TO DEVELOP FLOOD FORECASTING APPROACH OF AHMEDABAD, GUJARAT, INDIA

A Thesis submitted to Gujarat Technological University

for the Award of

Doctor of Philosophy

in

Civil Engineering

by

Ujas Deven Pandy
149997106023

under supervision of

Dr. Dhruvesh Prahladbhai Patel

GUJARAT TECHNOLOGICAL UNIVERSITY
AHMEDABAD

January – 2020

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ABSTRACT

Flood is disastrous for developing countries, its assessment are important for flood risk management. Ahmedabad is largest city and former capital of Gujarat and included under the 100 Smart city project of India is situated 151.6 km downstream for Dharoi dam. The city has experienced floods in the year 1973, 1983, 1988, 1991, 1993, 1998, 2003, 2004, 2005 and 2006. Going by statistics the city has experienced flood in almost consecutive three to four years. In addition, AMC has developed the Sabarmati River Front along the banks across the city which narrows the river section. Whether any study about risk of flood vulnerability has been done prior to the implementation of the project is not clear. How the river will react during flood situation is a matter of speculation for general public. Therefore, it is prime issue for Ahmedabad city to produce the accurate flood inundation map which predicts the depth and submergence areas of city. In year 2006, 8800 cumec water was released from Dharoi dam during monsoon due to continuous rainfall in the upstream of the Dharoi dam, Sabarmati River basin, resulting in the inundation in low lying area of Ahmedabad city. Gumbel's flood frequency analysis has been carried out using last 35 years annual discharge data to calculate discharge for various return periods till 100 years. The open source HEC-RAS model is one of the most popular hydraulic models. In 2014 a new version of HEC-RAS (HEC-RAS-v5) was released including 2D capabilities and having capabilities to import geometric data from HEC GeoRAS, hence used in present study to develop 1 D and 2D hydrodynamic model.

Present study describes the development of 1D and 2D hydrodynamic model using DEM generated cross sections of river for flood mitigation studies. 1D hydrodynamic model is generated for 39 km length of Sabarmati River from Chiloda Bridge to Vasana barrage covering two most important cities of Gujarat state, i.e. Ahmedabad and Gandhinagar. The River geometry has been generated using ALOS and Cartoset-1 digital elevation models. The cross section of the river has been derived at every 200 m using HEC GeoRAS software and validated with data collected from state and central government departments as well as field survey. The boundary condition for Chiloda Bridge and Vasana Barrage as upstream and downstream respectively has been fixed and model is simulated for flood event of year 2006 under the unsteady flow conditions. As outcomes, the discharge, water surface elevation, velocity and flow area has been derived and analyzed. Afterward the simulated flow and stages at known section is compared with the observed data and shows a significant correlation. When water begins to overflow it becomes a 2-D phenomenon and hence 2-D modeling is carried out. The present study is applied the new HEC-RAS-v5 to simulate the August 2006 flood event in the Sabarmati River. The results of 2D

simulation in form of maps for hydraulic properties like, depth of water, water surface elevation, velocity, arrival time and inundation area have been studied. The Stimulation showed many areas of the city getting inundated due to this event. It also provides important information like maximum depth and flow velocity of the flood for corresponding discharge at Chiloda Bridge. Model was run from 18th August, 2006, 18:00 hours to 23rd August, 2006 23:00 hours. Maximum depth, Velocity, inundation area and WSE observed have been mapped with Google map of Ahmedabad and Gandhinagar city for preparing flood inundation probability maps which can be utilized for flood mitigation and management for Ahmedabad city and preparing Emergency Action Plan (EAP) for city. The areas located on both the banks of river has been classified under high risk and low risk categories as per their chances of getting inundated for discharge equal to year 2006 flood event.

In the present analysis, the application of a HEC-RAS and HEC-GeoRAS open source software have been applied for prediction of a river stages and inundation area mapping, hence applicable for flood mitigation and management in developing countries under a scarcity of data, fund and skilled human resources. An evacuation plan for inhabitants can be planned based on flood inundation maps. The results generated from simulations can be further utilized for forecasting about inundation area and other hydraulic properties for corresponding discharge from upstream.

Acknowledgement

Undertaking this PhD has been a truly life-changing experience for me and it would not have been possible to do without the support and guidance that I received from many people. I am using this opportunity to express my gratitude to all those who supported me directly or indirectly all through the completion of this doctoral education.

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Ujas Deven Pandya

Table of Content

List of Abbreviation

NDMA	National Disaster Management Authority
DEM	Digital Elevation Model
DTM	Digital Terrain Model
1D	One Dimensional
2D	Two Dimensional
SWDC	State Water Data Center
AMC	Ahmedabad Municipal Corporation
CWC	Central Water Commission
GIS	Geographic Information System
DRIP	Dam Rehabilitation and Improvement Project
GOG	Government of Gujarat
GOI	Government of India
BISAG	Bhaskaracharya Institute for Space Applications and Geo-informatics
AUDA	Ahmedabad Urban Development Authority
MCM	Million Cubic meter
NH	National Highway
RL	Reduced Level
ALOS	Advanced Land Observing Satellite
ISRO	Indian Space Research Organization
WSE	Water Surface Elevation
XS	Cross section
LOB	Left of Bank
ROB	Right of bank

List of Figures

List of Tables

List of Annexure

CHAPTER – 1

INTRODUCTION

1.1 Background

Flood is defined as enormously high flows of water of surface water bodies like, rivers, streams, lakes, ponds, reservoirs, due to which water spills over the water bodies and inundates in surrounding area. As per, (NDMA, 2008), flood is additional water inundated due to insufficient capacity of rivers to carry a high volume of water from the upstream area within their banks following heavy rainfall . Among all the natural catastrophic events occurs worldwide, floods are the most recurrent and destructive to social, economical and environmental aspects of the vicinity. Among many reasons of floods, includes high intensity precipitation in catchment, change in river cross sections due to sedimentation, sudden dam failure, release of high discharge from dam etc.

Depending upon various factors like, velocity, geography and causes, floods may be mainly classified as, Fluvial (River) flood, ground water flood, pluvial flood and Surge (Coastal) flood. Fluvial flood results due to water spills over either or both the banks of river when discharge in river exceeds its normal water carrying capacity. The cities located on banks of river may face water logging due to this type of flooding. Pluvial flooding results from inefficient or inadequate urban drainage with respect to the corresponding rainfall in an urban area. In such cases of pluvial flood, existing drainage system of area is not capable of disposing high intensity torrential rain water and resulted in water logging in area. Groundwater flooding results where water table is higher and that water get inundated on surface. Surge or Coastal flooding occurs due to high tidal waves and affect the areas located on coastal shoreline.

India is one of the worst flood-affected countries in the world, as it is surrounded by the Arabian Sea, Indian Ocean and the Bay of Bengal. In India, major flood-prone areas occupy approximately 12.5 % of the country according to the Indian Geological Survey. Also, as per the National Flood Committee, nearly 40 million hectares area in India is vulnerable to flood. In India, among all natural hazards, river floods are most recurrent and often destructive causing extensive losses of infrastructures, cultivation, transportation, community health, livestock and human lives (Alam & Muzzammil, 2015). Gujarat has seen various damaging floods. Almost all major rivers in Gujarat pass through a wide stretch of very flat terrain before meeting to the Ocean. These flat lands of lower river basins are prone to be flooded every year.

Flood causes considerable damage to human lives and property almost every year. Since adoption of National Flood Policy by Government of India in 1954, it was realized that a total protection against flood by structural means alone is not possible and that optimum solution would consist of a mixture of structural and non-structural measures. Therefore, stress has been laid on non-structural measures like flood forecasting and warning, which is most important among such means to minimize the damage potential from floods (NDMA, 2008).

1.2 Hydrodynamic Modeling

In recent decades, the use of Geographic Information System (GIS) along with the hydraulic model has been used effectively for flood management and forecasting. GIS being capable of representing topographic features and hydraulic model being efficient in simulating flow for various return periods, all together they both give very good results, which can be further used for preparing flood risk and hazards maps. (Demir & Kisi, 2016). Nowadays researchers are using various hydraulic models in combination with GIS to simulate water flows. Many software packages like DWOPER, FLDWAV, MIKE-11, ISIS, SOBEK, CCHE2D, TUFLOW, Infoworks-2D, RiverFLO-2D etc. have been widely applied for simulating 1D and 2D flow in rivers (Taylor, 2012) (Bellos, 2012). Researchers like, (Timbadiya et al., 2014a) and (Pramanik et al., 2010) have developed a stage-discharge relationship using 1 dimensional MIKE 11 hydrodynamic model for lower Tapi river, India and Brahmani river, India respectively. Among all the available hydrologic models, freely available HEC-RAS (Hydrologic Engineering Center, River Analysis System) model is very handy means for forecasting of likely flood events to occur in future and used worldwide for flood prediction and management (Yamani et al., 2016). HEC-RAS is efficiently used worldwide to develop flood risk assessment and management for rivers like, Pahang River, Malaysia (Zainalfikry & Ghani, 2018) , Mert River, Turkey (Demir & Kisi, 2016), Martil River, Northern Morocco (Azouagh et al., 2018), Al Kahlaa and Hilla Rivers of Iraq (Awad, 2015a) (Luay Kadhim & Tawfeek, 2013) and many more. Also in India HEC-RAS based hydrodynamic model has been effectively developed for many rivers like Mahanadi (Parhi, 2012), Tapi (P. V. Timbadiya, Patel, & P.D., 2014) (Patel et al., 2017) and Yamuna (Kumar et al., 2017).The 2D modeling capabilities of HEC RAS can be effectively applied for floodplain mapping. Many researchers have effectively developed 2D HEC-RAS model for floodplain and urban areas using 90 m and 60 m resolution DEMs. (Nandurkar, More, & Despande, 2017) and (Quirogaa, Kurea, Udoa, & Manoa, 2016a)have developed 2D HEC RAS model for Pune city and Bolivian Amazonia using 30 m grid and 90 m grid SRTM DEM respectively. Application of DEM for development of efficient 2D HEC RAS model is widely performed in research community. Some of them are models of Indus River system, Pakistan using ALOS world 3D of 30 m grid (Rind, Ansari, Saher, Shakya, & Ahmad, 2018), Lower Mekong River, Cambodia using 30 m grid SRTM (Thol, Kim, Ly, Heng, & Sun, 2016), Dharla river and its floodplain, Bangladesh using 90 m grid SRTM and Lower Tapi basin and floodplain, Surat using 30 m and 90 m SRTM (Patel et al., 2017) which shows good agreement between integration of DEM generated geographic data and 2D HEC RAS hydrodynamic modeling for floodplain mapping.

1.3 Hypothesis/Problem Definition

The average rainfall of Ahmedabad is about 98.2 cm. Infrequent heavy torrential rain causes flood to the Sabarmati River and the city has experienced floods at every two to three years interval. Ahmedabad reach a record high flood level at 47.45 m at Subhash Bridge on 19th and 20th August, 2006 which is even more than the past recorded level of 44.09 m in year 1993. The city remains water logged for almost two to three days. Ahmedabad city is having mostly flat terrain except some part like Thaltej which is having little higher elevation than other part of city. This topographic condition of almost flat terrain makes the inundation condition even worse. The flood has affected the areas near Vadaj, Bapunagar, Naroda, Dani limada, Paldi, Ghatlodiya and Khanpur covering both the left and right banks of Sabarmati River.

For this study, after discussion with officials of state irrigation department and Ahmedabad municipal corporation followed by literature review the problem is defined as that release of heavy discharge such as of 8800 cumec (approximately 3 Lac cusec) from Dharoi Dam along with parallel rainfall in Ahmedabad and other in between Cities, made difficult to dispose accumulated water in the city causes water inundation in low lying area of city. As shown in Fig. 1, for release of 8800 cumec discharge from Dharoi dam in year 2006, the water surface elevation at Subhash Bridge and Vasana barrage almost reached their high flood levels of 45.34 m and 41.76 m respectively.

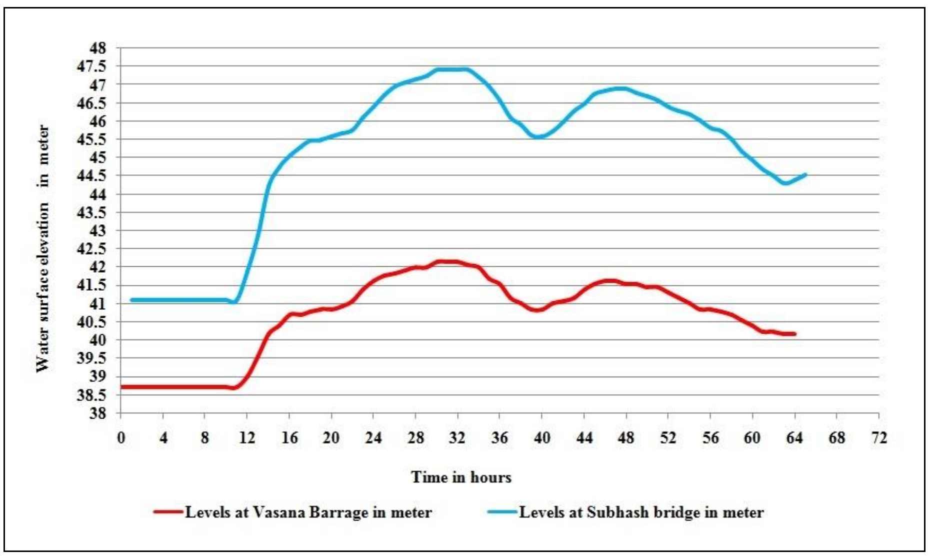


Fig. 1.1 Hydrographs at Subhash Bridge and Vasana Barrage for flood event of year 2006

To test this hypothesis, flood event of year 2006 has been considered and one dimensional and two dimensional hydrodynamic models have been developed for city to prepare inundation maps for different return periods.

1.4 Objectives

Following are the main objective of the present research;

- To develop 1 D unsteady model of Sabarmati river to predict stages at various locations along the study area.
- To analyze stages of river for various return period using 1 D steady hydrodynamic model
- To develop 2 D hydrodynamic model of Sabarmati river for flood inundation mapping
- To calibrate manning's roughness coefficient for study reach of Sabarmati river
- To develop flood inundation maps for Ahmedabad city for preparation of framework for flood forecasting and warning system.

1.5 Methodology

The study started with the review of literature of the past flood events of Sabarmati River and Ahmedabad and followed up by discussion of application of various models and their application for flood prevention and management activities of recent decades with officials of irrigation department and of Ahmedabad municipal corporation.

After studying literature regarding flood history of Sabarmati River and Ahmedabad city, the flood frequency analysis has been carried out using Gumbel's flood frequency analysis. In this analysis, annual peak discharge of past 35 years from 1981 to 2015 has been considered and following Gumbel's steps mean, standard deviation has been calculated. Flood flow for different return periods has been calculated using Gumbel's co-efficient K, mean and standard deviation. The graph has been plotted for return period versus flow discharge and best fit function for this graph comes out as 5^{th} order polynomial having R^2 as 0.995. This function has been used further in study to calculate future discharge for various return periods to run 1D steady flow in HEC RAS

For this study integration of Arc Map 10.0.1, HEC GeoRAS 10.0 and HEC-RAS 5.0.1 has been used to create geometric data and simulation of 1D and 2D hydrodynamic models of study reach of Sabarmati River. It has been observed from literature study that generally all hydrodynamic models are developed by taking mostly physically surveyed geometric data or by collecting surveyed data from concern government agencies. During data collection from various state and central government departments like SWDC, CWC and AMC it has been faced and observed that the availability of latest and authentic surveyed data specially required for development of hydrodynamic model is a major challenge. Hence, DEM generated geometric data produced in HEC GeoRAS has been used in this study for simulation of 1D and 2D HEC RAS hydrodynamic models. The geometric data produced using DEM has been validated with data from government agencies and data collected by field survey.

This study aims to develop 1D and 2D hydrodynamic model for study reach of Sabarmati River using DEM generated geometric data with the help of HEC GeoRAS and HEC RAS 5.0.1. Initially flood event of year 2006 has been simulated under unsteady condition by considering Saint-Venant equation for both 1D and 2D simulation. The 1D hydrodynamic HEC RAS model is simulated for unsteady condition to study and compare the stages of Sabarmati River in length of 39 km from Chiloda Bridge to Vasana Barrage. To produce river geometry, elevation data at every 200 m interval for the total 39 km length of study area has been extracted using HEC Geo RAS open source software using 10 resolutions Cartoset-1 DEM. For simulation of HEC RAS model, flow hydrograph at Chiloda Bridge and normal depth at Vasana Barrage has been considered as upstream and downstream boundary conditions respectively. The 1D model is calibrated for Manning’s roughness coefficient in range of 0.020 to 0.040 for year 2006 and validated for year 2007 as per guideline of DRIP, Govt. of India and Chow (1959). The 39 km long study reach has a very gentle slope. As there is no sharp curvatures has been observed, effect of meandering has been neglected by providing expansion and contraction coefficient as 0.3 and 0.1 respectively. Simulation of model under unsteady condition will give result in terms of water surface elevation, discharge, velocity, flow area and energy gradient slope. The study of over-spilling of water from both the banks has been carried out by comparing water surface elevation for maximum discharge with ground elevation of both the banks and based on its outcomes, areas to face inundation has been identified.

The 1D model computes water surface elevation and depth of water. It does not determine the direction. That is quite insufficient to actually determine which direction that water's going to go, which the 2D modeling can be very useful to in reality determine which direction the flow going to go. HEC-RAS 2D has a GIS interface and applies the finite volume method to solve unsteady flow equations that describe the two-dimensional nature of the flow (Gary W. Brunner, 2016). For this research 10 m grid Cartoset 1 DEM is used to create terrain and 2D flow area is delineated on this terrain with 10 m computational mesh. The upstream and downstream boundaries have been considered same as of 1D HEC RAS modeling. The value of Manning's roughness coefficient n is also a very important parameter that can be used in calibration of the two-dimensional model. In the land cover map prepared with the help of BISAG, manning's roughness values have been specified as per guideline of DRIP, Govt. of India with various land use categories like wasteland, agriculture, plantation, built up areas of high, medium and low intensity, river, canal, water bodies etc. in land cover layer. This land cover layer with relevant manning's roughness co-efficient has been associated with terrain data of study area which results in association of a Manning's roughness value with each computational cell faces. The 2D model has been simulated for year 2006 flood event with computational interval of 10 seconds. After simulation, various maps for parameters like discharge, velocity, water surface elevation, inundation boundary has been extracted for maximum discharge profile. All these parameters have been extracted for 20th August, 2006, 17:00 hours having maximum discharge value at Chiloda bridge. The flood inundation maps have been prepared for various discharges by mapping simulated parameters with ward map of Ahmedabad city.

Two different DEMs of resolutions 30m and 10m has been used to simulate 1D unsteady model and simulated values of stages at Subhash bridge gauging site has been compared with actual values to study and observe effect of 200m and 300m cross section spacing for both the DEMs of 30 m and 10m resolutions. Observed and simulated stage hydrographs at Subhash Bridge have been evaluated in terms of statistical parameter like root mean square error (RMSE), % error, mean absolute difference and mean difference. The result shows that model is highly sensitive towards grid interval of DEM used for creation of geometric data along with the spacing of cross section interval considered. Also it has been observed that if the cross section spacing has been decreased beyond optimum value, the flow become supercritical and gives unrealistic outcomes in terms of water surface elevation. This analysis will be helpful in deciding most appropriate cross section spacing for DEMs of 30 m and 10 m resolutions. Sensitivity of model has been analysed by comparing simulated and observed stages of river at Subhash Bridge using geometric data generated through two DEMs of 30 m resolution and 10 m resolutions along with cross section spacing of 300m and 200 m interval for 2006 and 2007 flood events considering manning's roughness coefficient ranging from 0.020 to 0.040.

1.6 Scope of study

- To provide base information for preparing the emergency action plan for Ahmedabad city for flood warning and mitigation management.
- Flood inundation map is useful for the local planning authority, engineers, rescue management system for planning for flood vulnerable areas and further utilized for developing forecasting system of city.
- The results will be useful in identification of the gap in the available infrastructure for prioritization of implementation of new infrastructure and plan for appropriate flood management and rescue system.
- It will be useful for the decision making process for all urban infrastructure development.

1.7 Thesis outline

The thesis contains total of 8 chapters. The 1st chapter gives brief introduction of entire work including general background, problem definition, objectives, methodology adopted and scope of study. The 2nd chapter contains review of literature for flood frequency analysis, various hydrodynamic modeling and integration of HEC GeoRAS and HEC RAS for development of 1D and 2D hydrodynamic model for channel. The chapter also contain case studies of 1D and 2D HEC RAS modeling in Indian and global scenario. The 3rd chapter discuss about study area of Sabarmati River and Ahmedabad city, flood history and major structures located on the Sabarmati River for the patch considered for study. The generation and collection of hydrological, spatial and geometric data is also illustrated in this chapter. The 4th chapter discussed about the methodology followed for study of flood frequency analysis and development of 1D and 2D hydrodynamic HEC RAS model. The 5th chapter discussed about results obtained from flood probability analysis and 1D and 2D simulation and their analysis. The 6th chapter demonstrates calibration of model in terms manning's roughness coefficient and its validation in terms of stage hydrograph at Subhash Bridge gauging site. The 7th chapter discuses about sensitivity of HEC-RAS model for different resolutions terrain and cross section spacing. The final 8th chapter consists of conclusions derived from results obtained and further recommendations to improve and strength work.

CHAPTER 2

LITERATURE REVIEW

2.1 Flood History

Floods are one of the most severe and most damaging environmental threats which harm facilities, government and private systems, the climate, the economy and human settlements to a large extent. The economic development in developed and developing countries has been hampered by frequent flood failures. Floods are a recurring occurrence that causes tremendous loss of life and harm to the economy, properties, infrastructure and public utility structures. The trend towards damage due to floods is increasing. It is a cause for concern. The average annual flood damage for the last 10 years is nearly Rs. 5000 crore, the highest over the previous 50 years compared to Rs. 2000 crore. This is due to many reasons, including rapid population growth and urban planning combined with increased economic and development activities in the flatlands and global warming (Alam & Muzzammil, 2015).

During periods of rain or snow, some of the water is stored in lakes or ground, some are consumed by plants and trees, others evaporate, and the remainder flows over the field as a surface runoff. Floods occur when the water cannot be drained by the swamp, ponds, streams, soil and vegetation. Water then flows off the surface in volumes that cannot be carried by rivers or contained in natural ponds, lakes and man-made dams. The floods of the river are usually attributed to heavy rain and sometimes to increased snowfall. A flood that rises rapidly, with little or no advance warning, is called a flash flood. Flash floods usually result from intense rainfall over a relatively small area, or if the area was already saturated from previous precipitation.

Hydrologist define flood as, "sudden increase in water discharge causing rise in water level peak in the channel or floodplain". The flood is commonly defined as, "temporary or permanent impounding of water due to overflowing of water from channel occurs when a normally dry land areas are temporary inundated due to overflowing of water at the natural or artificial confines of a river". When water in channel exceeds its carrying capacity, the surplus water spills over the banks of the channel and inundated the neighboring lower elevation area. This condition usually occurs in monsoon when river gets surplus water due to prolonged or heavy precipitation or due to release of water from storage structure located on upstream part of river. Floods are caused by many factors: heavy precipitation, severe winds over water, unusual high tides, tsunamis, or failure of dams, levels, retention ponds, or other structures that contained the water

Based on event, place and source floods can be mainly classified as River flood (fluvial flood), urban flood (Pluvial flood), Flash flood and coastal flood. The pluvial or urban flood occurs mainly because change in runoff action because of development of impervious surface due to building and roads having lesser infiltration capacity than natural plain. Also, torrential rain and inefficient drainage network makes the flooding condition worse in city area. Coastal flooding is known as the flood which takes place in the coastal area because of ocean waters. Unusually high levels of water in the ocean can be introduced to the land resulting in the coastal flooding due to natural events such as a tropical storm, a cyclone or concentrated offshore low pressure. Likewise, tidal effect taking place in the ocean due to earthquakes and volcanic action can also add to coastal flooding. Flash floods arise in a short period of time when there is a large amount of rain surge. This normally happens without or with no notice locally quite unexpectedly. Because of torrential rainfall or the rapid release of water from a dam, flash floods can occur.

India is situated in South Asia's stormy climate zone. The circulation of monsoon creates nearly 80-90 percent of the annual rain in June to September, over most areas of the nation. During this cyclonic time of unsettling impacts from the Middle Ocean and Bay of Bengal, the precipitation in Indian rivers is wider and overwhelming and frequently increases. Records show that in India there have been approximately 210 severe surges with real outcomes between 1966 and 1955, with close to seven surges each year. Of these, some massive floods overflowed the rivers and plunged many arrivals and humans (Rakhecha, 2002)

As per (NDMA, 2008), an average, every third year, India is hit by extreme floods. More than 40 mha is vulnerable to flooding out of a maximum geographical area of 329 million hectares.

In an average 75 lakh of land is affected by floods and more than 1600 livelihoods per year are lost. The annual average damage to crops and public services exceeds Rs. 1800 crores. The monetary value of flood damage shows a growing trend. Over the last five years the annual average losses for the last 10 years (1996-2005) were Rs. 4745, as opposed to the average loss for 53 years, of Rs. 1805 crores.

There are mainly two types of measures for flood management one being structural and the other is non-structural measures. Structural measures includes concrete reservoirs and dams, natural drainage, through redistribution of a part of the river to a region where water withholding and water harvest is not a concern, the bank protection of the river to limit overflowing of rivers, the expansion of the channel to raise the area of stream to carry higher discharge, the catchment area to be handled by Afforestation, construction of sea walls and other such works while non-structural actions include flood-scale zoning , flood forecasting and warning systems, flood-protecting, etc (Tripathi, 2015).

Gujarat saw many caustic floods in past and majority of its areas are vulnerable to flooding irrespective of size of catchment. Nearly every major river in the state passes a wide stretch of very flat land before meeting the ocean and towns located on the rich alluvial plains of these major rivers and cities located on their banks are more exposed to inundation.

The Sabarmati River which runs through the semi-arid west of India is a seasonal river ruled by monsoon and stays parched post monsoon. It sometimes supplies to very severe flooding in Ahmedabad city which damages social and economical life of nearby low lying area. Historical reports show that the early floods were strong in 1683, 1714, 1739, 1755, 1868, 1927, 1941, 1950 and 1992, which specify that there was a succession of flood incidents in the Sabarmati river (Sridhar & Chamyal, 2014). In past century the flood event of year 1973 and 2006 are considered as the highest having peak discharge of 16000 cumec and 8800 cumec approximately according to state irrigation department. As per (GOG, n.d.), approximately 10365 cumec discharge was recorded at Chiloda Bridge, Gandhinagar station in July 1993 having maximum gauge of 8.25 m in Sabarmati river. After that, in year 2006, 2011, 2015, 2017 the Sabarmati River and nearby low lying area got inundated due to combine effect of torrential rain and discharge release from Dharoi dam (https://guj-nwrws.gujarat.gov.in)

2.2 Hydrologic Modeling

Hydrological simulation is old since it began discreetly from the 1850s. The development in modeling has occurred at an increasing rate, primarily due to easy access to almost enormous software power (Singh, 2018). Modeling of flood simplifies the actual event. For an instance, a flood model of one specific river basin simulates natural flood events using real input data, hydraulic properties and boundary conditions of specific area. A flood model of a particular river basin based on different boundary conditions or input information can be achieved which have variable impact on performance based on input data. The flood risk activity or the hydraulic properties can therefore be calculated and calculated at a certain period of time by simulation. (Hodges, 2009) defined hydrodynamic modeling as, "the art and science of applying conservation equations for momentum, continuity, and transport to represent evolving velocity, density, and scalar fields". The hydrodynamic modeling offers imminent into spatial and temporal changes in physical process that are observed, but less apparent in field information (Hodges, 2014).

Hydrodynamic models proves to be very useful for flood hazard mapping specifically by using numerical modeling, physical modeling or using historical data mapping with past extreme flood events. The numerical modeling is the prime choice for researchers and hydrologists in development of flood hazard maps because of its efficiency to simulate real world event even with shortage of data and lesser cost in comparison of physical model. While the physical modelling can be utilized as supplementary and supportive tool in flood hazard mapping for very significant region associated with high social and economical damage. The method to use historical data mapping proves to be very helpful for calibration of different parameters used by numerical methods (Bellos, 2012). Mathematical models, which try to reproduce fluid movement, are hydrodynamic models and they characterize the movement of water through the resolution of formulas generated by the physics rules. The simulations can be categorized as 1D, 2D and 3D versions based on their spatial representation of flood plain stream (Teng et al., 2017).

Hydrodynamic modeling is very useful techniques in recent time having important role in flood risk assessment due to its capability of simulating flood events of real time. The results generated through it can be effectively used for emergency flood management like, adaptive traffic control, evacuation, planning of infrastructures (Jaipurkar, 2014).

(Devi, Ganasri, & Dwarakish, 2015) have studied various hydrological models like TOPMODEL, MIKESHE, variable infiltration capacity model (VIC) and soil and water assessment tool (SWAT) model and classified models as lumped and distributed model based on function of space and time and deterministic and stochastic model based on output values produced for single set input, static and dynamic model based time factor and event based and continuous model based on the output it produces. They further discussed about empirical model, conceptual model and physically based models. Researchers like, (S. Kumar, Jaswal, Pandey, & Sharma, 2017) have discussed about application of various 1D and 2D hydrodynamic models like, HEC-RAS, HEC-HMS, LISSFLOOD-FP, MIKE-11, MIKE-21 and TELEMAC-2D, DAMBRK, FLDWAV, SMPDBK, FLO-2D and suggested HEC-RAS as more proficient tool to produce more reliable outcomes in dam break analysis.

2.3 Integration of GIS and HEC RAS

In recent decades, the use of Geographic Information System (GIS) along with the hydraulic model has been used effectively for flood management and forecasting. GIS being capable of representing topographic features and hydraulic model being efficient in simulating flow for various return periods, all together they both give very good results, which can be further used for preparing flood risk and hazards maps (Demir & Kisi, 2016). Nowadays researchers are using various hydraulic models in combination with GIS to simulate water flows. Many software packages like DWOPER, FLDWAV, MIKE-11, ISIS, SOBEK, CCHE2D, TUFLOW, Infoworks-2D, RiverFLO-2D etc. have been widely applied for simulating 1D and 2D flow in rivers (Taylor, 2012) (Bellos, 2012).

Geographical data systems attach land coverage information with topographical data and other knowledge regarding geographic location-related processes and resources. Non-topographic data may include a type of soil, use of property, land cover, characteristics of the groundwater, and human-crafted structures on or below the surfaces of the earth as applicable to hydrological systems. Topography analysis is referred to as terrain modeling and the propensity of surface water to move downwards implies that terrain mapping has a strong hydrological significance (DeVantier & Feldman, 1993).

In recent time, due to increased human interference together with excessive rainfall, the risk of flooding has been increased even in areas which were previously not prone to inundation. Due to this, flood management is highly required which first and foremost demands for preparing flood zone maps for floodplain which can be further utilized in calculating river bed, designing infrastructure development schedule, developing flood prediction and warning system, planning flood mitigation and rescue operations and many more. Various methods like, aerial photography, mathematical models, past observation analysis etc has been practiced since long but GIS integrated with hydraulic and hydrologic models is one of the newest techniques which save time and money (Khaleghi, Mahmoodi, & Karimzadeh, 2015). GIS being capable of representing topographic features and hydraulic model being efficient in simulating flow for various return periods, all together they both give very good results, which can be further used for preparing flood risk and hazards maps (Demir & Kisi, 2016). In several studies geographical information system (GIS) is used to identify areas of flood haz ards with the inclusion of weather, geomorphology, topography, soil and demographic data to reduce loss of life, damages caused by flooding, and damages (Bera, Pal, & Bandyopadhyay, 2012).

2.4 HEC RAS and HEC GeoRAS

The U.S. Army Corps of Engineer's River Analysis System (HEC-RAS) is an integrated system of software comprised of a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities. The software was developed at Hydraulic Engineering Center (HEC), which is a division of the Institute for Water Resources (IWR), U.S Army Corps of Engineer. The first version of HEC-RAS was released in July 1995 and since that time many changes and updates are included till latest version 5.0 in 2015 (Brunner, 2016).

HEC-RAS is open source tool designed to perform one-dimensional, two-dimensional and coupled 1D and 2D hydraulic simulations for a full network of channels. HEC-RAS latest version supports the constant and unchanging water flow profile estimation, the integrated hydrodynamics of 1D and 2D, the measurement of sediment transport / mobile beds, water temperature analysis, water quality analysis and spatial modeling of many measured parameters. The main benefits are the standard geometrical software representation and common geometrical and hydraulic computation routines in all parts.

