Urban Flood Modelling using 1D Channel Network. Case Study of Nimes, France

By

Muhammad Adnan

(NUST201362221MSCEE15313F)

A Thesis submitted in partial fulfillment

of the requirements for the degree of

Master of Science

in

Water Resources Engineering and Management

DEPARTMENT OF WATER RESOURCES ENGINEERING AND MANAGEMENT

NUST INSTITUTE OF CIVIL ENGINEERING

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

NATIONAL UNIVERSITY OF SCIENCES AND TECHNOLOGY

SECTOR H-12, ISLAMABAD, PAKISTAN

(2017)

This is to certify that the

Thesis entitled

Urban Flood Modelling using 1D Channel Network. Case Study of Nimes, France

Submitted by

Muhammad Adnan

(NUST201362221MSCEE15313F)

Has been accepted in partial fulfillment of the requirements

Towards the award of the degree of

Master of Science in Water Resources Engineering and Management

____

Dr. Sajjad Haider

Associate Professor

NUST Institute of Civil Engineering (NICE)

School of Civil & Environmental Engineering (SCEE)

National University of Sciences & Technology, Islamabad

DEDICATED TO

My Beloved Parents and Family

ACKNOWLEDGEMENTS

I am very excited to express my deepest gratitude to all those who contributed in making this study possible, valuable and successful. First of all, I like to thank Almighty God for the wisdom, knowledge and perseverance He has been bestowed on me during whole of my life.

Then, I would like to extend my appreciation for my supervisor Dr. Sajjad Haider (NICE, NUST) for spending his enormous valuable time in teaching, guiding, being patient and providing moral and technical support to perform this research.

I am very grateful to the rest of my thesis guidance committee members, Dr. Hamza Farooq Gabriel (NICE, NUST), Dr. Shakil Ahmad (NICE, NUST), Dr. Kamran Syed (NICE, NUST) and Asst. Prof. Ammara Mubeen (NICE, NUST) for their willingness to serve as members of the thesis committee on a very short notice. Their participation in editing and their valuable suggestions for improving my thesis was quite valuable.

Beside my thesis guidance committee, I would like to thank Kashif Jamal and Muhammad Adnan Khan for providing me their technical support to carry out this research.

Last but not least, I like to thank my brothers especially, Kashif Ali, for providing me their support, love and lot of prayers throughout my studies. I am very grateful to all of you for your contribution and support.

(Muhammad Adnan)

ABSTRACT

Floods are one of the leading natural hazards which cause immense damage to properties as well as to human lives. Due to increasing climate impacts, the occurrence of flood events is also increasing. Amongst the different types of floods, flash floods are of severe type due to their intensity and short warning time. In urban areas, flash floods can be even more destructive, as their economic cost is more.

The measures to combat floods can be categorized into *structural* and *non-structural*. Numerical modelling figures among the non-structural measures for the flood management.

A wide variety of different flood simulation models are available for analyzing the flood propagation. For engineers, the interest is normally, in knowing the spatial and temporal history of flood depths and velocities and their peaks. To that end, Saint Venant equation based models are a potent tool for the engineers.

In urban areas, numerical flood modeling is rendered difficult due to heterogeneous milieu, consisting of number of large and small obstacle e.g. roads, streets, buildings, parks, parking areas. Nowadays, the bi-dimensional Saint Venant equation models are preferred due to their superior ability to predict the flow direction in channel-floodplain case i.e. strong lateral variation in topography. They, however, need large amount of data and observations to successfully run and validate the model. In addition, computational mesh building is laborious and in the absence of suitable pre-processing tools, is difficult to manage. In addition, many city centers have the configuration of a network of streets meeting at crossroads which lend itself better to a 1d flow simulation using 1d Saint Venant equation model supplemented by suitable junction models.

In this study, flood modelling in the French city of Nimes, is carried out using BASEMENT model which is a freeware using interconnected 1d channel network supplemented by available junction models. The aim is to evaluate the capacity of a freeware model in predicting an urban flash flood through the 1d approach using the in-built functionalities.

The model results were compared with the observed flood marks and also with the result of the 2d model. The model results were comparable to the 2d model results. The model needs to be expanded to include more junction flow models to predict the complex flow pattern at the

crossroads as well as to efficiently incorporate the complex crossroad configurations with 4 or more streets. We also require more detailed urban topographical datasets to test and evaluate the coupled flood modeling approach whereby large open spaces can be modeled in 2d and streets in 1d.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

1-D	One Dimensional
2-D	Two Dimensional
3-D	Three Dimensional
BASEMENT	Basic Simulation Environment
BC1	Base Case 1
BC2	Base Case 2
BC3	Base Case 3
CFL	Courant Friedrichs Lewy
DEM	Digital Elevation Model
FDM	Finite Difference Method
FEM	Finite Element Method
FVM	Finite Volume Method
GIS	Geographic Information System
GUI	Graphical User Interface
HEC-RAS	Hydrologic Engineering Centre- River Analysis System
NICE	NUST Institute of Civil Engineering
RMSE	Root Mean Square Error
SCEE	School of Civil and Environmental Engineering
SVE	Saint Venant Equation
SWE	Shallow Water Equation

Chapter 1

INTRODUCTION

1.1 GENERAL

Worldwide floods are the most devastating natural hazard impacting adversely the society and accounting for more lives lost than any other natural hazard. With the increasing impact of climate change the frequency of the floods occurrence is also increasing. There is very much a need of establishing flood risk plans and strategies (Ahmad et al., 2010).

Flooding in an urban range is a serious issue for urban planners. Many urban areas in Asia are old and they have developed according to their needs. This implies that layout of cities have slowly formed into complex framework (Boonya-aroonnet et al., 2002).

There are few factors which are considered important when a surface flood modelling exercise is carried out in urban environment. The factors are; cars and buildings which offer resistance to the flood flows , storage in buildings and green spaces which intercept the flows and complex geometry of the city makes simulation difficult(Mignot et al., 2006).

Highly dense areas are mostly covered by impervious material which results in increase in surface runoff and decrease in infiltration of water .Through adequate mitigation techniques flooding can be avoided at some scale (Mahmood Siddiqui et al., 2011).

For improvement in flood mitigation processes and risk assessment flood modelling is done by using suitable models which use different schemes. Many models have been introduced in recent years ranging from 1-D to 3-D. Recently coupled models have also been introduced like 1-D/1-D, 1-D/2-D. Which are easy and uses less computation time as compared to fully 2-D models. Depending on the availability of the data, time and purpose of research any suitable model can be used.

Factors which are considered important in flash flooding in urban areas are fast urbanization, imperviousness, steep bottom slope, overloaded drainage system and lack of green spaces. Also developments like building, parking, road etc. lead to increasing imperviousness which results in decrease in infiltration and higher generation of flows. These factors combine to cause flooding.

1.2 PROBLEM STATEMENT

Intense rainfall in short interval of time can produce flash floods in urban areas. France is one of the countries which is prone to such kind of flooding. Amongst the types of flooding flash floods are of a severe type and can cause substantial economic and social damages.

The area of our study is the city of Nimes in southern France which was subjected to flash flooding on 3rd October 1988. Nimes city has a history of flash flooding since 1350. The affected urban area Richelieu is located in the north eastern part of Nimes city. Along the north-south axis which is also the main flow direction the dimension of area is 1400m and along east-west axis is minimum of 220m and maximum 1050m, respectively. A railway embankment runs along the northern side. No river crosses the city Nimes and when it rains on the region the flow drains to Vistre River through temporary watercourses. Whenever the discharges become greater than conveyance limit of the watercourses, overflow occurs and water starts flowing north to south direction. Richelieu consists of large building such as hospitals and army barrack with streets and crossroads(Mignot et al., 2006).

The city of Nimes has a flooding history since 1350. Significant events of floods are 1988, 1998, 2002, 2004, 2005 and 2010. But the most devastating observed event occurred on 3rd October, 1988 when 420mm of rain fell in 8 hours resulting in overflow of watercourses and inundation of localities up to a depth of 3m. This event caused loss to buildings, houses and human life. According to (Duclos et al., 1991) homes of 45,000 people and 11,000 vehicles were damaged. Death of 9 people and 3 severe injuries were reported.

Floods are natural hazards and cannot be stopped but by proper planning and adopting suitable strategies its effects can be mitigated. Early flood forecasting, warning system and flood hazard maps can help a lot in risk management of floods. Thus study was carried out to thoroughly understand the flood flooding phenomena in urban areas. This will also help in understanding the related field case.

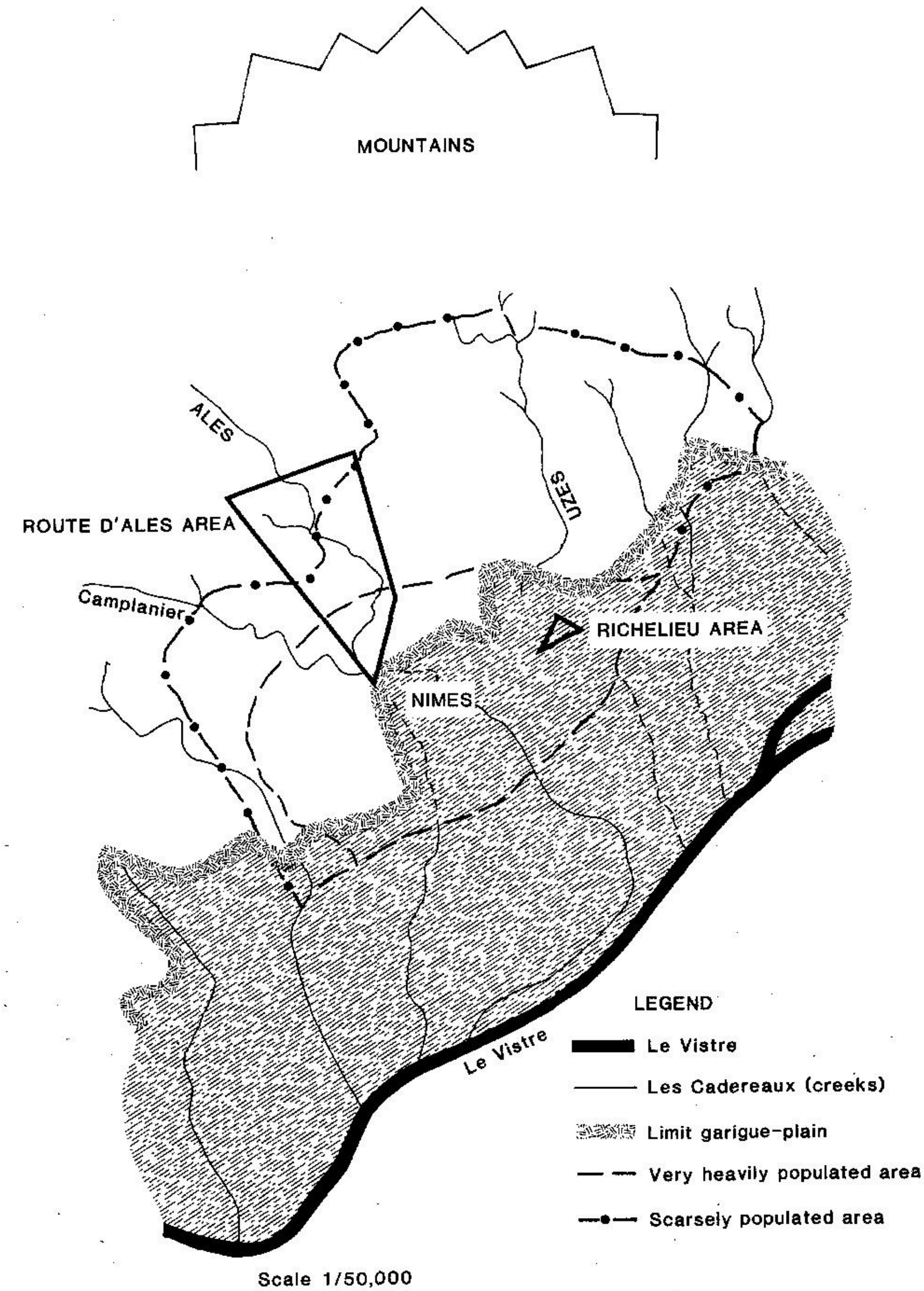


Figure 1.1: Nimes and its region (Duclos et al., 1991)

1.3 OBJECIVES OF THE STUDY

i. To apply the 1-D branching network solution technique to an urban downtown for modelling a flash flood within the framework of Freeware BASEMENT model.

ii. To compare the results of 1-D study with observed flood marks.

iii. To compare the results of 1-D study with a 2-D flood modelling results, for an assessment of the accuracy of the model.

1.4 SCOPE OF THE RESEARCH WORK

The present study is based on the event of flash flood in the city of Nimes France that occurred on 3rd October, 1988. Software BASEMENT has been used to model the event of 3rd October, 1988 and the results have been compared with 2-D model (Mignot et al., 2006). 1-D coupled network technique have been used in BASEMENT model and to know how useful this technique is as compared to 2-D modelling. Sequential, junction and bifurcation coupling is used at the boundary conditions. Also the aim of study is to develop a model which can also be applied in urban centres of Pakistan and to recommend better effective risk management techniques.

Chapter 2

LITERATURE REVIEW

2.1 GENERAL

Floods are said to be the most sensational association amongst man and his surroundings. Increasing population and economic activities near the river banks introduce danger to urban districts. Because of compelling floods occasions, flood modelling is gaining much interest all over the world. Past examination on all the model parameters, their perceptions, results and conclusions are assessed and certain derivations are made to seek after a very much corresponding study.

2.2 CAUSES AND ISSUES OF FLOODING

Natural disasters have always been a problem for the human being. Among those floods are of major concern which are very frequent, widespread natural disaster in the world. Therefore development of cost effective strategies are important to establish mostly for developing countries (Ahmad et al., 2010). Also due to change in climate frequency of floods and its magnitude is also increasing (Allan and Soden, 2008).

Surge developed by high intensity of precipitation or sudden release of water from reservoir or dam break is termed as flash flood (Calianno et al., 2013).

Most industrial cities in the world have problem of insufficient capacity of sewer system in heavy rainstorm situation. Due to insufficient capacity of sewers water overflows and runs on the surface of the streets. South/south east countries are having this issue (Mark et al., 2004).

Excessive overflow of water from the waterway like river or stream onto normal dry land is termed as riverine flood. This type of flooding is a relatively long duration phenomenon and may last for weeks (Clements, 2009).

2.3 FLOOD MODELLING APPROACHES

Natural disasters have always been a great problem for human beings. Among them floods are of great concern which are most frequent, widespread natural disaster in the world. Therefore development of cost effective strategies are important to establish mostly for developing countries (Kadam and Sen, 2012).

Using hydrologic and hydraulics models flood hazard maps are prepared. For flood inundation, flood plain delineation, many mathematical models are developed(Kadam and Sen, 2012). These models can be classified as 1-D, 2-D ,3-D and hybrid schemes(González-Sanchis et al., 2012; Kuiry et al., 2011).

1-D models are based on 1-D saint-venant equation (SVE) using finite difference method and finite element method schemes (Chaudhry, 2007; Cunge et al., 1980). In 1-D models finite difference method scheme is widely used because of its computational efficiency. Many models like DWOPER, FLDWAV, MIKE-11, ISIS, SOBEK (1D) etc. are used for dynamic 1-D simulation of rivers. 1-D modelling approaches have been used for the past 30 years. Many researchers favour 1-D modelling because of low computation effort, easy to calibrate/setup (Seyoum et al., 2012). As 1-D SVE is the governing equation for 1-D models, the assumptions made are; hydrostatic distribution of pressure, prismatic channel and uniform velocity across the channel (Chaudhry, 2007). Numerical methods used for solving 1-D SV equation are the finite difference method, finite volume method and finite element method (Chaudhry, 2007). The major drawback of 1-D models is inability of lateral flood wave diffusion(Seyoum et al., 2012).

Flood plains are extended flood areas. When the water overflows the banks of the river and move towards flood plains the flow doesn't remain 1-D. Therefore 2-D/3-D models are best to model the flood in flood plains. 3-D models are costly and need high resolution data so 2-D models are widely used(González-Sanchis et al., 2012; Hunter et al., 2005). MIKE 21, TELEMAC 2D are widely used 2-D models (Kadam and Sen, 2012). Shallow water equation (SWE) is governing equation for 2-d models (Chaudhry, 2007).

Recently an approach is becoming very popular in which river is modelled in 1-D and flood plain is modelled in 2-D. In Urban flood modelling same model is used as 1-D model in modelling streets flow and 2-D model in bifurcation/junction. This type of model combines accuracy and efficiency as compared to 1-D model and provides extra information on flow process (Hunter et al., 2005; Kuiry et al., 2011).

2.4 URBAN FLOOD MODELLING

Most parts of the world have the problem of insufficient capacity of the sewers. Especially, in South/South-East Asian countries, this problem is more severe because of high intensity local rainfall and poor drainage. Due to increase in population, developing countries are growing without any plan which makes this problem more severe. Due to insufficient capacity of sewers, overflow occurs and water starts flowing on the streets. In developing countries the frequency and extent of flooding has made them a good case for urban flood modelling as the data is available. A few example of urban flood problems are Mumbai (India), July 2000, Dhaka (Bangladesh), September 1996 and Jakarta (Indonesia), February 2002. Urban flooding creates infrastructural and economic problems as well as causing considerable damages to property (Mark et al., 2004).

According to (König et al., 2002) damages can be divided into three parts which consists of Direct damages; material damages caused by water or flowing water, Indirect damages, traffic disruption, labor and administrative cost and disease spreading and Social damages, decrease in property values and delay in developing projects.

With advances in Computer technology, many countries in the world, now use computer models to model minor or major floods. These models can help analyze the different interactions between rainfall and floods.

There are some factors which make modelling of an urban area difficult. The factors are trans critical flows, poor topography, wet/dry interfaces, complex geometry and presence of large building and obstacles (Mignot et al., 2006; Yoshioka et al., 2015).

The computer models can model the flow in 1-D, 2-D, 3-D and coupled 1-D/1-D, 1-D/2-D. According to (Ghostine et al., 2014) 1-D simulation provides a good platform for urban flood modelling but at the junction the flow is mainly 2-D/3-D (Neary et al., 1999; Weber et al., 2001). Fully 2-D models can be used in such situation which shows the good results but they need high resolution data and more computational time (Ghostine et al., 2014).

Coupled 1-D/2-D models are getting popular in which main flow is modeled in 1-D and flows at junction/bifurcation or floodplains are modeled in 2-D and has been successfully used (Gejadze and Monnier, 2007; Verwey, 2001). Ghostine, et al., 2014 used 1-D/2-D coupled model to model flow in channel in 1-D and at junction 2-D and compared the results with fully 2-D model and found the results satisfactory and also the computation time was very less as compared to 2-D model.

2.4.1 ONE DIMENSIONAL CHANNEL NETWORK

This method is capable of providing flow information at low computational cost. To set up the problem a lot of work for network layout is needed. Mostly the challenge is at the junctions where the flow is mostly 2-D. Another problem in 1-D channel network is when this has to be interact with larger area models. When the flooding is due to surrounding terrain a suitable coupling is needed between 1-D channel network model and flood plain. In this method 1-D saint venant equation is used. The computation procedure solve the equation for every reach/domain and boundary condition is applied at the end of every domain. At the end points of the domain boundary conditions and information of the flood is imposed according to the state of the flow which is conveyed to the next domain according to connecting nodes. The specific feature of this method is coupling of reaches (Alcrudo, 2004).

Usually conservation of mass and equality of total head is applied according to given formulae below

$$\sum_{j=1}^{N_k} Q_j S_j \quad = 0 \qquad\qquad k = 1 \ldots\ldots N$$

(Eq. 2.1)

Where

Q_j is flow of the reach, S_j is the sign of flow (entering or leaving the junction) and N_k is number of reaches joining the node K and NN is total number of nodes connecting the reaches.

