 5. Flood depth analysis for the urban area as a whole, considering the average water level at the outfall with 2 and 5 yr rainfall event is shown in Table 6.6.

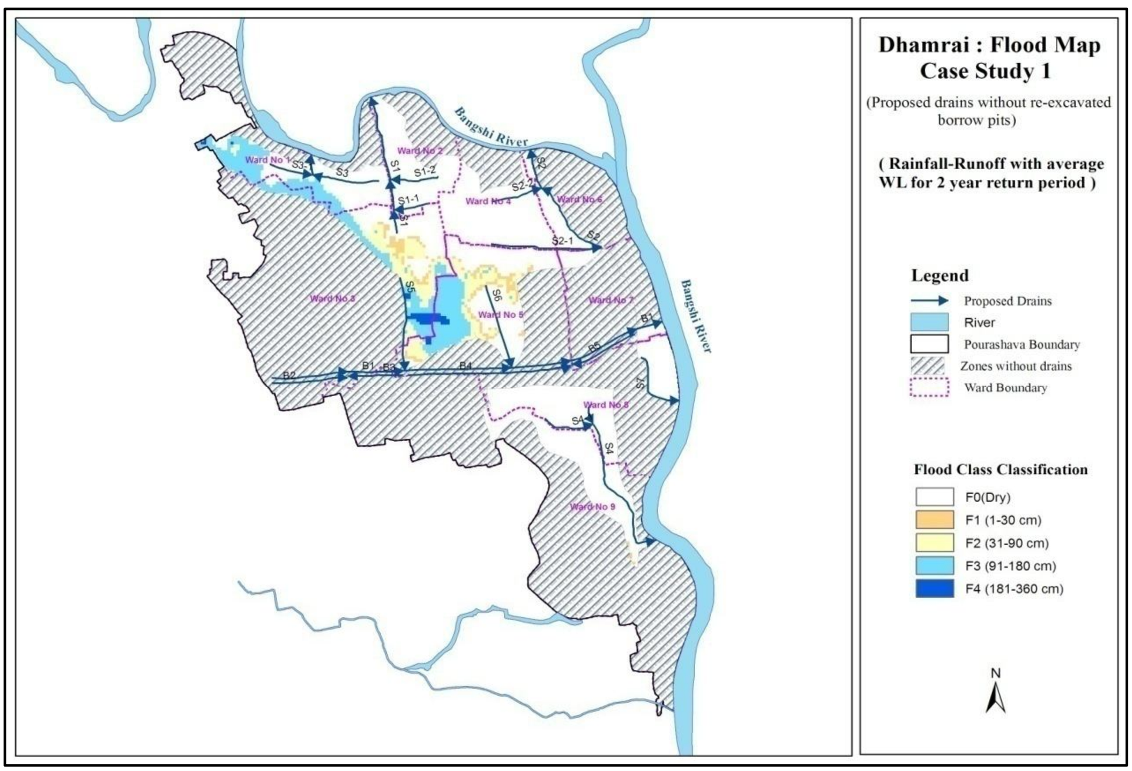


Figure 6.7: Flood Map considering Rainfall-Runoffs with average WL for 2 year return period S2(a)

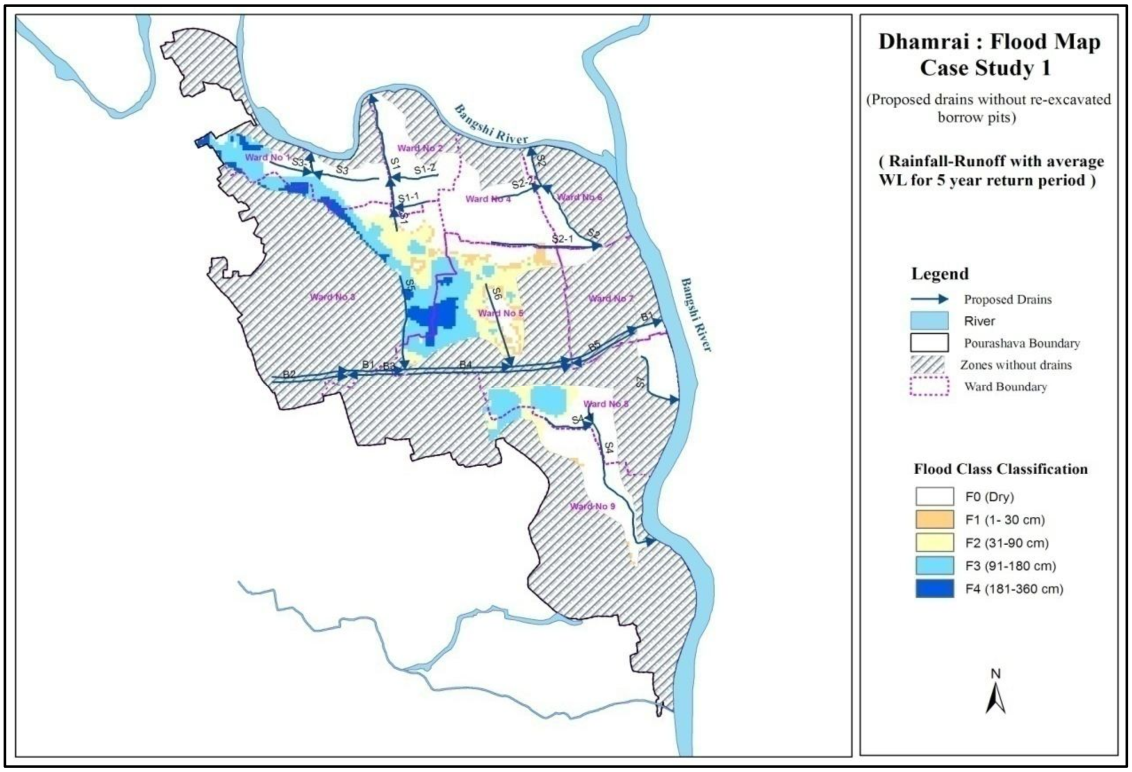


Figure 6.8: Flood Map considering Rainfall-Runoffs with average WL for 5 year return period S2(b)

Table 6.6: Flood Depth Analysis for Urban Areas considering RF and Average Backwater Effect

Location	**FF**	**F0**	**F1**	**F2**	**F3**	**Flood Free Area (FF+F0)**	**Eff. Area (FF to F3)** (Area unit in Ha)
Flood depth cm	water free	1-30	31-90	91-180	181-360		
Avg WL and 2 yr RF event	180.12	6.88	18.56	30.84	2.20	187.00	238.60
Avg WL and 5 yr RF event	143.80	8.24	28.88	45.24	12.44	152.04	238.60

iii) Flooding from Rainfall considering 1 in 5 year WL at the Outfall (S3)

Flood maps considering maximum water level of model results are shown in Figures 6.9 and **6.10** representing 2 and 5-year return period respectively with 1 in 5 year water level. Shallow to deep flooding occurs within some parts of Ward No 1 and 3 and very deep flooding in Ward No 5. Flood depth analysis for the urban area as a whole, considering 1 in 5 year water level at the outfall with 2 and 5 yr rainfall event is shown in Table 6.7. A comparison of percent of inundated area under different scenarios and simulations are represented in Table 6.8.

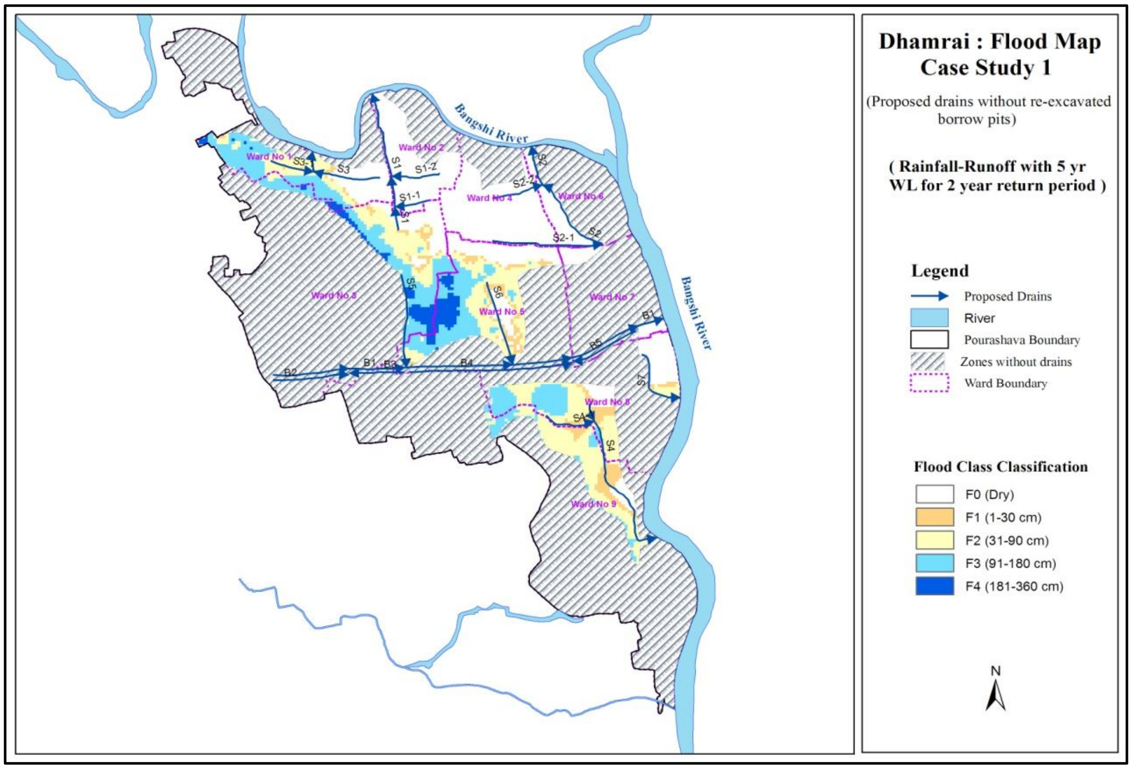


Figure 6.9: Flood Map considering Rainfall-Runoffs with 5yr WL for 2 year return period S3(a)

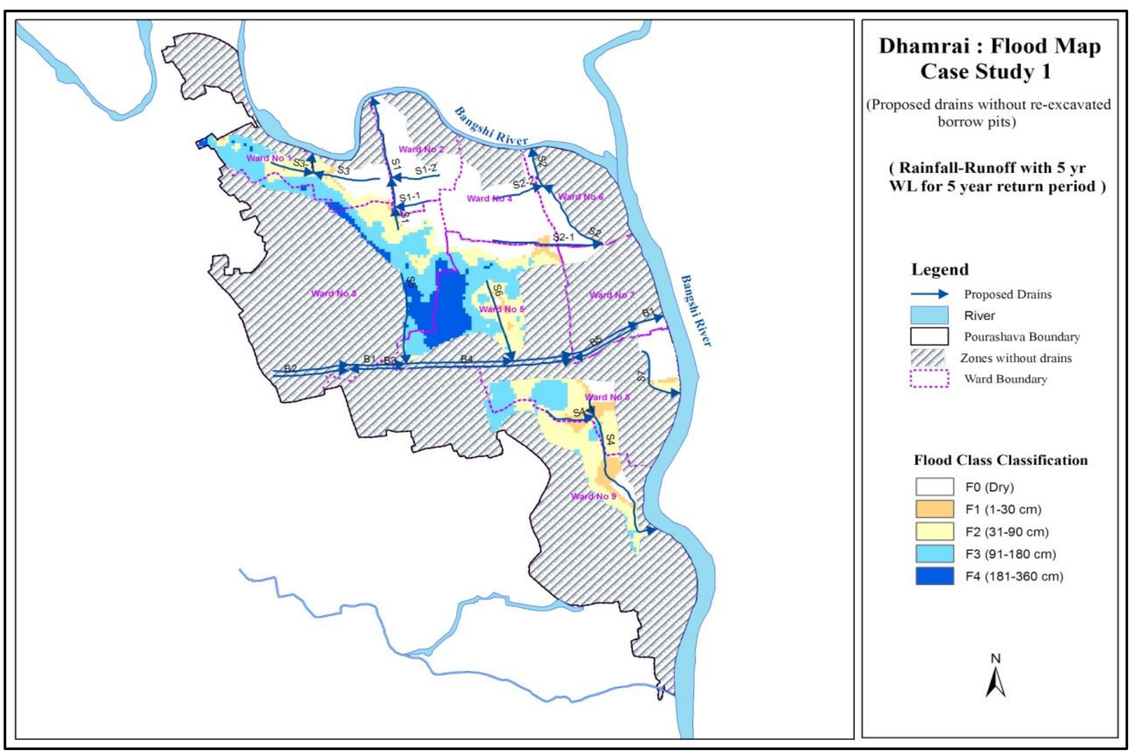


Figure 6.10: Flood Map considering Rainfall-Runoffs with 5yr WL for 5 year return period S3(b)

Table 6.7: Flood Depth Analysis for Urban Areas considering RF and 1 in 5 Year Backwater Effect

Location	FF	F0	F1	F2	F3	Flood Free Area (FF+F0)	Eff. Area (FF to F3)
Flood depth cm	water free	1-30	31-90	91-180	181-360		
1 in 5 yr WL and 2 yr RF event	108.00	15.20	56.92	48.56	9.92	123.20	238.60
1 in 5 yr WL and 5 yr RF event	95.32	11.44	57.80	54.04	20.00	106.76	238.60

Table 6.8: Percent of area inundated in urban areas under different scenario for Case Study 1

Scenarios	FF water free	F0 1-30 cm	F1 31-90 cm	F2 91-180 cm	F3 181-360 cm	Damaging Floods F1+F2+F3
2 yr RF event only S1(a)	88	2	6	4	0	10
5 yr RF event only S1(b)	63	4	14	17	3	33
Avg WL and 2 yr RF event S2(a)	75	3	8	13	1	22
Avg WL and 5 yr RF event S2(b)	60	3	12	19	5	36
1 in 5 yr WL and 2 yr RF event S3(a)	45	6	24	20	4	48
1 in 5 yr WL and 5 yr RF event S3(b)	40	5	24	23	8	55

In Table 6.8, the water logging extent with two different rainfall events with different outfall water level stages shows, if no improvements in the borrow pits are done, the water logging area increases to 22% for S2(a) and 48% for S3(a), indicating 12% and 38% increase in the inundated area comparing to the base S1(a) scenario. Similarly, the water logging area increases to 36% for S2(b) and 55% for S3(b), indicating 3% and 22% increase in the inundated area comparing to the base S1(b) scenario.