The development of HEC-GeoRAS has linked ARC/INFO to the U.S. Army Corps of Engineers Hydrologic Engineering Center River Analysis System (HEC-RAS). HEC-GeoRAS provides a graphical user interface (GUI) that enables the hydraulic engineer to create a HEC-RAS import file containing geometric attribute data from an existing digital terrain model (DTM), process water surface profile data exported from HEC-RAS, and perform floodplain mapping. HEC-GeoRAS represents a significant advance in linking a GIS with hydraulic modeling. While the integration has been performed specifically for ARC/INFO and HEC-RAS, the GeoRAS software is not exclusive to assisting with hydraulic analysis in HEC-RAS. GeoRAS successful utilizes the GIS technology to develop and refine geometric data for use in river analysis. Results from hydraulic analysis may be displayed in the GIS to establish floodplain extent and evaluate flood depth.

Geometric data is prime requirement for development of any hydrodynamic model. HEC-RAS demands for precise channel geometry in terms of cross section data or digital elevation model to perform 1D and 2D simulation of flow. Nowadays researchers are using various hydraulic models in combination with GIS to simulate water flows. Many software packages like DWOPER, FLDWAV, MIKE-11, ISIS, SOBEK, CCHE2D, TUFLOW, Infoworks-2D, RiverFLO-2D etc. have been widely applied for simulating 1D and 2D flow in rivers (Bellos, 2012) (Taylor, 2012).

Many researchers have effectively developed 1 D HEC-RAS model for streams across globally. For example, AL-Kahla river (Awad, 2015b) and Hilla River (Luay Kadhim & Tawfeek, 2013) of Iraq, Kalu river, Shri lanka (Nandalal, 2009), Lighvan Chai River, Iran (Khaleghi et al., 2015), Mert River, Turkey (Demir & Kisi, 2016), Matial River, Northern Morocco (Azouagh et al., 2018) across the world and Indian rivers like, Mahanadi (Parhi, 2013), Godavari (Sunil Kute, Sayali Kakad, Vrusjali Bhoye, 2015), Tapi (Prafulkumar V.

Timbadiya, Patel, & Porey, 2011a)(P. V. Timbadiya, Patel, & Porey, 2014) (Mehta & Ramani Maulik Joshi, 2014), Jhelum (Ahmad, Alam, Bhat, & Ahmad, 2016),Waingangā (Ingale & Shetkar, 2017), Yamuna (N. Kumar, Lal, Sherring, & Issac, 2017), Purna river (Azazkhan, Vadher, & Agnihotri, 2017) are successfully simulated using 1 D HEC-RAS model in recent past. Also, development of 2D HEC-RAS has been practiced for river basins and urban areas worldwide such for Koshi river basin (Kafle & Shakya, 2018), Bolivian Amazonia (Quirogaa, Kurea, Udoa, & Manoa, 2016b), Cambodia (Thol et al., 2016), Jiaxing city area (Rincón, Khan, & Armenakis, 2018), Kirsehir city area (Yalcin, 2019), Greater Toronto area (Rincón et al., 2018) and Lower Indus River basin (Rind et al., 2018). Researchers like, (Nandurkar et al., 2017) and (Rangari, Sridhar, Umamahesh, & Patel, 2019) have successfully utilized HEC-RAS for development of 2D hydrodynamic model for Pune and Hyderabad cities respectively in India. The combined use of 1D and 2D hydrodynamic model has been effectively utilized for development of coupled 1D/2D model for Dharla River basin(Navera, 2018) and Jamuna River Basin (Ali, Anik, & Khan, 2016) in Bangladesh and Lower Tapi River and Surat city (Patel et al., 2017) to develop flood inundation maps for various hydraulic parameters as a part of flood management activity.

2.5 One-Dimensional hydrodynamic modeling case studies

(Nandalal, 2009) have developed HEC-RAS model for 79 km length of Kalu River from Ratanpura to Kalutara in Shri Lanka with the aim of determining water levels across the river. The required geometric data to develop a model has been produced by filed survey and the river has been represented in form of total 86 surveyed cross sections using 1:10,000 topo-sheets. Initially the model was calibrated by using five different steady flow profiles and model computed water levels at three gauging stations namely, Ratanpura, Ellagawaa and Ptupaula have been compared with observed values of water levels. The author then simulated HEC-RAS model under unsteady condition for observed flood event of 17th to 20th May 2003 by utilizing Manning’s n calibrated from steady flow simulation which shows good agreement of observed and simulated values of water levels. The unsteady model has been simulated for fifty different flood events to establish probability relation with upstream and downstream flood levels. The developed model also provides extent of inundation on both banks of river and three dimensional views of inundated areas along the river. The author concluded that developed model can obtain water levels on downstream on base of upstream water levels and these results can further used to inform people regarding flow of water in downstream region by experience of upstream communities.

(Azouagh et al., 2018) have developed floodplain maps for 30 km patch of torrential natured Martil River, Morocco using HEC RAS and HEC Geo RAS tools included into the Arc GIS software. As the work required more precision, and DEMs of 30 m, 12 m and 10 m resolution didn't satisfy the need of study, combine use of aerial photographs of 2m resolutions along with DEM has been used to create geometry data in form of 146 cross section using HEC Geo RAS tool. Discharge data of past record of three stations from year 1970 to year 2013 were analyzed and maximum discharge for each stations has been considered for model simulation. The 1D HEC RAS model has been simulated for return period of 10 year, 20 year and 50 year for three flood zones. The results generated by model enable to identify flood zones and other hydraulic parameters like water levels, velocity. The authors concluded that results are consistent with morphology of area and the HEC RAS software along with aerial photographs and rich information of morphology of river is effective in delineating risk areas for flood management and mapping.

(Masood & Takeuchi, 2012) have developed 1D hydrodynamic model for 37 sq.km area of mid-eastern Dhaka using SRTM DEM of 90 m grid and past flood records of 32 years from 1972 year to 2004 year. As per the author, DEM can only simulate channel having width greater than twice the resolution of itself. Therefore it was not feasible to solve Balu River in study area having average width of 100 m using collected 90 m resolution SRTM DEM. So, authors re-sampled 90 m resolution DEM into 30 m resolution using bilinear interpolation method. The actual DEM was modified according to actual river cross section and by raising elevation and merged with original DEM to finally get modified DEM of 30 m resolution. By using Gumbel distribution, authors calculated maximum water level for 100 year return period and assigned that as upstream and downstream boundary condition. The results of RAS simulation was used in preparing risk zone in calculated as a product of inundation depth and exposure of people and property to flood. This study shows effectively integrated application of HEC RAS and DEM generated data for preparing information about flood risk management.

(Yarrakula, Deb, & Samanta, 2010) have developed flood forecasting model of Subernarekha river for 154 km length for forecasting flood levels by integrating HEC-RAS and GIS for producing geometric data as well as mapping of inundation and damage areas. Authors produced 239 cross section of river using 10 m grid DEM generated from Cartoset 1 stereo data procured from National Remote Sensing Centre (NRSC). The elevation data of Cartoset 1 DEM of 10 m resolution was compared and verified with topo-sheets by taking 233 random spot elevation points and found to be matched sensibly fit with topo-sheet data. Stage hydrographs at Jamshedpur and near Bhosraghat have been considered as upstream and downstream boundary conditions respectively. The model was calibrated by using stage hydrograph of year 1985 and 1988 and most suitable value of manning's roughness coefficient calculated as 0.11 for banks and 0.047 for channel. The calibrated hydrodynamic model have been used to validate discharge and water level for duration of 16tu June to 22nd September 1997 using statistical performance indices like, Nash-Sutcliffe Coefficient, index of agreement and percentage of deviation in peak. Authors have concluded that hydrodynamic model developed using 10 m grid Cartoset 1 DEM for Subarnarekha river can efficiently forecast floods for different return periods.

(P. V. Timbadiya, Patel, & Porey, 2014) have developed 1 D unsteady HEC-RAS model using physically surveyed geometric data and flood data of years 1998, 2003 and 2006 for Lower Tapi River. River geometry has been represented in form of 135 cross sections for 103.5 km length of lower Tapi river and manning's roughness coefficient has been considered as single value of 0.035 for entire river reach. The authors have selected flow hydrograph at Ukai dam as upstream boundary and stage hydrograph at Hope Bridge, Surat city as downstream boundary. The model was simulated for extreme flood years of 1998, 2003 and 2006 for 10 minute of computation time step and stage hydrographs at Kakrapar weir and Ghala station were studied. The comparison of observed and simulated stage hydrographs at two gauging stations were calculated and scrutinized in terms of statistical parameters like, percentage error, root mean square error, mean absolute difference and mean difference. The author concluded that HEC-RAS simulated stage hydrographs were in close agreements with observed stage hydrographs.

2.6 Two-Dimensional hydrodynamic modeling case studies

(Ali et al., 2016) have developed combined 1D and 2D hydrodynamic model using HEC RAS and GIS for Jamuna River Basin for the objective of development flood inundation maps. Geometry was prepared in HEC GeoRAS using 10 m grid re-sampled DEM for development of 1D HEC-RAS model of River excluding nearby floodplain. After importing 1D geometry in HEC-RAS, the neighboring floodplain of left and right bank of river was converted into 2D mesh of 300 m x 300 m cell size. The model simulation gives the calibrated manning's value of 0.032 for main channel and 0.035 for floodplain has been fixed. The validated HEC-RAS model has been employed to produce different water surface profiles for various flow conditions for duration of 5 years from 2004 to 2008. With the help of GIS, the simulation results have been used to develop flow inundation maps. The inundation extent has shown close agreement with MODIS data map. The study presented a systematic approach for preparation of flood inundation maps using coupled 1D and 2D model applying integrated used of HEC-RAS and GIS. The author also, confirms compatibility of HEC-RAS, Arc GIS and HEC GeoRAS for development of coupled 1D/2D model.

(Quirogaa et al., 2016b) have developed 2D HEC-RAS model for Bolivian Amazonia using geometric data produced using 90 m grid SRTM DEM and flood data of February 02, 2014 to March 02, 2014. The flow hydrograph and normal depth has been considered as upstream and downstream boundary at upstream and downstream extreme of Mamore River respectively. The author ensures the model stability by considering 15 second time step calculated from the Corant-Friedriches-Lewy condition. The study proves that the model is effective in generating the flood level and also has good match with the data reported by satellite images. On the other hand, the simulated model also gives supplementary data such as depth of water, velocity, duration, inundation boundary and help in analyzing potential flood management policies.

(Rind et al., 2018) have developed 2 dimensional hydrodynamic model for lower Indus river from Kotri Barrage to Arabian Sea for flood year 2010 by using HEC-RAS 2D along with Arc GIS and HEC GeoRAS tools for data preparation. The prime objective of study was to delineate the flood inundation maps and identify flood vulnerable areas for future flooding events. The researchers used freely available ALOS World 3D digital surface model of 30 m resolution to generate detail bathymetry of river sections incorporating GIS and model simulation was performed in HEC-RAS. The study utilizes Land Use Land Cover, recognized by the Landsat 8 Archive and later validated the same with Atlas by FAO and SUPARCO. The main aim was to distinguish between different land uses characteristics of classes included in the study area and to allocate different manning's n values to each individual class on the basis of these categories. Roughness value of alluvial and non-alluvial waterways have been allocated to each group as suggested by United States Geological Survey (USGS). The calibrated model of year 2010 on the basis of manning's roughness was then validated for year 2015 flood event. The simulation results of flood inundation were compared with MODIS data in terms of spatial match, percent match and measure of fit, "F" method. In order to develop and execute an effective flood management plan, this analysis addresses the design of an adaptive flood system that can be used on both local and regional scales. The authors suggest that by using higher resolution topographical information, the proposed template can be further enhanced and findings may theoretically be used for hazard assessment, flood insurance, the future flood forecasting and land use planning.

(Liu, Merwade, & Jafarzadegan, 2019) have compared the performance of frequently used HEC-RAS 1D, HEC-RAS 2D, LISFLOOD-FP sub grid and LISFLOOD-FP diffusive, in terms of their model structure and relative sensitivity to surface roughness categorization for same input data and boundary condition to perform unsteady flow analysis for four past flood events in United States. The application of these models to four different areas with different river geometry and classification of roughness shows that for a given set of roughness the geometry of the 1D or 2D models does not affect the performance of the sinuosity, length, and wavelength of the river. The quality of a 1D model is approximately similar to that of the 2D models used in the analysis with slightly better outcomes for the 2D models. The study shows that the quality of 2D models is influenced by low channel roughness and increases with rising channel roughness, allowing additional water to enter the floodplain while the quality of 1D model is influenced positively with increased roughness of floodplain. The researchers also compared uniform and distributed roughness classification in the floodplain and suggested that uniform surface categorization provides better results than distributed roughness categorization.

(Patel et al., 2017) have developed coupled 1D /2D hydrodynamic model for Tapi river and Surat city integrating GIS with HEC-RAS software. Researchers performed 1D modeling of Tapi River by using 299 numbers of cross sections at an average interval of 150 m to 200 m and 2D modeling by using geometric data produced from SRTM DEM of 30 m and 90 m resolutions. The researchers have replicated the year 2006 flood scenario and generated the outcome in terms of flood depth, level of water surface, flow velocity, time of arrival, distance and flooding in seven places. The model is simulated for two cases one being without bank protection and second is with bank protection work in order to find the risk of flooding in the future. The simulated results are validated with the regional flood level map and spatially located observed flood depth. In this study, the new version 5.0.1 of HEC-RAS for flood analysis in a 1D/2D setting was analyzed and concluded it as an appropriate way for decision-makers to consider in advance, in a specific location in the flood plain, the likelihood of flood rate, size, arrival time, depression and length. Also it supports the decision-makers to take the suitable choice to on basis of simulated results to minimize loss of people and infrastructure.

(Nandurkar et al., 2017) have aimed to develop urban flash flood modelling integrating remote sensing data and HEC-RAS hydrodynamic model. All the required map layers are prepared using QGIS software. Rainfall data of Pune district since year 1998 have been collected and based on data of heavy rainfall and average daily rainfall has been calculated for past 20 years data. Cross section data of Mula and Mutha Rivers has been calculated by using 1 m interval contour map and some part of Mutha River which is passing through city area where contour map is not available, was calculated by assuming regular interval of 1 m in triangular shape. The geometric data of Pune city has been extracted using 30 m resolution SRTM DEM. The researchers used recent Landsat data to calculate impervious surface further the urban area is classified as various vegetation, impervious surface and soil classes for assigning suitable Manning's roughness value to each of them. By simulating HEC-RAS model the researchers aimed to calculate depths of flood flow at various spots in the city. After analyzing the results generated, authors concluded that HEC-RAS is very effective in predicting water depths due to heavy rains in urban area and its entire model simulation is very data intensive which required detailed terrain model to develop accurate geometry. The authors further suggest that this work can be extended by providing vulnerability assessment and mitigation plans by incorporating more information through physical survey or RS and GIS.

CHAPTER-3

STUDY AREA AND DATA COLLECTION

3.1 Study Area

In this study, an attempt has been made to develop hydrodynamic model which can determine probable inundated area along the banks of river using DEM generated data. Availability of sufficient and reliable geometric and hydrological data is still a major challenge for most of the part of India. The Sabarmati River being one of the major rivers in western India, and flowing through two most important cities, Ahmedabad and Gandhinagar of Gujarat State, the river reach from Chiloda bridge to Vasana barrage has been considered as study area. The first Indian world heritage city, Ahmedabad has been considered as study area to findl flood depth and inundation areas corresponding to discharge at Chiloda Bridge, Gandhinagar approximately 38 km upstream of the Ahmedabad city.

3.1.1 Sabarmati River

Sabarmati is one of the major rivers in the western region of India. It is a non perennial river originating from Dhebar Lake at an elevation of 762 m near Tepur village in Kotdi Tehsil of the Udaipur district of Rajasthan state. The Sabarmati basin lies between 70°58’ to 73°51’ East longitudes and 22°15’ to 24°47’ North latitudes. The total catchment area of Sabarmati is 21,674 km^2 out of which 18,550 km^2 lies in Gujarat state. The river after traversing a distance of approximately 48 km in Rajasthan enters the Gujarat state and meets the Gulf of Khambhat after flowing 323 km in Gujarat in south west course. Sei, Siri and Dhamni are right bank and Wakal, Harnav, Hathmati, Khari and Watrak are left bank tributaries of the Sabarmati River. Gandhinagar, the capital of Gujarat state and Ahmedabad, the major industrial hub of Gujarat are located on the banks of this river.

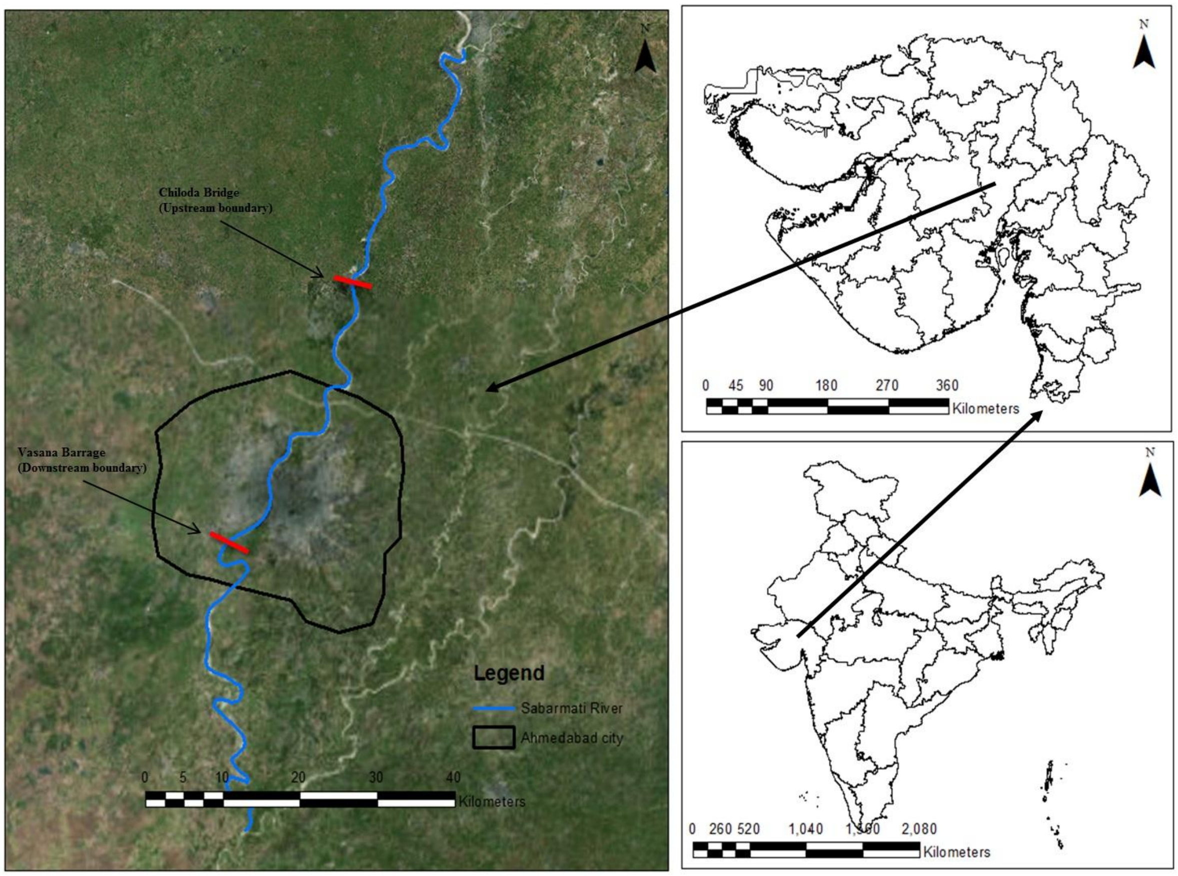


FIGURE 3.1 Study areas showing the Sabarmati River and the Ahmedabad city

The basin is located in semi-arid zone having average annual rainfall in range of 450 mm to 800 mm across various parts of basin. The Sabarmati basin extends over two states, Gujarat and Rajasthan having total area of 21,674 km^2. The basin is bounded by Aravalli hills on the north and north-east, by Rann of Kutch on the west and by Gulf of Khambhat on the south direction.

The wide river varies between 200 m and 1000 m in the study reach. The Sabarmati was susceptible to severe and frequent floods until the Dharoi Dam was built in 1976 (about 200 km upstream from Ahmedabad). The floods that Ahmedabad experienced in the latest past have been release from Dharoi Dam, which was announced twelve hours before the release of water from Ahmedabad. In 1976, a Vasna Dam was constructed downstream of Ahmedabad for water retention and irrigation on the Sabarmati River. The Narmada Canal, crossing the

River about 12 km upstream of Subhash Bridge is a part of major irrigation network. (http://sabarmatiriverfront.com/regional-history)

3.1.2 Ahmedabad city

Ahmedabad is Gujarat's biggest city and is India's seventh biggest. Sultan Ahmed Shah established the town in 1411 as the capital of the Gujarat Sultanate. In British India, the town became the home of the thriving textile industry known as' Manchester of the East.' The city is the administrative centre of Ahmedabad district, and was the capital of Gujarat after the bifurcation of the State of Bombay on 1st May 1960 to 1970; the capital was shifted to Gandhinagar after 1970. As per 2011,Census of India, Ahmedabad is the sixth-largest city and seventh-largest metropolitan area of India (Mundhe, Deshmukh, & Vyas, 2017). The City is Gujarat's center of education, IT, science and commercial sectors. Ahmedabad has been selected as one of the hundred Indian cities to be developed as a smart city and in July 2017, old city of Ahmedabad was declared as India's first world heritage city by UNESCO.

Ahmedabad city is located between $72^0$32'06" E to $72^0$35'14" E longitude and $22^0$59'01"N to $23^0$05'45"N in North-central Gujarat. The average elevation in Ahmedabad city is approximately 53 meters and most of the region is almost flat except for small hills of Thaltej area.

The city covers an area of 206 km^2 on the banks of the Sabarmati River. From mid-June to September, the Southwest Monsoon brings a damp climate. The average precipitation is 932 mm with low torrential rainfall. (Report of GSDMA 2006). The Sabarmati River divides the town into two physically separate areas of the East and the West. The 3rd segment of Ahmedabad has developing in eastern region during 19th century in the region of old villages on the outside edge of the old walled city. The old walled town extends along the tight curling roads, known as the "POLE" and consists of close cluster buildings in countless tightly packed houses. In this region there are numerous packed bazaars, the major train station and other administrative and commercial structures. The old town consists of a homogenous population with a distinct subculture of each. The eastern Ahmedabad other than old city carries the typical characteristics of trade area and cast based life style, industrial units with vast compound area and high walls, slums and chawls. The west section houses education organizations, contemporary buildings, multiplexes, broad highways, few residential colonies and slums.

The development of the Ahmedabad city is controlled by 2 state government bodies. The Ahmedabad Municipal Corporation (AMC) is responsible for the planning and policy implementation in core city, and the Ahmedabad Urban Development Authority (AUDA) plans and implements the policy decisions in newly developing urban areas.

The area of the city remains constant up to 1991. According to AMC about 20% growth is attributed to migration. The visible and the invisible population growth of the walled city and of the area just adjoining to the walled city have not been taken in to account while planning designing and implementing the sanitation work and rain drainage led to current flood vulnerability. Natural rainwater courses have been blocked by encroachments and illegitimate constructions which produces the floods combined with individual’s vulnerability. Thus vulnerability has been increased due to built environment and illegal construction. This increase in urbanization has led to elimination of retention capacity of the soil and increase the run off. The situation becomes extremely grim when the flooded river is not accepting this extra water and area along the both banks area inundated. The issue of storm water and solid waste is aggravating the flood problem.

3.1.3 Flood history

The average rainfall of Ahmedabad is about 932 mm (36.7 Inches). Infrequent heavy torrential rain causes flood to the River Sabarmati. As per AMC, the city has experienced floods at every two to three years interval. The Sabarmati River basin has a catchments area of about 21674 km^2. There are two major dams in upstream River Sabramati, one at Dharoi and second on a tributary Watrak River. These dams have been constructed with an objective of conservation of water for agriculture and drinking water purpose. The dams have a gated spillways and outflows aggravating the flood situation in downstream of dams. There is also a Vasna Barrage on river Sabarmati within Ahmedabad city. During flood situation the gates on Vasana barrage are opened to pass the excess water from the river.

Ahmedabad reach a record high flood level at 47.45 meters at Subhash Bridge on 19th & 20th August, 2006 surpassing the previous record level of 44.09 meters in 1993. Initial warning level is 44.09 meters and danger level is 45.35 meters. Discharge in the lower catchments was increased by releases from Dharoi and Watrak Dams. Inflow and outflow was almost same at the time of water released from both the Dams. This shows that both the dams have not been constructed with an objective of mitigating floods in the downstream area. The city remains flooded for two to three days during 19th August 2006 to 21st August 2006. The city is having mostly plane terrain apart from some portion of Thaltej Tekara area. The flood was concentrated in the areas of Bapunagar, Naroda,(Eastern Ahmedabad) Dani limada, part of Paldi, Ghatlodiya, Khanpur and Vadaj area which just outside the wall city. The densely populated slums are concentrated in these areas.

3.1.4 Major structures on Sabarmati River

There are many reservoirs on Sabarmati and its tributaries namely- Dharoi dam, Hathmati dam, Harnav dam, Guhai dam, Meshvo reservoir, Meshvo pick-up weir, Mazam dam and Watrak dam. For this study 39 km of Sabarmati River covering Ahmedabad and Gandhinagar cities has been considered. For development of hydrodynamic model, Chiloda Bridge, Gandhinagar and Vasana Barrage, Ahmedabad are fixed as upstream and downstream boundaries respectively. The Subhash Bridge gauging station which is located approximately 9km upstream of Vasana barrage has been considered for validation of results generated after simulation. The line diagram of the Sabarmati River from Dharoi dam to Vasna Barrage showing approximate distance is shown in Fig. 3.2 as follows;

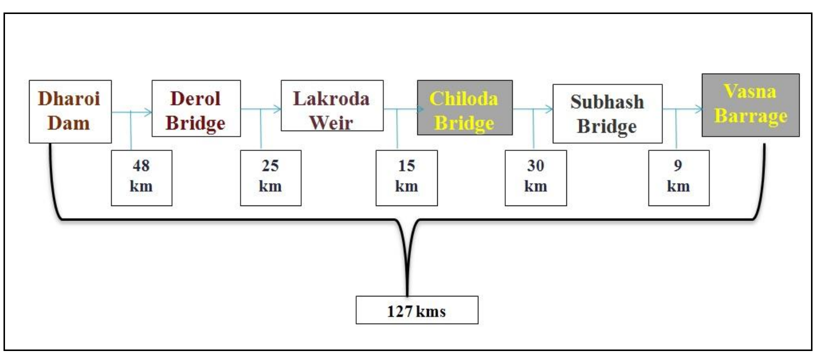


FIGURE 3.2 Line diagram of Sabarmati River showing study reach

3.1.4.1 Dharoi Dam

Dharoi dam constructed in 1978 is a major structure across the Sabarmati River which is located in Dharoi village of Mehsana district at approximately 165 km upstream of Ahmedabad city. Almost half of its catchment area, approximately 2640 km^2 from total of 5540 km^2 lies in Gujarat state. Dharoi is a composite dam constructed across Sabarmati River having 219 m long ogee type spillway. The top length of dam is 1207 m and maximum height above the lowest point of foundation is 46m. The catchment area of this project is nearly 5540 km^2. The live storage capacity of the reservoir is 776.5 MCM and gross storage capacity is 908.6 MCM.

FIGURE 3.3 Dharoi Dam on Sabarmati River

3.1.4.2 Chiloda Bridge

The Chiloda Bridge is high level bridge constructed on Sabarmati River near Borij village of Gandhinagar. The total length of the bridge is about 400 m. The bridge is having 11 spans of 36.58 m each. The design discharge and R.L. of design high flood level of Subhash Bridge are 11072 m^3/sec and 65.73 m respectively. The bridge is carrying Chiloda-Gandhinagar-Sarkhej road National Highway (NH) 8C.

FIGURE 3.4 Chiloda Bridge on Sabarmati River (Upstream boundary)

3.1.4.3 Subhash Bridge

The Subhash Bridge is high level bridge constructed on Sabarmati River near Sabarmati Ashram in year 1973. The total length of the bridge is 454.45 m. The bridge is having 5 spans of 76.25 m and 2 spans of 36.60 m each. The design discharge and high flood level of Subhash Bridge are 11350 cumecs and 32.025 m respectively. The bridge is carrying city road connecting Ranip and Shahibaug areas of Ahmedabad city.

FIGURE 3.5 Subhash Bridge on Sabarmati River (Gauging site)

3.1.4.4 Vasana Barrage

Vasana barrage was constructed in year 1976 in Ahmedabad for purpose of strengthening existing irrigation facility. It has total catchment area of 10619 km^2 covering 87 villages of 5 talukas of Ahmedabad district (source: https://guj-nwrws.gujarat.gov.in). Maximum observed flood at Vasana Barrage was in July 18, 1993 having discharge of 9783.37 cumecs. The barrages is having total 30 numbers of vertical gates among 26 numbers are on River and 4 numbers are on Fatewadi canal on Right bank of barrage. The length of Fatewadi canal is 6.9 km having capacity of 45.35 cumecs.

The Sabarmati River is having a meandering route of approximately 9 km just upstream of Vasana barrage with an average width range of 325m to 500 m, with two meandering loops at Gaikwad Haveli and Wadaj. The average reduced levels (RL) of the riverbed at Subhash Bridge and Vasna Barrage are 39.2 m and 37.4 m respectively. The height of the banks ranges from 4 to 9 m. A negative slope is observed from Sardar Bridge to Vasna Barrage. The edge is not clearly defined by embankments or retaining walls at most places. The river edge gently slopes down to the riverbed at several places, which have vegetation and have been encroached by slum settlements. The RL of the top gate of the Vasna Barrage is 41.756 m. Filling Vasna Barrage up to these level results in flooding of the nearby areas in monsoons.

FIGURE 3.6 Vasana Barrage on Sabarmati River (Downstream boundary)

FIGURE 3.7 Fatewadi Canal of Vasana Barrage

There are about 10 inline structures in the form of bridges connecting city roads, on Sabarmati River from Chiloda Bridge to Vasana Barrage. Details of some major bridge in route have been given in Table 3.1 as follows.

TABLE 3.1 Details of structures across the Sabarmati River in study reach

Sr. No	Name	Location	Year of Const.	Length (m)	Design Discharge (in m^3/sec)	Design H.F.L. (m)	Road with category
1	Indira Bridge	In 12 km near Hansol village	1983	400.04	14160	54.42	Road connecting Gandhinagar city to Airport at Ahmedabad
2	Subhash Bridge	Near Sabarmati	1973	454.45	11350	32.025	City road connecting Ranip and Shahibaug area
3	Gandhi Bridge	Near Income tax office	1940	457.5	11350	29.28	City road connecting Shahpur and Navarangpura areas
4	Nehru Bridge	Near Sadarbaug	1959	442.25	11350	30.5	City road connecting Khanpur Laldarwaja and Ellis Bridge areas
5	Ellis Bridge	Near Town hall	1892	433.41	14160	32.55	City road connecting west and east zone of Ahmedabad city
6	Sardar Bridge	Near Tagore hall	1939	510.57	11350	29.8	City road connecting Jamalpur and Paladi areas

(Source: Ahmedabad Municipal Corporation)

3.2 Data Collection

For development of hydrodynamic model in HEC-RAS, two prime requirements are hydrological information such as flow hydrograph, river gauge data, river stages and geometric information of river in form of elevation data along cross sections of river at desired interval, distance between consecutive cross sections for both left and right banks, Manning's roughness coefficient, normal depth at downstream etc. From these two main types of data, hydrological data are required for the past events to simulate and validate the model and corresponding channel geometry is required to provide terrain on which flow will be simulated. Literature review suggests that HEC-RAS model is highly dependent on channel geometry, and more realistic representation of river geometry facilitates better and sensible results. The data used for study along with its source and application in study is shown in Table 3.2.

TABLE 3.2 Details of data required and its application in study

Type of Data	Details	Source	Use in study
Hydrologic data	Hourly discharge and WSE	SWDC , CWC	Simulation of 1 D and 2D unsteady flow model in HEC-
Past records of discharge data for Sabarmati river	Yearly (1981-2015)	SWDC	Flood frequency analysis
River geometry data, cross sections, LOB, ROB etc	Every 200 m interval , Major bridge locations	CWC, SWDS, HecGeo RAS, Field Survey	Simulation of 1 D unsteady flow model
DEM of Ahmedabad city	ALOS (30 m grid) & Cartoset 1 (10 m grid)	ISRO, BISAG	2D unsteady flow modeling
Land use and land cover, ward map of Ahmedabad and Gandhinagar	Shape file, 10 m grid	BISAG, AMC	2D unsteady flow modeling

For development of 1D and 2D hydrodynamic model of Sabarmati River, Chiloda Bridge at Gandhinagar is considered as upstream boundary and Vasana Barrage at Ahmedabad is considered at downstream boundary. Subhash Bridge which is a gauging site of CWC, located approximately 9 km upstream of Vasana Barrage is considered for validation of simulated model parameter in term of water surface elevation.