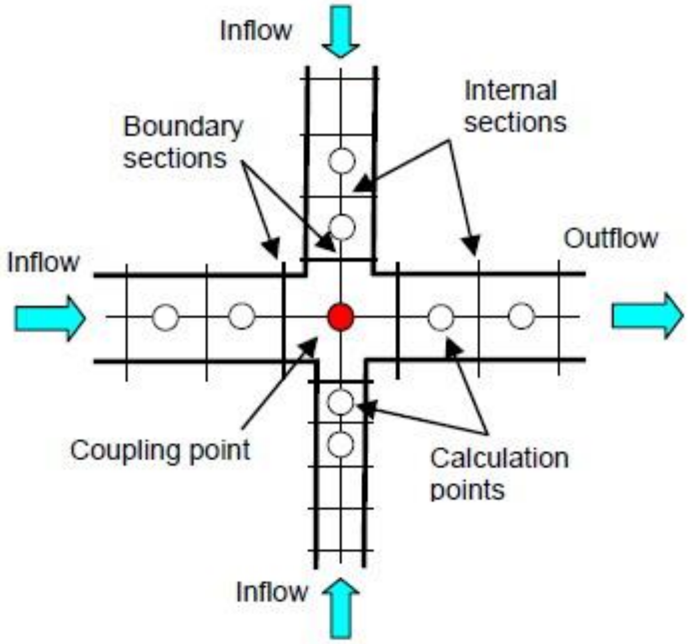


Figure 2.1: Coupling between diffferent reaches

The directions of flow path may not affect the simulations and may vary during the simulation process. They can be calculated from the hydraulic parameters of a particular domain (Mikdad and Zhang).

2.5 EFFECT OF URBAN ENIVRONMENT

Urban environment like building, green spaces, lawns, parks and parking play an important role in urban flooding. Increase in size of the city results in lot of impervious area; (building, parking, etc.) and decrease in Green spaces (parks, lawns etc.) which disturbs the capacity to hold the surface water runoff naturally (Rappaport and Sachs, 2003).

Imperviousness is the result of urbanization and is the reason for frequent flood occurrences. The relation between flood occurrence and impervious surface was developed in 1960 (Leopold, 1968; Seaburn, 1969). Increase in impervious area results in increase in surface runoff and decrease in infiltration of surface water (Leopold and Dunne, 1978). Increased surface runoff results in more frequent and severe floods (Brody et al., 2008). There is a link between urban environment and high peak discharges (Leopold, 1994). The lag time (difference between Centre of volume of precipitation and Centre of volume of surface runoff) decreases because of increased imperviousness (Hirsch et al., 1990). Wetland are considered to be natural flood mitigation sources (Lewis, 2001; Mitsch and Gosselink, 2000).

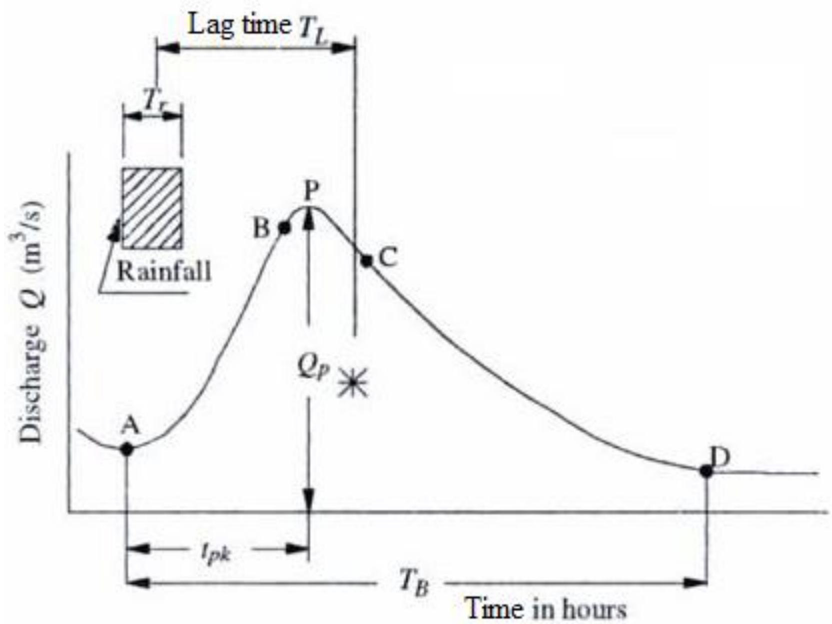


Figure 2.2: Lag time (Subramanya, 2005)

Fast urbanization is resulting in the replacement of vegetation areas which function as rainwater interceptors and store water in soil. Green spaces have a positive effect on water infiltration and storage in soil (Beard and Green, 1994). Runoff reduces due to green spaces (Cheng et al., 2008). Creation of green spaces is suggested by considering the recent flood occurrences (Dunnett and Kingsbury, 2004). The city managers are underestimating the positive role of green spaces and are more concerned towards construction of better drainage system which results in more financial cost (Zhang et al., 2012).

According to (Zhang et al., 2012) Green spaces can provide infiltration to storm water, purification of water and can also reduce the impacts of floods and droughts.

2.6 PREVIOUS STUDIES ON NIMES CITY AND SIMILAR URBAN FLASH FLOODING

(Paquier et al., 2003) Carried out a research on Nimes by comparing two models RUBAR20 2-D model and REMU 1-D model and found that 1-D models are good in highly dense urban area but 1-D/2-D coupled models should be used while 2-D model should be used in crossroads and large spaces.

(Mignot et al., 2006), used a 2-D Saint-venant equation model Rubar20 to simulate October 1988 flood in Nimes, France. Sensitivity analysis was carried out and 2002 flood event was used to calibrate the model and to plan mitigation measures in the affected urban area.

(Mikdad and Zhang) used REMU 1-D model which uses Finite Element method to simulate the 1988 flood event of Nimes.

(El Kadi Abderrezzak et al., 2008) used a 2-D depth averaged shallow water model to simulate 1988 flood event in Nimes France. He altered the strickler's coefficient in two simulations from 10 $m^{1/3}$/s to 40 $m^{1/3}$/s and compared the results with observed value.

(Paquier et al., 2015) gave a general idea from calculation of hydrograph to flood hazard map how the parameters like crossroads, building, obstacles and not including the parameters like sewers can produce the uncertainty in results of simulation of flood.

(Lhomme et al., 2005) developed a GIS based 1-D model and simulated the flood event of Nimes and compared it to fully 2-D model results. He setup a conceptual scheme which is used for Y-shaped pipe junction and applied it on X-shaped crossroads and found that results are good on steep slope streets but not good at crossroads.

(Ahmad et al., 2010) carried out research on Nullah Lai flooding by coordinating hydrological model with GIS. In this study HEC-RAS and HEC-GEORAS were utilized for estimation of flood zones of 2001 Flood event. Distinctive surge maps were setup in view of flood reoccurrence of 25, 50 and 100 years return periods.

(Mahmood Siddiqui et al., 2011) carried out research on Nullah Lai Pakistan using the 1-D model MIKE 11 by simulating the flood event of 2001. For further flood scenarios he ran the simulation for return period of 5, 10, 25 and 100 years based on frequency analysis of 3, 6, 9 and 12 hourly rainfall.

(Morales-Hernández et al., 2016) used 1-D/2-D coupled model for simulation of Tiber river and concluded that simulation time for 2-D models is more and also need high resolution data whereas simulation time for Coupled model is 15 times faster as compared to a 2-D model.

Chapter 3

STUDY AREA AND METHODOLOGY

3.1 GENERAL

Nimes city of France is located in northern area and have a long history of record severe flood events. Specifically the flood event occurred on 3rd October 1988 is very important. It was caused by rainstorm that generated 420mm of rainfall on northern hills which lead to inundation of several localities with water depth of up to 3m. Lives and property were damaged by this resulting flood. Due to its severe nature and devastating effects it attracts researchers (Mignot et al., 2006). The purpose of this study was to carry out 1D analysis using 1d coupled network technique using a freeware and compare the results with flood marks.

3.2 SITE DESCRIPTION

Nimes is a French city situated in the plain downstream from Seven Hills incorporating its northern area. “Richelieu” is the studied area situated in the north eastern part of city and is one of the most severely affected zone. The dimensions of the region are around 1400m along the North-South axis and that is likewise the main stream flow axis and a variable East-West width with a maximum 1050m and minimum 220m. A railway embankment runs all the way along the northern side. The western flank is shaped by hills and the eastern flank is constituted by railway line embankment. No rivers cross the Nimes. Whenever there is rain, the flow carried by the temporary water courses enters the urban zone and then flows in underground water courses until reaching the Vistre River downstream of city of Nimes. However when the flow exceeds the channel capacity, the water overflow occurs at upstream of the city and the flow enters the Northern part following the natural North-South slope within the streets. Richelieu is one of these areas where flow can enter through a few street underpasses underneath railway embankment. The drop in ground level in the studied area is 15m and 20m, respectively, representing steep slope. From the eastern and western underpass respectively to the southern boundary which represents an average slope higher than 1%. The part which is located in the upstream part of city is mainly composed of large building areas such as Hospital and military barrack with wide streets and crossroads. The central part consists of narrow regular streets network with right angle crossing. The southern part of the city consists of average width streets and still follows the main North-South Slope (Mignot et al., 2006).

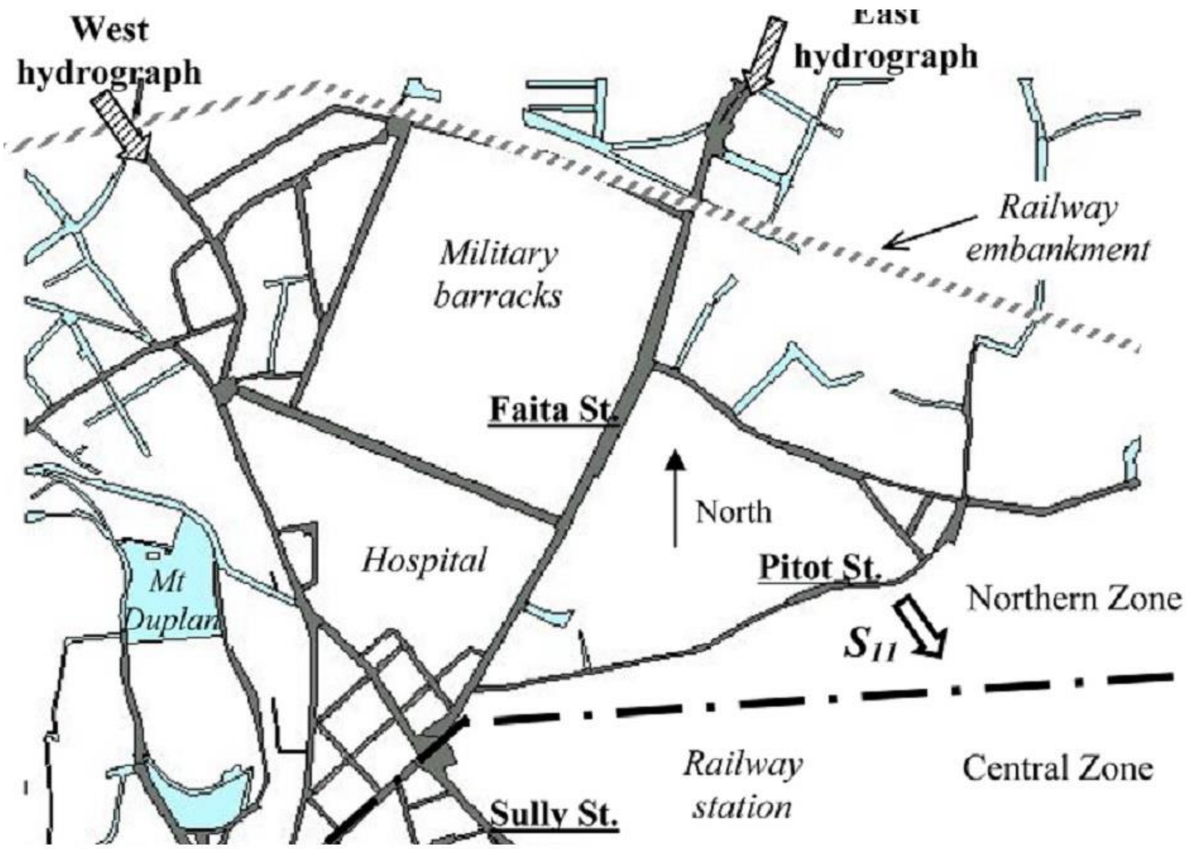


Figure 3.1: Nimes and its region (Mignot et al., 2006)

3.3 DATA COLLECTION

To run simulation the primary data is provided by the Public works department of the Gard County and the Technical Services of Nimes city. It is comprises of

225 cross-section profiles of the 46 streets of the studied area.

A map is also provided which contains built up places and ground elevations also showing the limits of flooded area. Immediately after the flood marks of flood elevation points 99 had been recorded and put on the map to show the location of measurement. About 85% of the marks are from the cross roads and other 15% are from front of the buildings in the streets. Rainfall data was measured at many locations in the area.

A French consulting firm BECOM performed a hydrological analysis on the upstream basins which are of area 10km^2 and 3.5km^2 starting from rainfall measurement to calculate the hydrographs at the upstream border of studied Richelieu area. The model is based on a conceptual linear tank approach similar to GR4 model(Perrin et al., 2003). It considered the hydrological processes for the urban areas and for the grass lands of the upstream basins. The model is calibrated by modelling various events and compared with flood marks on upstream of the Richelieu domain (railway embankment).

In computing the hydrographs at the upstream boundary of the study domain the important point was about the sewerage network. The sewerage network capacity for the studied domain was 4-7 m^3/s which was subtracted from the calculated inflow hydrographs. The sewerage network evacuation capacity is less than the total peak discharge of Nimes 1988 flood. Also the sewage network was full before the upstream hydrograph reached due to rainfall. As a result of hydrological modelling two hydrographs are computed northwest and northeast hydrographs shown in figure 3.2 which can be used as inputs at upstream boundary of the studied domain. Hydrographs were of 13 h span with time step of 15 min.

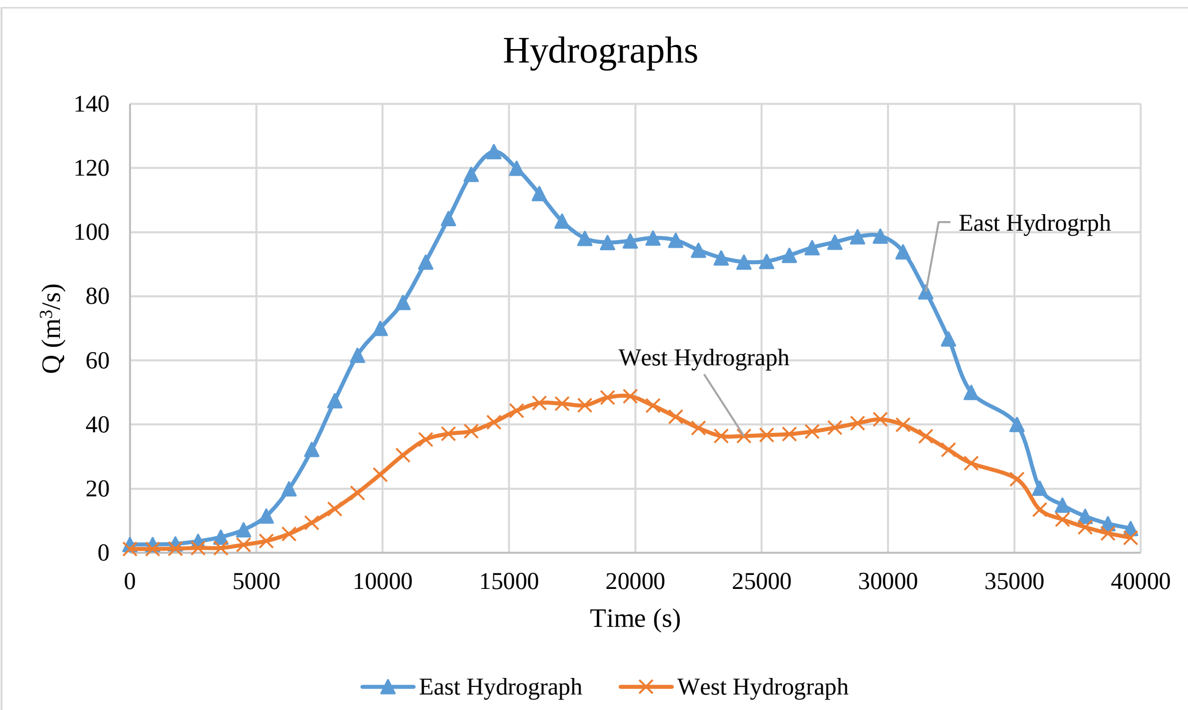


Figure 3.2: Inflow Hydrographs

Linear interpolation has been done to obtain the cross-sections of the street junctions. Additional points and information has been added for the complex topographical crossings. The buildings were considered as water tight and no details were introduced for them. In hydraulic modelling the flow exchange with sewage network is neglected. The DEM contains about 25,000 points described by three co-ordinates x, y, z.

3.4 FLOODS IN NIMES

Since 1350, numerous flood events have been recorded in the Nimes city. Floods events of 1988, 1998, 2002, 2005 and 2010 are of great importance. Major events are shown in the table 3.1.

Table 3.1: Characteristics of major rainfall events (Fleury et al., 2013)

Date	**Cumulative rainfall (mm)**	**Soil recharge**	**Rainfall duration(h)**	**Peak discharge in Alès cadereau (m^3/s)**
3 October 1988	420	Yes	8	< 300
27-28 May 1998	180	No	30	~20
8–9 September 2002	190	Yes	26	~40
November 2004	90	Yes	9	~20
6 September 2005	225	No	15	~30
8 September 2005	200	Yes	18	~80
7–8 September 2010	180	No	24	~12

3.4.1 3 October 1988 Flood Event

Rainfall event of the 3rd October, 1988 lasted 8 hours and generated 420mm which led to overflow of water courses. As a result inundation of localities up to 3m of water depth occurred causing considerable damages to life and property. Return period of this event was 150- 250 years (Mignot et al., 2006).

3.5 DATA COLLECTION

The studied area “Richelieu” located at northern part of the city of Nimes has the area of 1400m along north-south axis and is the main axis of flow. The maximum and minimum width along the east-west axis is 1050 and 220m respectively.

BECOM French consulting firm performed hydrological modelling on the studied area to compute the hydrograph at the upstream. Discharge capacity of sewerage network which is about (4-7 m^3/s) is subtracted from surface flow hydrograph. Building blocks were considered as watertight bodies and additional points were introduced for the complex cross roads.

3.6 GEOMETERY PREPARATION

A number of supporting programs are used in preparation of Geometry of area which includes Excel and Notepad++. BASEMENT accepts the geometry file of extension BMG. For preparation of supporting geometry files Excel (to calculate the distances between the points and distances between the cross-sections) and Notepad++ (for raw editing of BMG files) are used.