6.5.2 Case Study 2 (Proposed Drainage System with re-excavation of the Borrow Pits)

Inundation patterns has been analyzed after the re-excavation of the borrow pits. This section describes the improvement achieved with the proposed storm drains and re-excavated borrow pits.

i) Flooding from RF without Backwater Effect (S1)

It is clear from Figure 6.11 and Figure 6.12 that, no water logging occurs for 2 year return period whereas shallow to deep flooding occurs within some parts of Ward No 1, 3 and 5 for

5 year return period. Flood depth analysis for the urban area as a whole, considering the rainfall effect only is shown in Table 6.9.

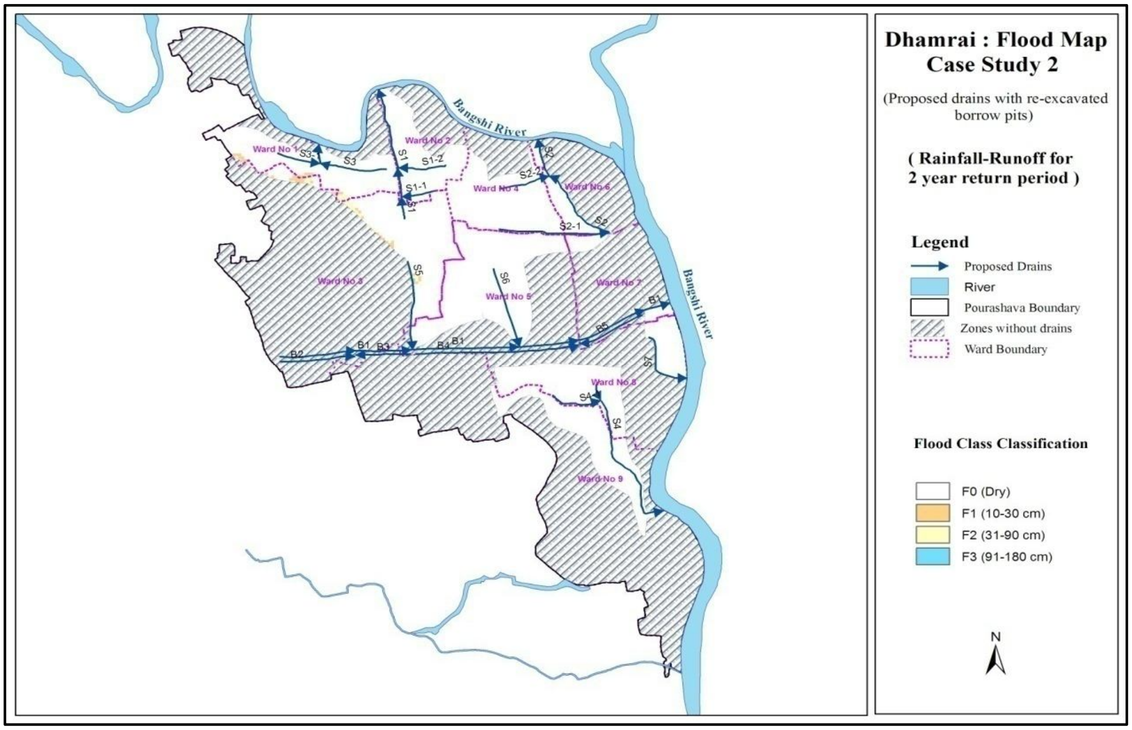


Figure 6.11: Flood Map considering Rainfall-Runoffs for 2 year return period S1(a)

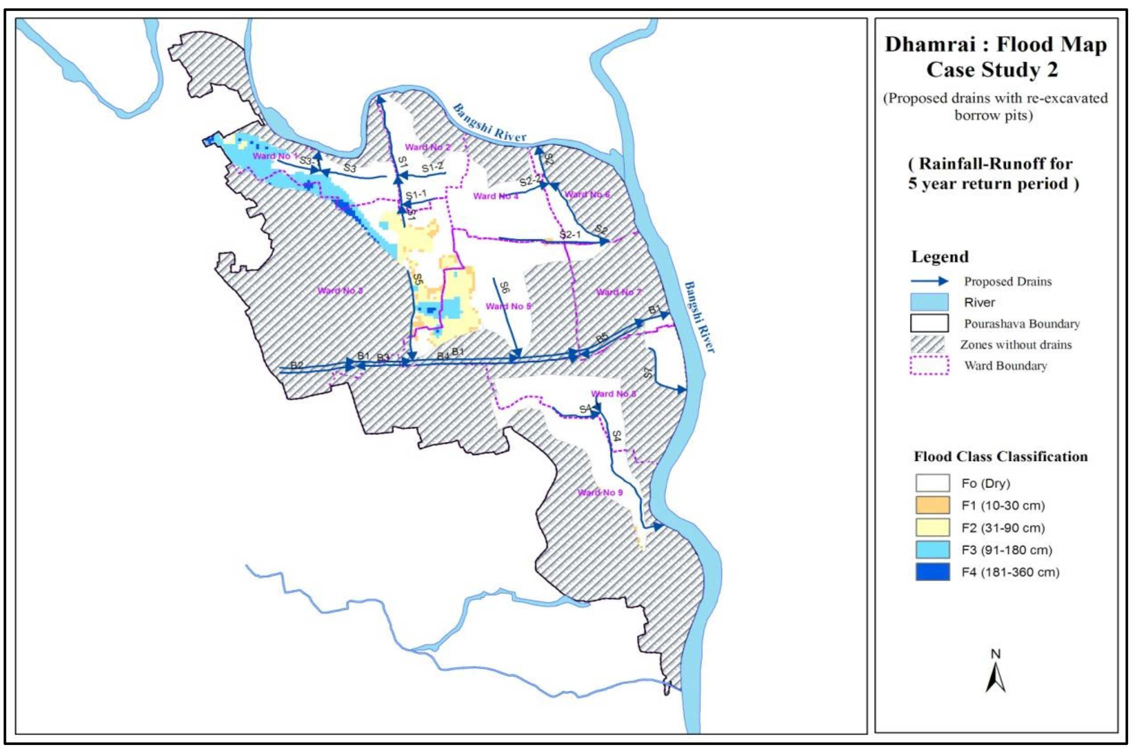


Figure 6.12: Flood Map considering Rainfall-Runoffs for 5 year return period S1(b)

Table 6.9: Flood Depth Analysis for Urban Areas considering RF Effect only

Location	**FF**	**F0**	**F1**	**F2**	**F3**	**Flood Free Area (FF+F0)**	**Eff. Area (FF to F3)** (Area unit in Ha)
Flood depth cm	water free	1-30	31-90	91-180	181-360		
2 yr RF event only	235.12	1.88	1.56	0.04	0.00	237.00	238.60
5 yr RF event only	190.64	5.44	18.92	21.16	2.44	196.08	238.60

ii) Flooding from Rainfall considering Average WL at the Outfall (S2)

Flood maps considering maximum water level of model results are shown in Figures 6.13 and 6.14 representing 2 and 5-year return period respectively with average water level. Shallow to deep flooding occurs within some parts of Ward no 1, 3 and 5. Flood depth analysis for the urban area as a whole, considering the average water level at the outfall with 2 and 5 yr rainfall event is shown in Table 6.10.

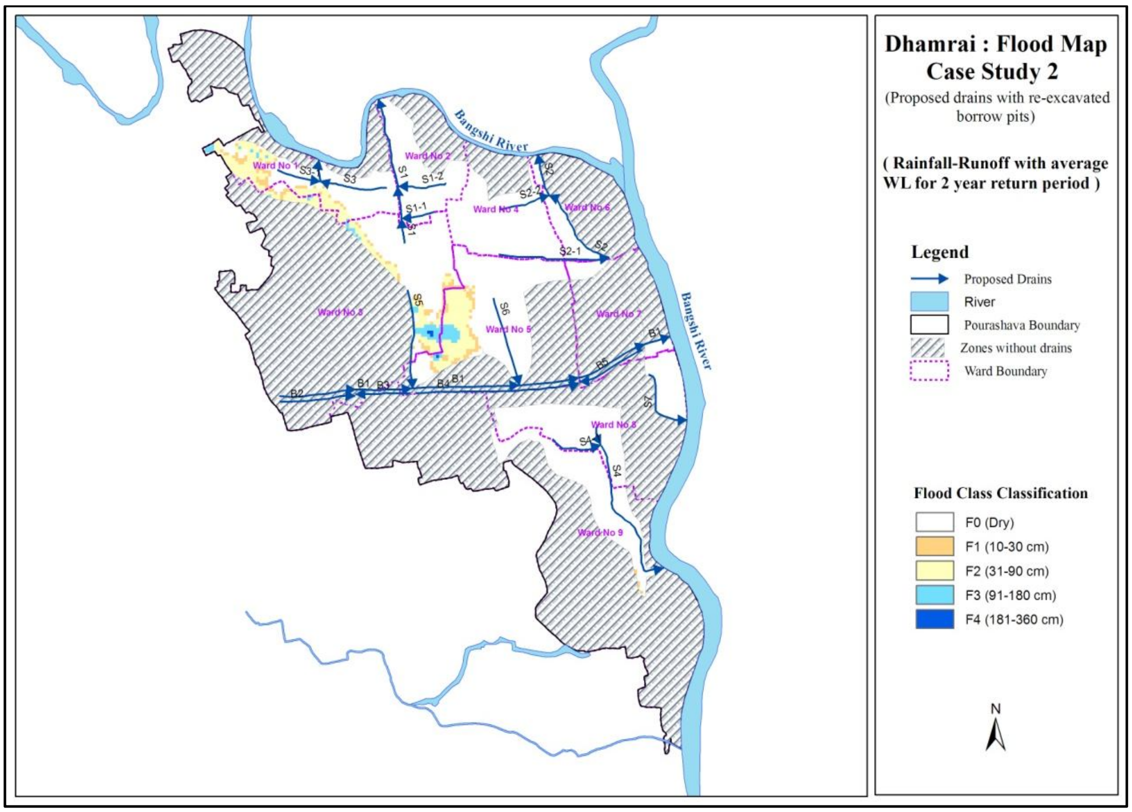


Figure 6.13: Flood Map considering Rainfall-Runoffs with average WL for 2 year return period S2(a)

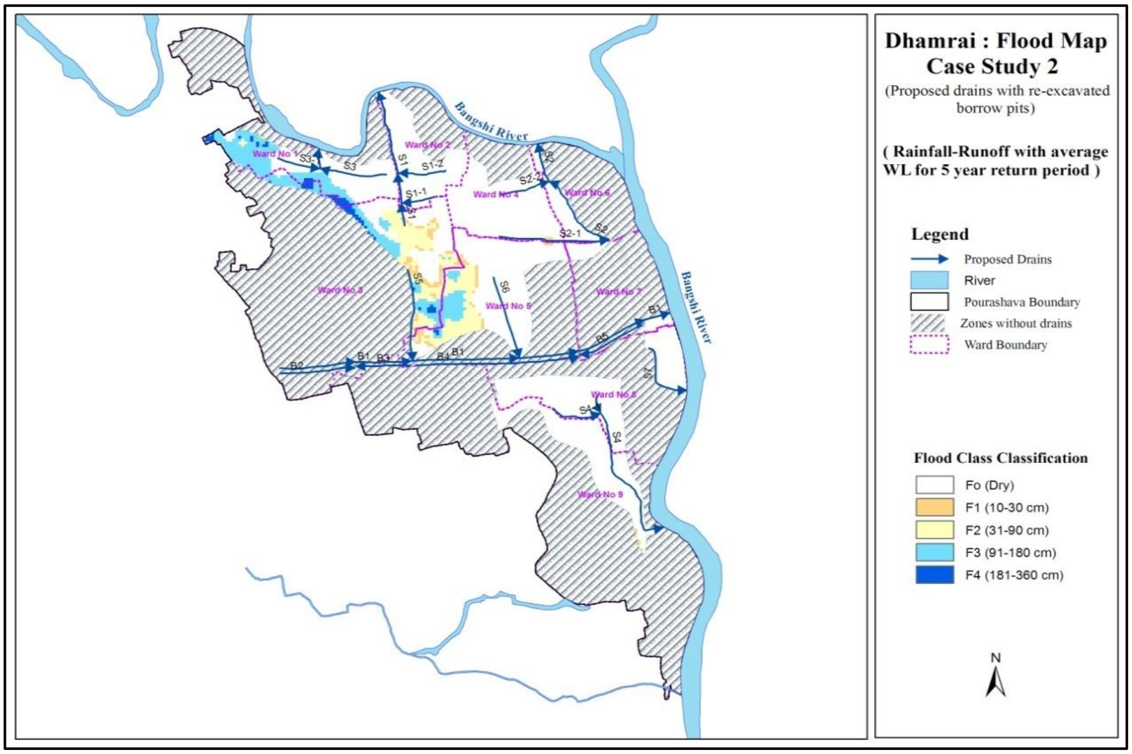


Figure 6.14: Flood Map considering Rainfall-Runoffs with average WL for 5 year return period S2(b)

Table 6.10: Flood Depth Analysis for Urban Areas considering RF and Average Backwater Effect

Location	**FF**	**F0**	**F1**	**F2**	**F3**	**Flood Free Area (FF+F0)**	**Eff. Area (FF to F3)** (Area unit in Ha**)**
Flood depth cm	water free	1-30	31-90	91-180	181-360		
Avg WL and 2 yr RF event	204.60	7.44	22.32	4.08	0.16	212.04	238.60
Avg WL and 5 yr RF event	187.92	5.68	19.40	22.32	3.28	193.60	238.60

iii) Flooding from Rainfall considering 1 in 5 year WL at the Outfall (S3)

Flood maps considering maximum water level of model results are shown in Figures 6.15 and 6.16 representing 2 and 5-year return period respectively with 1 in 5 year water level. Shallow to deep flooding occurs within some parts of Ward No 1, 3, 5 and 8. Flood depth analysis for the urban area as a whole, considering 1 in 5 year water level at the outfall with 2 and 5 yr rainfall event is shown in Table 6.7. A comparison of percent of inundated area under different scenario is represented in Table 6.11.