Before development of HEC RAS model, the flood frequency analysis has been done at Chiloda Bridge which is considered as upstream boundary for study area. Flood frequency analysis using Gumbel method has been used and required annual peak discharge data from year 1981 to 2015, i.e. 35 years has been collected from SWDC, Gandhinagar. Other hydrologic data at upstream and downstream boundaries in terms of hourly river gauge and water surface elevation has been collected from SWDC and AMC. The water surface elevation data to be used for validation of simulated results have been collected from CWC, Gandhinagar.

For creating geometry of the river in HEC-RAS, cross section data of River, distance between consecutive cross sections at left bank and right bank between study reach is required. The only available cross section data at Chiloda Bridge and Subhash Bridge has been collected from SWDC and CWC respectively. The cross section data at some of the approachable bridge locations have been collected through field surveys. For development of precise HEC RAS model simulation, aimed in this study, elevation data at every cross section interval of 200 m interval is required. In primary stage of study, open source Google Earth software was used to produce required geometric data at every 500, interval in Sabarmati River. The validation of Google Earth generated geometric data with government data and filed data was not in acceptable range. Therefore, to overcome unavailability of geometric data at desired interval of 300 m, HEC GeoRAS software and 30 m grid interval ALOS and 10 m grid interval Cartoset 1 has been used to generate required elevation data along the study reach of Sabarmati River which is discussed in detail in Ch- 4; methodology, in section 4.2.

To develop geometric data of study area through HEC Geo RAS, grid interval of DEM plays very important role. For this study, ALOS and Cartoset 1 of 30 m grid and 10 m grid respectively have been used. The cross section spacing at which river geometry have been produces also affects the results of simulation in 1D hydrodynamic modeling. Therefore, results of both the DEMs for different cross spacing of 300 m and 200 m have been studied and evaluated.

The Advanced Land Observing Satellite (ALOS) released in January 2006 is one of the Japanese Earth observing satellite program which follows the Japanese Earth Resources Satellite-1 (JERS-1) and Advanced Earth Observing Satellite (ADEOS) and will utilize advanced land-observing technology. The ALOS global digital surface model (AW3D30) is a data set having horizontal resolution of around 1 arc second (30 meters) as shown in figure 3.8. This data set has been generated by the Japan Aerospace Exploration Agency's (JAXA) using Advanced Land Observing Satellite "DAICHI" (ALOS) based on stereo mapping from the Panchromatic Remote-sensing Instrument for Stereo Mapping (PRISM) and is available free of charge. It is considered as one of the most precise publically available digital surface model and widely utilized in areas like mapping, cartography, disaster monitoring and management, water resources research, regional observation, etc.

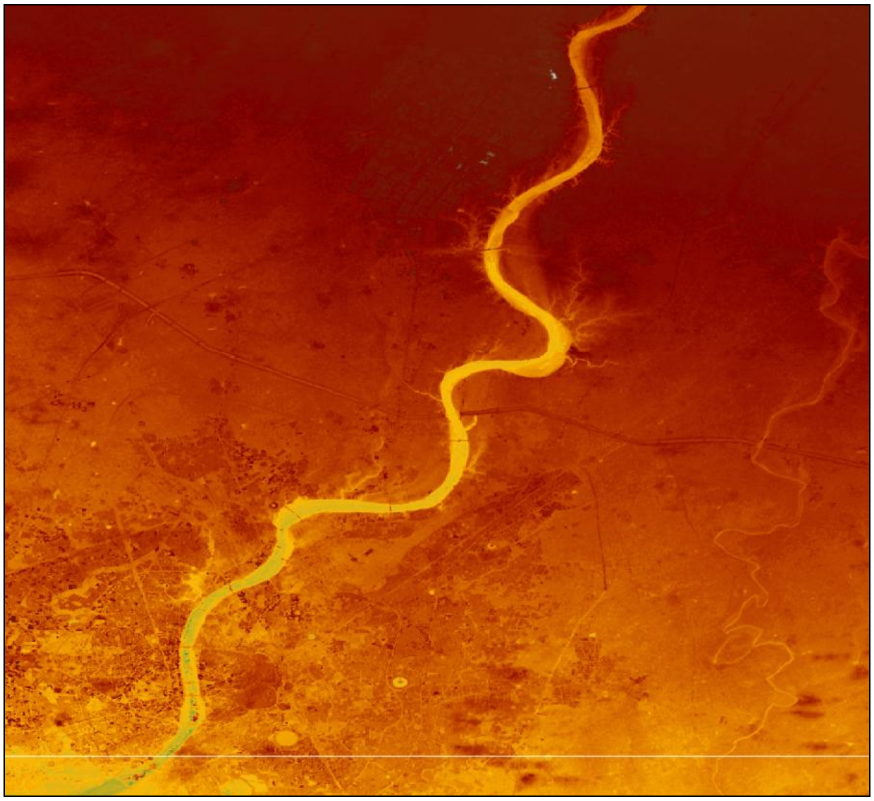


FIGURE 3.8 ALOS World 3D digital elevation model of 30 m grid

The Indian Space Research Organization (ISRO) has launched IRS-P5 (Cartosat-1) of 1560 kg on May 5, 2005 into 618 km Sun synchronous orbit. One of the most important objectives of this mission was to providing high-resolution satellite information. It consists of two panchromatic cameras with 2.5 m resolution which captures +26 degrees forward and -5 degrees rear images simultaneously. One of its key goals is to generate a Digital Elevation Model (DEM) and corresponding ortho-image for the entire country to facilitate large scale mapping in cartography and terrain modelling applications. The 2.5 resolutions of stereo images of study area have been purchased from ISRO. The DEM of 10 m resolution has been produced from stereo images under guidance of the Bhaskaracharya Institute for Space Applications and Geo-informatics (BISAG), Gandhinagar. In recent past the CartoDEM proves to be very functional in various subjects like run-off analysis, watershed planning, contour generation, drainage network analysis, river behavior and flood management, urban planning and management, design of hydraulic structures, design of pipeline or road network etc

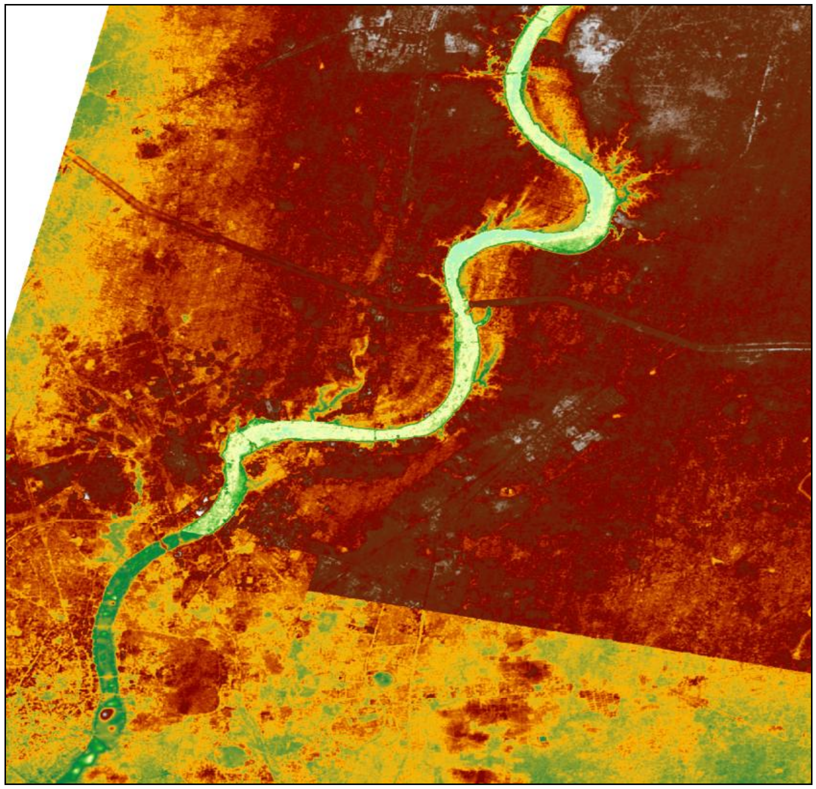


FIGURE 3.9 Cartoset 1 digital elevation model of 10 m grid

The manning's roughness coefficient for development of 1D model is selected by trial and error method and Chow (1959) in range of 0.020 to 0.040. The roughness coefficient for 2D model has been selected using shape file of land cover and land use map of 10 m grid in which roughness for various types of land surfaces has been selected as per field visits and the guideline of DRIP, GOI as shown in Table 3.3.

TABLE 3.3 Roughness considered for various land cover for development of 2D HEC RAS model

Land Use Land Cover (LULC) Code	Details	Manning’s n
1,2,5	Wasteland (land with scrubs)	0.1
14,16	Agriculture (crop land)	0.035
18	Plantation	0.160
30	River	0.025
31	Canal	0.04
100	Built-up area (rural)	0.1
101,800	Built-up (urban/cities) -medium	0.08
181	Others, Prosophis	0.160
300	Water bodies (pond/lake)	0.04
303	Water body (tank)	0.04
1000	Built-up (urban/cities)- high	0.15

The results in terms of discharge, WSE, velocity and inundation boundary of simulated 2D model has been mapped with ward map of Ahmedabad city collected from AMC to produce inundation probability map of the city.

CHAPTER 4
METHODOLOGY

4.1 Flood probability analysis – Gumbel's method

The aim of a flood-frequency analysis is to determine how often a particular area can expect a flood of a certain size. Estimation of recurrence intervals and probability of incidence of discharges help us to make flood predictions and can be further utilized for flood management planning. Recurrence interval is the length of time (years) between occurrences of a specific flood magnitude. The probability of happening of a particular discharge is the reciprocal of the recurrence interval. Flood frequency information can be determined from past information of peak discharge in any given year. For this study, probability of floods for related return period in range of 5 years to 100 years has been calculated by using peak discharge data of year 1981 to 2015, i.e. 35 years. Gumbel probability distributions has been used for simulating the future flood discharge situation using annual peak flow data (1981-2015) from Chiloda Bridge gauging station of the River Sabarmati. The predicted design floods of various return periods (T) i.e., 5, 10, 25, 50 and 100were obtained and compared.

Following steps have been performed for Gumbel's flood frequency analysis;

- First of all, all the n values of annual peak floods (x) have been arranged in descending order of its extent.
- The rank (m) of highest flood value has been assigned as 1 and so on for the rest of the values.
- The incidence of probability (P) and return period (T) has been calculated using equations m/ (n+1) and (n+1)/m. The values of P and T along with corresponding flood intensity gives plotting positions.
- Using mathematical steps statistical parameters like, mean x, squared mean x^2, mean of squares x^2 and standard deviation S have been calculated as shown in Table 4.1.

Table 4.1 Gumbel's flood frequency analysis using past data of yearly discharge

Year	Annual peak discharge x in 100 Cumec arranged in descending	Order No.	Return period T= (n+1) / m (years)	Probability P = m / (n+1) (%)	x^2
2012	0.00	35	1.03	0.97	0.00
2002	0.00	34	1.06	0.94	0.00
2001	0.00	33	1.09	0.92	0.00
2000	0.00	32	1.13	0.89	0.00
1999	0.28	31	1.16	0.86	0.08
1987	0.32	30	1.20	0.83	0.10
2004	0.33	29	1.24	0.81	0.11
2013	0.58	28	1.29	0.78	0.34
2003	0.61	27	1.33	0.75	0.37
1996	0.71	26	1.38	0.72	0.51
1986	0.77	25	1.44	0.69	0.59
2010	0.95	24	1.50	0.67	0.90
2009	1.06	23	1.57	0.64	1.11
1995	1.11	22	1.64	0.61	1.23
2014	1.15	21	1.71	0.58	1.32
1998	1.76	20	1.80	0.56	3.08
1989	3.10	19	1.89	0.53	9.60

2008	4.15	18	2.00	0.50	17.18
1982	4.72	17	2.12	0.47	22.23
1981	6.68	16	2.25	0.44	44.62
2005	8.28	15	2.40	0.42	68.53
1985	10.45	14	2.57	0.39	109.16
1984	10.58	13	2.77	0.36	111.85
1983	13.59	12	3.00	0.33	184.63
2011	14.81	11	3.27	0.31	219.47
1990	15.40	10	3.60	0.28	237.23
1997	18.95	9	4.00	0.25	359.00
1988	22.77	8	4.50	0.22	518.67
2007	24.42	7	5.14	0.19	596.38
1991	27.52	6	6.00	0.17	757.49
2015	29.48	5	7.20	0.14	868.86
1992	32.46	4	9.00	0.11	1053.53
1993	48.66	3	12.00	0.08	2367.36
1994	55.73	2	18.00	0.06	3105.45
2006	64.72	1	36.00	0.03	4188.83
[1]	[2]	[3]	[4]	[5]	[6]

- By using Gumbel's frequency factor table, frequency factor K of sample size n=35, have been selected for corresponding return period.
- By using relation x = x + KS, flood values for various return periods have been calculated which has been shown in Table 4.2 below.

Table 4.2 Gumbel's probability analysis for discharge and return periods

Return Period T	Mean x	S	K (From Gumbel's	KS	Flood Flow in (100 cumec)
[1]	[2]	[3]	[4]	[5]	[6]
5	12.17	16.86	0.851	14.35	26.52
10	12.17	16.86	1.516	25.56	37.73
15	12.17	16.86	1.891	31.88	44.05
20	12.17	16.86	2.152	36.28	48.45
25	12.17	16.86	2.354	39.69	51.86
30	12.17	16.86	2.52	42.49	54.66
50	12.17	16.86	2.979	50.23	62.40
60	12.17	16.86	3.142	52.97	65.14
75	12.17	16.86	3.341	56.33	68.50
100	12.17	16.86	3.698	62.35	74.52

- The discharge values of column [6] of Table 4.1 are plotted against return period of column [1] of the sameTable. Best fit function representing the nearest trend of probability of discharges versus respective return periods, has been found out using five most common mathematical functions and their respective R^2 values which has been further presented in chapter 5 ,section 5.1.

4.2 Geometry generation in HEC GeoRAS

For development of any hydrodynamic modeling either 1D or 2D, an accurate representation of ground surface elevations is a primary requirement. A high-quality terrain model precisely illustrate the elevations of the River and floodplain by including various physical features like, the channel banks and channel bed, and other high ground features such as roadways and levees by which the flow is directed. For this study, availability of geometric data at desired interval of 200m is a major challenge. The only data available with government agencies is at Chiloda Bridge and Subhash Bridge. Also, to produce elevation data at every 200 m interval for the entire patch of 39 km was time consuming and practically not possible. So, desired geometric data of study area has been generated using digital elevation model (DEM) namely, ALOS and Cartoset 1 of 30 m and 10 m resolutions respectively. Following are the stepwise procedure, adopted for generating geometric data in HEC GeoRAS tool.

- Add DEM in Arc Map using Add Data tool
- Select Terrain type and desired DEM in Layer setup for HEC-RAS Pre-Processing
- Create and digitize various RAS layers like Stream center line, Banks, flow paths and cross sections from RAS Geometry one by one as shown in Figure 4.1 below. The layers can be modified by using Editor Tool.
- Digitize stream center line in reference with DEM and available base map and assign suitable River code and reach code to stream.
- Assign Stream center line attributes like, topology, lengths, stations and elevations.
- Digitize bank lines in reference with selected DEM and base map
- Digitize flow path center lines in direction of flow in both sides along the river center line.

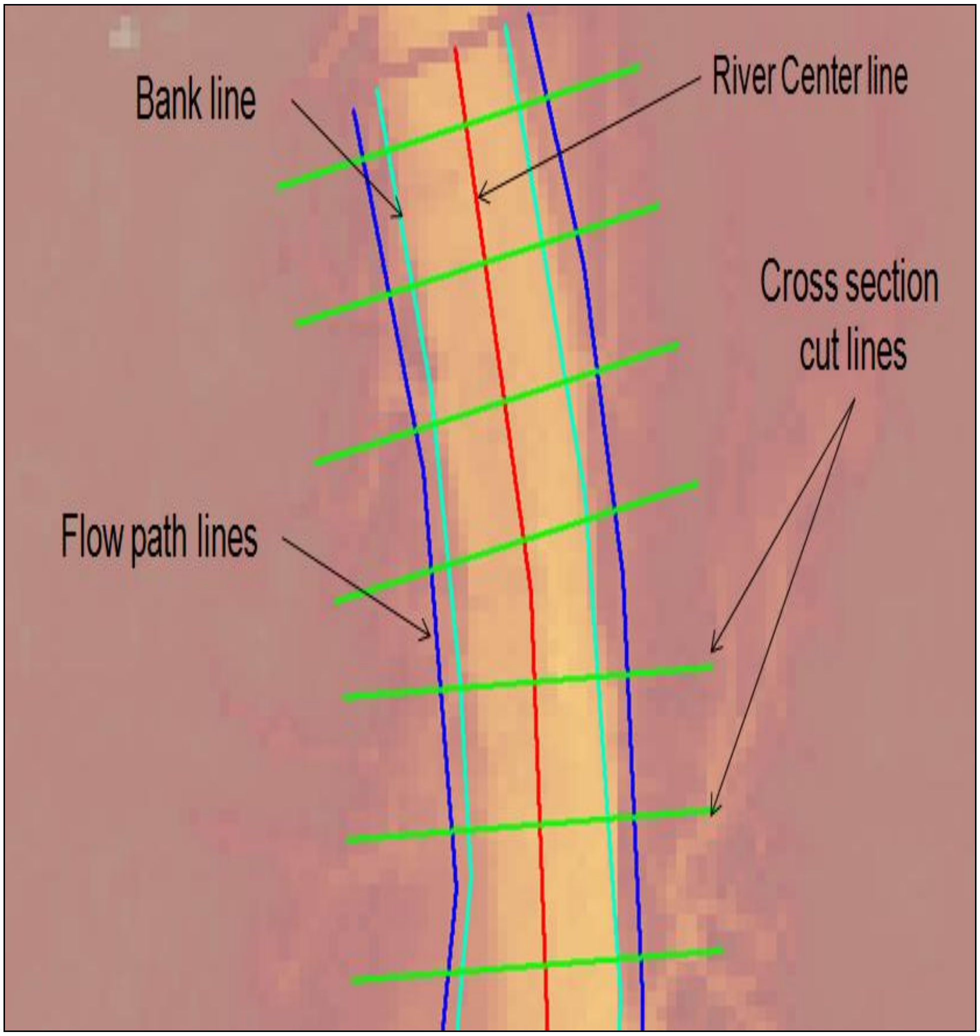


Figure 4.1 various layers representing River geometry created in HEC GeoRAS

- Assign line type attributes as right and left to relative flow path lines. For this, left and right banks are considered as per direction of flow.
- Generate Cross sections directly (of desire width and at desired interval) by using construct XS cut line tools or by manually digitizing. For generating cross sections directly, the maximum width of the channel has to be considered and accordingly single value of cross section width needs to be considered so that all the directly generated cross sections can either touch or cross the flow path lines on both the sides. For this study, cross sections are generated directly for spacing of 200 m and width of 1000 m as shown in Figure 4.2.

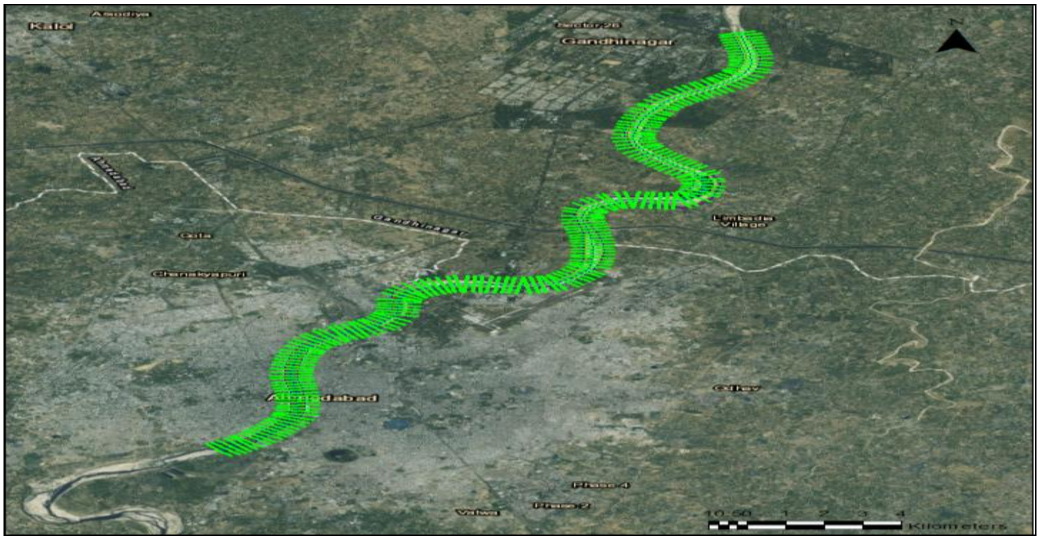


Figure 4.2 Cross sections on Sabarmati River generated in HEC GeoRAS

- Assign XS cut line attributes like River & Reach name, stationing, Bank stations, downstream reach lengths and Elevations from RAS Geometry tool.
- Export GIS to RAS using Export RAS data from RAS Geometry tool and save to your directory.
- Import Geometry data by selecting GIS format option in Geometric data window in HEC-RAS

4.3 One-dimensional (1D) hydrodynamic modeling

HEC RAS has ability to simulate one dimensional (1D) water surface profile under steady and unsteady condition for both natural and artificial channels. The water surface profiles are calculated between two consecutive cross sections of channel by solving energy equation using recurring procedure known as standard step method. (Chow, 1959) (HEC-RAS, 2016). The Physical laws which govern the flow of water in a stream are the principle of conservation of mass (Eq. 1), and the principle of conservation of momentum (Eq. 2). Both of these laws are expressed in the form of partial differential equations mathematically and usually referred to as continuity and momentum equations respectively and collectively referred as Saint-Venant equations. In the present study, one dimensional unsteady gradually varied flow simulation has been performed with the help of continuity and momentum equations.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial \frac{Q^2}{A}}{\partial x} + gA\frac{\partial H}{\partial x} + gA\,(S_o - S_f) = 0 \tag{2}$$

A = cross-sectional area normal to the flow; Q = discharge;

q= lateral inflow/outflow of the channel; g = acceleration of gravity;

H = water surface elevation, also called stage; S_o = bed slope;

S_f = friction slope; t = temporal coordinate,

x = longitudinal coordinate.

Saint-Venant equations, (1) & (2) are solved using the four-point implicit scheme, which is also known as the box scheme. This numerical scheme has been marginally stable when to run in a semi-implicit form, which corresponds to the weighting factor (θ) equal to 0.6 for unsteady simulation. In HEC-RAS, a default value of θ is 1, though; it permits to specify value ranging from 0.6 to 1. The box finite difference scheme is limited to its ability to handle transitions between subcritical and supercritical flow, because a diverse solution algorithm is requisite for diverse flow situation. This shortcoming can be purged in HEC-RAS by opting for a mixed-flow routine to patch solution in sub reaches.(Prafulkumar V. Timbadiya et al., 2011a)

To simulate 1 dimensional model in HEC-RAS, prime inputs are geometric data in terms of cross sections and bank levels, hydrologic data in terms of upstream and downstream conditions and manning's roughness coefficient. The required geometric data at every 200 m interval along the entire study reach has been generated using HEC GeoRAS tool in Arc GIS 10.0.1 software as describe in section 4.2. The hydrological data are required to assign upstream and downstream boundaries for study reach. The upstream boundary conditions can be given as either stage hydrograph or flow hydrograph of that location. The downstream boundary can be given as any of among stage hydrograph, flow hydrograph, rating curve and normal depth. For this study, flow hydrograph at Chiloda Bridge and normal depth at Vasana Barrages are considered as upstream and downstream boundary conditions respectively.

The flow chart of methodology adopted to simulate 1 D hydrodynamic model in HEC-RAS has been presented in Figure 4.3

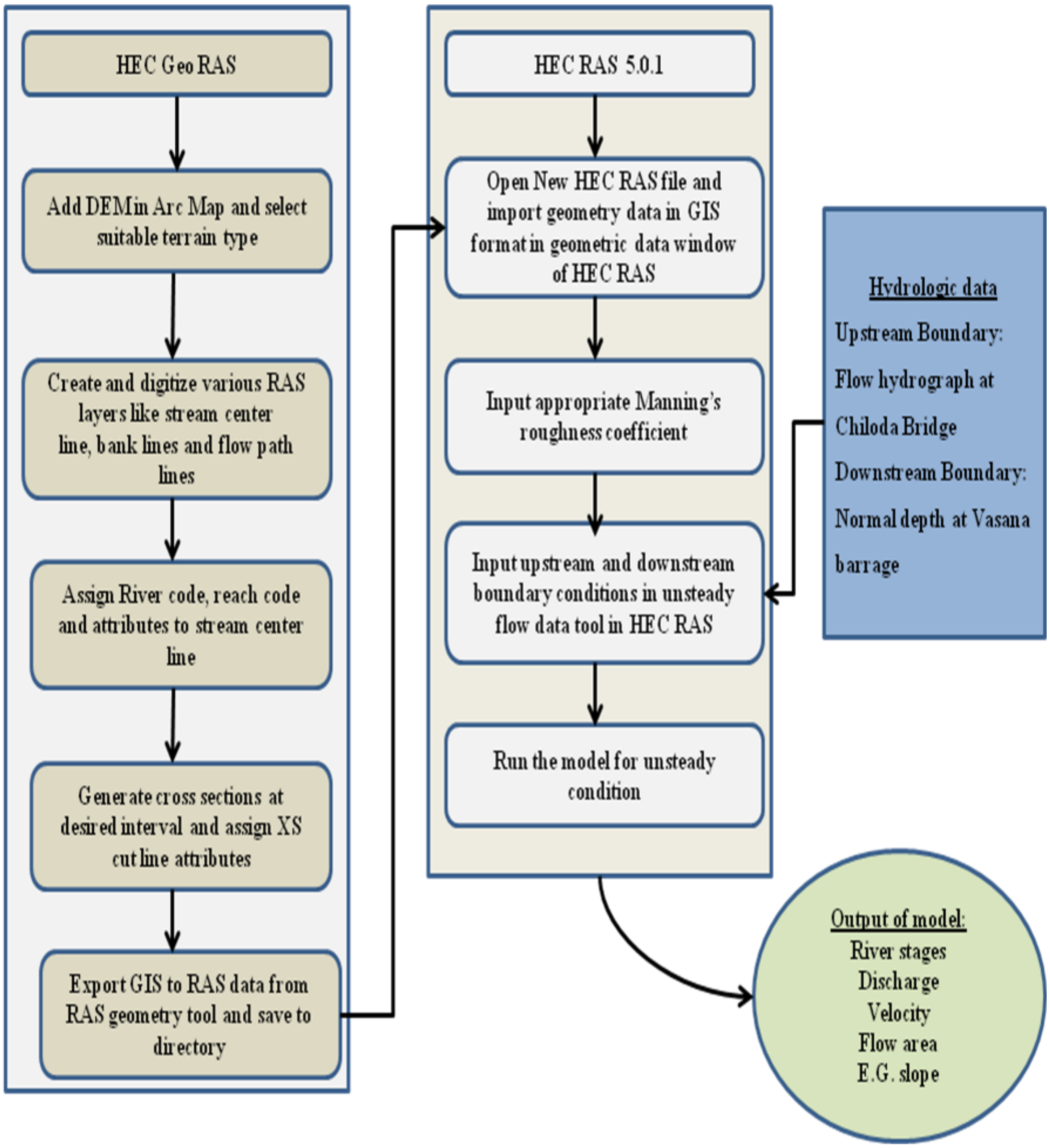


Figure 4.3 Methodology flow chart of 1D HEC RAS model

- Start a new project and save at desired location by giving appropriate name

- **Import Geometry data** from GeoRAS by selecting **GIS format** in Geometric data in HEC-RAS geometric data window as shown in Figure 4.4.

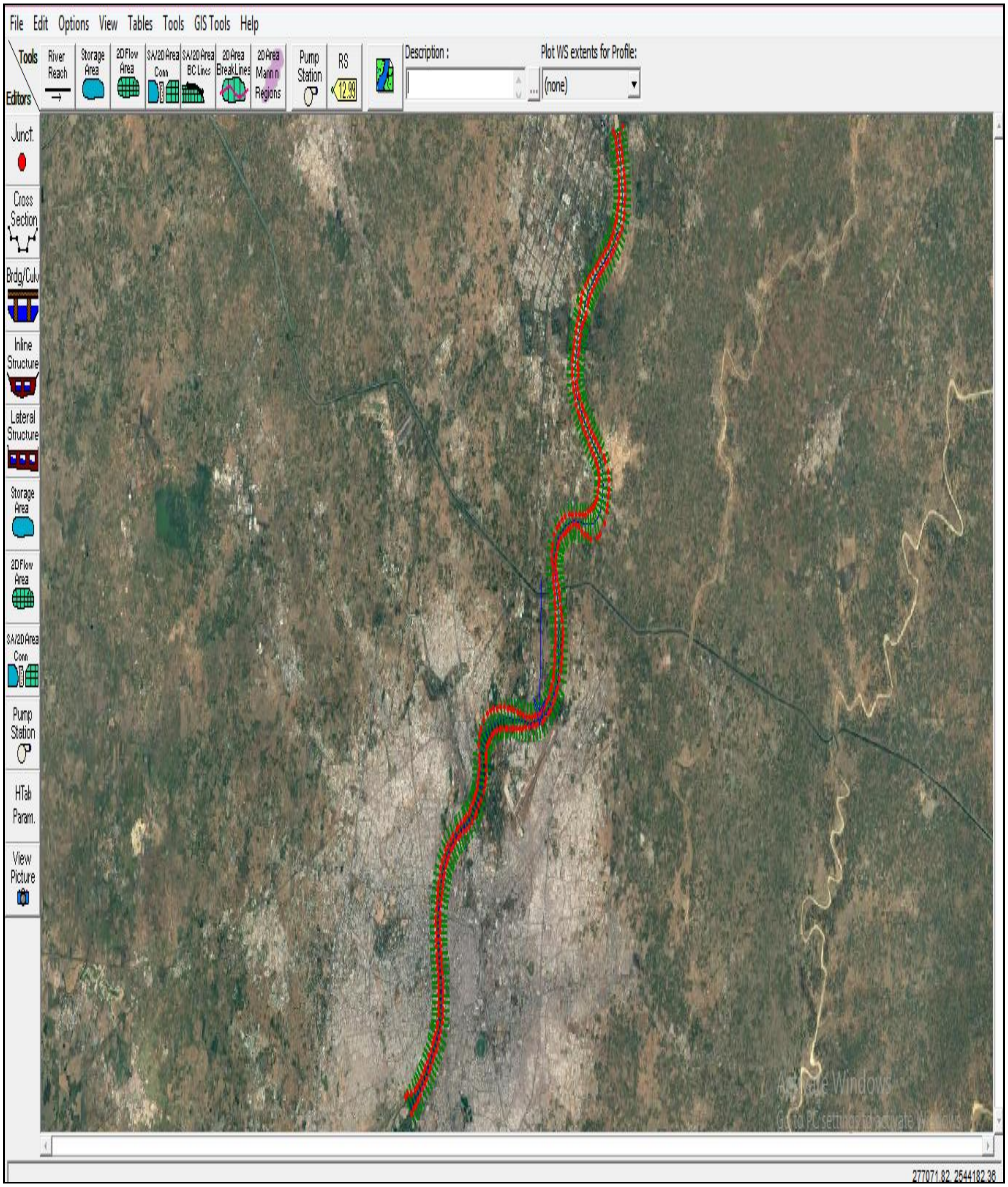


Figure 4.4 Geometric data imported from HEC GeoRAS in HEC RAS

- Input upstream and downstream boundary conditions in unsteady flow data tool in HEC-RAS. For 1D simulation upstream boundary condition is consider as flow hydrograph at Chiloda Bridge for the duration of 18/08/2006 (18:00 hours) to 23/08/2006 (23:00 hours) as shown in Figure 4.5. The downstream boundary condition is selected as normal depth of 0.00044, which was calculated by dividing difference of elevation between upstream and downstream boundary of river with total length. It can also be directly calculated in HEC-RAS.