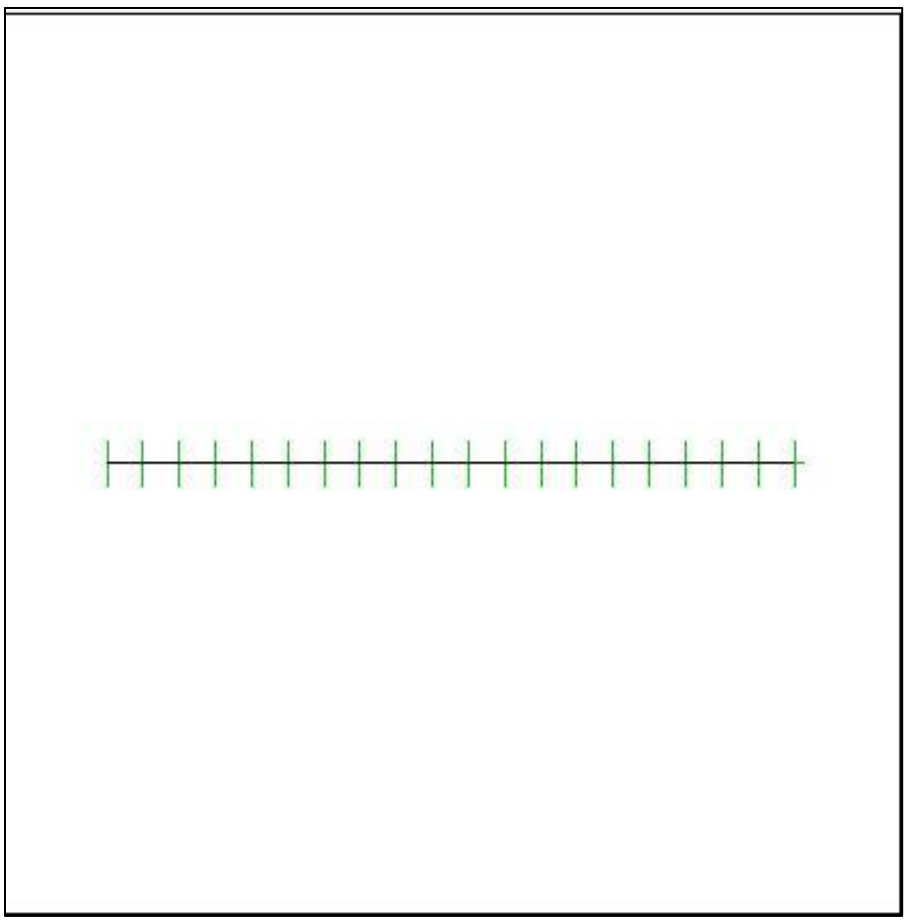


Figure 3.3: Plan of a domain consists of x-sections

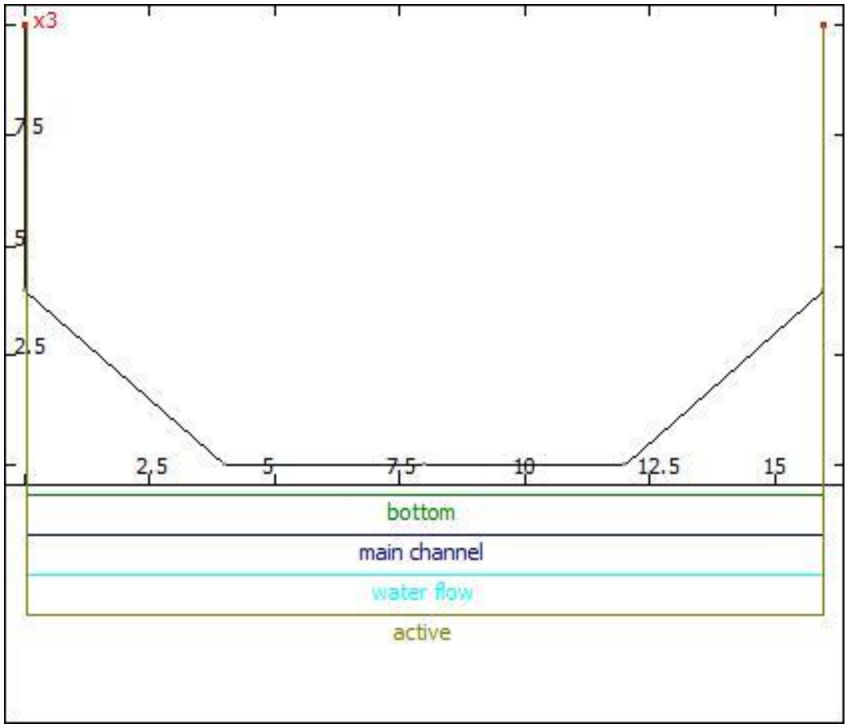


Figure 3.4: Cross-section view (m)

3.7 MODEL SETUP

After the generation of geometry of the study area couple based 1-D network of channels was made. Due to shortage of time, the 1-D flood simulations were carried out only on the northern part of the model as used by (Mignot et al., 2006). Thus, the southern part comprising of narrow streets, meeting at right-angle to each other was not included.

The models was set up as a coupled network of 1-D channels. In this method every street/channel of study area was considered as a single “domain”. It was then coupled to the other domains (streets) by employing the standard junction functionality available in the BASEMENT model, namely, sequential, junction and bifurcation. Suitable boundary conditions were provided to every domain at upstream and downstream to connect the domains with each other. At the upstream of first domains hydrograph boundary condition was provided and at the outlet of the model different boundary conditions like Zero Gradient, HQ relation and Weir were used. Whereas in between the other domains Coupling HQrelation at the downstream boundary conditions and coupling hydrograph at the upstream boundary conditions were used. For the reference case Strickler’s coefficient of 40 $m^{1/3}s^{-1}$ was used (Mignot et al., 2006). For other simulations Strickler coefficients of 20 and 30 ($m^{1/3}s^{-1}$) were used. The flow was introduced into the model by applying the eastern and western hydrographs. The value of CFL (Courant-Friedrichs-Lewy) 0.95 is used for every domain and every simulation. CFL is parameter used to control the time steps in computation. It is the ratio between spatial resolution and time step. It controls the stability of the computation. Minimum water depth parameter was chosen 0.05m for every domain.it is also an important parameter for stability of computation. If the water surface elevation at the cross section is greater than minimum depth the cross section is considered as wet. If the water surface elevation is less than minimum water depth than the cross section is considered as dry. Model is first tried on initially dry conditions but failed to run due to dry conditions. Models usually become instable under dry bed conditions due to rapid change in discharges and flow depths. So, appropriate conditions have to produce to start the computation. A steady hydrograph of 0.5 m^3/s on both inflow locations is introduced to produce initial wet conditions. After that the actual hydrograph is introduced and simulation is carried out.

3.8 CROSSROADS/JUNCTIONS MODELLING

Currently BASEMENT have the following 1-D coupled models available:

The sequential coupling is suitable if within the same street, there is hydraulic structure or parameters like slope, flow or geometry is varying significantly.

The junction connects three domains i.e. two upstream merge into one downstream domain.

The bifurcation is a reverse of junction, in which one upstream domain splits into two downstream domains. For further details see Annex-1.

BASEMENT model is incapable of modelling at junction with four channel/streets. During our model setup at certain points 4 streets/channels have to be modelled. For this, a strategy is adopted in which three channels/streets are connected with available function of junction and bifurcation whereas the fourth street is sequentially coupled with one upstream domain.

At couple of points there were junctions with two inflows and two outflows. In such situation a junction is used with two inflows and one outflow whereas the second outflow is sequentially coupled with one of upstream channel.

3.9 CALIBRATION OF STRICKLER'S COEFFICIENT

Strickler coefficient (K_s) is the inverse of Manning coefficient and it represents the effect of roughness to the flow (Chaudhry, 2007). Roughness coefficient basically represents the resistance to flow. This factor is of dire importance in computation of flood modelling. Manning, Chezy or Darcy-Weisbach's equations are widely used for calculation of flows and velocities in flood modelling. In this study Strickler's roughness coefficient is used in setting up of model's parameter. The right choice of roughness factor is the most important step of any flood modelling. Error of 50% in roughness factor can alter the peak discharges of the floods around 40% (Ballesteros et al., 2011). Some factors are listed which affect the value of roughness coefficient (Te Chow, 1959)

- Surface roughness
- Vegetation
- Channel geometry
- Alignment of channel
- Deposition of material and scouring
- Obstruction

To calculate the roughness coefficient is not an easy task and in many cases cannot be determined by just the roughness characteristics(Phillips and Tadayon). Especially for urban areas there are fewer studies on what value of roughness coefficient should be used. For this study, as it is totally an urban area a single value of roughness coefficient is used for all domains/channels. In this study instead of Manning's roughness coefficient Strickler's roughness coefficient is used. The values of Strickler's roughness coefficient is selected by help of past studies and trial and error method. The effect of Strickler's roughness coefficient is discussed later in model calibration section.

Chapter **4**

SIMULATIONS AND ANALYSIS

4.1 GENERAL

In this chapter an analysis of the results from the 1-D flood modelling simulations are presented. The factors which effect the simulation results like Strickler's factor, boundary conditions are discussed. The purpose of this chapter is to compare the results of simulations with field observations and the 2-D model results (Mignot et al., 2006). Some recommendations to get best results out of the model are also made.

4.2 SIMULATIONS PERFORMED

A number of simulations were performed in this study. Initially, simulation was done to calibrate the model to get best fit results termed as base/reference simulation. Other simulation were performed for assessing different boundary conditions and effect of roughness coefficient. The parameter which were kept constant in all simulation are;

- Upstream boundary condition for discharge i.e. east and west hydrograph of 1988 flood event.
- Geometry of the area under study i.e. 46 domains and 235 cross-sections.
- Courant-Friedrichs-Lewy number (CFL) = 0.95
- Minimum water depth 0.05m
- Initial Conditions

4.2.1 Duration of Simulation

In order to simulate the flood event of 3rd October 1988, the simulation time was set to 20000 seconds. Simulation time was selected on the basis that at 14000 to 15000 sec water levels attain the maximum depths and thereafter it continue to decrease. This time is, hence, sufficient to capture the flood dynamics.

4.2.2 Boundary Conditions

To solve SVE BASEMENT needs suitable upstream and downstream boundary conditions. Therefore two open boundary conditions were used i.e. hydrographs at upstream boundary conditions and HQ Relation, Zero gradient and Weir were used as downstream boundary conditions.

Zerogradient is the boundary condition used as downstream boundary condition. It is used for outflow from the computation domain. This boundary condition means the variables water depths and velocities at the outflow points will be zero.

HQrelation is the boundary condition used for the outflow of a computational domain. It is the relation between discharge and stage level. Either a file with stage level and discharges can be used or a slope is used. With slope the BASEMENT model calculate the outflow discharges assuming the normal flow.

At the model outlet, the correct boundary conditions needed to be applied as the extent of the model was not the same as in (Mignot et al., 2006). The strategy adopted was to use the 2-D model results at the model outlet to reproduce the same water depths at downstream boundary conditions of 1-D model.

The following simulations were carried out to assess the effect of boundary conditions

- BC1: Combination of boundary conditions
- BC2: Zero gradient boundary condition
- BC3: HQ relation or rating curve boundary condition

4.2.3 Calibration of Model for Roughness

Determination of roughness coefficient is important for the calibration of the model. It has a significant effect on the predicted water levels. The value of Strickler coefficient was selected to get the best results. It has already been mentioned that due to urban area a single value is kept for roughness factor for the whole model. The value of roughness coefficient has been taken from previous studies (El Kadi Abderrezzak et al., 2008; Mignot et al., 2006; Paquier et al., 2003). Further reference simulation was performed on different value of Strickler coefficient. Root mean square error (RMSE) is used to measure the efficiency of the results.

$$Root\ Mean\ Square\ Error = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(C_i - O_i)^2} \qquad \text{(Eq. 4.1)}$$

Where "C" is the calculated values and "O" is the observed value at each cross-section and "n" is the number of flood marks.

4.3 RESULTS

Simulations performed are divided into three cases termed as BC1, BC2 and BC3. BC1 is Base Case 1 and is calibrated to give best results. In BC1 the simulation performed using Strickler coefficient of 40 $m^{1/3}s^{-1}$ is termed as the reference case. Whereas BC2 and BC3 are cases for other simulations to observe the effect of different boundary conditions and roughness factor. RMSE method is chosen to compare the results of 1-D present study and 2-D (Mignot et al., 2006) with the observed flood marks. A total number of 20 observation points are selected to compare the results using RMSE. The position of the observation points is as shown in the figure 4.1.

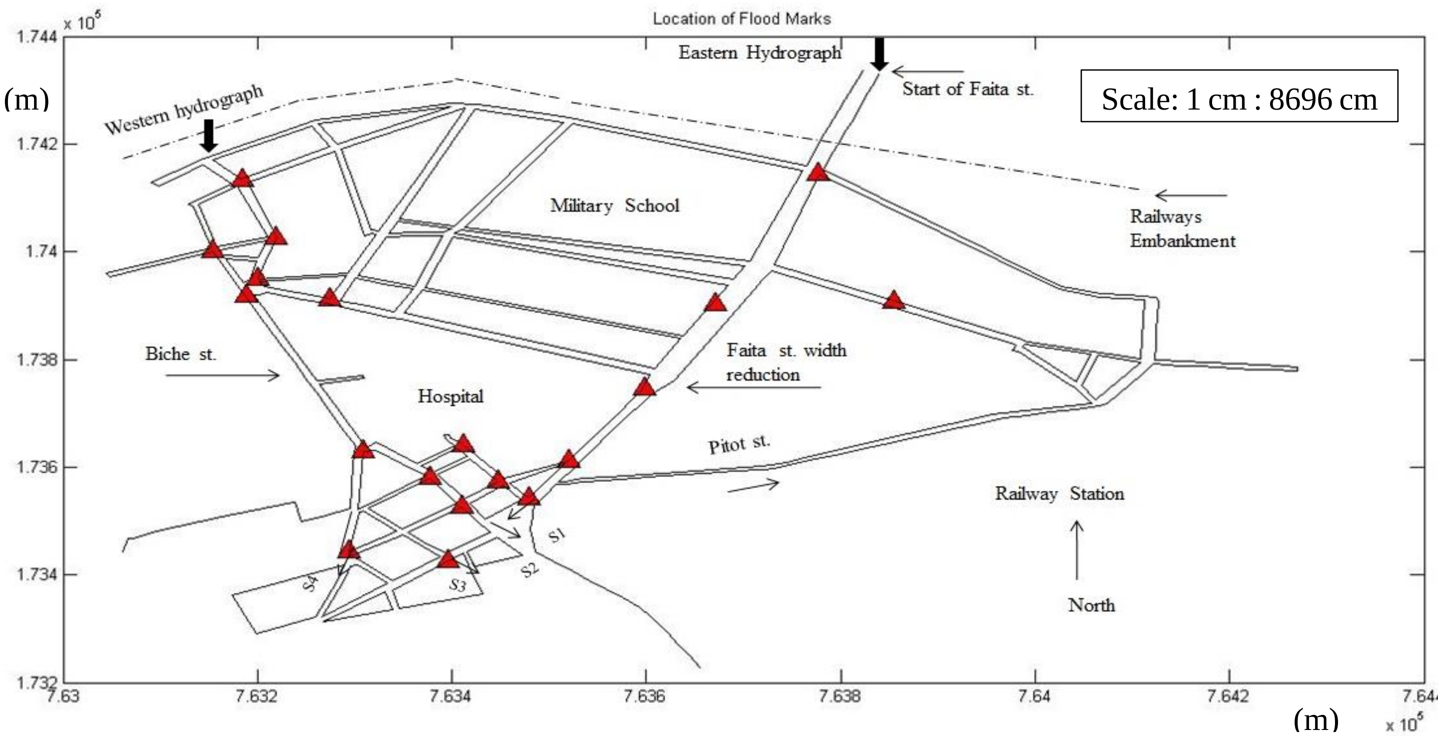


Figure 4.1: Location of Flood Marks

For the same observation points for 1-D (Present Study) and 2-D (Mignot et al., 2006) results compared with observed value using RMSE.

4.3.1 Base Case 1

In this case we used multiple boundary conditions at the outlet because the model is not full domain due to shortage of time and we have terminate the model at some location. So we tried to keep the same water depths at the outlet as of 2-D model (Mignot et al., 2006). To maintain that condition we changed the boundary conditions at the outlets S1, S2, S3 and S4. The model was run with Strickler's coefficient 20, 30 and 40 ($m^{1/3}s^{-1}$). Root Mean Square Error method is used to measure the efficiency of the results on 20 observation points as shown in Figure 4.1. The results of 1-D study is compared with 2-D models results in form of bar charts for different values of Strickler's coefficient i.e. 20, 30 and 40 ($m^{1/3}s^{-1}$).

4.3.2 Reference Case

In BC1 the simulation performed with Strickler coefficient of 40 $m^{1/3}s^{-1}$ which is adopted from (Mignot et al., 2006) is termed as Reference case. Flood marks are compared with results of 1-D coupled model. The difference between observed flood marks and simulated water depths is shown in the figure 4.2.

Since the model was cut in half and different boundary conditions were applied at the outlet to get the same water depths at the downstream boundaries of 1-D coupled model and also we used Strickler roughness factor 40 $m^{1/3}s^{-1}$. From the figure 4.2 we can see that at Faita Street near Railway embankment and at some points near the outlet simulated depths differences are in range of 0 to 0.5 m. At Faita Street reduction point the difference of water depth from observed flood marks is getting more than 1m. The model was also predicating the back water effect at that Faita street reduction point. At outlets difference of simulated water depths as compared to observed flood marks is below 1m.

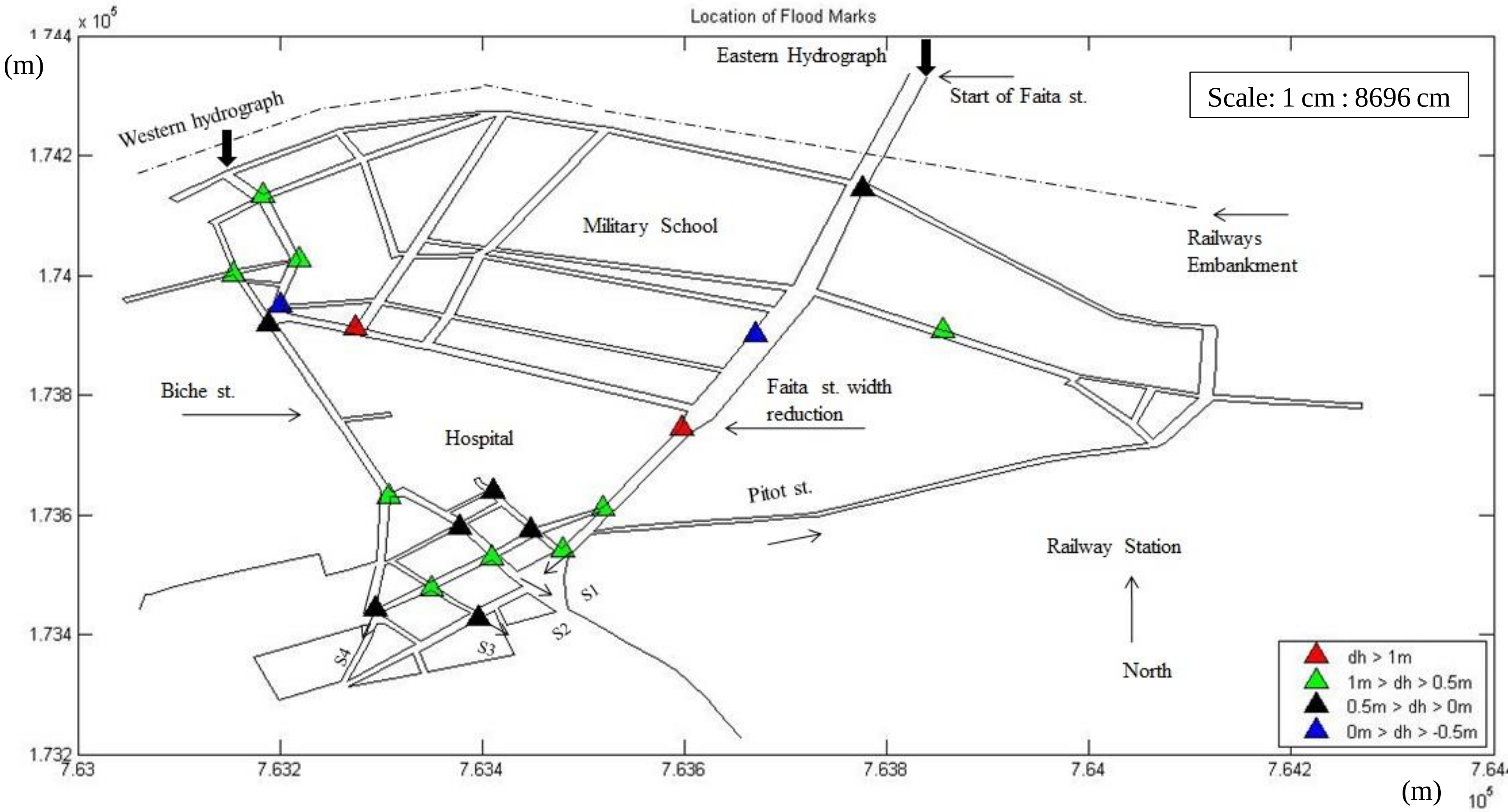


Figure 4.2: Comparison of simulated water depths with Observed Flood marks at K_s 40 $m^{1/3}s^{-1}$

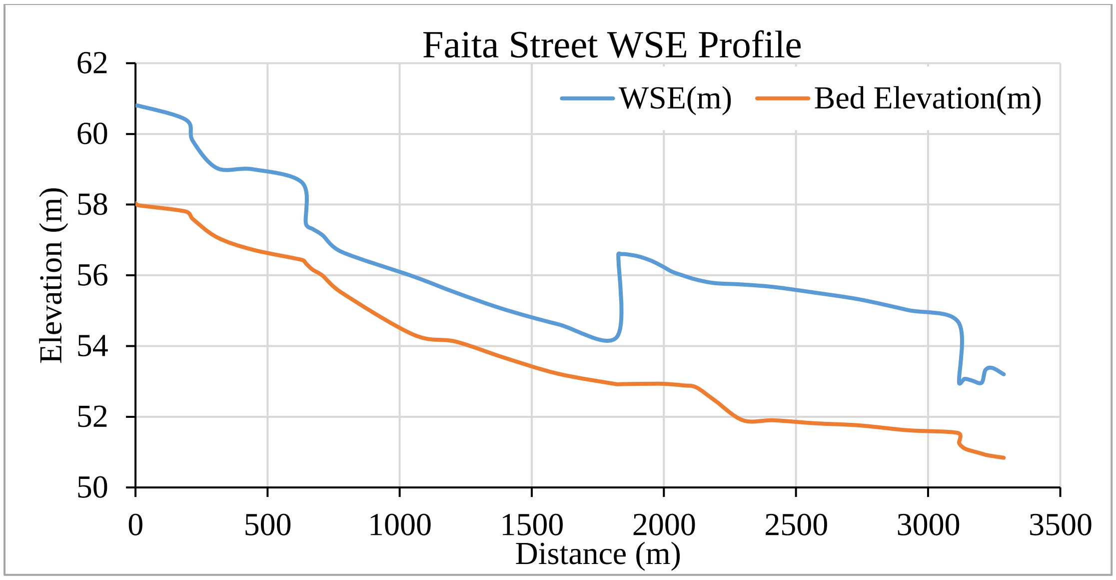


Figure 4.3: Water Surface Elevation of Faita Street

Figure 4.3 shows the water surface elevation of Faita Street at certain time step. The profile is truly representation of unsteady flow.