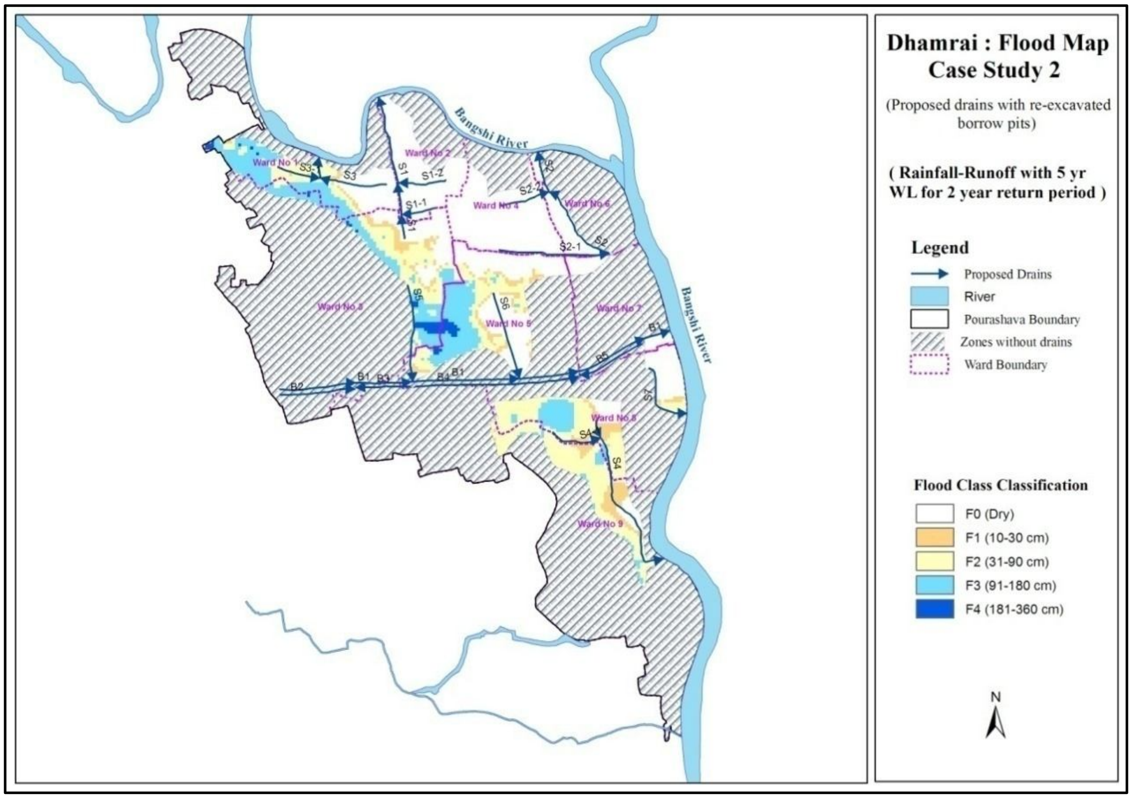


Figure 6.15: Flood Map considering Rainfall-Runoffs with 5yr WL for 2 year return period S3(a)

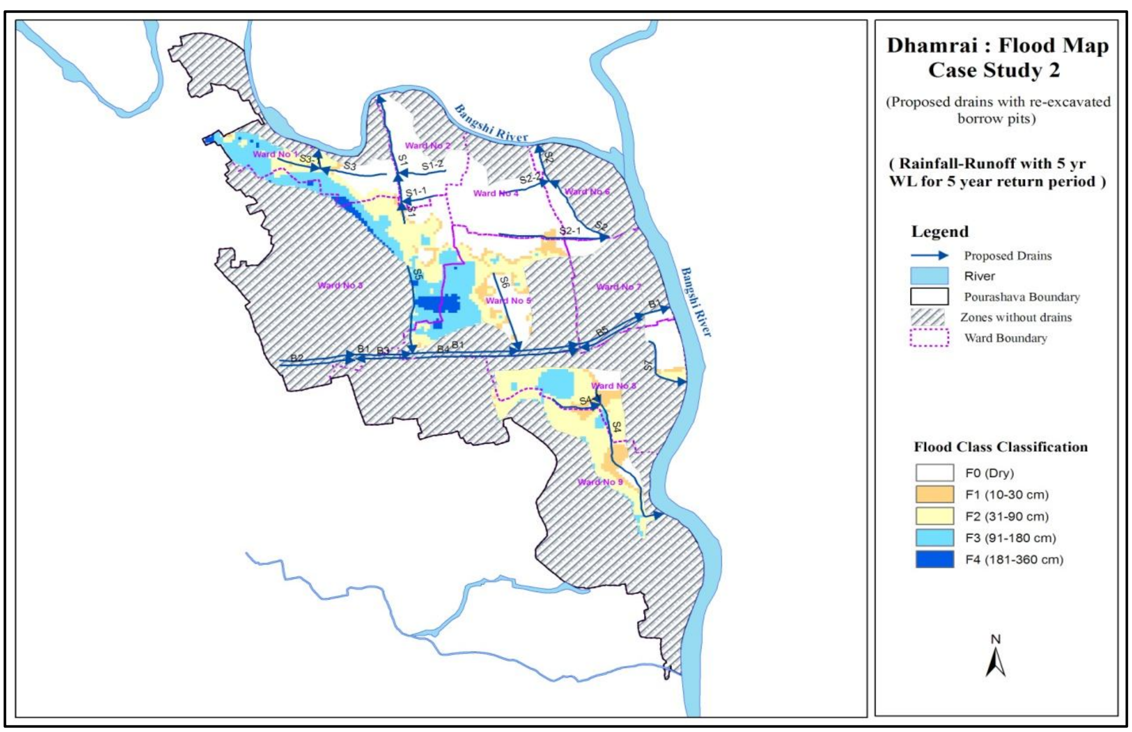


Figure 6.16: Flood Map considering Rainfall-Runoffs with 5yr WL for 5 year return period S3(b)

Table 6.11: Flood Depth Analysis for Urban Areas considering RF and 1 in 5 Year Backwater Effect

Location	FF	F0	F1	F2	F3	Flood Free Area (FF+F0)	Eff. Area (FF to F3) (Area unit in Ha)
Flood depth cm	water free	1-30	31-90	91-180	181-360		
1 in 5 yr WL and 2 yr RF event	120.20	15.96	58.56	40.68	3.20	136.16	238.60
1 in 5 yr WL and 5 yr RF event	109.44	16.60	63.20	42.96	6.40	126.04	238.60

Table 6.12: Percent of area inundated in urban areas under different scenario for Case Study 2

Scenarios	FF water free	F0 1-30 cm	F1 31-90 cm	F2 91-180 cm	F3 181-360 cm	Damaging Floods F1+F2+F3
2 yr RF event only S1(a)	99	0	1	0	0	1
5 yr RF event only S1(b)	80	2	8	9	1	18
Avg WL and 2 yr RF event S2(a)	86	3	9	2	0	11
Avg WL and 5 yr RF event S2(b)	79	2	8	9	1	19
1 in 5 yr WL and 2 yr RF event S3(a)	50	7	25	17	1	43
1 in 5 yr WL and 5 yr RF event S3(b)	46	7	26	18	3	47

In Table 6.12, the water logging extent with two different rainfall events with different outfall water level stages shows, if improvements in the borrow pits are done, the water logging area increases to 11% for S2(a) and 43% for S3(a), indicating 10% and 42% increase in the inundated area compared to the base scenario S1(a). Similarly, the water logging area increases to 19% for S2(b) and 47% for S3(b), indicating 1% and 29% increase in the inundated area compared to base scenario S1(b). Figure 6.17 to 6.19 represents the water logging extent for two case studies under different scenarios and simulations.

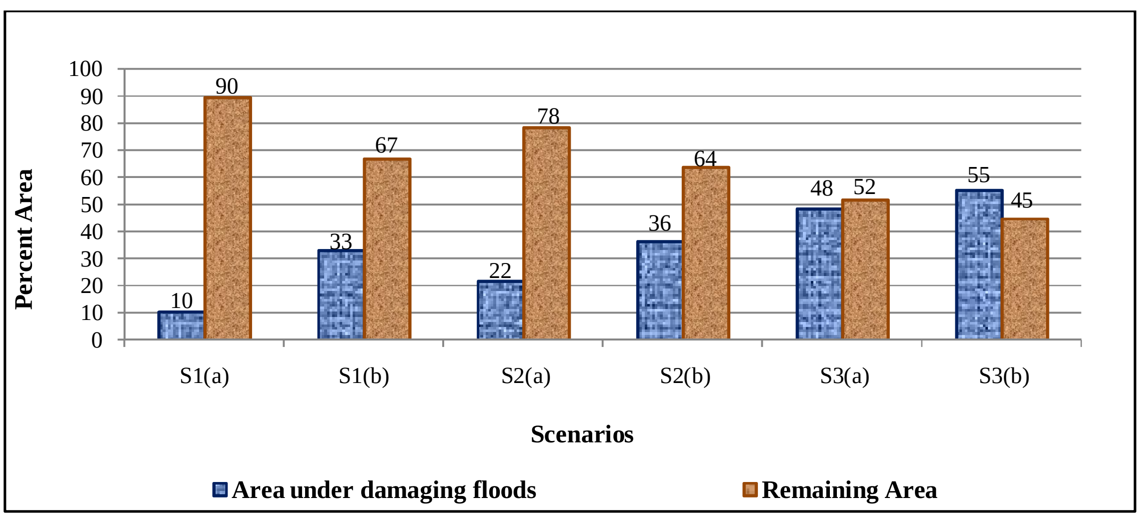


Figure 6.17: Water Logging extent with proposed drainage system for Case Study 1

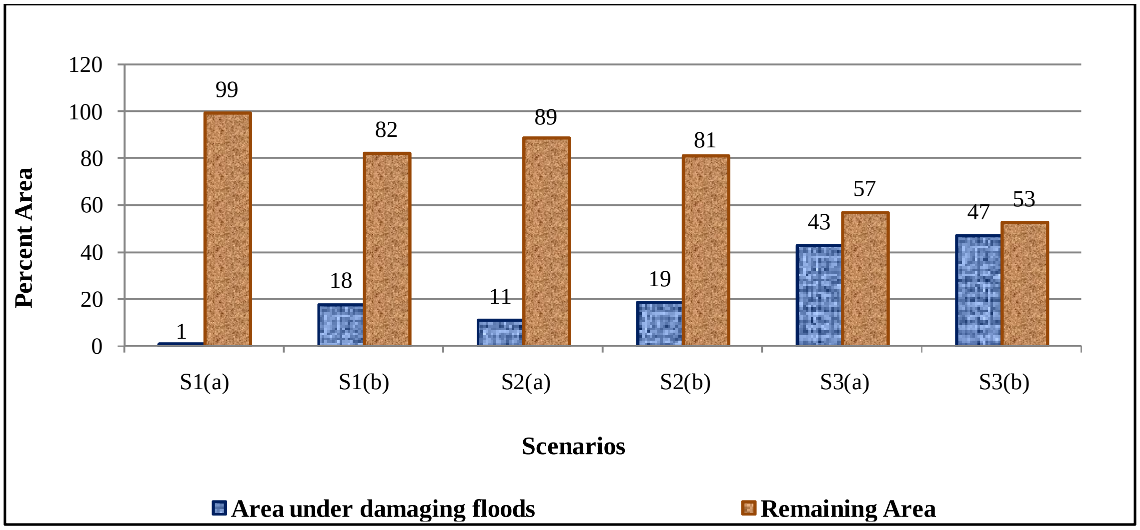


Figure 6.18: Water Logging extent with proposed drainage system for Case Study 2

The above Figure 6.17 and 6.18 indicates that, worse scenario exists for back water effects with 1 in 5 year outfall water level stages having only 52%, 45%, 57% and 53% of land above flood levels.

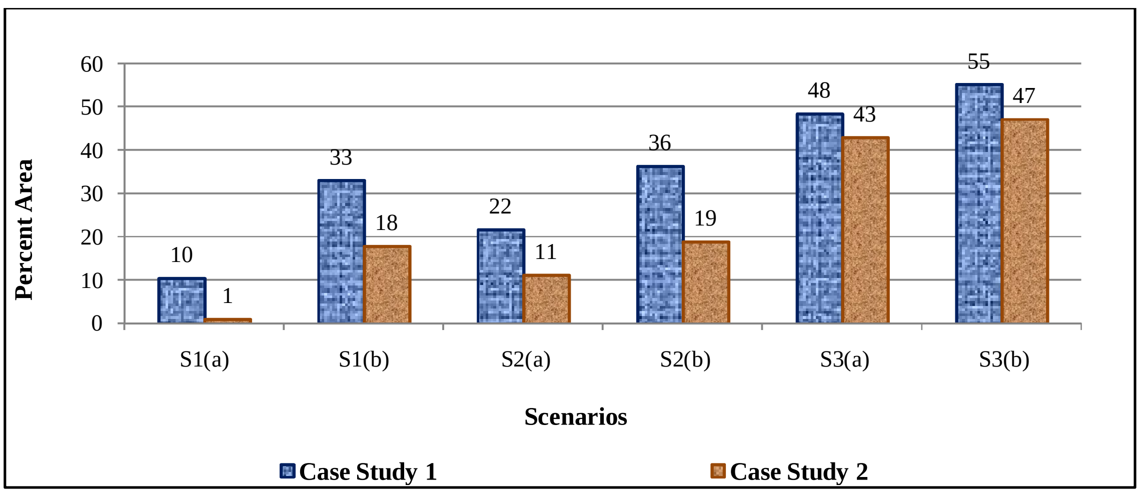


Figure 6.19: Comparison of Water Logging extent with proposed drainage system for Case Study 1 and 2

The above figure depicts that, due to improvement in the borrow pits, the flood extent makes an improvement of about 9%, 15%, 10%, 17%, 5% and 8% of land above flood levels, for three scenarios and simulations.

6.6 Estimation of Damages

Digital Elevation Model and flood depth maps for different rainfall events with different outfall water level stages are readily available for analytical purposes. The spatial resolutions of these maps are 20 x 20 m. In order to make the landuse data compatible with the above stated types it is rasterised with the same spatial resolutions. The study is only concerned with homestead, commercial and industrial areas so it is separated from the other types with queries. Major differences between flood depth and damage map is that, the flood depth maps depict the intensity of flood water all over the study area while the damage maps illustrate the pattern of damages. Table 6.13 and Table 6.14 represent landuse pattern distribution in the study area as a whole and only for the concerned urban area. The landuse map is presented in Figure 6.20.