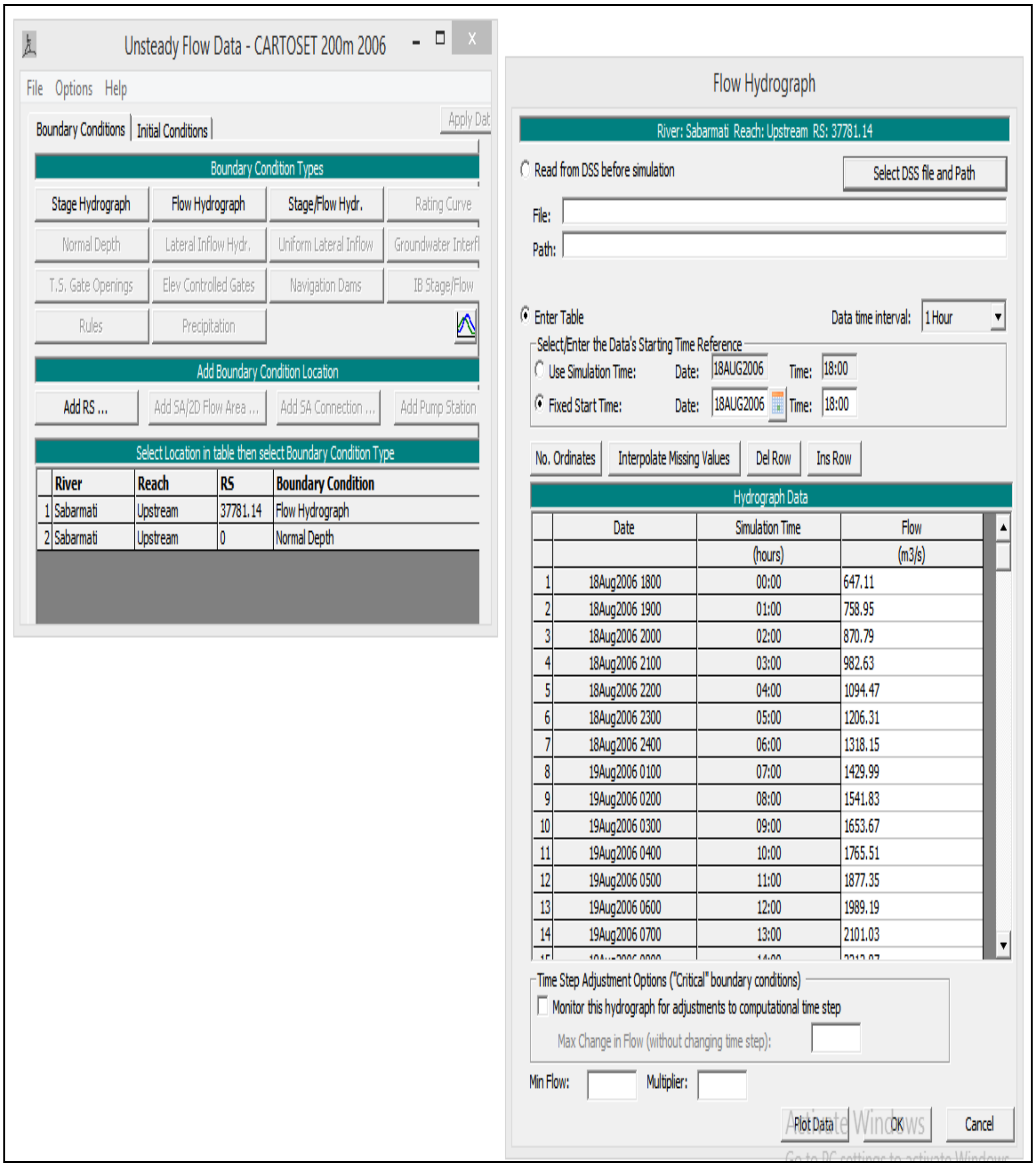


Figure 4.5 Boundary condition in unsteady flow window in HEC RAS

- Manning's value in range of **0.020 to 0.040** is used for trial and error to validate water surface elevation at Subhash Bridge for year 2006. Manning's value 0.025 gives the close match to observed value of stages at Subhash Bridge and considered for entire channel for simulation of 1D as well as 2D HEC RAS model.
- The model is simulated for 1 minute of computation time step and 1 hour of output storage interval for both years of 2006 and 2007 under unsteady condition as shown in Figure 4.6.

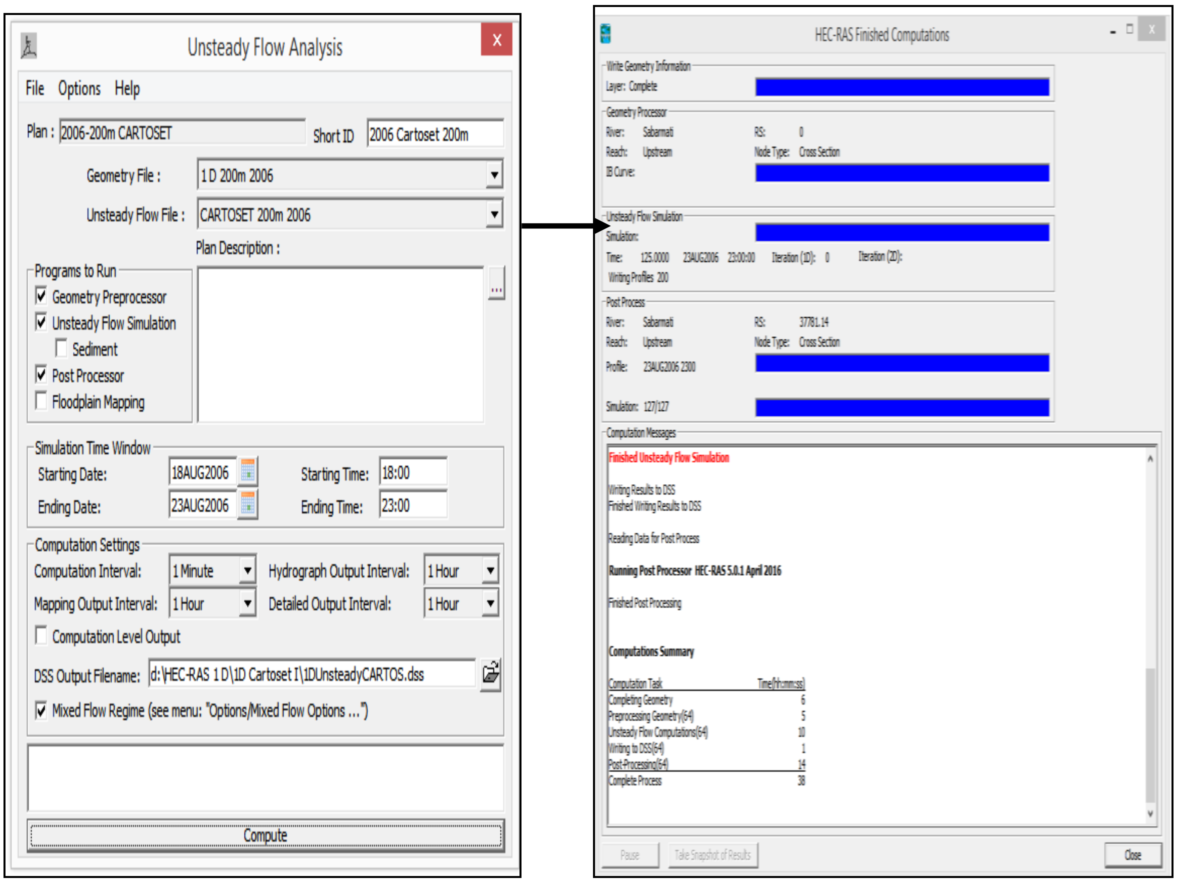


Figure 4.6 One dimensional HEC RAS model simulation in unsteady condition

After simulation of model, results in terms of various features like profile and cross section plots, rating curve, 3D multiple cross section plot along with animation which shows flow for each profile, flow and stage hydrographs, detailed output table at definite cross section locations, detailed output at intermediate structures like Bridges, Culverts, Weirs etc and review of errors, notes and warnings. Various hydraulic information like, discharge, water surface elevation, energy gradient slope, channel velocity, flow area, Froude number etc are available for each cross section and flow profile in tabular form after model simulation. Tabular output is available in two different layouts from which one shows results of hydraulic information at specific cross section point called as detailed output table and the other presents classified hydraulic information for several cross sections and several profiles called as profile summary table. Apart from these two, user defined output table containing desired table parameters can also be formed and utilized as one of the standard table for the particular plan.

4.4 Two-dimensional (2D) hydrodynamic modeling

The main channel and surrounding flood plain regions are a mixture of a river system. If a flood event goes above the depth of the main channel, the water surface elevations of the waterway expand into fluvial plains. For simulating elevated flows, 1D or 2D models with constant and unstable state assumptions. Two-dimensional models are using the ground as a constant surface, while 1D model only look at the river and inland geometries at distinct places along the channel length. The use of a constant terrain allows two-dimensional models to describe more precisely the lateral flow interactions between the main and the floodplains and their flood-planar storage impacts.

In 1D hydraulic modelling it is assumed that the flow only moves downstream into the longitudinal direction. The terrain in a 1D model is represented as a transversal system and the results are estimated at each cross-section of the average velocity and water depth as discussed in section 4.3. In a 1D model there are physical limitations that can be overcome both longitudinally and sideways by a 2D model, namely concurrent flow. For development of 2D hydrodynamic model in HEC-RAS, the entire study area needs to be delineated by a continuous polygon covering River, banks and floodplain and referred as “2D flow area” which corresponds to the extent of entirety of the simulated model. The 2D flow area is divided into a grid where each compartment is a control volume in which inflow and outflow is calculated for each side based on respective velocities.

The concept of 2D modeling is to discriminate the river and adjoining floodplain area into a group of different cells which are called as the grid cells. Each grid cell consists of levels and roughness details on behalf of the levels and friction of the terrain. HEC-RAS implements a sub-grid strategy to bathymetry. Each grid cell consists of various GIS cells, as shown in Figure 4.7, with the sub-grid bathymetry approach. There is a single elevation for every GIS cell. It is effectively a set of grid cells that make up the model of the land. It's the surface model which represents the ground.

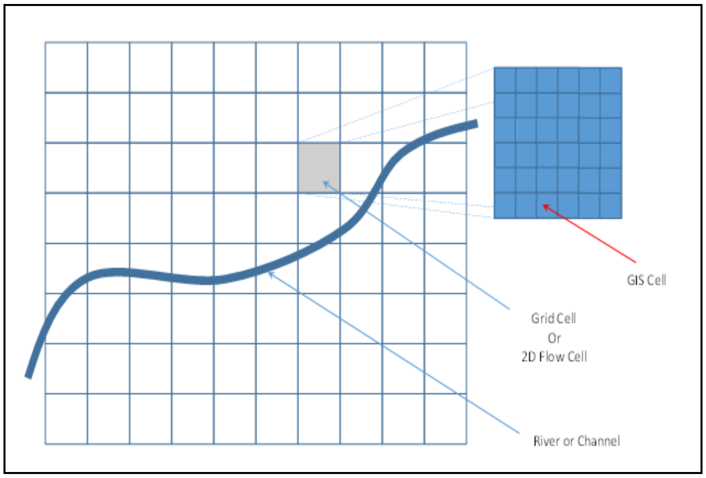


Figure 4.7 Channel and flood plain in form of Grid cell and GIS cell

The interface between two grid cells is called a cell face. The ground geometry at a face is composed of the ground elevations found from the GIS cells. Hydraulically, a cell face is the same as across-section and since ground geometry information is provided by GIS cell, various properties such as hydraulic radius, cross section area, wetted perimeter, conveyance etc can be calculated for desired water surface elevation. The 2D flow area editor allows the user to select a nominal grid size for the initial generation of the 2D flow area computation mesh. User have to specify the spacing between the computational grid cell centers in terms of DX and DY which indicates the generation of computational mesh that has grids that are X & Y everywhere except around break lines and the outer boundary. The computational mesh will control the movement of water through the 2D flow area.

For development of 2D model, a computational mesh is generated for Sabarmati river reach from Chiloda Bridge to Vasana barrage covering Gandhinagar and Ahmedabad cities. Cartoset 1 terrain of 10 m grid was used as a base for computational mesh for 2D flow area. Initially, 10 m X 10 m spacing of computation point was selected which generated the total 26, 65,342 grid cells. This grid was selected to be near to the original DEM (Cartoset 1 10m X

10m). But during the process it was found that the machine used for simulation having 64 bit operating system, Intel core i3 processor and 4 GB RAM was not able to calculate hydraulic property tables in RAS Mapper and giving "system out of memory" type of error. To move ahead with the simulation, the computational mesh was generated using 20 m spacing of computation points which generated the total of 6, 65,887 grid cells and was able to calculate hydraulic property table for each grid cell.

The new version of HEC RAS 5.0.1 have the option to develop a model either by using full momentum equation, commonly called as 2D Saint Venant equation or the 2D diffusive wave equation as shown below as equations (3), (4) and (5). (Gary W. Brunner, 2016) (Quirogaa et al., 2016a) (Patel et al., 2017)

$$\frac{\partial \zeta}{\partial p} + \frac{\partial p}{\partial x} + \frac{\partial q}{\partial x} = 0 \tag{3}$$

$$\frac{\partial p}{\partial t} + \frac{\partial}{\partial x}\left(\frac{p^2}{h}\right) + \frac{\partial}{\partial y}\left(\frac{pq}{h}\right) = -\frac{n^2 pg\sqrt{p^2+q^2}}{h^2} - gh\frac{\partial \zeta}{\partial x} + pf + \frac{\partial}{p\partial x}(ht_{xx}) + \frac{\partial}{p\partial y}(ht_{xy}) \tag{4}$$

$$\frac{\partial q}{\partial t} + \frac{\partial}{\partial y}\left(\frac{q^2}{h}\right) + \frac{\partial}{\partial y}\left(\frac{pq}{h}\right) = -\frac{n^2 qg\sqrt{p^2+q^2}}{h^2} - gh\frac{\partial \zeta}{\partial y} + qf + \frac{\partial}{p\partial y}(ht_{yy}) + \frac{\partial}{p\partial y}(ht_{xy}) \tag{5}$$

Where,

h is the depth of water in meter,

p and q are the explicit flow in the x and y directions ($m^2 s^{-1}$),

ζ is the surface elevation (m),

g is the acceleration due gravity ($m\ s^{-2}$),

n is the Manning roughness coefficient,

p is the water density ($kg\ m^{-3}$),

xx, yy and xy are the components of the effective shear stress and

f is the Coriolis (s^{-1}).

When the diffusive wave equation is selected, the primary terms of the momentum equations (2) and (3) are neglected. In HEC RAS software, diffusion wave equation is default but for this simulation the Saint Venant equation set called as full momentum equation has been selected. The Saint Venant equations are solved using a four point implicit finite volume algorithm commonly called as box scheme. The finite volume solution approximates the average integral on a reference volume and allows the more broad approach to indefinite meshes. The main reasons for selecting full momentum equation over diffusion wave equation for specific study area is

- The diffusion wave equation do not consider the change in velocity over the time and distance which are key concern during developing hydrodynamic model to simulate rapidly rising flood waves.

- The reach of Sabarmati River selected as study area is having spacious and well planned Gandhinagar on upstream and randomly developed slums area and illegal encroachment specifically on bank of river for approximately 15 km length of downstream resulting into abrupt change in water way of river. For this particular condition, the force related to sudden contraction and expansion can be more precisely calculated by full momentum equation.

- Also, the full momentum equation proves to be better choice for calculating water surface and velocity through bridges and around abutments and piers. As having 6 major bridges across the study reach of river between Chiloda Bridge to Vasana barrage the full momentum equation proves to be wise choice for simulation.

Apart from above study area specific reasons, the full momentum equations set can efficiently simulate mixed flow condition, rapid transmission of wave around and through structures, having tight bend in channel. Also, this equation set demands for very minor grid cells and little computational time steps (Gary W. Brunner, 2016)

There are mainly five types of external boundary conditions such as Flow hydrograph, Stage hydrograph, normal depth, rating curve and precipitation which can be used either as upstream or downstream boundary. Among these, the flow hydrograph and stage hydrograph can be applied as positive flow values entering 2D flow area and negative flow values flowing out of a 2D area respectively. The precipitation boundary condition can be applied directly to any 2D flow area as a time series of surplus rainfall. The normal depth and rating curve can only be used at downstream side of channel where flow will leave a 2D flow area. Flow hydrograph at Chiloda Bridge and normal depth at Vasana Barrage has been considered as upstream and downstream boundary conditions respectively. The manning's roughness factor for channel was selected as per the calibration results derived from 1 D simulation and for neighboring flood plain was selected as per the Dam Rehabilitation and Improvement Project (DRIP), govt. of India guidelines.

The selection of appropriated cell size and corresponding computational time steps which goes well with selected 2D area mesh for simulation event. The computational time step is function of cell size in which entire flood plain is alienated and the flow velocity moving through cells. HEC RAS provides guideline for selecting suitable time step interval for both diffusion wave equation and full momentum equation. As the full momentum equation has been selected for simulation of model for this study, the following equation (6) and (7) has been considered for calculating time step interval to ensure the stability of the model (Gary W. Brunner, 2016) (Patel et al., 2017).

$$C = \frac{V\Delta T}{\Delta X} \leq 1.0 \ \ (with\ maximum\ C = 3.0) \tag{6}$$

$$\Delta T = \frac{\Delta X}{V} \ \ (with\ C = 1.0) \tag{7}$$

Where,

C is the Courant Number,

V is the flood flow velocity (m s-1),

ΔT computational time step(s) and

Δx is the average cell size (m)

The model is simulated for year 2006 by considering computational interval of 10 second for Cartoset 1 of 10 m grid under unsteady condition and the results in form of map for various hydraulic properties like discharge, velocity, water surface elevation, inundation boundary and arrival time has been produced in RAS Mapper and studied. The stepwise methodology adopted for simulation has been illustrated in following flowchart in Figure 4.8.

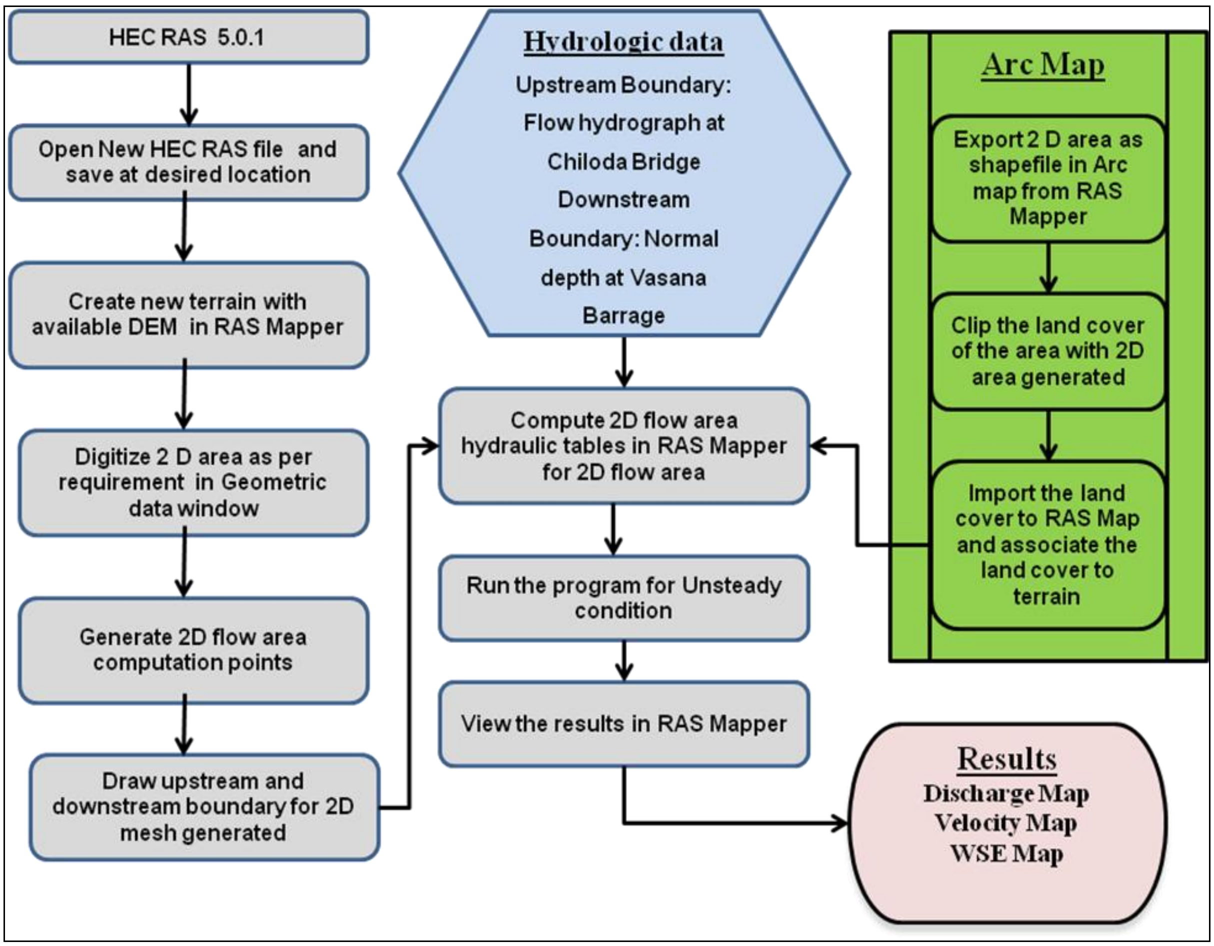


Figure 4.8 Methodology flow chart of 2D HEC RAS model

Stepwise procedure adopted to simulate 2D model in HEC-RAS software has been described as follows;

- Start a new project and save at desired location by giving appropriate name
- By using tool RAS Mapper, Create new terrain by right clicks on terrain layer and select already saved DEM by browsing. The RAS Mapper can import floating-point grid format

(*.fit), GeoTIFF (*tif) and other format. The terrain layers used to construct the terrain model for this study consisted of Cartoset 1 data set of 10 m grid and 2D flow area generated is divided into grid cell of size 20 m x 20 m. Thus, entire study area is characterized as total 665887 grid cell having average area of cell is 400.54 m^2.

- Open **Geometric data** from HEC- RAS and zoom in as per requirement
- Add **new 2D Flow area** and draw boundary as per requirement
- **Generate 2D flow area computation points** by giving desired values **through** 2D flow areas toolbar. Selection of desired **spacing DX and spacing DY** in computation point spacing can be entered. For this simulation both DX and DY are given as 20 (Figure 4.9).

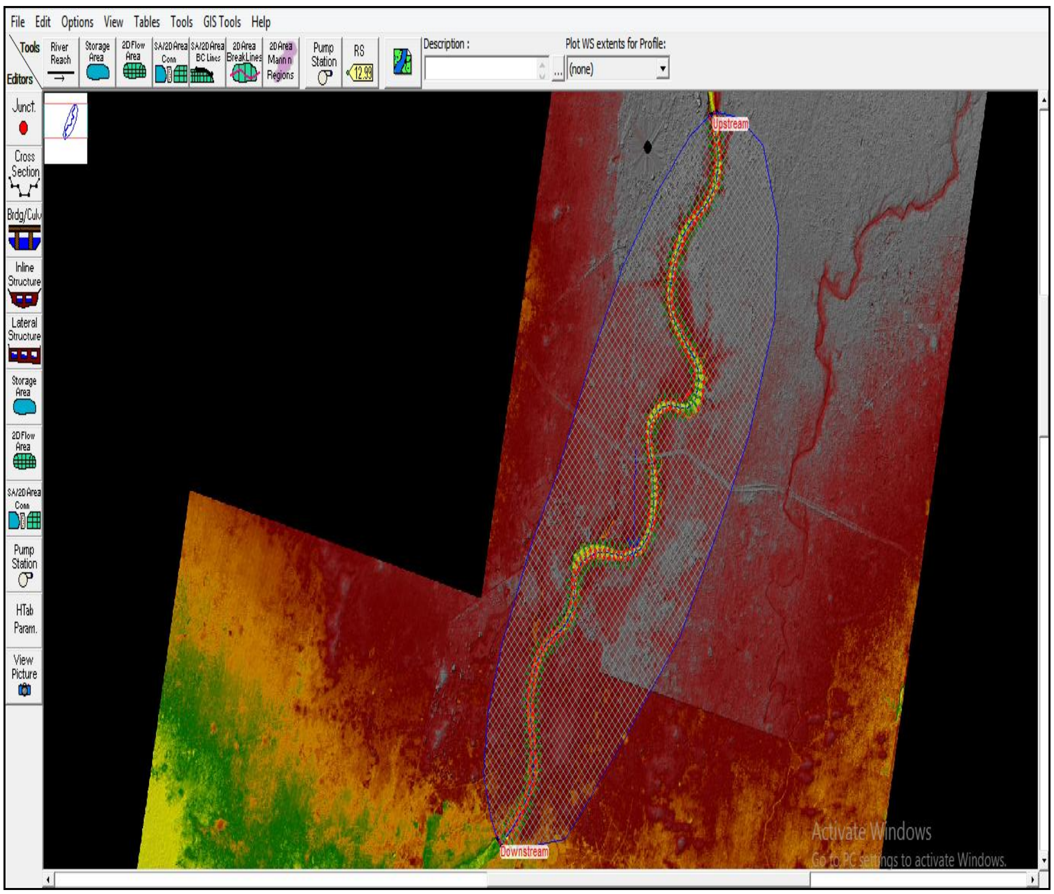


Figure 4.9 Generation of 2D computation mesh for study area

- In RAS Mapper, select **Export layer** by right clicking Terrain layer and select **current view as image**
- **Export terrain** in image format at desired location and save it by giving desired name.
- **Add map data layer** from specified location in img. format

- In geometric data window, **select layers to view in background** and click added map layer. The DEM in background of 2D mesh area is replaced with map layer of img. Format of the DEM.
- Draw upstream and downstream boundary lines by using tool **Add arc just outside 2D area for new boundary condition and** name it.
- Input **upstream and downstream boundary conditions** in unsteady flow data tool in HEC-RAS. For the simulation of 2D model in present research, following upstream and downstream conditions has been applied.

Upstream Boundary Condition:

Flow Hydrograph	Location – Chiloda Bridge
	Duration - 18/08/2006 (18:00 hours) to 23/08/2006 (23:00 hours)

Downstream Boundary Condition:

Normal depth	Location – Vasana barrage
	Value – 0.00044 (directly from 1D HEC-RAS model)

- The various land cover information can be generated in RAS Mapper and subsequently incorporated with selected geometric data of flood plain. Also, user can select suitable manning's n value and also can customize specific regions in which roughness value can be replaced with existing n value in land cover data set. The 2D flow area roughness value of n can be entirely define manning's n or can be utilized to regulate several manning's n value for flood plain for calibration of model. The HEC RAS can allow fetching land use information in shape file and gridded formats. The shape file of specific area can be generated in Arc GIS while gridded land cover information can be gained from other related sources (Gary W. Brunner, 2016). For this simulation, the shape file of 2D area generated in HEC RAS has been exported in Arc Map and has been clipped with land cover information collected from BISAG, Gandhinagar of same region as shown in Figure 4.10.

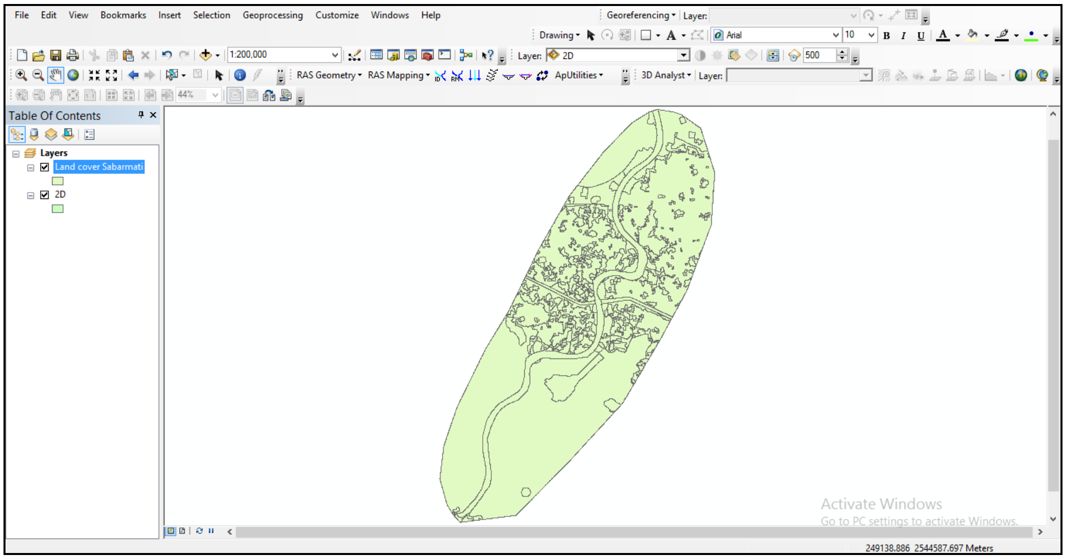


Figure 4.10 Land cover associated with 2D flow area in Arc Map

- The value of manning's roughness coefficient has been selected for different surfaces as suggested by guidelines for mapping Flood Risks Associated with Dam, DRIP, Government of India as specified in Table 4.3.

Table 4.3 Manning's roughness coefficient used to develop 2D HEC RAS model for different land covers

Land Use Land Cover (LULC) Code	Details	Manning's n
1,2,5	Wasteland (land with scrubs)	0.1
14,16	Agriculture (crop land)	0.035
18	Plantation	0.160
30	River	0.025

Land Use Land Cover (LULC) Code	Details	Manning’s n
31	Canal	0.04
100	Built-up area (rural)	0.1
101,800	Built-up (urban/cities) -medium	0.08
181	Others, Prosophis	0.160
300	Water bodies (pond/lake)	0.04
303	Water body (tank)	0.04
1000	Built-up (urban/cities)- high	0.15

- The land cover with appropriate roughness coefficient has been imported in RAS map and has been associated with terrain used for simulation and calculation of **2D flow area hydraulic tables** has been performed from 2D flow area layer in RAS Mapper as shown in Figure 4.11 below;

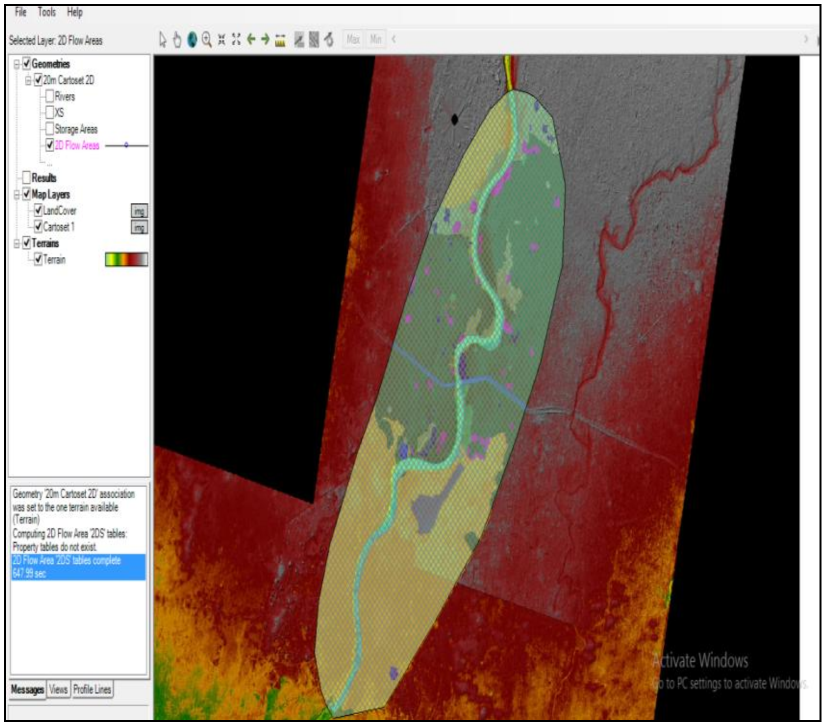


Figure 4.11 Study area associated with corresponding roughness coefficient in RAS Mapper

- Calculate **options and tolerances** from options and select desired equation set for simulation. For this simulation full momentum equation with theta value of 1 and water surface tolerance and volume tolerance equal to 0.003 has been considered as shown in Figure 4.12 below.