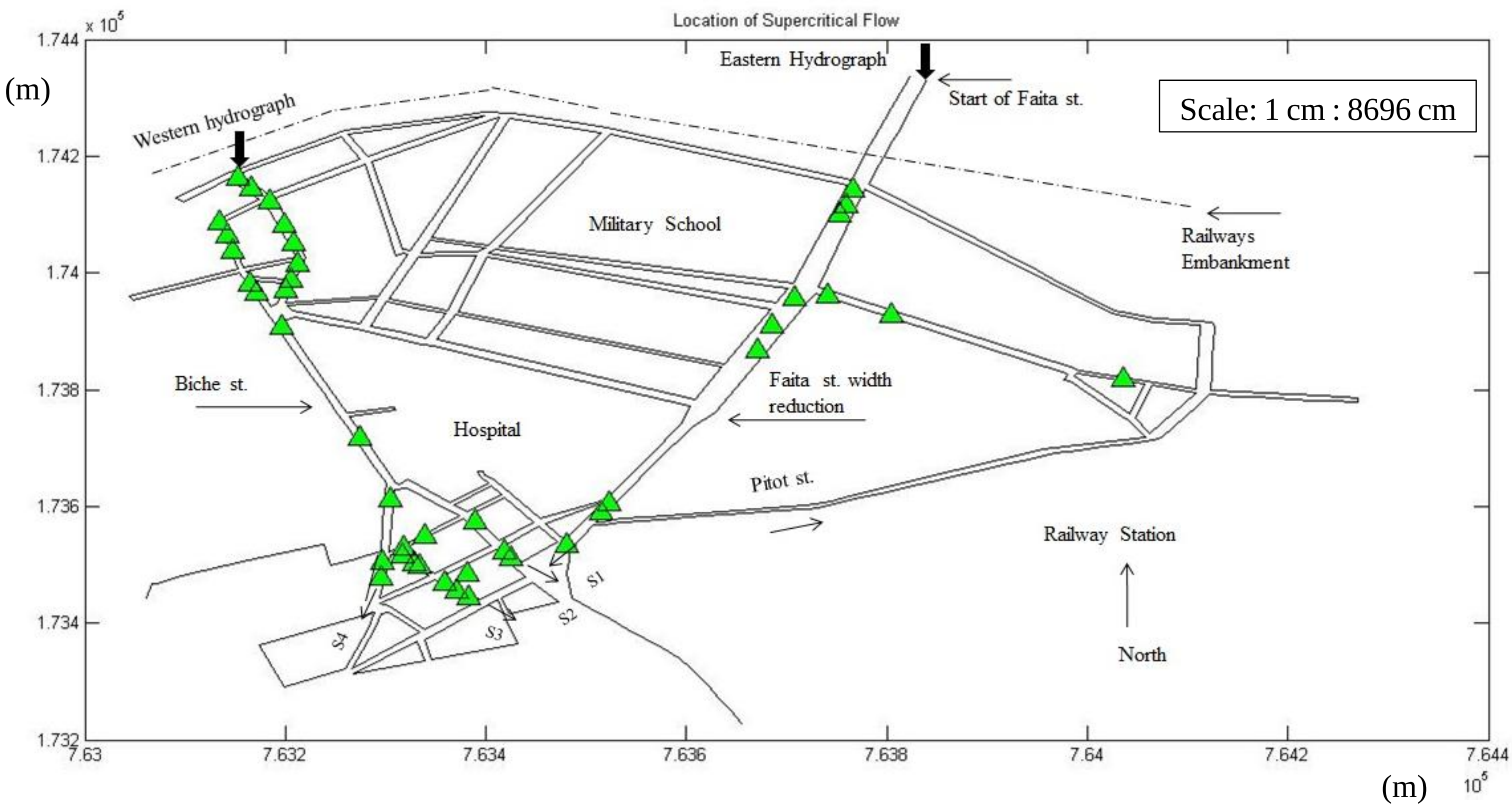


Figure 4.4: Location of Supercritical flows

Figure 4.4 shows the location of supercritical flows or hydraulic jumps. Still some freeware are unable to capture the flows transition from subcritical to super critical condition. So a robust shock capturing model was needed to capture the hydraulic jumps. BASEMENT model was quite capable of capturing the hydraulic jumps.

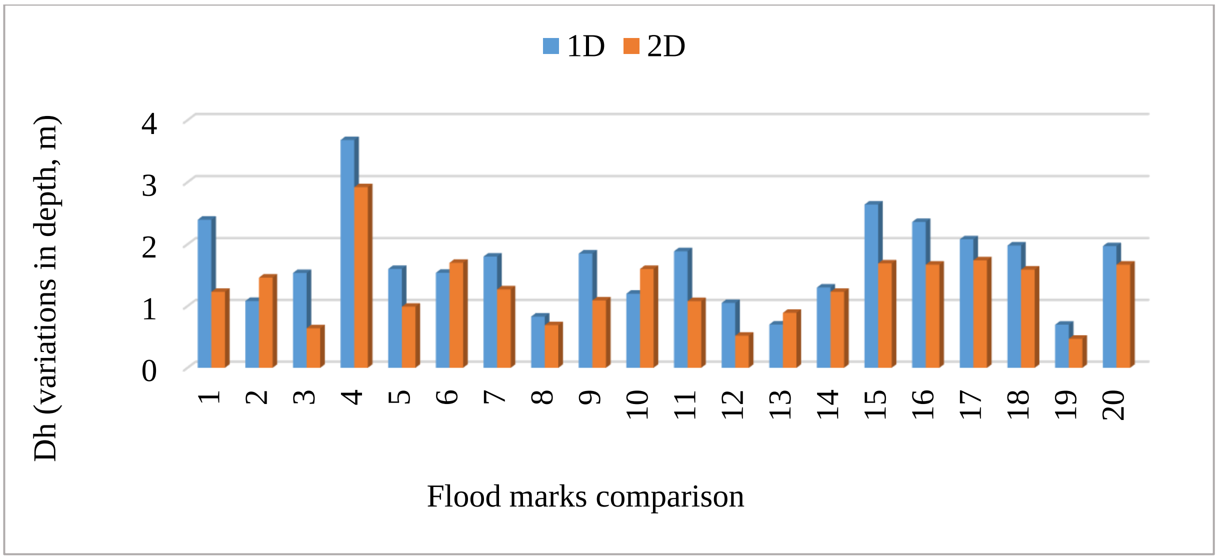


Figure 4.5: Flood marks comparison at Strickler's Coefficinet of 40 ($m^{1/3}s^{-1}$)

The results of simulation with Strickler's coefficient of 40 $m^{1/3}s^{-1}$ is compared with 2-D simulation results (Mignot et al., 2006) at 20 observation points as shown in the figure 4.5. From the bar chart it can be observed that results of 1D and 2D are quite comparable but at certain point 1D model is overestimating.

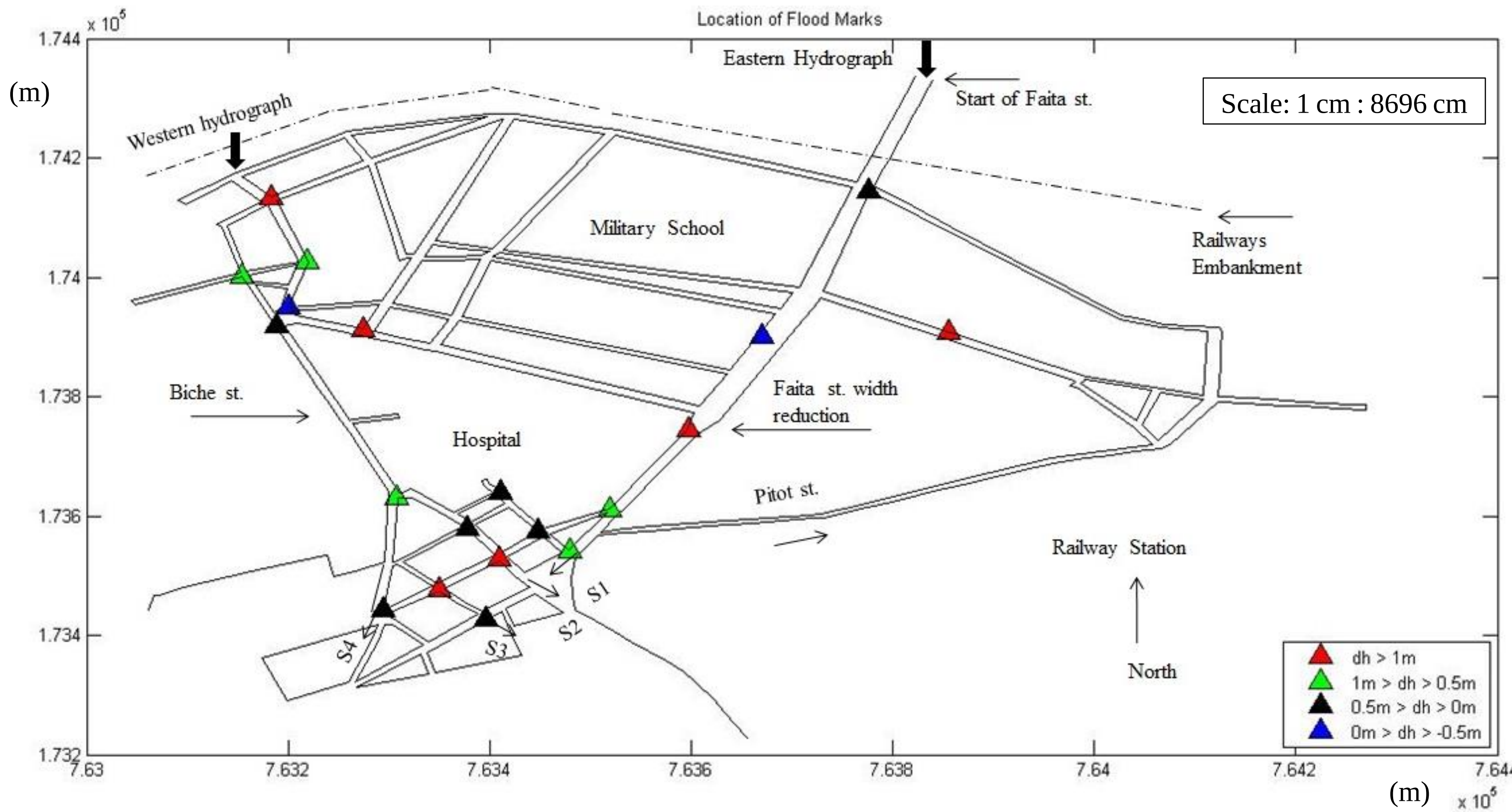


Figure 4.6: Comparison of simulated water depths with Observed Flood marks at K_s 30 $m^{1/3}s^{-1}$

Figure 4.6 shows the difference of water level of simulated water depths to observed flood depths at 20 marks. Since in this simulation roughness is increased ($K_s = 30$) the difference of water depths also increased. Increased roughness gave more water depth as compared to reference case. Due to this reason resulting simulated water depths at many points are more than 1 m as compared to observed flood depths.

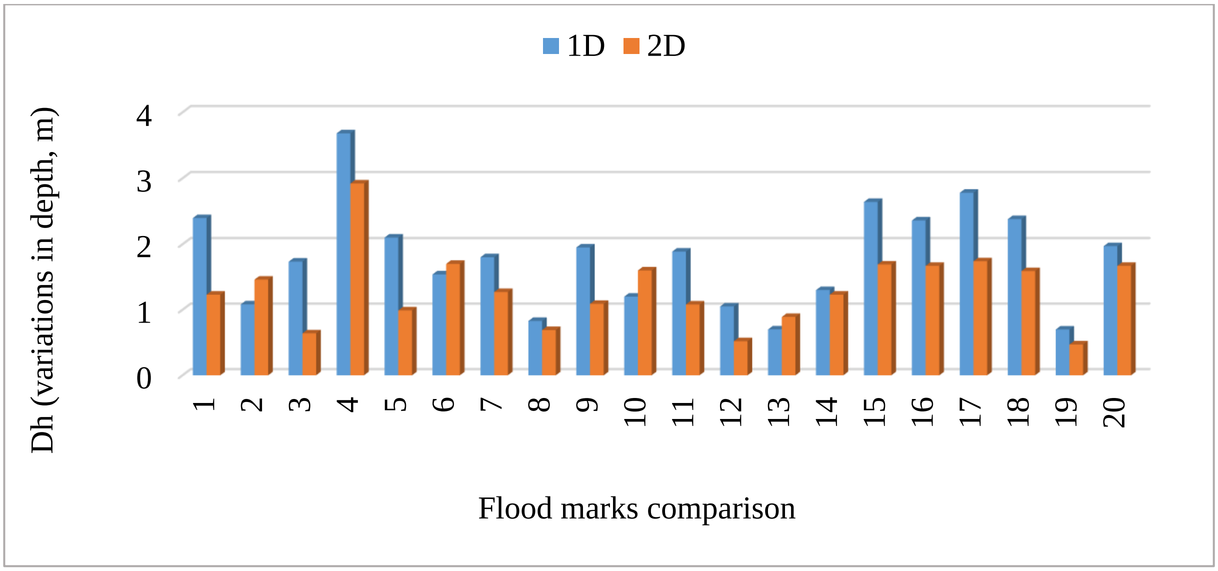


Figure 4.7: Flood marks comparison at Strickler's Coefficinet of 30 ($m^{1/3}s^{-1}$)

Figure 4.7 shows the difference of water depths of 1-D and 2-D models at Strickler's value of 30 $m^{1/3}s^{-1}$. There is no significant difference between the variations of depths between 1-D models results at Strickler's values 40 and 30 $m^{1/3}s^{-1}$ except at some points the water depth for Strickler's 30 $m^{1/3}s^{-1}$ is higher.

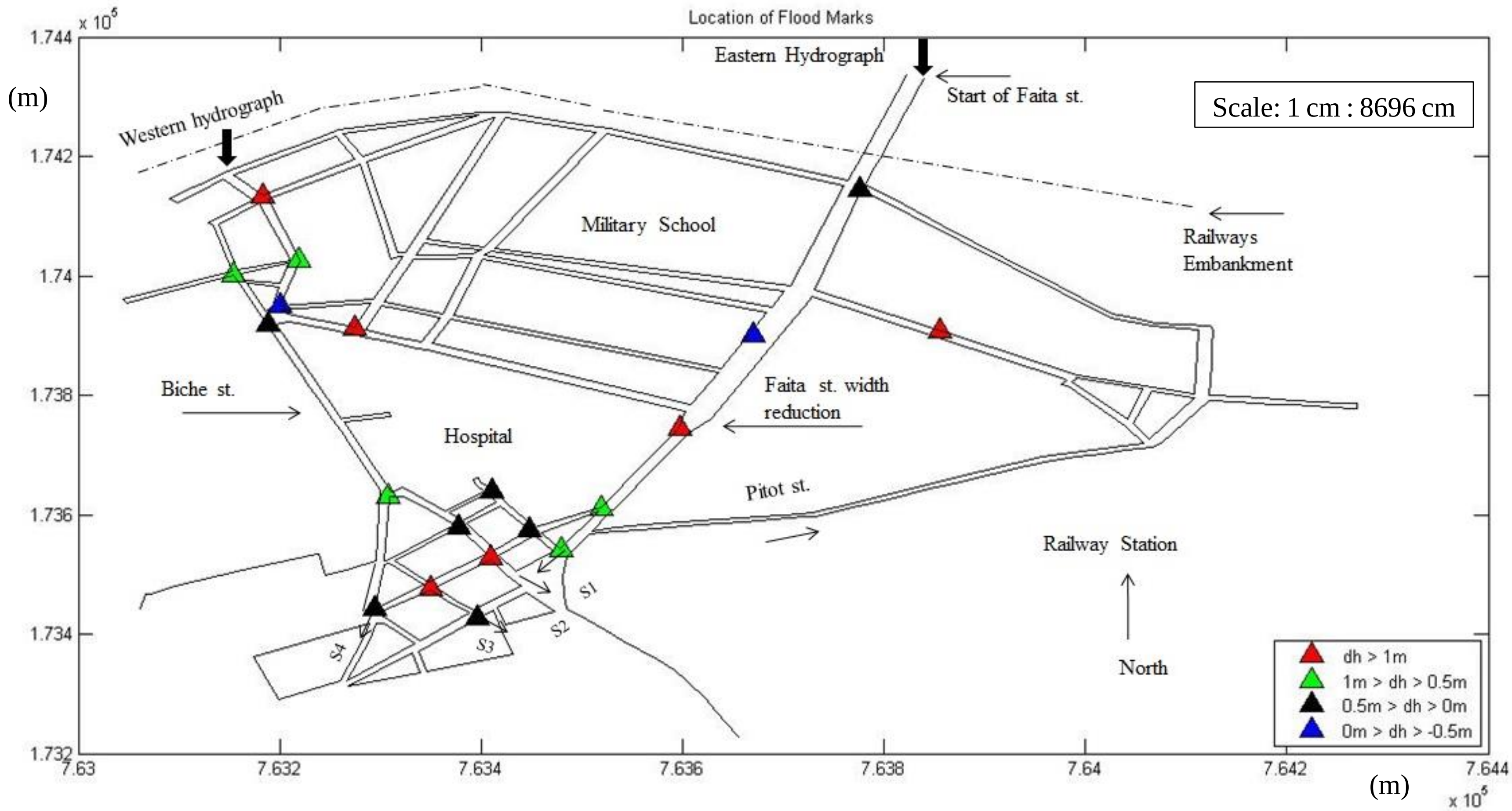


Figure 4.8: Comparison of simulated water depths with Observed Flood marks at K_s 20 $m^{1/3}s^{-1}$

Figure 4.8 shows the comparison of 1-D simulated water depths to the observed flood depths. In this simulation the Strickler coefficient of 20 $m^{1/3}s^{-1}$ is used which is high roughness factor, due to this reason the difference is increased at many flood marks up to more than 1m. In this simulation due to increase in roughness factor results of 1-D model significantly deviate from the reference case and give poor results when compared with observed flood depths.