Table 6.13: Landuse pattern distribution in the study area as a whole

Landuse Type	Area (Ha)	Percentage (%)
Agricultural Land	211	30.22
Commercial & Others	18	2.60
Homestead	397	56.79
Industry	54	7.71
Road	19	2.68
Total	**700**	**100.00**

Table 6.14: Landuse pattern distribution in the study area only for urban area

Landuse Type	Area (Ha)	Percentage (%)
Agricultural Land	15	6
Commercial & Others	11	4
Homestead	191	80
Industry	12	5
Road	10	4
Total	**238**	**100**

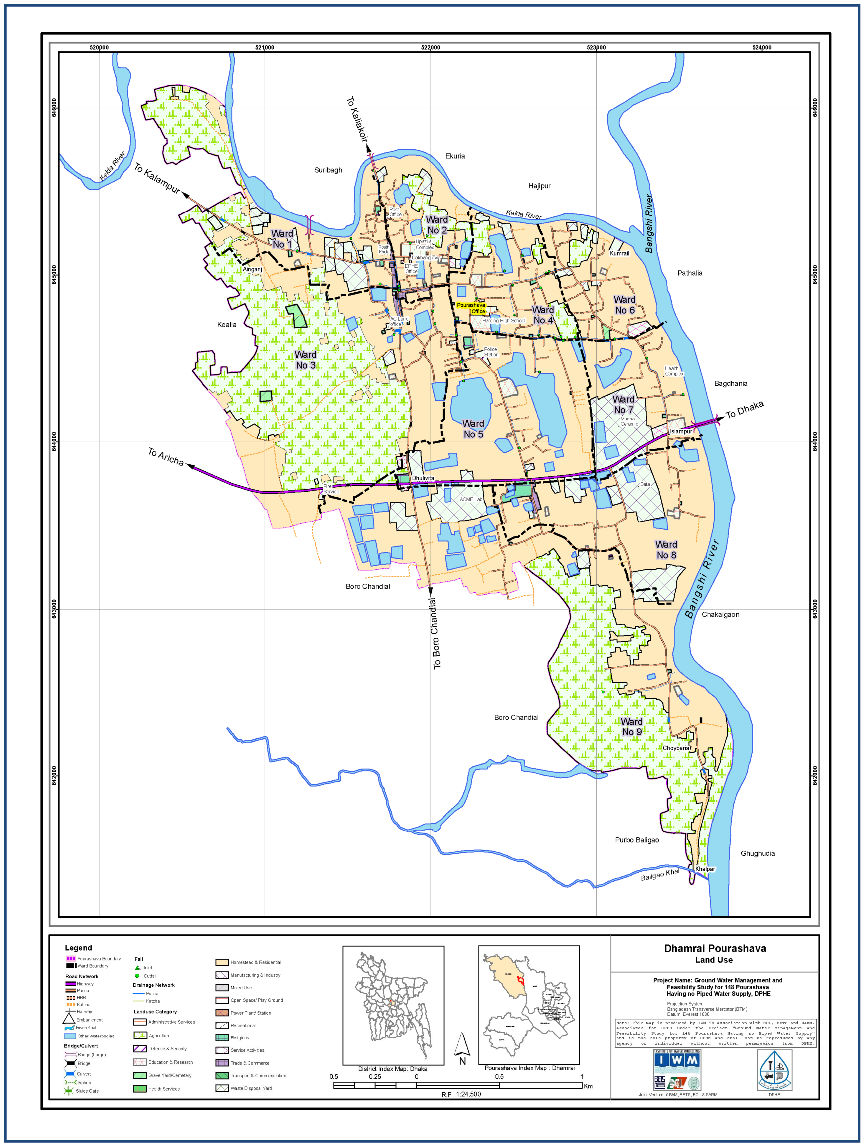


Figure 6.20: Landuse Pattern of Dhamrai

6.6.1 Case Study 1 (Proposed Drainage System without re-excavation of the Borrow Pits)

Damage maps are developed for the results obtained from Case Study 1. The damages are indicated for the homesteads, commercials and industrial sectors only. Figure 6.21 to 6.26 represents the types of damages under different rainfall event with different outfall water level stages. The percentage of damages, under three different damage patterns for homesteads, commercials and industrial are presented in Table 6.15 to Table 6.17. The damages are classified as low, moderate and severe damages.

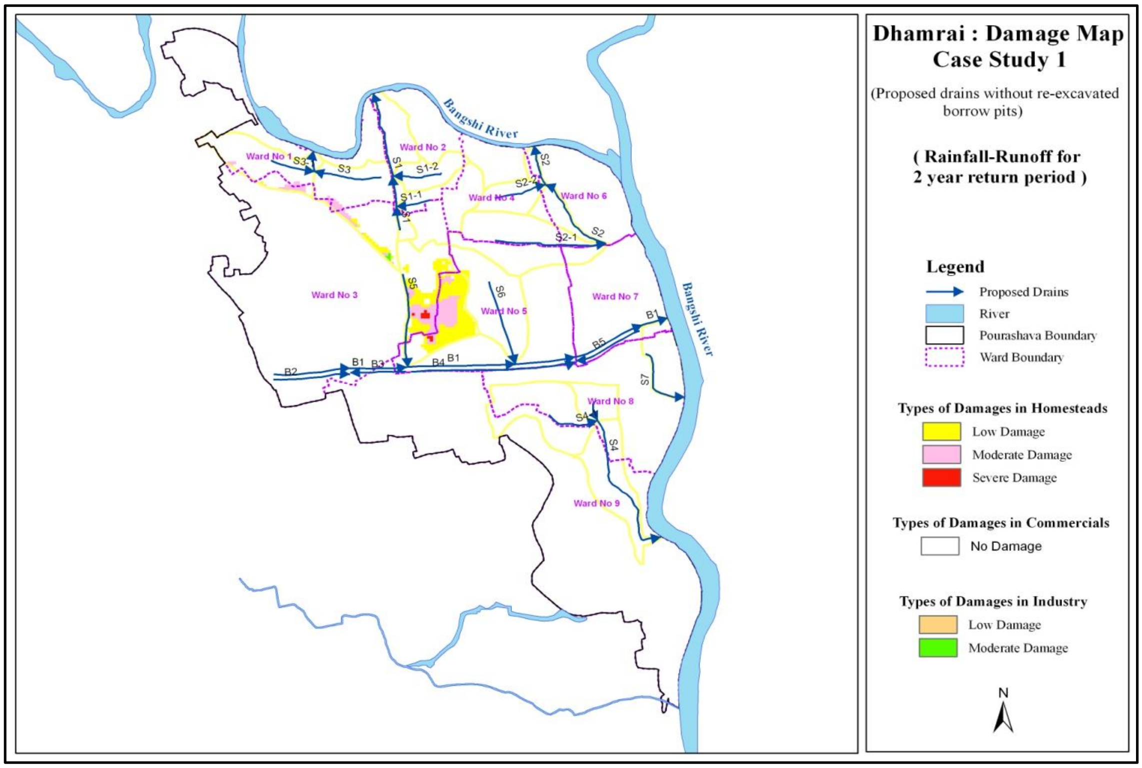


Figure 6.21: Damages from Rainfall-Runoffs for 2 year return period S1(a)

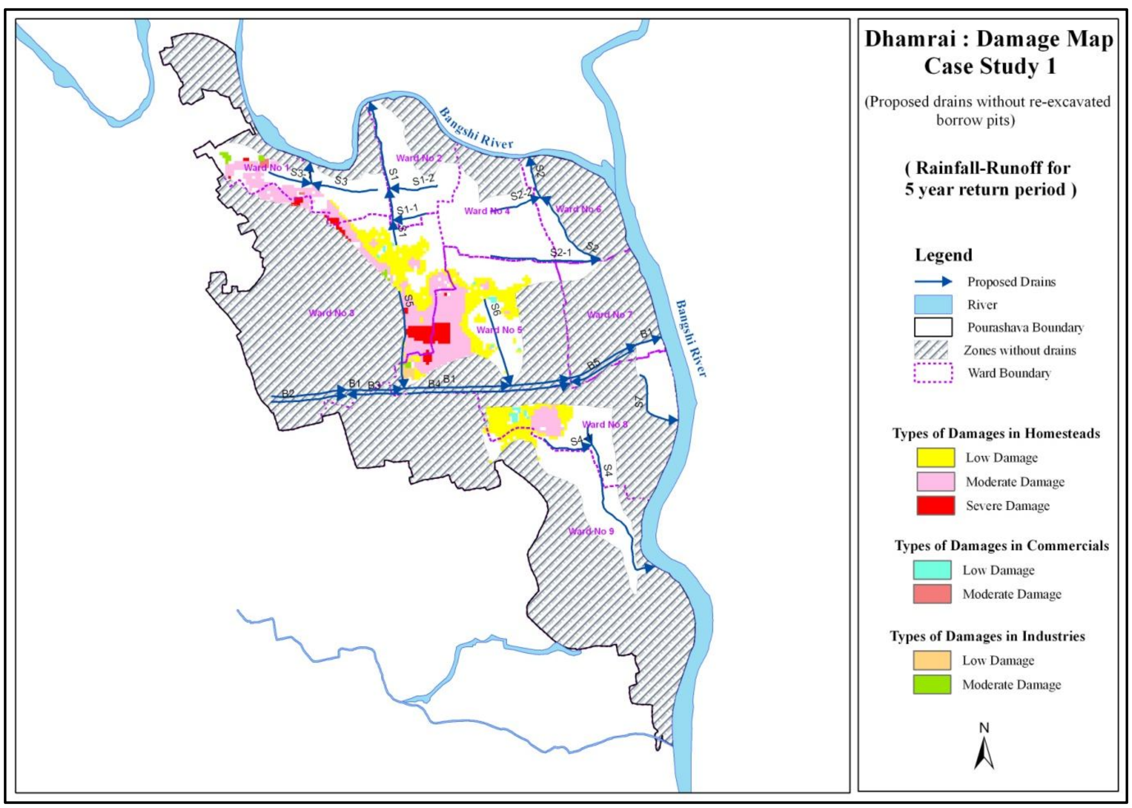


Figure 6.22: Damages from Rainfall-Runoffs for 5 year return period S1(b)

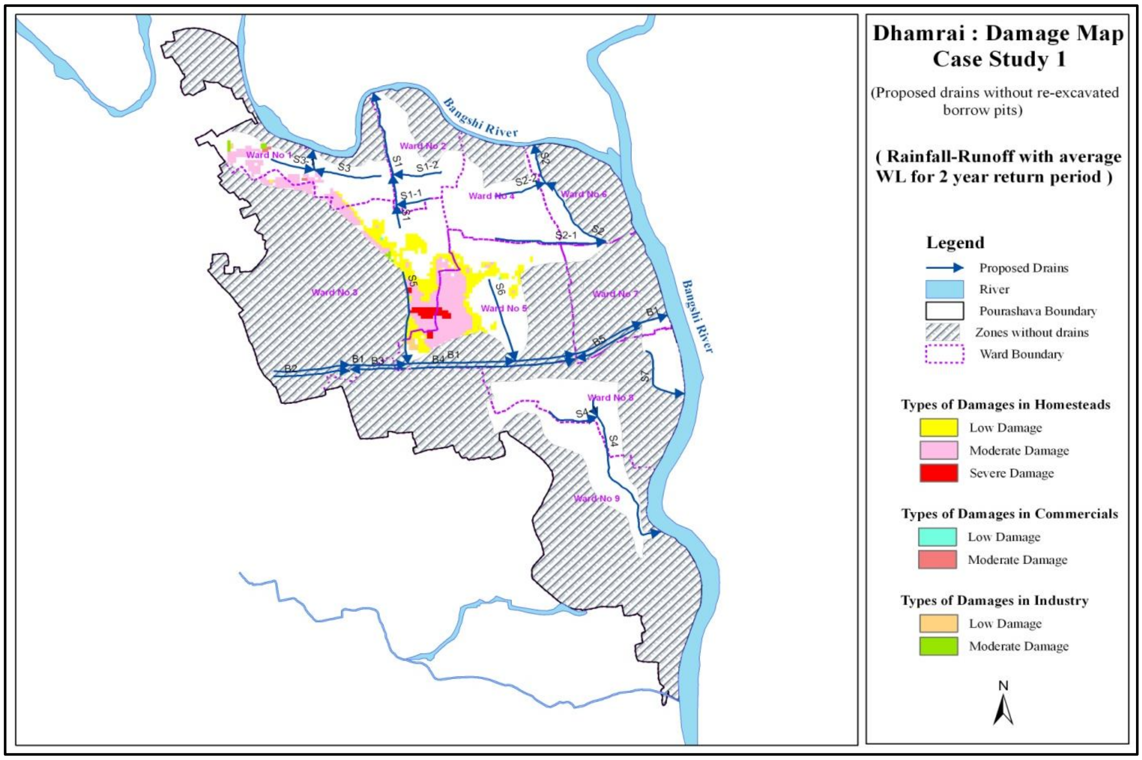


Figure 6.23: Damages from Rainfall-Runoffs with average WL for 2 year return period S2(a)

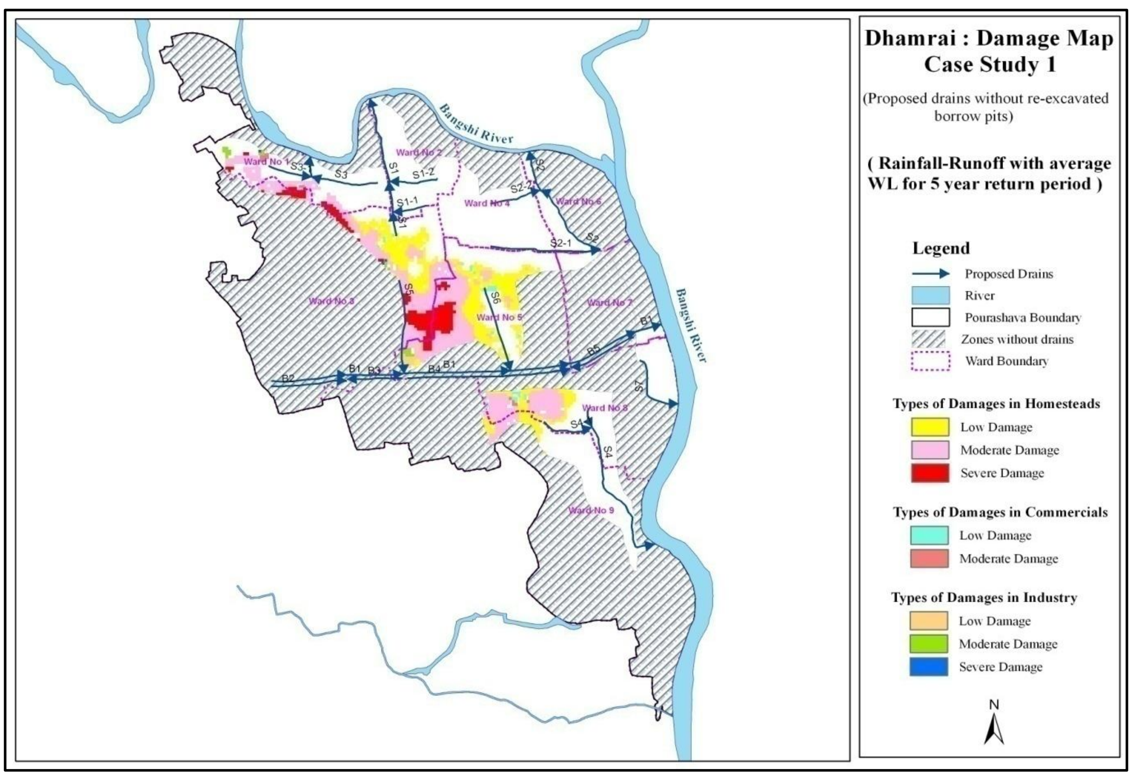


Figure 6.24: Damages from Rainfall-Runoffs with average WL for 5 year return period S2(b)

