

**MATHEMATICAL MODELLING FOR ANALYSIS OF DRAINAGE
CONGESTION PROBLEM OF POLDER 29 IN KHULNA DISTRICT**

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A Thesis Submitted

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Arpita Islam

Dedicated to

My Beloved Parents, the source of all inspirations in my life

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ABSTRACT

The coastal belt of Bay of Bengal lies in the southern part of Bangladesh faces severity of natural catastrophes such as drainage congestion, inundation of land etc at monsoon. To protect this area, polders are constructed during sixties. But waterlogging problem is getting intensified with time in southwest coastal region of Bangladesh due to gradual riverbed siltation, inadequate drainage channels and improper operation and maintainance of regulators and flapgates. There is a growing concern that these effects will exacerbate in future due to effects of climate change. In this study, Polder 29 has been selected as the study area because condition of most of the internal drainage channels is almost disagreeable and some gates at eastern and western periphery remain non-operational which results prolonged waterlogging. So, this study is formulated to analyze the existing drainage congestion problem of the study area and an adaptive measure is suggested to alleviate this problem.

GeoSWMM was used in this study to carry out stormwater analysis of the study area comprising 79.33 km² area. The model was calibrated and validated using the observed water level data (both inside and outside) of Kanchannagar sluice. Frequency analysis of rainfall and water level data was carried out for different return periods to estimate the design rainfall and water level. Adequacy of existing drainage channels was investigated by simulating the model at design storm scenario (1 in 10 year period). The result of the analysis reveals that some drainage channels are inadequate for conveying the design flow volume. To solve this problem, increasing cross-sectional area is suggested.

Flood inundation map was prepared to assess the waterlogging problem of the study area. The analysis reveals that 52.33% of the total polder area is waterlogged under the existing condition and application of the ideal pump facility can reduce this to 37.33%. Largest Kanchannagar catchment was modeled separately to understand the actual extent of reduction in inundation depth. The analysis shows that 38.70% of the total Kanchannagar catchment area is flooded at the existing scenario and re-excavation and addition of new drainage channels help reduce the waterlogged area to 11.26 %. The flooding loss was about 139860 m³ with this adaptive measure and after utilizing pump with capacities of 1, 2 and 3 cumec this loss is reduced to 136900 m³, 119400 m³ and 110280 m³ respectively

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LIST OF ABBREVIATIONS

ADB	Adaptation Development Bank
BMD	Bangladesh Meteorological Department
BMP	Best Management practice
BUET	Bangladesh University of Engineering and Technology
BWDB	Bangladesh Water development Board
CEGIS	Center for Environmental and Geographic Information Services
CEIP	Coastal Embankment Improvement Project
CEP	Coastal Embankment Project
CERP	Coastal Embankment Rehabilitation Project
DAP	Detail Area Plan
DEM	Digital Elevation Model
DHI	DHI Danish Hydraulic Institute
DND	Dhaka- Narayanganj-Demra
DOE	Department of Environment
DPHE	Department of Public Health Engineering
EIA	Environmental Impact Assessment
FIM	Flood Inundation Map
GIS	Geographic Information System
HYV	High Yielding variety
IDF	Intensity Duration Frequency
IFI	International Financial Institutions
IPCC	Intergovernmental Panel on Climate Change
IPSWAM	Integrated Planning for Sustainable Water Management
IWM	Institute of Water Modelling
KCC	Khulna City Corporation
KUD	Khulna Urban Drainage
MES	Meghna Estuary Study
MSL	Mean Sea Level
NAM	North American Mesoscale Model
NASA	National Aeronautics and Space Administration

NENT	Northeast New Territories
NGA	National Geospatial-Intelligence Agency
NIMA	National Imagery and Mapping Agency
NRCS	Natural Resources Conservation Service
NWNT	Northwest New Territories
PWD	Public Works Datum
RDII	Rainfall Dependent Infiltration/Inflow
SDM	Stormwater Drainage manual
SLR	Sea Level Rise
SRES	Special Report on Emissions Scenarios
SMRC	SAARC Meteorological Research Centre
SWMC	Surface Water Modelling Centre
TRM	Tidal River Management
U/S	Upstream
USGS	United States Geological Survey
ZPP	Zauro Polder Project

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

Stormwater is mainly originated from rainfall or melting of snow from the mountains and then generated on land surface. Soil absorbs much of the stormwater and plants hold some portion of it where it falls. Later the remaining part of the water conveys to natural channels. Unmanaged stormwater can create two major issues: one related to the volume and timing of runoff water (flooding) and the other related to potential contaminants that the water is carrying (water pollution). Excessive runoff from the upstream catchments triggers flooding in river by limiting channel conveyance capacity. The excess water spills over the banks and the floodplain becomes flooded by this spilling water. In rural areas, inundation mainly occurs while the extreme intensity rainfall produces greater amount of runoff and the channels do not have adequate capacity to drain away this water. As a result, water remains stagnant for prolonged period and induced flooding known as storm water flooding or waterlogging.

Geographically, Bangladesh is located at the tip of a funnel, through which huge amount of rain water discharged in monsoon to the ocean flushing over the country (Ali et al., 2013). This country is ranked as being at "high-risk" of multiple devastating hazards by Maplecroft's annual Natural Hazards Risk Atlas and the effect of climate change magnifies the governing factors of these natural hazards. The coastal belt of Bay of Bengal is in the south of Bangladesh, faces severity of natural catastrophes like drainage congestion, inundation of land at monsoon and unsteady morphological processes, fresh water scarcity in dry season. Apart from natural causes, human activities i.e changes in land use, blockage of natural drainage systems, silt arising from construction activities, indiscriminate land filling and lack of comprehensive maintenance of natural watercourses due to land access problems can also influence the prevalence of inundation. The temporary drainage congestion, which first appeared in 1982, gradually became permanent water logging to such an extent that, by 1990, an area of 100,600 hectares in Khulna and Jessore districts alone was permanently waterlogged (IFI, 2016). This congestion affects the biophysical environment and consequently also affects the production of fish and paddy as well as the other socio-

economic factors (Alam, 2007). Some vast areas of crop land remained under water for more than 10 years. Many villages were submerged, houses collapsed, transportation system in those areas ceased. As a result, people have been living on embankments and roads, livestock numbers have greatly decreased, procurement of fire wood and drinking water have become very expensive, education of children has been discontinued and waterborne diseases like diarrhoea and scabies become wide spread, people have great difficulties on finding work and hence the income of the poor people has sharply decreased (Alam, 2005).

Though coastal areas are protected by polders from sixties but this inundation problem is getting stronger in south-western part of Bangladesh with time. Height of all these polders varies between 3m to 7m (IWM, 2005a). About 11,915 km² in the coastal area is protected by coastal polders (MES II, 2001). IWM and CEGIS (2007) have studied that about 25 polders in the southwest region would experience severe drainage congestion due to 62 cm SLR. Among the affected areas of Khulna, Satkhira is the worst hit and experiencing severe and year-round water logging (Awal, 2014). Water congestion problem would be prolonged in the Khulna region due to Monsoon strengthening towards the end of the season along with increasing future projection in autumn rainfall (Mondal et al., 2013).

So, waterlogging is a pressing concern at the backdrop of climate change which causes enormous sufferings to the livelihood and adversely affects the environment. Therefore Polder 29, lies in south-west part of Bangladesh is selected as the study area. Due to unabated encroachment of canals, gradually filling up of low-lying flood plains, rivers, canals, water bodies and a lack of proper drainage and sewerage system, waterlogging has become a perennial problem for the dwellers of this region, especially in the rainy season (Roy, 2017). As a result, stormwater management has become a challenging task in the areas when the natural drainage system experiences relatively high water levels during rainy season or if the channel capacity decreased due to encroachment of the runoff detention areas. Smaller coastal communities often employ a simple system of surface conveyances, such as open ditches along roads and highways. Some larger communities have separate systems for stormwater and sewage. Regardless of the system, the low-lying nature of many coastal areas causes unique stormwater management challenges that are exacerbated by coastal flooding. Rain water mainly drained by gravity drainage system while wetlands and low lands work as a retention

storage. Flat topography in these areas makes this drainage a difficult approach. More frequent, higher, and longer-lasting high water events may drive up already high groundwater levels in some coastal communities. This change may reduce the soil's ability to absorb stormwater, thus increasing runoff. As the coastal polders are surrounded by tidal rivers so closed flapgates during high tide prevents the excess runoff to pass through the drainage sluices which ultimately results prolonged waterlogging.

The importance of stormwater modeling is increasing now a days due to urbanization, population growth and climate change. The first two trends induce a rapid growth of cities, making stormwater management even more challenging while at the same time a rising number of people is affected by the harmful effects of stormwater on the environment. These effects are expected to be exacerbated in future due to increased monsoon precipitation, higher transboundary water flows, and rising sea levels resulting from climate change which eventually increase the depth and extent of inundation. Considering the above mentioned factors, stormwater analysis is carried out using GeoSWMM for polder 29 which is currently facing excessive inundation problem and an improvement measure is suggested to alleviate this problem.

1.2 Objective of the Study

The objectives of the study have been described as follows:

1. To perform frequency analysis of rainfall for different return period and select the design rainfall
2. To set up and calibrate the hydrologic model of the study area using GeoSWMM
3. To check the adequacy of the existing drainage network for design rainfall conditions and generate inundation map for this condition
4. To suggest measures for the improvement of existing drainage system including the introduction of pumping facilities

Expected outcomes of the research are as follows:

1. Identification of water-logged areas from the depth-inundation mapping
2. Improvement of drainage congestion problem with suggested measures

1.3 Organization of the Report

Chapter One: This chapter provides background information to understand the context of the study. Research objectives with expected outcomes are also stated here.

Chapter Two: This chapter gives a review of relevant literature which is in connection with the present study. It gives a clear view of different types of works done on the study area and similar areas and it also gives information about related works using mathematical models.

Chapter Three: This chapter describes about theoretical background and description of GeoSWMM model, its various objects, operations and applications

Chapter Four: This chapter describes the theory and methodology followed in the present study. It includes a brief description of the study area, data source, data collection, data processing, data analysis along with model development,

Chapter Five: This chapter contains calibration and validation of the drainage model, detailed result analysis of calibrated model, and adequacy checking of drainage channels. Inundation map is presented here for design rainfall condition and effectiveness of adaptation measure is assessed here to lessen the drainage congestion of the selected polder area.

Chapter Six: Major findings of the study are shown as a conclusion in this last chapter. The suggestions and recommendations have also been stipulated here.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Bangladesh is one of the biggest and most densely populated deltas of the world, formed by sediments carried by the Ganges, Brahmaputra and Meghna River system (Bisquit, L. A., 2015). The effects of global warming and accompanying sea level rise are threatening many of the world's lowland areas so poldering is necessary to save this entity (Lineback and Gritzner, 2014). A polder is a large tract of low-lying wetlands which has no connection with outside water other than through manually operated devices. As a part of the Coastal Embankment Project (CEP), a series of embankments and polders were constructed in the 1960s around the Southern coastal region of Bangladesh (CEIP, 2015). Bangladesh has 139 polders, of which 49 are sea facing. These were constructed in order to protect the coast from tidal flooding, salinity incursion and water logging, resulted in high agricultural productivity on the coastal lands for 10-15 years (Bakuluzzaman and Naher, 2015). Regulators and other structures were built to control water intake and drainage of polder areas with the primary principle of improving agriculture. The gates of the regulators are closed during high tide to avoid inundation and opened during low tide in case of agricultural prerequisite.

Now-a-days the polders of the South-West coastal region of Bangladesh have been experiencing drainage congestion and subsequent waterlogging over the last few decades. The reasons include inadequate drainage channels within the polders to convey storm water runoff, increased sedimentation in rivers and drainage channels both within and outside polders, tidal influence at outfall and lack of proper operation and maintenance of regulators and flap gates (Awal, 2014). People are compelled to live in waterlogged condition in these areas; even many cultivated crop lands are permanently inundated losing valuable agricultural production especially rice. Climate change could further exacerbate this issue through changes in sedimentation and river-flow, increased monsoon rainfall and retarded discharge of rivers due to back water effect and sea level rise.

The study of polder 1, 2, 6 and 8 (IWM 2013c) has been carried out for solving the long standing drainage congestion problem in the Polder 1, 2, 6-8, 6-extension under Kalaroa, SatkhiraSadar, Debhata, Assasuni and Talaupzillas in the Satkhira district. The major interventions considered for removing drainage congestion are tidal river management by allowing natural tidal movement into embanked low-lying beel for sediment management, dredging/excavation of rivers, additional drainage and flushing regulators, , excavation of khals etc. Though TRM is an effective solution in drainage congestion, it requires huge land area requisition which is not always possible so left of the options can be considered for solving the problem taking into account the socio economic aspects.

Rahman (1995) in his project report stated that DPHE prepared the first master plan in 1968 to protect the developed area covering 75Km² area from floodwater and to drain internal storm water. BWDB (1975), following the previous master plan, prepared detailed scheme for an area of 145Km². In 1976 another scheme for internal drainage system was prepared by DPHE. Due to difficulty in financing both the proposals were not approved. Instead a crash program for removing standing water from Dhaka City, Internal Scheme was recommended and has been implemented by DPHE since 1976.

JICA (1887) has focused on a study of storm Water Drainage System Improvement Project in Dhaka City from November 1986 to October 1987 to solve the water logging problem. A proposal of improving 39.7 km of khals was given. Considering the floods of 1987 and 1988, the Govt. of Bangladesh finalized a committee in October 1988 for flood control and drainage of greater Dhaka. Afterwards the Government of Bangladesh established a committee for flood control plan for Greater Dhaka Metropolitan region which predicted mostly on the 1987 JICA study storm drainage system improvements for Dhaka city and the 1988 “Jansen Report” on causes of the flood and recommended solutions.

Khan (2006) taken up a study in the area lies between southern part of Noakhali and Lakshmipur districts of Bangladesh to assess CDSII (Char Development and Settlement Project) to find an integrated solution to the problem of waterlogging. MIKE-11 was calibrated and validated and several hydro-meteorological events i.e optimum size of regulators and drainage channels were assessed through a series of simulations considering 20 scenarios.

Shams and Hossain (2013) carried out a study by applying a numerical model to improve the drainage situation of DND area of Dhaka city. Rainfall Runoff and Hydrodynamic Models of MIKE11 have been setup as base model for analyzing the different drainage options. For this study three options are developed as per DAP report up to year of 2030 with population projection 225 per acre. The flood impact map has been generated from MIKE11 for two days consecutive rainfall. According to numerical model outputs the model shows that, installment of 4 pump stations at Shimrail (38.41 cumec discharge from drainage command area of 2994 ha), Adamjinagar (30.21 cumec, command area 2355 ha), Pagla (2.82 cumec, command area 220ha) and Fatulla (3.25 cumec command area 253 ha) provide the best suitable option with the design rainfall (245 mm) for 2 day 5 year return period.

Overloop (2006) studied about drainage control in water management of polders in Netharlands. He identified that water logging problem become severe over the past years as more area has been paved and storm events have become more extreme. In this paper a control method is presented that can effectively kept water in the upstream parts, until the downstream part can accommodate this amount of water. The method is based on upstream Proportional Integral-control with adaptation of the set point. The control is referred to as Cascade PI-Control. Basically, the goal of the control method is to fill the available storage equally in the whole area.

Idah et al., (2009) made a study for determining the causes of water logging in the pilot scheme of Zauro Polder Project (ZPP) at Nigeria. Researcher's identified numerous reasons for water logging. Topography of the area pilot scheme (ZPP) was the major reason of waterlogging as all those areas having elevations less than 100cm so there is no possibility of surface outflow. Second reason of water logging is lack of effective management to re-excavate/reconstruct the primary and secondary canals. Unplanned irrigation systems by local farmers and improper land grading etc. are also responsible for waterlogging phenomenon. Following recommendations were suggested to improve the conditions in the pilot scheme, which are re-excavation of the drains and the canals so that smooth inflow and outflow can be obtained. There should be adequate monitoring of the lateral canals in order to forestall illegal activities of the farmers.

CEGIS (2013) carried an EIA study at polder 39/2c of Pirojpur district under Blue Gold Program and found the peripheral boundary is not covered by embankments and people

of the area are suffering from tidal inundation twice a day. It becomes disastrous during high spring tides and monsoon due to drainage congestion in the low lying areas. No sluices have been constructed and all the internal Khals were kept open to the river. Two types of embankment has been proposed One is Interior-dyke having side slopes C/S 1:2 and R/S 1:3 with crest level 5.50 mPWD along the bank of Baleswar (Kocha), to the West and crest level 5.00mPWD along the bank of Pona River, and another is Marginal -dyke with crest level 4.50 mPWD and side slopes C/S 1:2 and R/S 1:2 along BaharKhal and Bhuter Khal to the east. A proposal of constructing 13 drainage sluices and re-excavation of drainage channels were also given to remove drainage congestion.

In CEIP-I study (CEGIS 2013) at Polder 35/1, it is found that currently about 30-40 percent of the polder area is facing problems of salinity and water logging primarily because of ineffective water control structures and silting of water channels. This situation is further compounded by unavailability of suitable ground water at shallow depth during April to May. Some suggestive measures are offered such as re-excavation of 17 drainage channels out of 90 water channels/ khals, replacement of Seventeen flushing inlets, re-sectioning the embankment to solve this drainage congestion problem.

2.3 Studies on Drainage Modeling Considering Climate Change

ADB (2010) entrusted IWM to carry out a study on the area under the Khulna City Corporation (KCC). During the study recurring and worsening waterlogging problems were found in this zone which will exacerbate by increased rainfall and sea level rise in near future. The Khulna Urban Drainage (KUD) model is especially developed using the MIKE11 software including the “NAM” component for rainfall run-off modeling for this project. The impact of climate change on KUD was analyzed considering the base case, which means the situation in 2010, 2030 and 2050 without climate change, and the reference case, which means the situation for the same years, but with climate change. The water logging condition for Khulna City was assessed for 5-yr and 10-yr return periods. Modeling of the current situation shows that 22% of Khulna City is under harmful waterlogging condition for a 10-year return period event. These area increases to 54% for the 2050 scenario if no improvements are done and the drainage system remains the same. The category of deep waterlogging, which causes the most damage, increases significantly in 2050. Some adaptive structural measures such as

widen/ deepen drainage channels in the problem areas, lay new drainage channels, and increase regulator size were suggested for water logging problem of Khulna in 2050 due to climate change condition. Modeling results show that harmful waterlogging area reduces from 29% to 14% after adaptation measures are taken.

Change, I. C. (2007) found that the projected sea-level rise will be occurred in the Bay of Bengal 15–62 cm at 2080. Sea level rise will aggravate the drainage congestion, water logging and flooding problems that are already severe in the urban and peri-urban areas of Khulna (ADB, 2010).

Mondal et al. (2013) made a comparative study on hydro-meteorological trends in south western part of Bangladesh. According to the study annual maximum tidal high water level is increasing at a rate of 7 - 18 mm. They have also analyzed the rainfall data for a period of 63 years (1948–2010) at Khulna region and found that the number of rainy days in a year and the maximum number of consecutive rainy days are found to be increasing in the southwest coastal region of Bangladesh. They also found that the rainfalls have increasing trends where most increment will be occurred during monsoon. As a result, water logging problems will become crucial in this region.

In CEIP-1 study (IWM 2013a) the AR4 projection of IPCC fourth assessment for sea level rise and rainfall has been considered in the mathematical model to determine the climate change condition in existing and improved drainage congestion for different options. To improve the drainage performance under anticipated climate change condition some options such as excavation of all the internal drainage khals of polders by 1 m and dredging of the peripheral rivers/khals by 2 m, modification of invert level of drainage structures, extra regulators and increase of vent numbers for some existing drainage structures etc. are devised and duly incorporated in the model setup for assessing their effectiveness. The results from the model revealed that the drainage congestion will improve significantly due to the implementation of these options. However, dredging of internal canals (1m) and peripheral River (2m) is proposed, also renovation of existing structure was also suggested.

Sarwar (2017) made a comparative study to assess the water logging problem for coastal polders namely 17/1 and 17/2 of Khulna district (Dumuria upazila). The research work has also analyzed the vulnerability of the existing situation of these polders with changing climate scenario of IPCC 2014 fifth assessment report and the

effectiveness of re-excavation of internal khals, dredging of Peripheral Rivers, Tidal River Management (TRM) and introducing pumping system against water logging problem applying as different scenarios. From the local model of Polder 17/1 it has been observed that maximum flood free area increased from base condition 17.26% to 69.96% in the final option and polder 17/2 from base condition 16% to 84% in final option under critical climate change condition. In this study, TRM is applied for reducing the water logging problem solution in the selected coastal polders though it is a long-term method of solution. Pumping is applied in this study for water level up to 1.0 m PWD and lowest at 0.50 m PWD for getting the highest benefit at extreme climate change condition.

An impact assessment of sea level rise on inundation, drainage congestion, salinity intrusion and change of surge level in the coastal zone of Bangladesh has done by IWM (2005). The study was conducted considering the potential effects of climate change for different sea level rise i.e. 14 cm, 32 cm, and 88 cm for the projected years 2030, 2050 and 2100. Mathematical models of the Bay of Bangle have been used to transfer the sea level rise in the deep sea along the southwest region rivers and the Meghna estuary. The study found that about 11% more area (4,107 sq.km) will be inundated due to 88 cm sea level rise in addition to the existing year 2000 inundation area under the same upstream flow. Due to 32 cm sea level rise, about 84% of the Sundarbans area becomes deeply inundated, and for 88 cm sea level rise Sundarbans will be completely lost. It also observed that 10% increase in wind speed of 1991 cyclone along with 32 cm sea level rise would produce 7.8-to 9.5 m high storm near Kutubdia- Cox's Bazar coast.

IWM (2007) has made a detailed analysis of the probable impacts of relative sea level rise on coastal livelihoods of Bangladesh. According to the 3rd IPCC predictions, the study considered the sea level rise for both low (B1) and high (A2) greenhouse gas emission scenarios. The study revealed that about 13% more area (551,000 ha) in the coastal region will be inundated in monsoon due to 62 cm sea level rise. The study also found that if 1991 cyclone will come with 27 cm SLR and increased intensity, Chittagong district will be suffered more and about 99,000 ha more area (18%) will be exposed to severe inundation (>100cm) compared to 1991 cyclone inundation. It was also observed that compared to 1991 cyclone inundation about 35,000ha area of Cox's Bazar district will be inundated severely (>100cm).

Chang et al. (2013) worked on improving drainage system in Tuku area of southern Taiwan. An integrated drainage–inundation model, combining a drainage flow model (SWMM) with 2-D overland-flow inundation model (2D diffusive-wave model) is used to assess the flood management approaches. Precipitation scenarios projected by the MPEH5 model for the 2020s period (2010–2039) were used to assess climate change impact on the inundation simulations in order to obtain inundation depth and its extent. The suggestive measures include increasing drainage capacity, using fishponds as retention ponds, constructing pumping stations, and building flood diversion culverts.

2.4 Studies on Drainage Modeling with Application of SWMM and GeoSWMM

Maliha (2017) carried out a study in Bogra city to design a drainage network system with limited data. As urbanization leads to increase of impervious surfaces thereby increasing runoff in the study area so provision for adequately planned drainage management system has become mandatory. The study was based on designing an adequate drainage system where there is scarcity of data. In this study, maximum flows and peak runoff in the design canals are estimated using the model GeoSWMM. The total runoff volume obtained from the model simulation for high intensity of rainfall is used to design the minimum cross section of the drainage canal. The cross section of the drainage conduit is designed based on 5 and 10 year return period.

Das (2017) carried out a study in Chittagong city to evaluate the present drainage situation, identify the future requirements and suggest improvement of the drainage network system to provide an area free from drainage congestion as rainfall induced flooding and water logging is a regular phenomenon in this area. GeoSWMM was used in this study to analyze the stormwater runoff. The total runoff volume obtained from the model simulation for high intensity of rainfall is used to design the minimum cross section of the drainage canal. She found the runoff-coefficients for each delineated subcatchments ranges from 0.6 to 0.7 for 5 and 10 year return period. The area of the drainage canals is determined by using continuity equation with the model simulated value of 21 streams peak flow (i.e. maximum stream flow from S21 is 140 cms) and non-silting non scouring velocity of the canals which is 0.8 m/s.

Islam (2017) applied GeoSWMM for the western part of Chattogram City for designing drainage conduits. The analysis showed that runoff –coefficients varies from 0.6 to 0.7.

Minimum drainage area was calculated for 5 and 10 year return period considering 0.8 m/s velocity. The maximum cross-sectional area of canal section was found 9.98m³.

Chowdhury (2016) selected Sirajganj district as the study area as every year monsoon spillage of the Jamuna River causes inundation. The study was conducted to assess the present drainage situation and suggest improvement of the drainage network system to provide an area free from water congestion within an acceptable environmental condition. Using the model SWMM, peak Runoff for 5 and 10 year return period of the each subcatchment is determined. The value of flow and depth obtained from the model and scatter plot comprising the flow of the conduit with the subcatchment runoff and node depth with the link flow measurement was taken into account for designing drainage conduit. The design depth of each conduit is kept 0.5 m larger than the model simulated value for considering any future adverse condition.

Paul (2015) applied SWMM to design a storm water drainage of AHS (Army Housing Scheme) Jolshiri Abason. The peak Runoff of the each subcatchment is analyzed using the model. The rainfall intensity has been evaluated from the IDF curve. Model simulated value of flow and depth and scatter plot comprising the flow of the conduit with the subcatchment runoff and node depth with the link flow measurement were used for conduit design. Pumping stations are introduced in each outlet as the water level of the Cannel is significantly lower than the HFL of Balu River during monsoon.

Ghosh and Hellweger (2011) did a comprehensive study using SWMM and the analysis part was done for 50 storm events. They found that the annual runoff is not independent on the spatial resolution of the model. The spatial resolution on peak flow simulation was altered. Peak flows decreased for large storms while an increase with an increasing level of aggregation was found for small storms.

Jang et al. (2007) applied SWMM for natural catchments and revealed that non-calibrated SWMM models were most accurate if the whole catchment was modeled as one single runoff unit. Those models provided good simulation results without calibration for three undeveloped Korean watersheds with areas ranging from 8.5 km² to 55.9 km².

Jiang et al., (2015) did a study employing SWMM model which simulated the urban flooding of Dongguan City in the rapidly urbanized southern China. It was found that

with the increase of the return period, the total precipitation, surface runoff, runoff coefficient increase, but the infiltration does not increase obviously as the study area has a very high impervious area rate. Finally, the results from the analysis showed that with 1-year return period precipitation, the studied area will have no flooding, but for the 2, 5, 10 and 20-year return period precipitation, the studied area will be inundated.

2.5 Studies on Waterlogging Problem in Khulna District

Awal (2014) identified in his study that sea-level rise and high tide is leading to prolonged water-logging in south-western part of Bangladesh. He took into account Jessore, Satkhira and Khulna district as the study area. According to him, plinth rising and elevating the local habitats and physical infrastructures can be considered as an immediate and short-term measure for the removal of water logging effect, whereas operation of Tidal River Management (TRM) technology might be considered for long-term or permanent solution for raising the low lands. He also suggested for excavation or re- excavation of dead or silted-up rivers, canals, ponds and irrigation channels to minimize the water logging problem in the region.

In the study of polder 36/1, it was revealed that crucial water logging problem exists in project area due to enormous sedimentation in the main drainage routes over the decades (IWM, 2013b). Drainage improvement of polders suggested two different option proposed considering Tidal River Management (TRM) and renovation of existing structure, dredging of silted up internal canals and peripheral rivers. Though TRM is an effective solution in drainage congestion, it requires huge land area requisition which is not always possible. However, option two is comparatively practical, considering the socio-economic context.

The Blue Gold Program management authority involved CEGIS to undertake EIA study (CEGIS, 2015) of polder 22. The study revealed that the most severe problem within the area is the saline water intrusion. But non-functioning water control structures like regulators, caused insufficient drainage and flashing capacity of the polder area that triggered drainage congestion. They proposed some interventions like re-sectioning embankment, repairing drainage sluices at Durgapur, Telikhali, and Darunmallik with lift gates in the country side, and flap gates in the river side and repairing one pipe sluice at the mouth of Gopipagla canal. An existing culvert at Fulbari severely impedes the drainage flow so reconstruction with a wider opening is suggested

to remove this flow disruption.

2.6 Drainage Congestion and Waterlogging in Polder 29

Drainage congestion has been identified as one of the major problems which impeding the development potential of polder 29. Almost all the khals inside the polder that are directly connected to the peripheral rivers, suffer from enormous drainage congestion. Gradual sedimentation in the Upper Bhadra River increased bed level of the river, water from the khals could not pass properly to the parent river (Upper Bhadra) which induces drainage congestion problems in the khals connected to Ghengrail. This is the reason why most of the sluice gates have been non-functional placed along the eastern periphery of the polder. Besides, most of the structures inside the polder are not performing properly upto the desired level and fail to drain out excess water during heavy rainfall thereby lead to drainage congestion. During the rainy season GWT lies within 1m depth and 2% of NCA is poorly drained. Water is drained from the soil poorly and the soil remains under water for 15 days to few months. As timely removal of surplus water is crucial it also accecerbates waterlogging issues. Most of the khals running through the polder area cannot cope with the increased rainfall occurrences during monsoon and post-monsoon periods leading to moderate to severe drainage congestions. Some of the severely affected khals are Arokhal, Asannagarkhal, Mora Bhadra River (locally termed as Mora Bhadrakhal), Bokultolakhhal, Telikhalkhal etc. As per opinion of the people, around 40% khals inside the polder (Bokultola, Arokhal, Telikhali etc.) suffer from severe drainage congestion, taking one week or more to properly drain out rainwater. In addition, almost 30% khals (Kanchannagar, Katakhal, Ruthimara etc.) suffer from moderate drainage congestions taking 2 to 6 days to drain out excess water (CEGIS, 2016). Due to insufficient drainage capacity of the khals, stormwater inundates agricultural land for 4 to 5 days and hampers agricultural productivity of Kharif-II crops specially (Brandsma, 1988).

2.7 Studies on Drainage Problem of Polder 29

The Blue Gold Program management authority entrusted CEGIS to carry out the EIA study (CEGIS, 2016) on polder 29. The study showed that there lies a major inundation problem due to gradual siltation of Upper Bhadra River and non-functional water management structures such as sluice gates, flap gates etc. During the rainy season the surplus water inundates agricultural land because of inadequate drainage capacity of the khals. In order to improve the drainage performance they proposed some interventions for the rehabilitation of the polder such as re-sectioning 16.16 km embankment, constructing retired embankment at Baro area and Jaliakhali, repairing the drainage sluices and outlets and excavation of nine drainage khals. It is expected that around 30% of khals adjacent to the periphery of the polder would be improved from drainage congestion in future. The cropping intensity is anticipated to increase by 15% after completion of the interventions. Interventions in Polder 29 will safeguard the polder against direct intrusion of tidal water.

Brandsma (1988) selected drainage unit sluice 6 of polder 29 as the study area. He proposed excavation of drainage khals and proper regulation of drainage flow at Bajiapati Bhul Baria to reduce the damage of HYV crop due to inundation. He found that division of drainage unit sluice 6 into a separate east and west part and construction of a new sluice in the east embankment dike will increase the drainage capacity of the drainage unit.

Bakuluzzaman and Naher (2015) found there lies a congestion in Ramkhali canal due to low elevation of land area, siltation in the canal bed. But according to the villagers of Kumaghata, non-operational sluice gate is the main reason of water logging. Sahas, joykhali and vanderpara are the important gates for the villagers. Sahas gate is blocked due to soil accumulation in front of the gate which is also responsible for drainage congestion. After consulting with the respondents of polder 29 they gave some suggestion like re-excavation of canal (Kakmari, Ramakhali, Patchakhal, Bhadra River) and restoration of Sahas, Golaimari and Chatchatia gate to minimize this inundation problem.

Summary:

From the review of the literature it is observed that many studies have been performed for analyzing waterlogging problem on some coastal Polders along with other areas of Bangladesh and other countries using SWRM, MIKE 11 etc. Some studies suggested taking measures like excavation of drainage channel, repairing sluices to solve water logging problem. But assessing waterlogging problem using drainage model and suggestion of adaptive measure i.e. pumping is unavailable for this study area. Polder 29 is one of the largest catchments that is facing inundation problem over the decades. So this study is carried out to identify drainage capacity of the internal khals and extent of inundation of this area using GeoSWMM and an improvement plan is suggested to lessen the water logging condition of this study area.

CHAPTER THREE

THEORITICAL BACKGROUND AND MODEL DESCRIPTION

3.1 General

Models describe our beliefs about how the world functions. In mathematical modeling, we translate those beliefs into the language of mathematics (Marion, G. 2008). It consists of simple to complex mathematical equations with linear and/or non-linear terms and ordinary or partial differential equation terms together with logical statements, expressing relations between variables and parameters. Overall, it is a process in which real-life problems are translated into mathematical language, solved within a symbolic system, and the solutions tested back within the real-life system (Verschaffel et al., 2002).

Models are constructed to study pertinent system responses as studying the real-world processes could be extremely time consuming, expensive and even dangerous. Therefore, a model can be viewed as a decision-making tool which provides approximate but the model equations are not sufficient to represent the total complexity of real world rather than the simplification (Karssenber, D. 2002). As noted by, a model will never describe every aspect of the real world and will contain aspects that have no corresponding counterpart in the real world. In general, a model is constructed in such a way that it is as simple as possible and as complex as needed to produce results of required resolution and accuracy (Sarwar, 2017)

3.2 Categorization of Models

Mathematical models could be classified in many different ways depending on the specific technique used, nature of input and output, discretization of the decision space and so on. These classifications are obviously overlapping and not mutually exclusive. Because of this, all the possible groups cannot be represented in a simple tree format (Sarwar2017). Different types of mathematical models which are mainly used in water resources are shown in Figure 3.1.

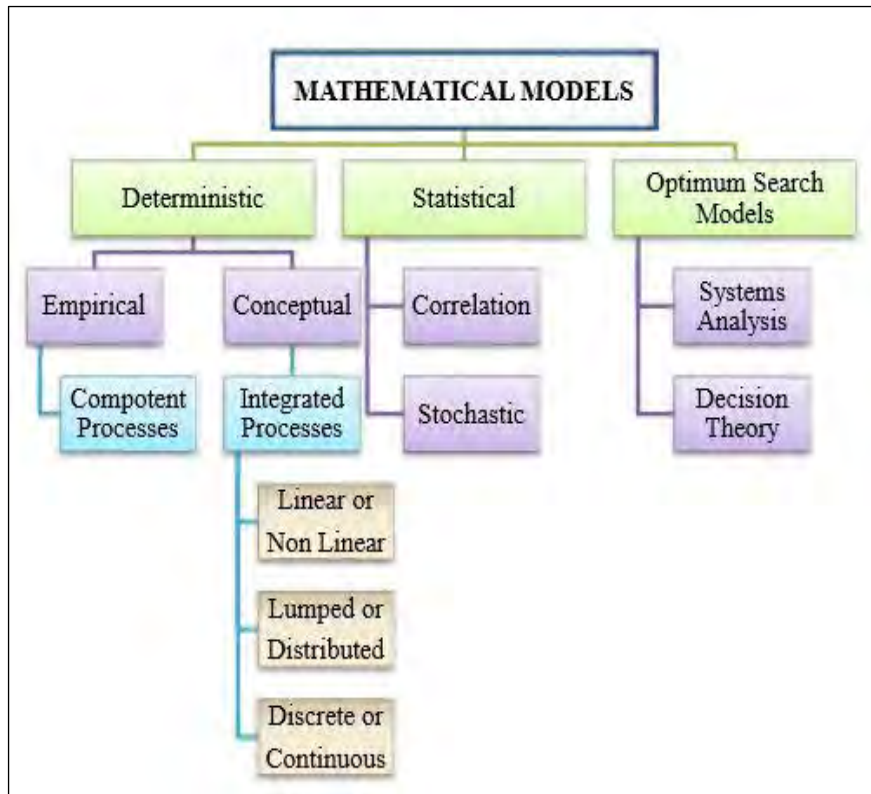


Figure 3.1: Classification of mathematical modeling

3.3 Choice of Model

Hydrological models express the related components of hydrological cycle where hydraulic model of a water or sewer system is used to analyze the system's hydraulic behavior. Choice of models depend upon the purpose throughout the world.

