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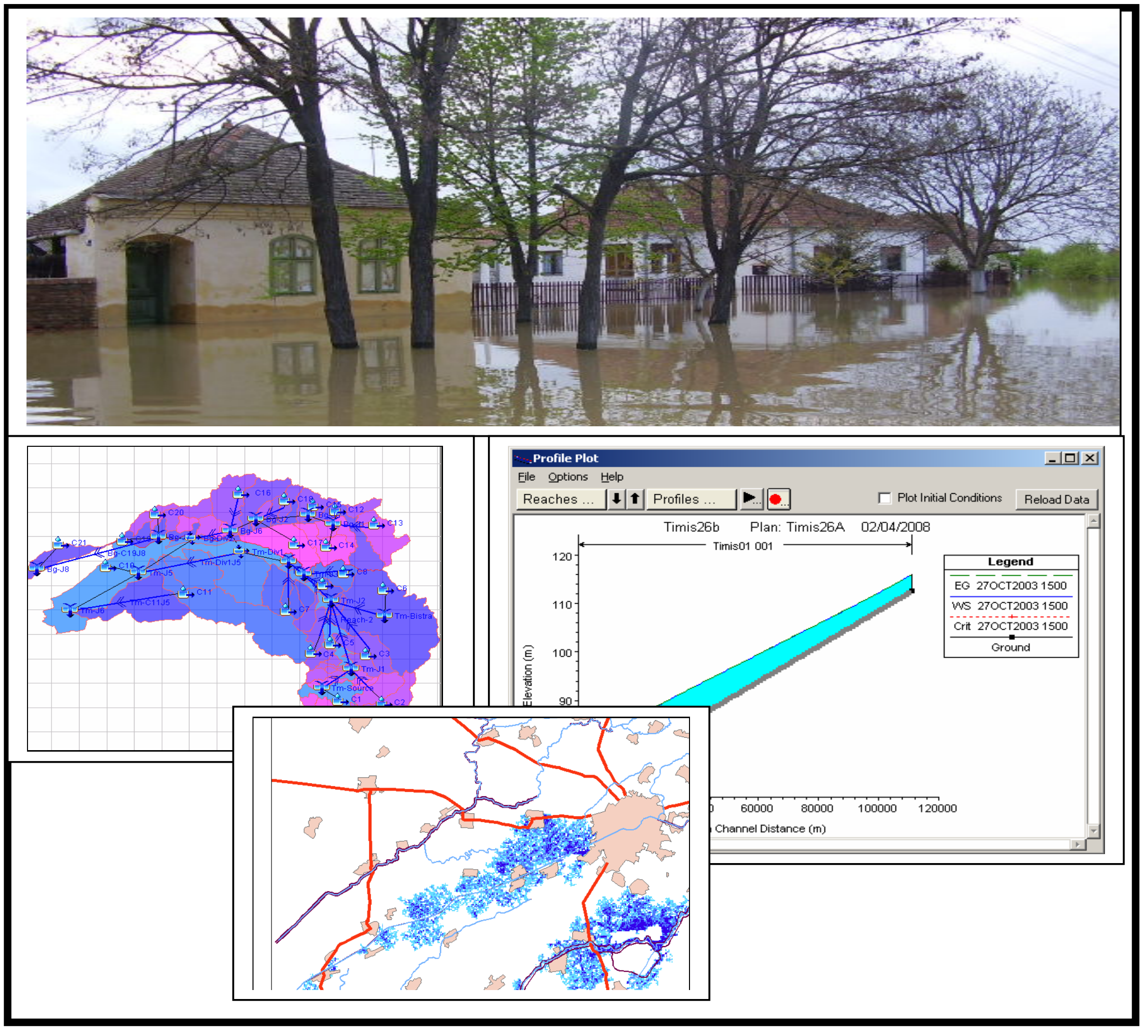

An Integrated modelling approach for flood hazard and mitigation analysis: A case study of Timis and Bega Basin, Romania.

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M.Sc. Thesis WSE-HI.08-09
April 2008

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An Integrated modelling approach for flood hazard analysis and mitigation: A case study of Timis and Bega Basins, Romania.

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This research is done for the partial fulfilment of requirements for the Master of Science degree at the UNESCO-IHE Institute for Water Education, Delft, The Netherlands

Delft
April 2008

This work is dedicated to my son
Trevor Moseti.

Abstract

Many regions throughout the world have experienced severe river flooding during the recent years. The Timis Bega Basin in Romania and Nzoia basin located in the western part of Kenya in East Africa, are no exception with many people affected every year by floods. This study approached the problem of flooding using an integrated modelling approach for flood hazard and mitigation assessment. The developed approach is based on the integration of a rainfall runoff model (HEC-HMS), 1D hydraulic model (HEC RAS), and 1D-2D SOBEK model. These models aided in analysis of flood generation from the catchment, propagation through the river and showed the floodplain inundation. GIS was used for spatial based visualization.

This study developed and tested the methodologies on Timis Bega basin. The applicability of these methodologies to Nzoia catchment was discussed on chapter 7. The stream network is schematized based on the result of a catchment analysis obtained from a Digital Elevation Model (DEM) using ArcView GIS and HEC-GeoHMS extension.

The rainfall-runoff model was calibrated for the Timis Bega catchment and correlation coefficients and the root mean square error of 0.55 and 11% for Lugoj station in Timis River and 0.72 and 2.7% for Balint station in Bega River were obtained. The calibrated rainfall runoff model results were provided as the upstream boundary conditions into the 1D hydraulic model. The discharges and levels computed by the 1D model were provided to the 1D-2D SOBEK model to compute the floodplain inundation. All the models showed sensitivity to the roughness coefficient. The rainfall runoff and the 1D hydraulic models showed that the peak magnitude is attenuated as the roughness coefficients for the catchment and main channel roughnesses respectively are increased. The 2D model showed increased velocities on the floodplain for lower roughness values, which resulted to a higher flooding extent as compared to high roughness values.