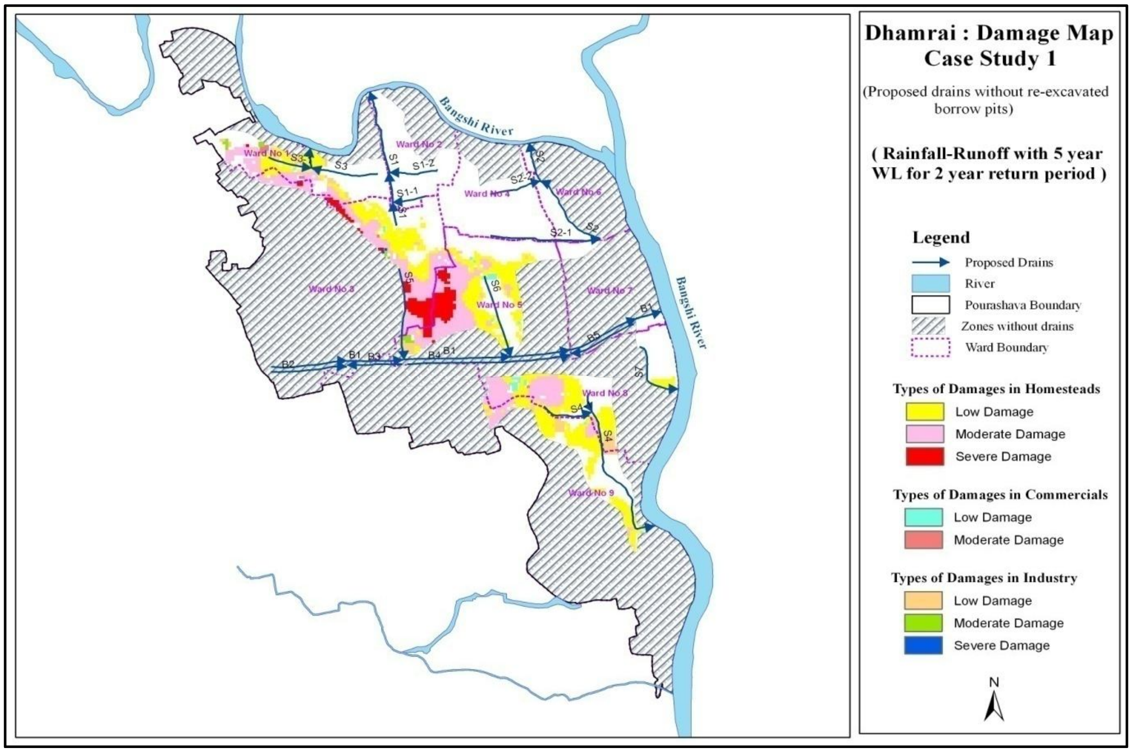


Figure 6.25: Damages from Rainfall-Runoffs with 1 in 5 year WL for 2 year return period S3(a)

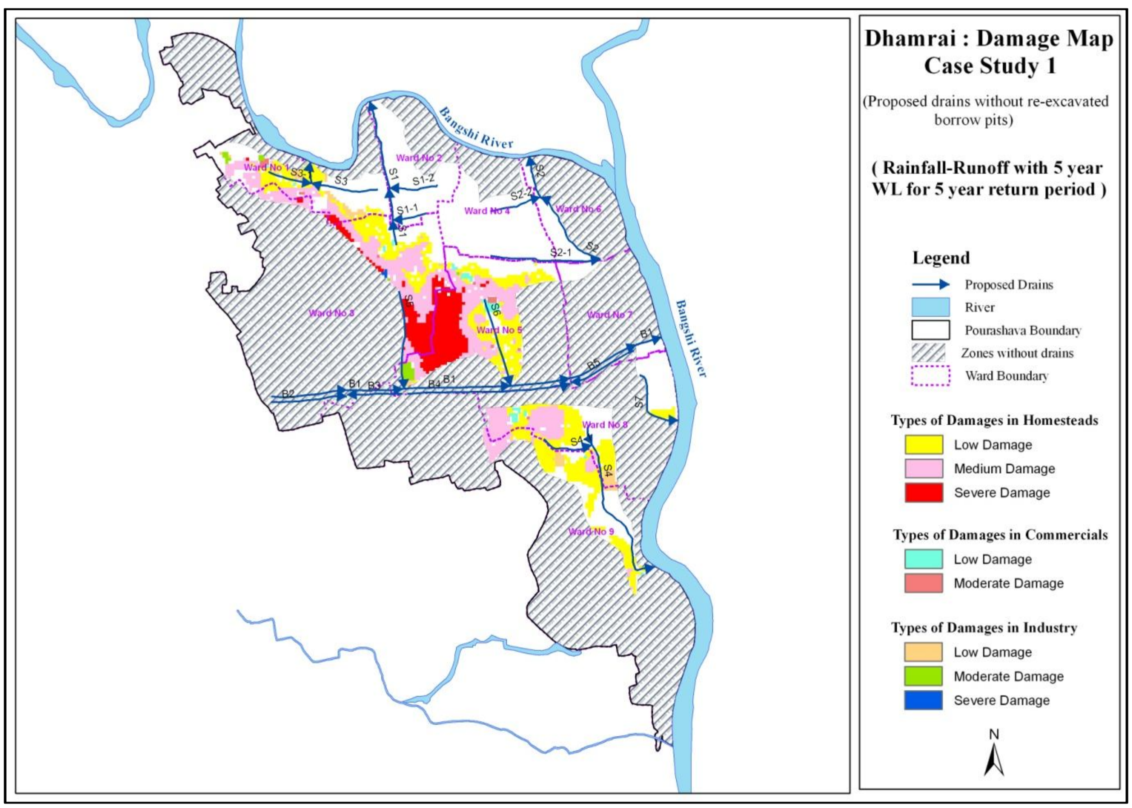


Figure 6.26: Damages from Rainfall-Runoffs with 1 in 5 year WL for 5 year return period S3(b)

Table 6.15: Percent of damaged areas in Homesteads

Scenarios	Low Damage	Moderate Damage	Severe Damage
2 yr RF event only S1(a)	7	4	0
5 yr RF event only S1(b)	15	16	3
Avg WL and 2 yr RF event S2(a)	9	12	1
Avg WL and 5 yr RF event S2(b)	13	20	5
1 in 5 yr WL and 2 yr RF event S3(a)	24	20	4
1 in 5 yr WL and 5 yr RF event S3(b)	24	23	9

Table 6.16: Percent of damaged areas in Commercials

Scenarios	Low Damage	Moderate Damage	Severe Damage
2 yr RF event only S1(a)	0	0	0
5 yr RF event only S1(b)	10	4	0
Avg WL and 2 yr RF event S2(a)	1	3	0
Avg WL and 5 yr RF event S2(b)	10	6	0
1 in 5 yr WL and 2 yr RF event S3(a)	12	4	0
1 in 5 yr WL and 5 yr RF event S3(b)	19	8	0

Table 6.17: Percent of damaged areas in Industries

Scenarios	Low Damage	Moderate Damage	Severe Damage
2 yr RF event only S1(a)	1	1	0
5 yr RF event only S1(b)	7	8	0
Avg WL and 2 yr RF event S2(a)	5	5	0
Avg WL and 5 yr RF event S2(b)	7	8	1
1 in 5 yr WL and 2 yr RF event S3(a)	35	7	1
1 in 5 yr WL and 5 yr RF event S3(b)	38	12	1

It is clear from the above tables that, severe damages will occur only in homesteads whereas commercial and industries undergo low to moderate type of damages. Severe damages within the homesteads increases 2% and 6% for the back water effect from river Bangshi.

6.6.2 Case Study 2 (Proposed Drainage System with re-excavation of the Borrow Pits)

Similar analyses were carried out for the results obtained from Case Study 2. Figure 6.27 to 6.32 represents the types of damages under different rainfall event with different outfall water level stages. The percentage of damages, under three different damage patterns for homesteads, commercials and industrial are presented in Table 6.18 to Table 6.20.

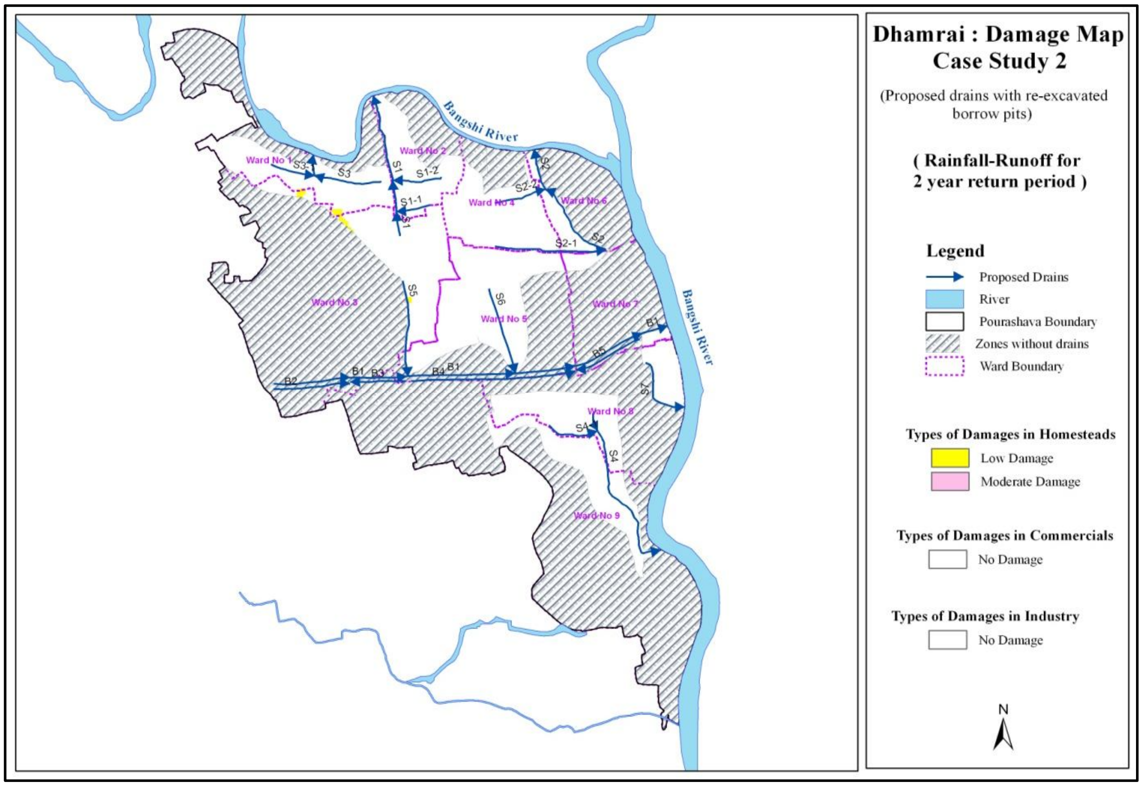


Figure 6.27: Damages from Rainfall-Runoffs for 2 year return period S1(a)

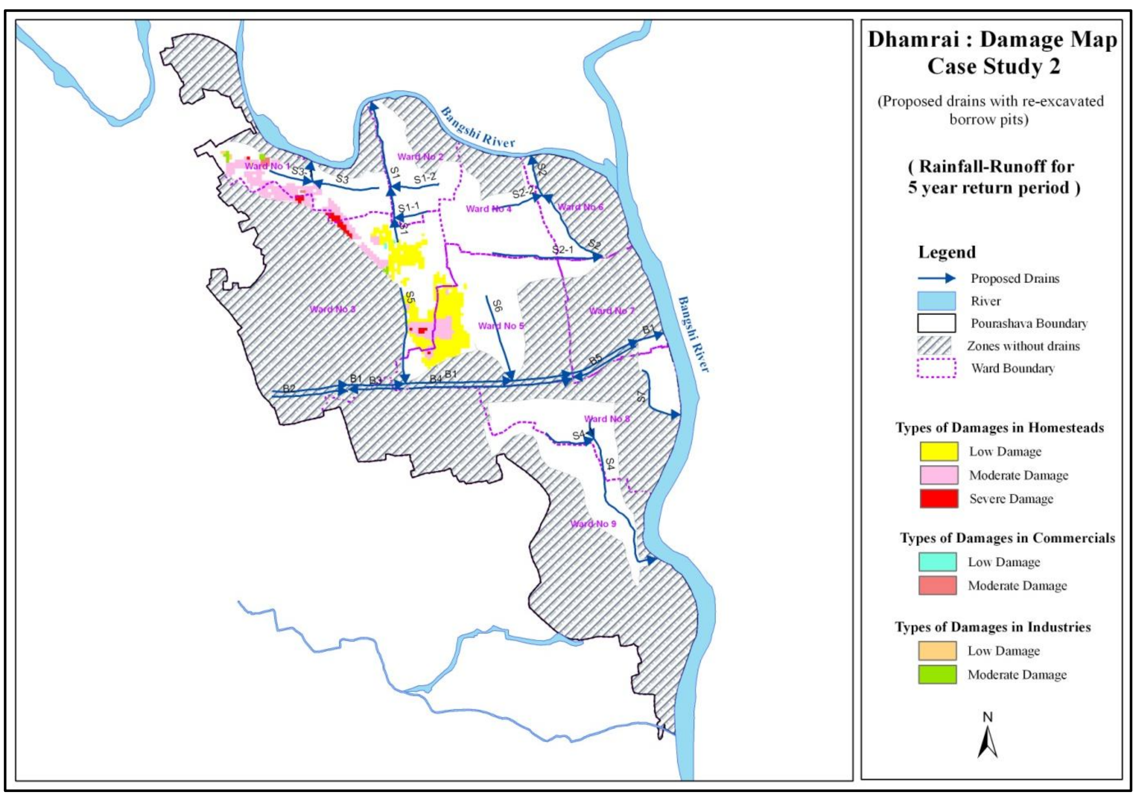


Figure 6.28: Damages from Rainfall-Runoffs for 5 year return period S1(b)