MIKE SHE

MIKE SHE includes all important aspects of hydrology. In order to divide rainfall into runoff, evapotranspiration and groundwater recharge, MIKE SHE is the fastest, most dependable way to produce accurate integrated models. MIKE SHE includes both a simple, semi-distributed overland flow method for rainfall runoff modeling and a 2D, diffusive wave, finite difference method for detailed runoff and flood modeling. It can simulate detailed flooding based on fine scale topography in a coarser numerical grid as well as detailed two-way exchange with rivers. MIKE SHE Studio is the ideal package for distributed rainfall-runoff modeling or basin wide water balance and water management studies.

MIKE 11- NAM

MIKE 11-NAM is a rainfall-runoff model that is part of the MIKE 11 RR module. It has been used by many researchers (Hafezparast et al., 2013). The NAM (Nedbør Affstrømnings Model) model is a deterministic, lumped conceptual rainfall-runoff model which is originally developed by the Technical University of Denmark (Nielsen and Hansen, 1973). In this model, the hydrological cycle is the basis of the quantitative simulation of water storage and flows in the watershed and its parameters represent an average value for the whole watershed. Generally, rainfall, potential evaporation and temperature are the input data needed for this model and the output of the model is the watershed outflow over time. DHI (DHI, 2009)

RORB

RORB is a rainfall-runoff model which is applicable for both rural and urban catchments. The application of RORB model is relatively user friendly (Choo, E. L. 2004). Various input parameters of this model are subcatchment area and landuse condition, channel type, length and slope, fraction imperviousness, rainfall and streamflow data. In addition, there are only two model parameters, m and k which a user has to determine for the rainfall-runoff process. This distributed model was used by Chow in 2004 to determine the runoff in Sg Klang Basin(urban) and Sg Bernam Basin(rural) located in Malaysia.

ILSAX

This model was developed for urban catchment. This model was further developed to the DRAINS model (O'Loughlin, G. 1993). Rainfall can be applied to the model either uniformly or in the standard version of ISLAX. In this model every inlet to the pipe system is modeled. It also includes loss model that uses Hortons equation. Hydrographs are generated using time area method. The program also allows modeling of pipes, box culvert and both regular and irregular natural channels.

RAFTS

The model is also known as „Regional Stormwater Drainage Model“. It consists of five modules of which two are used to convert rainfall into runoff. Laurenson's nonlinear runoff routing model (Laurenson, E. M. 1964) modified by Atiken (AITKEN, A.P.

1968) is contained in one and the other uses Phillip's infiltration equation (Philip, J. R. 1957). In this method each sub catchment is divided into ten sub areas and the rainfall excess is routed and summed through the ten sub areas using a non-linear storage. Hydrographs may be translated between sub-catchment outlets. RAFTS uses either initial loss or Phillip's infiltration equation in the rainfall loss module. This model can be used both for rural and urban catchments.

HEC-HMS

HEC-HMS is a semi-distributed mathematical model, developed by the US Army Corps of Engineers and is suitable for dendritic watershed systems (Scharffenberg, W. 2015). It is used to compute runoff hydrographs for a network of watersheds. The model evaluates infiltration losses, transforms precipitation into runoff hydrographs, and routes hydrographs through open channel routing. It does not include a detailed model of interflow or groundwater flow instead representing only the combined outflow as baseflow (Haque, 2018). HEC-HMS is used in combination with HEC-RAS for calculation of both the hydrology and hydraulics of a stormwater system or network.

WinTR-20

WinTR-20 (hydrologic model) is used to evaluate flooding problems, alternatives for flood control (reservoirs, channel modification, and diversion), and impacts of changing land use on the hydrologic response of watersheds. It is best suited to predict stream flows in large watersheds. It computes direct runoff and develops hydrographs resulting from any synthetic or natural rainstorm. The rainfall-frequency data will be used to develop site-specific rainfall distributions.

WinTR-55

WinTR-55 is a single-event rainfall-runoff small watershed hydrologic model. The model is an input/output interface which runs WinTR-20 in the background to generate, route and add hydrographs. The WinTR-55 generates hydrographs from both urban and agricultural areas at selected points along the stream system. Hydrographs are routed downstream through channels and/or reservoirs. Multiple sub-areas can be modeled within the watershed. A rainfall-runoff analysis can be performed on up to ten sub-areas and up to ten reaches. The total drainage area modeled cannot exceed 25 square miles.

HydroCAD

It is a computer aided design program for modeling the hydrology and hydraulics of stormwater runoff. Runoff hydrographs are computed using the SCS runoff equation and the SCS dimensionless unit hydrograph. For the hydrologic computations, there is no provision for recovery of initial abstraction or infiltration during periods of no rainfall within an event. The program computes runoff hydrographs, routes flows through channel reaches and reservoirs, and combines hydrographs at confluences of the watershed stream system.

SWMM

SWMM is a large, relatively sophisticated hydrologic, hydraulic and water quality simulated model developed by a consortium of American engineers for the US Environmental Protection Agency (EPA) (Wurbs, R. A. 1994). The model is capable to simulate watersheds with complex storm sewer networks, detention storage and water treatment facilities. It has been globally applied for modeling drainage system, regional water quality management planning and rehabilitation purposes.

GeoSWMM

The GeoSWMM model, a geospatial stormwater and wastewater management model, has been developed as an add-in to ESRI's ArcGIS 10.x. (Rossman and Huber, 2016). It is a dynamic rainfall-runoff simulation modeling tool for a single event or long-term (continuous) simulation of runoff quantity and quality. It is built upon EPA's SWMM 5 model which requires users to manually enter data through its interface GeoSWMM makes the input data preparation extremely easy and efficient through the ArcMap™ based tools, geodatabase and mapping features. It allows users to simulate flow, velocity of water, depth of water and water quality for projects ranging from a simple site development to a complex urban watershed involving open channels, sewers, pumps, hydraulic structures and Best Management Practices (BMPs).

By integrating SWMM with ArcMap, GeoSWMM helps users to significantly reduce the time it takes to prepare model input, increase the accuracy of input data, store and manage data in a database environment and visualize and inspect model objects on a map. It allows users to run simulation and view output without ever exiting the GIS environment.

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

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

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

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

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

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

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

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

Applications of GeoSWMM

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

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

3.4 GeoSWMM Conceptual Model

3.4.1 Introduction

GeoSWMM conceptualizes a drainage system as a series of water and material flows between several major environmental compartments which also contain GeoSWMM objects. The overall hydrological process of GeoSWMM is shown in Figure 3.2

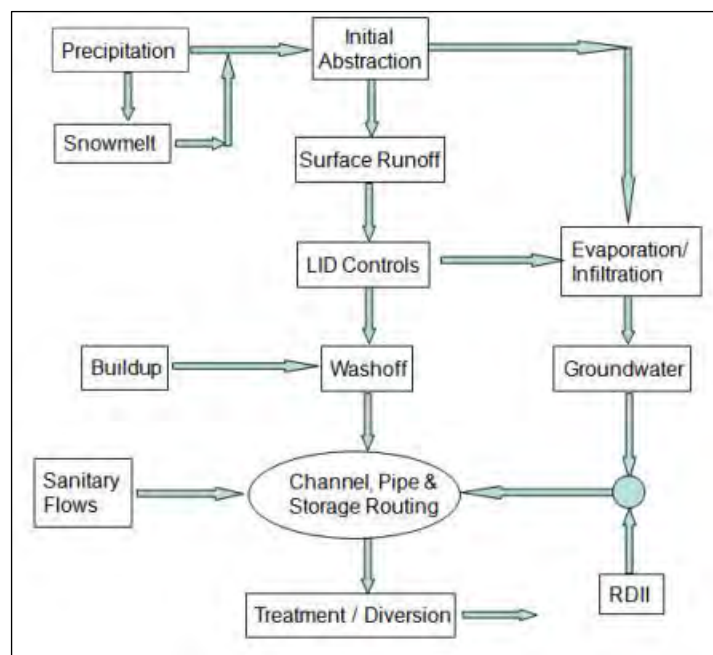


Figure 3.2: Process modeled by GeoSWMM

The atmosphere compartment, from which precipitation falls and pollutants are deposited onto the land surface compartment. GeoSWMM uses rain gage objects to represent rainfall inputs to the system. The land surface compartment receives precipitation from the atmospheric compartment in the form of rain or snow; it sends outflow in the form of infiltration to the groundwater compartment and also as surface runoff and pollutant loadings to the transport compartment. The groundwater compartment receives infiltration from the land surface compartment and transfers a portion of this inflow to the transport compartment. This compartment is modeled using aquifer objects. The transport compartment contains a network of conveyance elements (channels, pipes, pumps, and regulators) and storage/treatment units that transport water to outfalls or to treatment facilities. Inflows to this compartment can come from surface runoff, groundwater interflow, sanitary dry weather flow, or from user-defined hydrographs. The components of the transport compartment are modeled with node and link objects.

Not all compartments need appear in a particular GeoSWMM model. For example, one could model just the transport compartment, using pre-defined hydrographs as inputs. Simulation procedure that GeoSWMM used to model stormwater runoff is shown in the Figure 3.3

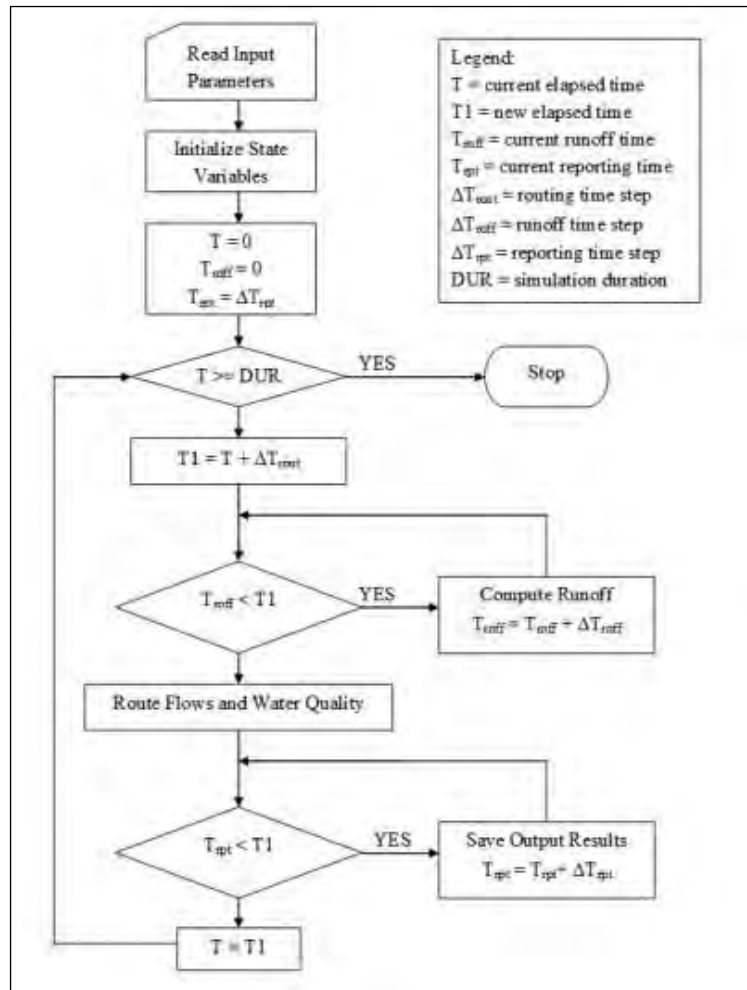


Figure 3.3: Flow chart of GeoSWMM's simulation procedure

3.4.2 Visual Objects

Figure 3.4 depicts how a collection of GeoSWMM's visual objects might be arranged together to represent a stormwater drainage system. These objects can be displayed on a map in the GeoSWMM workspace. Though there are several objects, but the following sections describe the objects which are used in this study.

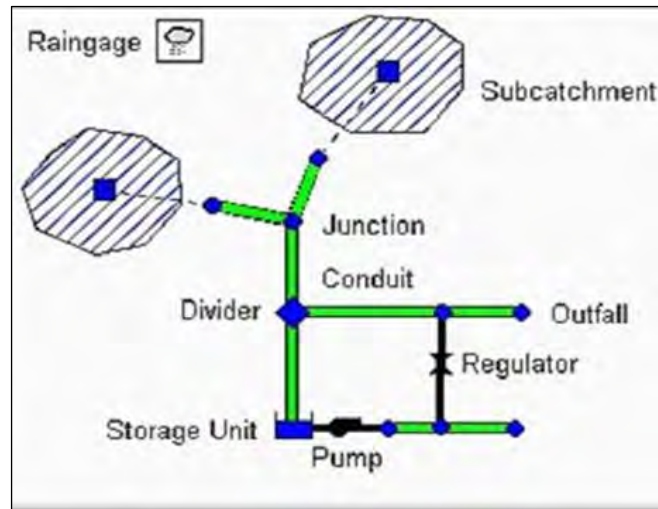


Figure 3.4: Example of physical objects used to model a drainage system (Source: User Manual for EPA-SWMM 5.0)

Rain Gages

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

The principal input properties of rain gages include:

1. Rainfall data type (e.g., intensity, volume, or volume or cumulative volume)
2. Recording time interval (e.g., hourly, 15-minute, etc)
3. Source of rainfall data (input tinme series or external file)
4. Name of rainfall data source

Subcatchments

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

Subcatchments can be divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Impervious areas are themselves divided into two subareas - one that contains depression storage and another that does not. Runoff flow from one

subarea in a subcatchment can be routed to the other subarea, or both subareas can drain to the subcatchment outlet. Infiltration of rainfall from the pervious area of a subcatchment into the unsaturated upper soil zone can be described using three different models which are discussed later.

The other principal input parameters for subcatchments include:

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

Junction Nodes

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

The principal input parameters for a junction are:

- Invert elevation
- Height to ground surface
- Pondered surface area when flooded (optional)
- External inflow data (optional)

Outfall nodes

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

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

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

The principal input parameters for outfalls include:

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

Conduits

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

Most open channels can be represented with a rectangular, trapezoidal, or user-defined irregular cross-section shape. In this research, the later shape is used by inserting a transect object which define how the channel's depth varies with distance across the cross-section. GeoSWMM uses the Manning's equation to express the relationship among the flow rate (Q), cross-sectional area (A), hydraulic radius (R), and slope (S) in all conduits. For standard U.S. units:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \quad (3.1)$$

Here n is the Manning roughness coefficient. The slope S is computed from the elevation of a conduit's end node inverts and its offsets.

$$S = \Delta y / \Delta x \quad (3.2)$$

Where,

Δy = the difference in elevation

Δx = the horizontal distance between the invert at each end of the conduit.

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

The principal input parameters for conduits are:

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

Calculation of entry/exit loss:

Energy losses can occur at bends, contractions, or enlargements in conduit geometry and also be associated with flows entering a conduit from a larger water body or flows exiting a conduit to a larger water body.

A minor loss (exit/entry loss) is represented as the product of a loss coefficient and the local velocity head for a specific location i along a conduit:

$$\Delta H_L = K_{m,i} \frac{U_i^2}{2g} \quad (3.3)$$

where ΔH_L is the minor head loss (ft), $K_{m,i}$ is a loss coefficient, and U_i is flow velocity (ft/sec). The location index i is 1 for an entrance loss based on the conduit's upstream velocity, 2 for an exit loss based on its downstream velocity, or 3 for an average loss based on its average velocity. Minor losses can be included in the St. Venant momentum equation for a conduit by treating them as a loss per unit length, h_L in the same way that the friction slope S_f is treated. This modified version of the momentum equation is:

$$\frac{\partial Q}{\partial t} + \frac{\partial(Q^2/A)}{\partial x} + gA \frac{\partial H}{\partial x} + gAS_f - U q_L/2 + gAh_L = 0 \quad (3.4)$$

where $h_L = \sum_{i=1}^3 K_{m,i} U_i^2 / (2gL)$ with L being the conduit length.

In GeoSWMM manual, entry and exit loss co-efficient values are provided for box culverts, pipes etc which ranges between 0 to 1 but in case of open channels 0.3 for entry loss co-efficient and 1 for exit loss co-efficient was suggested (Rossman, 2016). Alternatively, Manning's roughness value can be adjusted to cover the impact of these entry and exit losses.

Operation of flapgates:

Flapgates prevent any reverse flow through conduits. Operation of the gate (opening and closing) is performed by computing head difference of each time step between inside and outside of the flapgates.

Pump

Pumps are links used to lift water to higher elevations. A pump curve describes the relation between a pump's flow rate and conditions at its inlet and outlet nodes. Four different types of pump curves are used in GeoSWMM and they are given in Figure 3.5. The on/off status of pumps can be controlled dynamically by specifying startup and shutoff water depths at the inlet node or through user-defined Control Rules. Rules can also be used to simulate variable speed drives that modulate pump flow.

The principal input parameters for a pump include:

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

Whenever a pump link is encountered in either the dynamic wave or kinematic wave methods its new flow is found directly from its pump curve using whatever values were last computed for nodal heads and volumes.

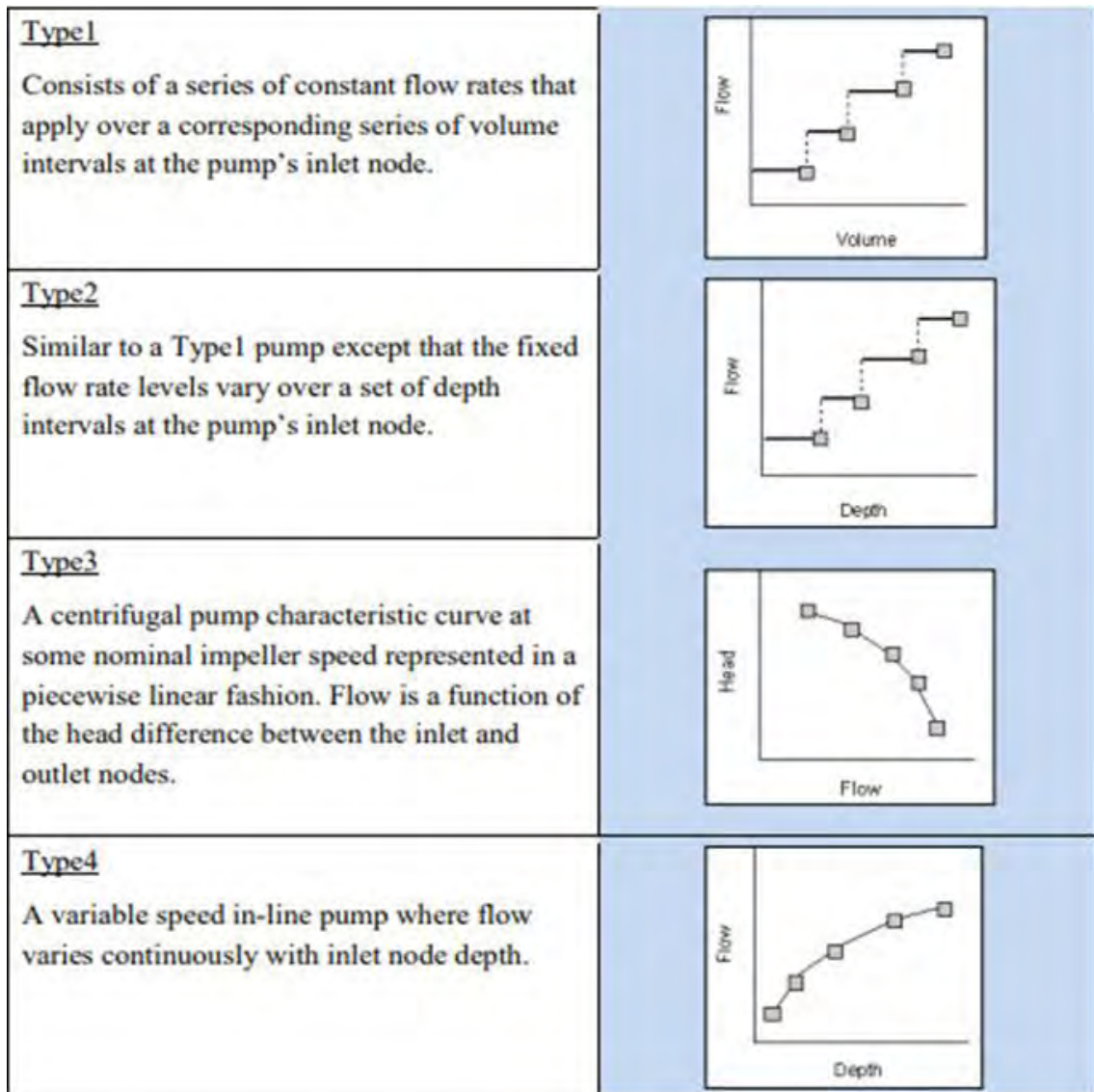


Figure 3.5: Different types of pump curves (Source: User Manual for EPA-SWMM 5.0)

Besides the four curves there is another pump named as “Ideal” pump where flow rate equals the inflow rate at its inlet node. No curve is required. The pump must be the only outflow link from its inlet node. Used mainly for preliminary design.

3.4.3 Non-Visual Objects

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

Climatology

Temperature

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

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

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

Evaporation

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

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

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

If rates are read directly from a climate file, then a set of monthly pan coefficients should also be supplied to convert the pan evaporation data to free water-surface values. An option is also available to allow evaporation only during periods with no precipitation. In this study, a single constant value 3mm/day (CEGIS, 2016) is used for simulation

Wind Speed

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

Transects

Transects refer to the geometric data that describe how bottom elevation varies with horizontal distance over the cross section of a natural channel or irregular-shaped conduit. Fig. 3.5 displays an example transect for a natural channel.

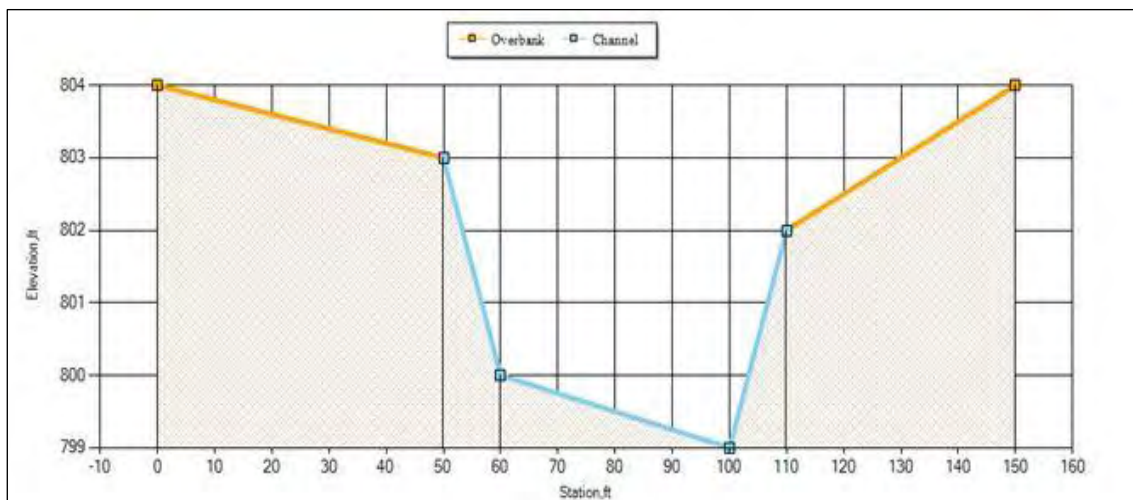


Figure 3.6: Example of a natural channel transect

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

Control Rules

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

Simple time-based pump control

Rule R1
If simulation time > 8
Then pump 12 status = On
Else pump 12 status = Off

Pump station operation

Rule R3A
If node N1 depth > 5
Then pump N1A status = On
RULE R3B
If node N1 depth > 7
Then pump N1B status = On
RULE R3C
If node N1 depth <= 3
Then pump N1A status = Off
And Pump N1B status = Off

Land Uses

Land Uses are categories of development activities or land surface characteristics assigned to subcatchments. Examples of land use activities are residential, commercial, industrial, and undeveloped. Land surface characteristics might include rooftops, lawns, paved roads, undisturbed soils, etc. Land uses are used solely to account for spatial variation in pollutant buildup and washoff rates within subcatchments.

The GeoSWMM user has many options for defining land uses and assigning them to subcatchment areas. One approach is to assign a mix of land uses for each subcatchment, which results in all land uses within the subcatchment having the same pervious and impervious characteristics. Another approach is to create subcatchments that have a single land use classification along with a distinct set of pervious and impervious characteristics that reflects the classification.

Curves

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

- Storage - describes how the surface area of a Storage Unit node varies with water depth

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

Time Series

Time Series objects are used to describe how certain object properties vary with time.

Time series can be used to describe:

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

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

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

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

Time Patterns

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

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

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

3.5 Computational Methods

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

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

3.5.1 Surface Runoff

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

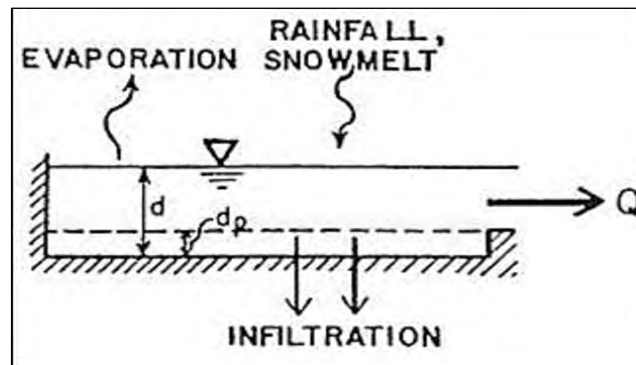


Figure 3.7: Surface runoff (Illustrative diagram)

3.5.2 Infiltration

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

Horton's Equation

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

$$f_p = f_\infty + (f_0 - f_\infty)e^{-k_d t} \quad (3.5)$$

Where,

f_p = infiltration capacity into soil (ft/s)

f_∞ = minimum or equilibrium value of f_p at $t=\infty$ (ft/s)

f_0 = maximum or initial value of f_p at $t=0$ (ft/s)

t = time from beginning of storm (s)

k_d = decay co-efficient (s^{-1})

Green-Ampt Method

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

Curve Number Method

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

In this study, Hortons equation is used to calculate infiltration because this is the widely used method for drainage modeling (Rossman, 2016). Though curve number method is used in other studies but this is applicable where the study area is large because the soil map required for this contains a coarser resolution and as this study area is small, therefore Horton's methods suits the best to calculate infiltration. The necessary input parameter used in this study is selected from GeoSWMM user manual.

3.5.3 Flow Routing

Flow routing within a conduit link in GeoSWMM is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e., the Saint Venant flow equations). There are four routing methods i.e Steady flow routing, Kinematic wave routing, Diffusive wave routing and Dynamic wave routing. GeoSWMM considers three routing methods which are given below:

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

Steady Flow Routing

Steady Flow routing represents the simplest type of routing possible (actually no routing) by assuming that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The normal flow equation is used to relate flow rate to flow area (or depth). This type of routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal or pressurized flow. It can only be used with dendritic conveyance networks, where each node has only a single outflow link (unless the node is a divider in which case two outflow links are required). This form of routing is insensitive to the time step employed and is really only appropriate for preliminary analysis using long-term continuous simulations.

Kinematic Wave Routing

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

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

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

Dynamic Wave Routing

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

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

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

In this study Dynamic Routing method is used because some part of subcatchments contain adverse slope gradient and there lies a significant backwater effect due to d/s flow restrictions at the outfalls.

Each of these routing methods employs the Manning equation to relate flow rate to flow depth and bed (or friction) slope.

3.5.4 Surface Ponding

Normally in flow routing, when the flow into a junction exceeds the capacity of the system to transport it further downstream, the excess volume overflows the system and is lost. An option exists to have instead the excess volume be stored atop the junction, in a ponded fashion, and be reintroduced into the system as capacity permits. Under Steady and Kinematic Wave flow routing, the ponded water is stored simply as an excess volume. For Dynamic Wave routing, which is influenced by the water depths

maintained at nodes, the excess volume is assumed to pond over the node with a constant surface area. This amount of surface area is an input parameter supplied for the junction.

Alternatively, the user may wish to represent the surface overflow system explicitly. In open channel systems this can include road overflows at bridges or culvert crossings as well as additional floodplain storage areas. In closed conduit systems, surface overflows may be conveyed down streets, alleys, or other surface routes to the next available open channel.

3.6 Frequency Analysis for Selecting Design Storm and Water Level

Frequency of a hydrologic event, such as the annual peak flow is the probability that a value will be equaled or exceeded in any year. This is more appropriately called the exceedence probability, $P(F)$. The reciprocal of the exceedence probability is the return period T in years. The length of record should be sufficient to justify extrapolating the frequency relationship. For example, it might be reasonable to estimate a 50-year flood on the basis of a 30-year record, but to estimate a 100-year flood on the basis of a 10-year record would normally be absurd (Neill 1973). Viessman and Lewis (1996) noted that as a general rule, frequency analysis is cautioned when working with shorter records and estimating frequencies of hydrologic events greater than twice the record length.

Frequency analysis can be conducted in two goat ways: one is the analytical approach and the other is the graphical technique in which flood magnitudes are usually plotted against probability of exceedence.

3.6.1 Analytical Frequency Analysis

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

In this present study, all the distributions are analyzed for rainfall data to find out best-fit frequency distribution function for various return period. Finally design rainfall value is selected based on the analysis for developing an adaptation measure.

3.6.2 Probability Distribution Function

Extreme Value Distribution

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

$$F(x) = \exp \left[- \left(1 - \kappa \frac{x-u}{\alpha} \right)^{1/\kappa} \right] \quad (3.6)$$

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

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

$$F(x) = \exp \left[- \exp \left(- \frac{x-u}{\alpha} \right) \right] \quad -\infty \leq x \leq \infty \quad (3.7)$$

The parameters are estimated by

$$\alpha = \frac{\sqrt{6}}{\pi} s \quad \text{and} \quad u = \bar{x} - 0.5772\alpha \quad (3.8)$$

Eq (4.2) can be expressed as

$$F(x) = e^{-e^{-y}} \quad (3.9)$$

where y is the reduced variate defined as

$$y = \frac{x - u}{\alpha} \quad (3.10)$$

Solving Eq (4.4) for y :

$$y = -\ln \left[\ln \left(\frac{1}{F(x)} \right) \right] \quad (3.11)$$

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

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

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

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

$$y_T = -\ln \left[\ln \left(\frac{T}{T-1} \right) \right] \quad (3.12)$$

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

$$x_T = u + \alpha y_T \quad (3.13)$$

3.6.3 Frequency Analysis Using Frequency Factors

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

$$x_T = \bar{x} + K_T s \quad (3.14)$$

where x_T is a flood of specified probability or return period T , \bar{x} is the mean of the

flood series, s is the standard deviation of the series; and K_T is the frequency factor and is a function of return period and type of probability distribution, as well as coefficient of skewness for skewed distributions, such as LP3.

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

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

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

$$K_T = \frac{\sqrt{6}}{\pi} \left[0.5772 + \ln \left\{ \ln \left(\frac{T}{T-1} \right) \right\} \right] \quad (3.15)$$

Lognormal Distribution

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

- $y_i = \log x_i$
- Compute the mean, \bar{y} and standard deviation s_y
- Compute $y_T = \bar{y} + K_T s_y$

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

So, K_T can be taken from Table

- Finally compute $x_T = \text{antilog } y_T$

Log-Pearson Type- III Distribution

The recommended procedure for use of the LP3 Distribution is to convert the data series to logarithms and compute:

- $y_i = \log x_i$
- Compute the mean, \bar{y} and standard deviation s_y ,
- Compute coefficient of skewness

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

- Compute $y_T = \bar{y} + K_T s_y$
Where, K_T is taken from defined Table
- Finally compute $x_T = \text{antilog } y_T$

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

Goodness-of-Fit Tests

A probabilistic goodness-of-fit (GOF) test is an important tool for assessing the closeness of a theoretical model in representing the frequency distribution of observations (Ott, 1995). The goodness of fit of a probability distribution can be tested by comparing the theoretical and sample values of the relative frequency or the cumulative frequency function. In the case of the relative frequency function, the χ^2 – test is used and with cumulative frequency function the Kolmogorov-Smirnov test is used. The minimum values of chi square represent the best fit distribution of data.

Chi-Square Test

The test statistic is given by

$$\chi^2 = \sum_{i=1}^k \frac{n[f_s(x_i) - p(x_i)]^2}{p(x_i)} \quad (3.16)$$

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

To describe the χ^2 test, the χ^2 probability distribution must be defined. A χ^2 distribution with $\nu = k-l-1$ degrees of freedom (l is the number of parameters used in fitting the proposed distribution) is the distribution for the sum of squares of ν independent standard normal random variables z_i . The critical χ^2 distribution function is taken from a table suggested by Haan (1977). A confidence level is chosen for the test; it is often expressed as $1-\alpha$, where α is termed the significance level.

Confidence limit

Statistical estimates are often presented with a range, or confidence interval within which the true value can be reasonably expected to lie. The size of the confidence interval depends on the confidence level β for which there is significance level α , given by $\alpha = (1-\beta)/2$. The upper boundary and lower boundary values of the confidence interval are called the confidence limits. For example, if $\beta = 90\%$ then $\alpha = 0.05$ or 5%.

CHAPTER FOUR

METHODOLOGY AND MODEL DEVELOPMENT

4.1 General

Various kinds of data of previous and current years have been collected and compiled in order to develop mathematical hydrologic and hydrodynamic model. These data are the basis for further analysis and interpretation of model results leading to accurate assessment of hydrologic and hydrodynamic condition of the study area. According to the modeling requirements, satellite images (DEM), rainfall data, water level data and cross-sections of khals within the study area have been collected. This chapter describes a brief description about the selection of study area, data source, data collection, data processing and analysis of data along with model development.

4.2 Brief Description of the Study Area (Polder 29)

4.2.1 Background

The Polder was conceived in the year of 1960 under Coastal Embankment Project (CEP). Construction of the Polder was started in 1966 and completed in 1971 and was one of the two polders selected as pilot project implementation under the Delta Development Project in 1988 (CEGIS, 2016). The original concept of construction of this Polder was only to protect the coast from tidal flooding, salinity incursion and water logging resulted in high agricultural productivity on the coastal lands for 10-15 years (Bakuluzzaman and Naher, 2015). Parts of the polder are threatened by drainage congestion which could worsen with changes occurring with on-going climate change.