The integrated model was used to assess an extreme flood event, which was realised in 2005 in Timis Bega basin. A flood mitigation measure of intentional dyke breach was modelled to evaluate whether the flooding extents to more economic areas could be reduced. Various affected areas were mapped and the vulnerable areas identified. First the flood event was modelled and it was realized that the model predicted an area of 22,631 ha as compared to the estimated from the actual event as 25,000 ha.

A mitigation measure of intentional dyke breach was assessed and it was realized to considerably reduce the flood extents. Based on the reservoirs in the catchment and their total volume there is a potential for storage. However, when both storage and dyke breaching are considered they were realized to be more effective in reducing the flood effects. The system was used to show decision support tools that can be used for flood management in application to intentional dyke breach. The importance of forecasting was emphasized and the flood timeline for Timis was obtained to be 10 hr and 8 hr fpr Bega river in the 2005 event. The applicability of the integrated modelling approach to the Nzoia river has been discussed in chapter 7.
Keywords: Modelling, extreme events, hazard, mitigation, reservoir, dyke breach

Acknowledgement

I would like to thank my parents, Mr. and Mrs. Onyari, my sisters and brothers for their unending love and support throughout my life and education and my husband Evans for his infinite support and love, which have propelled me through this study. Much love and thanks to my son Trevor for being an easy baby to have allowed me to study with him around.

I would like to sincerely thank the Nile Basin Initiative for awarding me this scholarship and made it possible for me to study; as well i would like to appreciate the extra support that they offered me.

My sincere appreciation also goes to Dr. Popescu, my supervisor for her input, ideas, feedback, support and mentorship throughout the course of this study. I wish to thank Dr. Jonoski, and Ir. Schalk Van Andel for their mentoring and guidance provided to make this study a success. My sincere appreciation to Dr. Micha Werner for his inputs and ideas that were incorporated this study.

I am sincerely grateful to all the staff in the HydroInformatics department for their sincere support throughout my study in UNESCO-IHE. I sincerely thank our secretary Jos Bult for her kindness and genuine love to my family, and for being there for us all the time.

Finally I am thankful to everyone at UNESCO-IHE and my friends Kirsten Feijn, Salima, Lucy, Anne Lermete, and Vanessa among others who gave me support in all ways and made my stay here a memorable and valuable experience.

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List of symbols

HEC	Hydrologic Engineering Center
HMS	Hydrological Modelling System
RAS	River Analysis System
2D	Two Dimensional
1D	One Dimensional
GIS	Geographical Information System
C	Chezy coefficient
D	Diffusion coefficient
F	Darcy-Weisbach resistance coefficient
g	Gravitational acceleration
H	Water Level
n	Manning resistance coefficient
P	Perimeter
q	Lateral inflow/outflow per unit length
Q	Discharge
R	Hydraulic radius
S	Channel side slope
S_f	Friction slope
S_0	Bed slope
U	Velocity
ρ	Fluid density
τ	Reynolds stress
β	Momentum correction coefficient

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1. Introduction

1.1 Background

Floods remain one of the most frequent and devastating natural hazards worldwide. While forecast and warning systems in existence at the time of past events can significantly contribute to the reduction of losses, the potential for further prevention of avoidable losses with the technological advances through system improvements remains considerable.

Flood modeling can help to understand flood generation and identify the potential areas to be inundated, thus allowing for planning to reduce the damage caused by floods by giving early warning to communities downstream especially in floodplains which will be affected. At the same time modeling can be used to evaluate various flood mitigating measures in order to determine which alternative will be economically and environmentally feasible given the prevailing conditions. Integrated modeling has proved to be a necessity due to the complexity of the interactions and different components involved between the river and its floodplain.

Various studies have been carried out on flood modeling showing interactions between rivers and floodplains, as well as flood forecasting at key points. In Blackburn (2006) study, it showed that flood forecasting systems can combine flood routing models i.e. hydrologic that are used to obtain the flood peak by routing flood events between streamflow gauging stations, and hydraulic models to simulate flood propagation based on detailed channel geometry in order to forecast flood levels at key sites. However in areas with complex river flow conditions, two dimensional models are used for spatial hydraulic analysis.

Stewart (1999) study considered modeling of Lowland River reaches which contain complex within-reach hydrological interactions. The paper clearly shows that river and floodplain flow are the most important processes in terms of flood modelling in lowland systems, although it points out that there are often important lateral inflows and interactions between the river and the floodplain that affect the propagation of the flood wave. The paper developed a modelling approach based on a two-dimensional finite element hydraulic model of river and floodplain flow, which was linked to a series of simple hydrological models to simulate catchment runoff. The simulations showed that the model is able to predict flood hydrographs for a series of flood events, under a range of different hydrological conditions. However, more complex spatially and temporally distributed models appear to be required if predictions of the flood inundation extent are desired.

Extreme historical events can not only support flood awareness in realised scenarios but also used as reference for analysis of potential extreme cases under present conditions this was emphasized by Buchele (2006) paper. However, in reconstructing historical discharges further investigation on historical hydraulic boundary conditions is required, though due to limited historical data availability and quality major uncertainties can be expected.

This study considered to develop an integrated model based on a hydrologic model, one and two-dimensional finite element hydraulic models for river and floodplain flow simulations. The hydrologic model was used for the whole catchment for flood routing which was linked to a one dimensional hydraulic model for the main channel to propagate the flood through the river and finally it was linked also to a two dimensional hydraulic model for the floodplain analyses. GIS was loosely coupled with these models for pre and post processing.

In this study two case studies were evaluated, Timis-Bega in Romania and Nzoia catchment in Kenya. The methodologies were developed and tested for Timis Bega basin. The applicability of these methodologies on Nzoia catchment is discussed in chapter 7.

In Timis-Bega basin, the current river flood forecasting uses empirical models which show relationships of the flow at a downstream point to that at the upstream station. The gauge relations are based on flow and water elevation with the effect of lateral inflows automatically contained in the empirical relation. However, these models have a limitation in that the flood spatial and temporal variability is not known precisely.