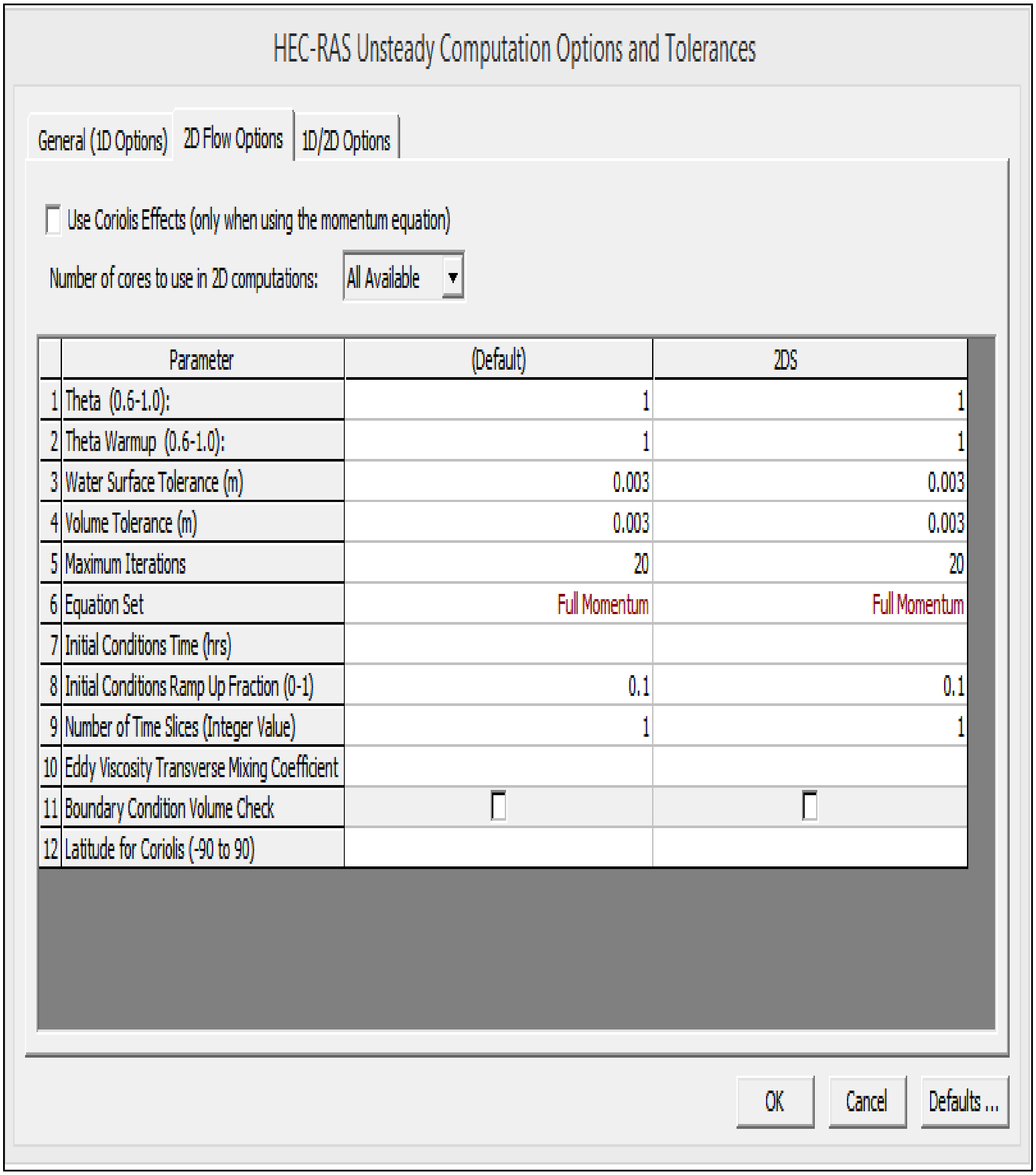


Figure 4.12 Selection of options and tolerance for unsteady simulation

- **Run the program** for 10 sec computational interval under unsteady condition from 18th August 2006, 18:00 hours to 23rd August 2006, 23:00 hours as shown in Figure 4.13 (a) and 4.13 (b) below;

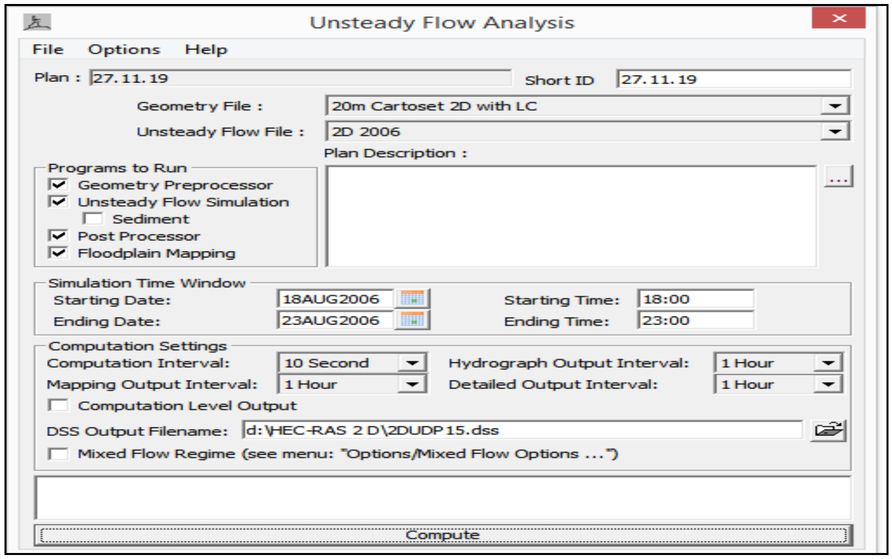


Figure 4.13 (a) Unsteady flow analysis window for 2D HEC RAS model

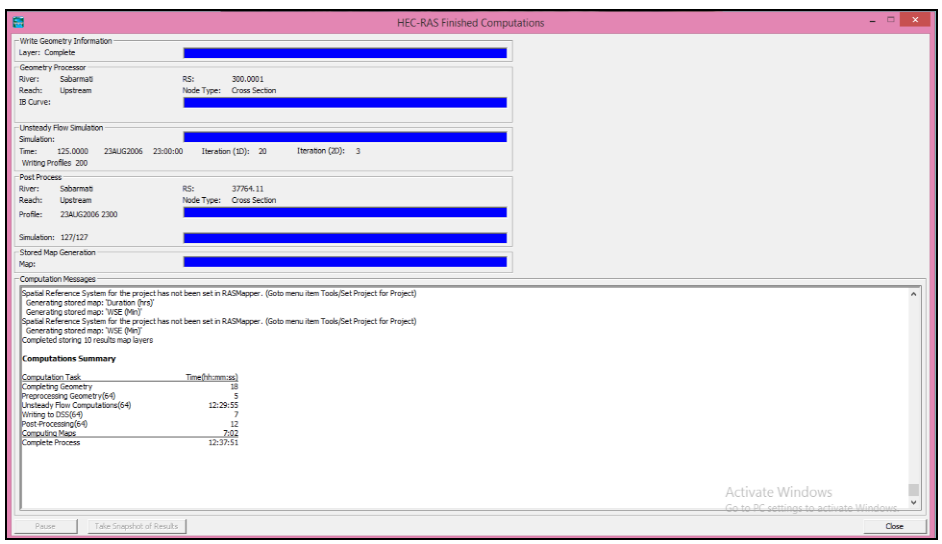


Figure 4.13 (b) Unsteady flow simulation for 2D HEC RAS model

After simulation of flow, output in terms of map of various hydraulic parameters like discharge, velocity, inundation boundary, water surface elevation and arrival time has been produced for every one hour interval for the duration of 18th August 2006, 18:00 to 23rd August 2006, 23:00 in RAS Mapper. These maps have been saved as raster file and imported in Arc GIS. The raster files of discharge, velocity, inundation boundary, water surface elevation and arrival time with corresponding area ward map has been mapped and analyzed, which will gives results of specific hydraulic parameter for particular region in study area.

CHAPTER 5

CALIBRATION AND VALIDATION

5.1 Calibration of Manning’s roughness

When any hydrodynamic model is created, the key objective is to bring it closer to its actual condition through the specific choice of all variables, according to the actual condition. There should be a tolerable spectrum of variation between the real and simulated phenomenon that can be achieved through model tuning. Depending on the model's significance, a range of criteria must be optimized to obtain best-compatible outcomes for trials and errors. Hydrodynamic simulations are applied to estimate the frequency and variability of flooding at different scales. Hydrodynamic models are calibrated, so that the best possible descriptive parameters for the resistance to natural flow are found. The testing of hydrodynamic systems has been more practical and quicker in recent years, as earth observation services and computer-based optimization techniques have improved.

The output of hydrodynamic model depends on many variables like, mesh size for 2D model and cross section spacing for 1D model, time step interval considered for simulation, roughness of surface etc. Among all, the manning's coefficient is typically used as a calibration parameter in hydrologic modeling as it describes the resistance to flow of water on surface. Since it cannot be measured, it is determined empirically or implicitly by laboratory and field techniques, which necessarily involve a large amount of trial and error (Oubennaceur, Chokmani, Nastev, Tanguy, & Raymond, 2018). As suggested by (Prafulkumar V. Timbadiya, Patel, & Porey, 2011b), The flow of water, the water levels and other hydraulic properties are interconnected and highly depends on the roughness of the channel . The calculation of the channel roughness parameter is of key importance in the open-channel flow analysis, especially in hydraulic modeling. Channel roughness is a highly variable parameter that depends on many variables including surface roughness, vegetation cover, channel course, channel irregularities etc.(Parhi, 2012) (Parhi, 2013).

For this study, the flood data of year 2006 has been considered for calibration of manning's roughness, "n". In this research, attempt has been made to calibrate roughness coefficient of Sabarmati River from Chiloda Bridge, Gandhinagar to Vasana Barrage, Ahmedabad considering manning's n in range of 0.02 to 0.04 as per (Chow, 1959) for flood banks with heavy stand of timbers with flood reaching below branches which is dominating feature in study area considered (NIH, 2007). By using aforementioned data and subsequently, different values of roughness coefficient have been used to justify their adequacy for simulation of flood event of year 2006 in study reach. As recommended by (NIH, 2007), flow contribution from the cross sectional area next to the banks is relatively much lesser than channel segment and change in depths of flow are largely unaffected by varying roughness of banks of river. Considering the fact of that the roughness of banks have less significance on depth of flow, the single value of manning's n have been considered for entire study reach of Sabarmati River for calibration purpose. The various single values of manning's n used for whole study reach for year 2006 flood and ALOS DEM of 30 m resolution along with flow duration and gauging site for calibration and validation are shown in Table 5.1.

TABLE 5.1 Simulation duration, manning's "n" and gauging site considered for calibration of model

Flow year	Simulation duration	Manning's n	Gauging site
2006	18th August 2006, 18:00 hours to 23rd August 2006, 23:00 hours	0.020, 0.025, 0.030, 0.035, 0.040	Subhash Bridge

In study reach of Chiloda Bridge to Vasana Barrage, there is only one gauging site located at Subhash Bridge which is approximately 9 km upstream of Vasana Barrage in Ahmedabad city and managed by Central Water Commission (CWC), government of India. Due to unavailability of sufficient number of gauging site in study reach, the calibration and validation of model has been performed at Subhash Bridge site only. The 1 D HEC-RAS model of Sabarmati River has been simulated under unsteady condition for calibration of manning's roughness "n" for year 2006 flood event and stages of river at gauging site simulated for various manning's n has been compared with the actual values of stage hydrograph of the same site (Figure 5.1).

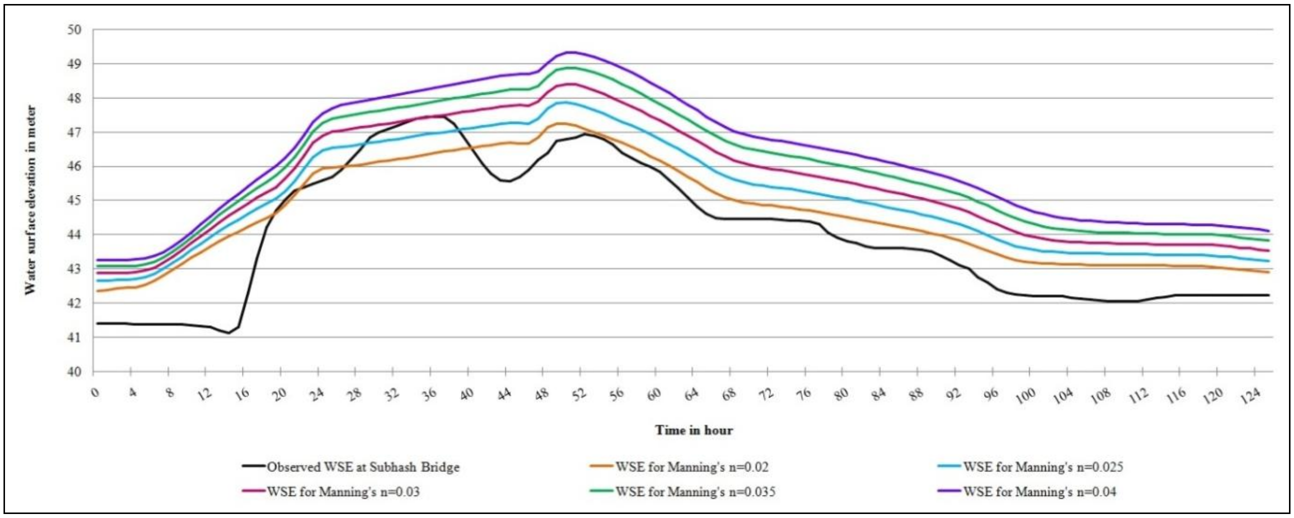


Figure 5.1 Comparison of observed and simulated stage hydrographs for various manning's roughness value for year 2006 at Subhash Bridge gauging site

The observed and simulated values of stage hydrograph of Sabarmati River at Subhash Bridge gauging site has been compared through statistical parameters like root mean square error (RMSE) (Prafulkumar V. Timbadiya et al., 2011a) (P. V. Timbadiya, Patel, & P.D., 2014) (Luay Kadhim & Tawfeek, 2013) mean absolute difference, mean difference and Nash-Sutcliff efficiency (NSE) (Parhi, 2012) as shown in Table 5.2 below.

TABLE 5.2 Comparison of observed and simulated stage hydrographs at Subhash Bridge for year 2006 flood event for calibration of manning's n

Simulation duration	Manning's n	RMSE	Mean absolute difference	Mean difference	NSE
18th August 2006, 18:00 hours to 23rd August 2006, 23:00 hours	0.020	0.83	0.62	-0.03	0.82
	0.025	0.95	0.8	-0.62	0.64
	0.030	1.26	1.6	-1.09	0.57
	0.035	1.62	2.63	-1.51	0.48
	0.040	1.97	1.89	-1.89	0.41

After studying above Figure 5.1 presenting graphical comparison of observed and simulated water surface elevation for Manning's roughness ranging from 0.2 to 0.4 and analyzing these parameters in various statistical parameters, it has been decided that, though Manning's roughness of 0.020 gives good agreement in terms of RMSE and mean absolute difference, it has lower value of peak stage than the observed one. From any flood model it has been expected that it should give output nearer to the actual one and strictly not less than the actual values to avoid under estimation of future flow. Considering this, fact the Manning's n value of 0.025 has been considered for simulation as though giving moderate satisfaction in terms of statistical output but better in accordance with peak value of actual stage in River.

5.2 Validation of hydrodynamic model

To evaluate the performance of calibrated model for manning's roughness equal to 0.025, flood data of year 2007 has been used for validation purpose. The observed stage hydrograph at Subhash Bridge gauging site for flood year 2007 has been compared with simulated stage hydrograph considering calibrated manning's roughness value of 0.025 (Figure 5.2).

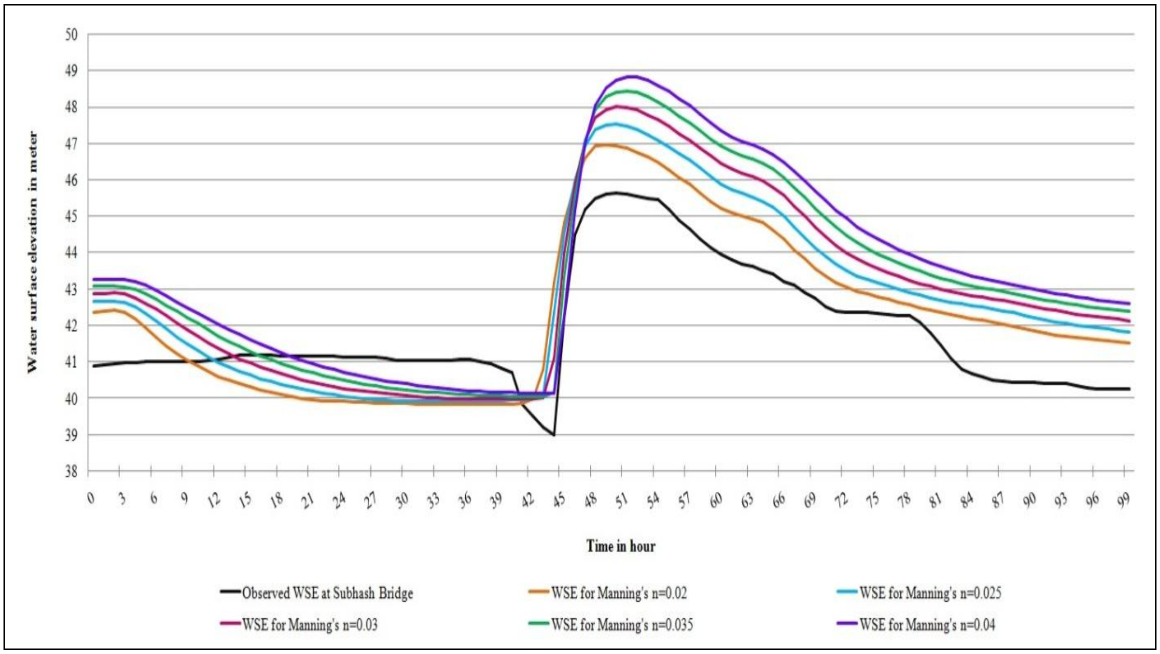


Figure 5.2 Comparison of observed and simulated stage hydrographs for various manning's roughness value for year 2006 at Subhash Bridge gauging site

The calibrated manning's roughness of 0.025 for Sabarmati River in study reach, has been validated for flow data of year 2007 by comparing observed and simulated stage hydrographs at Subhash Bridge gauging site. The simulated stage hydrograph using calibrated manning's n value of 0.025, has been also evaluated using various statistical parameters like RMSE, mean absolute difference, mean difference and NSE as shown in Table 5.3 below;

TABLE 5.3 Validation of observed and simulated stage hydrographs at Subhash Bridge for year 2007 flood event for calibrated manning's n

Simulation duration	Manning's n	RMSE	Mean absolute difference	Mean difference	NSE
7th July 2007, 21:00 hours to 11th July 2007, 24:00 hours	0.025	1.22	1.11	-0.47	0.82

From figure 5.2 and statistical parameters calculated and shown in table 6.3 shows that by taking into account manning's n value of 0.020 the developed model can predict river stages through being on little higher values. Though results given by the statistical parameters does not comes out as desired but considering the unavailability of sufficient hydrologic data and geometric data for validation in study reach, the developed model can at least give idea about trend and pattern of river water level in study area. Also, being on little higher side it eliminates probability of underestimating the flow and makes it safer considering illegal encroachments on river banks resulting in narrowing the water way in channel.

Thus, considering both the statistical results as well as results of site inspection along with limitation of only one gauging site for validation, it has been decided to consider the manning's roughness of 0.025 for Sabarmati River for development of 1D as well as 2D HEC-RAS simulation.

CHAPTER 6

RESULT AND ANALYSIS

6.1 Flood probability analysis

As discussed in chapter 4, section 4.1, the flood frequency analysis has been carried out for Sabarmati River by considering past 35 year data from year 1981 to year 2015 using Gumbel's flood frequency distribution. The various mathematical functions with corresponding coefficient of correlation value R^2 has been studied and compared with trend of graph plotted using Gumbel's distribution as shown in Table 6.1 below.

Table 6.1 Functions and respective R^2 for Gumbel's probability graph

Function	Equation	R^2
Exponential	$y = 36.64e^{0.008x}$	0.742
Linear	$y = 0.440x + 36.19$	0.863
Logarithmic	$y = 15.66\ln(x) + 1.445$	0.999
Polynomial (2nd order)	$y = -0.005x^2 + 0.987x + 27.95$	0.956
Power	$y = 17.38x^{0.326}$	0.974

As a result of trial and error for finding best fit trend of discharge over different return periods, logarithmic function shows the most accurate value of R^2 as 0.999 corresponding to discharge calculated from Gumbel's probability plot as shown in Figure 6.1.

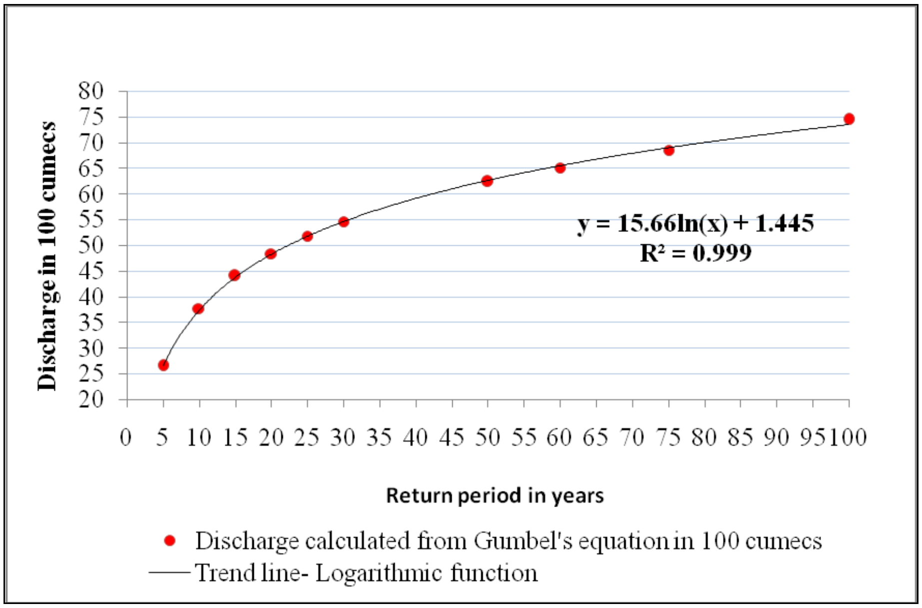


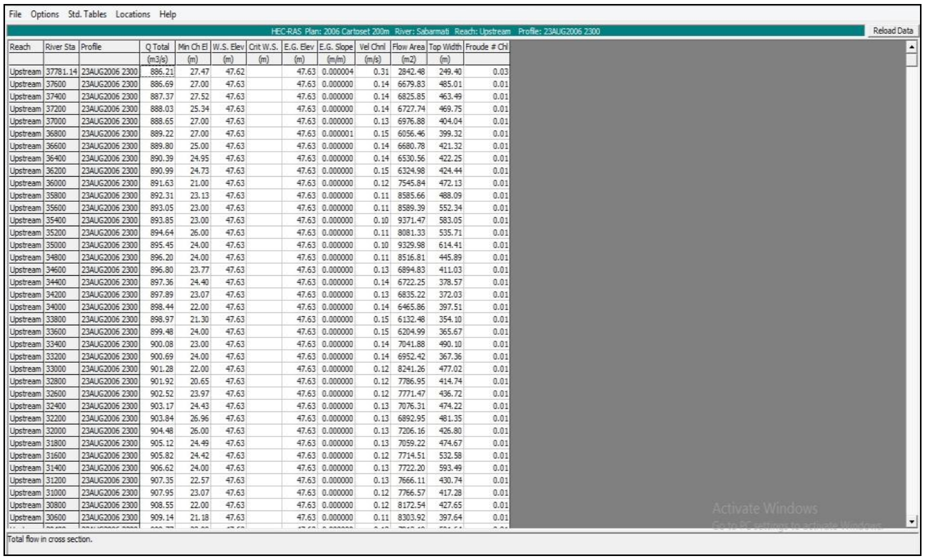
Figure 6.1 Gumbel's flood probabilty analysis

Thus, from above figure 5.1 it has been observed that logarithmic function is best fit to trend of discharge and further used to calculate peak discharge for any return period. The discharge calculated using this frequency distribution has been further used to simulate one dimensional HEC-RAS model under steady condition as discussed in section 6.2.2.

6.2 One dimensional HEC-RAS model

6.2.1 Unsteady flow

To develop 1 dimensional unsteady model for the flood event of the year 2006, open source HEC-RAS 5.0.1 has been used. The flow hydrograph of the respective year at Chiloda Bridge, Gandhinagar and normal depth at Vasana barrage, Ahmedabad have been considered for the upstream and downstream boundary conditions respectively. The flow is simulated under unsteady condition by selecting the computational time step of 1 minute in HEC-RAS. As discussed in chapter 6, section 6.1, Manning's n has been taken as **0.025** for entire river bed and for right and left banks of a total of 39 km patch of the Sabarmati River. HEC-RAS gives output in terms of various hydraulic properties like water surface elevation, flow velocity, discharge, flow area, top width, Froude number, critical velocity, and energy gradient after simulation. The results after simulations can be achieved in tabular form as shown in Figure 6.2 in which hydraulic properties of each profile has been illustrated.



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HEC-RAS Plan: 2006 Cartoset 200m River: Sabarmati Reach: Upstream Profile: 23AUG2006 2300

Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Upstream	37781.14	23AUG2006 2300	886.21	27.47	47.62		47.63	0.000004	0.31	2842.48	249.40	0.03
Upstream	37600	23AUG2006 2300	886.69	27.00	47.63		47.63	0.000000	0.14	6679.83	485.01	0.01
Upstream	37400	23AUG2006 2300	887.37	27.52	47.63		47.63	0.000000	0.14	6825.85	463.49	0.01
Upstream	37200	23AUG2006 2300	888.03	25.34	47.63		47.63	0.000000	0.14	6727.74	469.75	0.01
Upstream	37000	23AUG2006 2300	888.65	27.00	47.63		47.63	0.000000	0.13	6976.88	404.04	0.01
Upstream	36800	23AUG2006 2300	889.22	27.00	47.63		47.63	0.000001	0.15	6056.46	399.32	0.01
Upstream	36600	23AUG2006 2300	889.80	25.00	47.63		47.63	0.000000	0.14	6680.78	421.32	0.01
Upstream	36400	23AUG2006 2300	890.39	24.95	47.63		47.63	0.000000	0.14	6530.56	422.25	0.01
Upstream	36200	23AUG2006 2300	890.99	24.73	47.63		47.63	0.000000	0.15	6324.98	424.44	0.01
Upstream	36000	23AUG2006 2300	891.63	21.00	47.63		47.63	0.000000	0.12	7545.84	472.13	0.01
Upstream	35800	23AUG2006 2300	892.31	23.13	47.63		47.63	0.000000	0.11	8585.66	488.09	0.01
Upstream	35600	23AUG2006 2300	893.05	23.00	47.63		47.63	0.000000	0.11	8589.39	552.34	0.01
Upstream	35400	23AUG2006 2300	893.85	23.00	47.63		47.63	0.000000	0.10	9371.47	583.05	0.01
Upstream	35200	23AUG2006 2300	894.64	26.00	47.63		47.63	0.000000	0.11	8081.33	535.71	0.01
Upstream	35000	23AUG2006 2300	895.45	24.00	47.63		47.63	0.000000	0.10	9329.98	614.41	0.01
Upstream	34800	23AUG2006 2300	896.20	24.00	47.63		47.63	0.000000	0.11	8516.81	445.89	0.01
Upstream	34600	23AUG2006 2300	896.80	23.77	47.63		47.63	0.000000	0.13	6894.83	411.03	0.01
Upstream	34400	23AUG2006 2300	897.36	24.40	47.63		47.63	0.000000	0.14	6722.25	378.57	0.01
Upstream	34200	23AUG2006 2300	897.89	23.07	47.63		47.63	0.000000	0.13	6835.22	372.03	0.01
Upstream	34000	23AUG2006 2300	898.44	22.00	47.63		47.63	0.000000	0.14	6465.86	397.51	0.01
Upstream	33800	23AUG2006 2300	898.97	21.30	47.63		47.63	0.000000	0.15	6132.48	354.10	0.01
Upstream	33600	23AUG2006 2300	899.48	24.00	47.63		47.63	0.000000	0.15	6204.99	365.67	0.01
Upstream	33400	23AUG2006 2300	900.08	23.00	47.63		47.63	0.000000	0.14	7041.88	490.10	0.01
Upstream	33200	23AUG2006 2300	900.69	24.00	47.63		47.63	0.000000	0.14	6952.42	367.36	0.01
Upstream	33000	23AUG2006 2300	901.28	22.00	47.63		47.63	0.000000	0.12	8241.26	477.02	0.01
Upstream	32800	23AUG2006 2300	901.92	20.65	47.63		47.63	0.000000	0.12	7786.95	414.74	0.01
Upstream	32600	23AUG2006 2300	902.52	23.97	47.63		47.63	0.000000	0.12	7771.47	436.72	0.01
Upstream	32400	23AUG2006 2300	903.17	24.43	47.63		47.63	0.000000	0.13	7076.31	474.22	0.01
Upstream	32200	23AUG2006 2300	903.84	26.96	47.63		47.63	0.000000	0.13	6892.95	481.35	0.01
Upstream	32000	23AUG2006 2300	904.48	26.00	47.63		47.63	0.000000	0.13	7206.16	426.80	0.01
Upstream	31800	23AUG2006 2300	905.12	24.49	47.63		47.63	0.000000	0.13	7059.22	474.67	0.01
Upstream	31600	23AUG2006 2300	905.82	24.42	47.63		47.63	0.000000	0.12	7714.51	532.58	0.01
Upstream	31400	23AUG2006 2300	906.62	24.00	47.63		47.63	0.000000	0.13	7722.20	593.49	0.01
Upstream	31200	23AUG2006 2300	907.35	22.57	47.63		47.63	0.000000	0.13	7666.11	430.74	0.01
Upstream	31000	23AUG2006 2300	907.95	23.07	47.63		47.63	0.000000	0.12	7766.57	417.28	0.01
Upstream	30800	23AUG2006 2300	908.55	22.00	47.63		47.63	0.000000	0.12	8172.54	427.65	0.01
Upstream	30600	23AUG2006 2300	909.14	21.18	47.63		47.63	0.000000	0.11	8303.92	397.64	0.01

Total flow in cross section.

Figure 6.2 Profile output table for 1 dimensional HEC-RAS model

Apart from tabular form, the graphical output in terms of 3-dimensional profile plot of river (Figure 6.3) has been studied.

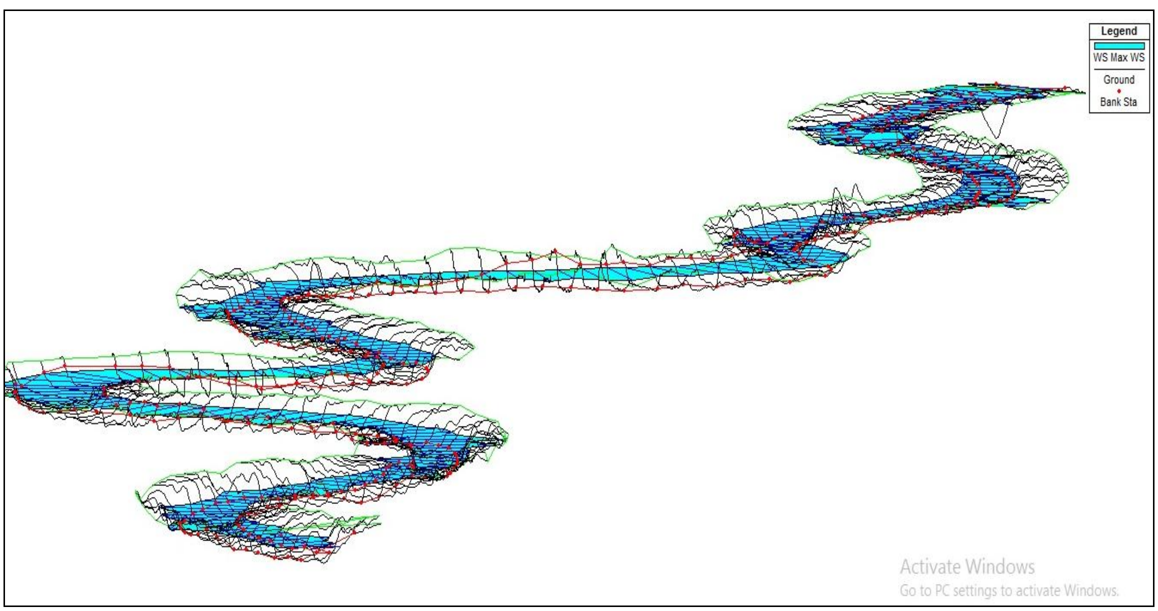


Figure 6.3 Three dimensional perspective plot of Sabarmati River for maximum water surface profile

Further, details for individual sections at Chiloda Bridge (upstream boundary), Subhash Bridge (gauging station considered for validation) and Vasana Barrage (downstream boundary), has been obtained for analysis as shown in Figure 6.4 (a), (b) and (c) respectively.