Khulna district associated with 10 polders. Among them polder no 29 has been selected as the study area because according to BWDB sources, there lies a major inundation problem due to inadequate drainage and as a result livelihood and agriculture are badly affected.

4.2.2 Location of the Study Area

The polder is situated in South-Western hydrological region of Bangladesh with administrative jurisdiction lying with the Khulna operation and maintainance Division-1, BWDB, Khulna. It is surrounded by the Upper Bhadra River in the east and Ghengrail River in the west. Polder 29 covers the entire Bhandarpara union as well as Sarappur union of Dumuria upazila small portion of Dumuria union, more than half of Sahas union of Khulna district. It also consists a small portion of Surkhali union of BatiaghataUpazilla, Khulna District under its coverage. Figure 4.1 shows the boundary of the study area comprising approximately 7933 ha.

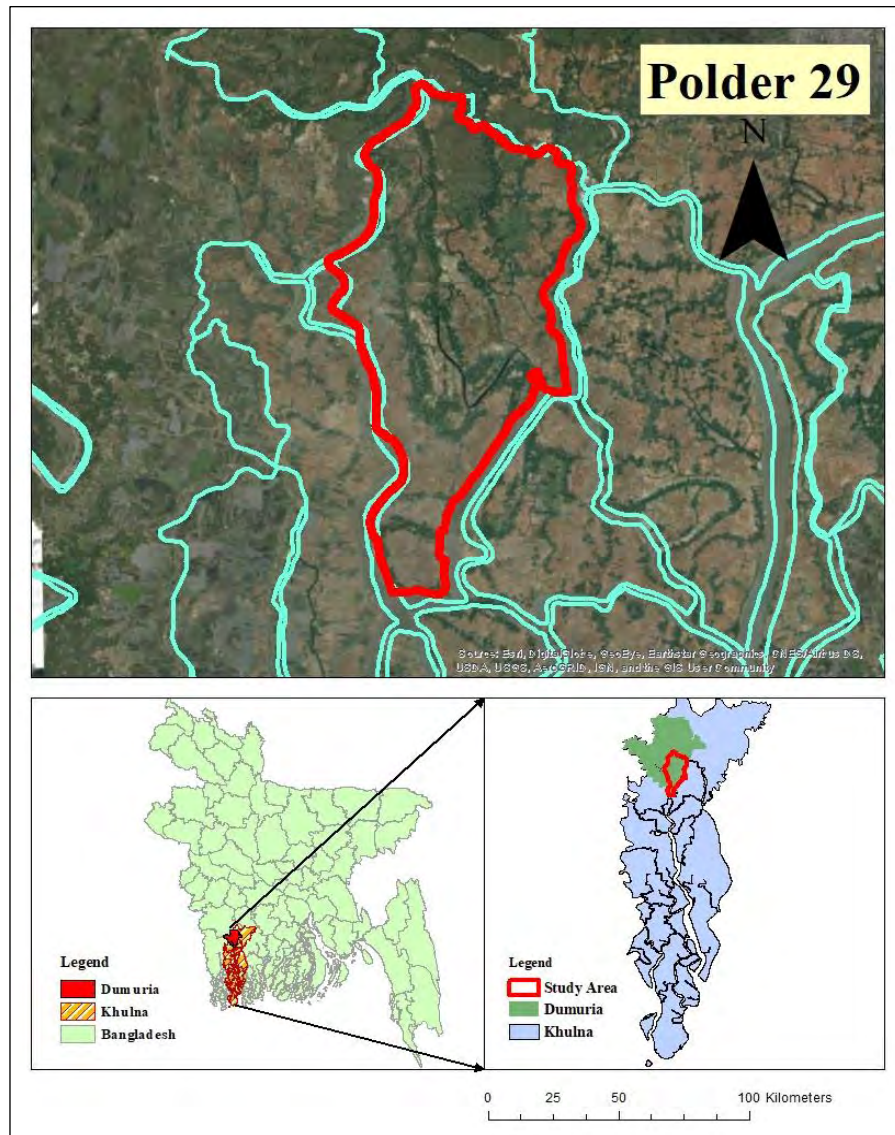


Figure 4.1 Location map of the study area

4.2.3 Topography of the Study Area

Digital Elevation Model (DEM) infers that topographically the area is flat by analyzing the reduced Levels (RLs). It varies from 0.46 to 4.4 mPWD inside the polder with average RL of around 3mPWD. From the DEM, it is found that around 50% lands of the areas have elevation between 1 to 3 mPWD. The elevations showed a very minor downward sloping from north to south, which eventually draws water from the up- stream basins to the Ghengrail and Bhadra rivers. Most of the land elevations near the polder periphery are higher than that of the central areas.

4.2.4. Land Use of the Study Area

The gross area of the polder 29 is about 7933 ha of which 4443 ha is net cultivable area (NCA). The NCA is about 56% of the gross area. The land use pattern comprises of 20.5% settlements, 7%water bodies (river/khals/ponds), 1% road and 15.5% gher of the gross area. Fishers and CEGIS fisheries expert reported that 390 ha (7% of NCA) is under rice cum fish culture (CEGIS, 2016)

4.2.5. Land Type of the Study Area

Land type defined as a process of classifying cultivated land based on the seasonal inundation depth of normal flooding. According to Soil Resource Development Institute (SRDI, 1988), five land types (High land, Medium High Land, Medium Low land, Low land and very low land) have been classified. The entire polder area falls under medium highland (F1) category which is normally flooded between 0-90 cm depth of water continuously for more than two weeks to few months during the monsoon season.

4.2.6. Soil Salinity of the Study Area

Generally, the study area lies in the semi saline zone which means the river water surrounding the polder area is saline during part of the year. Soil salinity inside the polder area increases gradually over the years (SOLARIS, 2006). During field visit, local farmers and Agriculture officers reported non-functional water control structures are mainly responsible for salinity intrusion inside the polder area. Soil and water salinity gradually increases from January and reaches maximum level in

the month of March-April and then decreases due to onset of monsoon rainfall (CEGIS, 2016). From early July until early November the salinity of the river water does not exceed 2000 uS/cm (Brandsma, 1988).

4.2.7 Meteorology

Rainfall

Rainy season is nominal in the study area in comparison to other districts of the country (BWDB, 2015). May to second half of October are the wettest months with highest rainfall intensity whereas November to February are the driest months of the year with negligible rainfall. Average rainfall during the wettest period is around 1750 mm, while 150 mm rainfall is recorded during the rest of the year. According to the trend analysis of 31 years (1978 to 2008), the highest and lowest values of rainfall are usually observed during the months of July (343 mm) and December (7 mm) respectively (CEGIS, 2016)

Temperature

Seasonal variation of temperature in this delta area is distinct. Temperature data of last 30 years (1948 to 2008) revealed that mean maximum temperature ranges from 19.3°C to 30.4°C with the highest temperature experienced in the month of May. There is also notable fluctuation in minimum temperature, which varies between 15.37°C to 25.2°C with the lowest temperature experienced in the month of January.

Relative Humidity

Relative humidity values are usually higher in the coastal areas compared to the other parts of the country because of having a greater extent of water bodies leading to increased evaporation. The value depends on the effect of atmospheric water vapors coupled with temperature. Relative humidity value ascents above 85% in monsoon (June to September), and starts decreasing from post monsoon season following the monsoon rainfall (CEGIS, 2016).

Wind Speed

A record of last 35 years (1978 to 2012) estimated average monthly wind speeds is highest in April (around 160 kph) and lowest in November (around 40 kph). During

cyclone Sidr (2007) and Aila (2009), 1 minute sustained wind speeds were recorded as 260 kph and 120 kph respectively in Khulna district.

Sunshine Hour

The average sunshine hour data has been collected from Khulna BMD station (1990-2010). It was found that from October to May, daily average sunshine hours are higher than 7 hours, but due to increased extent of cloud cover in monsoon (June to September the values drop below 5 (CEGIS, 2016).

4.2.8 Water Resources System

The water resources system of the Polder area plays an indispensable role in fulfilling the demand of the surrounding ecosystem and depends livelihood for a significant amount of people. It is the source of water supply which and plays a significant role in assimilating and diluting wastes, attenuating and regulating drainage, recharge into the aquifer, and maintaining the environment for aquatic habitats (BWDB, 2015).

River and Khal System

Polder 29 is located 75 km away from the Bay of Bengal and surrounded by the lower Bhadra (east) and Ghengrail (west) rivers. These two rivers fall into the Sibsa River System, and are directly fed by the oceanic tides (CEGIS, 2016). The delta area is subjected to diurnal tide. River adjacent to western boundary (Ghengrail River) receives water from Bay of Bengal during high tide which feeds the peripheral Ghengrail and Upper Bhadra rivers. The Arokkal, Ramakhali khal, Asannagar khal etc. are the tributaries of the Ghengrail which maintain the water resources functions of the river.

On the other hand, the hydrological functions of the Upper Bhadra River are supported by a number of its tributaries namely, Kata khal, Bakultola khal, Kanchannagar khal etc. The area is intersected by a number of creeks and khals such as Ruhimatrakhal, MoraBhadra khal, Ratankhali khal, Jaliakhali Khal, Telikhali Diversion Khal, which are connected to the drainage network, mostly through regulators. The river and khal systems of the surrounding polder is shown in Figure 4.2.

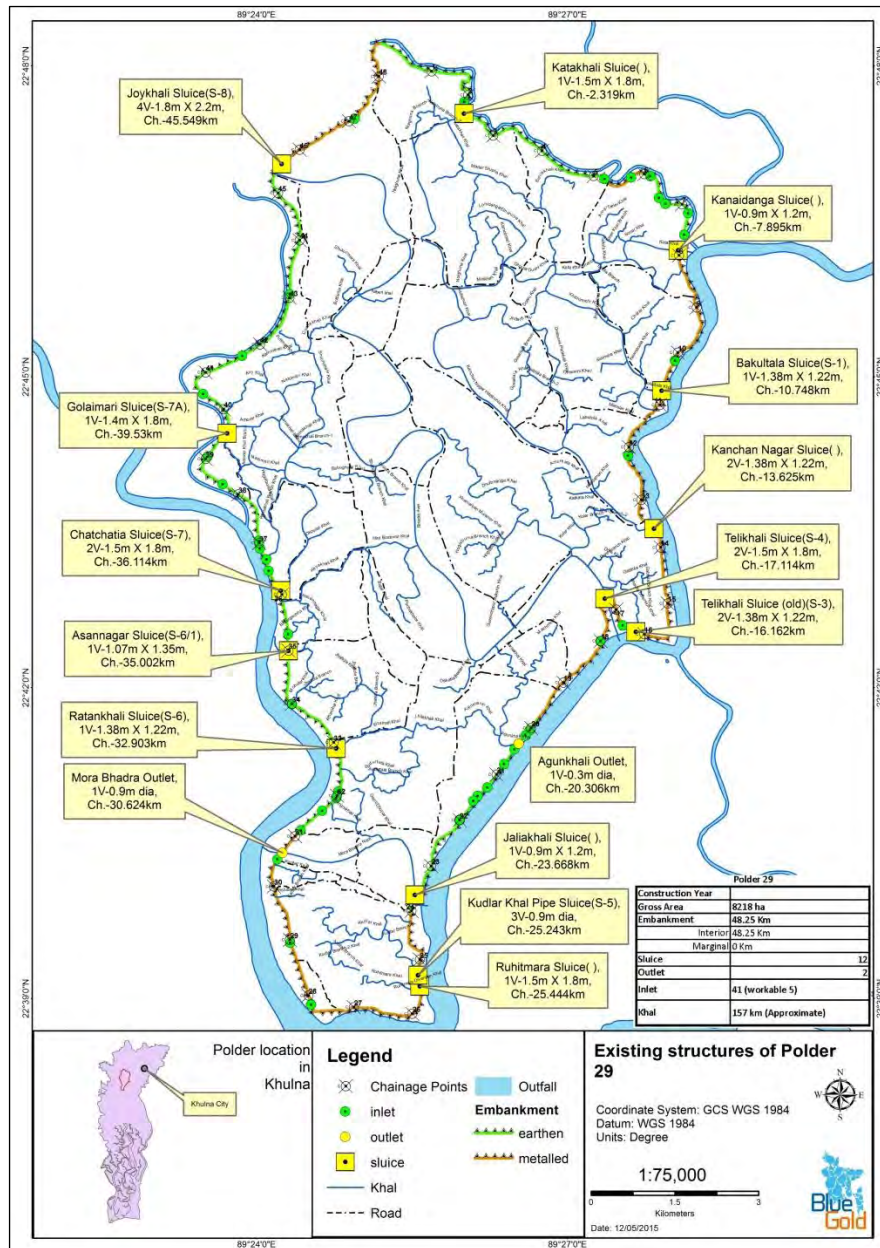


Figure 4.2: River and khal system of the study area (Source: CEGIS, 2016)

These internal water courses accelerate the flow circulation inside the polder, when needed. During low tide, tidal water level decreases through the peripheral water courses and reaches the Bay of Bengal. Both flapgates and sluice gates are present at drainage outlets for flow regulation. During high tide and heavy rainfall the flap gates remain closed to prevent the entry of outside water inside the polder but during low tide, flap gates become operational and the excess water from the polder area discharges through these gates.

Surface Water:

The surface water level data was collected (Figure 4.3 and Figure 4.4) from two BWDB stations located at Dumuria (Upper Bhadra River) and at Sutarkhali (Ghengrail River) for 30 years (1986 to 2016). Water levels during high tide vary from 1.26 to 3.58 mPWD at Dumuria and 2.78 to 4.58 mPWD at Sutarkhali. On the other hand, the low tidal water levels range from +0.04 to -2.4 mPWD at Dumuria, and to 0.35 to -1.8 mPWD at Sutarkhali.

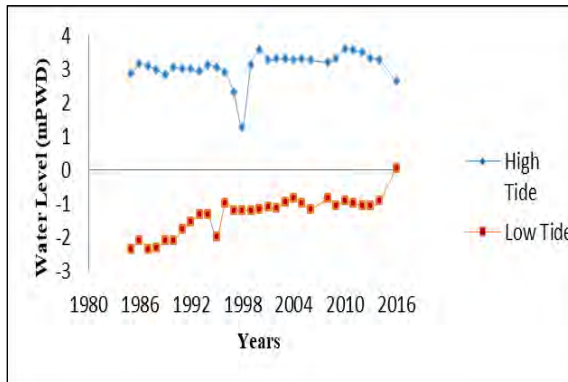


Figure 4.3: Surface water level of upper Bhadra River

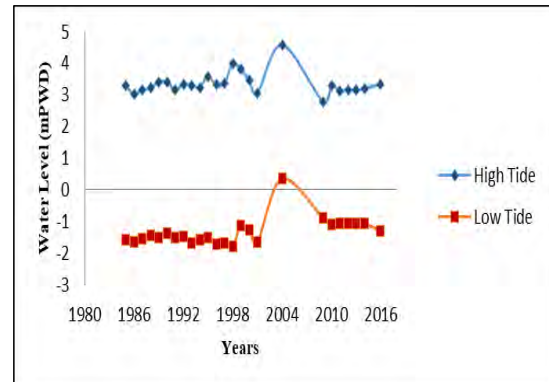


Figure 4.4: Surface water level of Ghengrail River

Ground Water:

Ground water level data has been collected and analyzed from the observation well named as KHU005 (at Dumuria). A very slight fluctuations in the GWT was found with respect to ground level. A lowest value of 4.1m in July and highest value of 2.5m in January was recorded for the year 2007 (CEGIS, 2016). In the dry season, increased dependency of the local people on ground water lowers the GWT but during monsoon, the higher availability of surface water leads to higher recharge of ground water sources (BWDB, 2015)

4.2.9 Present Status of Water Management Infrastructures

The polder is enclosed by embankments and numerous water management infrastructures such as, sluices, drainage outlets, flushing inlets to serve drainage and flushing purpose, ensure sustainable management, optimal use and equitable sharing of water resources.

Embankment

The Embankment length is 49 km having a top width ranges between 3.7 m to 3.8m. It provides protection against salinity intrusion and tidal surges. On both riverside and countryside existing side slopes varies from 2.15m to 2.25m. The existing condition of the embankment is good in most portions but deteriorates at two locations such as Baro aria and Jaliakhali due to erosion. In 2014, at Jaliakhali one retired embankment has already been constructed by the local community.

Water Control Structures and Culverts

BWDB constructed 14 drainage sluices and 2 drainage outlet within the polder for maintaining drainage operation. Among these, 6 sluice gates have been repaired and 5 others have been constructed under IPSWAM project from 2003 to 2011. But repairing needs to be done for some structures. The gate at Golaimari, Jaliakhali, Ramkhalikhal were either missing or found non-operating due to damages of the wheels and shafts which were used to hoisting the gates. In addition to this, river bed raised up due to continuous siltation which caused sluice gate at Golaimari and Telikhali non-functional. Chatchatia gate cannot flushing out water rapidly for the difference between high canal/river bed and low elevation of agricultural land (Bakuluzzaman and Naher, 2015).

Present Status of Drainage Khals

Over the years, siltation, topsoil erosion and other land filling activities have reduced number of water courses within the polder gradually. As a result, the present condition of most of the internal drainage khals is completely disagreeable. The condition of AroKhal, Asannagar Khal, Golaimarikhali and Kata khal is the worst among all. Most of the area of AroKhal is covered with grass and water course is almost non-existent. During dry season the khals are usually blocked off by the sluice gates to prevent the entry saline water, whereas in wet season, these khals are used to drain the surplus water out of the polder. But due to presence of damaged sluice gates at khal openings some khals carry saline water during dry season. But in recent years increased siltation raised the bed level of khals which hampers the flow circulation inside the polder. Only Mora Bhadra Khal was identified as a perennial water course. Approximately 35% khals (Telikhali khal, Bokultola khal etc.) carry reduced amount flow, varying from 2~5 feet deep. During dry season, around 30% number of khals at Mora Bhadra, Asannagar, Ramakhali

etc. carry water upto 5~7 feet depth. Almost 20% khals (Arokhal, Golaimarikhali, Kata khal etc) inside the polder carry no water during dry season.

4.3 Field Visit

A visit to the study area (Polder 29) had been made on 9 August, 2016. Kanchannagar sluice is selected as water level calibration location because it receives water from maximum area of the watershed. This sluice was visited on the very first day in the morning and the rest of the sluices were visited on the second day. The pictures of internal drainage channels and sluice gates of some locations are given in Appendix A. From the field study the following results are obtained which contributed in this study for further understanding and analysis:

- It was observed that the gates (flap and sluice) at Kanchannagar sluice were operational and a greater amount of flow pass through this sluices to Bhadra River. The local people gave an idea about the rise of water level there and according to them during high tide, water level rises upto 3.42 m PwD of the total height of flap gate.
- During a visit to Bokultala khal, it was found that a new gate was constructed inside the khal which was previously broken and now working as a well-functioning gate. Approximately 1.5 to 2 cumec water passes through this gate according to BWDB engineer.
- At Telikhali old, there was a two vent sluice of which only one was operational. Due to, damages of the wheels and shafts other one remained non operational
- A good amount of flow was observed at Telikhali-New due to proper functioning of two flap gates and 2-vent sluices..
- The gates at Golaimari, Ramkhali khal were either missing or found non-operating.
- Some debris as well as water hyacinths were detected at the gate openings of Keyakhali and Asannagar khal which hamper the natural flow through the structures.

- The present condition of some drainage khals i.e Arokhal, Asannagar Khal, Golaimarikhhal and Kata khal was found completely disagreeable.
- Width of the water course of Kata khal has decreased from a very high range of 30 to 40feet to 2feet. The rest of the outlets could not be visited because of the condition of newly constructed roads and transportation inconveniences.

4.4 Data collection

4.4.1 Digital Elevation data

Digital Elevation Model (DEM) data of 30m resolution (Figure 4.5) for year 2004 has been collected from Shuttle Radar Topography Mission (SRTM) website (<http://srtm.csi.cgiar.org>). The SRTM DEM was found to contain more surface detail and roughness than the TOPO DEM. It has been produced using radar images gathered from NASA's shuttle.

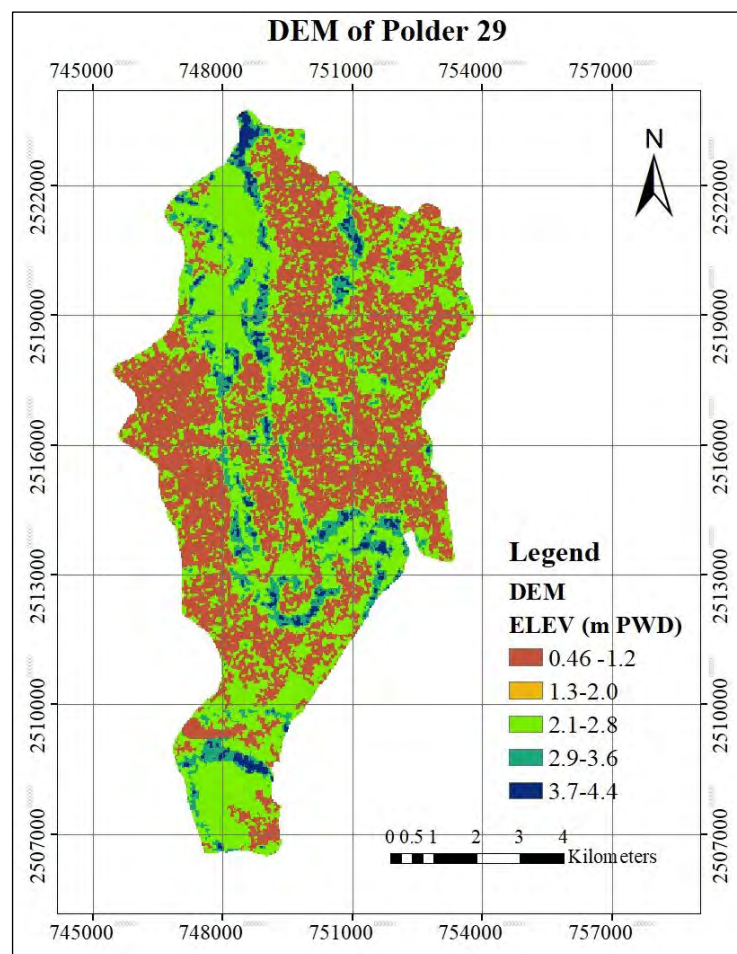


Figure 4.5: DEM of Polder 29 (Source: Weydah, 2017)

4.4.2 Rainfall Data

Rainfall data has been collected from BMD for frequency analysis to determine design rainfall. Though the rainfall station does not lie inside the polder area, this is the only nearby station. Due to this reason rainfall data collected from this station has been used in this research. For calibration and validation of the model, GPM data at 30 minutes interval has been retrieved from US Geological Survey GPM satellite for more precision.

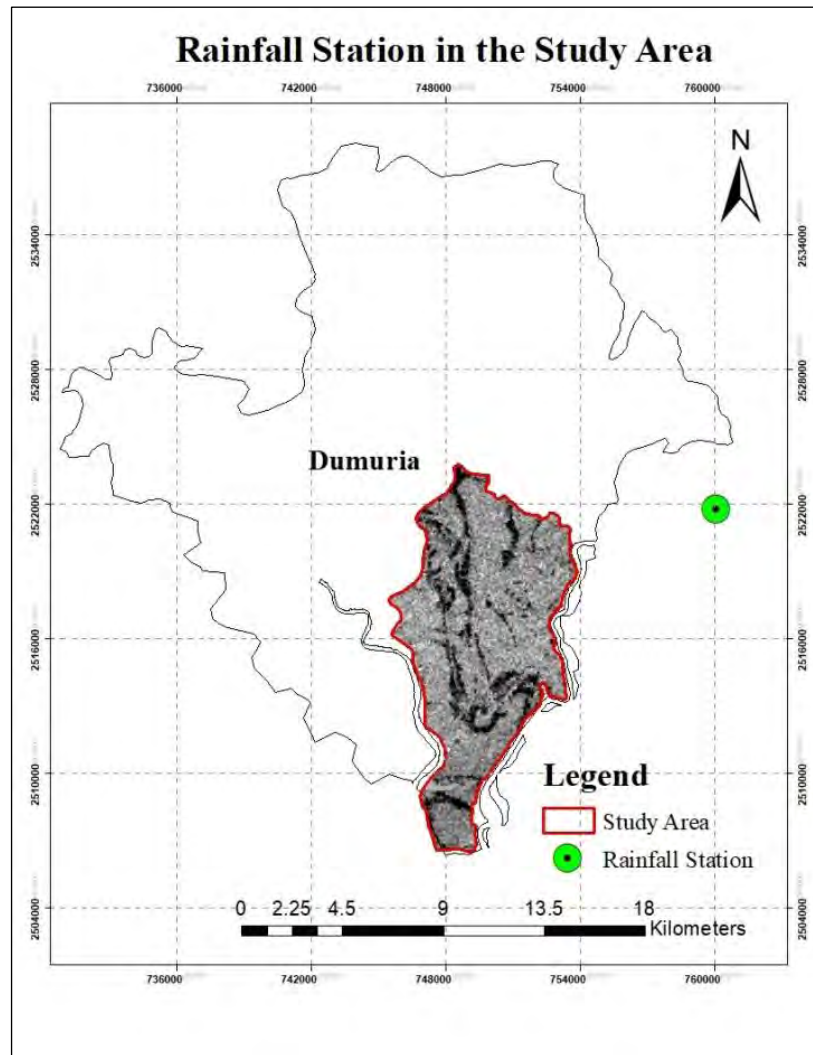


Figure 4.6: BMD station in the study area

The GPM is a joint US–Japan space programme designed to measure tropical rainfall worldwide with regular spatial and temporal distribution. This programme is established and regulated by National Aeronautics and Space Administration (NASA). The GPM Core Observatory design is an extension of TRMM’s highly successful rain-sensing package which provides global precipitation measurements with improved accuracy, coverage and dynamic range for studying precipitation characteristics. GPM is also

expected to improve weather and precipitation forecasts through assimilation of instantaneous precipitation information. Relative to TRMM, the enhanced measurement and sampling capabilities of GPM offers many advanced science contributions and societal measure.

4.4.3 Cross Section Data

Canal cross sections of the study area have been collected from Morphology Department of Bangladesh Water Development Board for the year 2015. These natural channels mainly conveyed the stormwater of the area. The canal network with outfall locations and sample cross sections are shown in the following Figure 4.7 and Figure 4.8

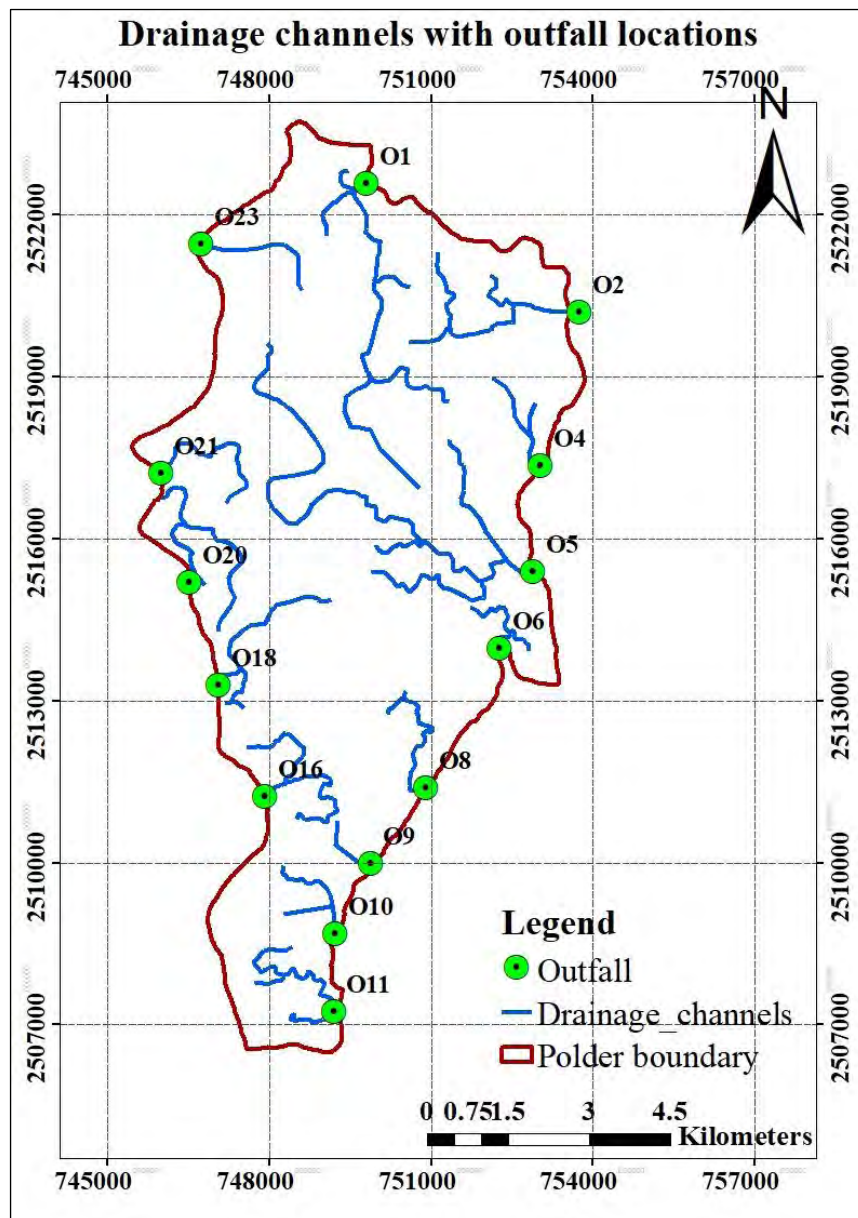


Figure 4.7: Canal network and outfall locations of the study area

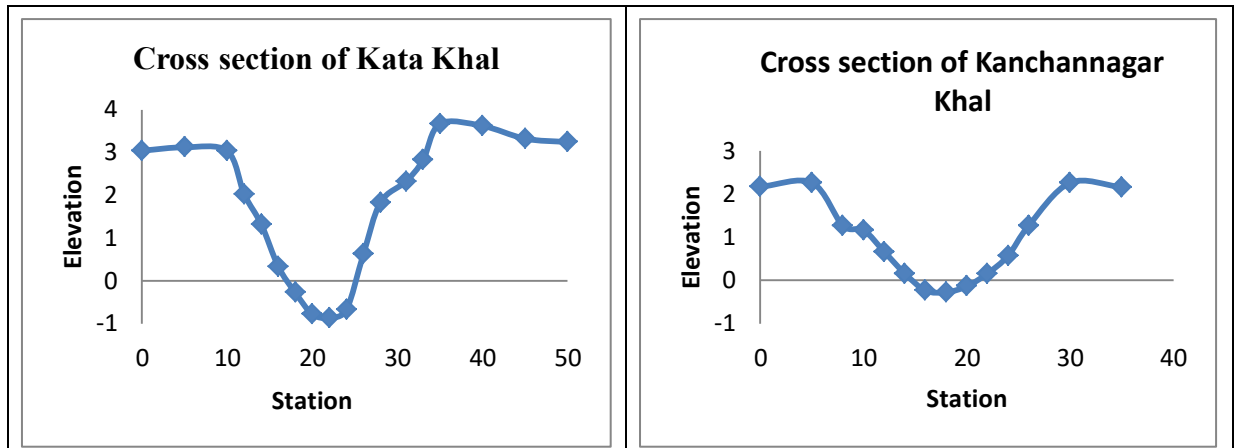


Figure 4.8: Canal cross section of Kata khal and Kanchannagar khal

4.4.4 Land Use Map

Land use map is required for calculation of % imperviousness of each subcatchment. Six types of land use are taken into account in this study. They are: Agricultural land, Settlement, Road, Gher (Shrimp culture), Pond and River or Canal. The presented land use map is collected from CEGIS which was prepared in 2015 for analyzing environmental impact assessment of Polder 29. Land use map is shown in the Figure 4.9

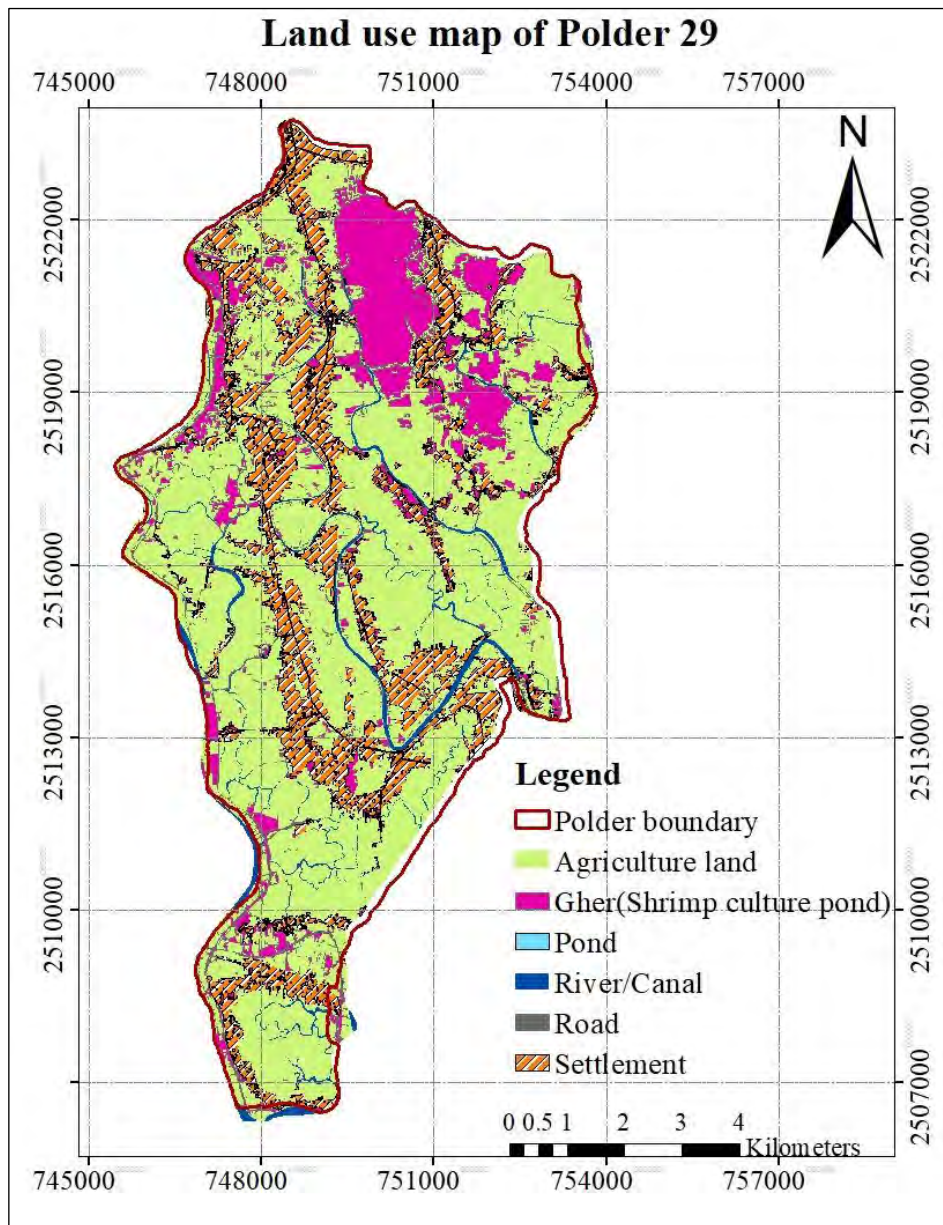


Figure 4.9: Land use map of the study area

4.4.5 Water Level Data

Water level data of the stations SW 28 and SW29 (1986-2016) has been collected from BWDB for frequency analysis. The location of these stations is shown in Figure 4.10 Kanchannagar sluice was selected as the calibration location as it receives water from major portion of the catchment. A surveyor was appointed under Nuffic-Niche BGD-155 Project to measure water level both inside and outside of Kanchannagar sluice. The data was collected during August and September, 2016. The summary of data collection is presented in Table 4.1.