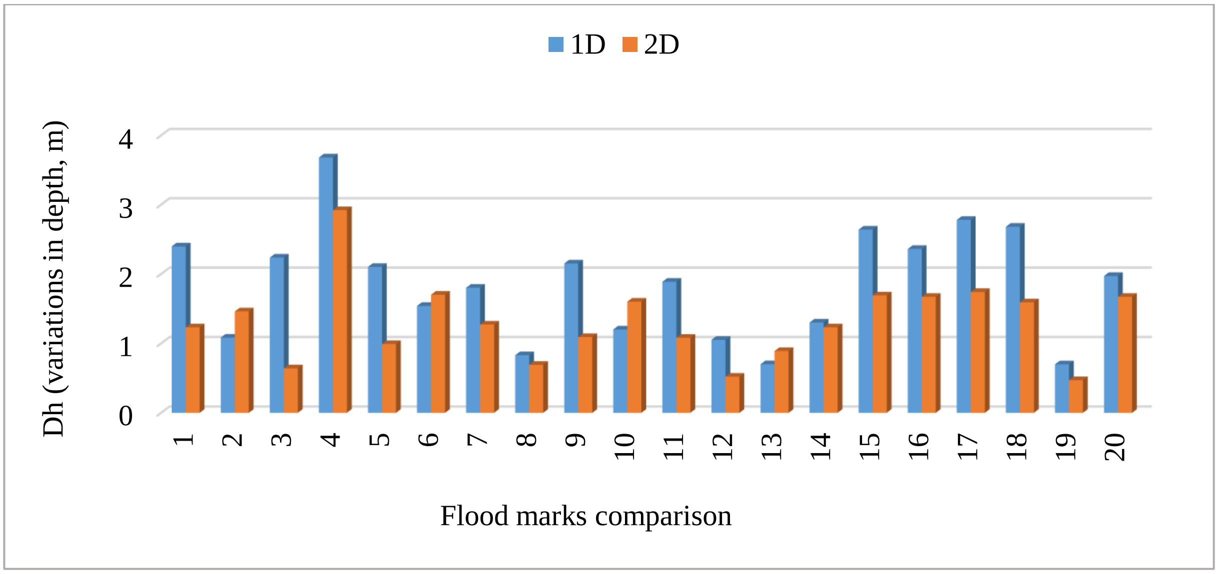


Figure 4.9: Flood marks comparison at Strickler's Coefficinet of 20 ($m^{1/3}s^{-1}$)

Figure 4.9 shows the results of Strickler's coefficient 20 $m^{1/3}s^{-1}$ which is high roughness value. At this roughness value the model gives more depths at most of the points. This results in more water depths as compared to 2-D simulated water depths. From the analysis we can conclude that with the increasing roughness the flow depths are also increasing. Reason behind the overestimating of the 1-D model results is the presence of uncertainties.

4.3.3 Base Case 2

In base case 2 every parameter is same as base case 1 except the outlet boundary conditions. The downstream boundary conditions at outlets s1, s2, s3 and s4 provided are HQrelation i.e. rating curve with slope of 2. When rating curve boundary condition is selected with slope, the model BASEMENT calculate outflow assuming normal flow. The variations in the water depths at observation points are given below. Similarly as Base case 1 simulations are carried out for roughness factor of 20, 30 and 40 $m^{1/3}s^{-1}$.

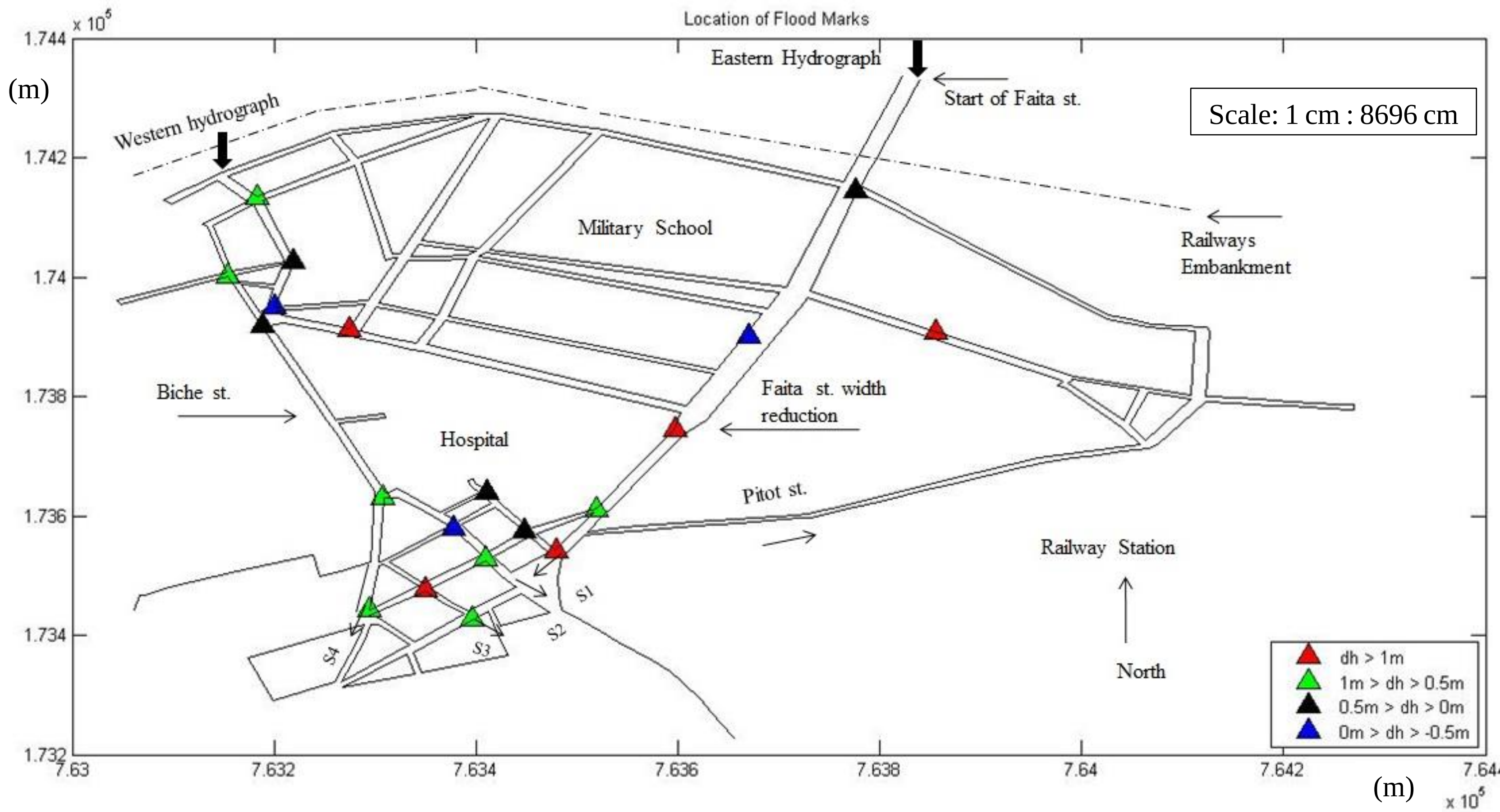


Figure 4.10: Comparison of simulated water depths with Observed Flood marks at K_s 40 $m^{1/3}s^{-1}$

Figure 4.10 shows the difference of 1-D simulated water depths with observed flood depths. As we applied rating curve as the outlet boundary condition. At the outlet S1, the difference of water depth level is more than 1 m. As compared to the reference case in this case at more points the difference of water depth is more than 1m. Whereas at other outlet the difference lies between 0 to 0.5m. Comparing with 2-D model results the figure 4.11 shows the difference of water depth level.

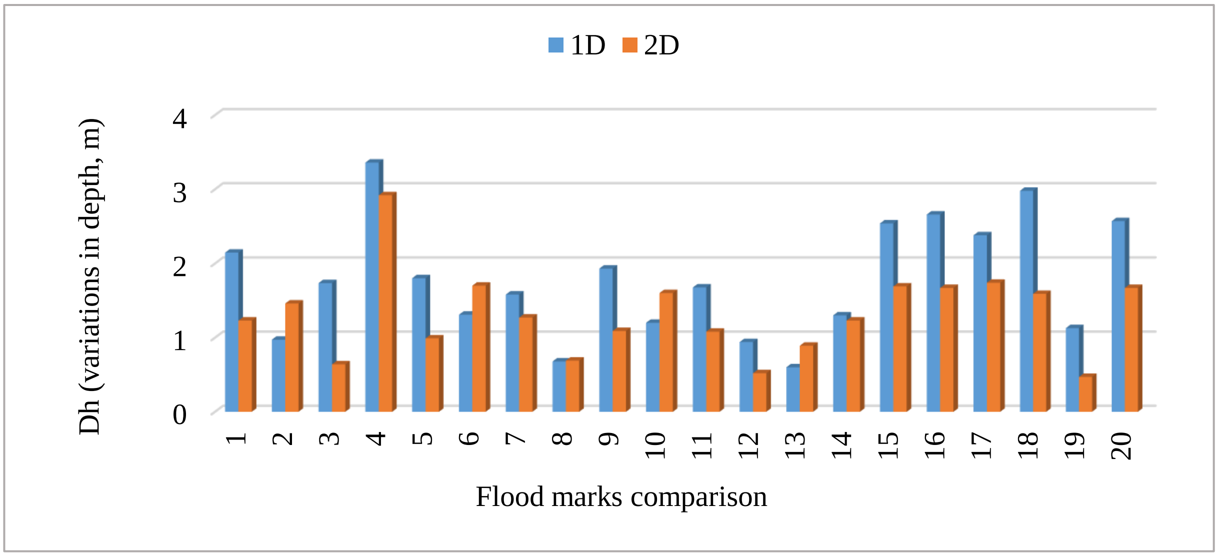


Figure 4.11: Flood marks comparison at Strickler's Coefficinet of 40 ($m^{1/3}s^{-1}$)

Figure 4.11 shows the difference of 1-D simulated results with 2-D model results and clearly from the bar chart graph it can be noted that at 40 $m^{1/3}s^{-1}$ 1-D results are not that significant as compared to reference case.

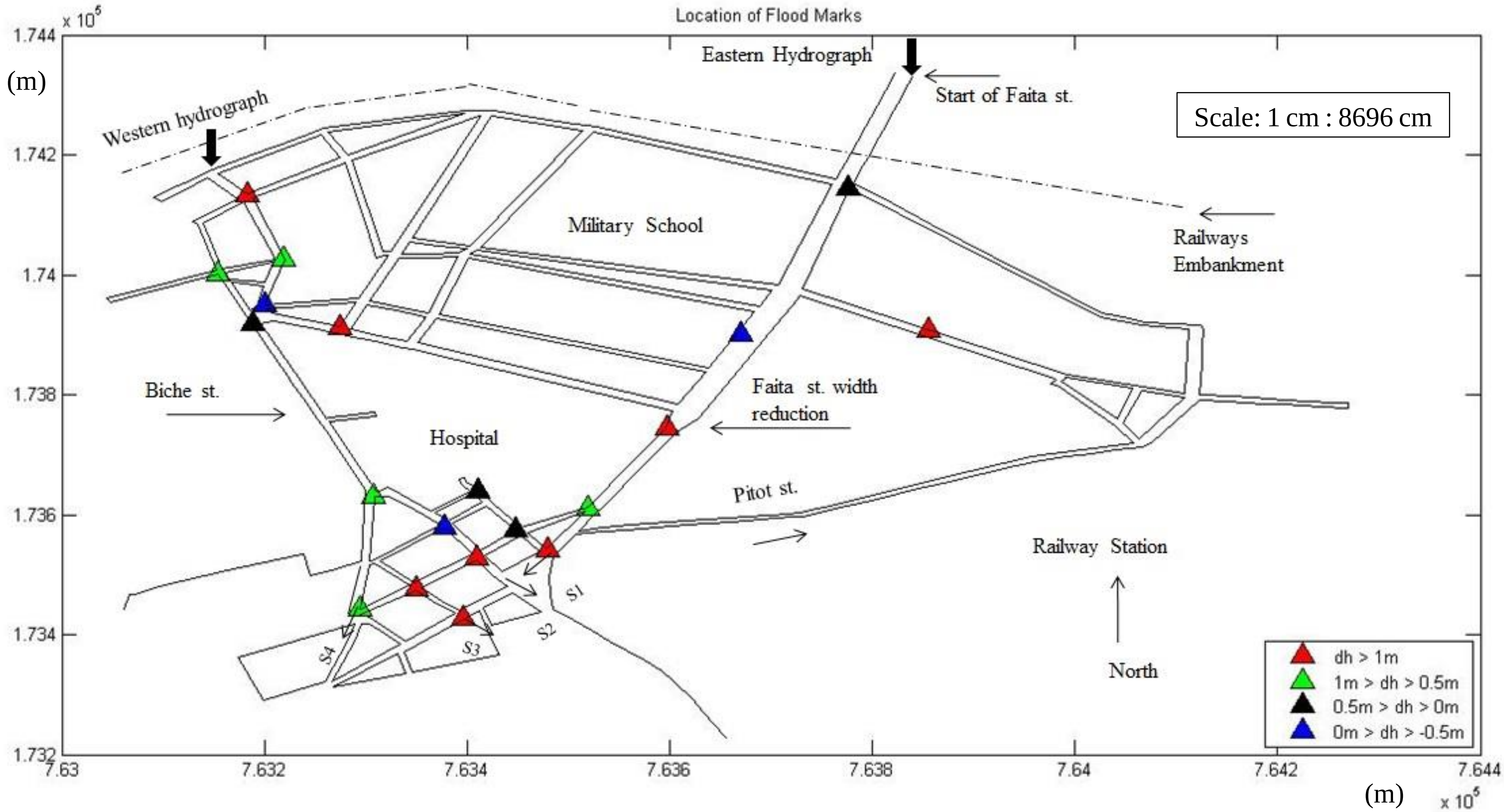


Figure 4.12: Comparison of simulated water depths with Observed Flood marks at K_s 30 $m^{1/3}s^{-1}$

In Figure 4.12 it is clearly seen that more points are getting depths more than 1m as compared to observed flood marks as compared to previous case (40 $m^{1/3}s^{-1}$). S1, S2 and S3 are getting more than 1m difference of water level than observed flood level. The point at Faita street reduction also gets more water depth whereas at Biche Street the difference of water level is less than 1m as compared to observed water levels. Due to increase in roughness the depths are also increasing.

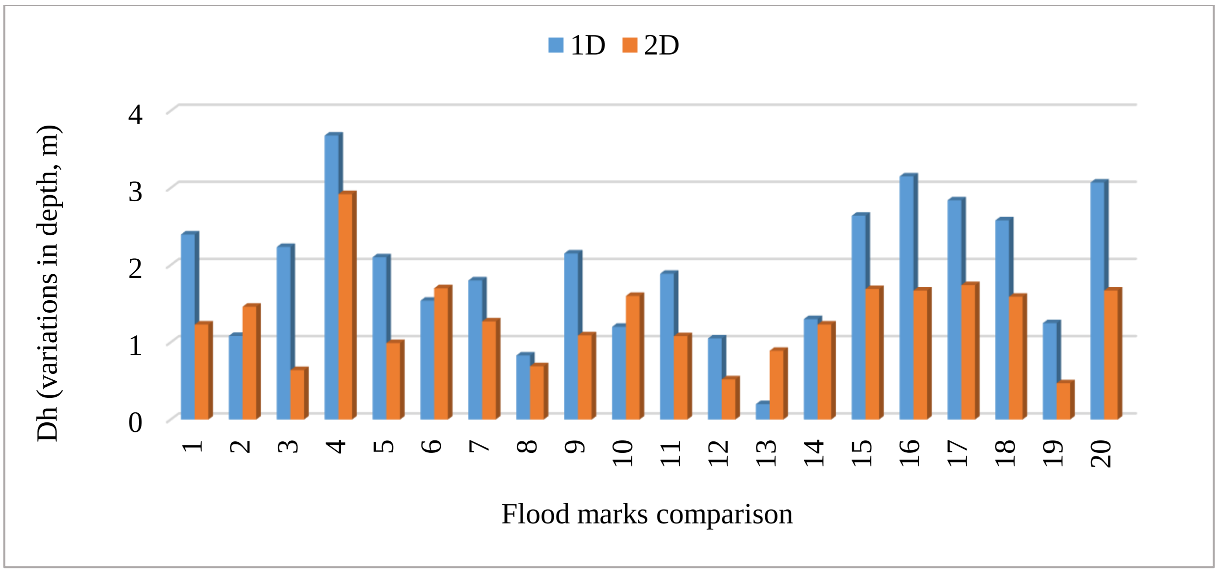


Figure 4.13: Flood marks comparison at Strickler's Coefficinet of 30 ($m^{1/3}s^{-1}$)

In this figure 4.13 shows the simulated 1-D results comparing with 2-D results and clearly shows 1-D results are less significant as compared to 2-D results.