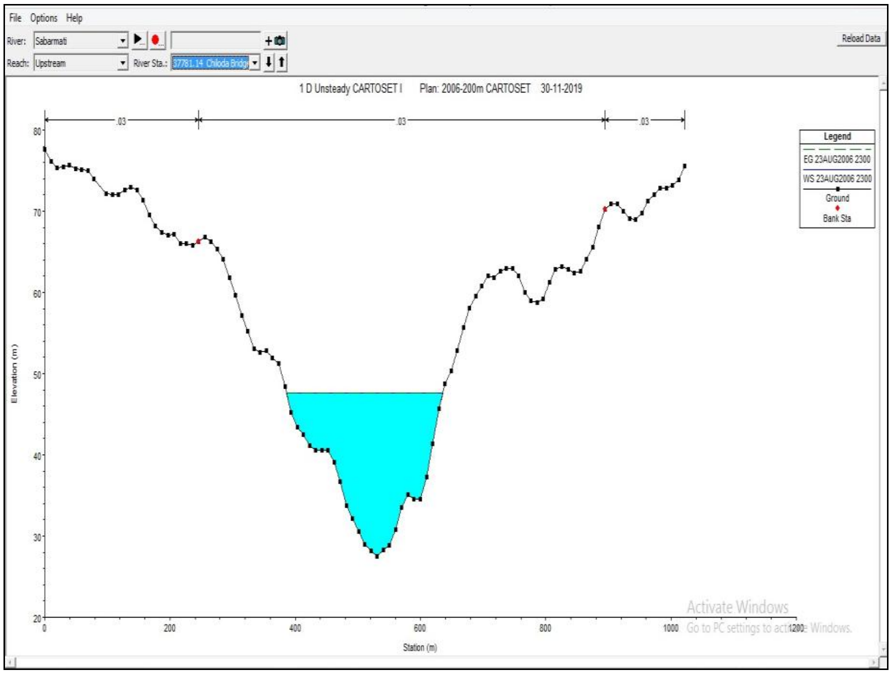


Figure 6.4 (a) Cross section of Sabarmati River at Chiloda Bridge (Upstream boundary)

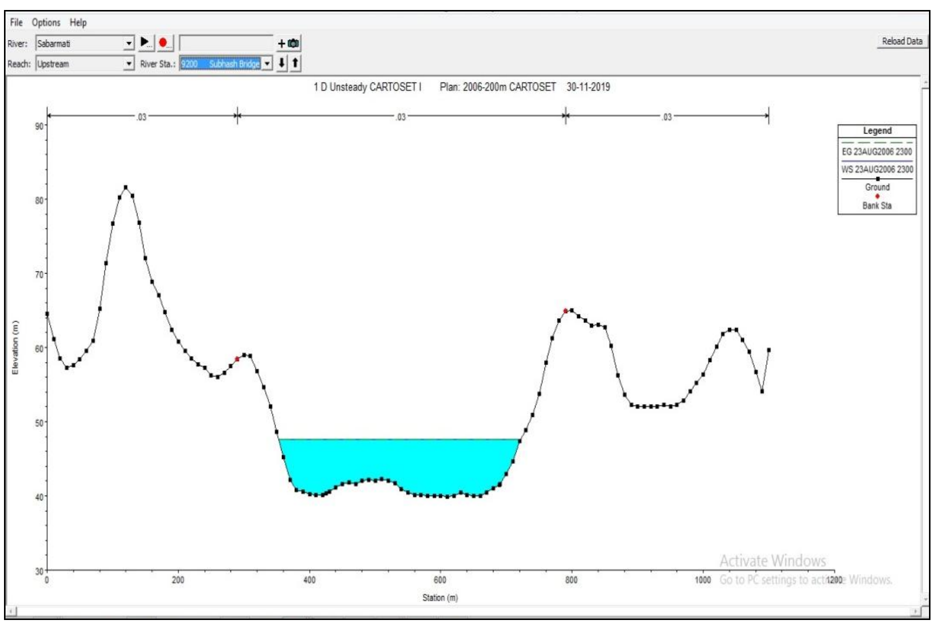


Figure 6.4 (b) Cross section of Sabarmati River at Subhash Bridge (Gauging station for validation)

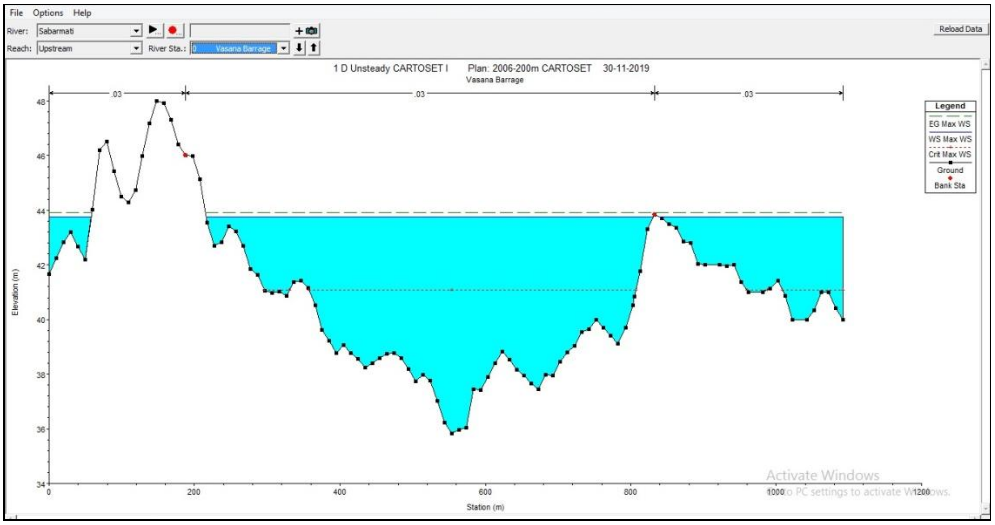


Figure 6.4 (c) Cross section of Sabarmati River at Vasana Barrage (downstream boundary)

The stage hydrograph of the year 2006, as shown in Fig. 6.5 is within an acceptable range with actual hydrograph with nearly equal peak flood value of actual stage hydrograph, while the simulated stage hydrograph of the year 2007 as shown in Fig. 6.6 gives higher flood peak than the actual one.

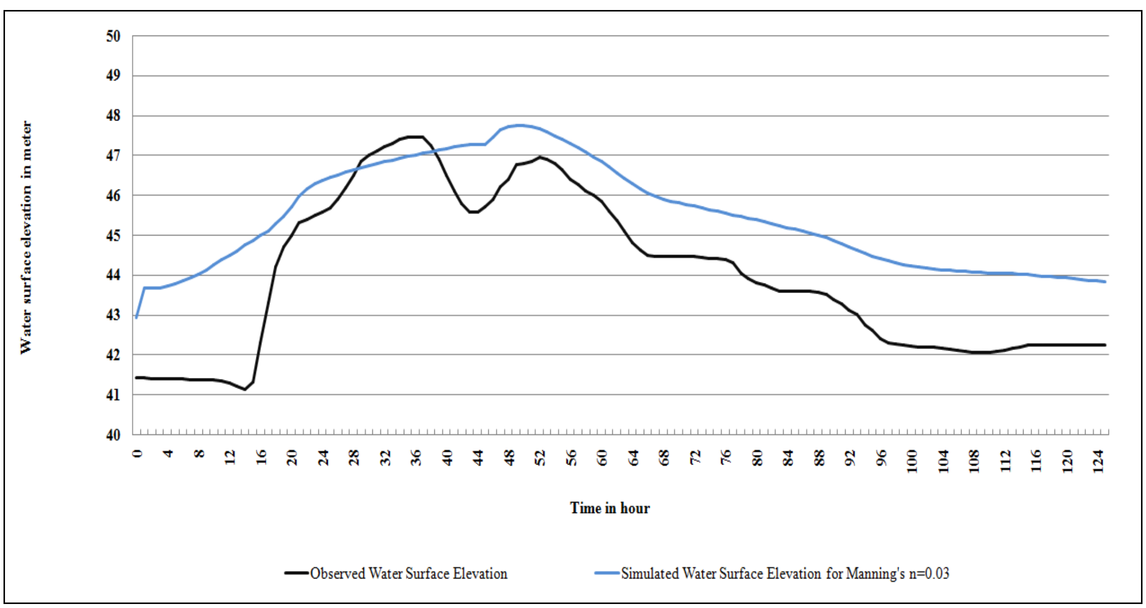


Figure 6.5 Comparision of observed and simulated stage hydrograph at Subhash Bridge for year 2006

For year 2006, the simulated stage hydrogaph at Subhash bridge shows higher stages in range of average 1 to 1.5 meters for most of the river length but from 19th August 2006, 23:00 hours to 20th August 2006, 08:00 hours the simulated stage hydrograph shows drop of 0.1 to 0.5 m than observed stages of river.

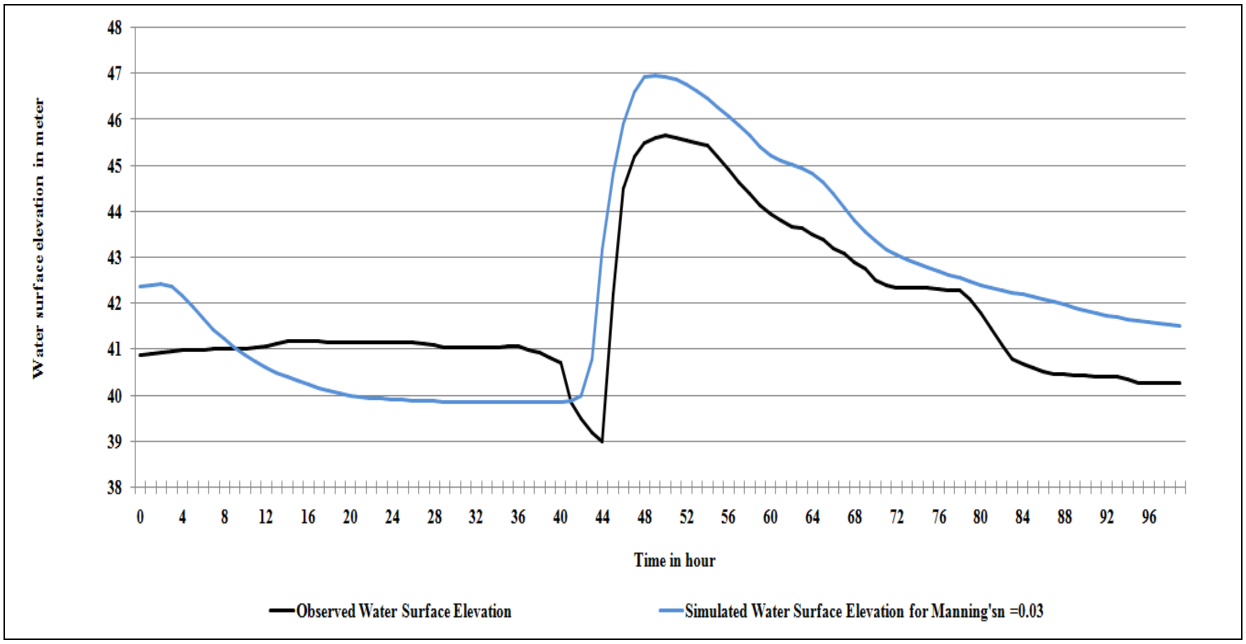


Figure 6.6 Comparision of observed and simulated stage hydrograph at Subhash Bridge for year 2007

For year 2007, the simulated stage hydrogaph at Subhash bridge shows lower values in range of 0.3 to 1.2 m than observed stages for time period of 8th July 2007, 08:00 hours to 9th July 2007, 13:00 hours. For rest of the flood duration, the values of simulated stages of river is higher than observed stages of river in range of 1-2 m. The maximum value of observed water level at Subhash Bridge for year 2007 was 45.65 m on 9th July 2007, 23:00 hours , which was given as 46.94 m by model, thus resulting approximately 1.2 m higher value of water level in River.

For analysis purpose, the simulated value of water surface elevation at Subhash Bridge gauging site has been compared in terms of RMSE, Average absolute difference and average difference (Table 6.2) with observed value for both the years 2006 and 2007.

TABLE 6.2 Calculation of root mean square error (RMSE), average absolute difference and average difference of water surface elevation at Subhash Bridge gauging site

Flood Year	RMSE	Avg. absolute Difference (in meter)	Avg. difference (in meter)
2006	1.65	1.49	-1.44
2007	1.66	1.41	-0.97

As shown in Table 6.2, RMSE for both the year 2006 and 2007 are 1.65 and 1.66 respectively which are within satisfactory range. The average absolute difference for the year 2006 is 1.49 meter and 1.41 meter for the year 2007 which also shows the fair agreement of the simulated model. The value of average difference between observed and simulated water surface elevation for year 2006 and 2007 are -1.44 and -0.97, the negative value shows that output of model is in higher side than actual value. It has been observed that for both criteria of the comparison year 2007 data gives more fair results than the year 2006. The stage hydrograph for both the years gives slightly higher values which may be due to high discharge, which makes excess water to spill over any of the banks and becomes 2 dimensional in nature. For such events, the combined 1D/2D hydrodynamic simulation should be developed for preparing inundation maps. Also, for more realistic results, sediment transport during flood should also be considered.

The water surface elevation for maximum flow of water at Chiloda Bridge on August 20, 2006 at 17:00 hours has been compared with ground elevations of left bank and right banks of the Sabarmati River as shown in Fig.6.7 and 6.8 respectively. The detail output of maximum water surface elevation for flood event of year 2006, along the left and right bank of the Sabarmati river has been shown as annexure-I. It has been observed that out of total 197 cross sections, 129 numbers on left bank and 95 numbers on right bank has higher water surface elevation than ground level of their respective banks. It indicated that approximately 65.48% area on left bank and 48.22 % area on right bank has possibilities of water spill and inundation for discharge equal or more than 6472 m^3/s.

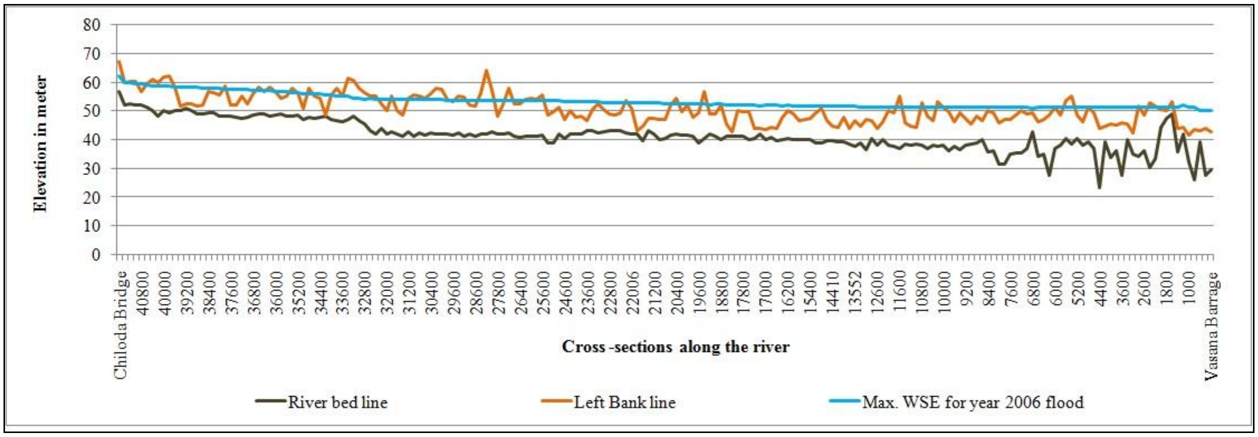


Figure 6.7 Water surface elevation for maximum discharge on 20th August, 2006, 17:00 hours along left bank of river for year 2006 flood

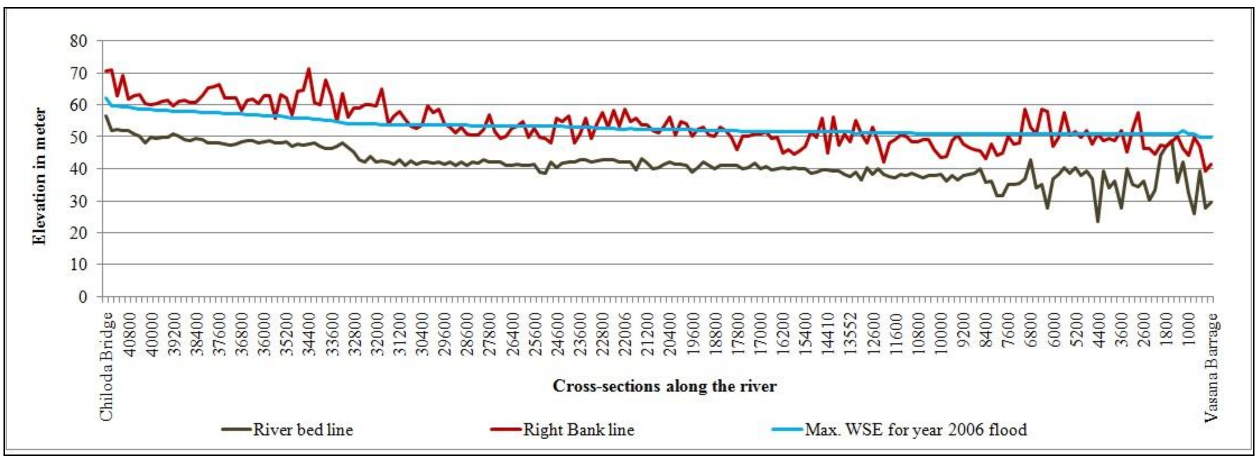


Figure 6.8 Water surface elevation for maximum discharge on 20th August, 2006, 17:00 hours along right bank of river for year 2006 flood

As per the graphs, the critical cross sections identified are 7800, 5000 and 6000 on left bank of river which specify areas of Dudheshwar, Khanpur and Shahpur respectively in Eastern Ahmedabad. On Right bank of Sabarmati, cross sections between 7600-8600, 7000, 6400 and 2000 showing more concentration of flood which indicate Ashram road, Vadaj, Usmanpura and Paldi area in Western Ahmedabad. On left bank, simulated discharge at Dudheshwar, Khanpur and Shahpur was found to be 6538 m^3/s, 4758.54 m^3/s and 5368 m^3/s respectively, which is higher than its actual discharge carrying capacity. Also, water surface elevation at

Dudheshwar, Khanpur and Shahpur comes out as 51.12 m, 51.02 m and 51.03 m respectively which is greater than their respective levels of banks of 45.07 m, 50.13 m and 47.31 m and so resulted in water spill in specific area. The encroachment in river bed in form of slum area makes difficult for river water to flow easily. Also, on right bank, areas like Ashram road, Usmanpura and Vadaj having simulated discharge of 6960 m^3/s, 5614 m^3/s and 6087 m^3/s exceeding River's safe carrying capacity. The water surface elevation at Ashram road, Usmanpura and Vadaj come out as 51.03 m, 51.06 m and 51.14 m respectively which is higher than bank levels of 46.53 m, 47.16m and 48.95 m respectively indicating over-spilling of excess water from River.

6.2.2 Steady flow

The one dimensional simulation of flow under steady condition has been performed for probable discharge of return period of 20, 25, 30, 50, 60, 75 and 100 years using HEC-RAS. The probable discharge calculated for various return periods using Gumbel's flood frequency analysis has been used to simulate steady HEC-RAS model. The geometric data and hydrologic data have been considered same as of unsteady flow simulation. The maximum water surface elevation for each profile has been simulated and maximum water surface elevation of 20 year and 100 year return period have been plotted against elevations of left bank and right bank as shown in figure 6.9 and 6.10 respectively to check for overflowing of water from the either banks.

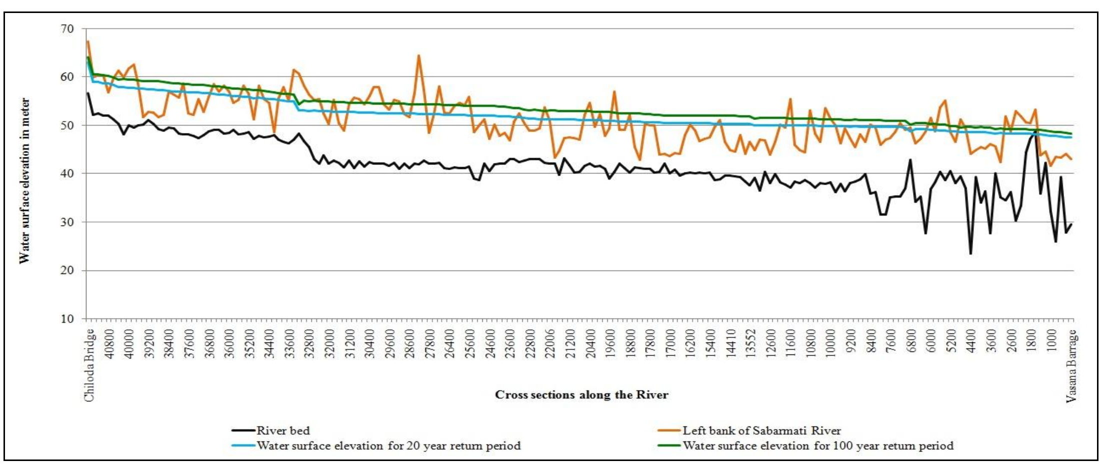


Figure 6.9 Water surface elevation for maximum discharge for 20 years and 100 years return period along left bank of Sabarmati river

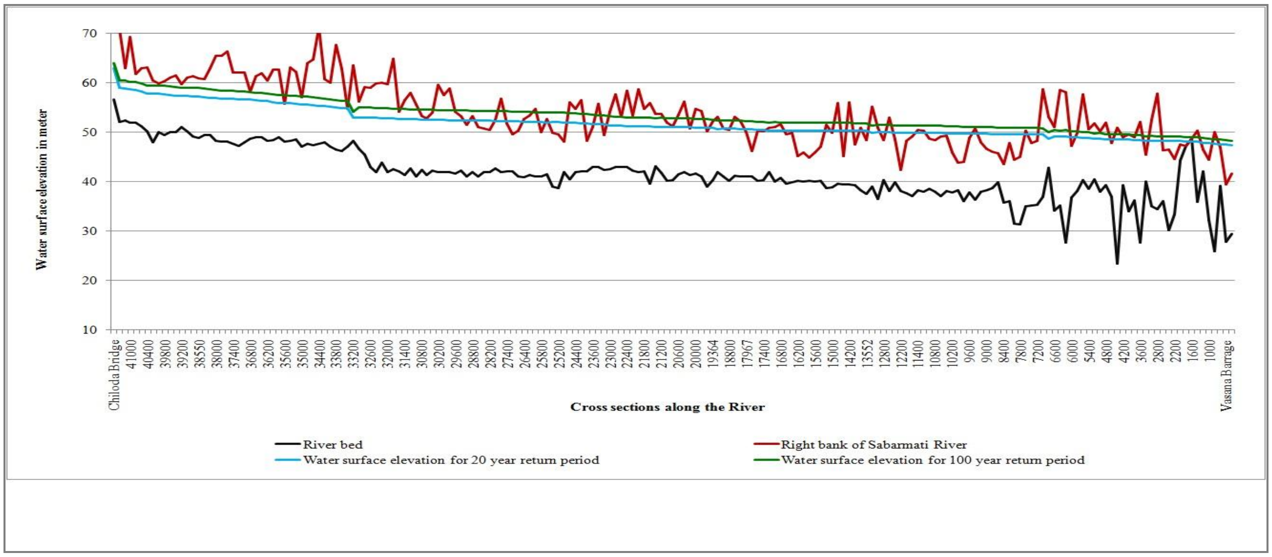


Figure 6.10 Water surface elevation for maximum discharge for 20 years and 100 years return period along right bank of Sabarmati river

From the above graphical representation of maximum water surface elevation corresponding to elevations of left and right bank gives probability of water to get spill over from the particular banks. It has been observed that the left bank representing east side of river is more vulnerable to water spill than right bank representing west side of river. The river is represented as total 197 number of cross sections from Gandhinagar city of Ahmedabad city and probability of number of cross sections on both the sides of banks has been calculated to check the vulnerable zone for high water level for different return periods along the study reach of the Sabarmati river as shown in Table 6.3 below.

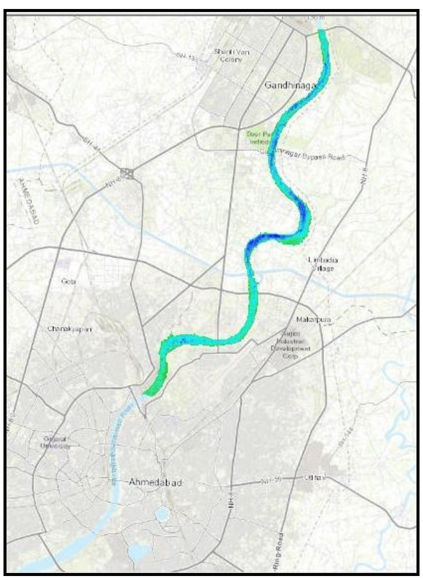
TABLE 6.3 Calculation of probability of inundation of areas along left and right bank of Sabarmati River for various return periods

Profiles	Return Period in years	Discharge in cumec	Left Bank (East)		Right Bank (West)	
			No. of C/s Over topped	% of C/s Over topped	No. of C/s Over topped	% of C/s Over topped
PF 1	20	4845.27	116	**58.88**	70	**35.53**
PF 2	25	5185.84	119	**60.41**	72	**36.55**
PF 3	30	5465.72	122	**61.93**	73	**37.06**
PF 4	50	6239.59	132	**67.01**	87	**44.16**
PF 5	60	6514.41	137	**69.54**	89	**45.18**
PF 6	75	6849.93	140	**71.07**	96	**48.73**
PF 7	100	7451.83	146	**74.11**	101	**51.27**

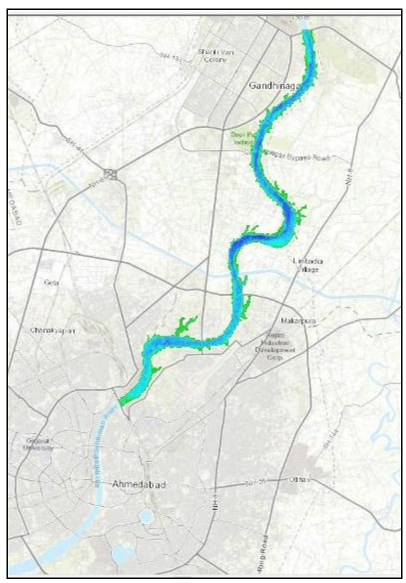
The probable peak discharge for return periods of 20, 25, 30, 50, 60, 75 and 100 years calculated from Gumbel's frequency distribution has been simulated under steady condition and maximum water surface levels have been compared for elevations of both the left (east) and right (west) banks of Sabarmati river for all the 197 numbers of cross sections. The cross sections having elevation lesser than relative water surface elevation on corresponding bank has marked as unsafe and have probability of water spill. As shown in Table 6.3, the percentage of cross section prone to spilling of water during high discharge on left bank is 58 % and for right bank is 35.33 % for 20 year return period while for 50 year return periods it has come out as 67.01 % on left bank and 44.16 % on right bank. For all the return periods, it has been observed that left bank of Sabarmati River which represents east side of old wall city is more prone to water spill from higher water levels in River than the right bank of city locating on west side of river considered as new Ahmedabad city. Though, during literature study and discussion with government authorities it has been learn that west side of city faces more inundation problem than east side due to insufficient storm water drainage network and difficulty of rainwater to drain out during torrential rain.

6.3 Two dimensional HEC-RAS model

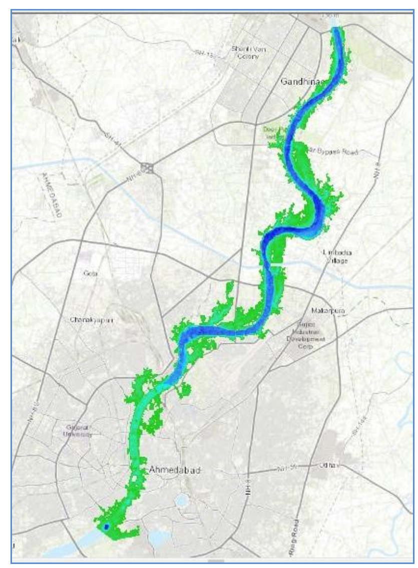
The flood event of year 2006 has been simulated for the time of 18 August 2006 18:00 hour to 23rd August 2006 23:00 hours under unsteady condition. The calibrated manning's roughness value of 0.025 has been considered for Sabarmati River and for floodplain the values of other surfaces roughness has been considered as per guidelines given by DRIP, India. The Cartoset 1 DEM of 10 m grid interval has been considered as terrain and upstream and downstream boundaries are considered same as considered in 1 D model. The 2D HEC-RAS model simulated results in terms of maps of various hydraulic properties like, depth, velocity, water surface elevations and inundation boundaries for every 4 hours has been stored and mapped with ward map of Ahmedabad city for analysis.



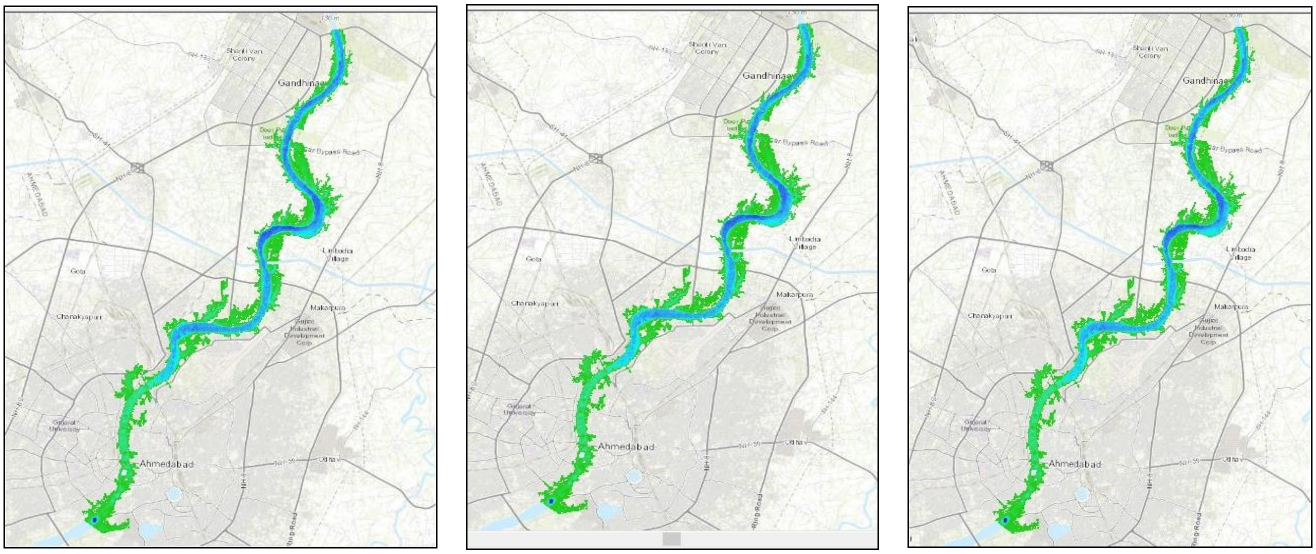
19 August 2006 11:00 h, Chiloda
Discharge = 2548.37 cumecs



19 August 2006 23:00 h, Chiloda
Discharge = 4219.23 cumecs



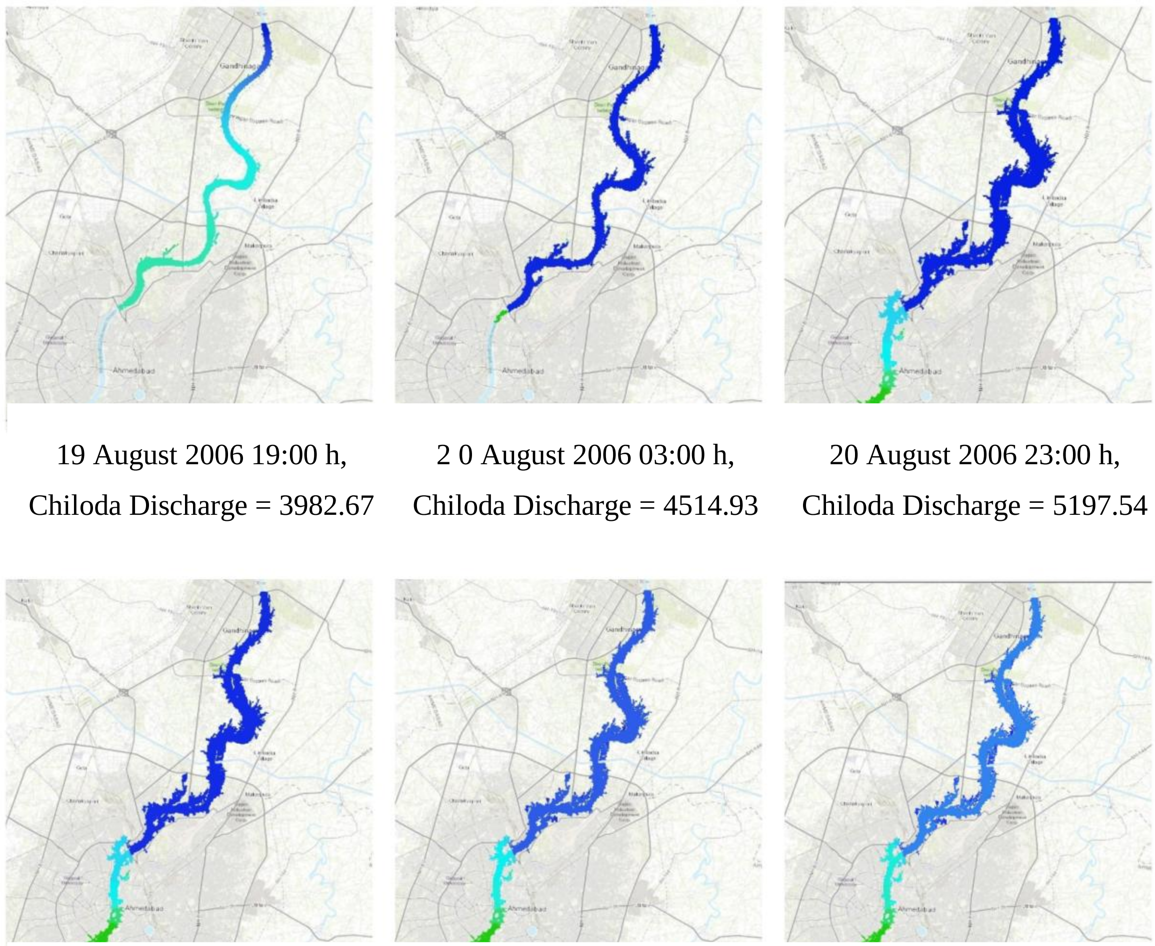
20 August 2006 17:00 h, Chiloda
Discharge =6472.12 cumecs



21 August 2006 15:00 h, Chiloda Discharge = 2577.69 cumecs | 22 August 2006 19:00 h, Chiloda Discharge = 1119.4 cumecs | 23 August 2006 23:00 h, Chiloda Discharge = 886.21 cumecs

Figure 6.11 Simulated flood depth of Sabarmati River in year 2006 corresponding to the discharge at Chiloda Bridge, Gandhinagar

The depth of flood of 2006 year flood event has been shown in Figure 6.11 for corresponding discharge at Chiloda Bridge. The Simulated results show that on 19 August 2006, 11:00 hours the maximum depth of water was 12 m and 17 m in the river near Raysan village and Koba village respectively and average 10 m near Koteshwar with corresponding discharge of 2548.37 cumecs at Chiloda Bridge. Further during 19 August 2006, 23:00 hours to 20 August 2006, 17:00 hours, due to increase in discharge from 4219.23 cumecs to 6472.12 cumecs at Chiloda Bridge, the average depth of water increased in range of 27 m to 30 m in river near Gandhinagar and Ahmedabad and water start spilling in surrounding areas having maximum depth of 3.5 m near Deer park, 3.2 m in Indroda village, 1.3 m in Dholakuva village, 1.5 m in GNLU, 2.2 m near Koba village, 4.12 m near Koteshwar, 3.5 m near Usmanpura and 3.8 m near Wadaj on right bank. The maximum depth of 2 m near NIPER-A, 2.8 m near IITGn campus, 1.5 m near Basan village, 5.2 m near GIFT city, 1.8 m Sadar bazaar, 2.8 m Dudheshwar, 3.8 m near Shahpur and 4 m near Khanpur and 3.2 m near Asarva on left bank of Sabarmati River. At the same time, between 20 August 2006, 17:00 hours to 21 August 2006, 15:00 hours, though the discharge at Chiloda bridge decreased from 6472.12 cumecs to 2577.69 cumecs the area exposed to different flood depths increases rapidly; then remains almost constant for maximum depth of flood in surrounding area up to maximum of 4.5 m in Ahmedabad. The depth of flood start decreasing at 22 August 2006, 19:00 hours due to decrease in corresponding discharge of 1119.4 cumecs at Chiloda Bridge resulting in decreased depth of 1.2 m in areas near banks.