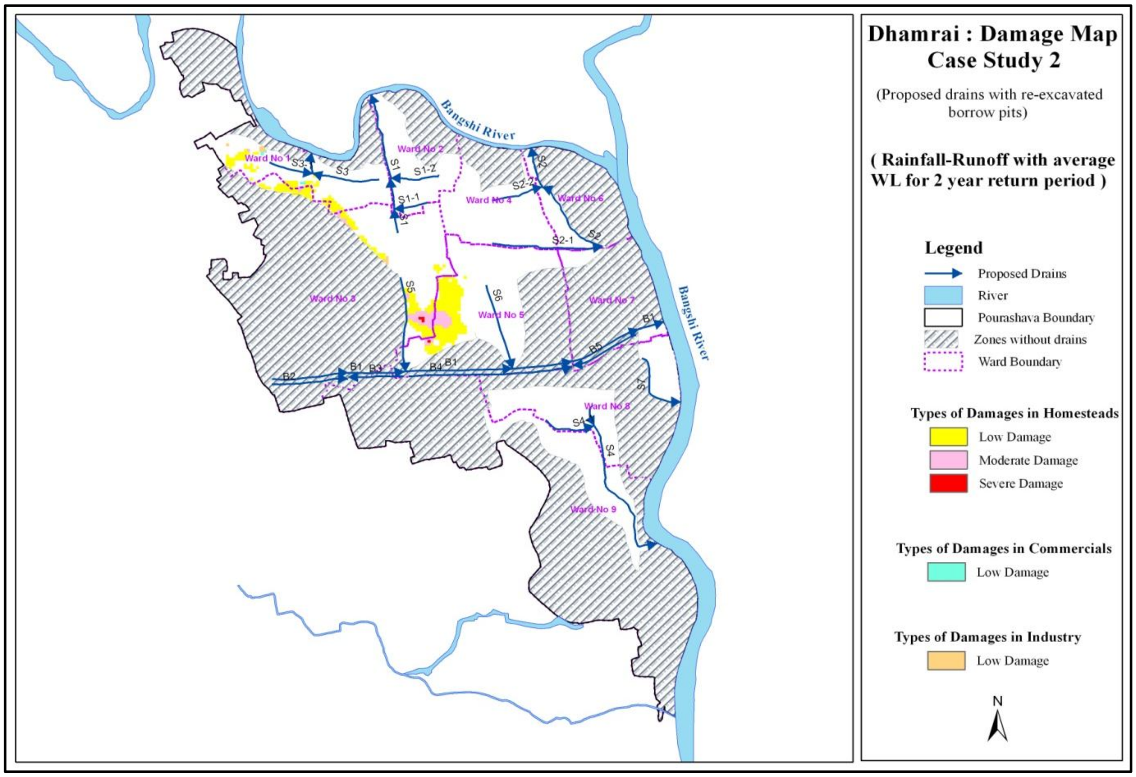


Figure 6.29: Damages from Rainfall-Runoffs with average WL for 2 year return period S2(a)

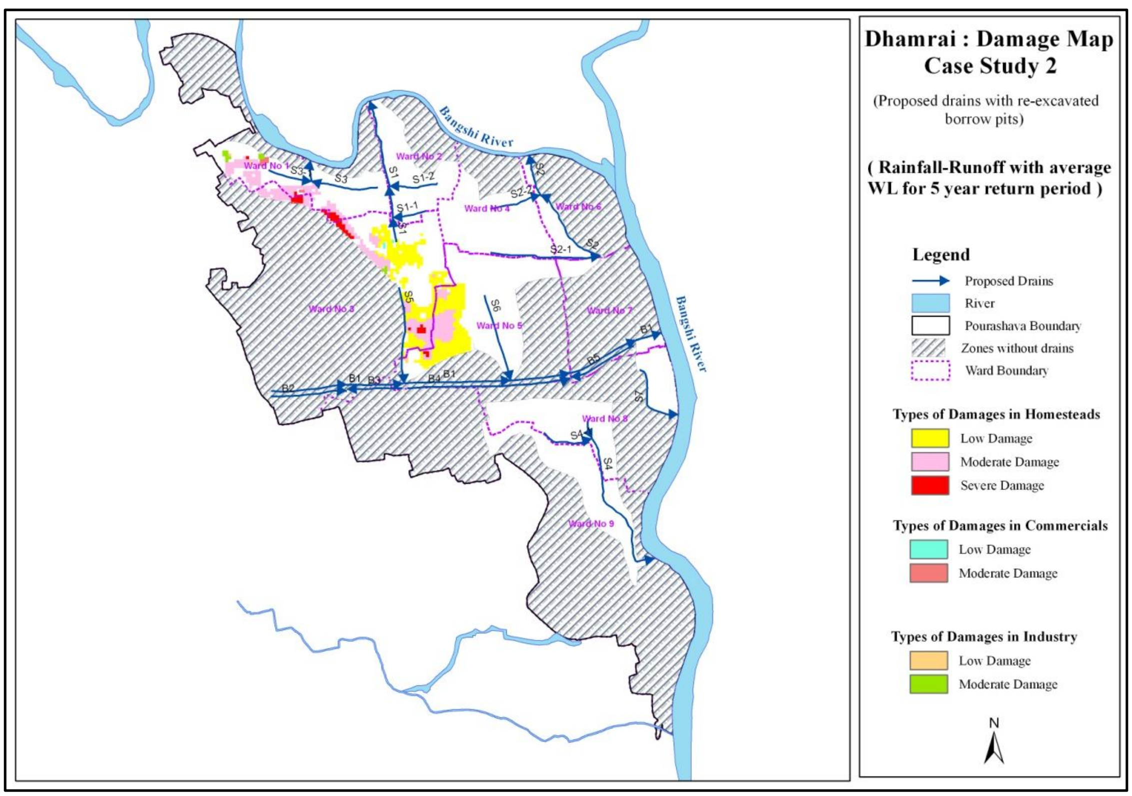


Figure 6.30: Damages from Rainfall-Runoffs with average WL for 5 year return period S2(b)

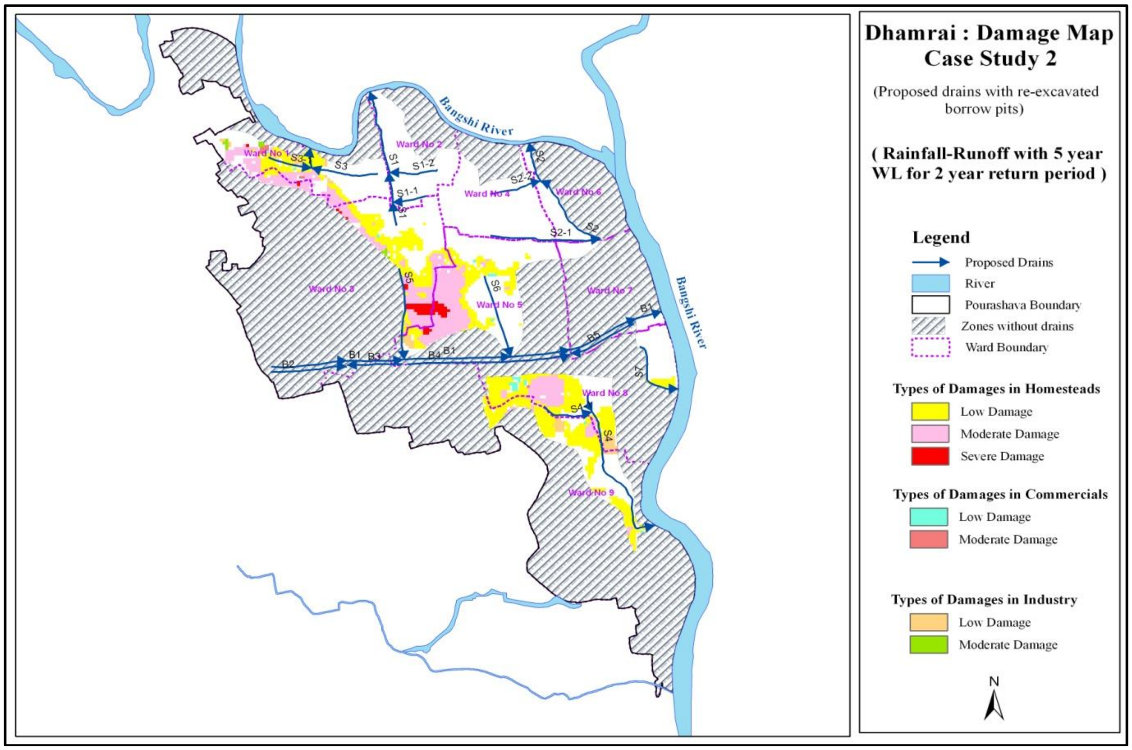


Figure 6.31 Damages from Rainfall-Runoffs with 1 in 5 year WL for 2 year return period S3(a)

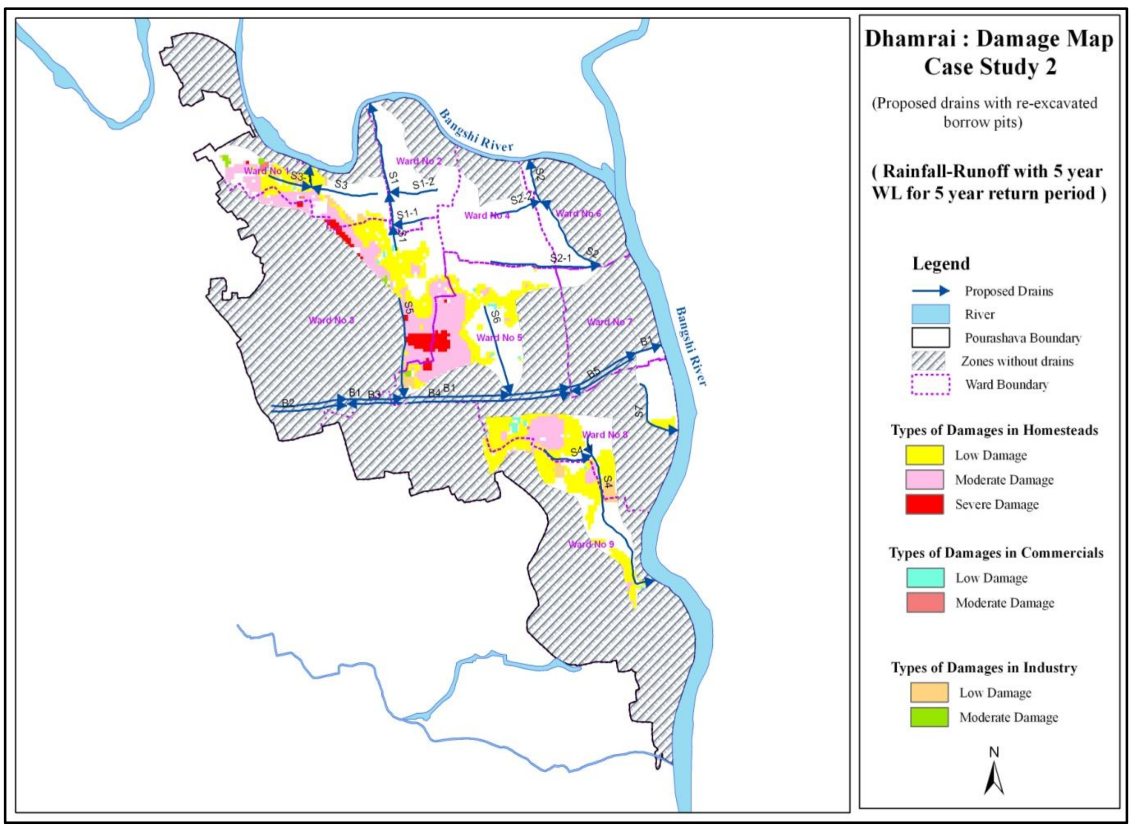


Figure 6.32: Damages from Rainfall-Runoffs with 1 in 5 year WL for 5 year return period S3(b)

Table 6.18: Percent of damaged areas in Homesteads

Scenarios	Low Damage	Moderate Damage	Severe Damage
2 yr RF event only S1(a)	1	0	0
5 yr RF event only S1(b)	9	7	1
Avg WL and 2 yr RF event S2(a)	8	2	0
Avg WL and 5 yr RF event S2(b)	9	8	1
1 in 5 yr WL and 2 yr RF event S3(a)	25	16	1
1 in 5 yr WL and 5 yr RF event S3(b)	27	18	3

Table 6.19: Percent of damaged areas in Commercials

Scenarios	Low Damage	Moderate Damage	Severe Damage
2 yr RF event only S1(a)	0	0	0
5 yr RF event only S1(b)	1	4	0
Avg WL and 2 yr RF event S2(a)	2	0	0
Avg WL and 5 yr RF event S2(b)	1	4	0
1 in 5 yr WL and 2 yr RF event S3(a)	9	4	0
1 in 5 yr WL and 5 yr RF event S3(b)	12	4	0

Table 6.20: Percent of damaged areas in Industries

Scenarios	Low Damage	Moderate Damage	Severe Damage
2 yr RF event only S1(a)	0	0	0
5 yr RF event only S1(b)	2	5	0
Avg WL and 2 yr RF event S2(a)	3	0	0
Avg WL and 5 yr RF event S2(b)	2	5	0
1 in 5 yr WL and 2 yr RF event S3(a)	32	5	0
1 in 5 yr WL and 5 yr RF event S3(b)	37	7	0

It is clear from the above tables that, moderate type of damages will occur mostly in homesteads and industries with comparatively less damages within the commercial enterprises. A comparison of damages between two cases in homesteads, commercials and industries are shown in Table 6.21 to Table 6.23.

Table 6.21: Comparison of damages in Homesteads

Damage	Case Study 1		Case Study 2	
	S1(a)	S1(b)	S1(a)	S1(b)
	Area (Ha)	Area (Ha)	Area (Ha)	Area (Ha)
Low Damage	13.52	29.28	1.04	17.04
Moderate Damage	8.36	31.36	0.04	14.00
Severe Damage	0.48	5.12	0.00	1.48
Total Damages	**22.36**	**65.76**	**1.08**	**32.52**
	S2(a)	**S2(b)**	**S2(a)**	**S2(b)**
Low Damage	16.52	25.60	15.96	17.60
Moderate Damage	22.92	37.56	3.44	15.16
Severe Damage	2.04	9.20	0.16	2.08

Total Damages	**41.48**	**72.36**	**19.56**	**34.84**
	S3(a)	**S3(b)**	**S3(a)**	**S3(b)**
Low Damage	46.48	45.32	48.28	51.60
Moderate Damage	38.52	43.28	31.20	33.64
Severe Damage	8.60	18.08	2.76	5.12
Total Damages	**93.60**	**106.68**	**82.24**	**90.36**

The above table states that, damage area decreases to 1% and 32% for rainfall effect only, after the improvement in borrow pits. Similarly, damage decreases to 20%, 34%, 82% and 90% for back water effects It means that about 21%, 33%, 22%, 38%, 11% and 16% improvements of damaged areas in homesteads can be attained.