In 2005, a flood event occurred that inundated a vast area with great damage. The flood was forecasted by the weather and hydrological models and warnings were issued in advance. There were three dyke breaks causing the area in between Bega and Timis Rivers to be inundated. However, due to the nature of the model used in the determination of the flooding, many questions came up. For instance, could the lead time for issuance of the warnings be improved? Could the areas that were inundated for the flood event and return period be identified? How could the flood effects be reduced to more economic areas?

1.2 Problem description

Flood forecasting using empirical models is handy especially in checking water levels at particular points within the river system, but is limited in identifying of flood spatial extent. Flood regulation measures initiated in Timis-Bega basins with the aim to alleviate the effect of recurrent floods include dykes, some permanent and non-permanent reservoirs, canals network, pumping stations; and a double connection between Timiş and Bega Rivers. However, these measures have not been able to control floods especially the extreme events. In 1912, 1966, 2000, and 2005 major flood events were recorded with great damages. In the recent event the water level in the dykes rose up with the flood wave overtopping and finally breaching.

Flood warnings were issued by use of weather and hydrological forecasts and actually people were evacuated from the flooding area however the lead time was short and it is possible there could have been false alarms. In some cases warnings were given one day earlier, while others on the same day.

This study tried to identify the flood spatial extent for the recent flood event in 2005 and further tried to regulate the flows and storages in the existing reservoirs so as to control flood volumes.

Renyi (1995) mentioned that there is a research gap in studies that consider both overflowing and breaching in flood protection measures. In cases where extreme rainfall events occur causing flooding, urgent decisions are required especially in dyked areas on which area is to be breached to avoid or reduce losses. This present study will assess the possibility of intentional breaching when dyked areas are flooded and there is need to breach to avoid excessive losses at more economic areas.

The rationale of this study is that flood modeling can help to understand how floods are generated and propagate downstream. Again flood hazard and vulnerability when represented by detailed spatial information can form a good basis for regional development of flood management concepts, planning, evaluation of flood protection measures, and for preparedness and prevention strategies.

1.3 Research Questions

The main questions that arise include;

1. How is the coupling hydrologic lumped conceptual model HEC-HMS, a 1D hydraulic HEC-RAS and Sobek 2D model developed?
2. What is the extent of inundation for the historical extreme event of 2005 for the Timis Bega basin?
3. To what extent can reservoirs in the Timis Bega basin reduce flooding volumes?
4. Can an integrated modeling approach be used to demonstrate decision making as a tool to reduce flooding effects?
5. How do the methodology and modelling tools apply to the Nzoia catchment in Kenya?

1.4 Objectives

The main objective is to integrate hydrologic lumped conceptual model HEC-HMS, hydraulic 1D HEC-RAS and Sobek 2D models for flood forecasting to improve on warning issuance and operation of the current system to minimize flood levels.

Specific objectives

1. To understand and develop an integrated model consisting of hydrologic lumped conceptual model HEC-HMS, hydraulic 1D HEC-RAS and Sobek 2D models.

2. To simulate inundation for the extreme flood event of 2005.

3. To assess the potential of reducing the flood magnitude through storage in the reservoir.

4. To demonstrate how an integrated modeling approach can be used as a decision making tool to reduce flooding effects.

5. To show how the methodology and modeling tools used in this study apply to the Nzoia catchment in Kenya.

2. Literature review

2.1 General Introduction

Flood forecasting can be defined as the use of real-time precipitation and streamflow data in rainfall-runoff and streamflow routing models to forecast flow rates and water levels for periods ranging from a few hours to days ahead, depending on the size of the watershed or river basin.

Flood forecasting mainly involves two approaches the flood routing component specifically hydrologic and hydraulic flood routing techniques. Hydrologic models are based on empirical storage–flow relations to approximate momentum effects and are very economical from a data perspective. Hydraulic models are deterministic and therefore require additional physical data describing the channel geometry and flow resistance characteristics.

The main disadvantage of hydrologic models is that they provide only discharge hydrographs as output, and this hydrograph output can only be generated at the gauging stations used in their calibration. In contrast, hydraulic routing models have the ability to produce output describing both water level and discharge hydrographs and between gauging stations. In addition, unlike hydrologic models, hydraulic models can be applied to dynamic problems, such as dam breaches as explained by Blackburn (2002).

Hydraulic modelling is applied in rivers and floodplains taking into account their complexities. When discharge in a river exceeds bankfull discharge, it changes from inbank to overbank flow as explained by Knight (2006). The study further explains that a significant change in the complexity of the flow behavior results due to differences in velocities between the main channel and the floodplain flows which produce strong lateral shear stresses.

Water level dynamics is the most important characteristic of the flood when considering flooding of floodplains and also in the risk analysis of the civil protection purposes. However water levels need high resolution simulations because of local variations in cross section geometry or the integration of flow with structures like bridges Rabuffetti (2005).

Blackburn (2002) determined whether it is possible to combine flood forecasting and flood levels determination using hydraulic flood routing techniques. In particular, the study was focused to find out if it was possible to route an open channel flood using natural channel geometry in short sub reaches where flood levels are required especially in cities and town sites and using approximate channel geometry between these sites. For this investigation, the 1987 summer flood event, on the Peace River was routed over approximately 800 km reach using a rectangular channel approximation, except through the towns where natural channel geometry was used. This hybrid geometry approach to flood routing was found to improve the peak stage accuracy considerably, as compared to earlier results obtained using rectangular geometry throughout. The results of this case study indicated that flood forecasting and flood level determination can be combined operationally using hydraulic flood routing techniques. The results suggested as well that channel roughness is the primary factor affecting the accuracy of the peak discharge, while channel gradient is critical to obtaining the correct timing for peak arrival. Neither peak magnitude nor time of arrival appears to be particularly sensitive to the actual channel geometry. However, actual channel geometry is required to obtain accurate water level forecasts in sub reaches of interest.