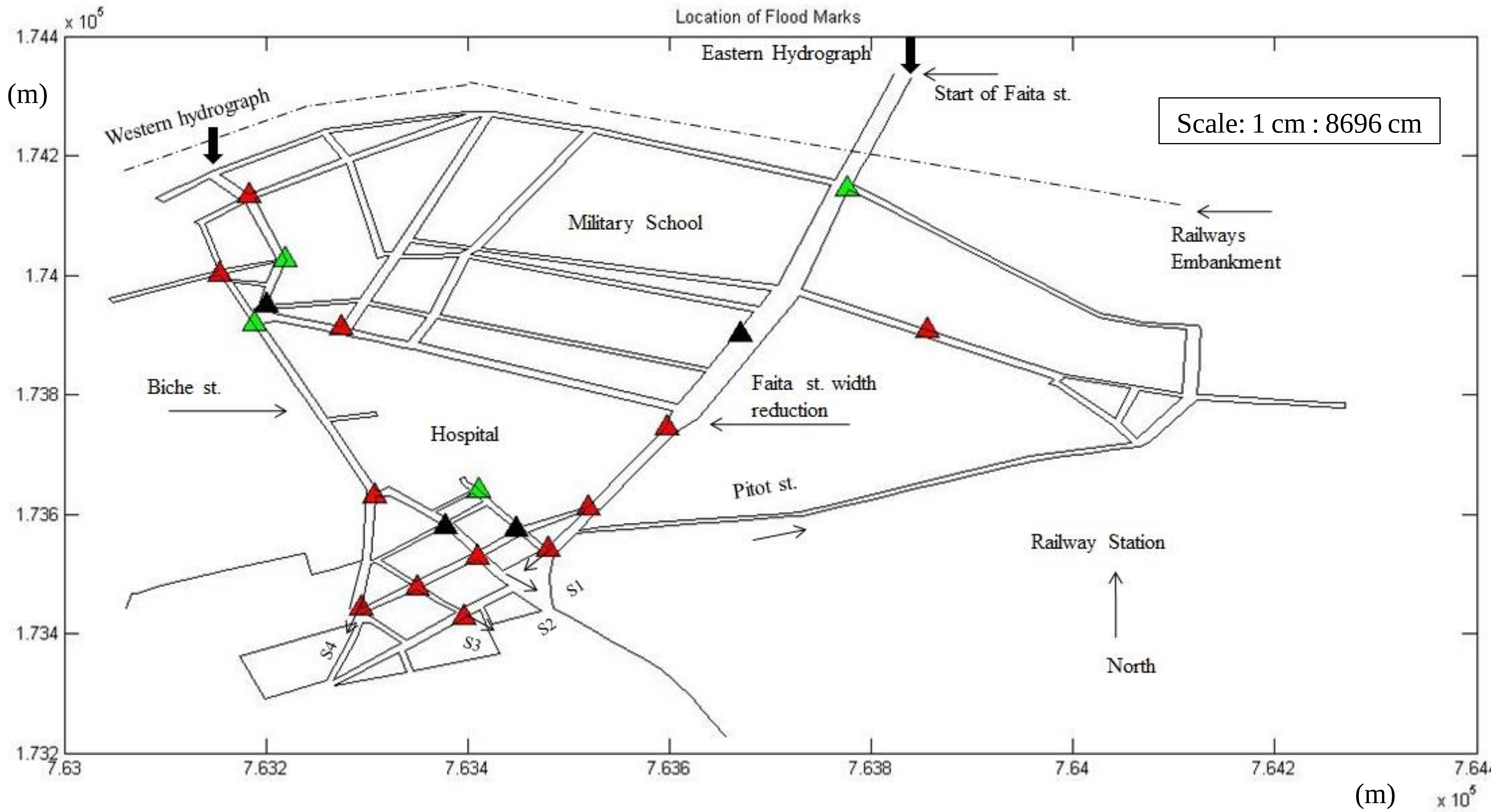


Figure 4.14: Comparison of simulated water depths with Observed Flood marks at K_s 20 $m^{1/3}s^{-1}$

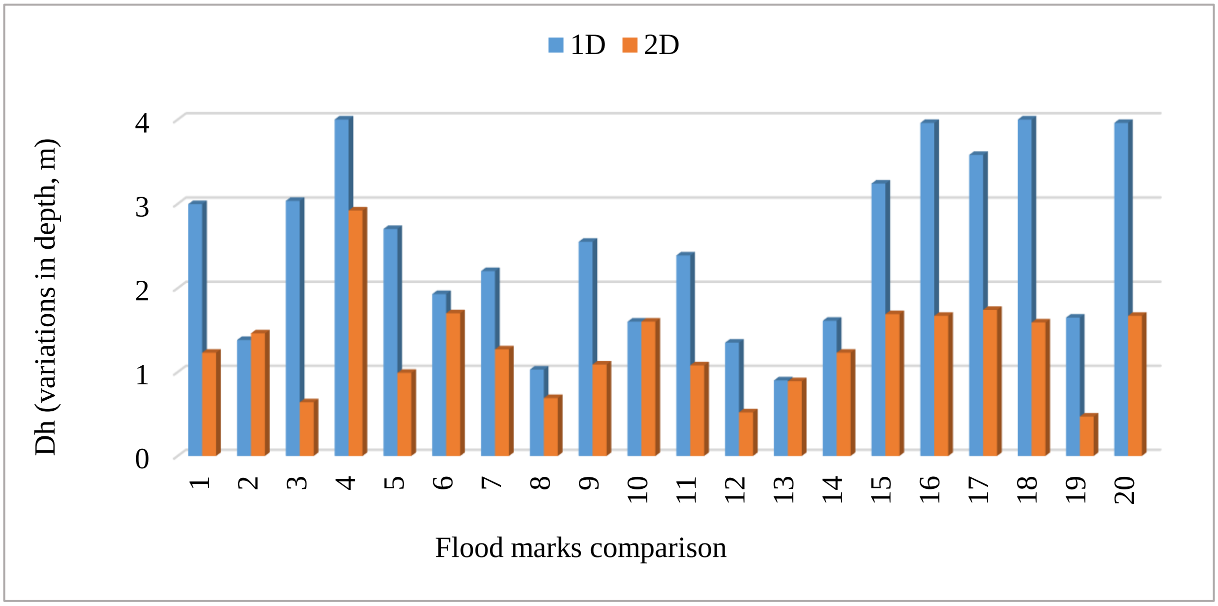


Figure 4.15: Flood marks comparison at Strickler's Coefficinet of 20 ($m^{1/3}s^{-1}$)

Figure 4.14 shows the comparison of simulated water level depths with observed flood depths at Strickler roughness factor of 20 and it can be seen that many points are getting more than 1m difference and no point at all is getting less water level difference than observed flood level. The reason behind this is using more roughness factor which is leading to bad results. This can also be verified in figure 4.15 where results varies significantly from 2-D results and give poor results. Since we neglect the effects of cutting the model in half in this case the results are significantly different from Base case 1. RMSE value for this case shows that as compared to base case 1, variation in depths for this case is far from 2-D results and give more water depths at almost all observation points.

4.3.4 Base Case 3

In base case 3 every parameter is same as base case 1 except the outlet boundary conditions. The outlet boundary conditions at outlets provided in this case are Zero Gradient. At zero gradient boundary condition, at outflow point the main flow parameters water depth and velocities become zero. The variations in the water depths at observation points are given below. Similarly as Base case 1 simulations are carried out for roughness factors of 20, 30 and 40 $m^{1/3}s^{-1}$. Figures 4.16, 4.17 and 4.18 show the comparison of simulated water depths as compared to observed flood depths, whereas figures 4.19, 4.20 and 4.21 show comparison of 1-D simulated water depths and 2-D model water depths. The same pattern can also be seen here, with the increase of roughness factor results are getting worse.

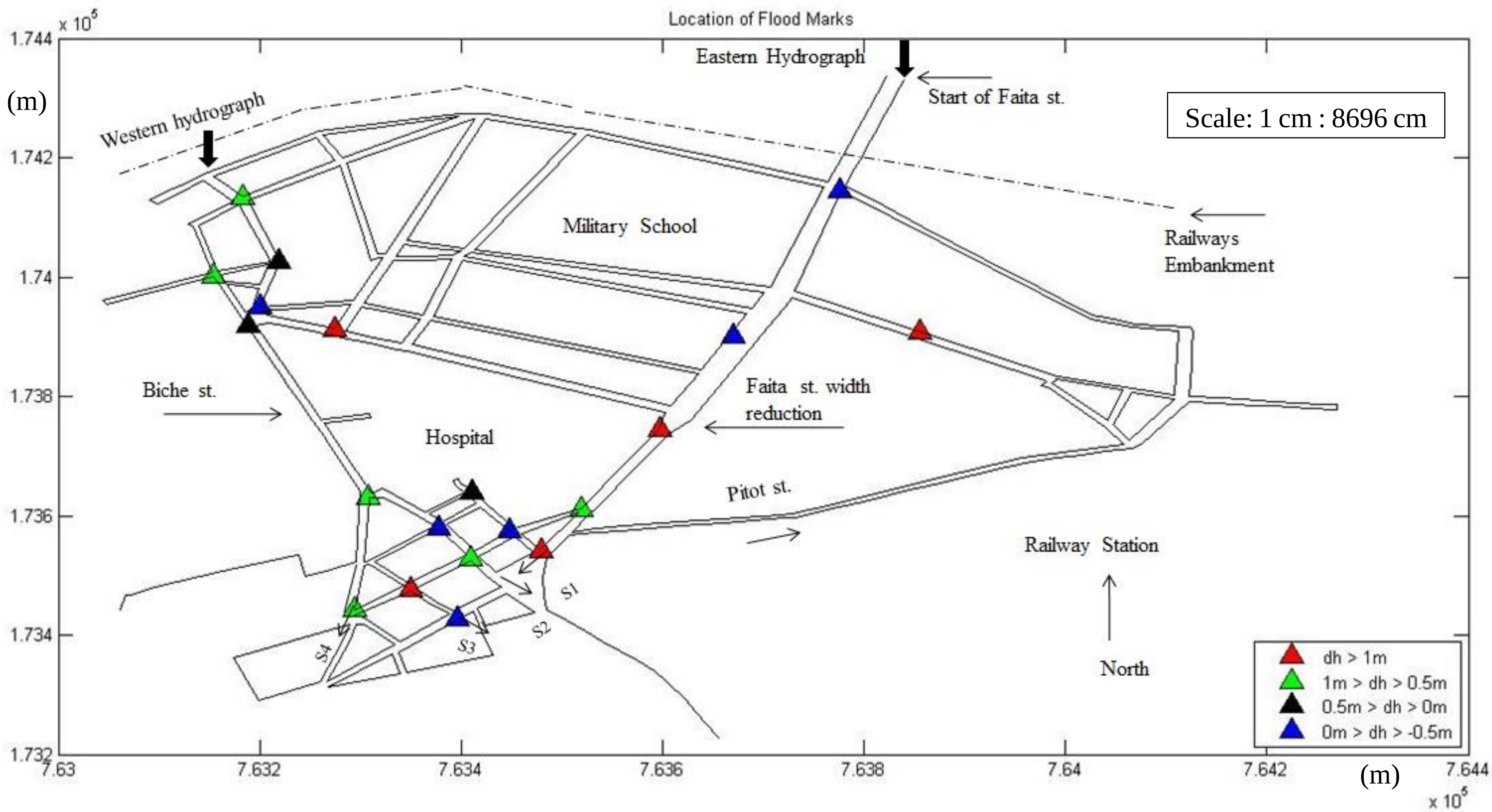


Figure 4.16: Comparison of simulated water depths with Observed Flood marks at K_s 40 $m^{1/3}s^{-1}$

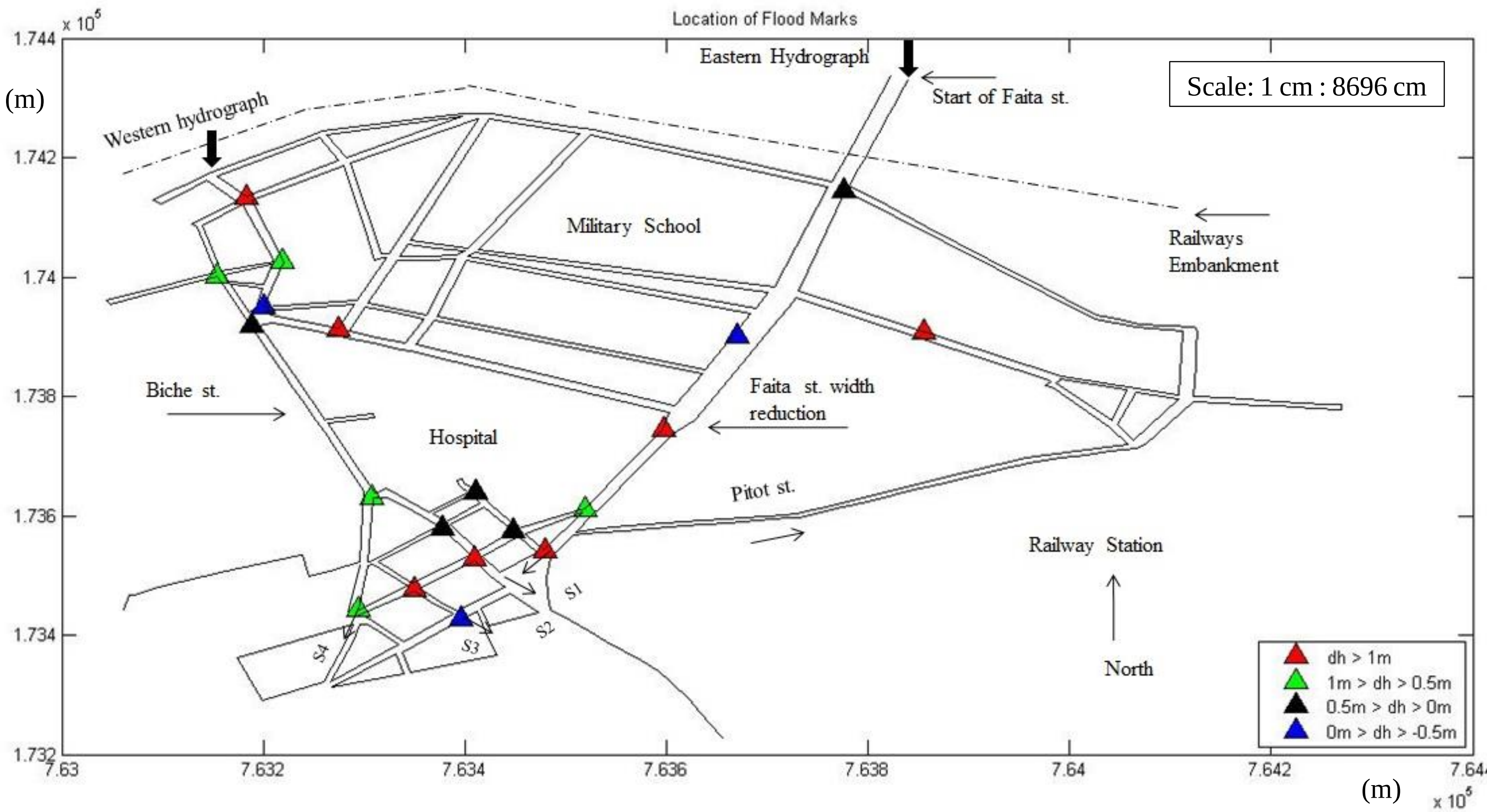


Figure 4.17: Comparison of simulated water depths with Observed Flood marks at K_s 30 $m^{1/3}s^{-1}$

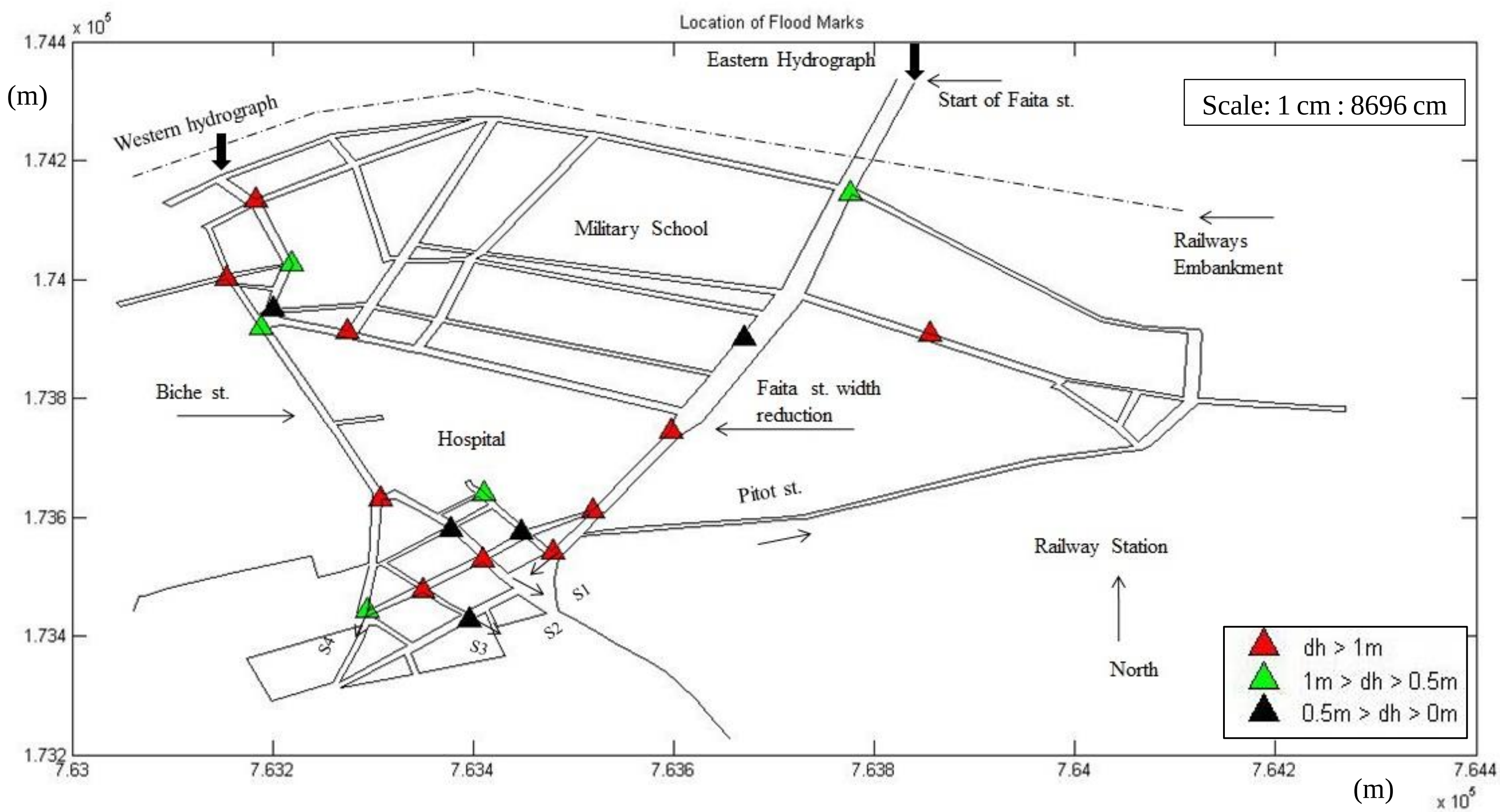


Figure 4.18: Comparison of simulated water depths with Observed Flood marks at K_s 20 $m^{1/3}s^{-1}$

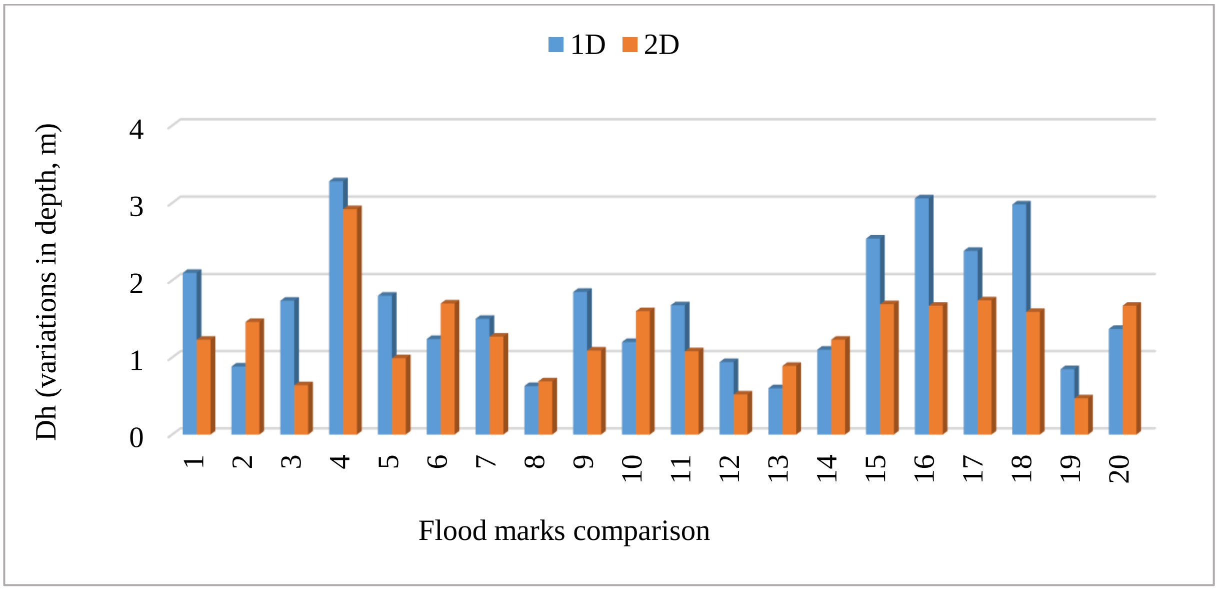


Figure 4.19: Flood marks comparison at Strickler's Coefficinet of 40 ($m^{1/3}s^{-1}$)