Table 6.22: Comparison of damages in Commercials

Damage	**Case Study 1**		**Case Study 2**	
	S1(a)	**S1(b)**	**S1(a)**	**S1(b)**
	Area (Ha)	**Area (Ha)**	**Area (Ha)**	**Area (Ha)**
Low Damage	0.00	1.04	0.00	0.12
Moderate Damage	0.00	0.44	0.00	0.44
Severe Damage	0.00	0.00	0.00	0.00
Total Damages	**0.00**	**1.48**	**0.00**	**0.56**
	S2(a)	**S2(b)**	**S2(a)**	**S2(b)**
Low Damage	0.08	1.08	0.20	0.12
Moderate Damage	0.28	0.60	0.00	0.40
Severe Damage	0.00	0.00	0.00	0.00
Total Damages	**0.36**	**1.68**	**0.20**	**0.52**
	S3(a)	**S3(b)**	**S3(a)**	**S3(b)**
Low Damage	1.24	2.04	0.92	1.24
Moderate Damage	0.48	0.84	0.44	0.44
Severe Damage	0.00	0.00	0.00	0.00
Total Damages	**1.72**	**2.88**	**1.36**	**1.68**

Table 6.23: Comparison of damages in Industry

Damage	**Case Study 1**		**Case Study 2**	
	S1(a)	**S1(b)**	**S1(a)**	**S1(b)**
	Area (Ha)	**Area (Ha)**	**Area (Ha)**	**Area (Ha)**
Low Damage	0.08	0.80	0.00	0.20
Moderate Damage	0.16	0.92	0.00	0.52
Severe Damage	0.00	0.00	0.00	0.00
Total Damages	**0.24**	**1.72**	**0.00**	**0.72**
	S2(a)	**S2(b)**	**S2(a)**	**S2(b)**
Low Damage	0.56	0.80	0.36	0.20
Moderate Damage	0.52	0.92	0.00	0.60
Severe Damage	0.00	0.12	0.00	0.00
Total Damages	**1.08**	**1.84**	**0.36**	**0.80**
	S3(a)	**S3(b)**	**S3(a)**	**S3(b)**
Low Damage	3.96	4.24	3.56	4.20
Moderate Damage	0.84	1.40	0.60	0.84
Severe Damage	0.08	0.16	0.00	0.00
Total Damages	**4.88**	**5.80**	**4.16**	**5.04**

Table 6.22 to Table 6.23 clearly states that, very little area will be damaged for different scenarios in commercials and industrial sectors.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The following conclusions has been sorted from the study

- Bangshi River, Kekla River and Baligao khal are identified as the major and natural drainage routes for the study area.
- Around 238 ha area of land has been identified and considered as the urban area with good road access for estimation of storm runoffs. However, homestead covers an area of about 191 Ha whereas commercial enterprises and industrial sectors cover only 11 and 12 Ha of land respectively.
- The area of Dhamrai is comparatively high ranging between 3.68 mPWD and 10.73 mPWD. General sloping pattern of this area is east to west. But the proposed drainage system has been planned and designed for maintaining most of the slope towards west to east and towards north, for draining to its outfall channel Bangshi.
- About 58% of land was found to be above the average flood level while some land is subjected to shallow depth (less than 30 cm) of flooding. The rest of the land ranges from moderate to very deep flooding. It is assessed that, about 14%, 14% and 7% of land is subjected to moderate (30-90 cm flood depth), deep (90-180 cm flood depth) and very deep (more than 180 cm flood depth) flooding.
- S1 through S7 are identified as the 7 major drains which are proposed with storm drains for the study area. S1 through S4 and S7 drainage systems have priority needs while S5 and S6 drainage systems are proposed in view of future extensions for the study area.
- The results obtained from Case Study 2, showed better improvements within the drainage system in terms of extent of flooding, compared to that of Case Study1.
- The results in scenario S1 under Case Study 2 showed, about 9% and 15% improvements in the flooded areas compared to Case Study 1. Under Case Study 2, the proposed drainage system will function smoothly for 2 year rainfall event and only 18% of land will go under flood levels for 5 year rainfall event.
- Under Case Study 2, there will be no water logging problem within Ward No. 8 for Scenario S1 and S2.
- In scenario S2 and S3, when considering the back water effects there is an increase in the flooded area specially which is limited to the low-lying parts of the drainage network. The effect will be severe within the low lying parts of Ward No 1, 3, 5 and 8. The results indicate that, 11% and 19% of land will experience flooding due to 1 in average year flooding event whereas, 43% and 47% of land experience flooding due to 1 in 5 year flooding events under Case Study 2.

- About 10% and 17% improvements in the flooded areas was possible to obtain for 1 in average year flooding events under Case Study 2 compared to that of Case Study 1. Whereas, for 1 in 5 year flooding events the improvements was only 5% and 8%.
- Part of outfall reaches of drains may have less drainage efficiency for a short period when the water level at the outfall is high in monsoon. However, the proposed drainage network will be effectively functional for the pre and post monsoon storm.
- Due to flooding, most of the damages will occur in homestead areas but majority of the damage lies within first category of damages. Only 3% of area undergoes severe damages within the homestead areas under Cases Study 2 whereas, commercial and industrial sectors experience no severe damages.
- Most of the inundations and damages are limited to Ward No 1, 3, 5 and 8.
- None of the scenarios indicate any additional areas in risk of flooding and damages in the future.

7.2 Recommendations

The following recommendations has been sorted from the study

- It is recommended to place several rainfall gauges in the study area to be able to compare the different rainfall data as this reduces the effect of local rainfall.
- It is observed from the model results, that due to the existence of pocket depression areas in Ward no. 1, 3, 5 and 8; drainage congestions are not improved even with rainfall flooding, except for scenario S1(a) of Case Study 2. Construction of some tertiary drains connecting the proposed secondary drains will improve the flooding conditions within these wards.
- Raising of low lands with earth fill can be another option to improve the flooding conditions within the wards affected to flooding. It is recommended that such land is raised to the similar level of high land (not less than 7.14 mPWD).
- It is observed that, there will be severe effect from the spills of Bangshi River in the north and west-middle areas of the study area. Provision of some control structures within the crossing points of the drainage system and road networks and having its operational system to remain closed during the monsoon when the outfall river stage is high, can prevent the affected wards from flooding.
- Provision of retention basins to store storm runoff from the contributing catchments and pumping out the runoffs to the river can also be another way to prevent the affected wards from flooding but this is very costly for a small townships.

- Re-sectioning of existing borrow-pits is proposed to the extent of its original geometry with the provision of adequate hydraulic connectivity. The study shows better result and improvements with the re-excavated borrow pits.

- The borrow pits must be in continuous flowing condition and thus reaching the outfall channel Bangshi. At least 2 nos. additional drainage structures are needed for its continuous flow to Bangshi in relating with today's situation.

- Zones 21& 22 of the study area will drain over the land across the study areas boundary to the Baligao khal and finally route and drain to Bangshi River. The concerned authority must have institutional linkages with all relevant line agencies for the continuation of drainage provision of Zones 21 & 22 in view of long term consideration.

- Detailed socio-economic studies considering households, public health, and urban planning issues need to be undertaken to understand current coping and adaptation situations.

- The objective of developing the Urban Drainage Model is to create a tool for Dhamrai officials, which would enable the simulation of the performance of the complex drainage system. Maintaining such a model has become a good practice in decision making to support alleviating drainage problems for future urban developments. For this reason, it is recommended to continue working on the gradual improvement of the Dhamrai Urban Drainage Model along with other urban developments. The model can also be used as a tool for planning.

7.3 Recommendations for Future Studies

If further studies about urban flooding are to be performed in this study area, some suggestions to improve the performance are as follows:

- As the urban areas around the study area is likely to be enlarged in the future, it is recommended to investigate and plan the extent of urbanizations.

- Planning of the flood plain zones depending on various level of vulnerability i.e. pre and post monsoon flooding, drought and salinity, agricultural sectors etc.

- Identifying weakness of the drainage system under climate change scenarios and finding adaption options.

- Use of more than one water level recorder to perform a better calibration and validation. For this reason, one water level recorder should be placed to measure the water level in Bangshi river which can be put in the model as a varying external water level. Simultaneously, the water level in the drainage network should be measured to be used in the calibration and validation.

- A study on combined sewer system i.e. wastewater disposed from sewerage pit and septic tanks, industrial and domestic waste water can provide a framework and solution against pollution to be occurred in the outfall channel Bangshi. For this, MOUSE model can be used to provide a useful planning tool for the Municipal areas.

- A further study on the water quality analysis and modelling, in the Outfall channel can be carried out.

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Annex A (Hydro-Meteorological & Hydrometric Data)

Annex A-1: Rainfall Intensity of Dhaka (mm/hour)

Time (mins)	Dhaka Tr = 1.1 year		Dhaka Tr = 2 year		Dhaka Tr = 5 year		Dhaka Tr = 10 year	
4	86		109.8		128.3		139.9	
6	83.9		106.4		124.5		136.1	
8	81.7		103.2		121.1		132.6	
10	79.4		100.1		117.8		129.4	
12	77.2		97.2		114.8		126.3	
14	75.0		94.5		112.0		123.4	
16	72.8		91.9		109.3		120.7	
18	70.7		89.5		106.7		118.1	
20	68.6		87.1		104.3		115.7	
22	66.6		84.9		102.0		113.3	
24	64.6		82.8		99.8		111.1	
26	62.7		80.8		97.7		108.9	
28	60.9		78.8		95.7		106.9	
30	59.2		77.0		93.8		104.9	
32	57.5		75.2		91.9		103.1	
34	55.9		73.5		90.2		101.3	
36	54.3		71.9		88.5		99.5	
38	52.8		70.4		86.9		97.8	
40	51.4		68.9		85.3		96.2	
42	50.0		67.5		83.8		94.7	
44	48.7		66.1		82.4		92.2	
46	47.5		64.8		81.0		91.7	
48	46.2		63.5		79.6		90.3	
50	45.1		62.3		78.3		89.0	
52	43.9		61.2		77.1		87.7	
54	42.9		60.0		75.9		86.4	
56	41.8		58.9		74.7		85.2	
58	40.8		57.9		73.6		84.0	
60	39.9		56.9		72.5		82.8	
62	38.9		55.9		71.4		81.7	
64	38.0		54.9		70.4		80.6	
66	37.2		54.0		69.4		79.6	
68	36.3		53.1		68.4		78.5	
70	35.5		52.3		67.5		77.5	
72	34.8		51.4		66.5		76.6	
74	34.0		50.6		65.7		75.6	
76	33.3		49.8		64.8		74.7	
78	32.6		49.1		64.0		73.8	
80	31.9		48.3		63.1		72.9	
82	31.3		47.6		62.3		72.1	
84	30.7		46.9		61.6		71.2	
86	30.1		46.3		60.8		70.4	
88	29.5		45.6		60.1		69.6	
90	28.9		45.0		59.4		68.9	
92	28.4		44.3		58.6		68.1	
94	27.8		43.7		58.0		67.4	
96	27.3		43.1		57.3		66.7	
98	26.8		42.6		56.6		66.0	
100	26.3		42.0		56.0		65.3	

Time (mins)	Dhaka		Dhaka		Dhaka		Dhaka	
	Tr = 1.1 year		Tr = 2 year		Tr = 5 year		Tr = 10 year	
102	25.8		41.4		55.4		64.6	
104	25.4		40.9		54.8		63.9	
106	25.0		40.4		54.2		63.3	
108	24.5		39.9		53.6		62.7	
110	24.1		39.4		53.0		62.0	
112	23.7		38.9		52.5		61.4	
114	23.3		38.5		51.9		60.8	
116	22.9		38.0		51.4		60.3	
118	22.5		37.6		50.9		59.7	
120	22.2		37.1		50.4		59.1	

***Note: the rainfall intensities given above have been adjusted to long term average values to be consistent with the Dhaka daily rainfall short term to long term rainfall ratios*

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Annex A-2: Rainfall Conversion Factor Calculation

i) Dhaka Rainfall Analysis

Year	Max daily (mm)	Year	Max daily (mm)
1953	90	1982	146
1954	n/a	1983	133
1955	115	1984	151
1956	326	1985	92
1957	73	1986	176
1958	137	1987	138
1959	n/a	1988	135
1960	141	1989	118
1961	185	1990	94
1962	116	1991	123
1963	189	1992	90
1964	114	1993	140
1965	177	1994	74
1966	257	1995	83
1967	125	1996	150
1968	145	1997	121
1969	86	1998	118
1970	152	1999	141
1971	251	2000	158
1972	231	2001	61
1973	168	2002	120
1974	106.7	2003	93
1975	143	2004	341
1976	163	2005	128

Year	Max daily (mm)	Year	Max daily (mm)
1977	100	2006	185
1978	128	2007	152
1979	108	2008	126
1980	91	2009	333
1981	81		

Nos of yrs:	55yrs	
Long term average (1953-09):	55yrs	144
Std Dev (1953-09):	55yrs	61
Short term average (1988-07):	20yrs	131
Std Dev (1988-08):	20yrs	58

Ratio 1= Dhaka Long term avg./Dhaka Short term avg. (1988-07)=	1.10

ii) Dhamrai (Savar) Rainfall Analysis

Year	Max daily (mm)	Year	Max daily (mm)
1988	194.3	1998	183
1989	101.59	1999	145
1990	210	2000	171
1991	225	2001	76
1992	105	2002	82
1993	98	2003	44
1994	127.6	2004	166
1995	95	2005	87
1996	152	2006	105
1997	93	2007	89.5

Short term average (1988-07); 20 yrs	127.5
Std Dev (1988-07); 20 yrs	50.1
Ratio 2= Dhamrai short term avg./Dhaka short term avg. (1988-07)= 0.97	
CF for Dhamrai Rainfall Intensities= Ratio1xRatio2= 1.07 using RF of 1988-07	