In the recent developments in flood forecasting, advances in the subject are yet to be achieved on the basis of extending forecast lead time several days into the future. Arduino (2005) paper shows that this can be achieved through recent advances in numerical weather predictions at various time ranges and spatial resolutions. Concluding that whereas flood forecasting systems combine hydrological and hydraulic models, the driving tools are environmental especially the meteorological variables like precipitation, temperature and evaporation which are updated from radar borne measurements and ground based gauge networks. This interlinked chain of models to predict water levels and discharges as well as distributed water depths and velocities, are core in flood forecasting.

The need for comprehensive, standardised and georeferenced information on floods for political and economic decision-making is explained by Barredo (2007). In this paper it is pointed out that relevant, accurate and up-to-date data is an important aspect for resource distribution, mitigation programmes, disaster monitoring and assessment. Despite this, there is a lack of spatial and thematic accurate global data for floods. In Europe, historic data on flood losses and casualties are neither comprehensive nor standardised, thus making long-term analyses at continental level difficult. Thus, a map and a catalogue of the major flood events for the last 56 years in the European Union (EU) has been developed, the study sort to alleviate the lack of homogeneous and georeferenced information on flood disasters for large periods in Europe and to give a picture of the current situation for major floods in the EU on the basis of past events and current trends.

2.2 River Flood Hydraulics

2.2.1 Introduction to River Hydraulics

Naturally rivers flow in the lowest areas in a given topography with their discharges flowing inbank and this results in identifiable river channels. However it happens that sometimes hydrological conditions vary with high rainfall and thus higher discharges occur that cause the channel to flow in an overbank condition, resulting in an increased flow area, depth and width.

In an inbank flow condition flows may be treated as if they were predominantly one-dimensional flows in the streamwise direction. However, overbank flows must be treated differently since three-dimensional processes begin to be especially important, particularly at the interaction between the main channel and the floodplain.

The flow structure of a river that is either straight or meandering channel can be represented mathematically by use of equations of fluid flow. The following section explains in brief the reduction of the mathematical equations from 3D to 2D depth averaged and to 1D flow in a river section.

2.2.2 Three dimensional flow

The three dimensional Reynolds averaged Navier Stokes equations describe the general motion of turbulent flow. Taking flow in one co-ordinate, as in the case of a river, the equation can be written as in equation 2-1 for a small cross sectional area for an open channel.

$$\underbrace{\rho\left[\frac{\partial UV}{\partial y}+\frac{\partial UW}{\partial z}\right]}_{A}=\underbrace{\rho g S_0}_{B}+\underbrace{\frac{\partial}{\partial y}\left(-\rho\overline{uv}\right)}_{C}+\underbrace{\frac{\partial}{\partial z}\left(-\rho\overline{uw}\right)}_{D} \qquad \textbf{2-1}$$

Where A is the secondary flow term, B is the Weight component term C and D are the Reynolds stresses for vertical and horizontal planes respectively and x, y, and z are the streamwise, lateral and vertical directions respectively. U,V,W are temporal mean velocity components in the {xyz} directions, ρ is fluid density, S_0 is channel bed slope, g is gravitational acceleration and $_{yx}\tau$ and $_{zx}\tau$ are Reynolds stresses on planes perpendicular to the y and z directions, respectively.

Figure 2-1 shows the essential difficulty in analysing even a simple steady uniform flow in a prismatic channel due to the various forces involved at any given point. The Navier-Stokes equations apply at a single point in the fluid such as at point J. The driving gravity force is balanced by the two Reynolds stress terms which control the vertical and lateral shearing processes arising from friction forces on the channel bed and sides and also the secondary flows traverse to the mean streamwise direction of flow with velocity components V and W.

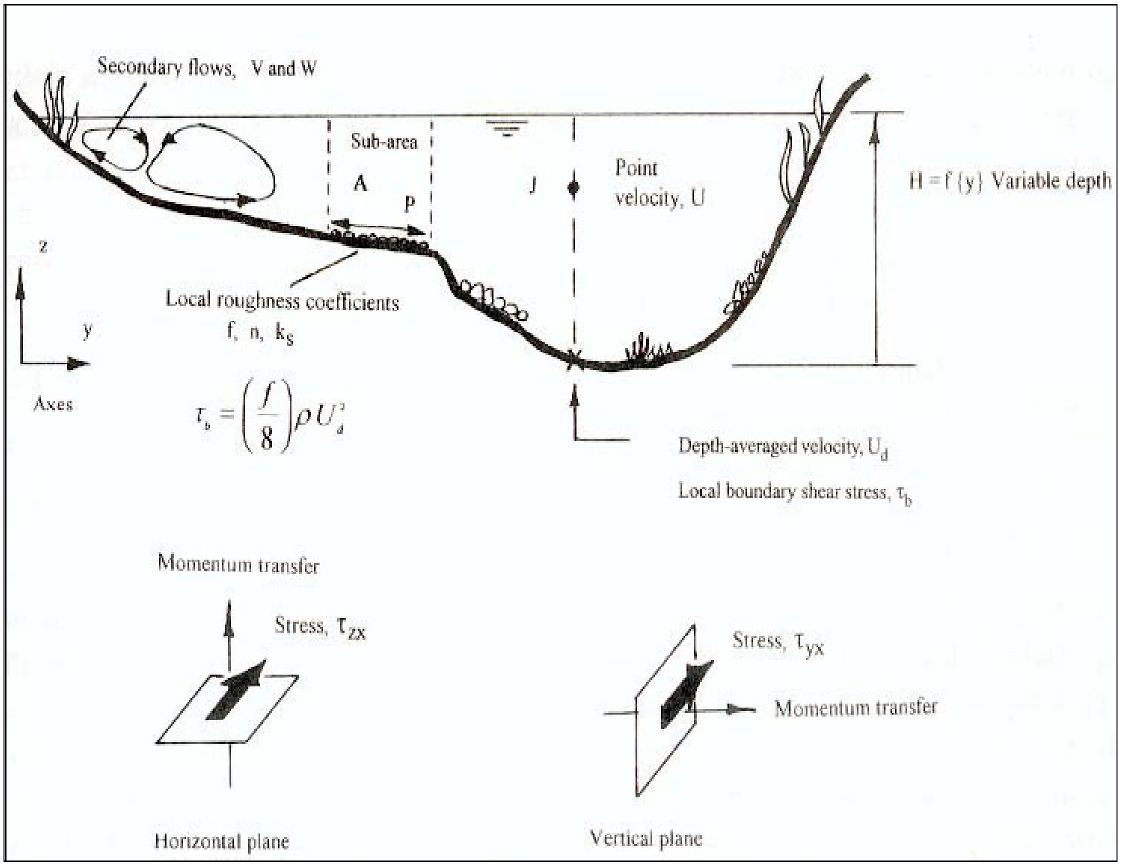

Figure 2-1 Flow in a channel (after Knight & Shiono, 1996)