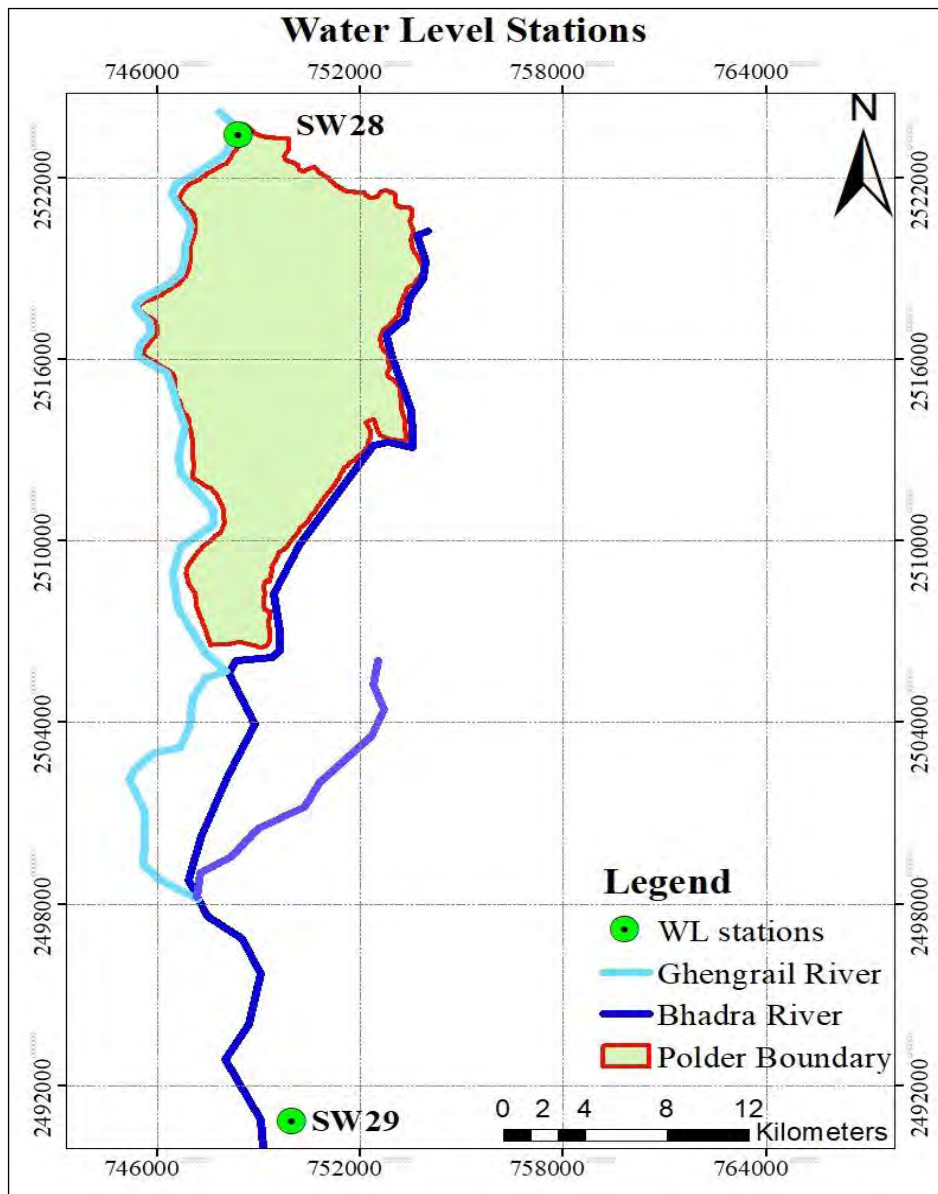


Figure 4.10: Location of water level stations

Table 4.1 Summary of Collected Data

Data type	Station/ Location	Source	Time period
Digital elevation model (DEM)	Polder 29	Shuttle Radar Topography Mission (SRTM)	2003
Daily rainfall	Khulna	Bangladesh Meteorological Department (BMD)	1948-2016
3-hourly rainfall	Khulna	Bangladesh Meteorological Department (BMD)	2016
GPM data(30-minutes interval)	Polder 29	US Geological Survey GPMsatellite	August and September, 2016
Land use map	Polder 29	CEGIS	2015
Cross section of drainage channels	Polder 29	BWDB	2015
Water level	Both inside and outside of Kanchannagar sluice	Nuffic-NICHE BGD 155 Project	August and September, 2016
Water level	Dumuria (SW 28) and Sutarkhali (SW 29)	BWDB	1986-2016

The overall methodology of the study is shown in the following Figure 4.11

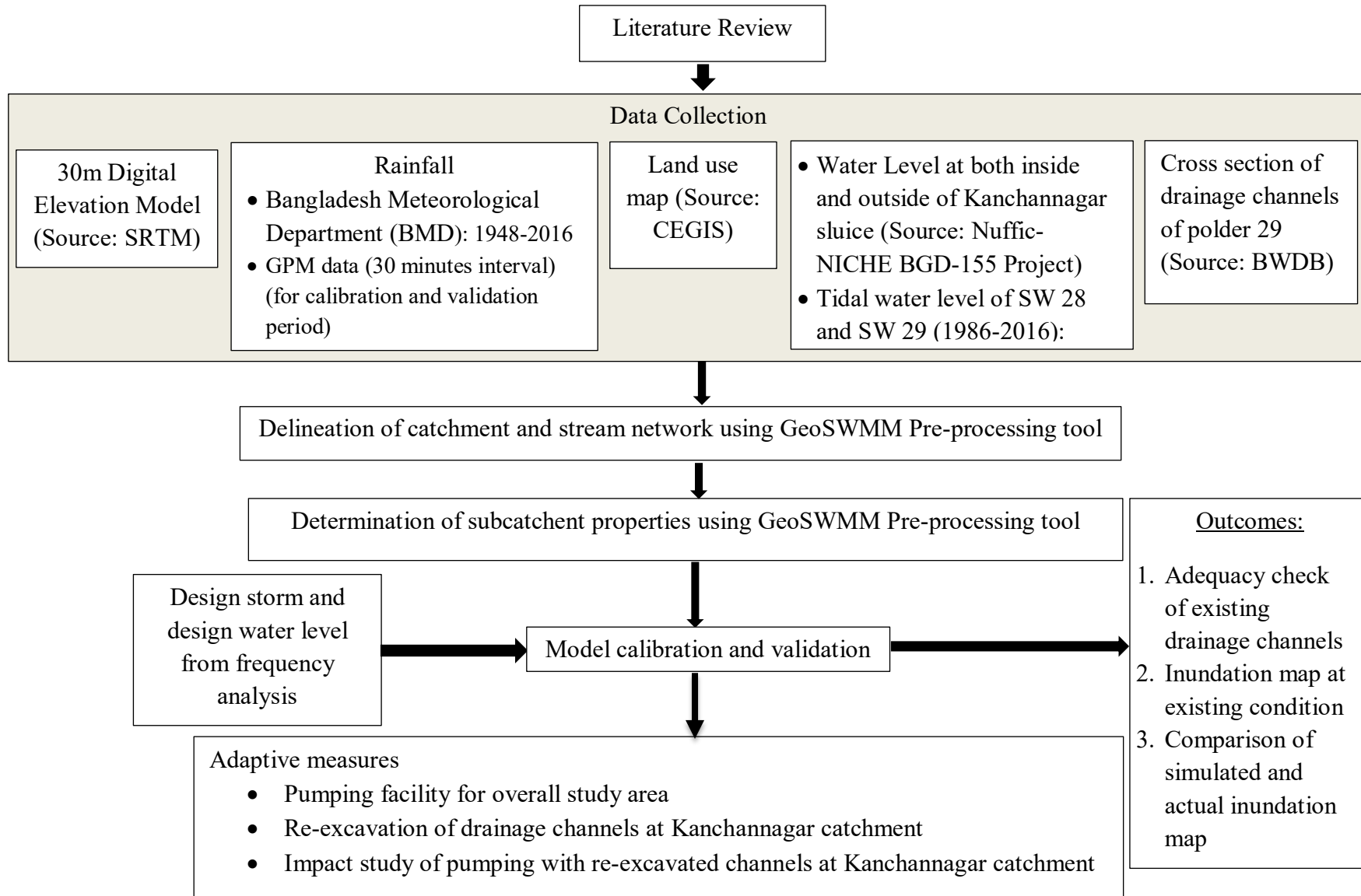


Figure 4.11: Overall methodology of the study

4.5 Data Analysis

4.5.1 Analysis of Rainfall Data

Historical rainfall data (1948-2016) has been collected from BMD. Frequency analysis is performed for fitting theoretical probability distributions to given dataset. In this research, four distributions are considered. They are Extreme Value I (EVI) Distribution, Normal Distribution, Log-Normal Distribution and Log-Pearson Type III (LP3) Distribution. These distributions measure the properties of hydrologic data. Chi square test is also conducted in this study because minimum value of chi square calculated among the four methods would represent the suitable distribution. The frequency analysis has been done for annual maximum value of daily rainfall for the period 1948-2016. Here, 1-day maximum annual rainfall was selected for the analysis in spite of 3-day or 5 -day consecutive rainfall because in drainage modeling higher peak can be obtained while considering the short period of rainfall. In this case, rainfall intensity is higher compared to the continuous approach.

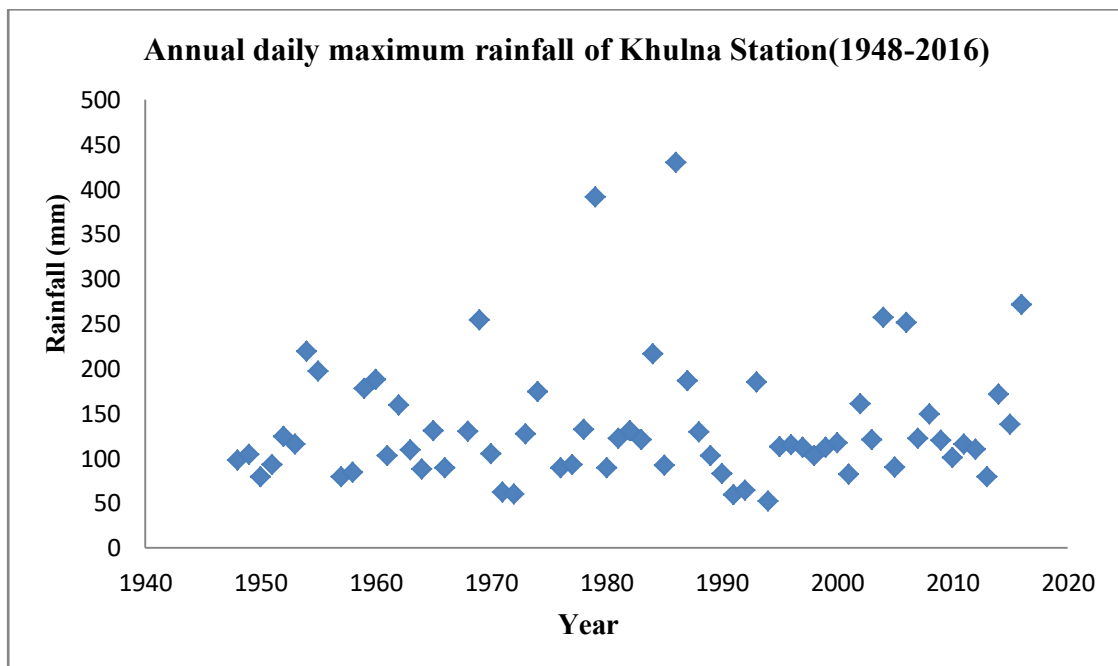


Figure 4.12: Annual maximum value of rainfall at Khulna station (1948-2016)

4.5.2 Analysis of Water Level Data

The study area is surrounded by the lower Bhadra river in the east and Ghengrail river in the west. These two rivers are directly fed by the oceanic tides. So, the delta area is subjected to diurnal tide. Water level data for 30 years (1986-2016) was collected from two tidal water level stations located at Dumuria, SW28 (Upper Bhadra River) and at Sutarkhali, SW29 (Ghengrail River). The runoff from the study area mainly flows out to these rivers. Frequency analysis was performed for annual maximum values of high tide level and minimum low tide level of these two rivers using Gumble distribution method as this is one of the most suitable methods to determine the design water level. This analysis is necessary for this study because it determines the design high tide and low tide water level for various return periods and later they are used as a d/s boundary condition while simulating the design storm scenario. Tidal water level variations for the two stations are shown in the following Figure 4.13 and Figure 4.14

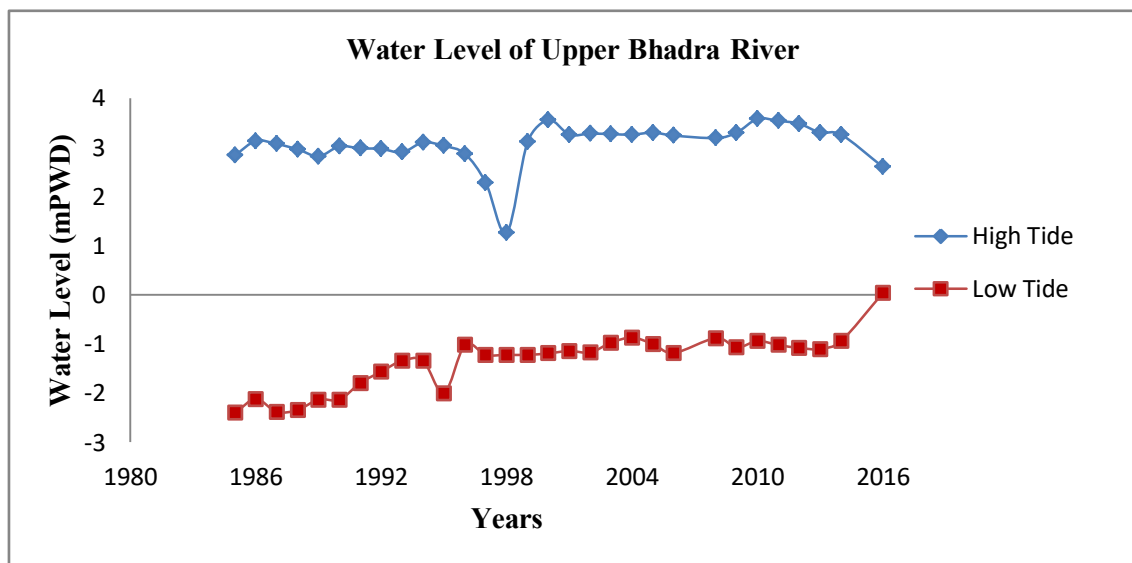


Figure 4.13: Annual maximum WL for high tide and minimum WL for low tide of upper Bhadra River

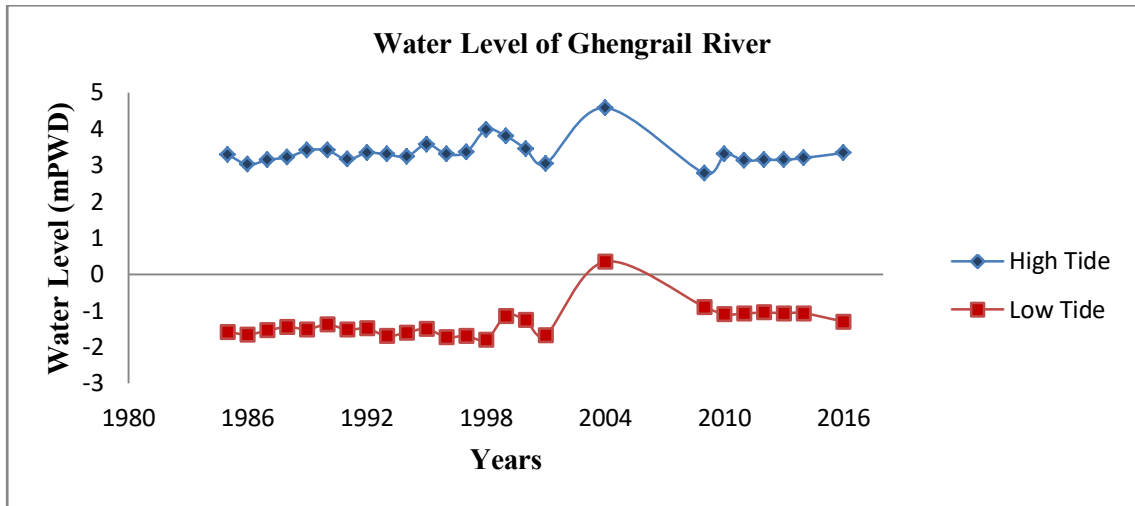


Figure 4.14: Annual maximum WL for high tide and minimum WL for low tide of Ghengrail River

From the above figures, it is observed that the water levels during high tide varies from 1.26 to 3.58mPWD at Dumuria and 2.78 to 4.58 mPWD at Sutarkhali. On the other hand, the low tidal water levels range from +0.04 to -2.4 mPWD at Dumuria, and to +0.35 to -1.8 mPWD at Sutarkhali.

4.6 Model Development

Digitizing Existing Streams

Existing stream network is digitized from Google Earth for better examination of the portrayed streams with the actual streams of the study area. This network of the study area is given in the model as Conduit layer. The conduits are open channels, for which cross-sections are given as irregular transects.

Watershed Delineation

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

A catchment may contain smaller watersheds, called subcatchments or subwatersheds. The outlet's points define the shape and spatial extent of the subcatchment boundaries. The number of the subcatchments is equal to the number of outlets. The stream network

that conveys water from a number of subcatchments to a common outlet looks like the veins of a leaf, with the position of the outlet at the end of the midrib (Fig. 4.16).

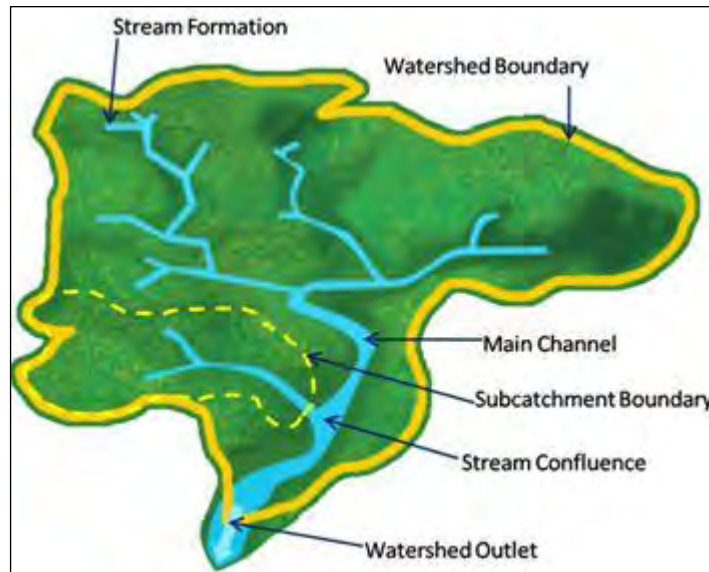


Figure 4.15: A schematic diagram of watershed components

The first step to set-up a Geo-SWMM model is watershed delineation which divides the total study area into small sub catchments according to the understanding of the drainage system and flow direction of storm-water. The tool provides the utmost flexibility in assigning desired outlets (or pour points) and generating their contributory watershed areas in a GeoSWMM project. The Watershed Delineation Tool works with user supplied DEM data (in raster or grid format) to segment the model catchment boundary into multiple subcatchments. This process is known as automatic delineation. The accuracy of catchment delineation, therefore, depends on the resolution (e.g. grid cell size) of the DEM data. The catchment delineation process involves a set of successive spatial operations in ArcGIS that require a clear understanding. The computation techniques applied by the Watershed Delineation Tool are similar to that of the Hydrology toolset of ArcGIS 10.2. The basic operations undertaken by the tool are illustrated in a flow chart as shown in the following Figure 4.16

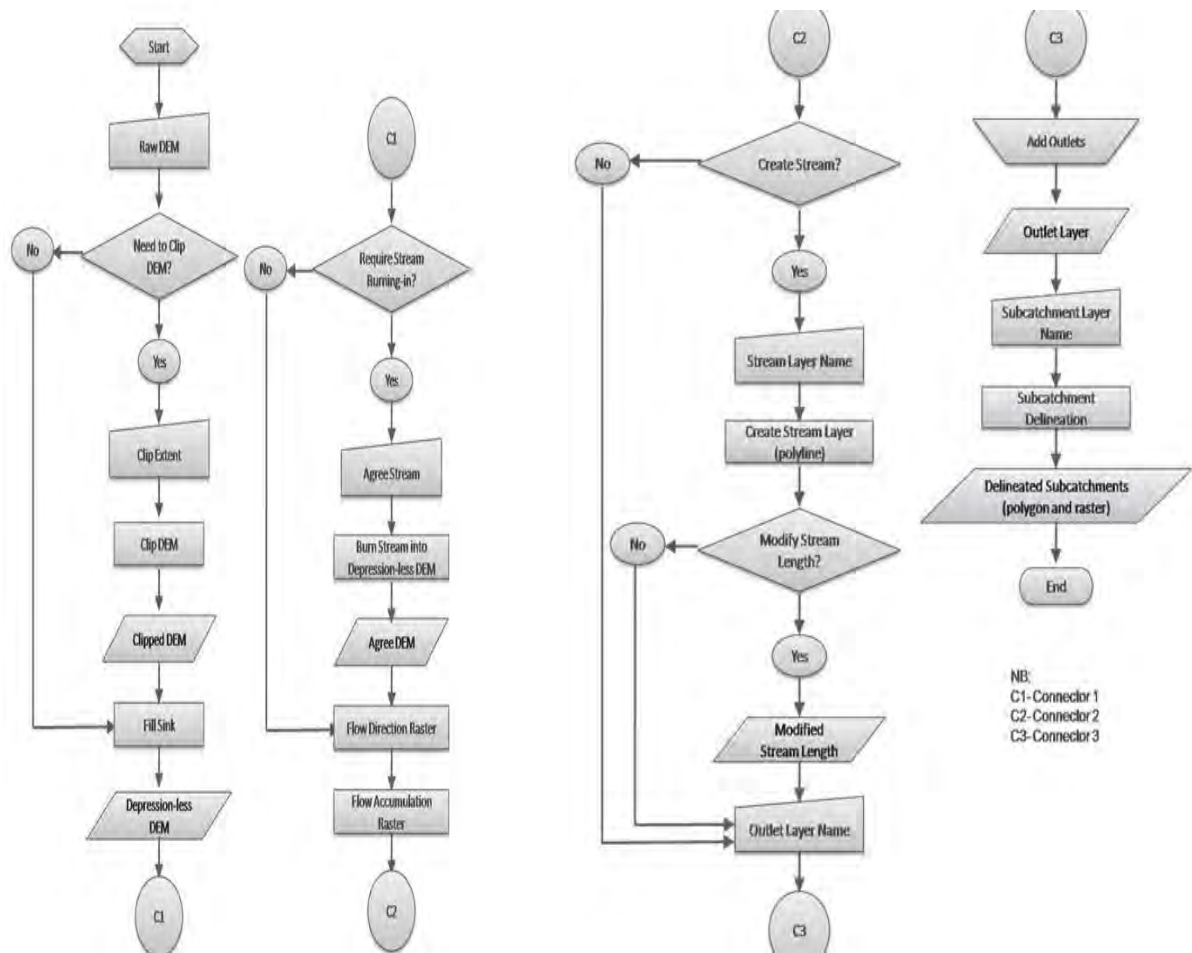


Figure 4.16: Flow chart of watershed delineation process (Source: GeoSWMM user manual)

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

The tool analyzes and processes DEM data applying an 8-pour point approach in order to determine the direction of flow. The Flow Direction Raster generated in the interim period contains the values that represent the direction of water movement. However, in some terrain data users may need to define flow paths exactly along the existing stream

layout. In such case, the "burning-in" technique is applied with the available stream layer in the DEM data. The user specified vector stream layer is called AGREE stream and the resulting DEM is called AGREE DEM. AGREE is a surface reconditioning system for DEMs (Hellweger, 1997). The system adjusts the surface elevation of the DEM to be consistent with the vector stream (AGREE stream). The burning-in technique allows the user to abruptly lower the cell elevations by a fixed amount along the stream line in the target DEM. This process is also known as DEM reconditioning. If the flow direction raster is produced using the AGREE DEM then the flow path automatically becomes aligned with the AGREE stream layer.

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

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

In this work, the canal network is previously known, so stream formation is not necessary. The stream is burnt in the DEM so that all the canals are included in the operation. In the next step, outlets are created using automatically generated outlet option and outlet number is adjusted with study area using Google Earth. These are incorporated in the model as „Junction“ and „Outfall“. Finally, the catchments are delineated on the basis of these junctions. The subcatchments are shown in the following Figure 4.17

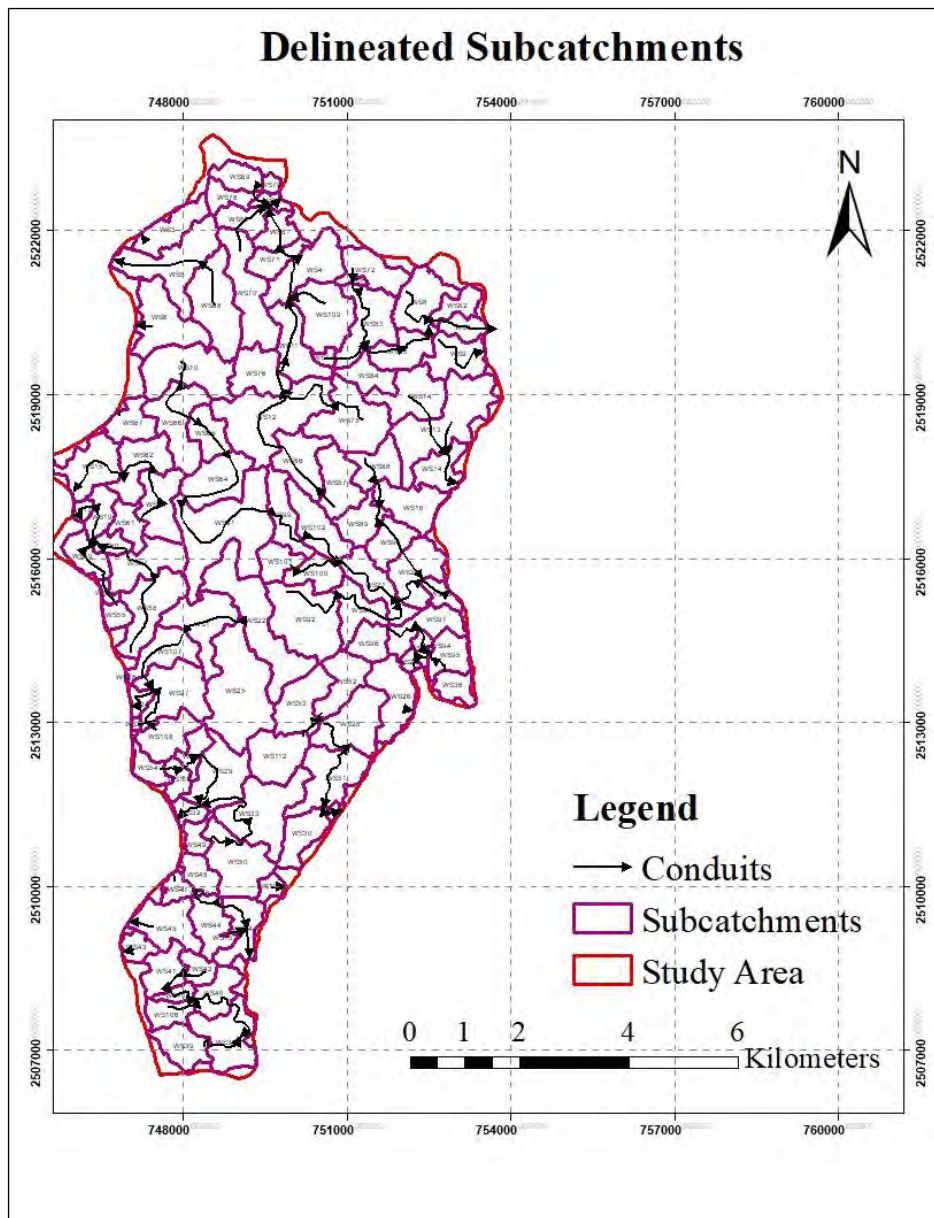


Figure 4.17: Watershed delineation of the study area

Calculation of Subcatchment Properties:

Slope Calculation

The "Slope Calculation" tool works with a Digital Elevation Model (DEM) dataset to find the average surface slope of the model subcatchments. The values are calculated in percentage rise. The characteristics of the required input features in the Slope Calculation tool are described below:

DEM Raster

This is a raster dataset that contains elevation values of the model subcatchment area.

Subcatchment Layer

This is a polygon feature layer that represents the subcatchments.

Subcatchment Slope Field

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

The calculated average slope of the subcatchments is shown in the Figure 4.18

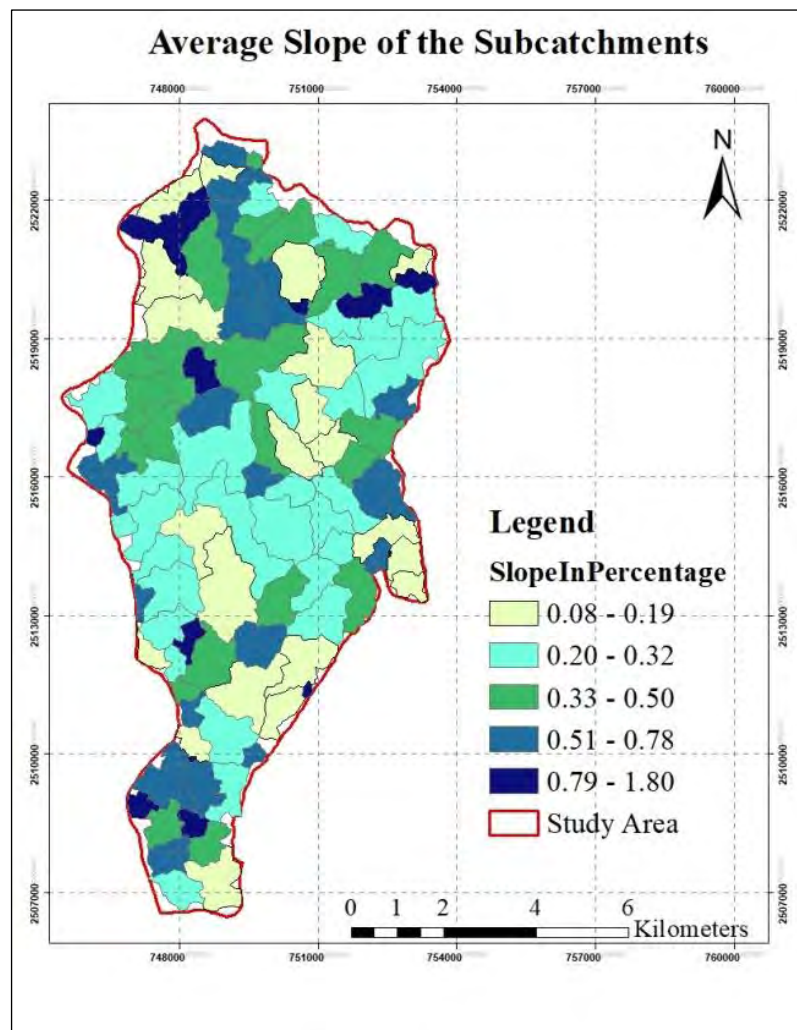


Figure 4.18: Average slope (%) of the subcatchments

Imperviousness Calculation:

The impervious parameter describes the percentage of total impervious surfaces compared to the total area of a subcatchment. It is often used as a calibration parameter (Choi and Ball, 2002) as it is not always possible to physically define it due to the fact

that many surfaces are in reality partially impervious. In this study, land use map and google earth images were used for subcatchment parameterization.

The "Imperviousness Calculation" tool works with a grid or a polygon feature layer data set that represents surface imperviousness in order to find the area-weighted average surface imperviousness of the model subcatchments. In an imperviousness raster, each cell value represents the fraction (or the percentage) of impervious area within that cell.

The characteristics of the required input features in the imperviousness calculation tool are described below:

Imperviousness Layer

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

Imperviousness Field Name

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

Subcatchment Layer

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

Subcatchment Imperviousness Field

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

Figure 4.19 represents imperviousness values for each subcatchment. It is one of the most sensitive parameters that controls the discharge and water level. For this reason, after the calculation these values are verified with the actual google earth image.

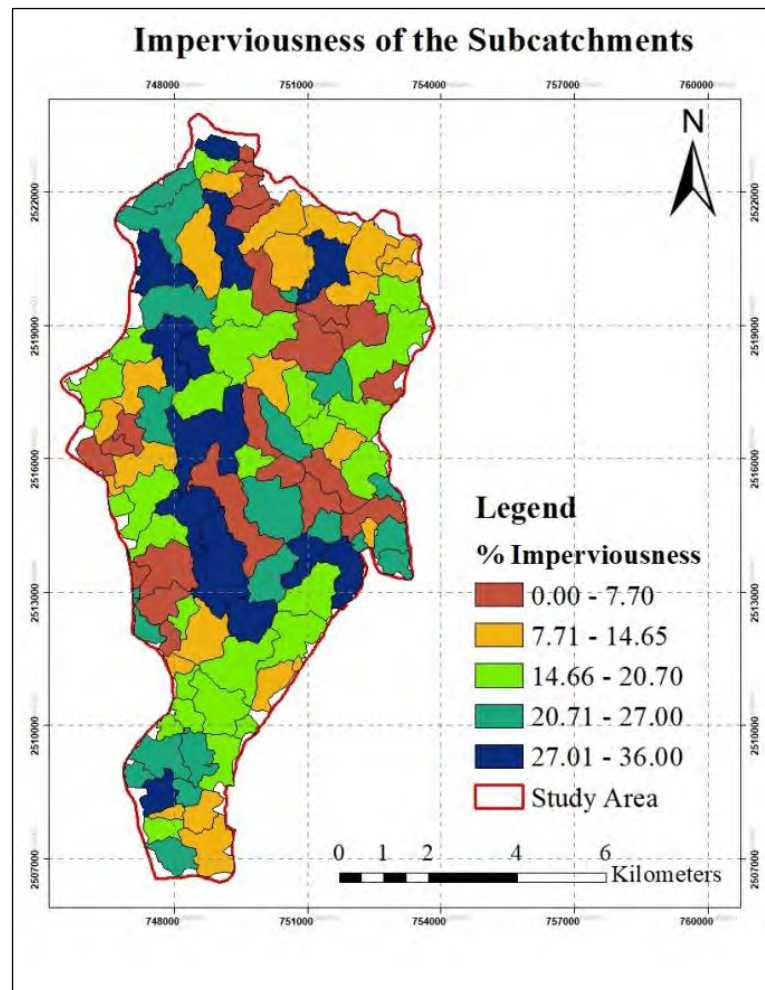


Figure 4.19: Imperviousness of the subcatchments

Subcatchment Characteristic Width Calculation:

The characteristic width is a special hydrologic parameter for a subcatchment that affects the time of concentration and determines the shape of the runoff hydrograph. It depends on the characteristics of the overland flow length and the shape of the subcatchment. If the overland flow line is represented by the physical width of an ideal rectangular shaped subcatchment, then the characteristic width will be the physical length of that rectangle (Fig. 4.21). However, in reality, subcatchments are shaped like non-uniform polygons and therefore the width cannot be estimated in a straightforward manner (Fig. 4.22). It is one of the key **calibration parameters** in a SWMM model because it can significantly alter the runoff hydrograph properties. According to Rossman (2010) and Gironas et al. (2009), the width parameter can be calculated by dividing the subcatchment area by the length of the longest overland flow path in the area. In case several flow paths exist, their maximum lengths should be averaged.