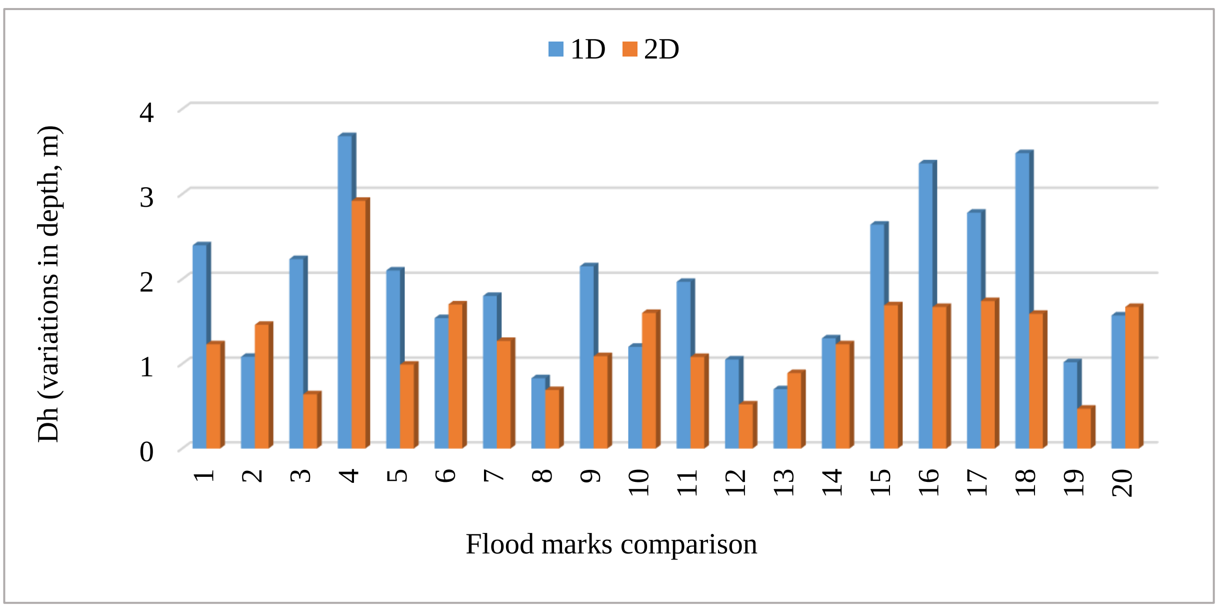


Figure 4.20: Flood marks comparison at Strickler's Coefficinet of 30 ($m^{1/3}s^{-1}$)

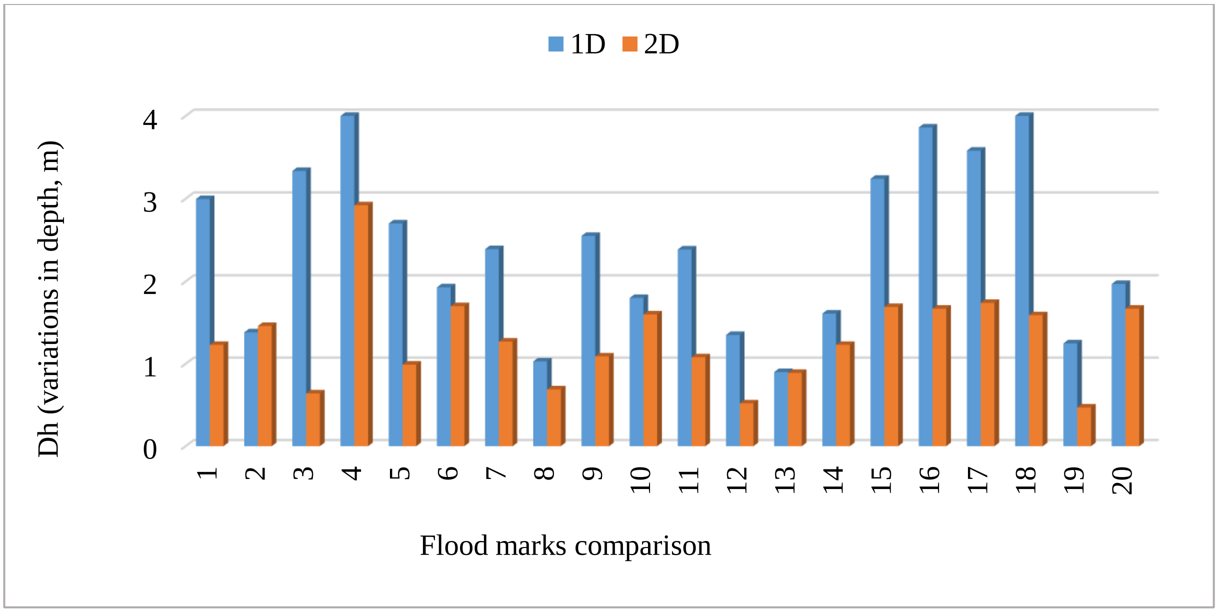


Figure 4.21: Flood marks comparison at Strickler's Coefficinet of 20 ($m^{1/3}s^{-1}$)

4.4 DISCUSSION

Above Figures show the variation in water levels with change of Boundary conditions and roughness coefficients. In the Following Table 4.1 statistical analysis shows the results of above simulation.

Table 4.1: RMSE values for 20 observation points for 1-D and 2-D

	RMSE (2-D) K_s=40 (m)	RMSE (1-D) K_s=40 (m)	RMSE (1-D) K_s=30 (m)	RMSE (1-D) K_s=20 (m)
Base case1	0.41	**0.49**	0.57	0.63
Base case 2		**0.60**	0.68	0.91
Base case 3		**0.66**	0.75	0.93

Table 4.1 shows that the Base case 1 at Strickler's coefficient 40 gives much close results to 2-D (Mignot et al., 2006). Accuracy keeps decreasing with increase in roughness factor. Similarly other boundary conditions, in which the effect of cutting the model is not taken in consideration give poor results. The reasons behind the poor results from base case 2 and base case 3 are that

in both cases the flow is free to outflow from outlets whereas in base case 1 we tried to create the same water depths which will be there if the model is complete and those depths are taken from 2-D model results. Also there are some streets in which very small amount of water flows in and make the street act as reservoir. These kind of streets are neglected as the BASEMENT is unable to model such scenarios. Also the geometry of the study area is complex and there are curvatures in the geometry but due to 1-D modelling Channels/streets are taken as straight. Which can be a factor in variations of water levels. Since the study area is urban area one value of Strickler's coefficient is used for the whole model.

Many cities have geometry in the form of network of streets meeting at crossings. It is quite laborious to model the vast area using 2-D modelling. Hence 1-D model is more suitable to these conditions. 1-D requires less data and are computationally efficient. Hence more flooding scenarios can be analyze in less time. 1-D model suffers from inadequacy at large crossroads and parking where the flow is mostly 2-D or 3-D so the complex flow interactions and head losses are poorly represented by a 1-D model and more detailed topography is needed to refine the results so 2-D models should be used at larger spaces.

In BASEMENT we are provided with 3 junction models but junctions having more than 3 streets are problematic to model. Hence we have to improvise. To improve the performance of 1-D model, 2-D model should also be utilized in conjunction. Large spaces should be modeled in 2-D which can improve the accuracy. For junction having more than 3 streets proper junction should be added to 1-D model.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

A historic urban flood of intense magnitude was simulated by using a 1-D SV equation model of street-link node type model. The main advantage of the simulation is small execution time compared to more elaborate 2-D models.

An analysis of simulated water levels with observed flood marks was carried out and RMSE was found to be 0.49m, while the same value for 2-D model was found to be 0.41m. It is therefore, concluded, that 1-D models of an urban downtown has a, more or less, comparable accuracy as more sophisticated 2-D models. The former models demand for less resources as far as data acquisition is concerned and require less time.

From the analysis it can conclude that

- 1-D study results for the Reference case are in close agreement to 2-D results and for all simulation reference case results give minimum error comparing to observed flood marks.
- When comparing the roughness factor the Strickler roughness factor of 40 give better results because when we decrease the Strickler roughness factor the model give overestimated results.
- The 1-D model can be made more effective if more comprehensive library of flow distribution models representing the junction are made available. This would allow the user to select the appropriate junction model based on the no. of participating streets, shape and roughness of the junction.
- Roughness values should be related to the type of the use: roads, streets, parks, lawns. Different roughness factor should be used for different classes instead of using single roughness factor for whole domain.

5.2 RECOMMENDATIONS

1-D saint venant equations have been used to simulate the flood event through an urban area. During this simulation obstacles like cars, bus stops etc. and building which were acting like storage to surface water. They were neglected in these simulations. To get more accurate local results these parameter should be included in these simulations(Haider et al., 2003; Paquier et al., 2003). Following are the recommendations which should be considered when simulating a flood event in dense urban area.

- BASEMENT should have a functionality to model the junction with fours streets/channels which it is lacking currently and can model with three streets/channels.
- At some streets the water gets stagnant. There is no provision in BASEMENT to model these domains which should be fixed.
- Data collection at the downstream is also necessary to set the accurate boundary conditions at the downstream.
- Flow in underground sewers should be included for better assessment of flood volume.
- Water storage in buildings, basements and streets should also be modelled.
- Different roughness coefficient should be used considering the fixed or mobile obstacles, width of the streets and various urban features as these may lead to increment to resistance to the flow.
- Urban databases should be developed on the basis of land record automation for better management of urban areas. It would also help in flood disaster analysis.
- It is recommended that same study can be carried out using 1-D/2-D coupling where large spaces can be modeled in 2-D and streets/channels in 1-D.
- Scope of study should be increased to cover the whole area of Richelieu.
- This same approach can be used to other basins of Pakistan which are prone to potential flood risk. Minawali, Dera Ghazi Khan, Rajanpur and Nowshera are some of them.

REFERENCES

Ahmad, B., Muhammad, S., Butt, M. J., and Dahri, Z. H. (2010). Hydrological modelling and flood hazard mapping of Nullah Lai. *Proc Pak Acad Sci, 47*(4), 215-226.

Alcrudo, F. (2004). Mathematical modelling techniques for flood propagation in urban areas. *Project report: IMPACT Project*.

Allan, R. P., and Soden, B. J. (2008). Atmospheric Warming and the Amplification of Precipitation Extremes. *Science, 321*(5895), 1481-1484. doi: 10.1126/science.1160787

Ballesteros, J., Bodoque, J., Díez-Herrero, A., Sanchez-Silva, M., and Stoffel, M. (2011). Calibration of floodplain roughness and estimation of flood discharge based on tree-ring evidence and hydraulic modelling. *Journal of Hydrology, 403*(1), 103-115.

Beard, J. B., and Green, R. L. (1994). The role of turfgrasses in environmental protection and their benefits to humans. *Journal of Environmental Quality, 23*(3), 452-460.

Boonya-aroonnet, S., Weesakul, S., and Mark, O. (2002). Modeling of Urban Flooding in Bangkok. 1-14. doi: 10.1061/40644(2002)274

Brody, S. D., Zahran, S., Highfield, W. E., Grover, H., and Vedlitz, A. (2008). Identifying the impact of the built environment on flood damage in Texas. *Disasters, 32*(1), 1-18.

Calianno, M., Ruin, I., and Gourley, J. J. (2013). Supplementing flash flood reports with impact classifications. *Journal of Hydrology, 477,* 1-16. doi: 10.1016/j.jhydrol.2012.09.036

Chaudhry, M. H. (2007). *Open-channel flow*: Springer Science & Business Media.

Cheng, J., Yang, K., and Xu, Q. (2008). Rainfall-runoff storage-infiltration effect of LUCC in highly urbanized region on a catchment’s scale: Shanghai urban green space system as an example. *Acta Ecologica Sinica, 28*(7), 2972-2980.

Clements, B. (2009). *Disasters and public health: planning and response*: Butterworth-Heinemann.

Cunge, J. A., Holly, F. M., and Verwey, A. (1980). Practical aspects of computational river hydraulics.

Duclos, P., Vidonne, O., Beuf, P., Perray, P., and Stoebner, A. (1991). Flash flood disaster-nîmes, France, 1988. *European journal of epidemiology, 7*(4), 365-371.

Dunnett, N., and Kingsbury, N. (2004). *Planting green roofs and living walls* (Vol. 254): Timber Press Portland, OR.

El Kadi Abderrezzak, K., Paquier, A., and Mignot, E. (2008). Modelling flash flood propagation in urban areas using a two-dimensional numerical model. *Natural Hazards, 50*(3), 433-460. doi: 10.1007/s11069-008-9300-0

Fleury, P., Maréchal, J.-C., and Ladouche, B. (2013). Karst flash-flood forecasting in the city of Nîmes (southern France). *Engineering Geology, 164*, 26-35.

Gejadze, I. Y., and Monnier, J. (2007). On a 2D 'zoom'for the 1D shallow water model: Coupling and data assimilation. *Computer methods in applied mechanics and engineering, 196*(45), 4628-4643.

Ghostine, R., Hoteit, I., Vazquez, J., Terfous, A., Ghenaim, A., and Mose, R. (2014). Comparison between a coupled 1D-2D model and a fully 2D model for supercritical flow simulation in crossroads. *Journal of Hydraulic Research, 53*(2), 274-281. doi: 10.1080/00221686.2014.974081

González-Sanchis, M., Murillo, J., Latorre, B., Comín, F., and García-Navarro, P. (2012). Transient two-dimensional simulation of real flood events in a Mediterranean floodplain. *Journal of Hydraulic Engineering, 138*(7), 629-641.

Haider, S., Paquier, A., Morel, R., and Champagne, J. (2003). *Urban flood modelling using computational fluid dynamics.* Paper presented at the Proceedings of the Institution of Civil Engineers. Water and maritime engineering.

Hirsch, R., Walker, J., Day, J., and Kallio, R. (1990). The influence of man on hydrologic systems. *IN: Surface Water Hydrology. Geological Society of America, Boulder, Colorado. 1990. p 329-359, 29 fig, 7 tab, 66 ref.*

Hunter, N. M., Horritt, M. S., Bates, P. D., Wilson, M. D., and Werner, M. G. (2005). An adaptive time step solution for raster-based storage cell modelling of floodplain inundation. *Advances in Water Resources, 28*(9), 975-991.

Kadam, P., and Sen, D. (2012). Flood inundation simulation in Ajoy River using MIKE-FLOOD. *ISH Journal of Hydraulic Engineering, 18*(2), 129-141. doi: 10.1080/09715010.2012.695449

König, A., Sægrov, S., and Schilling, W. (2002). *Damage assessment for urban flooding.* Paper presented at the 9th International Conference on Urban Drainage, Portland, Oregon USA.

Kuiry, S. N., Sen, D., Bates, P. D., and Yan, D. (2011). Application of the 1d-Quasi 2d Model Tinflood for Floodplain Inundation Prediction of the River Thames. *ISH Journal of Hydraulic Engineering, 17*(1), 98-110. doi: 10.1080/09715010.2011.10515036

Leopold, L. B. (1968). *Hydrology for urban land planning: A guidebook on the hydrologic effects of urban land use*: US Government Printing Office Washington, DC, USA.

Leopold, L. B. (1994). *A View of the River*: Harvard University Press.

Leopold, L. B., and Dunne, T. (1978). Water in environmental planning. *New York, 818p*.

Lewis, W. M. (2001). *Wetlands explained: wetland science, policy, and politics in America*: Oxford University Press.

Lhomme, J., Bouvier, C., Mignot, E., and Paquier, A. (2005). One-dimensional GIS-based model compared to two-dimensional model in urban floods simulation. *10th International Conference on Urban Drainage, Copenhagen/Denmark,*.

Mahmood Siddiqui, Q. T., Hashmi, H. N., Ghumman, A. R., and ur Rehman Mughal, H. (2011). Flood Inundation Modeling for a Watershed in the Pothowar Region of Pakistan. *Arabian Journal for Science and Engineering, 36*(7), 1203-1220. doi: 10.1007/s13369-011-0112-2

Mark, O., Weesakul, S., Apirumanekul, C., Aroonnet, S. B., and Djordjević, S. (2004). Potential and limitations of 1D modelling of urban flooding. *Journal of Hydrology, 299*(3-4), 284-299. doi: 10.1016/s0022-1694(04)00373-7

Mignot, E., Paquier, A., and Haider, S. (2006). Modeling floods in a dense urban area using 2D shallow water equations. *Journal of Hydrology, 327*(1-2), 186-199. doi: 10.1016/j.jhydrol.2005.11.026

Mikdad, O., and Zhang, B. The hydrological risk of urban rainfall events A surface flow simulation tool.

Mitsch, W. J., and Gosselink, J. G. (2000). The value of wetlands: importance of scale and landscape setting. *Ecological economics, 35*(1), 25-33.

Morales-Hernández, M., Petaccia, G., Brufau, P., and García-Navarro, P. (2016). Conservative 1D–2D coupled numerical strategies applied to river flooding: The Tiber (Rome). *Applied Mathematical Modelling, 40*(3), 2087-2105. doi: 10.1016/j.apm.2015.08.016

Neary, V., Sotiropoulos, F., and Odgaard, A. (1999). Three-dimensional numerical model of lateral-intake inflows. *Journal of Hydraulic Engineering, 125*(2), 126-140.

Paquier, A., Mignot, E., and Bazin, P.-H. (2015). From Hydraulic Modelling to Urban Flood Risk. *Procedia Engineering, 115*, 37-44. doi: 10.1016/j.proeng.2015.07.352

Paquier, A., Tanguy, J., Haider, S., and Zhang, B. (2003). Estimation des niveaux d'inondation pour une crue éclair en milieu urbain: comparaison de deux modèles hydrodynamiques sur la crue de Nîmes d'octobre 1988. *Revue des sciences de l'eau/Journal of Water Science, 16*(1), 79-102.

Perrin, C., Michel, C., and Andréassian, V. (2003). Improvement of a parsimonious model for streamflow simulation. *Journal of Hydrology, 279*(1), 275-289.

Phillips, J. V., and Tadayon, S. *Selection of Manning's roughness coefficient for natural and constructed vegetated and non-vegetated channels, and vegetation maintenance plan guidelines for vegetated channels in Central Arizona*.

Rappaport, J., and Sachs, J. D. (2003). The United States as a coastal nation. *Journal of Economic growth*, *8*(1), 5-46.

Seaburn, G. (1969). *Effects of urban development on direct runoff to East Meadow Brook, Nassau County, Long Island, New York*: US Government Printing Office.

Seyoum, S. D., Vojinovic, Z., Price, R. K., and Weesakul, S. (2012). Coupled 1D and Noninertia 2D Flood Inundation Model for Simulation of Urban Flooding. *Journal of Hydraulic Engineering, 138*(1), 23-34. doi: 10.1061/(asce)hy.1943-7900

Subramanya, K. (2005). *Engineering Hydrology, 4e*: Tata McGraw-Hill Education.

Te Chow, V. (1959). Open channel hydraulics.

Verwey, A. (2001). *Latest developments in floodplain modelling-1D/2D integration.* Paper presented at the 6th Conference on Hydraulics in Civil Engineering: The State of Hydraulics; Proceedings.

Vetsch, D., Ehrbar, D., Peter, S., Rousselot, P., Volz, C., Vonwiller, L., et al. (2017). System manuals of BASEMENT: Version.

Weber, L. J., Schumate, E. D., and Mawer, N. (2001). Experiments on flow at a 90 open-channel junction. *Journal of Hydraulic Engineering, 127*(5), 340-350.

Yoshioka, H., Unami, K., and Fujihara, M. (2015). A dual finite volume method scheme for catastrophic flash floods in channel networks. *Applied Mathematical Modelling, 39*(1), 205-229. doi: 10.1016/j.apm.2014.05.021

Zhang, B., Xie, G., Zhang, C., and Zhang, J. (2012). The economic benefits of rainwater-runoff reduction by urban green spaces: A case study in Beijing, China. *Journal of environmental management, 100*, 65-71.

ANNEX-1

BASEMENT

INTRODUCTION

"The Basic Simulation Environment for Computation of Environmental flows and Natural Hazards simulation. This software was developed at the Laboratory of Hydraulics, Hydrology & Glaciology (VAW) of the Swiss Federal Institute of Technology (ETH) at Zurich, Switzerland."