2.2.3 Two dimensional flow

Usually river engineers are only concerned with the parameters at the boundaries and therefore equation 2-1 above has to be integrated over the depth, width or area. This means that the resulting 3D flow fields in figure 2-2 below has to be simplified.

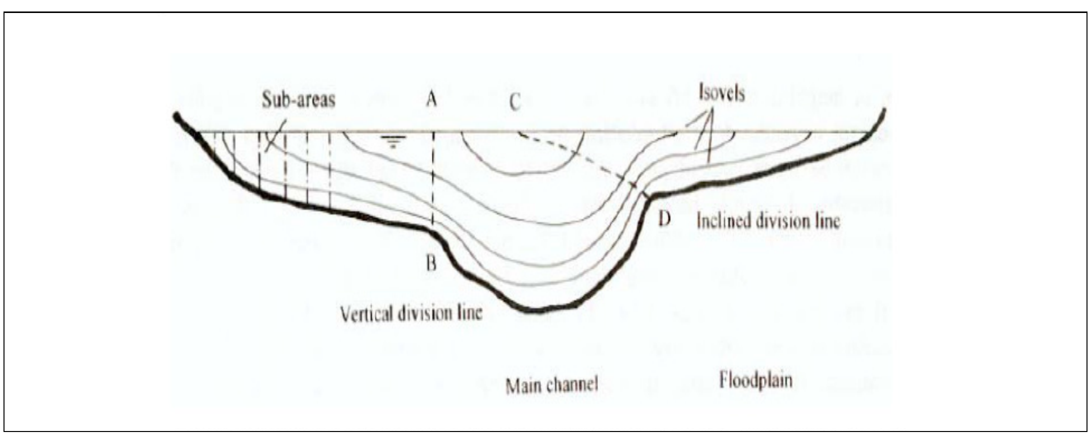

Figure 2-2 Channel subdivision methods for calculation of discharge (after Knight & Shiono, 1996)

Usually lateral distributions are of importance in rivers and due to this integration over the depth is undertaken resulting to a simplified depth-averaged form of the equation

2-2.

$$\rho g H S_0 - \tau_b \left(1 + \frac{1}{s^2}\right)^{\frac{1}{2}} + \frac{\partial}{\partial y}\left\{H \bar{\tau}_{yx}\right\} = \Gamma \qquad \textbf{2-2}$$

Where f, λ and Γ are the local friction factor, dimensionless eddy viscosity and secondary flow parameters respectively.

2.2.4 One dimensional flow

The flow of water in channels is governed by the Navier-Stokes equations. A one dimensional version of these equations are known as St. Venant equations. The resistance laws which are generally adopted for open channel flow are those based on steady flow and include the Darcy-Weisbach, Manning and Chezy formulae. These resistance laws essentially relate the conveyance capacity of the channel to the cross-sectional shape, longitudinal bed slope and resistance parameters.

The resistance to flow in a river channel can be subdivided into the following components that are partially interconnected:

- bed grain roughness,
- form resistance associated with large-scale bed undulations,
- flow resistance associated with irregular and asymmetric cross-sectional shape,
- roughness height of flexible vegetation,
- flow resistance of stiff vegetation,
- flow resistance caused by the momentum exchange between the main channel and the floodplain,
- flow resistance caused by the momentum exchange between vegetated and non vegetated section,
- sinuosity,
- large obstructions, e.g. rocks and woody debris, and
- Ice cover.

Instead of integrating over the depth, the equation 2-3 is integrated over the cross-sectional area of the channel. For instance the Manning (1857) equation is expressed as:

$$Q = \left(AR^{2/3} S_f^{1/2}\right) / n \qquad \textbf{2-3}$$

Where n is the resistance coefficient, A is the cross sectional area, Ris the hydraulic radius and S_f is the frictional slope.

2.2.5 Compound Channels Flows

A compound channel is generally visualized as a two-stage channel consisting of a main channel and a wider overbank flow channel usually referred to as a floodplain which inundates during high flows, see figure 2-3. In an attempt to compute discharge conveyed in such a compound channel it is realised that it is very complex because of the change in the resistance material from the main channel, to the floodplain this varies because in the floodplain vegetation of even buildings could be expected as compared to the main channel where boulders or even in highly maintained rivers sand and small stones could be found. Again, the lateral momentum transfer between the main channel and the floodplain does decrease the discharge in the main channel and increase the discharge on the floodplain. The irregularities in the topography which result in cross-sectional irregularities further make it difficult to compute compounded channel conveyance.