Channelized flow should never be included in this flow length, which typically reduces the maximum flow length to less than 200 meters

The "Subcatchment Characteristic Width Calculation" tool estimates the characteristic width property for the model subcatchments. It primarily works with a subcatchment layer and a conduit layer (or stream layer) to measure the characteristic width.

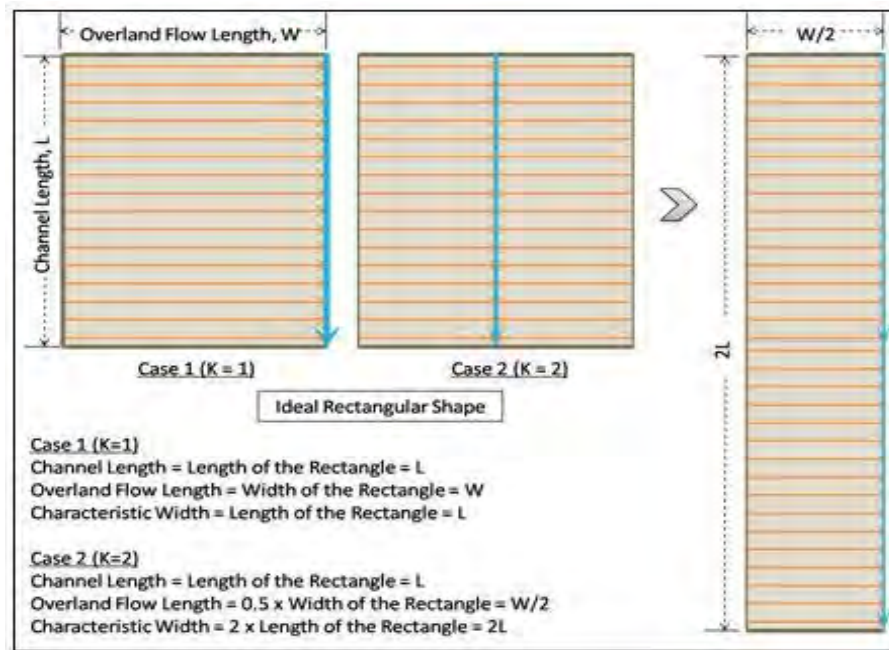


Figure 4.20: Characteristic width of an ideal rectangular shaped subcatchment

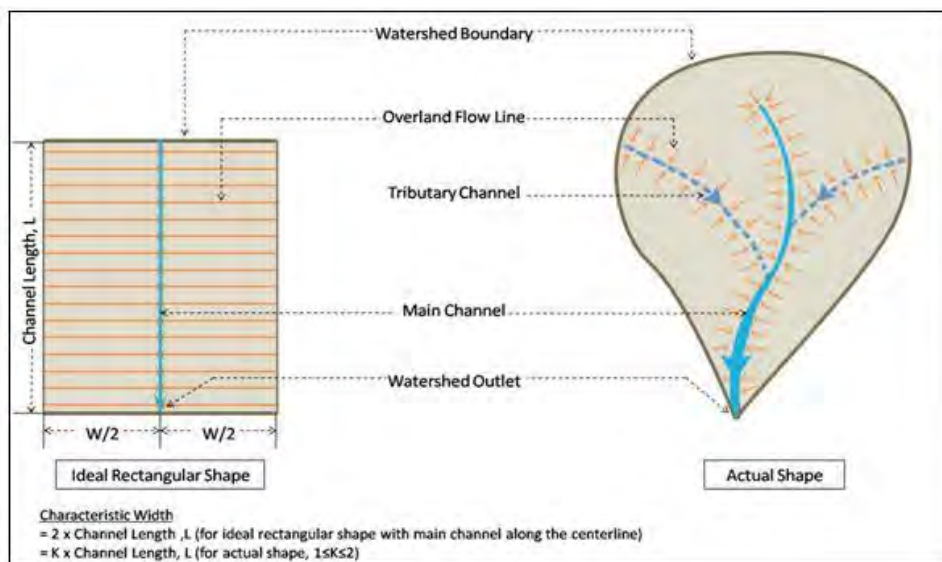


Fig.4.21: Characteristic width for a non-uniform shaped subcatchment

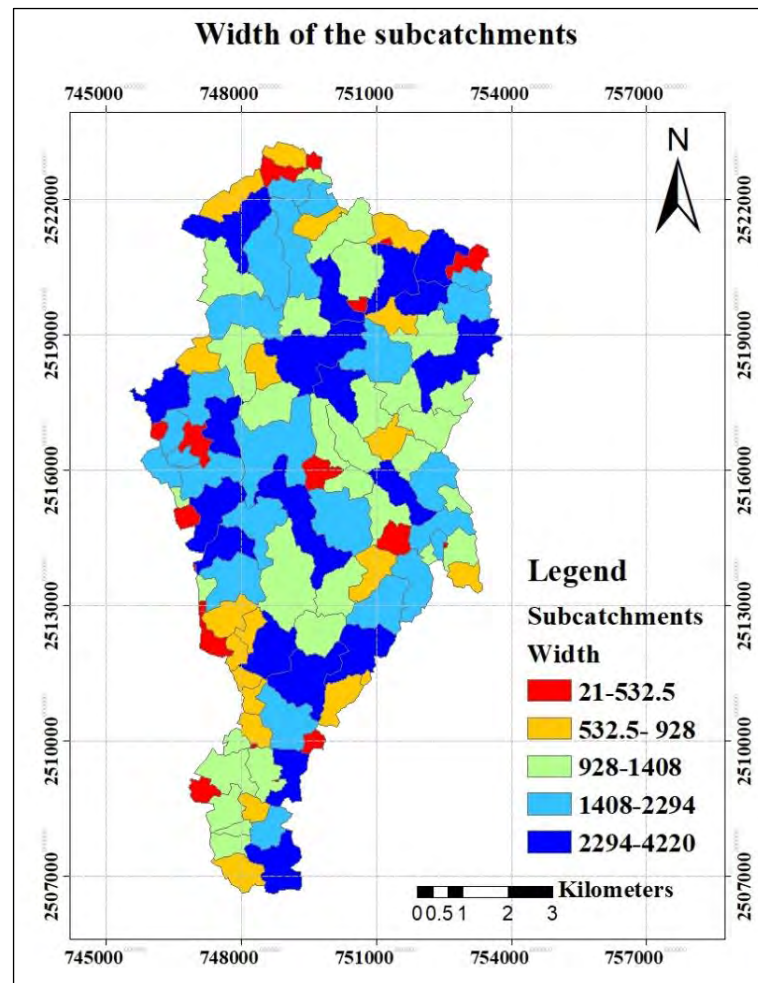


Figure 4.22: Characteristic width of the subcatchments

Land Use Reclassification

Land use data is given as a polygon feature format and an additional column is added in the attribute field containing land use codes. Lands are classified into six categories. They are: settlement, agricultural land, roads, Gher, Pond, Canal/ River. After the land use reclassification task is complete, the values of the Manning's roughness (N-value) and the depth of maximum depression storage (D- value) are assigned to the reclassified land use types.

Depression Storage

SWMM treats the pervious and impervious parts of a subcatchment separately. And thus both may be given independent values of depth of depression storage. One can also define a portion of the impervious area to have no depression storage at all. This could be realistic on steep roofs, for example. The SWMM User's Manual (Rossman, 2010) suggests some literature values for the depression storage. For impervious areas, the

values range from 1.3 to 2.5mm and for lawns from 2.5 to 5.1mm. The highest value is given for forest litter (7.6 mm). These values are not very exact and depression storage actually is one of the common calibration parameters used for SWMM parameterization (Choi and Ball, 2002)

Based on the values above, the depression storage for subcatchments were set to around 2.54- 3.81 mm for impervious subcatchment fraction and to 1.27 to 7.62 mm for pervious areas.

Manning's roughness co-efficient n for overland flow

Manning's n determination for channels, floodplains, and watershed surfaces by site visits, field experience, and photographs remains a widely used approach by hydrologists (Barnes, 1967; Aldridge and Garrett, 1973; Arcement and Schneider, 1989; Sellin et al., 2003) The selection of Manning's n should not be treated solely as an intuitive process but rather with engineering judgment applied in a standardized set of procedures (Arcement and Schneider, 1989; Wu et al., 1999; Tsihrintzis et al., 2001; Jain et al., 2004). It has been proposed that better estimation of Manning's n would improve the performance of hydrologic models Furthermore, estimating Manning's n is subjective because the surface roughness is dependent on the surface granular structure, complex interactions due to the elevation change, surface irregularity, flow depth, vegetation density, scale, and obstructions (Arcement and Schneider, 1989 ; Jain et al., 2004 ; Vieux, B. E. 2001). Indeed, some have proposed the accurate estimation of Manning's n is impractical because of its empirical nature and approximate estimation techniques (Kidson et al., 2006)

In Geo-SWMM model, this value differs for impervious and pervious areas. For roads, the N-value was taken 0.013. This value is exactly same as ordinary concrete lining according to the usual manual of Geo-SWMM. For settlement with vegetation, 0.19 was used as N-value for better approximation.

For pervious areas, The N-value ranges between 0.025 to 0.24. For river / canal the value was considered as lowest, The other values were a compromise between the values for woods with light underbrush (0.4), woods with dense underbrass (0.8) and dense grass (0.24) (Rossman, 2010)

Infiltration

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

- Dry soils (with little or no vegetation):
 - Sandy soils: 5 in/hr
 - Loam soils: 3 in/hr
 - Clay soils: 1 in/hr
- Dry soils (with dense vegetation): Multiply values of dry soils by 2
- Moist soils:
 - Soils which have drained but not dried out (field capacity): Divide values obtained from dry soils with both the cases by 3.
 - Soils close to saturation: Choose value close to min. infiltration rate.
 - Soils which have partially dried out: Divide values from A and B by 1.5 - 2.5.
 - Min. Infil. Rate: Minimum infiltration rate on the Horton curve (in/hr or mm/hr). equivalent to the soil's saturated hydraulic conductivity
 - Decay Const.: Infiltration rate decay constant for the Horton curve (1/hours). Typical values range between 2 and 7.
 - Drying Time: Time in days for a fully saturated soil to dry completely. Typical values range from 2 to 14 days.
 - Max. Infil. Vol.: Maximum infiltration volume possible (inches or mm, 0 if not applicable). It can be estimated as the difference between a soil's porosity and its wilting point times the depth of the infiltration zone.

For each of the subcatchment, these values were assigned considering the land use properties of the study area.

CHAPTER FIVE

RESULTS AND DISCUSSION

5.1 General:

After calibration and validation of the model work, the model is used for checking the adequacy of existing drainage channels and developing the inundation depth for design rainfall condition. It is clearly observed that this inundation mainly occurred due to presence of heavy rainfall along with the closed flap gates during high tide. The excess water in the drainage channel could not pass through the flap gates because of high tide level in the outside river. To mitigate and improvement of drainage congestion problem, various structural and non-structural methods are practicing all over the world. In this study, to prevent this inundation problem, pumping facility has been designed for 1-day maximum annual rainfall of 10 year return period as an improvement option.

5.2 Water Level Calibration and Validation:

Rainfall and Boundary condition:

30-minutes precipitation data was retrieved from US Geological Survey GPM satellite for more precision. The model was run for single storm event as the information about opening and closing time of flapgate at kanchannagar outlet was not properly known for a continuous basis. The model is calibrated for 30 August, 2 and 5 September of 2016 and validated for 21 and 27 September, 2016. The boundary (O5) and calibration location (J182) are depicted in the Figure 5.1. Precipitation data and water level data for d/s boundary condition are shown in Figure 5.2 and Figure 5.3 respectively.

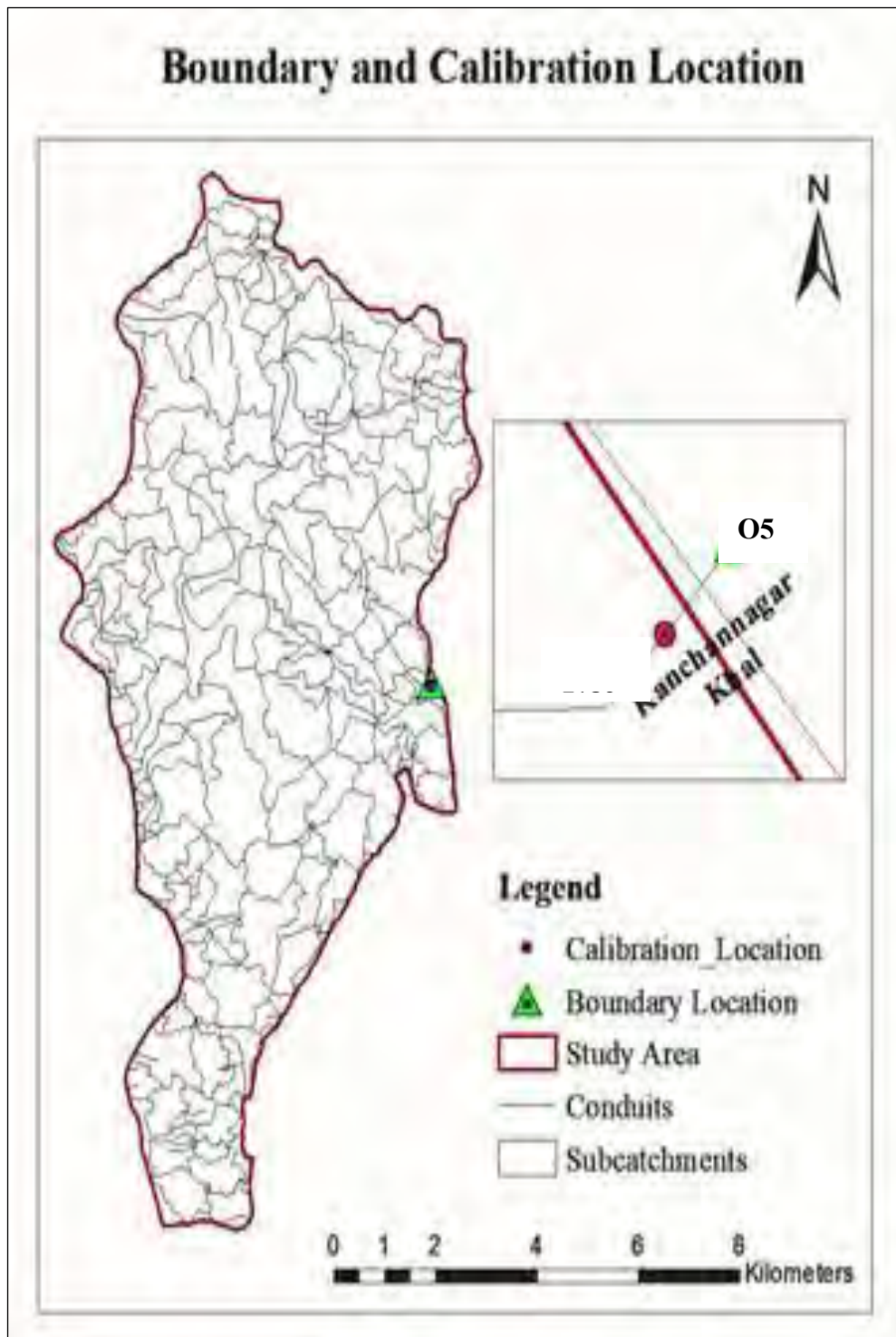


Figure 5.1: Boundary and calibration location of Polder 29

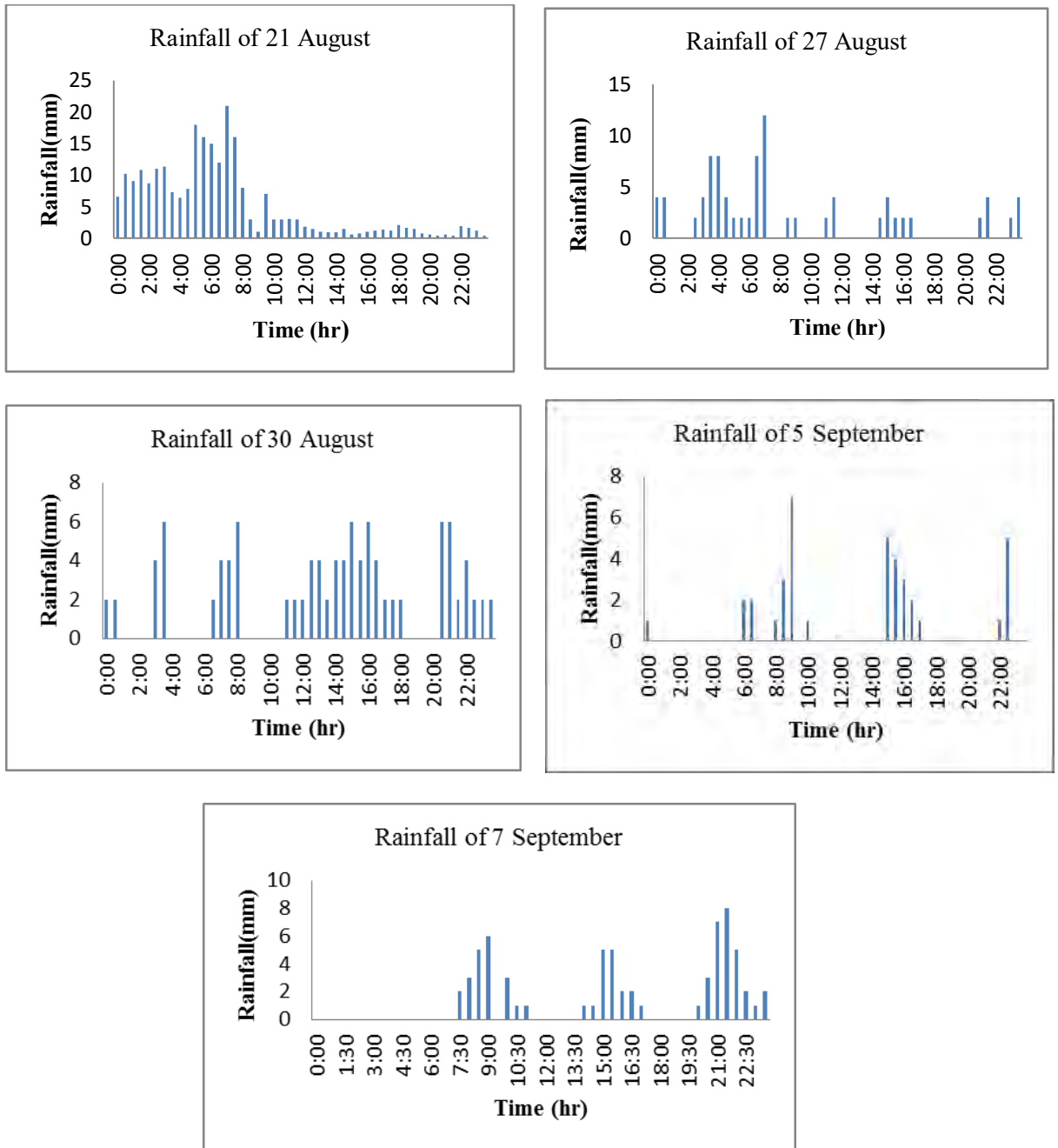


Figure 5.2: Precipitation data for calibration and validation, 2016

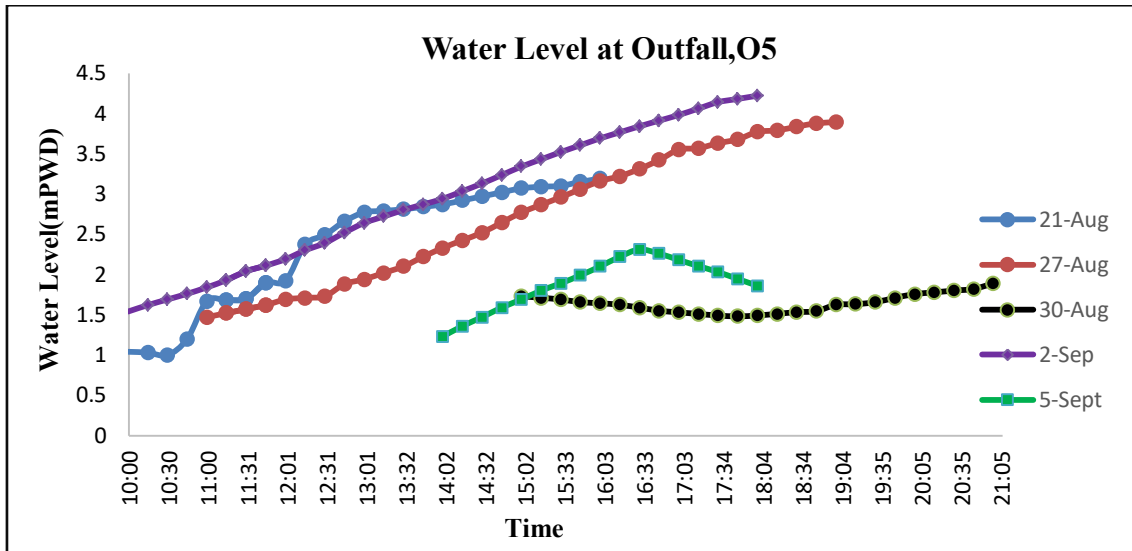


Figure 5.3: Water level boundary condition at outfall O5, 2016

This water level data is used as a boundary condition for each single storm event at the outlet of Kanchannagar sluice. About 1500 ha area contributes water to this outlet which is 22% of the total catchment area. Due to this reason, this outfall has been selected as a boundary location.

Calibration

Rainfall and boundary data of 30 August, 2 and 5 September, 2016 was used for calibration. After initial parameterization, initial depth of the contributory junctions and Mannig's N value have been adjusted to match the simulated WL with the observed WL. The model was calibrated with observed water level at Kanchannagar canal which are shown in the following Figure 5.4, Figure 5.5 and Figure 5.6

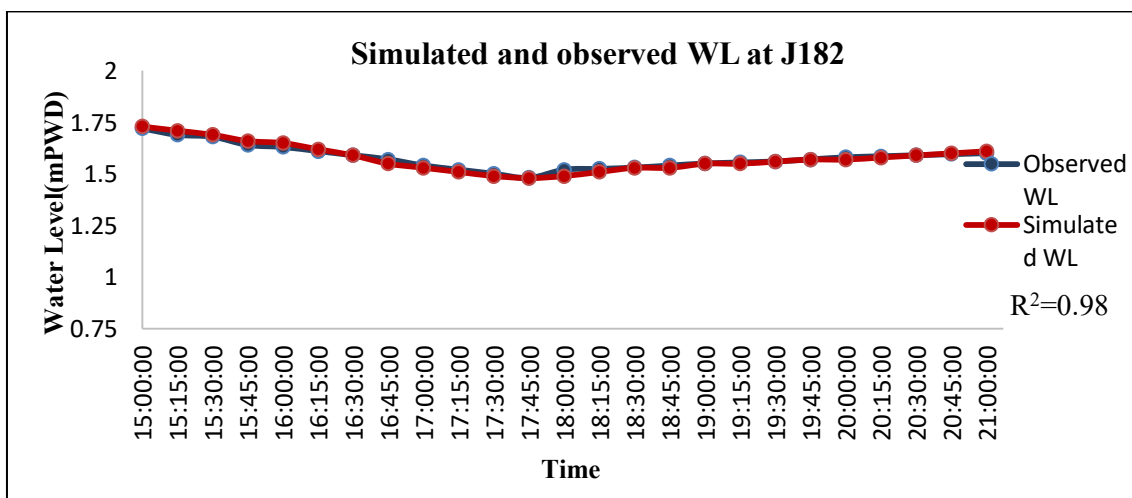


Figure 5.4: Calibration of WL for 30 August, 2016

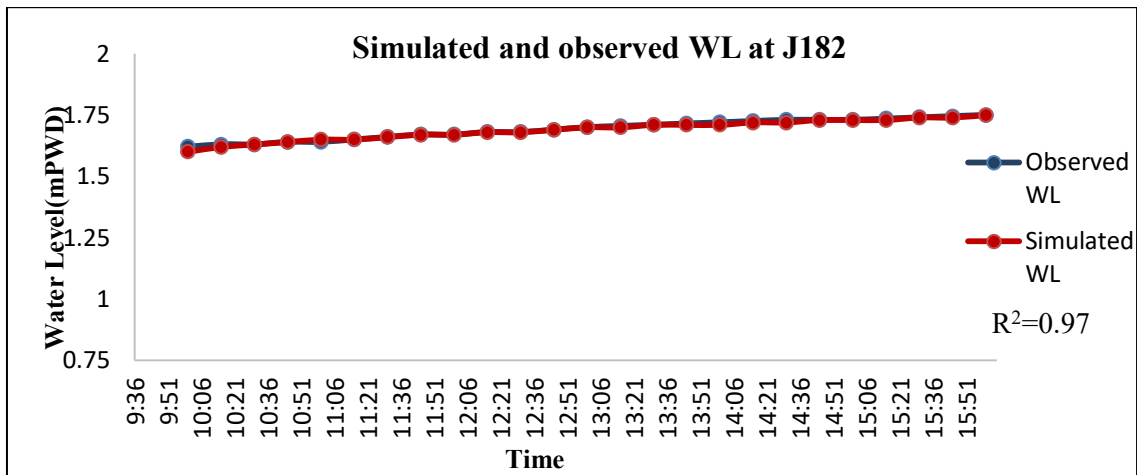


Figure 5.5: Calibration of WL for 2 September, 2016

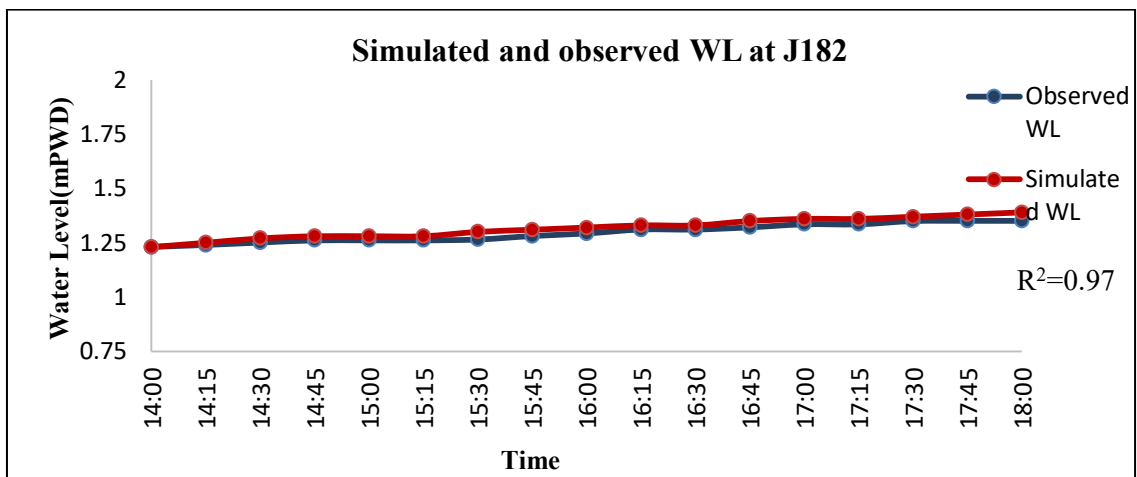


Figure 5.6: Calibration of WL for 5 September, 2016

It is clearly seen that the simulated value matched very well with the observed value and Pearson's R value is above 0.95 for all the three days which means it is a very good correlation.

Validation:

After calibration, the model was validated for 21 and 27 August of 2016. Pearson's R value lies in the ranges between 0.75 to 0.94 for this validated model.

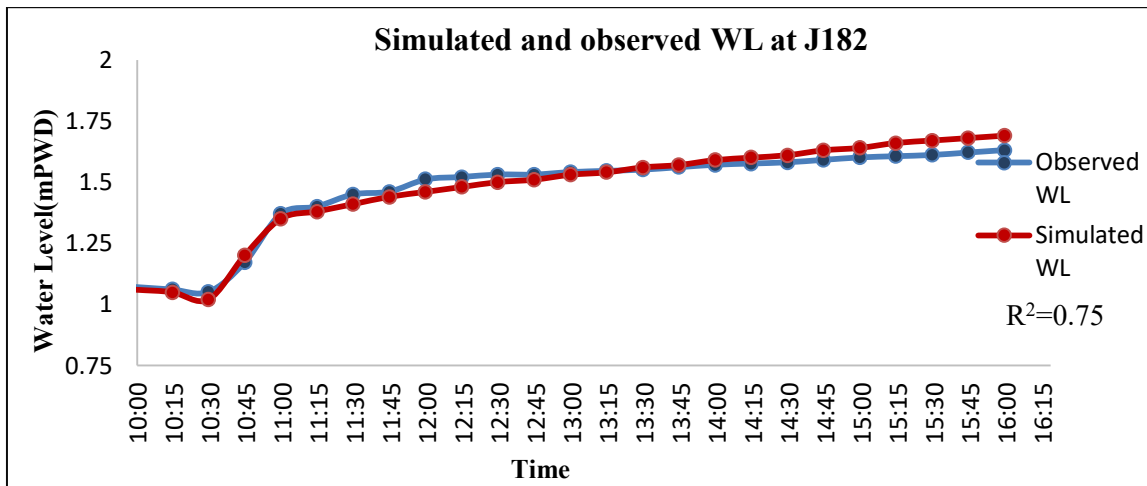


Figure 5.7: Validation of WL for 21 August, 2016

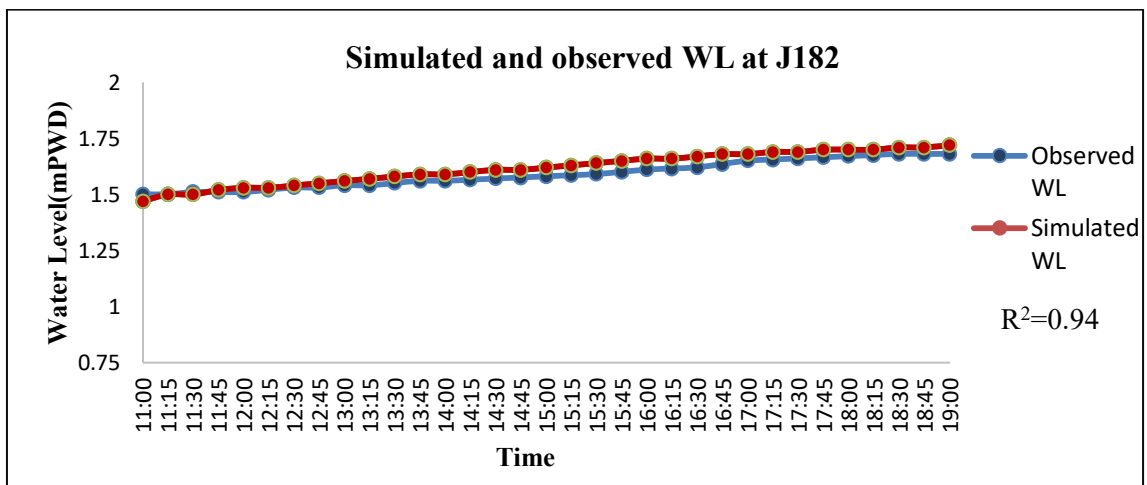


Figure 5.8: Validation of WL for 27 August, 2016

5.3 Comparison of Inundation Map with Satellite Image

In order to validate inundation extent of this model with the actual scenario, the calibrated model was applied to find the flood depth and to match with actual inundation. For this purpose, actual inundation map of 27 August, 2016 was collected from Sentinel-2 satellite image. The Sentinel-2 mission is a land monitoring constellation of two satellites that provide high resolution optical imagery and provide continuity for the current SPOT and Landsat missions. The mission provides a global coverage of the Earth's land surface every 10 days with one satellite and 5 days with 2 satellites, making the data of great use in on-going studies (ESA, Earth Online). For better comparison purpose, the image has been categorized as water and non-water

zone. Figure 5.9 shows the comparison between observed flood map and model simulated flood map. Common places of inundated area between model simulated flood map and observed flood map have been marked by circle. From figure it can be seen that the inundation areas between simulated and observed are adequately alike.