CAPABILITIES

The software BASEMENT comprises of these functionalities(Vetsch et al., 2017)

- Modelling of Flow behavior at steady, unsteady and transcirtical state in 1-D and 2-D.
- 1-D and 2-D bedload transport modelling.
- 1-D and 2-D advection, diffusion equation for pollutants and sediment transport.
- 3-D Subsurface flow module
- Choice of selection of different solution approaches.

COMPONENTS

The simulation tool can be subdivided into three parts. The Mathematical-physical module which consists of Governing equations, The Computational Grid represents the discrete form of topography and the numerical module which is about solving the equations.

BASEMENT 1-D

Basement 1-D module also called BASEchain is based on Saint-venant Equations for unsteady 1-D flow. The assumptions made are

- Pressure distribution is hydrostatic.
- Channel is prismatic.
- Uniform velocity distribution across the channel.
- For unsteady flow steady flow resistance laws are applicable.

Flow in channel can be defined by two of its variable. To analyze the flow two of three conservation laws are needed (Vetsch et al., 2017).

$$\partial V / \partial t + g\, \partial/\partial \mathrm{x}\, (v^2/2g + \mathrm{y}) = g(\mathrm{s}_\mathrm{o} - s_f)$$

[Eq. 1]

$$\partial A / \partial t + \partial Q / \partial x = q_l$$

[Eq. 2]

Where;

g : Gravitational Acceleration (m/s^2)

Q: Rate of discharge (m^3/s)

V : Flow velocity (m/s)

S_o : Bed slope

S_f : Energy slope

Y : Water depth (m)

q_l : Sink / source term per unit length (m^2/s)

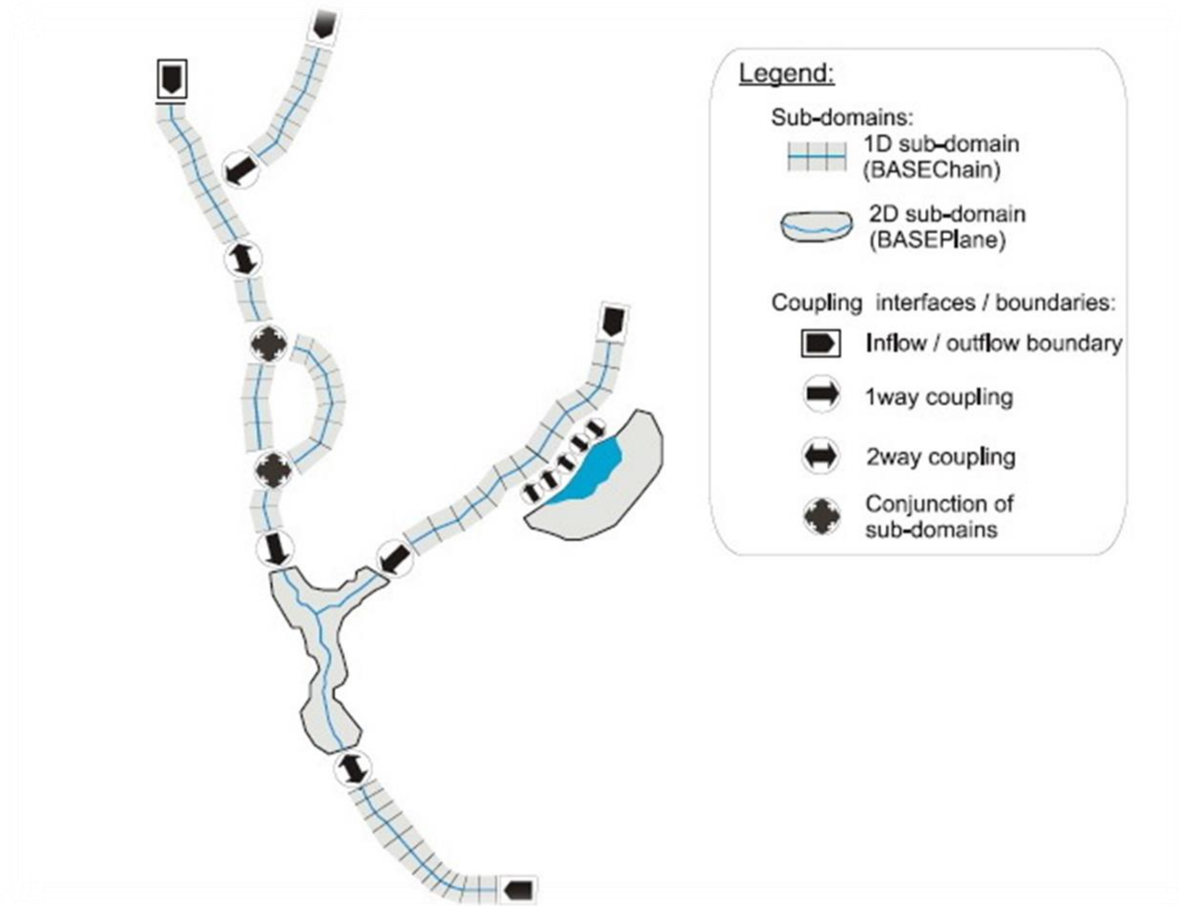
A : Cross – sectional area (m^2)

Eq. 4.1 and Eq. 4.2 are SV Momentum and Continuity Equations in 1-D respectively(Chaudhry, 2007).

Model coupling

Introduction

BASEchain is 1-D module of BASEMENT to simulate a single domain and BASEplane is 2-D module of BASEMENT to run 2-D simulation on a domain. BASEMENT has the capabilities to connect sub-domains for combined hydrodynamics simulations. These coupled simulations can be applied from simple river junction/bifurcation to integrated 1-D/2-D modelling. The coupling mechanism allow users to couple hydrodynamic simulation as well as morphological simulation. In the figure a river network showing different type of model coupling is illustrated.



River network with multiple BASEchain (1-D) and BASEplane (2-D) sub-domains and several coupling interfaces (Vetsch et al., 2017)

Few application of coupled simulation are

Instead of going for simulation for a large domain, the domain can be divided into smaller sub-domains through different interfaces of coupling. This will reduce the complexity of the problem and reduce the problem set up time. Also the extension of existing and calibrated models can be cased up with coupled simulations without redesigning the models (Vetsch, et.al, 2015).

The domain in which hydraulic structures are present can be divided in smaller sub-domains which can be coupled via hydraulic structure like weirs and gates.

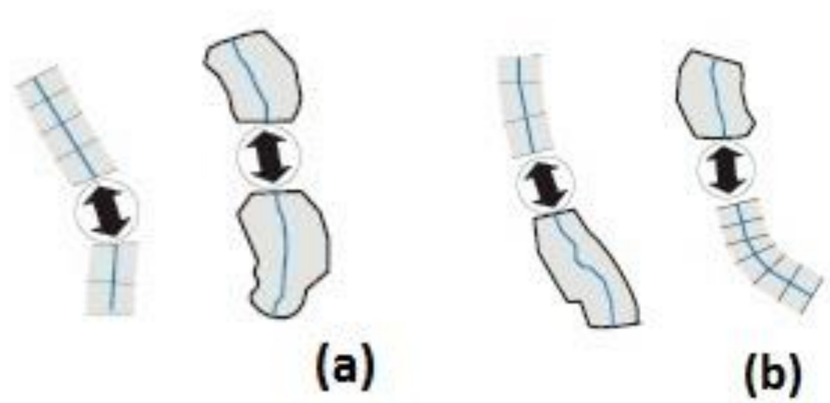
Coupled simulation can be helpful for mixed-dimensional approaches e.g. main channel can be modeled in 1-D whereas the floodplains can be modeled in 2-D. The benefits of fast 1-D simulation can be combined with detailed 2-D simulation flow characteristics. Also the required data and preparation of data can be minimized using mixed-dimensional modeling approaches.

Types of Coupling

Below different types of couplings are explained.

Sequential Coupling

Sub-domains can be combined sequentially through coupling interfaces at upstream or downstream boundaries. This type of coupling is called sequential coupling. This type of coupling can be used between sub-domains (1-D / 1-D, 2-D / 2-D) but also with mixed-dimensional like (1-D / 2-D, 2-D / 1-D) as shown in the figure.



(a) 1-D / 1-D, 2-D / 2-D (b) 1-D / 2-D, 2-D / 1-D

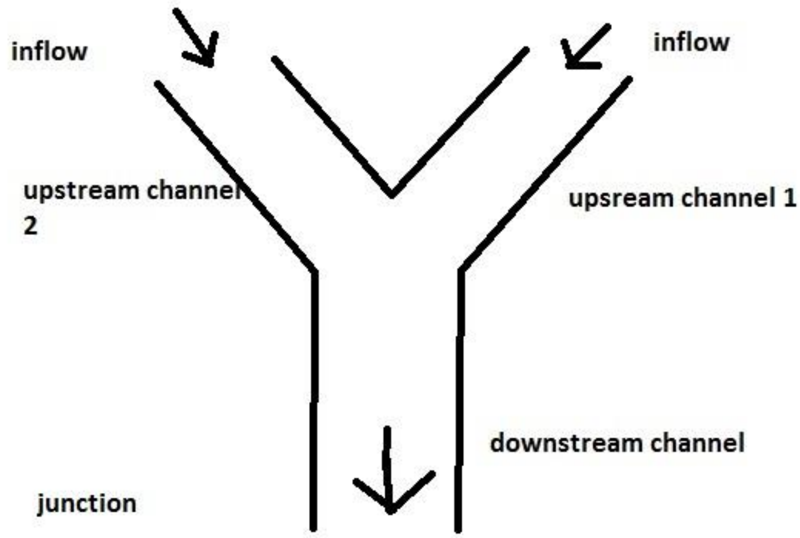
Sequential / Riemann Coupling

This type of coupling can be used to combine the sub-domains over the boundaries or external sources. For example, a downstream sub-domain input hydrograph can be combined with weir outflow of upstream sub-domain.

The Riemann coupling can also be used which sets the Riemann solver and allows flow in any direction which needs special connection between the sub-domain coupling interfaces.

Junction

Two upstream sub-domains connecting with one downstream sub-domain through coupling interfaces is called junction. BASEMENT allows simplified junction modelling of river network in 1-D as shown in the figure.

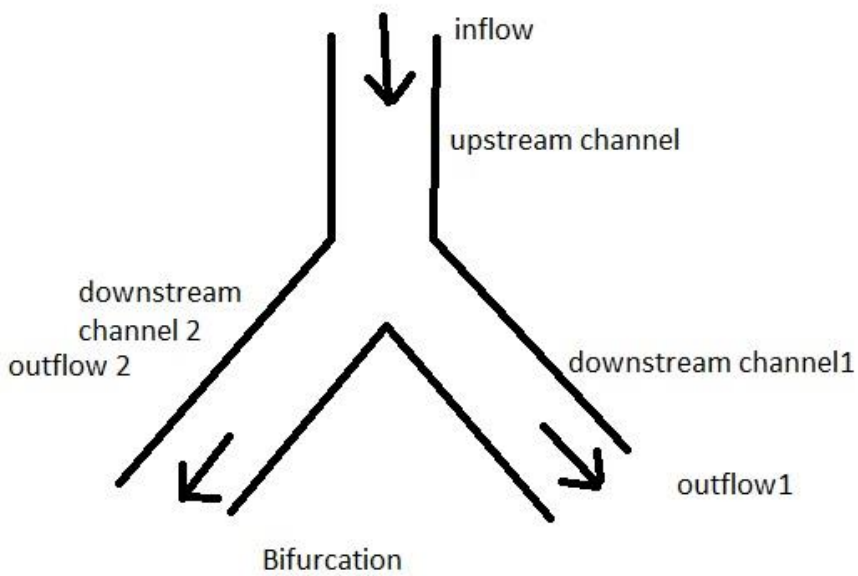


Junction

This coupling allows flow in any direction and special connection points needed at coupling interface boundaries.

Bifurcation

An upstream sub-domain connected with two downstream sub-domains via coupling interfaces is called Bifurcation. BASEMENT allows simplified modelling of Bifurcation within 1-D river network as shown in figure.



Bifurcation

ConfluenceWSE coupling can also be used in addition to bifurcation and junction in which a common water surface elevation (WSE) is established at the confluence point. This coupling allows flow in any direction and needs special connection points on coupling interfaces boundary.

Lateral Coupling

For integrated 1-D / 2-D modelling, a 1-D sub-domain can be laterally coupled with 2-D sub-domain. The coupling interface is established on the river channel between 1-D cross sections and corresponding 2-D mesh points.

COUPLING MECHANISMS

Explicit approaches have been adopted in coupling of sub-domains which means that data is exchanged explicitly between the domains at certain interval of time. This approach as compared to implicit approach is much simpler especially for mixed-dimensional sub-domains.

One Way Coupling

Coupling in which data exchange happens from upstream to downstream only is termed as one way coupling. It has the advantage that upstream sub-domain can run simulation irrespective of downstream sub-domain and flow variables can pass from upstream to downstream over certain interval of time. Due to one way coupling the information cannot travel from downstream sub-domain to upstream sub-domain. This type of coupling is not used where back water effect has to be taken into account.

Two Way Coupling

Coupling in which mutual data exchange happens in sub-domains at certain interval of time is termed as two way coupling. In Two way coupled sub-domains backwater effect can influence the upstream sub-domain. Instead of the data exchange happening sequentially from upstream sub-domain to downstream sub-domain, the sub-domains are executed simultaneously.

In a river network both techniques can be used. One way coupled sub-domains are executed from upstream to downstream whereas in two way coupled sub-domains are executed as a single subdomain within the sequence.

EXCHANGE CONDITIONS

In coupled simulation data is exchanged through coupling interfaces which are mostly boundary conditions and source terms. The following table shows the exchange of variables in coupling interfaces

Possible exchange conditions between coupling interfaces

Direction of exchange	Type of coupling	Exchange variables		
In downstream direction	Boundary condition and sources	discharge, "Q"	bed load, "$q_{b,g}$"	concentration, "C_g"
In upstream direction	Boundary conditions and sources	Water surface elevation, "Z_s"		

Assumption made for flexible and effective coupled simulation are

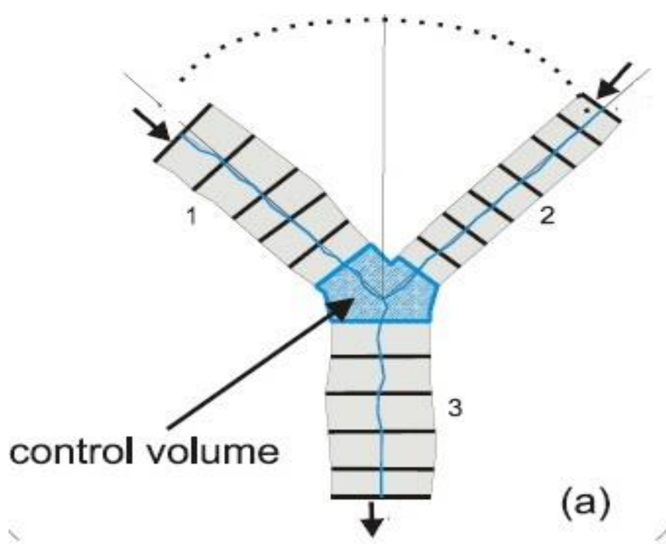
- It is assumed that flow conditions at coupling interfaces are known a priori and do not change during the simulation except only in lateral coupling.

- At coupling interfaces the 1-D cross-sections or 2-D mesh elements should be very close and have same geometries. This will reduce the possible error during the simulations at coupling interfaces and will avoid abrupt changes during simulation at coupling interfaces due to changes in the geometries.
- Flow is assumed orthogonal at coupling interfaces, i.e. the directional x and y flow components in 2-D are not exchanged separately.
- In 2-D coupling the average data is exchanged. This simplifies the coupling setup since no restriction is applied regarding geometries and number of cell at boundaries.

Exchange Conditions for River Junctions (1-D)

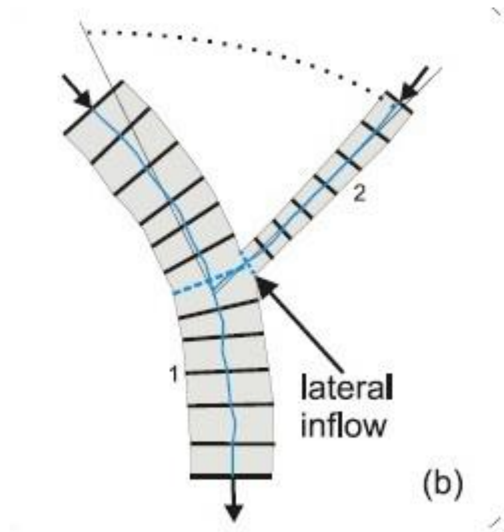
Within a river network at some locations river branches flow together or river bifurcate in many branches. Flow characteristics at the junctions/bifurcations are multidimensional. A 2-D modelling approach will be suitable for such locations but if have to be modeled in 1-D special coupling concepts are required.

There are two different approaches used for such junction modelling in BASEMENT. Not more than three branches are allowed in this modelling approach. In the first approach, three river branches meet and mutually exchange data (a). Mass and momentum conservation principles are applied over a defined control volume. Equal water surface elevation and balancing discharges are assumed in this simple approach.



Modelling of river junction via first approach

The second approach is lateral inflow of the tributary into a river at specified location (b). The discharge and sediment are passed from tributary to the main river via the source term.



Modelling of junction via second approach

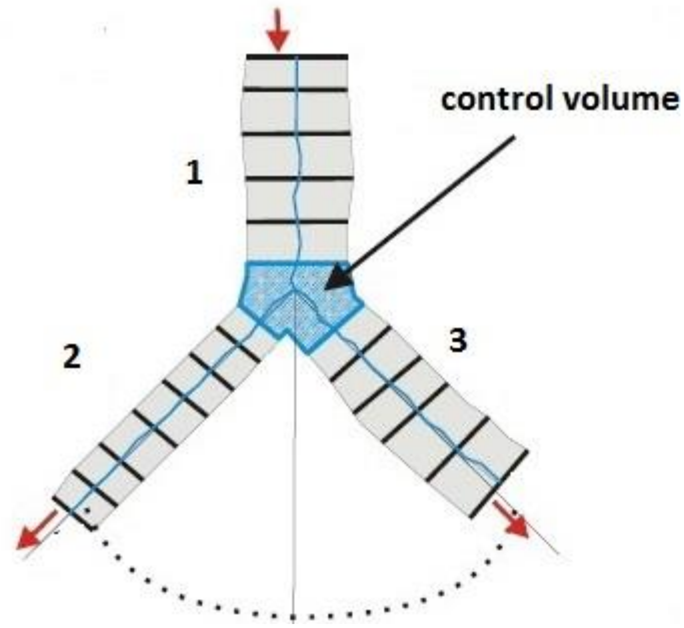
Exchange condition for modelling of a junction are

Exchange conditions for river junctions

	Exchange conditions	No. of equations
Discharge	$Q_{up1} + Q_{up2} = Q_{down}$	1
Bed load	$Q_{up1\_bed,\ g} + Q_{up2\_bed,\ g} = Q_{down\_bed,\ g}$	g = 1 … n
suspension	$Q_{up1}C_{up1,g} + Q_{up2}C_{up2,g} = Q_{down}C_{down,g}$	g = 1 … n

Exchange Conditions for Bifurcations 1-D

When a river distributes into two branches the flow and sediments should also be distributed into the two downstream branches. The distribution factor ϕ value must be chosen considering the local condition. The water surface elevation is averaged in two downstream sub-domains and then passed to upstream sub-domain.



Modelling of river bifurcation

Exchange conditions for river bifurcation is

Exchange conditions for river bifurcation

	Exchange conditions	No. of equations
Discharge	$Q_{up} = \phi\ Q_{down1} + (1-\phi)\ Q_{down2}$	1
Bed load	$Q_{up\_b,g} = \phi\ Q_{down1\_b,g} + Q\ (1-\phi) Q_{down2\_b,g}$	g = 1…. n
suspension	$Q_{up}C_g = \phi Q_{down1}C_{down1,g} + (1-\phi)Q_{down2,g}C_{down2,g}$	g = 1…. n