19 August 2006 19:00 h, Chiloda Discharge = 3982.67

2 0 August 2006 03:00 h, Chiloda Discharge = 4514.93

20 August 2006 23:00 h, Chiloda Discharge = 5197.54

21 August 2006 03:00 h, Chiloda Discharge = 4347.82

22 August 2006 03:00 h, Chiloda Discharge = 2071.35

23 August 2006 23:00 h, Chiloda Discharge = 886.21

Figure 6.12 Simulated flood water levels of Sabarmati River in year 2006 corresponding to the discharge at Chiloda Bridge, Gandhinagar

Also, as shown in figure 6.12, behavior of water surface elevation in the Sabarmati River for flood event of year 2006 has been studied which indicates the gradual increase in water levels at 19 August 19:00 hours the maximum WSE recorded in river as 45 m with corresponding discharge of 3982.67 cumecs at Chiloda Bridge. The right and left bank starts spilling near. Koteshwar and GIFT city area respectively on 20 August 2006, 03:00 hours with maximum water surface elevation being 52 m with corresponding discharge of 4514.93 cumecs and at maximum discharge of 6472.12 cumecs at Chiloda Bridge on 20 August, 17:00 hours the highest water surface elevation reached up to maximum of 58 m, inundating low lying areas located on both the banks of River but the areas of old city like Shahpur, Khanpur and Asarva on east bank and new city area like Usmanpura and Vadaj on west banks were affected the most as shown in Fig.6.13 below,

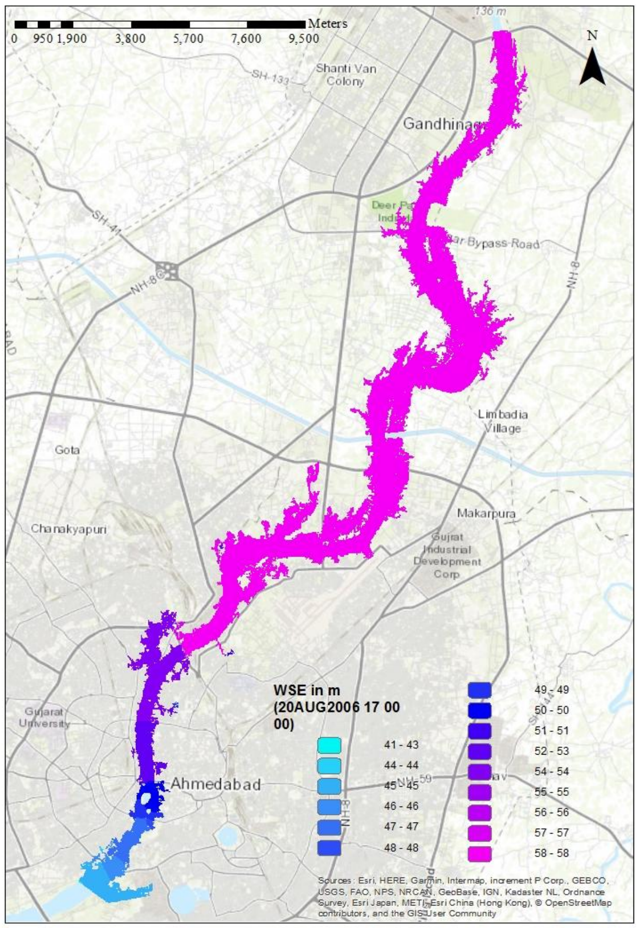


Figure 6.13 Water surface elevation map of Ahmedabad and Gandhinagar City, 20 August 2006, 17:00 hours (Peak discharge profile)

The Figure 6.14 below signifies the time taken by flood water to arrive at specific area along the study reach of the Sabarmati River for the maximum discharge of 6472.12 cumecs at Chiloda Bridge on 20 August, 17:00 hours.

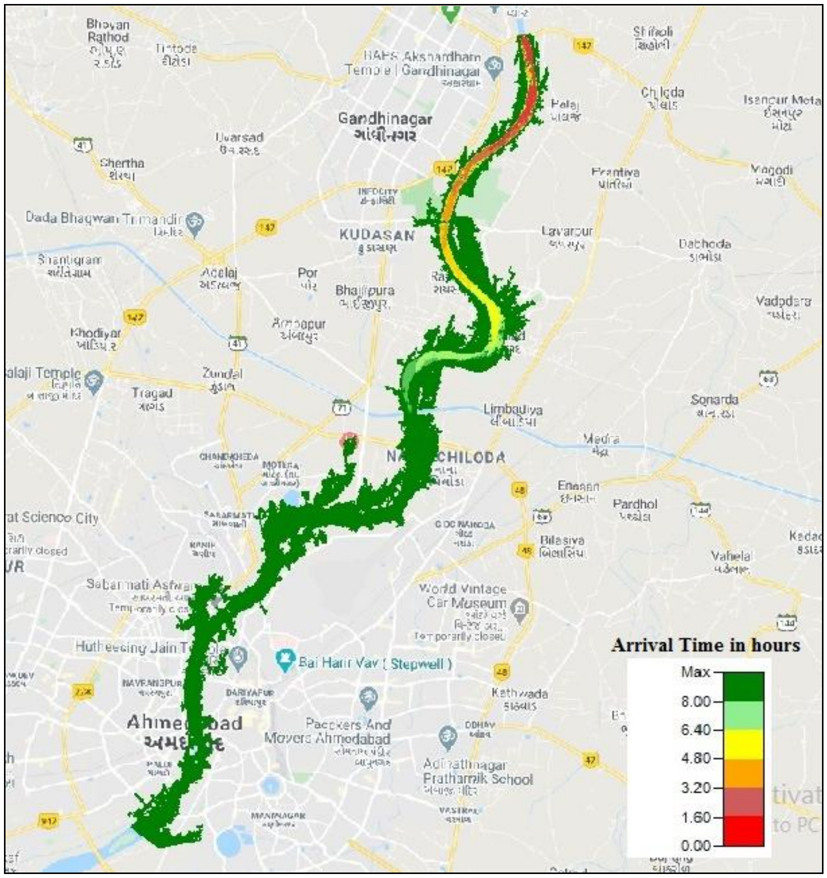


Figure 6.14 Arrival time of water to the areas along study reach for maximum discharge at Chiloda Bridge

It has been observed from the above Figure 6.14 that during initial period of around 2 hours water arrivel till Kudasan village of Gandhinagar and almost within next two hours it reaches upto Valad and Koba villages. It has been observed from map that it takes water approximately 8 hours to reach its maximum water level of 58 m in river and to spill over the banks resulting in inundation in surrounding areas.

As the velocity is very important property of flow of water, the velocity distribution of flood water during entire flooding event has been studied. As, shown in figure 6.15 below, on 20

August 2006, 17:00 hours the discharge at Chiloda bridge reaches its peak value of 6472.12 cumecs and from the above figures of 6.13 and 6.14, it has been clearly observed that the low lying areas on left bank and right banks remains flooded for almost 34 hours till 22 August 2006, 03:00 hours. It has been clearly observed from figure 6.15 that for peak discharge duration the velocity of flow remains mostly in range of 0 to 1 m/sec for entire study area which indicates longer duration of water retention and prolonged time of inundation. The higher value of flood velocity indicates speedy disposal of inundated water from the water logged areas. As the study areas of the Sabarmati River has comparatively flat terrain with very mild slope towards downstream, the lower value of flow velocity increase the inundation time and obstruct the disposal of water through gravity.

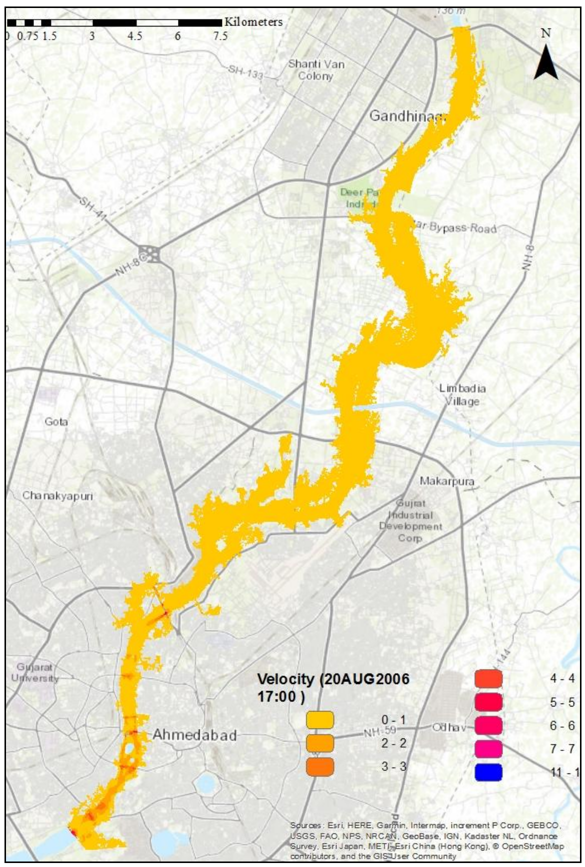


Figure 6.15 Velocity distribution maps of Ahmedabad and Gandhinagar City, 20 August 2006, 17:00 hours (Peak discharge profile)

The inundation area of year 2006 flood event during 18 August, 18:00 hours and 23 August, 23:00 has been calculated from maps generated in RAS mapper. The percentage of area inundated during flood event has been found out for total 269 km^2 area covering Ahmedabad and Gandhinagar cities along banks of Sabarmati River as shown in table 6.4 below;

TABLE 6.4 Time, discharge at Chiloda Bridge and percentage area inundated of Sabarmati River for year 2006 flood event

Sr. No	Date and Time	Discharge at Chiloda Bridge in cumecs	Total Area Inundated in km^2	% Area Inundated
1	18 August 2006, 19:00 hours	758.95	0.075	0.03
2	18 August 2006, 23:00 hours	1206.31	0.29	0.11
3	19 August 2006, 03:00 hours	1653.67	0.6	0.22
4	19 August 2006, 07:00 hours	2101.03	0.87	0.32
5	19 August 2006, 11:00 hours	2548.37	1.05	0.39
6	19 August 2006, 19:00 hours	3982.67	1.27	0.47
7	19 August 2006, 23:00 hours	4219.23	1.44	0.54
8	20 August 2006, 11:00 hours	4988.06	2.74	1.02
9	20 August 2006, 17:00 hours	6472.12	4.14	1.54
10	20 August 2006, 18:00 hours	6259.69	4.1	1.52
11	20 August 2006, 23:00 hours	5197.54	3.82	1.42
12	21 August 2006, 03:00 hours	4347.82	3.95	1.47
13	21 August 2006, 15:00 hours	2577.69	3.79	1.41
14	22 August 2006, 03:00 hours	2071.35	3.78	1.41
15	22 August 2006, 19:00 hours	1119.4	2.85	1.06
16	23 August 2006, 03:00 hours	1102.76	2.84	1.06
17	23 August 2006, 15:00 hours	1054.94	2.84	1.06
18	23 August 2006, 23:00 hours	886.21	2.74	1.02

As shown in table 6.4 above, at discharge of 4219.23 cumecs at Chiloda Bridge on 19 August 2006, 23:00 hours the banks of Sabarmati started to get overflow and out of total 269 km^2 area considered for 2D simulation, 1.44 km^2 area inundated which is 0.54 % of total area of Ahmedabad and Gandhinagar cities and almost 4.14 km^2 area which is 1.54 % of study area got inundated on 20 August 2006, 17:00 hours for peak discharge of 6472.12 cumecs at Chiloda Bridge. Then after approximately 5 hours on 20 August 2006, 23:00 hours, discharge at Chiloda Bridge gradually start decreasing up to 5197.5 cumecs and corresponding inundation also decreased as 3.82 km^2 areas which is 1.42 % of total study area. Then, gradually, with decrease in discharge from 4347.82 cumecs on 21 August 2006, 03:00 hours to 1119.4 cumecs on 22 August 2006, 19:00 hours the percentage of inundated area also decreased from 1.47 % to 1.06 % of total area.

From the simulated results, it has been found out that the maximum area was inundated for discharge of 6472.12 cumecs at Chiloda bridge on 20 August 2006, 17:00 hours where almost 4.14 km^2 area out of 269 km^2 area was inundated as shown in Figure 6.16 as follows;

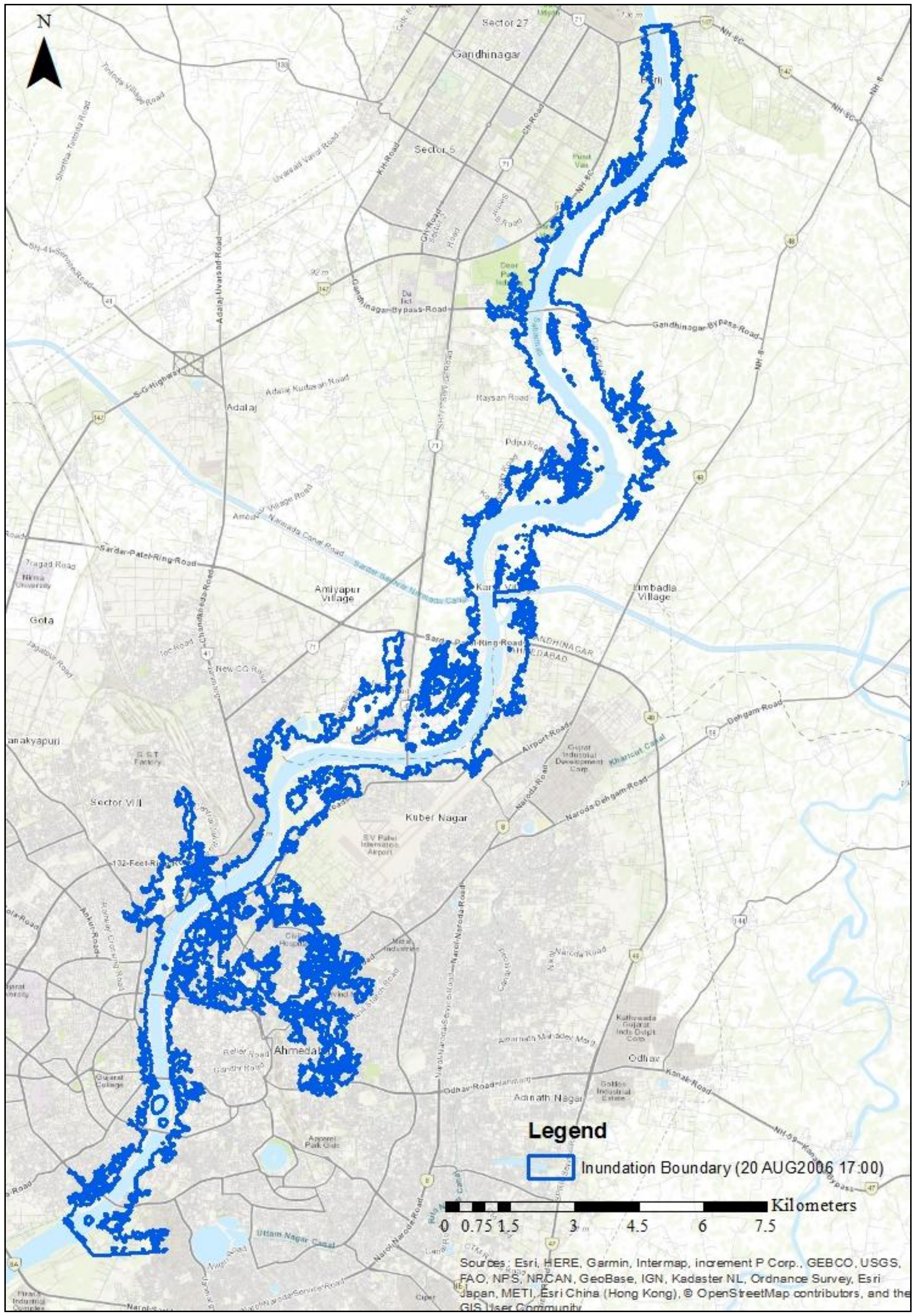


Figure 6.16 Flood inundation boundary of Sabarmati River, 20 August 2006, 17:00 hours (Peak discharge profile)

The mapping of inundated area for peak discharge of 6472.12 cumecs at Chiloda bridge has been done with base map of study area individually in two parts from which one being before

Narmada main canal covering Gandhinagar city and second being after Narmada main canal covering Ahmedabad city as shown in Figure 6.17 (a) and (b) respectively.

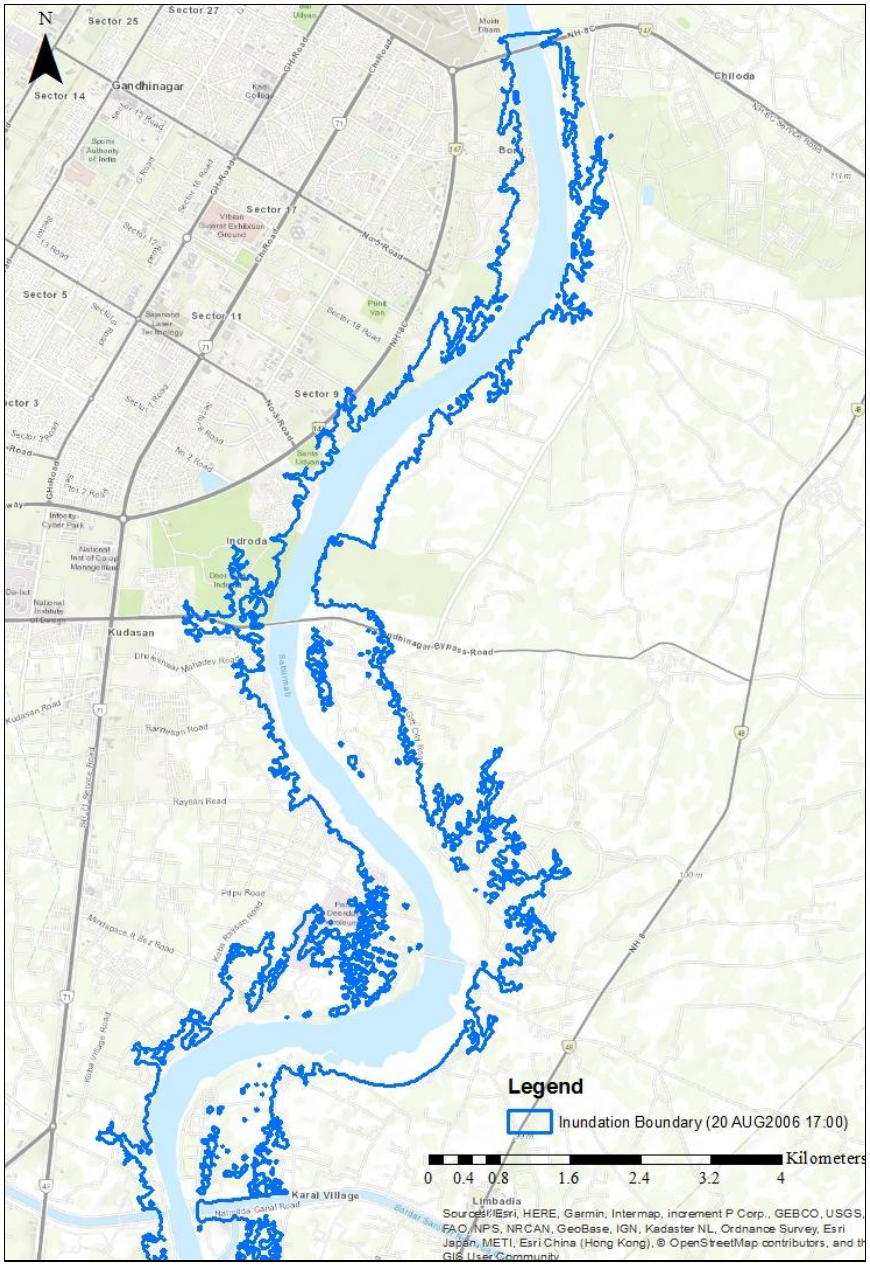


Figure 6.17 (a) Flood inundation boundary of Sabarmati River at Gandhinagar city, 20 August 2006, 17:00 hours (Peak discharge profile)

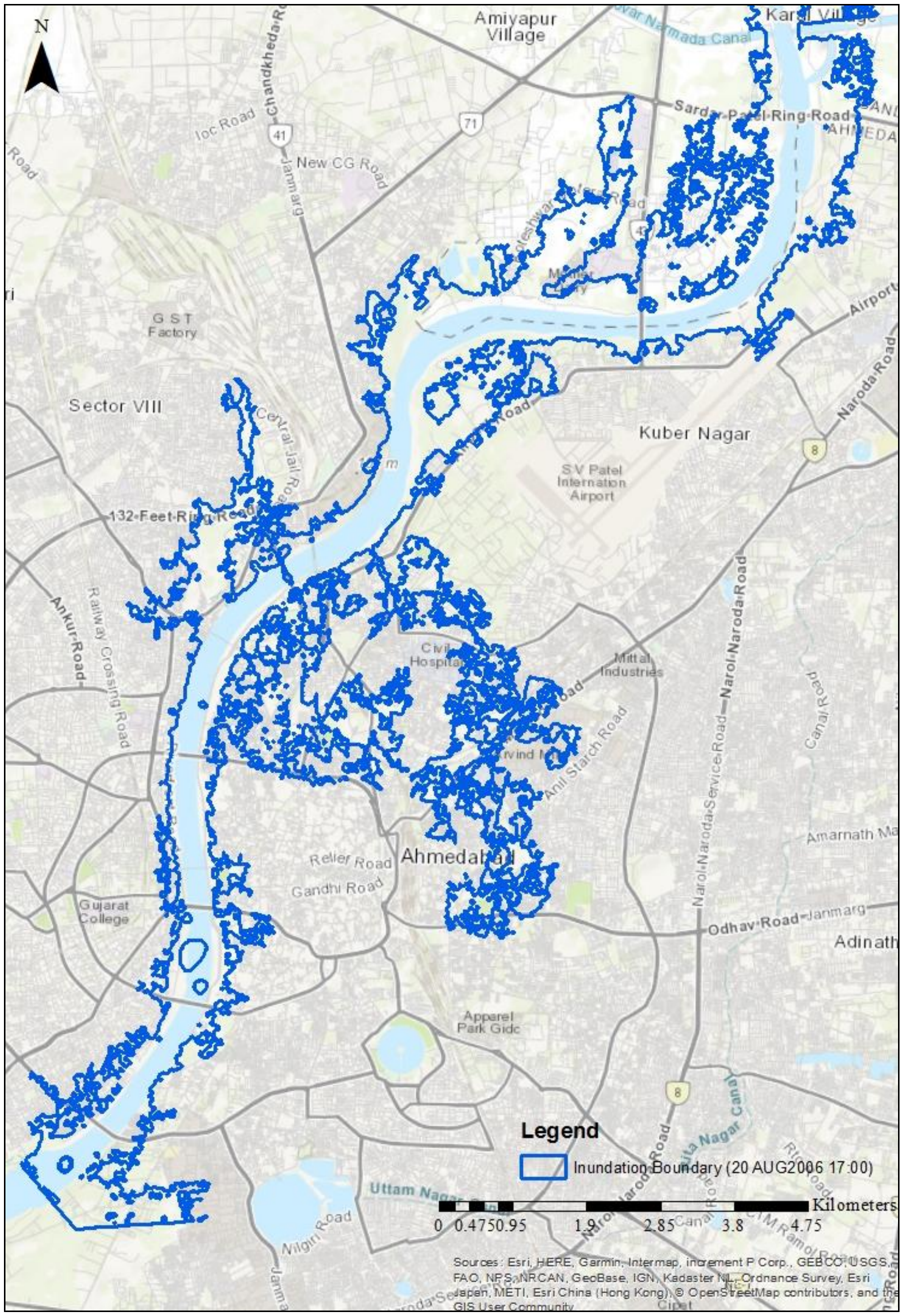


Figure 6.17 (b) Flood inundation boundary of Sabarmati River at Ahmedabad city, 20 August 2006, 17:00 hours (Peak discharge profile)

From the above Figures 6.17 (a) and (b) the areas of Gandhinagar and Ahmedabad cities having depth of water more than 3 m are considered at high risk and depth of water less than 3 m are considered as at low risk for water inundation. This classification has been done as per their risk to get inundated at maximum discharge of 6472.12 cumecs at Chiloda Bridge as shown in Table 6.5 below;

TABLE 6.5 Classification of areas according to their risk to get inundated for peak discharge of year 2006 flood event of Sabarmati River

City	Right Bank		Left Bank	
	High Risk	Low Risk	High Risk	Low Risk
Gandhinagar	Deer Park, Indroda	Borij, Indroda, Sector-9, Gujarat National Law University (GNLU), Koba, Dholakuva, Nabhoi Village	GIFT City	Palaj village, IIT-Gandhinagar, NIPER-A
Ahmedabad	Koteshwar, Sardar Patel Stadium, Ashram road, Vadaj, Usmanpura	Institute of Plasma Research (IPR), Torrent Power house, Sabarmati, Paldi, Vasana	Asarva, Khanpur, Shahpur, Raipur	Sadar bazaar, Shahibaug, Dudheshwar, Astodia, Behrampura

From the above Table 6.5, it has been understood that for the discharge equal to or more than flood event of year 2006, the areas Deer park, Indroda of Gandhinagar and Koteshwar, Sardar Patel Stadium, Ashram road, Vadaj and Usmanpura of Ahmedabad on right bank and areas of GIFT city of Gandhinagar and Asarva, Khanpur, Shahpur and Raipur of Ahmedabad on left bank of Sabarmati River are at high risk and severely face water inundation while areas of Borij, Indroda, sector-9, Gujarat National Law University (GNLU), Koba village, Dholakuva village, Nabhoi Village of Gandhinagar and Institute of Plasma Research (IPR), Torrent Power house, Sabarmati, Paldi, Vasana of Ahmedabad on right bank and Palaj village, IIT-Gandhinagar, NIPER-A of Gandhinagar and Sadar bazaar, Shahibaug, Dudheshwar, Astodia, Behrampura of Ahmedabad on left bank are at low risk and moderately face water inundation.

The results generated from 2D simulation in terms of Inundation maps, the areas along the banks of Sabarmati River can be classified on basis of their respective risk of getting inundated for any future value of corresponding discharge at upstream boundary which is Chiloda Bridge for this study. The output generated from 2D simulation can be further utilized in preparing forecasting maps for any value of future discharge of Ahmedabad city and corresponding early warning system can be established using simulated results which will prove to be great help to evacuate people and livestock for future release from upstream.

CHAPTER 7

SENSITIVITY ANALYSIS

The sensitivity of model can be observed by the output it delivers corresponding to change in the input variable parameters, which can be maneuvered to give best fit of model. The (McCuen, 1973) has defined sensitivity as, "the rate of change in one factor with respect to change in another factor", and mentioned it as mathematical tool which is proficiently important in development, calibration and validation of hydrologic models. (Forghanparast, Siosemarde, Mohamadi, & Merufinia, 2014) have investigated various relationships of manning's n in fixed riverbed and moving river bed. This study concludes that bed's roughness is a function of particle's shape, type and size and roughness coefficient increases with increase in changes of cross section. Also, the sensitivity analysis of roughness and cross section spacing has been studied by (Li et al., 2014) in which he concluded that at greater flow rate, change of water depth is more sensitive to roughness and cross section spacing. Thus, the primary response variables that have been used in this research to quantify sensitivity of 1dimensional HEC RAS model are DEM resolution and cross section spacing which has been utilized to generate river geometry. Both these variable are prime input of the HEC-RAS modeling as entire model runs on the basis of geometry of channel created using topographic data.