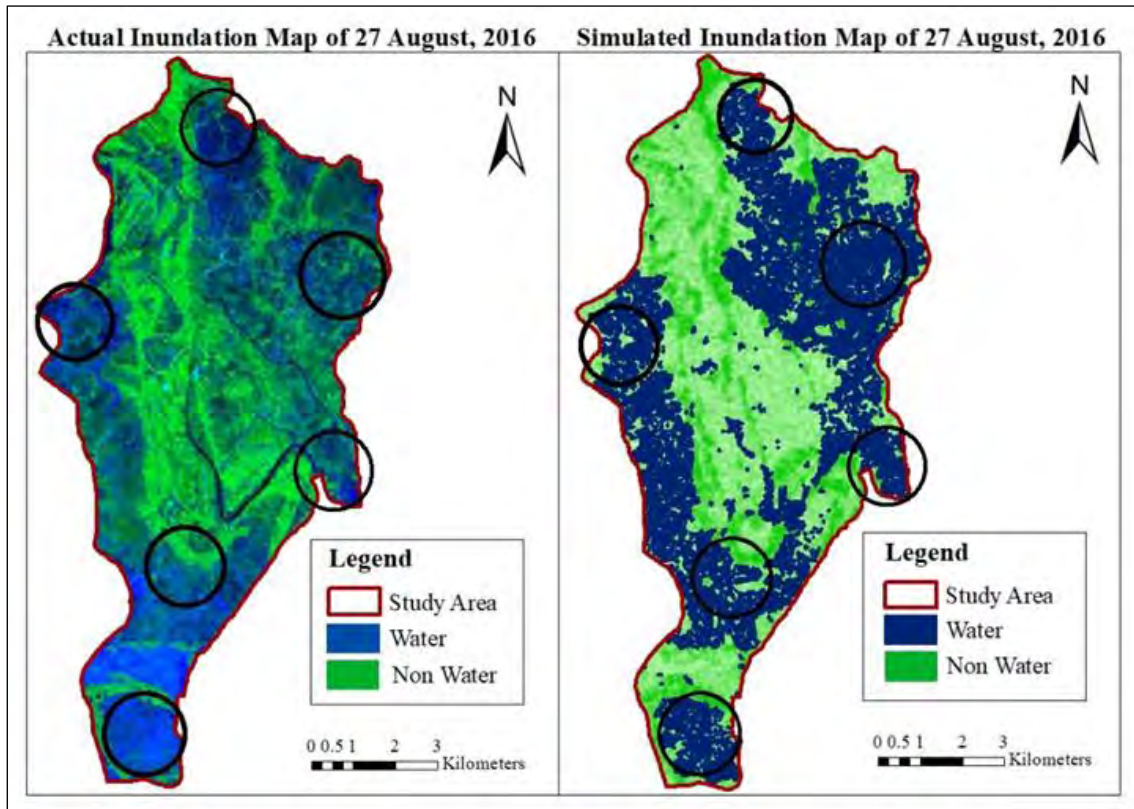


Figure 5.9: Actual and simulated inundation of 27 August, 2016

5.4 Design Rainfall

A design storm has a specific depth and a preselected duration. The specific depth of a design storm has a desired exceedance frequency. With the use of a catchment rainfall-runoff model, peak flow resulting from the input of a design storm is assumed to have the same exceedance frequency as the input design storm. Design storms with different exceedance frequencies are constructed based on results from the statistical analysis of historical rainfall data

A return period of rainfall, also known as a recurrence interval is an estimate of the likelihood of rainfall to occur. It is a statistical measurement typically based on historic data over an extended period of time, and is used usually for risk analysis. When dealing with structure design expectations, the return period is useful in calculating the

riskiness of the structure. For this reason, it is necessary to specify return period for analysis and design of stormwater drainage.

Time of concentration is a concept used in hydrology to measure the response of a watershed to a rain event. Small drainage areas will have a short time of concentration and this will produce a high intensity and high peak discharge per unit of area. Thus, for small areas it is not economical to design for long return periods such as 50 years or 100 years.

Design rainfall of 5 years return period has been selected for designing different types of drains in Mymensingh Pourashava during the preparation of stormwater drainage plan and temporary pumping was suggested only during extreme flood event (Khan, 2015). In Master Plan for Greater Dhaka Protection Project of Bangladesh Flood Action Plan no. 8A (FAP 8A), design rainfall of 5 years return period is applied for Khal Improvement and Trunk Drain (JICA, 1991)

In storm water drainage manual of Hongkong, a return period of 10 years was suggested for determining design capacity for pumps as it was assumed that within this period the pump can handle rainstorm runoffs collected inside the polder (SDM, 2018). Also a return period of 2-5 years was recommended for intensively used agricultural land and 10 years was suggested to use for village drainage including internal drainage system under a polder scheme. According to the research work of San Diego State University, for urban drainage of small catchment, design rainfall return period is 5 to 10 years (Schmitt et al., 2004). A study (Fortunato, 2014) had been done to define a methodology to identify the optimal rainfall return period for the design of urban drainage systems. The choice of the optimal return period is made minimizing the total costs of the system. The sewer network is dimensioned for a set of possible design rainfall return periods, and the corresponding construction, maintenance and operation costs are evaluated. For each scenario, the total expected damage for flooding caused by rainfall events with return period greater than the design one is then estimated by hydraulic simulation. The optimal solution for the case study is the adoption of design rainfall period of 10 years.

Ten (10) years return period (REF: i) Design Workshop Small Scale Water Control Structures, Volume-1, BWDB and CIDA, Northwest Hydraulic Consultant, 1993, ii) Standard Design Manual, Volume – I: Standard Design Criteria, BWDB) has been

selected to estimate design rainfall and to observe the drainage condition of the selected polder area.

Table 5.1 Statistical distribution of annual maximum value of daily rainfall (mm) for Khulna

	Return Period(Years)	EVI- Distribution	Normal Distribution	Log- Normal Distribution	Log Pearson Type III Distribution
Estimated Quantile	5	185.76	194.35	175.44	171.74
	10	226.83	225.22	216.35	216.54
	20	278.71	258.17	258.97	283.814
	50	317.21	279.43	294.77	342.85
	100	355.42	298.57	331.22	409.01
Goodness of fit test statistics	Chi-square value	9.24	73.38	3.19	4.01
	Critical Chi- square value (95%)	5.99	5.99	3.84	3.84

From the analysis, the lowest χ^2 value is found for Log-Normal Distribution which is 3.19 and it is also smaller than the critical value 3.84 for 95% confidence limit. So the annual maximum value of rainfall series fits the Log Normal Distribution and 216.35 mm rainfall

is selected as a design storm for checking the adequacy of drainage channels and preparation of inundation map. In the JICA (1987) study, the rainfall pattern of heavy rainfall was studied and the duration of heavy rainfall was found as six hours. The following distribution ratio (Table 5.2) is applied here for the design storm condition

Table 5.2: Percentages of rainfall for various time steps

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

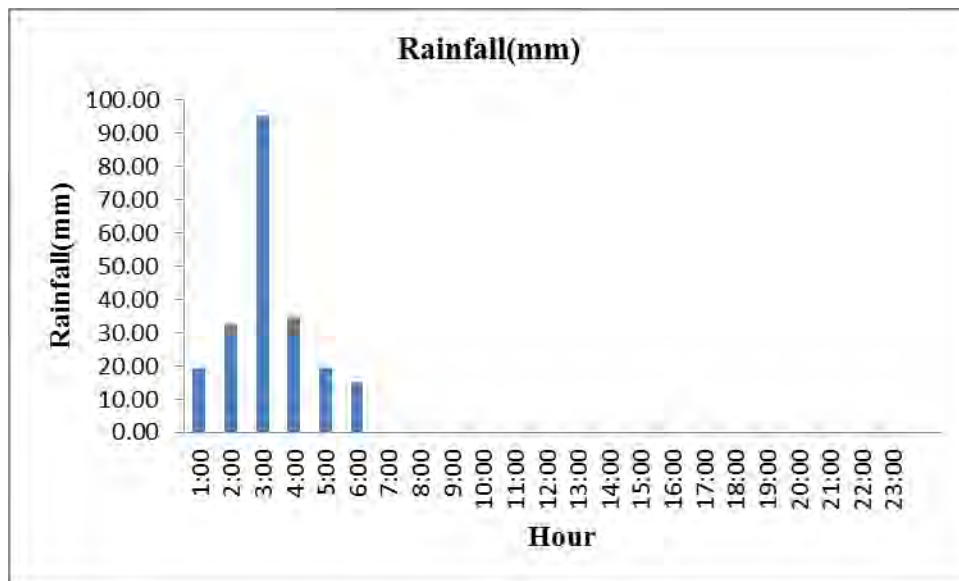


Figure 5.10: Distribution of annual maximum value of 1 in 10 year rainfall

5.5 Design Water Level:

The results of the frequency analysis using Gumble method is given below:

Table 5.3: Statistical distribution of annual maximum value of high tide and minimum value of low tide at Dumuria and Sutarkahli stations

	Return Period(Years)	Dumuria Station (SW28)		Sutarkhali Station (SW29)	
		High Tide	Low Tide	High Tide	Low Tide
Estimated Quantile (Gumble Distribution)	5	3.42	-0.92	3.64	-0.97
	10	3.63	-0.66	3.80	-0.78
	20	3.83	-0.38	3.96	-0.57
	50	3.99	-0.20	4.07	-0.44
	100	4.12	-0.03	4.17	-0.32
Goodness of fit test statistics	Chi-square value	2.72	2.03	2.49	2.98
	Critical Chi-square value (95%)		3.84		3.84

The statistical analysis revealed that, Gumble distribution fits the historical dataset and the high and low tidal value of the above mentioned stations for 10 year return period was used as a d/s boundary condition in this study. For Kanchannagar outfall (O5) tidal water level data (1in 10year period) is shown in Figure 5.11 and for other outfalls the tidal series data is given in Appendix B.

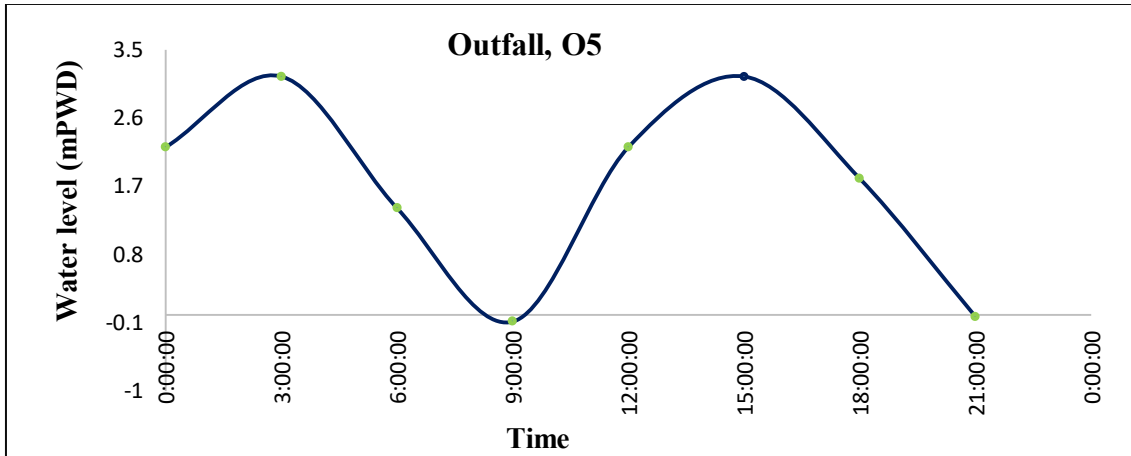
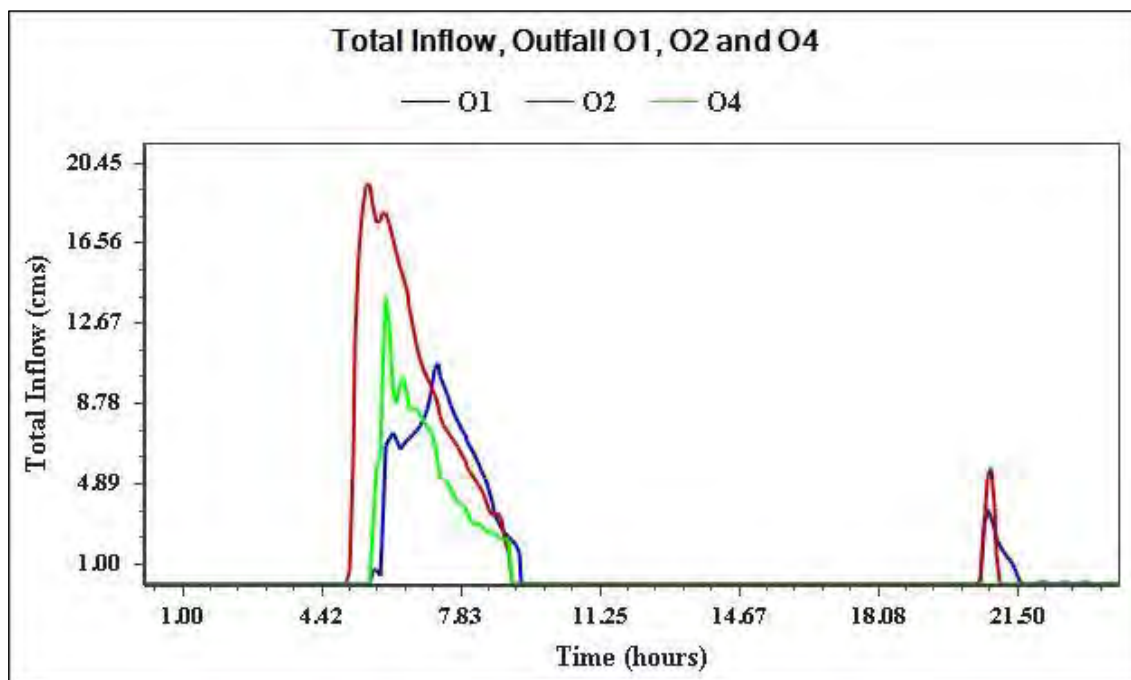
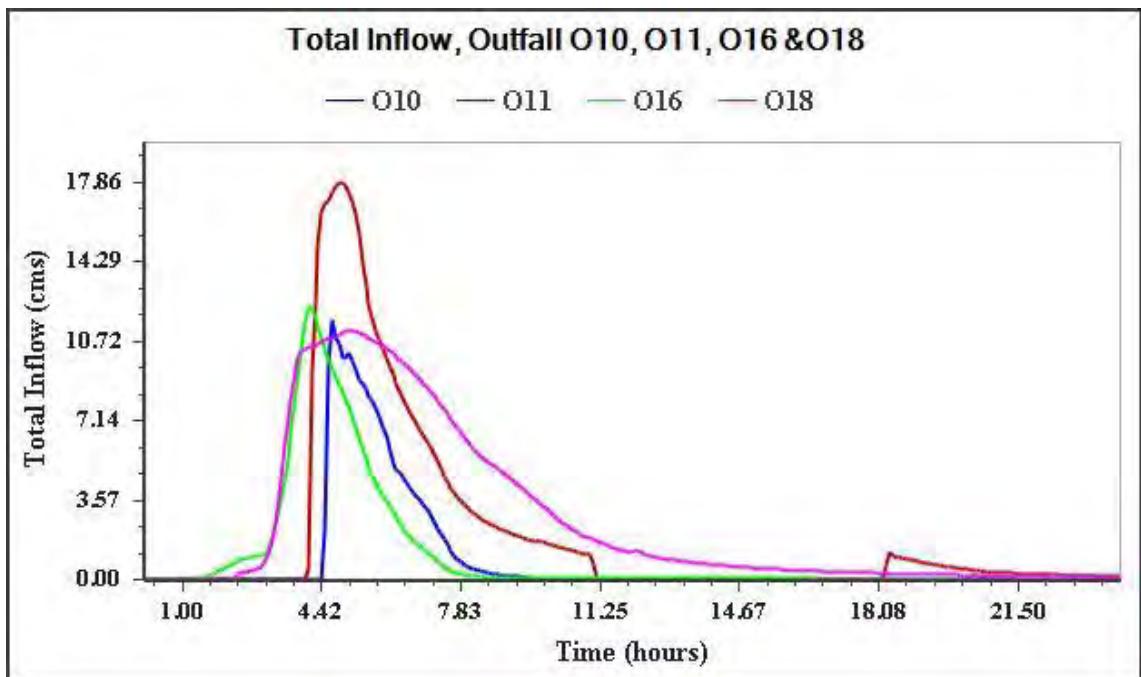
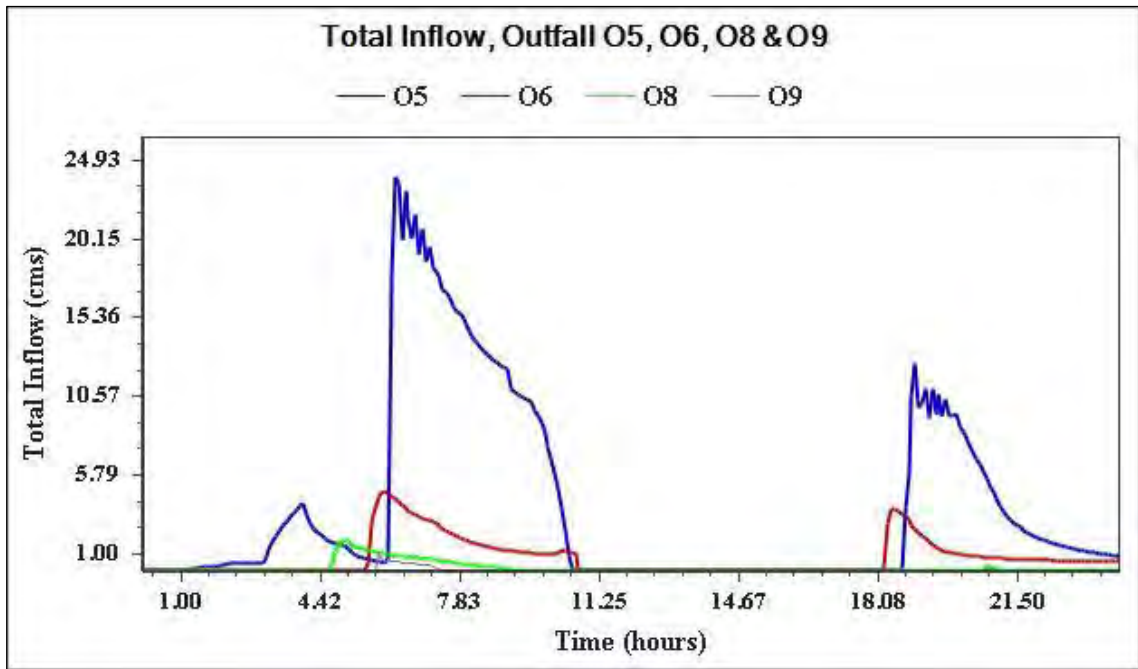


Figure 5.11: Tidal water level data for Kanchannagar outfall, O5 for 1 in 10 year period

5.6 Adequacy Check of Existing Drainage Channels:

The model is simulated under the design rainfall and design water level condition. The flow hydrographs at the outfalls are shown in the Figure 5.12





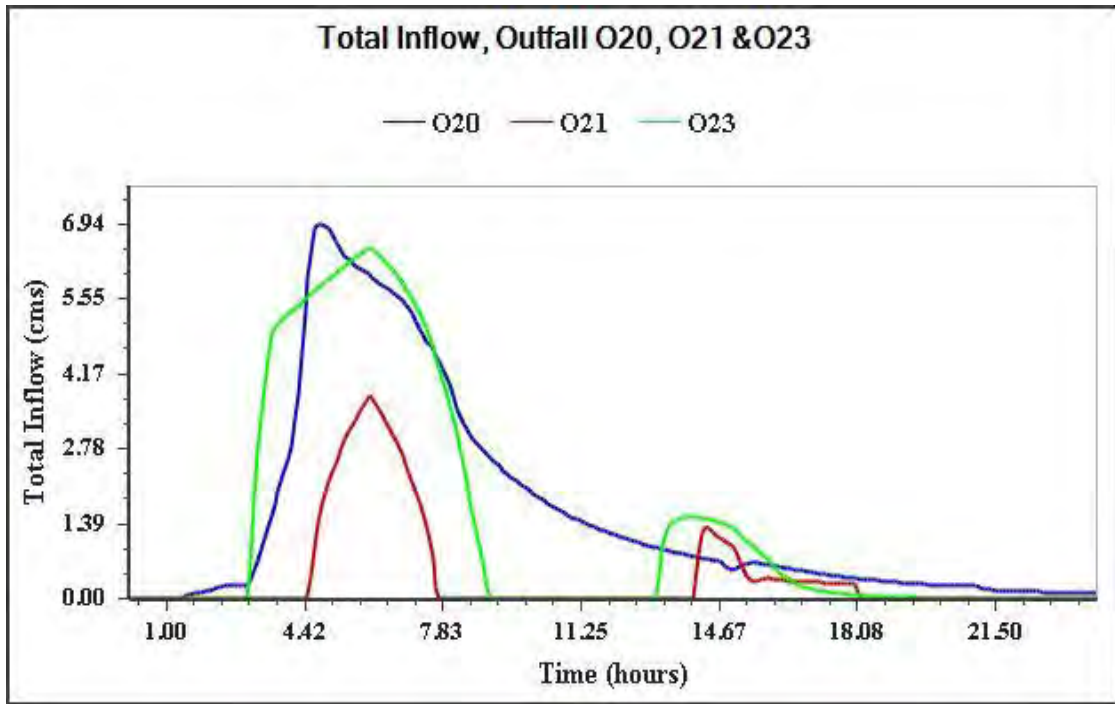


Figure 5.12: Flow hydrograph at outfalls of Polder 29

After the simulation it was observed that ten of the conduits are not adequate enough to carry the maximum flow obtained from the design rainfall scenario. For these conduits the required cross sectional areas are calculated and shown in the following table 5.4:

Table 5.4: Proposed cross sectional area for inadequate drainage channels

Conduit Name	Provided X-sectional Area (m ²)	MaxFlow/FullFlow	Required X-sectional Area (m ²)
C12	3.25	1.1	3.58
C23	22.79	1.02	23.25
C26	7.56	1.69	12.78
C27	6.54	1.94	12.69
C28	8.29	1.22	10.11
C53	13	1.62	21.06
C62	4.36	1.17	5.10
C73	12.41	1.2	24.10
C97	1.79	1.06	1.90
C108	3.21	1.01	3.25

Model simulated water depth is higher than the existing maximum depth for the conduits C3, C5, C6, C9, C15, C24, C31, C34, C35, C104 and C190 (total 11) due to tidal water level at the corresponding outfalls. As the flapgates remained closed during high tide level at the outside, the inside water could not pass through the outlets. As a result, there was a rise in water level in the respective conduits where the velocity ranged from 0.16 to 0.54m/s. If max flow vs full flow ratio is 1 then it indicates that the simulated water level exceeds the given depth of the drainage channel. From the study, conduits C10, C01, C13, C66, C65, C64, C70, C77, C81, C82, C101, C92, C105 and C106 (total 14) have higher water level at d/s node than the u/s node which made these conduits inadequate for drainage. So the u/s depth of the above mentioned conduits are found to be inadequate to convey the produced runoff.

5.7 Inundation Mapping for Design Storm Condition

Information on flood inundation extent is important for understanding the effects of flooding in that particular area, water storage volumes, and evaluating condition of the existing drainage channels. It provides important information, like depth and spatial extent of flooded zones. Therefore, in any waterlogged area, it is essential to prepare flood inundation map (FIM) to understand its extent and then provide a suitable improvement measure to mitigate this condition. In this study, FIM is prepared for 1 day design storm scenario showing the area of different land classes (F0, F1, F2, F3 and F4) using model results. In this model, the flood depth is calculated by subtracting the DEM elevation from the generated maximum water surface elevation. In this study land classification F0 is defined for water depth 0.0m to 0.30m, F1 is defined for 0.30m to 0.90m, F2 is defined for 0.90m to 1.80m F3 is defined for 1.80m to 3.6m and F4 land is classified for water depth >3.6m. Flood Free (FF), F0 and F1 land is considered as total flood free area for agricultural purposes.

5.7.1 Assessment of Effectiveness of Present Drainage System During Design Flood Event

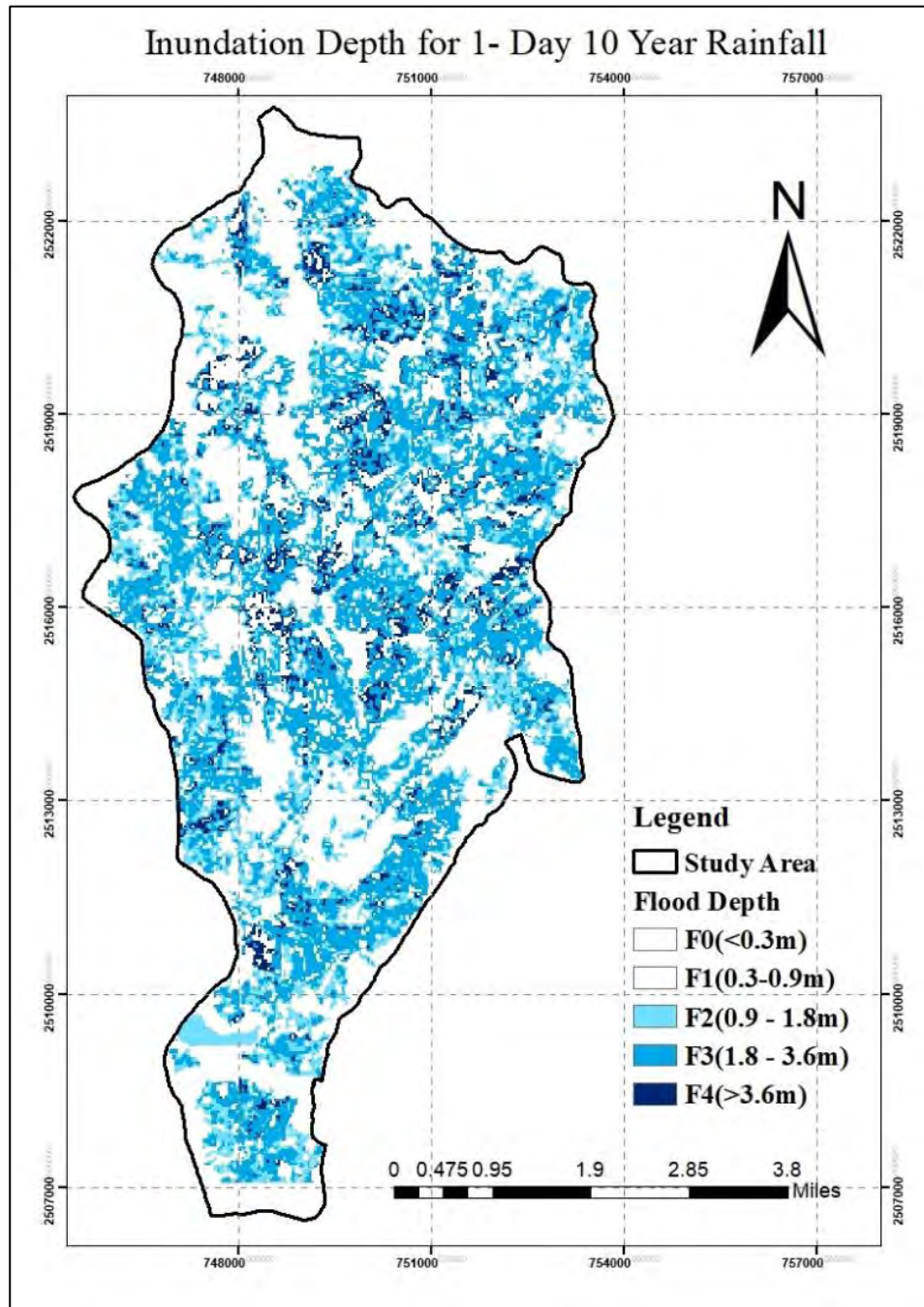


Figure 5.13: Inundation depth map of polder 29 for 1-day 10 year rainfall at existing condition

Inundation map has been prepared for 1in 10-year design rainfall on the existing drainage system. From the analysis it is seen that about 41.07% area remains flood free (FF). About 0.30% area lies below 0.3m inundation depth (F0) and 6.31% of the total polder area is under water depth ranged from 0.30m to 9.0 m (F1). Approximately 16.34% area lies within water depth 0.9-1.8m (F2). About 30.81% area remains under deeply inundation depth (F3) and only 5.18% area remains under water depth more than 3.6m (F4). The Inundation depth map of the study area for 1-day duration and land class at existing condition is shown in the Table 5.5.

Table 5.5: Inundation depth of Polder 29 for design flood event at existing drainage condition

Inundation depth considering design storm event	% Inundated area							
	Flood Free, FF	F0	F1	F2	F3	F4	Total flood free area (%)	Total flooded area(%)
		Inundation up to 0.3m	Inundation 0.3 to 0.9m	Inundation 0.9 to 1.8m	Inundation 1.8 to 3.6m	Inundation > 3.6m	(FF +F0+F1)	(F2+F3+F4)
Area (ha)	3258	24	500	1296	2444	411	3282	4151
Percentage (%)	41.07	0.30	6.31	16.34	30.81	5.18	41.37	52.33

5.8 Adaptive Measure

The present condition of the selected polder is vulnerable. Some hydraulic structures i.e sluice gates, flap gates are fully or partially damaged and are non-functioning due to which internal drainage congestion is prevalent in this area. The other structures, though functional, are in an extremely bad condition. The gates are corroded by saline water and concrete surface of the structures are in a very deplorable condition. Due to continuous siltation, the internal drainage channels have become inadequate to carry significant amount of flow. Considering these conditions, an improvement measure i. pumping facility has been suggested for this area. This option is taken based on the fact that during monsoon season when the outside WL of the river is experiencing high tide,

no water can be passed through the gates under gravity and the WL inside the polder will increase which may results drainage congestion. Also there is a possibility that the polder area will be waterlogged in near future and gravity drainage would not be viable. In this case the best probable suggestive measure can be pumping to remove this surplus water and reduce the inundation depth.

5.8.1 Pumping Facility at Existing Drainage Condition for Overall Study Area

From the previous discussion, it was found that about 52.33% of the total area is waterlogged at design storm condition. To mitigate this problem, 7 pumps were used at outfall O1, O2, O5, O8, O11, O16 and O20. The location of pump setting is based on the extent of inundation within this polder area. Here, ideal pump condition was applied to simulate the model as a preliminary design condition. The operation of the pump was performed by adjusting the startup and shut-off depth of the inlet node of the pump. These depth values are based on the maximum depth attained at each inlet node after simulating the model at design rainfall event. Pump locations at the study area are shown in the following Figure 5.14 and the inundation condition after applying the pump is shown in the Figure 5.15

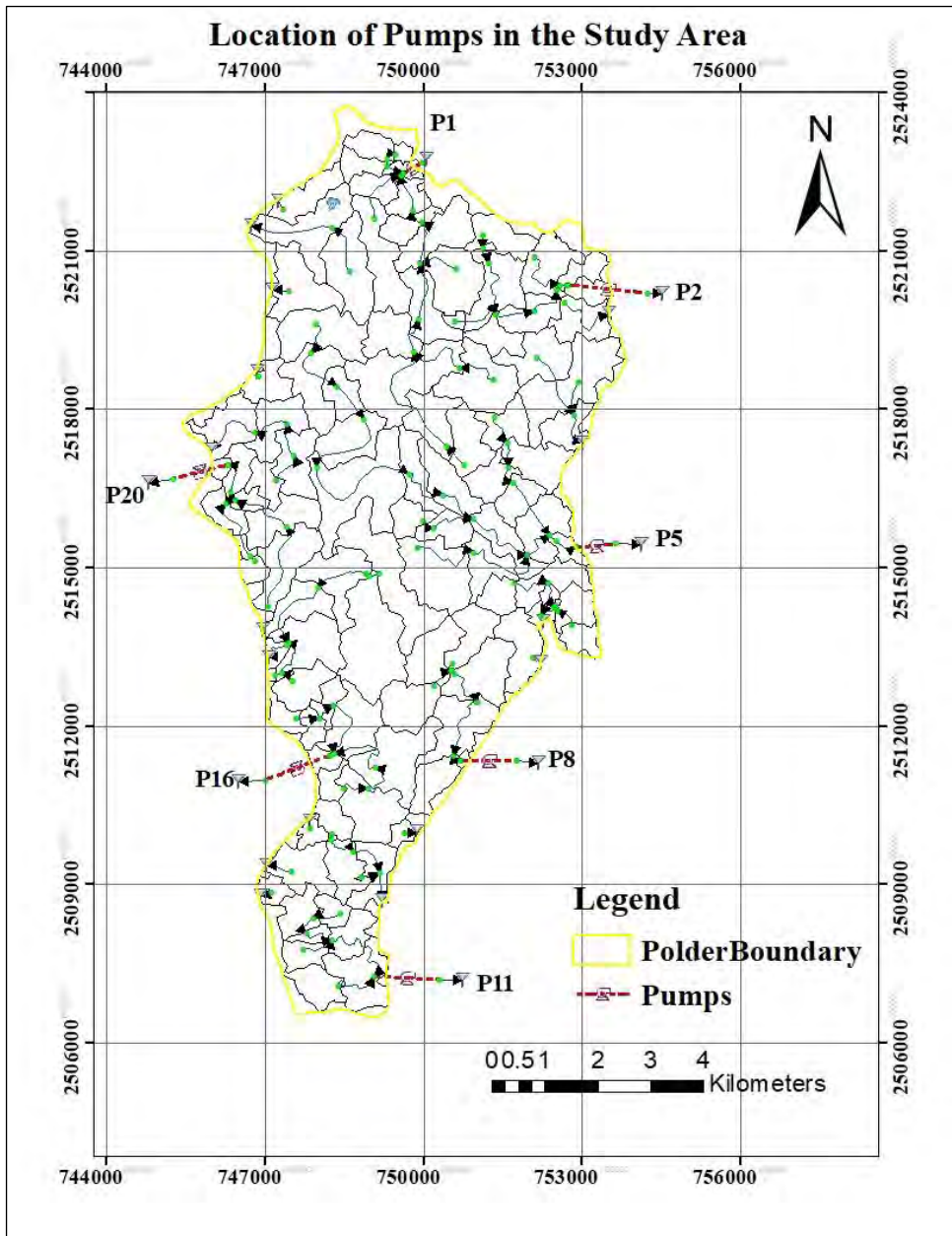


Figure 5.14: Location of pumps in the study area

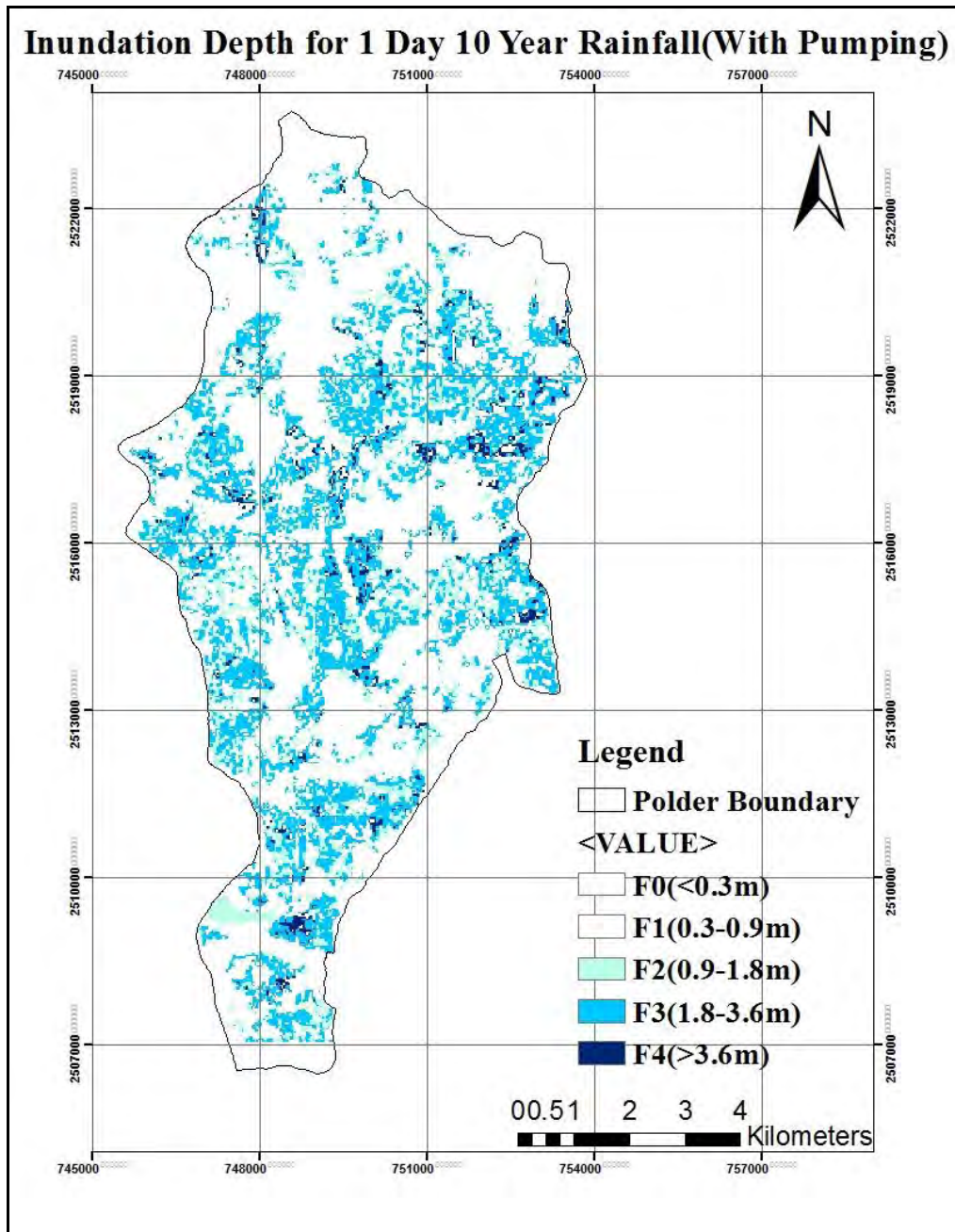


Figure 5.15: Inundation depth map of polder 29 for 1-day 10 year rainfall (with pumping)

Table 5.6: Inundation depth of Polder 29 for design storm scenario (with pumping)

Inundation depth considering design storm event (with pumping)	% Inundated area							
	Flood Free (FF)	F0	F1	F2	F3	F4	Total flood free area (%)	Total flooded area (%)
		Inundation up to 0.3m	Inundation 0.3 to 0.9m	Inundation 0.9 to 1.8m	Inundation 1.8 to 3.6m	Inundation > 3.6m	(FF +F0+F1)	(F2+F3+F4)
Area (ha)	4262	28	681	1364	1417	180	4971	2961
Percentage (%)	53.72	0.35	8.58	17.2	17.86	2.27	62.67	37.33

Table 5.6 showed that total flood free area increased from 47.68% to 62.67% and this area is suitable for people movement, agricultural production. The higher inundation depth area (F4) decreased from 5.18% to 2.27 % after applying the pumping facility. Previously area that fall under the land classes F2 and F3 were 47.15% (total) and after applying the improvement option this area reduced to 35.06% compared to the base condition. As this facility is applied for this large area considering the ideal pump, therefore actual extent of inundation depth reduction is not clearly visible from the map.