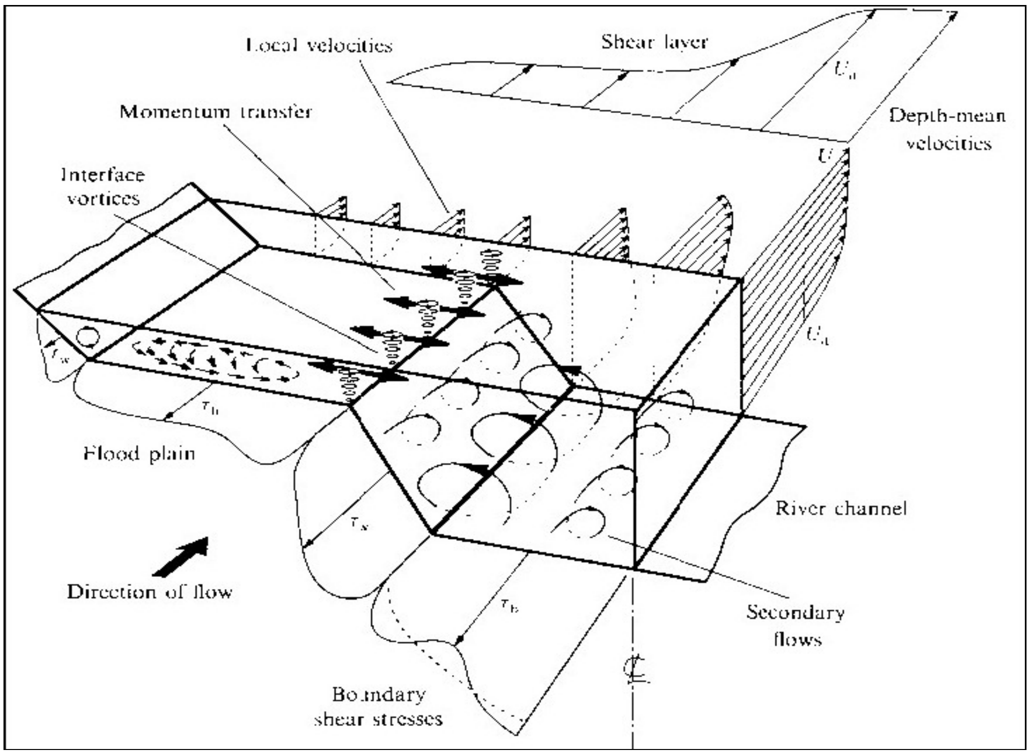

Figure 2-3 : Flow structures in a straight two-stage channel (after Kinght & Shiono, 1996).

However, various studies have been carried out in an attempt to understand the compound channel conveyance and their interrelated factors that affect the flow. In Sellin (1964) study it was realised that when a river rose above bank-full discharge the overbank flow reduced the velocities of the flow contained within the main river channel due to an intensive vortex shedding at the boundary of the main channel and the floodplain and that maximum average velocities were present in near bank-full stage.

In Pasche & Rouvé (1985) observations were made that when there is no floodplain vegetation, the slope of the bank between the main channel and the floodplain especially the width of the floodplain has a significant effect on the shear stress at the interface; but when the floodplain is vegetated, the slope has no significant influence on the shear stress, although the width of the floodplain has, especially when the vegetation is very dense.

Thornton *et al.* (2000) found out that apparent shear stress at the interface of the main channel and the floodplain cannot only be quantified as a function of the local turbulence at the interface but also it was realised that it is influenced by the velocities, flow depth and vegetation density on the main channel and floodplain.

Knight (2006) explains that when discharge in a river exceeds bankfull discharge, it changes from inbank to overbank flow, a significant change in the complexity of the flow behavior results due to differences in velocities between the main channel and the floodplain flows which produce strong lateral shear layers, which lead to generation of organized plan form vortices induced by inflection point instability.

When overbank flows occur, there are major changes in the river which result and require special considerations the abrupt change at the bankfull stage, major interactions between main river and floodplain flows. The proportion of flow between sub-areas, roughness differences between river and floodplains i.e global, zonal and local friction factors, significant variation of resistance parameters with depth and flow regime and flood routing parameters basically the wave speed and attenuation among others.

In flood problem discharge and stage or water level are the two primary parameters. Knight (2006), shows from laboratory and field stage-discharge curves for overbank, that in general Q increases with depth H, but once bankfull is reached under certain circumstances there is an actual reduction in Q despite a larger flow area, Q increases significantly due to increased flow area, with the slope of the h versus Q curve decreasing as the width of the floodplain increases.

2.2.6 Flood Routing

Cunge, Holly & Verwey (1980) derived and showed that for unsteady one-dimensional flow in an open channel, the principles of mass and momentum conservation lead to the St. Venant equations, equation 2-4 and 2-5. The following assumptions are taken into account in developing the momentum and continuity equations:

- Velocity is constant, and the water surface is horizontal across any channel section.
- All flow is gradually varied, with hydrostatic pressure prevailing at all points in the flow. Thus vertical accelerations can be neglected.
- No lateral, secondary circulation occurs.
- Channel boundaries are fixed; erosion and deposition do not alter the shape of a channel cross section.
- Water is incompressible (uniform density), resistance to flow can be described by empirical formulas, such as Manning's and Chezy's equation.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad \textbf{2-4}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\beta \frac{Q^2}{A}\right) + gA\left(\frac{\partial h}{\partial x} + s_f - s_0\right) = 0 \qquad \textbf{2-5}$$

Where

Q =discharge,
A =cross sectional area of flow
q = lateral inflow/outflow per unit length.

For a momentum correction coefficient, β, equal to 1.0, the momentum correction coefficient, equation may be expressed in terms of the section mean velocity, u, to give the friction slope, S_f, as in equation 2-6:

$$\underbrace{\underbrace{\overbrace{S_f = S_0}^{\text{Steady uniform flow}} - \frac{\partial h}{\partial x}}_{\text{Steady non-uniform flow}} - \frac{U}{g}\frac{\partial U}{\partial x} - \frac{1}{g}\frac{\partial U}{\partial t}}_{\text{Unsteady non-uniform flow}} \qquad \textbf{2-6}$$

Flow categories can be defined according to the number of terms used in the equation above. Steady Uniform flow will imply that the weight force balances the resisting shear force applied around the boundary wetted perimeter. Under these conditions Manning or Darcy-Weisbach equations apply. In steady uniform flow s_f=s_0 combining the equation above with resistance law e.g. manning equation yields the relationship between the unsteady Q and unsteady discharge Qn as in equation 2-7:

$$\underbrace{\underbrace{\overbrace{Q = Q_n}^{\text{Kinematic wave}}\left[1 - \frac{1}{S_0}\frac{\partial h}{\partial x}\right.}_{\text{Diffusion wave}} \left. - \frac{U}{gS_0}\frac{\partial U}{\partial x} - \frac{1}{gS_0}\frac{\partial U}{\partial t}\right]^{1/2}}_{\text{Full dynamic wave}} \qquad \textbf{2-7}$$

Where, these terms are grouped to indicate different levels of flood routing model i.e. Kinematic, diffusive and fully dynamic wave.