Annex A-3: 2 Day Maximum Yearly Rainfall of Savar Station for Dhamrai

Year	Annual 2-d maximum rainfall (millimetre)*	Year	Annual 2-d maximum rainfall (millimetre)*
1962	297.20	1987	167.68
1963	109.50	1988	264.09
1964	195.80	1989	154.89
1965	184.20	1990	250.00
1966	166.90	1991	289.99
1967	160.70	1992	165.00
1968	196.60	1993	153.00
1969	116.80	1994	128.00
1970	118.10	1995	153.00
1971	139.40	1996	214.00
1972	144.80	1997	160.00
1973	133.40	1998	208.00
1974	135.90	1999	235.00
1975	371.10	2000	196.00
1976	250.20	2001	125.00
1977	150.10	2002	149.00
1978	199.40	2003	66.00
1979	268.00	2004	314.00
1981	160.00	2005	139.00
1982	208.30	2006	203.00
1983	249.00	2007	144.50
1984	261.40	2008	122.00
1985	190.70		
1986	163.89		

*** Data Source: Daily rainfall data of BWDB*

Annex A-4: Maximum Water Levels at Kaliakoir Station and Bangshi

Year	Annual maximum water level (mPWD)		Year	Annual maximum water level (mPWD)	
	Kaliakoir	Dhamrai		Kaliakoir	Dhamrai
1978	7.20	6.41	1994	6.20	5.24
1980	9.29	8.33	1995	7.74	6.78
1981	8.15	7.19	1996	7.26	6.3
1982	7.37	6.41	1997	7.42	6.46
1984	9.39	8.43	1998	9.26	8.3
1985	8.90	7.94	1999	7.49	6.53
1986	8.21	7.25	2000	8.50	7.54
1987	9.33	8.37	2001	7.93	6.97
1988	10.47	9.51	2002	8.50	7.54
1989	7.00	6.04	2003	7.82	6.86
1990	7.94	6.98	2004	9.64	8.68
1991	8.01	7.05	2006	6.21	5.25
1992	6.30	5.34	2007	9.30	8.34
1993	8.07	7.11	2008	7.80	6.84

Annex B (Analytical Analysis of Drains)

Annex B-1: Estimation of Drain Flows

$Q_p = C_s C_r IA/360$

Drain Ref.	Drain Type	Drain Length (m)	Total Cont. Area (ha)	Chainage (m)	Sec. Length (m)	Sec. Cont. Area (ha)	Cum Area (ha)	Runoff Coeff. C_r	Storage Coeff. C_s	Effective Area (ha)	Max Overland Dist. (m)	Avg. Ground Slope 1:	Time of Entry (mins)	Travel Time (mins)	Time of Conc. (mins)	Rainfall Intensity,I (mm/hr)	Peak Flow,Q_p (m^3/s)
S1-1	Rect.	227	18	0-227	227	18	18	0.4	0.7	5.04	350	1200	27.00	3.78	30.78	81.27	1.14
S1-2	Rect.	332	8	0-332	332	8	8	0.4	0.7	2.24	200	1200	18.00	5.53	23.53	88.65	0.55
S1	Rect.	989	38	0-169	169	2	2	0.4	0.7	0.56	120	1200	12.00	2.82	14.82	99.43	0.15
				169-384	215	19	21	0.4	0.7	5.88	120	1200	12.00	6.40	18.40	94.71	1.55
				384-989	605	17	38	0.4	0.7	10.64	120	1200	12.00	16.48	28.48	83.48	2.47
S2-1	Rect.	743	28	0-743	743	28	28	0.4	0.7	7.84	300	1200	24.00	12.38	36.38	76.34	1.66
S2-2	Rect.	352	8	0-352	352	8	8	0.4	0.7	2.24	180	1200	16.00	5.87	21.87	90.53	0.56
S2	Rect.	899	44	0-610	610	34	34	0.4	0.7	9.52	150	1200	14.00	10.17	24.17	87.95	2.33
				610-899	289	10	44	0.4	0.7	12.32	150	1200	14.00	14.98	28.98	82.99	2.84
S3-1	Rect.	300	17	0-300	300	17	17	0.4	0.7	4.76	450	1200	33.00	5.00	38.00	75.02	0.99
S3	Rect.	597	43	0-453	453	25	25	0.4	0.7	7.00	650	1200	44.00	7.55	51.55	65.51	1.27
				453-598	145	18	43	0.4	0.7	12.04	650	1200	44.00	9.97	53.97	64.05	2.14
S4-1	Rect.	355	22	0-355	355	22	22	0.4	0.7	6.16	400	1200	30.00	5.92	35.92	76.72	1.31
S4	Rect.	1217	48	0-157	157	7	7	0.4	0.7	1.96	250	1200	21.00	2.62	23.62	88.55	0.48
				157-1217	1060	41	48	0.4	0.7	13.44	250	1200	21.00	20.28	41.28	72.47	2.71
S5	Rect.	661	31	0-661	661	31	31	0.4	0.7	8.68	350	1200	27.00	11.02	38.02	75.00	1.81
S6	Rect.	602	23	0-602	602	23	23	0.4	0.7	6.44	350	1200	27.00	10.03	37.03	75.80	1.36
S7	Rect.	494	11	0-494	494	11	11	0.4	0.7	3.08	200	1200	18.00	8.23	26.23	85.76	0.73

<u>Runoff Estimates for unplanned (without proposed drains at present) zones</u>

Drain Ref.	Sec. Length (m)	Sec. Cont. Area (ha)	Cum Area (ha)	Runoff Coeff. C_r	Storage Coeff. C_s	Effective Area (ha)	Max Overland Dist. (m)	Avg. Ground Slope 1:	Time of Entry (mins)	Travel Time (mins)	Time of Conc. (mins)	Rainfall Intensity,I (mm/hr)	Peak Flow,Q_P (m^3/s)
Zone 8	850	120	120	0.4	0.7	33.6	900	1200	57.00	14.17	71.17	55.29	5.16
Zone 9	700	10	10	0.4	0.7	2.8	550	1200	39.00	11.67	50.67	66.05	0.51
Zone 10	900	47	47	0.4	0.7	13.16	1800	1200	97.00	15.00	112.00	41.67	1.52
Zone 11	200	37	37	0.7	0.8	20.72	1600	1200	88.00	3.33	91.33	47.62	2.74
Zone 12	250	7	7	0.4	0.7	1.96	250	1200	21.00	4.17	25.17	86.87	0.47
Zone 13	400	16	16	0.4	0.7	4.48	250	1200	21.00	6.67	27.67	84.28	1.05
Zone 14	250	8	8	0.4	0.7	2.24	350	1200	27.00	4.17	31.17	80.90	0.50
Zone 15	550	23	23	0.4	0.7	6.44	250	1200	21.00	9.17	30.17	81.84	1.46
Zone 16	510	6	6	0.4	0.7	1.68	200	1200	18.00	8.50	26.50	85.48	0.40
Zone 17	365	6	6	0.4	0.7	1.68	150	1200	14.00	6.08	20.08	92.64	0.43
Zone 18	1050	16	16	0.4	0.7	4.48	165	1200	15.00	17.50	32.50	79.69	0.99
Zone 19	475	14	14	0.4	0.7	3.92	390	1200	30.00	7.92	37.92	75.08	0.82
Zone 20	350	24	24	0.4	0.7	6.72	400	1200	30.00	5.83	35.83	76.79	1.43
Zone 21	1058	97	97	0.4	0.7	27.16	300	1200	24.00	17.63	41.63	72.21	5.45
Zone 22	550	41	41	0.4	0.7	11.48	600	1200	41.00	9.17	50.17	66.37	2.12

Annex B-2: Design of Drain Section

(Using Manning's Formula, $Q_d = A\ [1/n][R^{2/3}][S^{1/2}]$

Drain ID	Runoff from Model	Slope 1:	Type P/ S	Top Width (m)	Bottom Width (m)	Design Depth (m)	Manning's Roughness,n	Flow Area, A (m^2)	Wetted Perimeter,P (m)	Hydraulic Radious, R=A/P	Design Capacity,Q_d (m^3/s)	Design velocity,V (m/s)	Actual drain depth* (m)	Remarks
S1-1	1.18	500	S	1.12	1.12	0.75	0.014	0.84	2.62	0.32	1.25	1.49	0.90	RCC Drain
	1.18	500	S	1.12	1.12	0.75	0.014	0.84	2.62	0.32	1.25	1.49	0.90	RCC Drain
S1-2	0.59	1000	S	0.75	0.75	0.90	0.014	0.68	2.55	0.26	0.63	0.93	1.05	RCC Drain
	0.59	1000	S	0.75	0.75	0.90	0.014	0.68	2.55	0.26	0.63	0.93	1.05	RCC Drain
S1	0.17	1000	S	0.65	0.65	0.40	0.014	0.26	1.45	0.18	0.19	0.71	0.55	RCC Drain
	0.17	1000	S	0.65	0.65	0.40	0.014	0.26	1.45	0.18	0.19	0.71	0.55	RCC Drain
	1.57	1000	S	1.16	1.16	1.14	0.014	1.32	3.44	0.38	1.57	1.19	1.29	RCC Drain
	2.56	1000	S	1.42	1.42	1.34	0.014	1.90	4.10	0.46	2.57	1.35	1.49	RCC Drain
S2-1	1.67	1000	S	1.18	1.18	1.20	0.014	1.42	3.58	0.40	1.72	1.21	1.35	RCC Drain
	1.67	1000	S	1.18	1.18	1.20	0.014	1.42	3.58	0.40	1.72	1.21	1.35	RCC Drain
S2-2	0.60	1000	S	0.80	0.80	0.85	0.014	0.68	2.50	0.27	0.64	0.94	1.00	RCC Drain
	0.60	1000	S	0.80	0.80	0.85	0.014	0.68	2.50	0.27	0.64	0.94	1.00	RCC Drain
S2	2.35	1000	S	1.36	1.36	1.32	0.014	1.80	4.00	0.45	2.37	1.32	1.47	RCC Drain
	2.35	1000	S	1.36	1.36	1.32	0.014	1.80	4.00	0.45	2.37	1.32	1.47	RCC Drain
	2.89	1000	S	1.45	1.45	1.44	0.014	2.09	4.33	0.48	2.89	1.39	1.59	RCC Drain
S3-1	1.03	1000	S	1.10	1.10	0.90	0.014	0.99	2.90	0.34	1.09	1.10	1.05	RCC Drain
	1.03	1000	S	1.10	1.10	0.90	0.014	0.99	2.90	0.34	1.09	1.10	1.05	RCC Drain
S3	1.29	1000	S	1.00	1.00	1.18	0.014	1.18	3.36	0.35	1.32	1.12	1.33	RCC Drain
	1.29	1000	S	1.00	1.00	1.18	0.014	1.18	3.36	0.35	1.32	1.12	1.33	RCC Drain
	2.24	1000	S	1.30	1.30	1.34	0.014	1.74	3.98	0.44	2.26	1.30	1.49	RCC Drain
S4-1	1.34	900	S	1.30	1.30	0.85	0.014	1.11	3.00	0.37	1.35	1.22	1.00	RCC Drain
	1.34	900	S	1.30	1.30	0.85	0.014	1.11	3.00	0.37	1.35	1.22	1.00	RCC Drain
S4	0.50	1000	S	0.85	0.85	0.65	0.014	0.55	2.15	0.26	0.50	0.91	0.80	RCC Drain
	0.50	1000	S	0.85	0.85	0.65	0.014	0.55	2.15	0.26	0.50	0.91	0.80	RCC Drain
	2.74	1000	S	1.44	1.44	1.40	0.014	2.02	4.24	0.48	2.77	1.37	1.55	RCC Drain
S5	1.83	1000	S	1.60	1.60	0.92	0.014	1.47	3.44	0.43	1.88	1.28	1.07	RCC Drain
	1.83	1000	S	1.60	1.60	0.92	0.014	1.47	3.44	0.43	1.88	1.28	1.07	RCC Drain
S6	1.40	1000	S	1.30	1.30	0.92	0.014	1.20	3.14	0.38	1.41	1.18	1.07	RCC Drain
	1.40	1000	S	1.30	1.30	0.92	0.014	1.20	3.14	0.38	1.41	1.18	1.07	RCC Drain
S7	0.78	500	S	1.10	1.10	0.55	0.014	0.61	2.20	0.28	0.81	1.35	0.70	RCC Drain
	0.78	500	S	1.10	1.10	0.55	0.014	0.61	2.20	0.28	0.81	1.35	0.70	RCC Drain

Design for the re-sectioning of the Borrow Pits

Drain ID	Runoff from Model	Slope 1:	Type P/ S	Top Width (m)	Bottom Width (m)	Design Depth (m)	Manning's Roughness,n	Flow Area, A (m^2)	Wetted Perimeter,P (m)	Hydraulic Radious, R=A/P	Design Capacity,Q_d (m^3/s)	Design velocity,V (m/s)	Actual drain depth* (m)	Remarks
B2	0.45	2000	P	2.30	0.95	0.45	0.014	0.73	2.57	0.28	0.50	0.69	0.65	Re-sectioning
	0.45	2000	P	2.30	0.95	0.45	0.014	0.73	2.57	0.28	0.50	0.69	0.65	Re-sectioning
B3	0.46	2000	P	2.30	0.95	0.45	0.014	0.73	2.57	0.28	0.50	0.69	0.65	Re-sectioning
	0.46	2000	P	2.30	0.95	0.45	0.014	0.73	2.57	0.28	0.50	0.69	0.65	Re-sectioning
B4	1.02	2000	P	2.90	0.95	0.65	0.014	1.25	3.29	0.38	1.04	0.84	0.85	Re-sectioning
	1.02	2000	P	2.90	0.95	0.65	0.014	1.25	3.29	0.38	1.04	0.84	0.85	Re-sectioning
B5	0.89	2000	P	2.69	0.65	0.68	0.014	1.14	3.10	0.37	0.93	0.81	0.88	Re-sectioning
	0.89	2000	P	2.69	0.65	0.68	0.014	1.14	3.10	0.37	0.93	0.81	0.88	Re-sectioning
B1	4.19	1800	P	4.68	1.20	1.16	0.014	3.41	5.38	0.63	4.23	1.24	1.36	Re-sectioning
	4.19	1800	P	4.68	1.20	1.16	0.014	3.41	5.38	0.63	4.23	1.24	1.36	Re-sectioning
	7.91	1800	P	5.93	1.55	1.46	0.014	5.46	6.81	0.80	7.93	1.45	1.66	Re-sectioning
	9.86	1800	P	6.46	1.72	1.58	0.014	6.46	7.42	0.87	9.92	1.54	1.78	Re-sectioning
	12.96	1800	P	7.30	2.35	1.65	0.014	7.96	8.30	0.96	13.04	1.64	1.85	Re-sectioning
	14.11	1800	P	7.51	2.35	1.72	0.014	8.48	8.55	0.99	14.20	1.67	1.92	Re-sectioning
	14.11	1800	P	7.51	2.35	1.72	0.014	8.48	8.55	0.99	14.20	1.67	1.92	Re-sectioning