For the above reasons, the largest Kanchannagar catchment (19% of the total polder area) is considered here because it comprises about 20% of the total waterlogged area and an adaptive measure is taken to improve the drainage congestion of this study area.

5.8.2 Adaptive Measure for Kanchannagar Catchment

The location of the largest Kanchannagar catchment is shown in the following figure 5.16

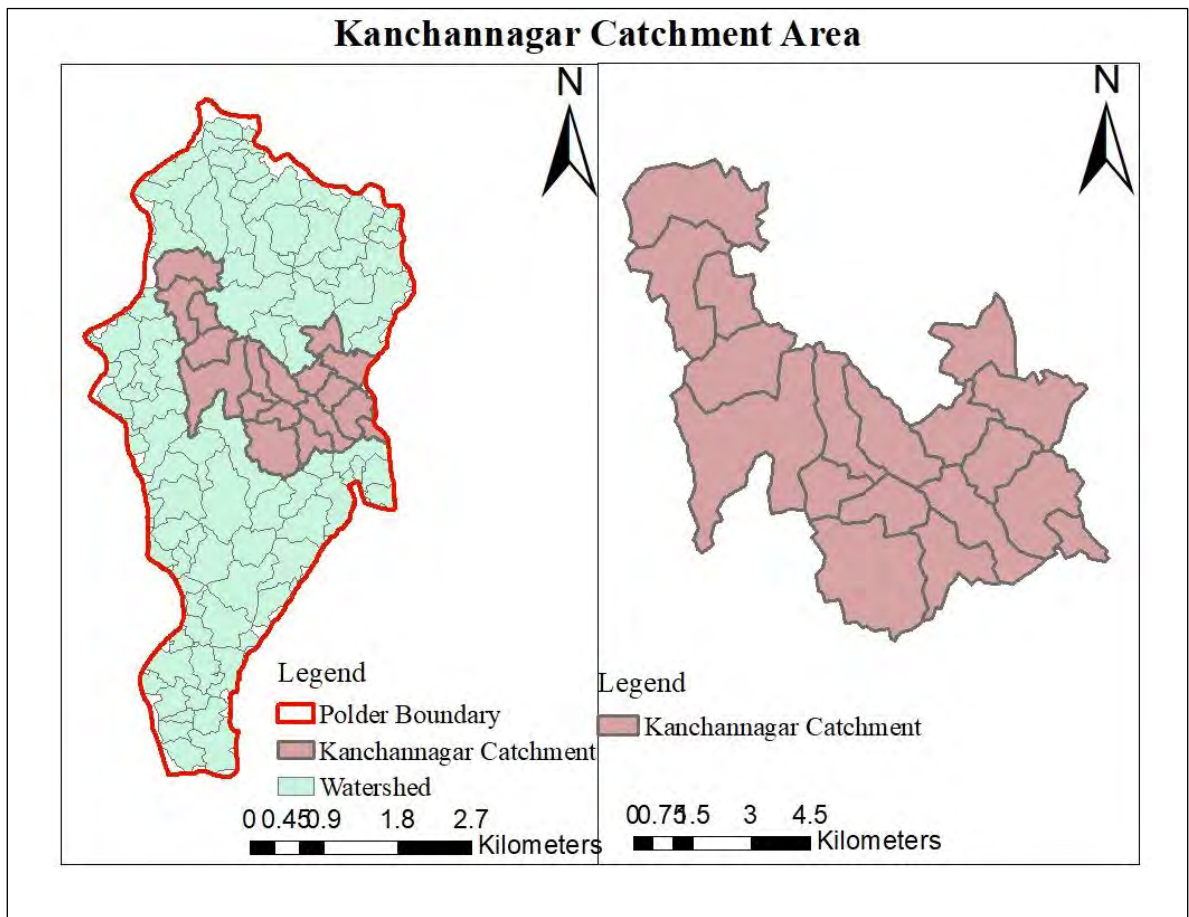


Figure 5.16: Location of Kanchannagar catchment of Polder 29

Firstly, to improve the waterlogging problem at existing condition, pumps with capacities of 8, 12 and 16 cumec were used for this area to reduce the inundation depth. This capacity is selected based on the depth and total inflow of the most d/s node of the catchment which is shown in Figure 5.17

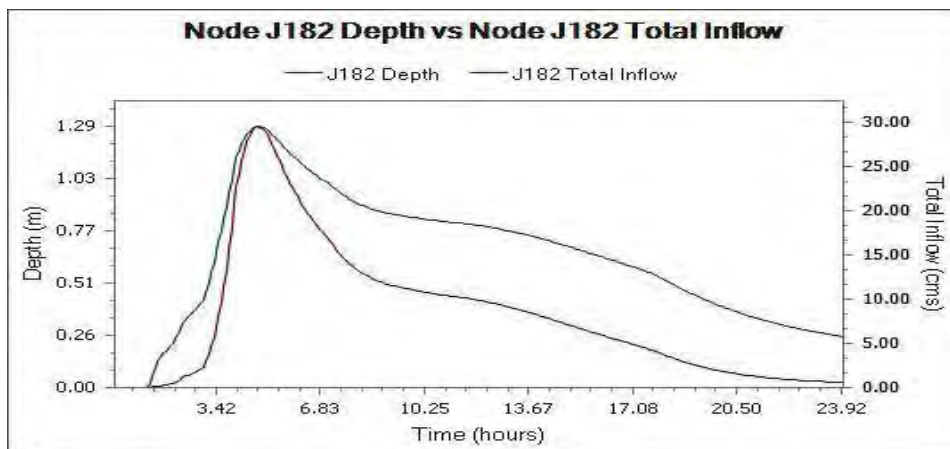


Figure 5.17: D/s node depth vs total inflow graph at pump inlet, J182

Type-2 inline pump is used here where the flow increases incrementally with inlet depth. The pump would switch on when the depth of water at inlet node of the pump exceeds 0.5 mPWD and switch off when the level falls to 0.25 mPWD. The results after application of pumping facility at this selected catchment are given in Figure 5.16

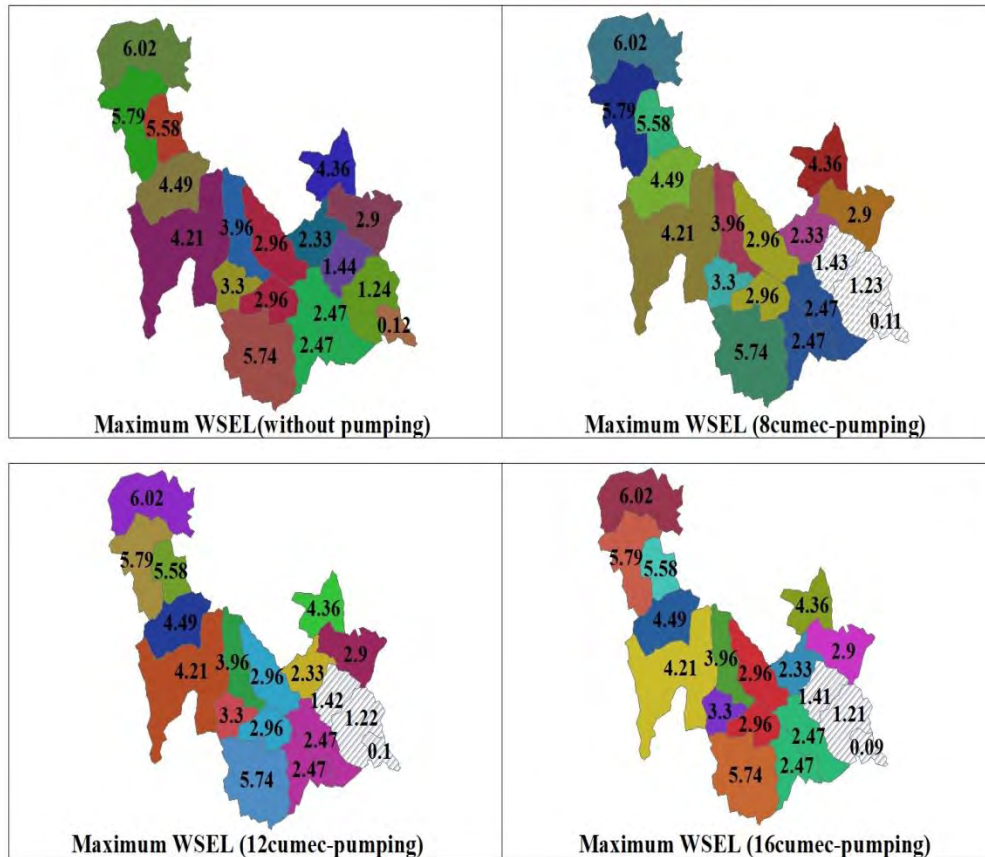


Figure 5.18: Change of HGL before and after pump application

Impact Study

Figure 5.18 showed that maximum WSEL decreased for only three catchments but the other WSEL remained same. It occurred due to reduction of conveyance capacity of the channels from u/s to d/s and presence of very flat gradient throughout the catchments. As a result, water could not pass through the conduits. It is more clearly understood from the WSEL profile of the drainage channels and this profile is shown in the Figure 5.19.

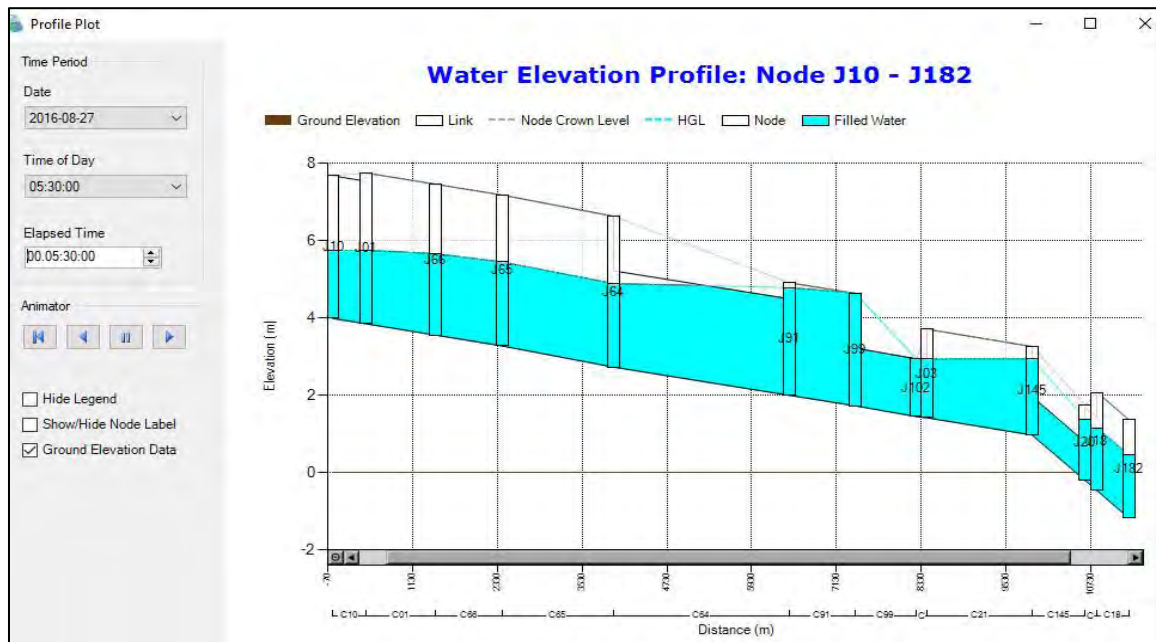


Figure 5.19: Water Surface Elevation (WSEL) profile of drainage channels for Kanchannagar catchment

The results of the analysis revealed that channel modification or proposed new channels can work as an appropriate improvement measure to overcome this adverse waterlogging effect. Re-excavation is capable of improving conveyance capacity of the channels which ultimately reduces floodplain inundation. If re-excavation alone is unable to lessen the floodplain inundation then new channels need to be proposed to minimize this effect.

5.8.3 Re-excavation of Drainage Channels at Kanchannagar Catchment:

Among the 23 drainage channels, eleven of them are re-excavated and modified to increase carrying capacity. These channels are selected based on the adequacy check results of this study. Before excavation, about 796630 m³ of total 1104700 m³ of water inundates the floodplain and some nearby areas. After modification of the channels, 543420 m³ of water lost from the system (31.8% reduction). To reduce the floodplain consumed volume, the channel capacity should be increased more but this is not feasible due to presence of high GW table of this area, therefore excavation should be done considering the location of GW table. So, five channels are proposed for construction in this study to alleviate this inundation extent. The locations are selected based on the inundation depth found from the design storm scenario. Location and cross sections of

re-excavated and proposed drainage channels are shown in the Figure 5.20 to Figure 5.22

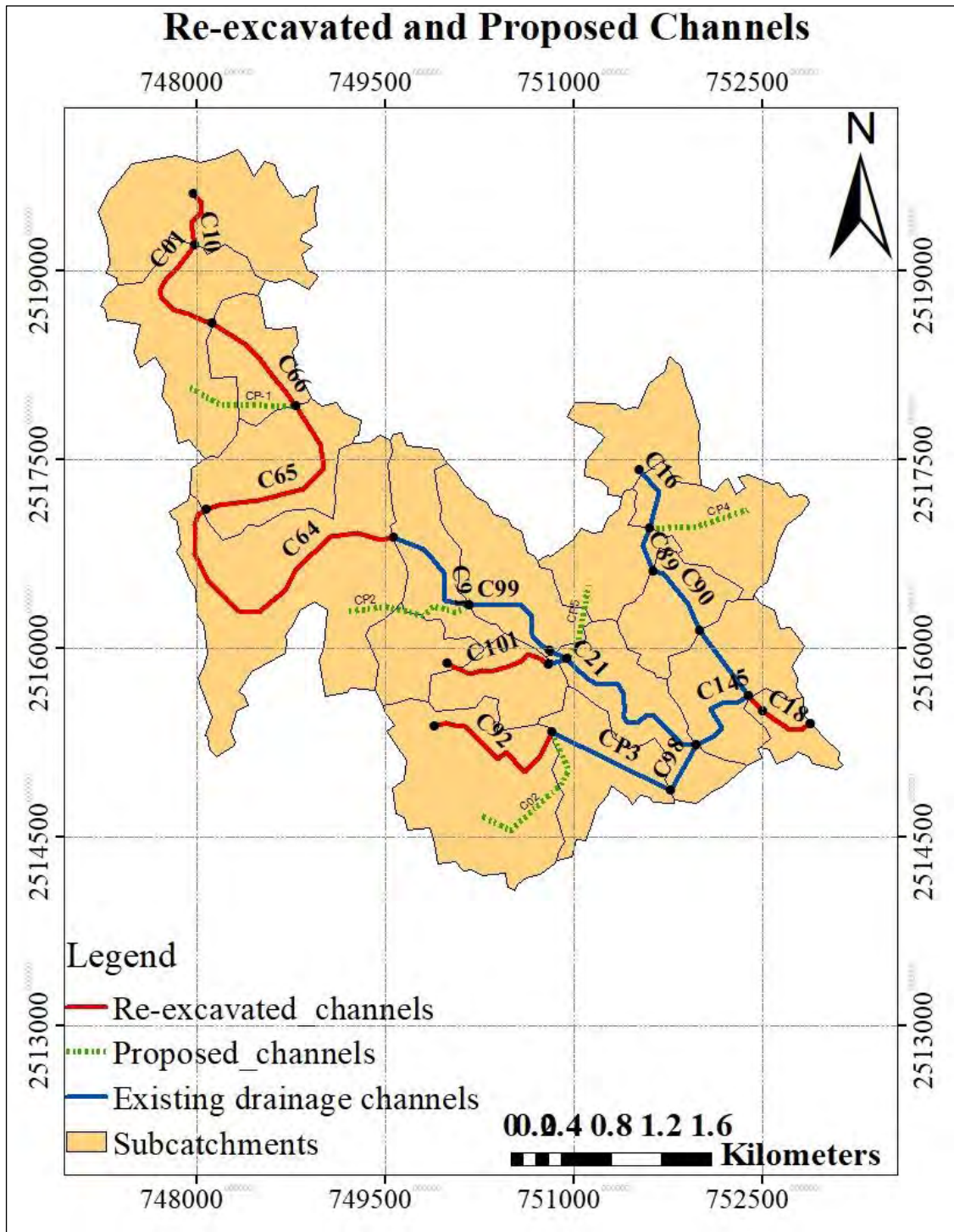
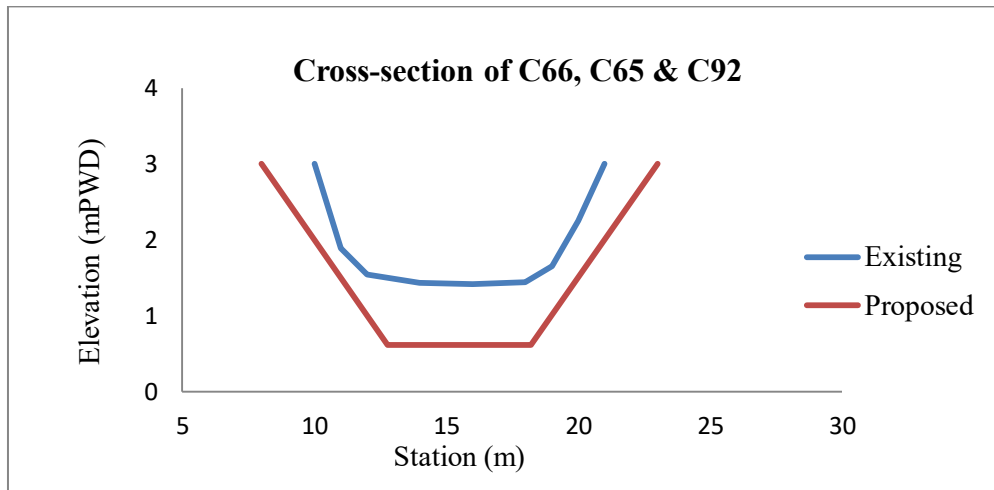
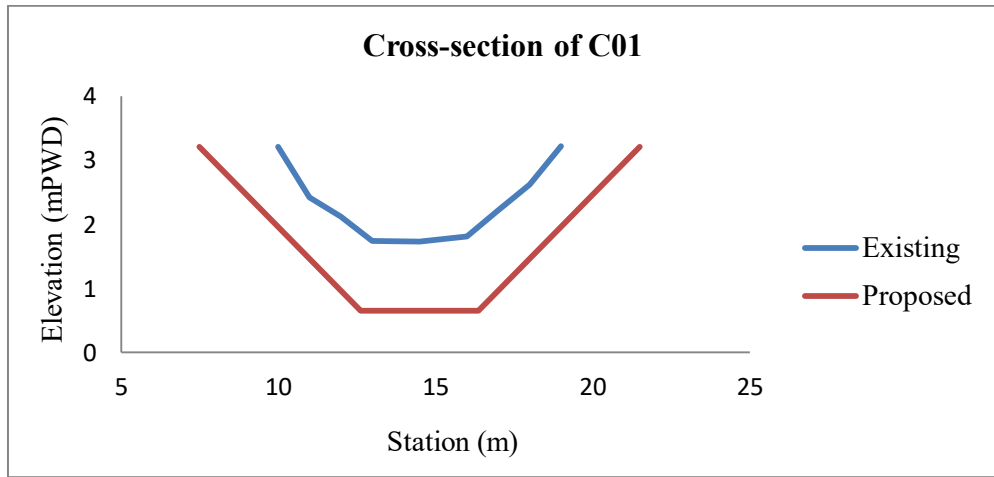
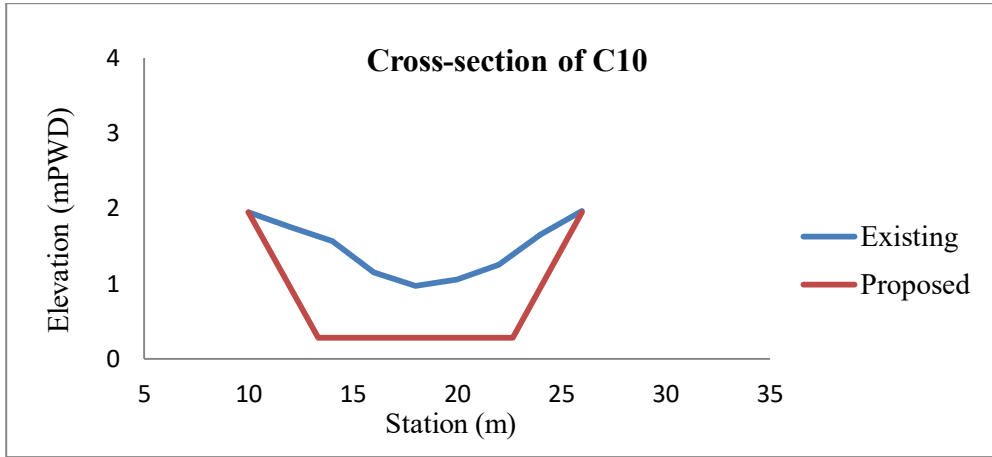


Figure 5.20: Location of re-excavated and proposed drainage channels of Kanchannagar catchment



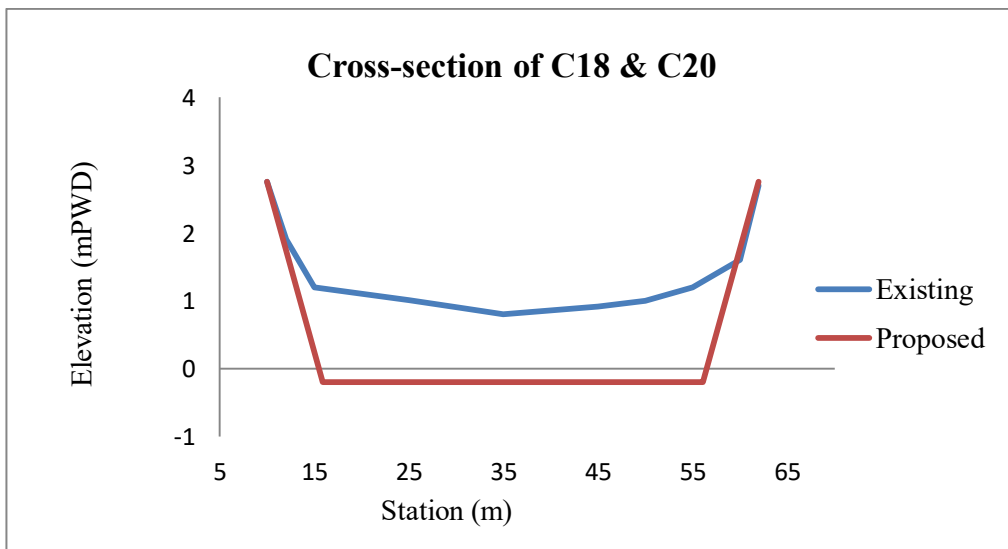
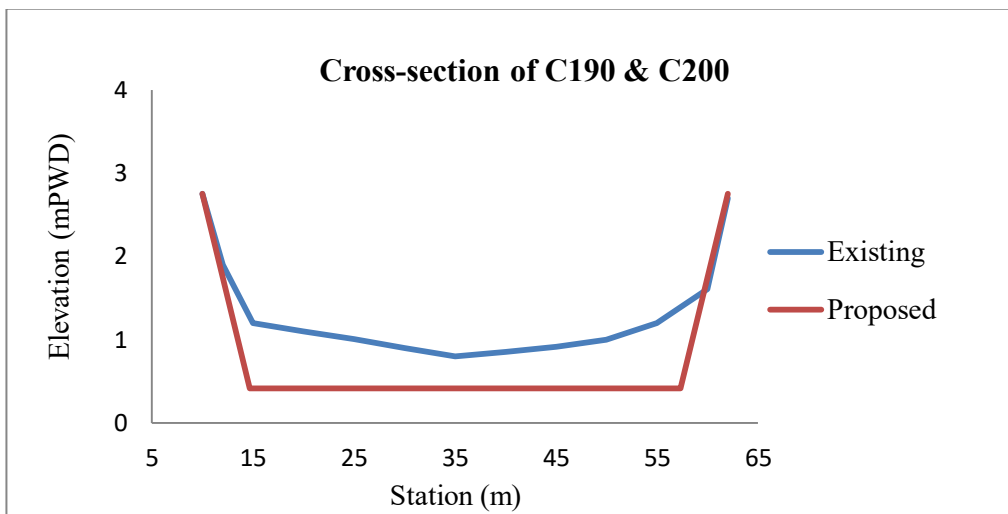
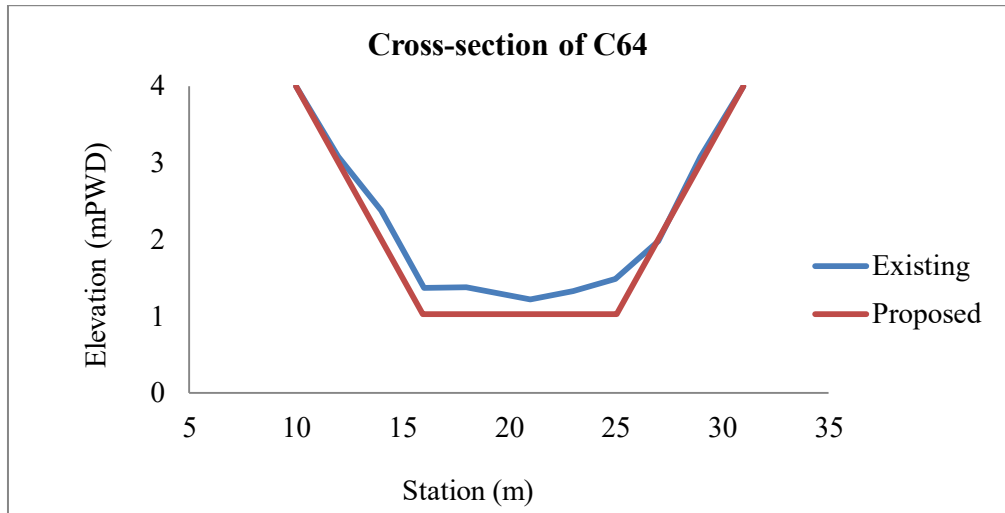


Figure 5.21: Cross sections of re-excavated channels of Kanchannagar catchment

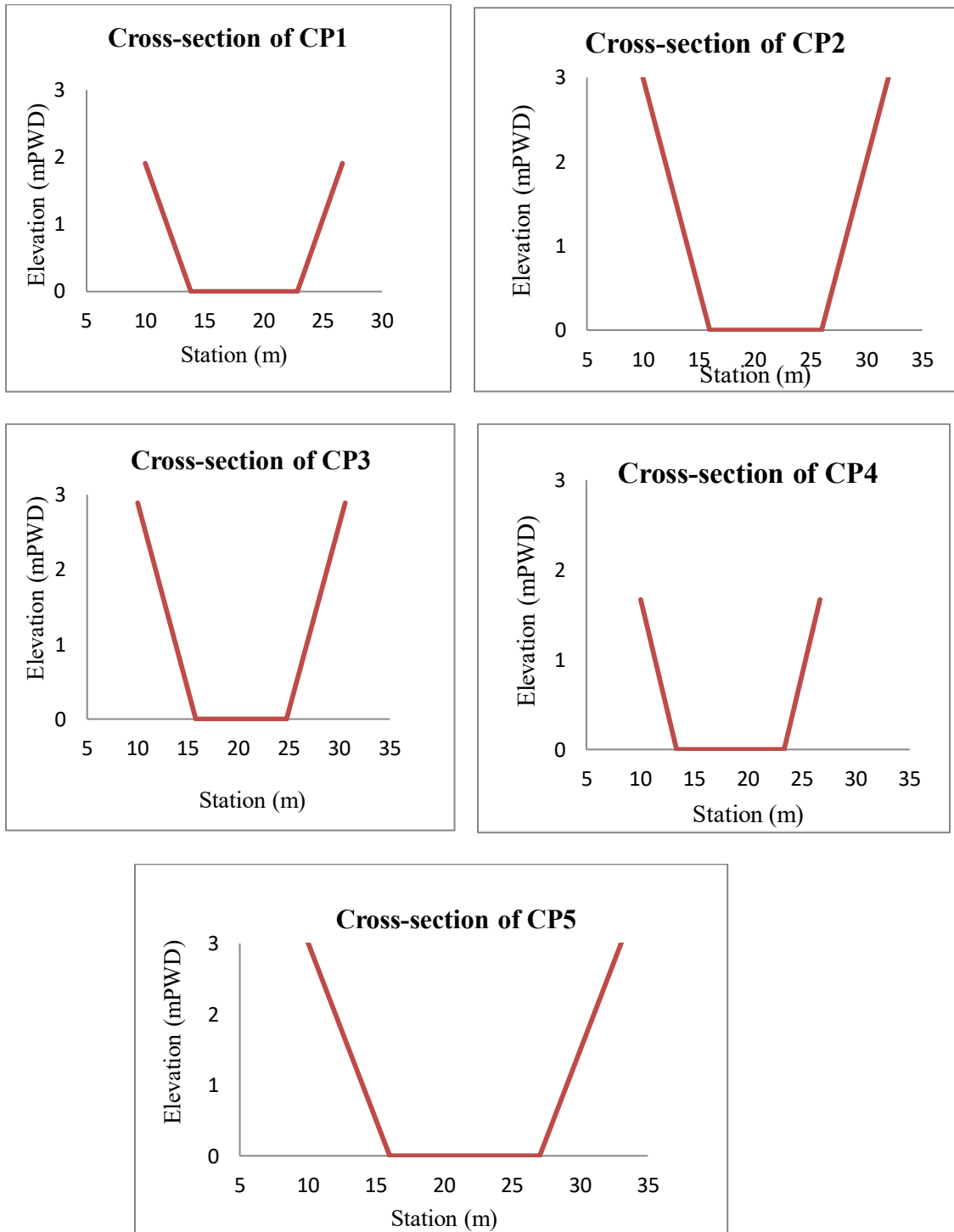


Figure 5.22: Cross sections of new proposed channels of Kanchannagar catchment

The model is simulated with these modified and new proposed drainage channels and the results are discussed below:

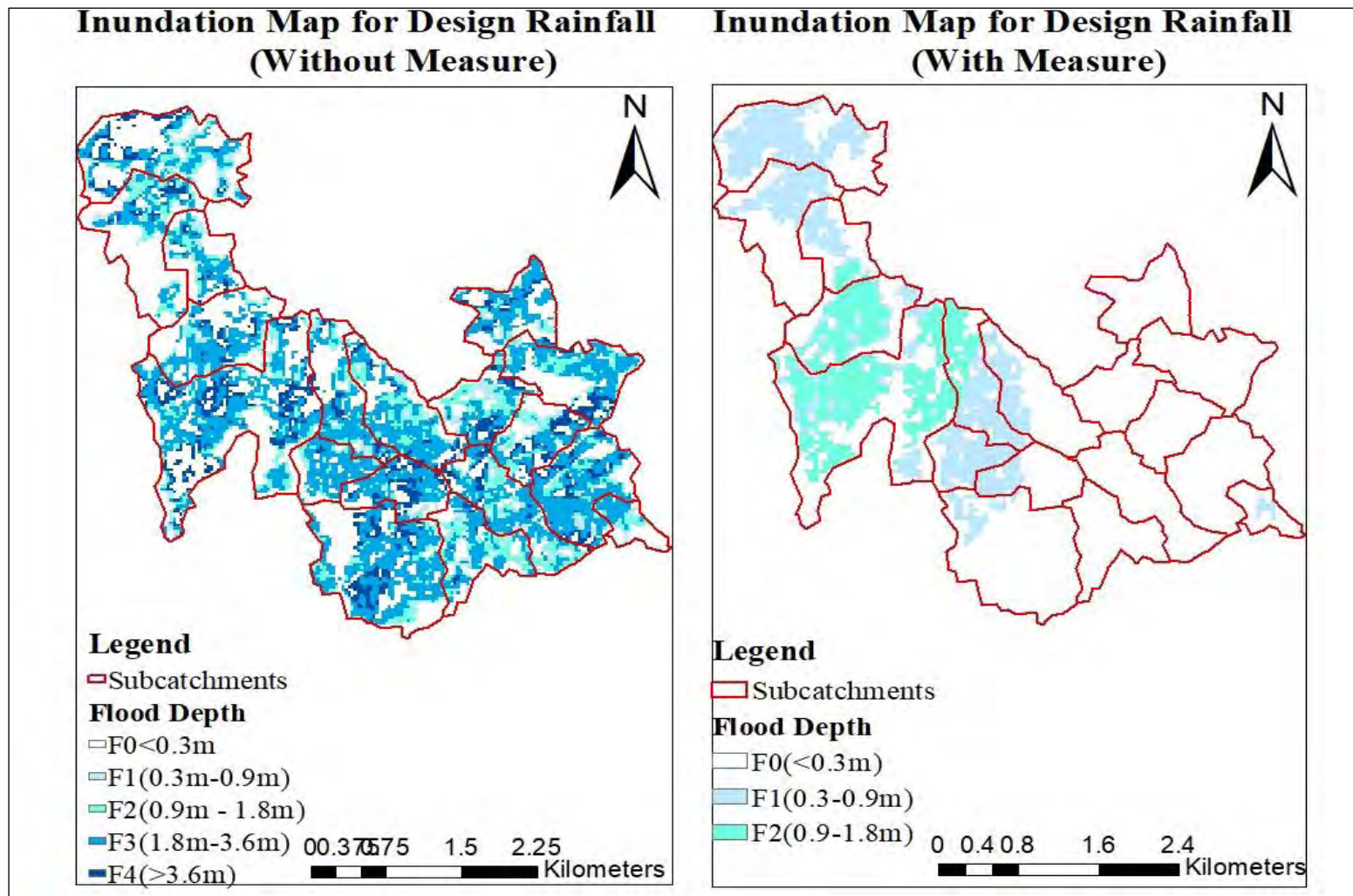


Figure 5.23: Comparison of inundation map before and after applying adaptive measure at Kanchannagar catchment

Table 5.7 Comparison of inundation depth of Kanchannagar catchment before and after applying adaptive measure

Inundation depth considering design storm event	Inundated area (ha)							
	Flood Free, FF	F0	F1	F2	F3	F4	Total Flood Free Area (%)	Total Flooded Area (%)
		Inundation up to 0.3m	Inundation 0.3 to 0.9m	Inundation 0.9 to 1.8m	Inundation 1.8 to 3.6m	Inundation > 3.6m	(FF+F0+F1)	(F2+F3+F4)
without any measure	579.46	0.93	73.75	183.02	531.11	131.73	654.14 (43.61%)	845.86 (38.70%)
with adaptive measure	892.3	152.31	283.45	168.94	0	0	1336.70 (89.11%)	168.94 (11.26%)

Impact Study:

Inundation map shown in Figure 5.23 portrayed that this measure decreased the inundation extent significantly. Now the channels can carry most of the design flows which ultimately helps in reducing the system flow loss or floodplain consumed volume. Table 5.7 showed that this modification along with proposed new sections can effectively reduce internal flood. It is found that total flood free area increased from 654.14 ha to 1336.70 ha (about 50% increase) and total waterlogged area decreased from 845.86 to 168.94ha (almost 80% reduction).