The convective-diffusion equation

The diffusion model results from combining equation 2-4 and 2-7 to give the convective –diffusion equation, which is represented as

$$\frac{\partial Q}{\partial t}+C\frac{\partial Q}{\partial x}=D\frac{\partial^2 Q}{\partial x^2} \qquad \textbf{2-8}$$

Where
C is Kinematic wave speed and D is the diffusion coefficient given by:

$$C=\frac{1}{B}\frac{\partial Q}{\partial h} \qquad \textbf{2-9}$$

$$D=\frac{Q}{(2BS_0)} \qquad \textbf{2-10}$$

Discharge in a channel during a flood event has characteristics of a wave that translates and attenuates; however in river engineering C and D are functions of discharge Q, as shown by equation 2-9 and 2-10. The gradient of the stage discharge curve is related to the kinematic wave speed by equation 2-9 and it indicates that during a flood C will vary with Q as dQ/dh and B change with time.

2.3 Modelling Approaches

Models in hydrology cover a wide spectrum of approaches, Chow (1988) attempted to sort these modes in various categories using three key parameters; randomness (deterministic or stochastic), space (distributed or lumped), and time (steady or unsteady). Any given model will lie on any of the categories; however it can also fall in more than one category. Other classifications which consider concepts applied in the model in describing the behavior of the system to be modelled include; black box, physically based and conceptual models, See figure 2-4.

Black box models: These are models in which a relation is established between the inputs and outputs of system, usually data driven modelling. In setting up these models, a large amount of inputs and known outputs are required to establish a reliable relationship. This relationship usually does not reflect the physics between inputs and outputs.
Physically based models: Physically based models are complex models based on established physical principles, as given in appropriate assumptions and laws.

Conceptual models: Conceptual models offer a practical compromise between physically based and black-box models. They rely on empirical descriptions of the processes considered as dominant. Generally these models are applied either as lumped models or semi-distributed. Vast amounts of distributed attribute data as in the physically based models are not necessary, but a drawback is that the models rely heavily on calibration with a suitable set of observed response data required.

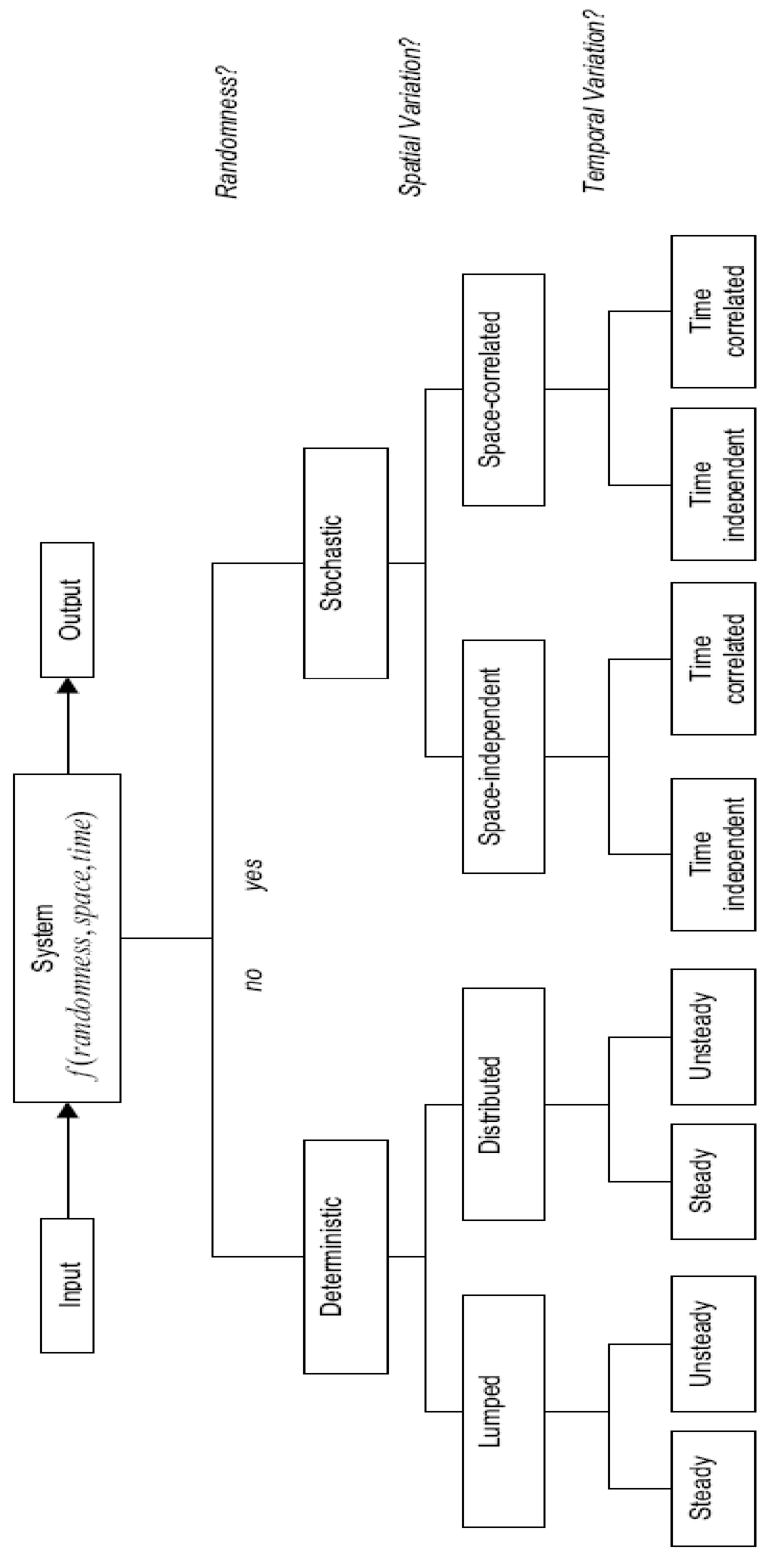

Figure 2-4 : Taxonomy of hydrological models (after Chow et al. 1988)