The sensitivity of 1dimensional model in terms of water surface elevation has been analyzed for varying values of manning's roughness, grid size of DEM and cross section spacing. The sensitivity of simulated water surface elevation corresponding to 30 m resolution ALOS 3D and 10 m resolution Cartoset 1 DEM and cross section spacing of 300 m and 200 m has been calculated. The sensitivity of aforesaid variables has been checked for two flood events of year 2006 and year 2007. The flow chart considered for sensitivity analysis is shown in Figure 7.1 as under;

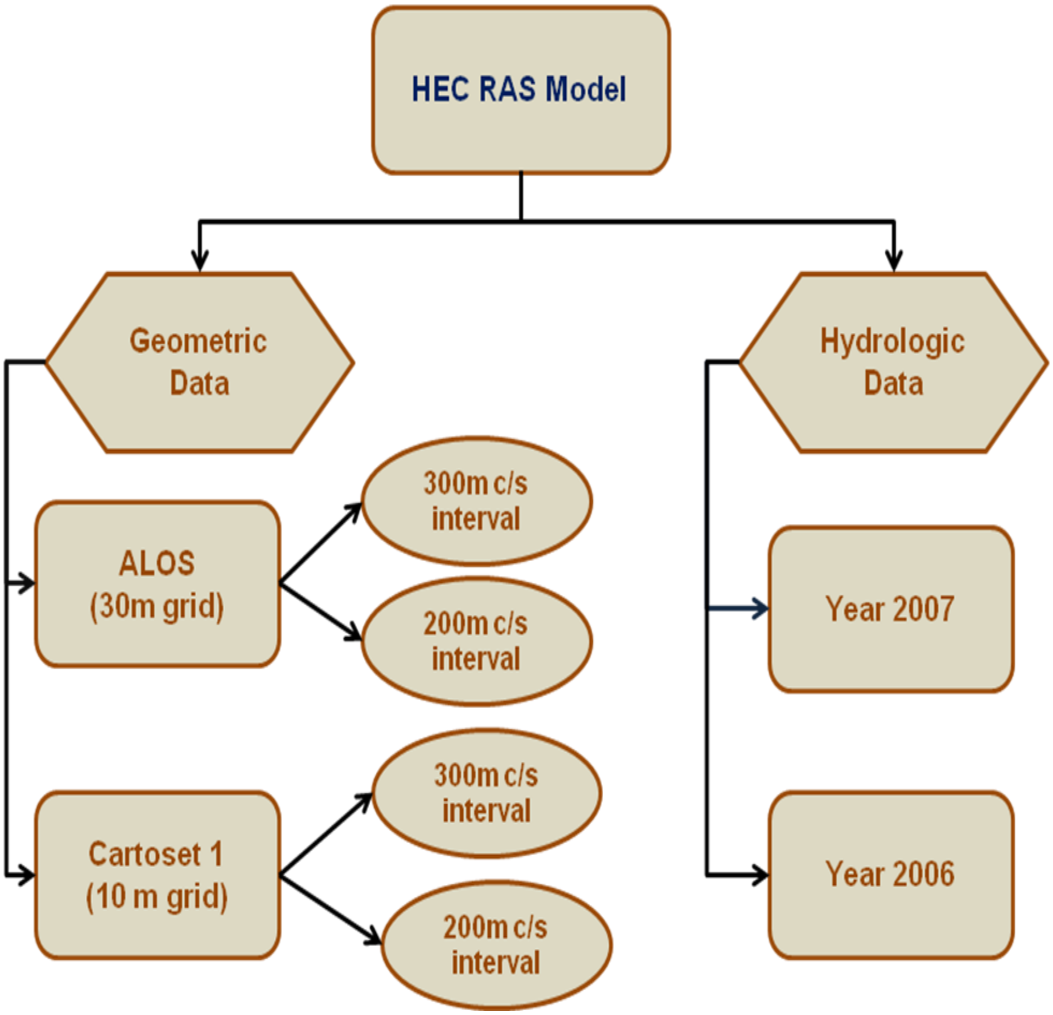


Figure 7.1 Flow chart of sensitivity analysis

The sensitivity of simulated water surface elevation for two DEMs of 30 m grid ALOS and 10 grids Cartoset 1 with cross section spacing of 300 m and 200 m has been evaluated through RMSE, mean absolute difference and mean difference as shown in Table 7.1, Table 7.2, Table 7.3 and Table 7.4. The lesser value of RMSE, mean absolute difference and mean difference indicates good conformity of simulated values with the observed value of river stages.

TABLE 7.1 Sensitivity of stages of river for manning's roughness for 30 m grid ALOS DEM and 300 m cross section spacing

Roughness co-efficient	RMSE		Mean Absolute Difference		Mean Difference	
	2006	2007	2006	2007	2006	2007
Manning's n= 0.020	0.83	1.08	0.62	0.87	-0.03	-0.08
Manning's n= 0.025	0.95	1.22	0.8	1.11	-0.62	-0.47
Manning's n= 0.030	1.26	1.44	1.6	1.33	-1.09	-0.8
Manning's n= 0.035	1.62	1.68	2.63	1.52	-1.51	-1.09
Manning's n= 0.040	1.97	1.91	1.89	1.69	-1.89	-1.36

TABLE 7.2 Sensitivity of stages of river for manning's roughness for 30 m grid ALOS DEM and 200 m cross section spacing

Roughness co-efficient	RMSE		Mean Absolute Difference		Mean Difference	
	2006	2007	2006	2007	2006	2007
Manning's n= 0.020	4.47	3.6	3.93	3.15	1.41	1.52
Manning's n= 0.025	5.17	3.93	4.85	3.62	-0.75	1.98
Manning's n= 0.030	4.61	4.44	4.39	4.13	-2.47	-0.24
Manning's n= 0.035	5.32	4.24	5.14	3.91	-0.96	1.3
Manning's n= 0.040	4.93	-	4.81	-	-4.51	-

It has been observed from table 7.1 and 7.2, that 1D HEC-RAS model is highly sensitive towards not only resolution of DEM but also on spacing of cross section considered for generating river geometry. As seen from table 7.1 and 7.2, for 30 m grid interval ALOS DEM gives better results at 300 m cross section spacing and further decrease in spacing of cross section as 200 m, gives much higher values of RMSE, mean absolute difference and mean difference for simulated stages of river and it even more increases with increase in roughness of manning’s coefficient value up to 0.04 for both the year 2006 and year 2007. Also, at cross section spacing of 200 m interval, model becomes unstable for manning’s roughness equal to 0.04 for data set of year 2007.

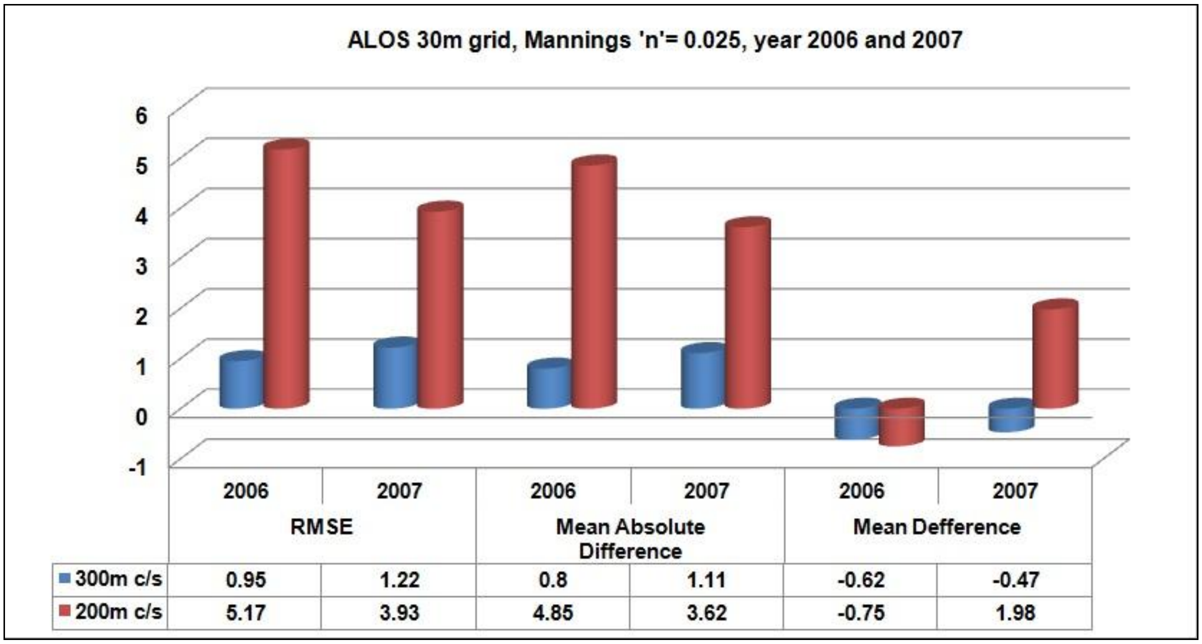


Figure 7.2 Sensitivity of river stages for 300 m and 200 m cross section spacing using ALOS 30 m DEM and Manning’s ‘n value of 0.025

As shown in above figure 7.2, the sensitivity of cross section spacing of 300 m and 200 m has been compared using calibrated Manning’s ‘n’ value of 0.025 using ALOS DEM for year 2006 and 2007. For year 2006, cross section spacing of 300 m interval gives RMSE equal to 0.95 and 1.22 for year 2006 and 2007 respectively while cross section spacing of 200 m interval gives RMSE of 5.17 and 3.93 respectively which are much higher for both the years. Similarly, the values of mean absolute difference for year 2006 and 2007 gives value of 0.80 m and 1.11 m respectively for 300 m cross section spacing and 4.85 m and 3.62 m respectively for 200 m cross section spacing respectively. The comparison shows that all the three statistical evaluation parameters of RMSE, mean absolute difference and mean difference shows better results for both the years of 2006 and 2007 for cross section spacing of 300 m interval for ALOS DEM in case of Manning’s ‘n’ value of 0.025.

TABLE 7.3 Sensitivity of stages of river for manning's roughness for 10 m grid Cartoset 1 DEM and 300 m cross section spacing

Roughness co-efficient	RMSE		Mean Absolute Difference		Mean Difference	
	2006	2007	2006	2007	2006	2007
Manning's n= 0.020	4.93	4.35	4.79	3.86	-4.79	3.47
Manning's n= 0.025	5.07	3.89	4.93	3.24	-4.93	2.3
Manning's n= 0.030	5.18	5.12	5.03	4.8	-5.03	-1.87
Manning's n= 0.035	5.22	4.59	5.06	4.26	-5.06	-2.23
Manning's n= 0.040	5.26	3.72	5.09	3.2	-5.09	-1.29

TABLE 7.4 Sensitivity of stages of river for manning's roughness for 10 m grid Cartoset 1 DEM and 200 m cross section spacing

Roughness co-efficient	RMSE		Mean Absolute Difference		Mean Difference	
	2006	2007	2006	2007	2006	2007
Manning's n= 0.020	1.06	1.99	0.83	1.59	-0.21	-1.41
Manning's n= 0.025	1.19	2.31	0.97	2	-0.7	1
Manning's n= 0.030	-	1.5	-	1.32	-	-0.72
Manning's n= 0.035	1.65	1.66	1.49	1.41	-1.44	-0.97
Manning's n= 0.040	1.78	1.82	1.63	1.49	-1.61	-1.2

Also, as seen from Table 7.3 and 7.4, where 10 m grid Cartoset 1 DEM has been used to check the sensitivity of model regarding cross section spacing of 300 m and 200 m by statistical parameters, which shows that for Cartoset 1 DEM the decrease in cross section spacing from 300 m to 200 m gives better results for relative values of manning's roughness coefficient for all the three statistical parameters considered for analysis.

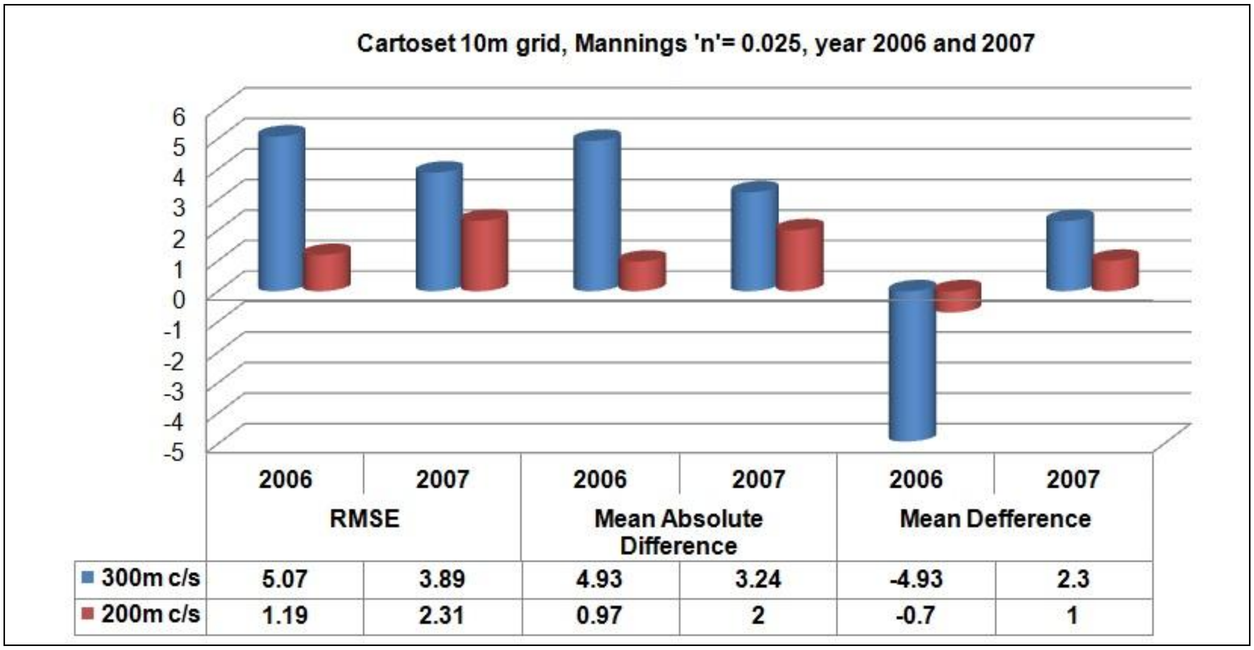


Figure 7.3 Sensitivity of river stages for 300 m and 200 m cross section spacing using Cartoset 10 m DEM and Manning's 'n value of 0.025

As shown in above Figure 7.3, the sensitivity of cross section spacing of 300 m and 200 m has been evaluated using calibrated Manning's 'n' value of 0.025 using Cartoset DEM for year 2006 and 2007. For year 2006, cross section spacing of 300 m interval gives RMSE equal to 5.07 and 3.89 for year 2006 and 2007 respectively while cross section spacing of 200 m interval gives RMSE of 1.19 and 2.31 respectively which are much lesser for both the years. Similarly, the values of mean absolute difference for year 2006 and 2007 gives value of 4.93 m and 3.24 m respectively for 300 m cross section spacing and 0.97 m and 2.0 m respectively for 200 m cross section spacing respectively. The comparison shows that all the three statistical evaluation parameters of RMSE, mean absolute difference and mean difference shows better results for both the years of 2006 and 2007 for cross section spacing of 200 m interval for Cartoset DEM in case of Manning's 'n' value of 0.025.

By studying above results it has been found out that for 30 m ALOS the minimum allowable cross section spacing is 300 m while for high resolution Cartoset DEM it can be reduced up to 200 m. It indicates that higher resolution DEM permits the closer spacing of cross section of river which gives more accurate and better geometric representation of channel.

In this study, further attempt has been made to check cross section spacing of 100 m for both the DEMs of 30 m and 10 m grid interval but requirement of HEC GeoRAS to accept single value of width of cross section for entire river length for automatic generation of cross section, it was not achieved. Because the study reaches of Sabarmati have variation of width from approximately 350 m to 1000 m and due to intrusion of closely spaced cross section with each other was not giving acceptable range of geometric data.

The above analysis shows that simulated model highly depends not only on manning's roughness but resolution of DEM and spacing of cross section considered for generating river geometry. The study shows that the minimum value of cross section spacing equal to 300 m for 30 m grid ALOS and 200 m for 10 m grid Cartoset 1 gives better results for model parameters. Also, it has been observed that more accurate results can be achieved by decreasing cross section interval in higher resolution DEMs. The appropriate combination of grid interval and corresponding cross section interval accompanied with suitable roughness coefficient can develop highly efficient model to predict stages and other hydraulic parameters in river.

CHAPTER 8

CONCLUSION AND RECOMMENDATION

8.1 Conclusion

The present study attempts to know efficacy and applicability of DEM generated cross sections along with HEC- RAS model for estimating water surface elevation in a river and water inundation in nearby flood plain through development of one dimensional and two dimensional models respectively. The developed 1D and 2D models proves to be relevant means for decision makers to explore in advance the flood depth, velocity, water surface elevation, arrival time at specific location and flood inundation boundary for future flood event corresponding discharge at upstream. By utilizing the simulated results, the concern authorities will take the suitable action in precise time to reduce the damage of property and lives. The salient research findings are summarized herewith from 1D and 2D HEC-RAS hydrodynamic modeling:

- The 1D HEC-RAS model can efficiently predict the river stages across the study reach of the Sabarmati River. Results generated by comparing various stages in river with corresponding bank levels indicates probable areas to be inundated for corresponding discharge from upstream. Even with limitation of 1D HEC-RAS model and lack of sufficient number of gauging stations for validation of results, the model has potential to effectively calculate river stages for relative discharge from upstream and further utilized to update regarding flood situation in surrounding. The present approach will reduce the modeling complexity of 1D modeling and fill the gap for required modeling data sets for preparation of an Emergency Action Plan for Dharoi Dam.

- The results simulated from 1D steady hydrodynamic model resourcefully suggest probable percentage of area inundated on left and right banks of river, which can be further utilized by planners and decision makers in designing flood protection approach and in preparation of flood inundation maps as a part of developing flood forecasting system.

- As a part of development of 1 D unsteady model, the manning's value has been calibrated through trial and error as comes out as 0.025 for study reach. Also, the effect of change in output of model corresponding to DEM resolution and cross section spacing has been checked which indicates high variation simulation results corresponding to terrain resolution corresponding cross section spacing considered to generate river geometry.

- During the study, it has been identified that most of the present research used mostly surveyed data for development of hydrodynamic model. To generate accurate geometry of river the physical survey proves to be costly and labors process and it has been professed that data availability and reliability is major constrain for specific study reach. So, replacing the processing of river geometry entirely by using remotely sensed data and high resolution DEM accelerates the modeling approach in such a way that it will be a very useful tool for hydraulic engineers to apply development of hydrodynamic modeling even for data scare regions. The developed methodology becomes a beacon light for researchers and decision-makers to envisage a decision-making system in data scare areas.

- The simulated results of 1D unsteady model identified that for flood event of year 2006, approximately 65.48% area on left bank and 48.22 % area on right bank has possibilities of water spill and inundation for discharge equal or more than 6472 cumecs. Also, the results of 1D steady simulation suggest that for return period of 20, 25, 30, 50, 60, 75 and 100 years the left bank consisting of old Ahmedabad city is more prone to over bank water spilling than right bank consist of new Ahmedabad city.

- From studying, the simulated results of 1D and 2D hydrodynamic model, it has been observed for discharge equal to or more than 6472.12 cumecs, the right bank area of Indroda village and left bank area of GIFT city of Gandhinagar are at high risk of water inundation. And for Ahmedabad city, Koteshwar, Sardar Patel Stadium-Motera, Ashram road, Vadaj, Usmanpura on right bank and Asarva, Shahpur, Khanpur and Raipur on left bank are at high risk of flood inundation for aforesaid discharge. The output generated from 2D simulation can be further utilized in preparing inundation maps for any value of future discharge and further development of flood forecasting system of Ahmedabad city.

The developed methodology presents strong helpful substantiation of the potentiality of combined use of HEC-RAS and HEC-GeoRAS for flood inundation modeling. The assessment of the HEC-RAS is an significant pace for successful and improved development of the hydrodynamic model and thus can provide significant aide in developing flood mitigation strategies for any similar case across the world. The future scope of the study includes dam break analysis using 1D and 2D model and based on that the emergency action plan for city can be developed using same modeling platform.

8.2 Recommendation

- Currently, the HEC-RAS cannot be effectively used for modeling bridges in 2D flow area due to unavailable tools. Considering this fact, the 2D model is simulated without considering any hydraulic structures across the study reach of Sabarmati River.
- The DEM used to generate geometry of River for 1D and 2D model has collected the information of year 2005. The Sabarmati Riverfront development project was carried out since year 2007 and has been completed in year 2011 for the length of approximately 11 km upstream of Vasana Barrage so for simulation of any flood event after year the latest DEM should be considered for more realistic results.
- Though covering two very important cities of Ahmedabad and Gandhinagar, there is no sufficient and reliable data were available for validation of geometric data as well as gauge data of Sabarmati River as there is only one gauging station allotted at Subhash Bridge. The limitation of only one gauging station in study reach is one of the major constraints for validating simulated results. Also, if more gauging station would have been established and made available for particular patch of Sabarmati River, the efficiency of developed model can be further improved.
- The simulation results are highly sensitive towards DEM resolutions, so more accurate results can be achieved using 5m or lesser grid interval DEM for generation of geometric

data of River. Also, unavailability of fund to carry physical survey to produce river cross sections at desired interval of 300 m interval and less, the entire geometry has to be developed using DEM data which can be improved by combining satellite data with modern tools like drone survey with financial support.

- The technical details of bridges across the study reach was not sufficiently available which can be further included for development of model. Also, it has been strongly suggested that in future, the development of hydrologic model must include the recently completed storage structure, Sant Sarovar at approximately 6 km downstream of Chiloda Bridge for more reliable results.

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List of Publications

1. Paper published on the, Review of application of open source HEC-RAS for 1 dimensional hydrodynamic modeling - Global and Indian scenario, *Journal of Emerging Technologies and Innovative Research (JETIR),* Volume 6, Issue 4, April 2019, ISSN-2349-5162 (UGC approved).
2. Paper published on the, Application of open source Google image for river bathymetry delineation for 1d hydrodynamic modeling, *International Journal of Research and Analytical Reviews (IJRAR),* Vol.6, Issue 1, March 2019, E-ISSN: 2348-1269 (UGC approved).
3. Presented Poster on, 1D HEC-RAS hydrodynamic modeling of river flow simulation using DEM extracted river cross-sections- A case of Sabarmati River, Gujarat, India, European Geo-science Union (EGU-2018), April-2018, Vienna, Austria.
4. Presented paper on, River cross section delineation from the Google earth for development of 1d HEC-RAS model – a case of Sabarmati River, Gujarat, India, HYDRO-2017 International, LDCE, Ahmedabad, India, December -2017.(International conference)

Annexure-I

Sr. No.	C/S ID	R.L. of Left Bank in m	Max. WSE in m	R.L. of Right Bank in m
1	Chiloda Bridge	67.39	62.1	70.34
2	41400	59.84	59.66	70.73
3	41200	60.36	59.6	62.95
4	41000	60.36	59.39	69.27
5	40800	***56.73***	59.26	61.84
6	40600	59.72	59.03	62.94
7	40400	61.25	58.57	63.12
8	40200	59.85	58.58	60.53
9	40000	61.74	58.49	59.92
10	39800	62.43	58.44	60.34
11	39600	58.55	58.3	61.05
12	39400	***51.66***	58.17	61.54
13	39200	***52.69***	58.06	59.76
14	39000	***52.54***	58.05	61.09
15	38800	***51.69***	58.05	61.4
16	38550	***52.16***	57.99	60.86
17	38400	***56.86***	57.82	60.84
18	38200	***56.35***	57.69	62.97
19	38000	***55.67***	57.66	65.44
20	37800	58.65	57.54	65.49
21	37600	***52.35***	57.49	66.3
22	37400	***52.06***	57.43	62.09
23	37200	***55.43***	57.33	62.18
24	37000	***52.73***	57.29	62.06
25	36800	***55.91***	57.21	58.34
26	36600	58.53	57.05	61.39
27	36400	***56.85***	56.96	61.9
28	36200	58.22	56.86	60.52
29	36000	56.88	56.64	62.75
30	35800	***54.67***	56.49	62.75
31	35600	55.16	56.53	***55.71***
32	35400	58.13	56.41	63.12
33	35200	56.48	56.3	62.25
34	35000	***51.16***	56.02	57.06
35	34800	58.21	56.04	64.09
36	34600	***55.43***	55.92	64.69

Sr. No.	C/S ID	R.L. of Left Bank in m	Max. WSE in m	R.L. of Right Bank in m
37	34400	***54.55***	55.72	71.19
38	34200	***48.52***	55.64	60.84
39	34000	55.6	55.45	60.01
40	33800	57.85	55.17	67.76
41	33600	55.12	55.04	62.88
42	33400	61.38	54.94	***54.7***
43	33200	60.59	54.45	63.65
44	33000	58.2	54.14	56.23
45	32800	56.37	54.08	59.17
46	32600	55.27	54.13	58.98
47	32400	55.34	54.09	59.93
48	32200	***52.43***	54.01	60
49	32000	***50.2***	54.01	59.73
50	31822	55.16	53.93	64.97
51	31578	***50.34***	53.91	54.12
52	31400	***48.83***	53.89	56.45
53	31200	54.36	53.86	57.99
54	31000	55.63	53.87	55.67
55	30800	55.38	53.82	***53.34***
56	30600	54.33	53.81	***52.89***
57	30400	55.99	53.77	53.96
58	30200	57.85	53.73	59.59
59	30000	57.82	53.72	57.52
60	29800	54.3	53.67	58.82
61	29600	***53.25***	53.66	54.17
62	29400	55.22	53.65	***53.21***
63	29200	54.94	53.65	***51.49***
64	28800	***52.2***	53.64	***53.21***
65	28600	***51.71***	53.63	***51.14***
66	28400	56.64	53.62	***50.79***
67	28200	64.42	53.61	***50.51***
68	28000	57.18	53.59	***52.52***
69	27800	***48.4***	53.59	56.86
70	27400	***52.48***	53.57	***51.85***
71	27000	57.94	53.53	***49.59***
72	26600	***52.61***	53.49	***50.27***
73	26400	***52.41***	53.48	***52.73***
74	26200	54.05	53.46	***53.42***

Sr. No.	C/S ID	R.L. of Left Bank in m	Max. WSE in m	R.L. of Right Bank in m
75	26000	54.61	53.42	54.79
76	25800	54.1	53.38	***50.05***
77	25600	55.84	53.33	***52.63***
78	25400	***48.51***	53.36	***49.88***
79	25200	***49.97***	53.36	***49.67***
80	25000	***51.23***	53.34	***48.11***
81	24600	***47.22***	53.31	56.05
82	24400	***50.1***	53.29	54.76
83	24000	***47.85***	53.2	56.52
84	23774	***48.35***	53.17	***48.27***
85	23600	***46.86***	53.07	***51.02***
86	23400	***50.74***	52.97	55.79
87	23200	***52.44***	52.95	***49.49***
88	23000	***50.43***	52.78	54.23
89	22800	***48.85***	52.69	57.76
90	22600	***48.8***	52.71	53.16
91	22400	***49.32***	52.62	58.41
92	22200	53.66	52.54	53.47
93	22006	***50.93***	52.59	58.77
94	21800	***43.23***	52.65	54.73
95	21600	***44.72***	52.59	56
96	21400	***47.31***	52.55	53.77
97	21200	***47.4***	52.56	53.73
98	21000	***47.25***	52.55	***51.95***
99	20800	***46.98***	52.53	***51.19***
100	20600	***52.11***	52.51	53.61
101	20400	54.65	52.47	56.27
102	20200	***49.68***	52.44	***50.83***
103	20000	***52.22***	52.37	54.72
104	19800	***47.7***	52.36	54.28
105	19600	***49.26***	52.34	***50.17***
106	19364	56.92	52.24	52.27
107	19200	***49.01***	52.07	53.08
108	19000	***48.98***	52.17	***50.73***
109	18800	***52.09***	52.16	***50.45***
110	18400	***45.5***	52.11	53.08
111	18200	***42.88***	52.07	52.2
112	17967	***50.23***	52	***49.98***

Sr. No.	C/S ID	R.L. of Left Bank in m	Max. WSE in m	R.L. of Right Bank in m
113	17800	***49.99***	51.96	***46.23***
114	17600	***49.97***	51.85	***50.41***
115	17400	***43.96***	51.84	***50.25***
116	17200	***44.03***	51.7	***50.93***
117	17000	***43.59***	51.78	***51.06***
118	16800	***44.26***	51.76	***51.65***
119	16600	***44.03***	51.77	***49.58***
120	16400	***47.9***	51.75	***49.91***
121	16200	***50.03***	51.76	***45.24***
122	16000	***48.92***	51.74	***45.97***
123	15800	***46.72***	51.73	***44.88***
124	15600	***47.21***	51.73	***45.92***
125	15400	***47.43***	51.71	***47.05***
126	15200	***49.57***	51.69	***51.55***
127	15000	***51.09***	51.69	***49.92***
128	14800	***46.6***	51.68	55.86
129	14410	***44.86***	51.69	***45.24***
130	14200	***44.45***	51.67	56.11
131	14000	***47.92***	51.64	***47.6***
132	13800	***44.03***	51.65	***50.92***
133	13552	***46.57***	51.62	***48.41***
134	13188	***44.8***	51.16	55.18
135	13000	***46.98***	51.32	***50.91***
136	12800	***46.77***	51.28	***48.39***
137	12600	***43.87***	51.33	53.03
138	12400	***46.47***	51.27	***48.52***
139	12200	***50.16***	51.29	***42.32***
140	11800	***49.48***	51.28	***48.24***
141	11600	55.39	51.27	***49.15***
142	11400	***45.99***	51.26	***50.49***
143	11200	***44.89***	51.25	***50.33***
144	11000	***44.42***	51.24	***48.7***
145	10800	53.07	51.2	***48.41***
146	10600	***48.2***	51.19	***49.17***
147	10400	***46.55***	51.12	***49.25***
148	10200	53.48	51.11	***45.97***
149	10000	51.33	51.11	***43.87***
150	9800	***49.95***	51.05	***44.02***

Sr. No.	C/S ID	R.L. of Left Bank in m	Max. WSE in m	R.L. of Right Bank in m
151	9600	***46.21***	51.05	***48.83***
152	9400	***49.35***	51.04	***50.71***
153	9200	***47.29***	51.06	***47.95***
154	9000	***45.49***	51.11	***46.69***
155	8800	***48.09***	51.08	***46.01***
156	8600	***46.53***	51.03	***45.79***
157	8400	***50.15***	51.08	***43.5***
158	8200	***49.49***	51.09	***47.82***
159	8000	***45.99***	51.11	***44.42***
160	7800	***47.02***	51.12	***45.07***
161	7600	***47.25***	51.12	***50.3***
162	7400	***48.62***	51.13	***47.81***
163	7200	***50.36***	51.14	***48.23***
164	7000	***48.95***	51.14	58.68
165	6800	***49.55***	50.92	53.14
166	6600	***46.24***	51.06	51.13
167	6400	***47.16***	51.06	58.56
168	6200	***48.68***	51.08	58.1
169	6000	51.51	51.03	***47.31***
170	5800	***48.66***	51.05	***49.96***
171	5600	53.69	51.02	57.75
172	5400	55.09	51.04	***50.6***
173	5200	***48.58***	50.99	51.79
174	5000	***46.47***	51.02	***50.13***
175	4800	51.24	50.98	51.93
176	4600	***49.33***	51.01	***47.77***
177	4400	***44.01***	51.03	***50.92***
178	4200	***44.85***	51.02	***49.06***
179	4000	***45.44***	51.04	***49.56***
180	3800	***45.07***	51.03	***49.02***
181	3600	***46.04***	51.03	52.09
182	3400	***45.64***	51.02	***45.46***
183	3000	***42.42***	51.04	52.54
184	2800	51.79	51.04	57.77
185	2600	***48.76***	51.05	***46.35***
186	2400	52.82	51.05	***46.58***
187	2200	51.8	51.06	***44.67***
188	2000	***50.6***	51.07	***47.56***

Sr. No.	C/S ID	R.L. of Left Bank in m	Max. WSE in m	R.L. of Right Bank in m
189	1800	50.37	51.07	***47.22***
190	1600	53.18	51.07	***48.74***
191	1400	43.81	51.08	***50.38***
192	1162	44.45	52	46.51
193	1000	41.61	51	44.48
194	800	43.48	51	50.1
195	600	43.33	50	47.01
196	400	44.03	50	39.52
197	Vasana Barrage	42.93	42	41.66

Annexure-XX

Gujarat Technological University

PhD *Viva Voce* Report

TITLE OF THE THESIS: " To develop flood forecasting approach of Ahmedabad City, Gujarat, India"

Name of the Scholar	Enrollment No.	Day & Date of Public Viva Voce	Discipline/ Branch	Venue
Ujas Deven Pandya	149997106023	14th August 2020, 1:30 p.m. onwards	Civil Engineering	GTU

Based on the thesis defense of above mentioned PhD Thesis, the overall recommendation on the thesis is as follows (Please tick any one of the following option):

- [x] The performance of the candidate was satisfactory. We recommend that he/**she**√ **be awarded the PhD Degree.**
- [ ] Any further modifications in research work recommend by the panel after 3 months from the date of first *viva-voce* upon request of the Supervisor or request of Independent Research Scholar after which *viva-voce* can be re-conducted by the same panel again. The suggestions for improving the thesis based on the discussions during the oral examination is detailed in a separate sheet to be incorporated in the thesis.
- [ ] The performance of the candidate was unsatisfactory. We recommend that he/she should not be awarded the PhD Degree. A separate sheet is enclosed describing unsatisfactory performance.

Further, it is certified that the examiner who participated in the thesis defense through electronic medium (if any), have confirmed the above recommendation after the *viva-voce* (through email as attached; if any) and the same may be considered sufficient record for acceptance.

BOARD OF EXAMINERS:

Sl No	Name	Designation	Institute	Signature
1.	Dr. Dhruvesh Patel	**Supervisor/ Co-Supervisor***	PDPU, Gandhinagar	D. P. Patel
2.	Dr. Sudhir Kumar Singh	**External Examiner 1**	University of Allahabad, UP	Sudhir Kumar Singh
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*The Co-Supervisor may sign in place of Supervisor if he/she has been assigned with the academic and administrative affairs/ responsibilities of the above mentioned scholar.

Encl.:

1) Separate sheet for suggestions / comments on the thesis (if any) endorsed by the Supervisor/ Co-Supervisor and the external examiners. The same to be provided to the scholar for revision/ modification in the thesis.
2) Email of external examiner (if any) who participated in the thesis defense through electronic medium.
3) Undertaking for final submission of hard copy of Ph.D. thesis & CD.

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Honorable Vice Chancellor