5.8.4 Impact Study of Pumping After Channel Modification at Kanchannagar Catchment:

A pump with capacities of 1, 2 and 3 cumec is applied to observe its impact on flooding extent of Kanchannagar catchment. The inflow hydrograph at pump inlet before and after pump application is shown in the Figure 5.24

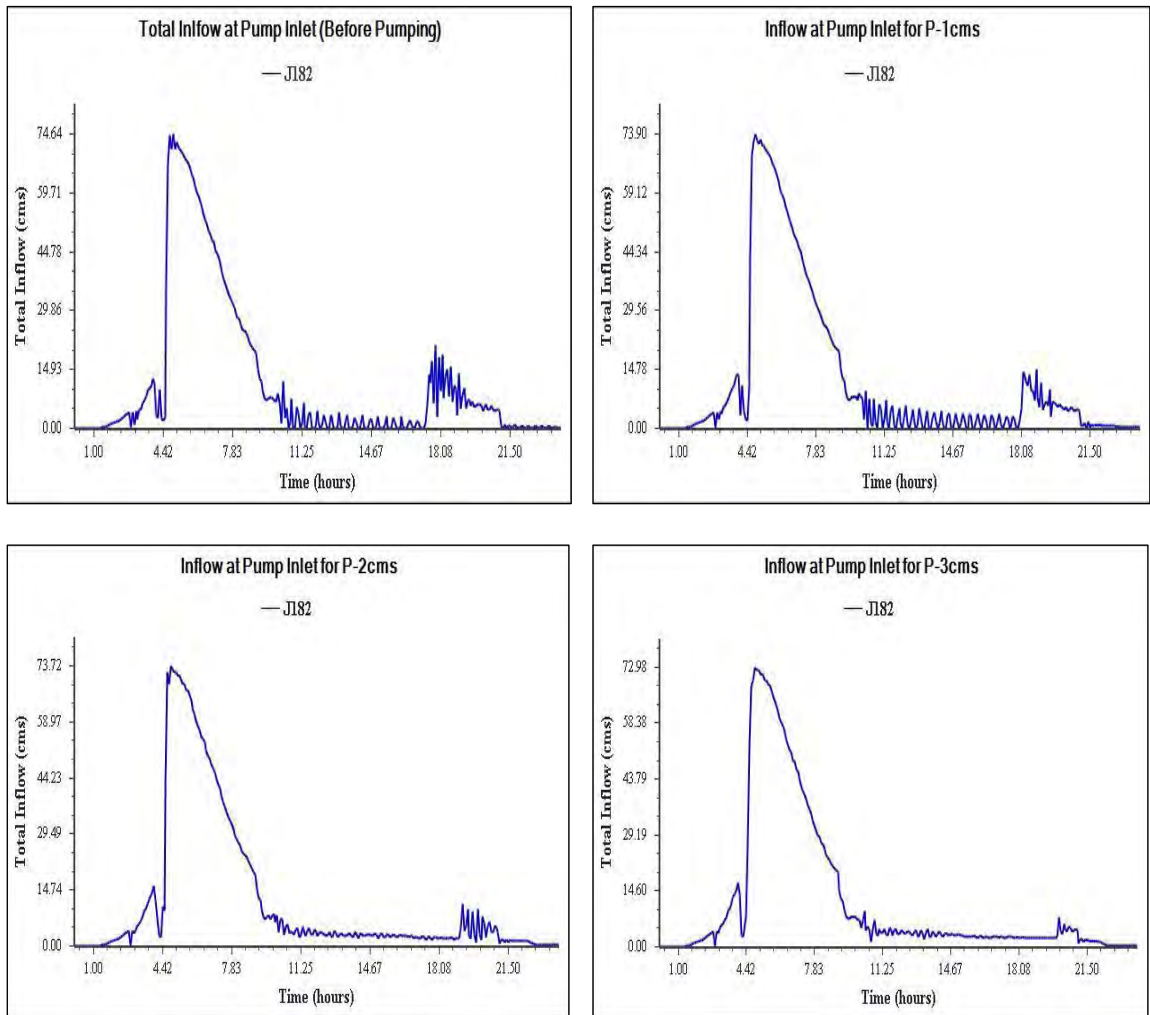


Figure 5.24: Inflow hydrograph at pump inlet, J182

From the Figure 5.24 it is seen that the peak value at pump inlet reduced by 0.74, 0.92 and 1.66 cumec with 1, 2 and 3 cumec pumping respectively and this facility also helps in reduction of system flooding loss and the results are shown in the following Table 5.8

Table 5.8: Model output for Kanchannagar catchment

	Existing condition without pumping	Channel re-excavation without pumping	Channel re-excavation with 1-cumec pumping	Channel re-excavation with 2-cumec pumping	Channel re-excavation with 3-cumec pumping
Surface runoff volume (m³)	1104740	1104740	1104740	1104740	1104740
Floodplain consumed volume (m³)	846630	139860	136900 (2.6%)	119400 (14.6%)	110280 (21.2%)
Volume carried by channels (m³)	258140	964880	967830	985340	994460

Table 5.8 depicted that, at existing condition a larger amount (846630m³) of surface runoff volume (1104740 m³) flooded the nearby areas. Channel excavation with some new proposed channels reduced this flooding loss and 139860 m³ of water now inundates the area which means with this improvement measure, most of the design flows is conveyed through the drainage channels and this helped in improving the inundation extent. The table also showed that pump application also worked as an effective measure to reduce more flooding volume. Pump with capacities of 1, 2 and 3 cumec is able to reduce floodplain consumed volume by 2.6%, 14.6% and 21.2% respectively.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions of the Study

In this research, the main task is to assess the drainage problem in the Polder 29 and find out the sustainable solutions to improve the condition. During monsoon and high tide, the river water level rises and the water from the canal cannot pass through the drainage sluices which causes inundation problem in this area. In this study, adaptive measures such as re-excavation of existing channels, proposal of constructing new channels and application of pumping are employed to improve the drainage condition. In near future, gravity drainage would be hampered and therefore drainage through pumps is considered here so that the natural channels can serve as the main drainage route to the river.

- The hydrological and hydrodynamic model Geo-SWMM is applied in this study to model Polder 29 comprising 7933 ha area. Water from the drainage channels mainly discharged to Ghengrail River in the west and Upper Bhadra in the east. The model is calibrated and validated using the observed water level data (both inside and outside) of Kanchannagar sluice.
- Rainfall data from 1948-2016 and water level data (1986-2016) for SW 28 and SW29 have been analyzed to select the design rainfall and design high and low tide level. From the analysis, 1-day maximum annual rainfall for 1 in 10 year return period was selected as design storm which was 216.35 mm. Design high and low tide level was found 3.66 and -0.66 mPWD for SW28 and 3.60 and -0.78 mPWD for SW 29. These values are used as a d/s boundary condition to simulate the model in design storm scenario.
- The analysis revealed that ten of the conduits are not adequate enough to carry the maximum flow and for them cross-sectional areas have been suggested to improve their capacity. For some conduits, water level at d/s node is higher than the u/s node and due to this flow cannot pass smoothly from u/s. In that case, gradient can be devised for those selected conduits to improve the condition.

- For validation of inundation extent of the model, actual inundation map of 27 August, 2016 from Sentinel-2 satellite image is compared with the inundation pattern found from the simulated model and the extent is adequately alike.
- Inundation map of whole study area has been prepared for the design flood event and it is found that 52.33% area is waterlogged (F2, F3 and F4) where depth ranged from 0.9 m to greater than 3.6m and 47.68% area (FF, F0 and F1) is flood free totally (depth 0 to 0.9m) which can be used for agricultural purposes.
- After using pumping facility (ideal pump) as an improved option, it is found that the total flood free area (FF, F0 and F1) increased from 47.68% to 62.67%. About 17.2% and 17.86% area lies under land classes F2 and F3 (depth 0.9-3.6m) respectively and only 2.27% area lies under deep inundation zone (F4) in this improved option.
- To identify the actual extent of reduction in inundation depth, the largest Kanchannagar catchment is modeled separately. Re-excavation of existing drainage channels and proposal of constructing new channels at high inundation depth region have been suggested to lessen the drainage congestion problem. The analysis revealed that, waterlogged area decreased by 80% compared to the existing condition whereas total flood free area increased from 43.6% to 89.11% for the largest catchment.
- The flooding loss was about 139860 m³ with this adaptive measure and 136900 119400 and 110280 m³ after utilizing pump with capacities of 1, 2 and 3 cumec respectively.

6.2 Recommendations for Future Study:

In this research work, analysis based on real data has been carried out and the results have already been discussed. Some recommendations can be summarized as part of the ongoing researches. The recommendations have been summarized below:

- In this study surveyed cross-sections of most of the canals are used and some are assumed based on DEM and google earth image. So to understand the actual inundation condition with precision and to suggest adaptive measure fine resolution DEM, actual cross section of all drainage channels, invert level of

drainage sluices, tidal water level of all outlets and the accurate closing and opening time of flapgates need to be known.

- As there is a possibility of urbanization in this area, therefore future land use scenario should be taken into account.
- Some future improvement scenario such as, remodeling of drainage sluices including addition of new vent and modification of invert level, possibility of constructing retention pond as a storage and its effect can be considered in future.
- Plinth rising and elevating the local habitats and physical infrastructures above the maximum water level of each subcatchments found from the simulated model considering various return periods can be other probable solutions for this study area.

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APPENDIX A

A.1 Existing status of sluice gates and drainage channels of Polder 29



Figure A.1: Kanchannagar sluice



Figure A.2: Newly constructed gate inside the Bokulatola khal



Figure A.3: Sluice gate at Telikhali Old



Figure A.4: Sluice gate at Telikhali New



Figure A.5: Golaimari sluice



Figure A.6: Asannagar sluice covered by water hyacinths



Figure A.7: Silted up course along the Aro khal



Figure A.8: Narrow course of Asannagar khal



Figure A.9: Narrow course of Golaimari Khal



Figure A.10: Silted up bed of Kata khal

APPENDIX B

B.1 Tidal water level series at outfall locations for 1 in 10 year return period:

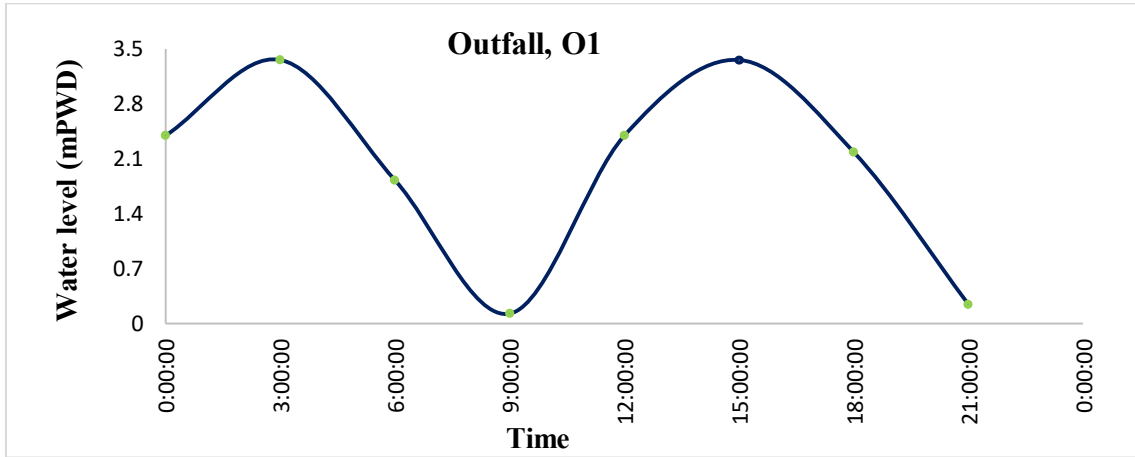


Figure B.1: Tidal water level at outfall, O1

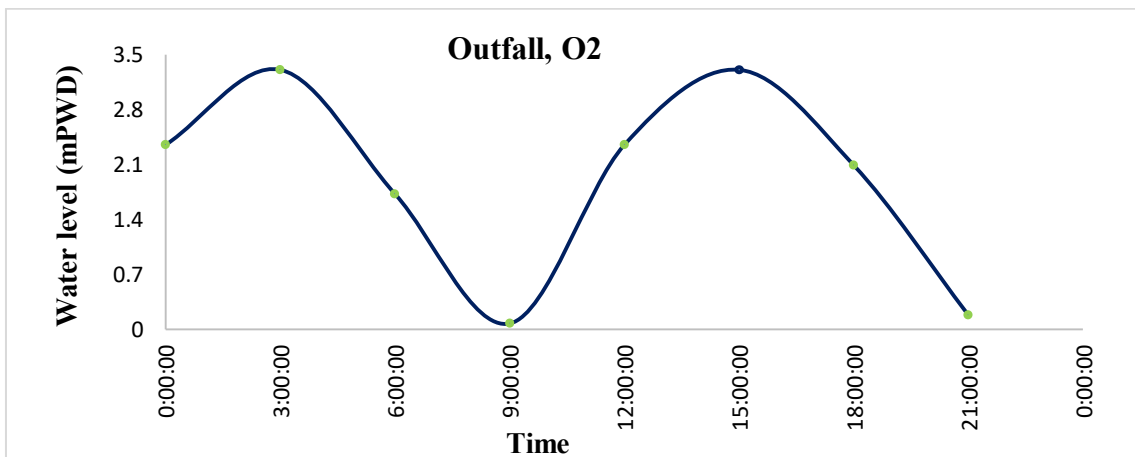


Figure B.2: Tidal water level at outfall, O2

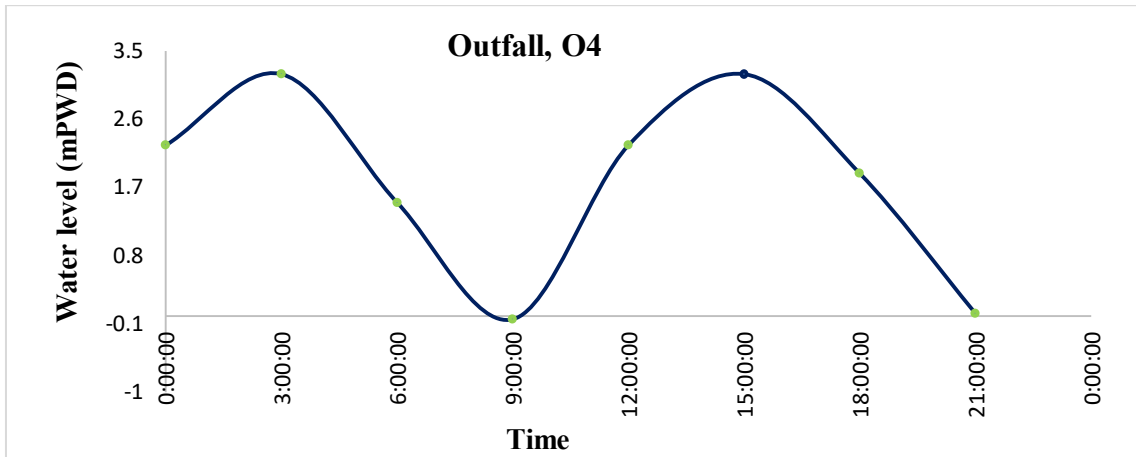


Figure B.3: Tidal water level at outfall, O4

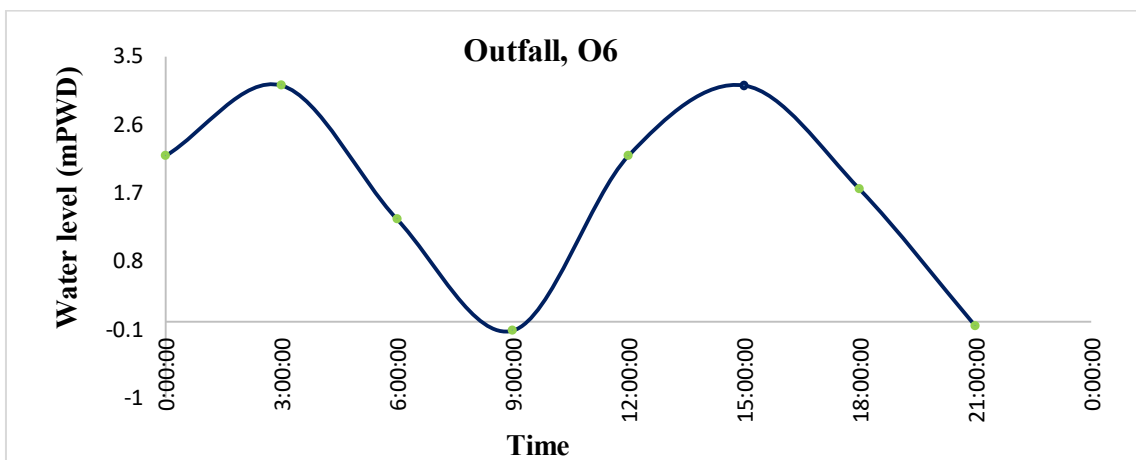


Figure B.4: Tidal water level at outfall, O6

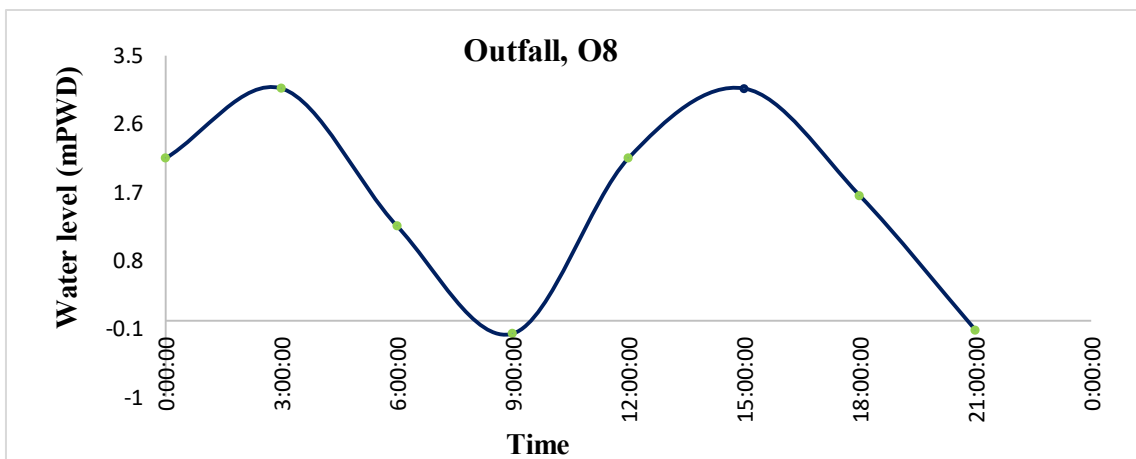


Figure B.5: Tidal water level at outfall, O8

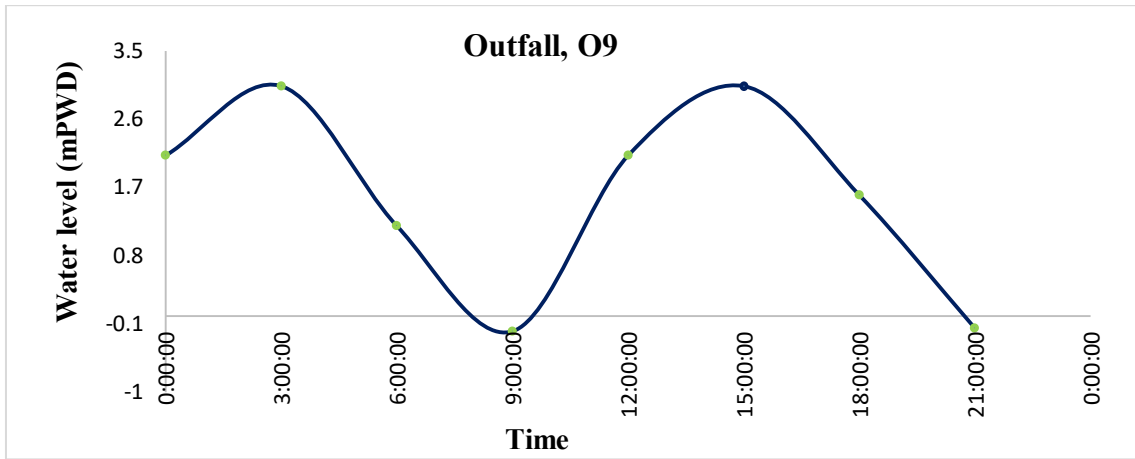


Figure B.6: Tidal water level at outfall, O9

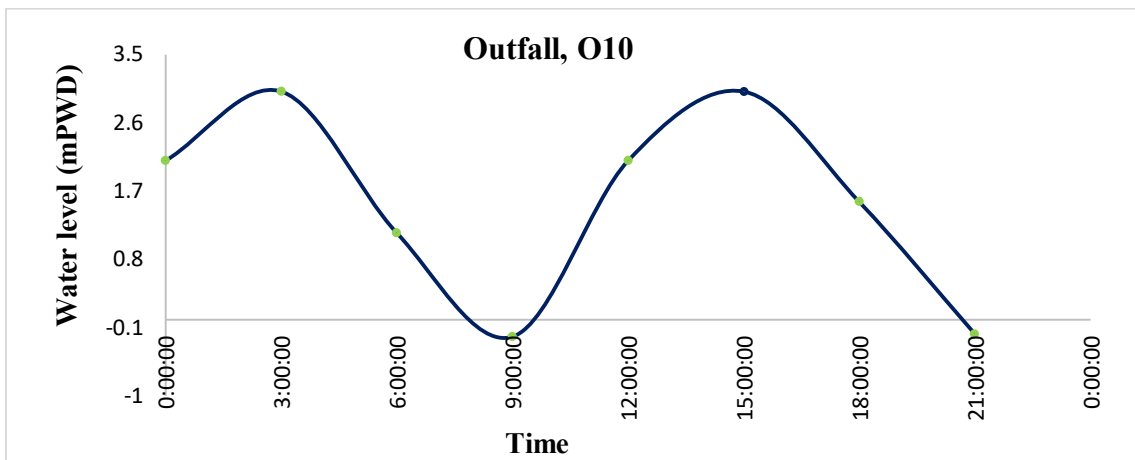


Figure B.7: Tidal water level at outfall, O10

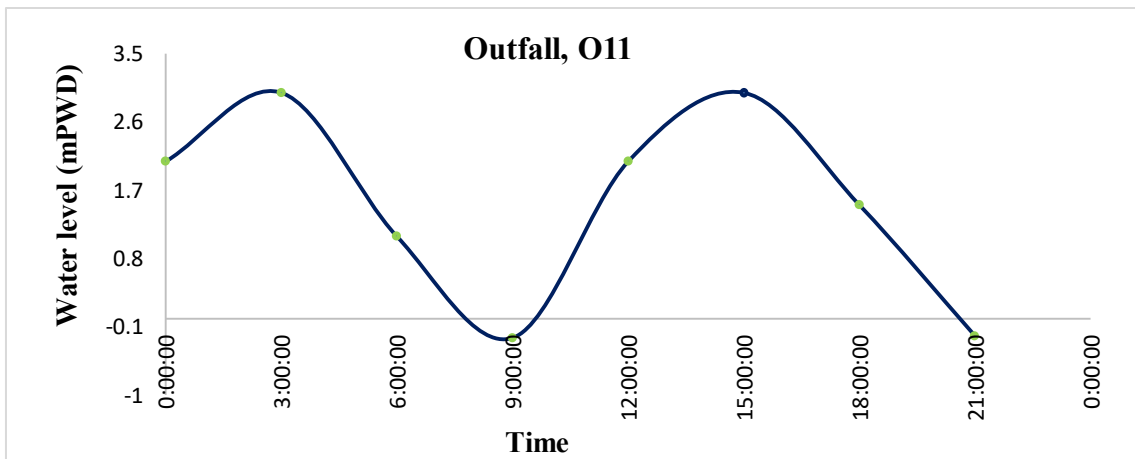


Figure B.8: Tidal water level at outfall, O11

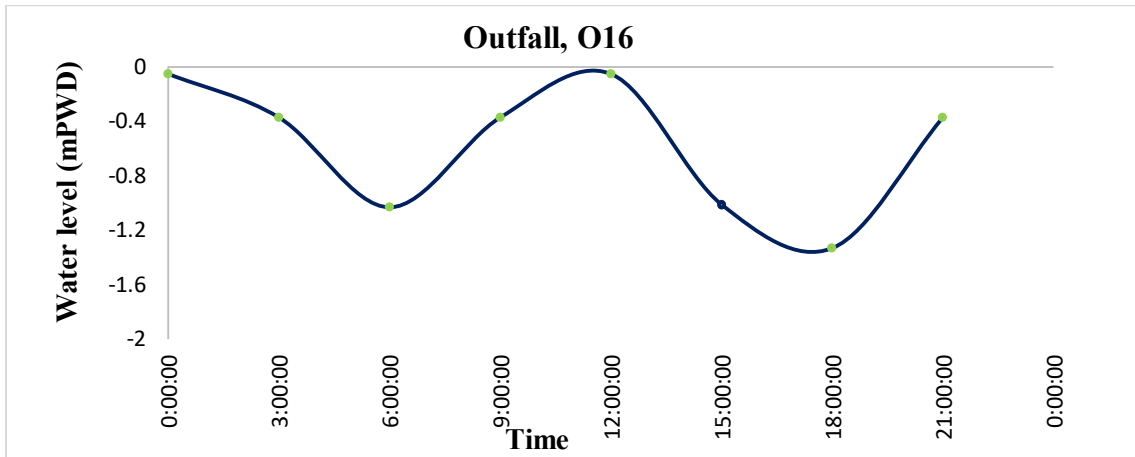


Figure B.9: Tidal water level at outfall, O16

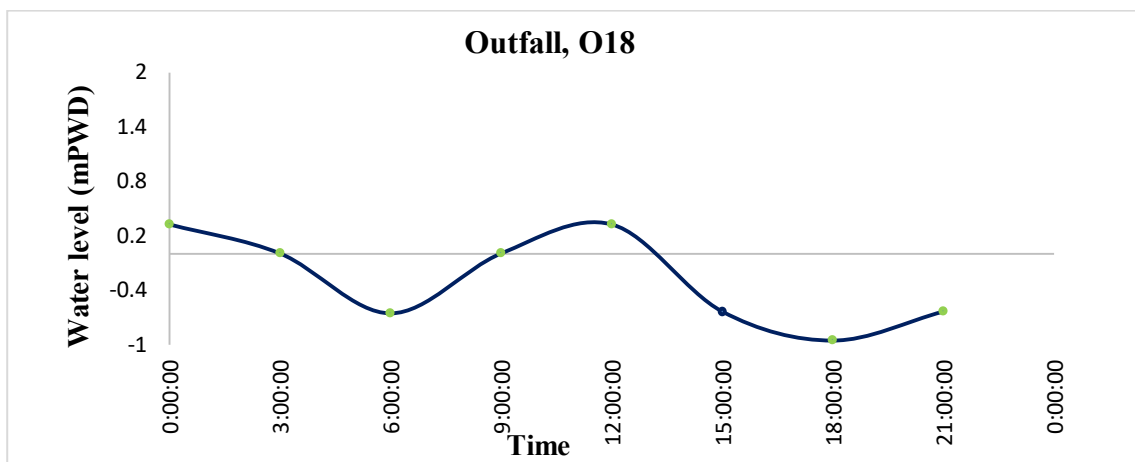


Figure B.10: Tidal water level at outfall, O18

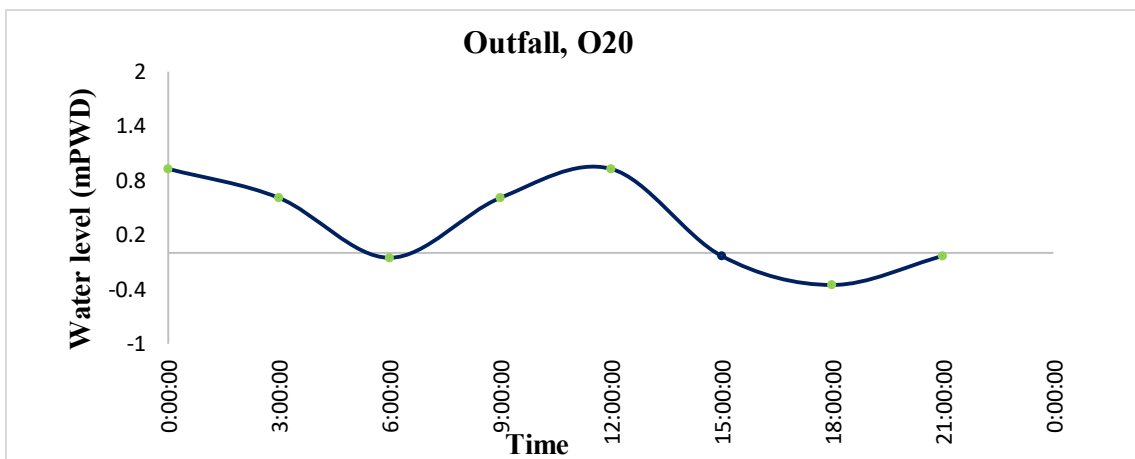


Figure B.11: Tidal water level at outfall, O20

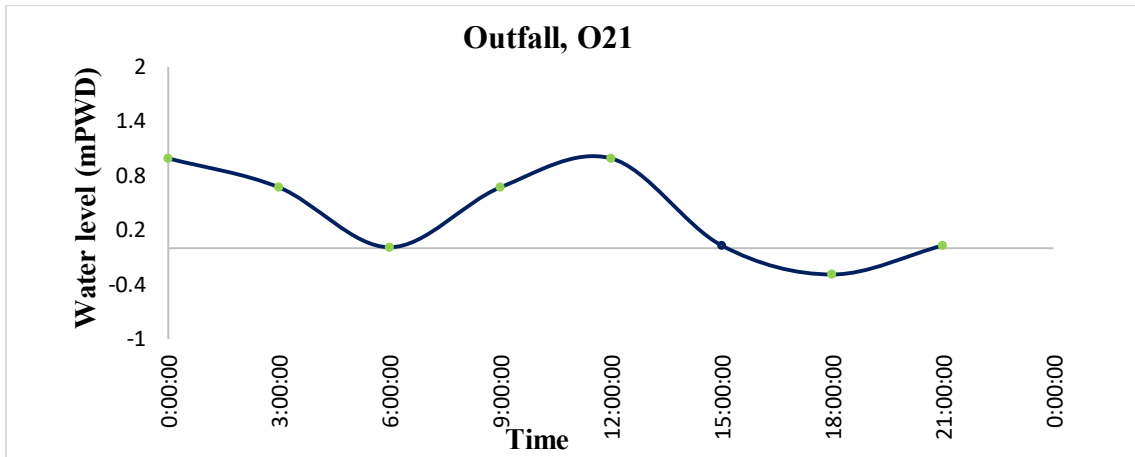


Figure B.12: Tidal water level at outfall, O21

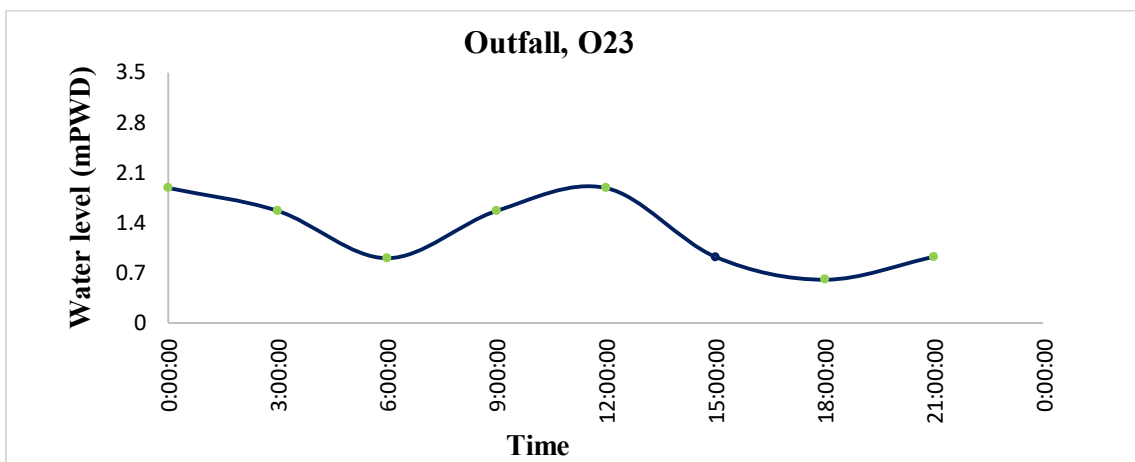


Figure B.13: Tidal water level at outfall, O23