2.4 Modelling Tools Used in this Study.

2.4.1 Data storage using HEC-DSSvue

Data Handling

Hydrologic Engineering Center -Data Storage System,HEC-DSS is used to manage time series and tabular data. This system is used for storing a variety of data in a standardized format and improving efficiency in hydrologic engineering. HEC-HMS and HEC-RAS can access data from a DSS file when it is properly referenced. Each data stored in the DSS file is stored with unique six part pathname which include; river basin, location of gage identifier, data type, starting date, time interval of data and user defined descriptor of data. Using these parts it is easier for the model to query and manage data.

2.4.2 Hydrological modelling using HEC-HMS

Introduction to hydrological modelling

In water resource management, models have proved to be a vital tool in relating rainfall to the obtained discharges in various catchments. Attempts have been made to analyze historical rainfall, infiltration, evaporation and streamflow data in developing predictive relationships. However, transformation of rainfall to runoff is complex as it involves many processes.

When rainfall exceeds the infiltration rate at the surface, excess water begins to accumulate as surface storage in small depressions governed by surface topography, as they fill, overland or sheet flow may begin to occur and the flow concentrates into small rivulets channels which the flow into streams. Contributions to a stream can come from shallow subsurface via interflow of baseflow and contribute to the overall discharge hydrograph from a rainfall event.

Rainfall-runoff simulations were based on Hydrologic Engineering Center's Hydrologic Modeling System HEC-HMS version 3.1.0, which is a deterministic, conceptual, lumped model that is designed to simulate the precipitation–runoff processes of single reach or dendritic watershed systems representing the land phase of the hydrologic cycle. HEC-HMS is developed by the US Army Corps of Engineers. Figure 2-5 shows the various interactions in the catchment resulting into runoff.

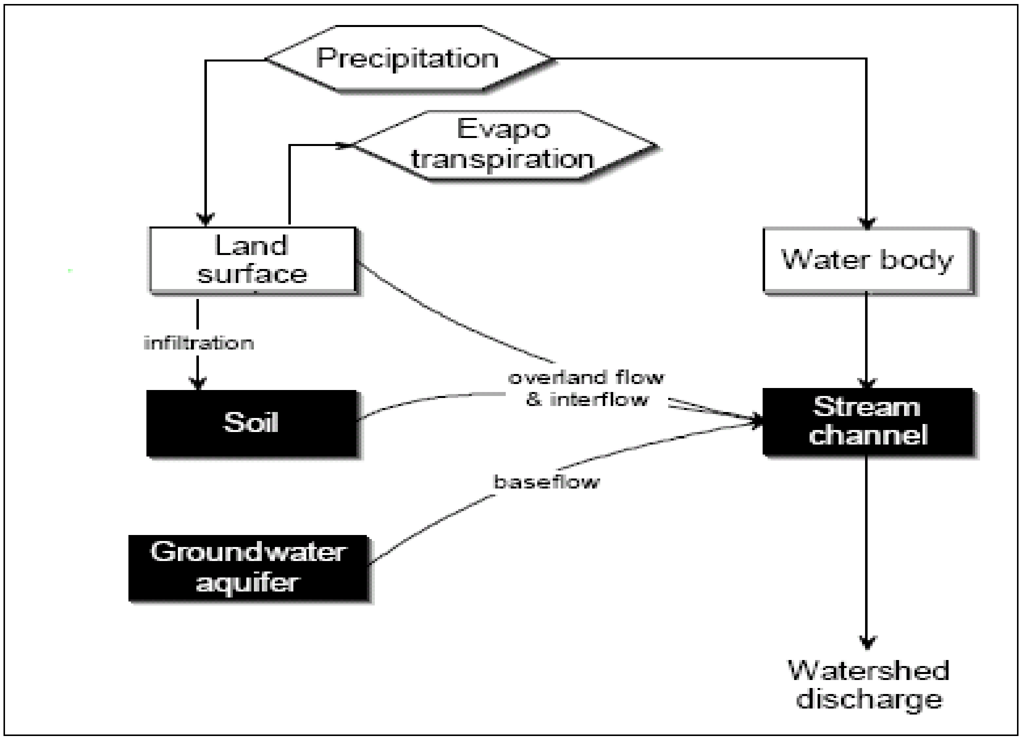

Figure 2-5 : Typical HEC-HMS representation of watershed runoff(after HEC-HMS manual)

Precipitation

Precipitation can be computed by either historical data. Historical precipitation data are useful for calibration and verification of model parameters, for real time forecasting. The computation of precipitation for each subbasin using historical gauge data together with areal weighting coefficients for each subbasin can be used.

The precipitation data must be input in the model at a constant time interval which should not necessarily be the same as the computation interval time Bedient (2002).

Mean-areal Precipitation Depth Computation

The required watershed precipitation depth can be inferred from the depths at the watershed gages using an averaging scheme given in equation 2-11below:

$$P_{MAP} = \frac{\sum_i (W_i \sum_t P_i(t))}{\sum_i W_i} \qquad \textbf{2-11}$$

Where

P_{MAP} = total storm mean areal precipitation (MAP) depth over the watershed;

$p_i(t)$ = precipitation depth measured at time t at gage i;

w_i = weighting factor assigned to gage/observation i.

If gage i is not a recording device, only the quantity pi (t), the total storm precipitation at gage i, will be available and used in the computation. Common methods for determining the gage weighting factors for MAP depth computation include:

- Arithmetic mean. This method assigns a weight to each gage equal to the reciprocal of the total number of gages used for the MAP computation. Gages in or adjacent to the watershed can be selected.
- Thiessen polygon. This is an area-based weighting scheme, based upon an assumption that the precipitation depth at any point within a watershed is the same as the precipitation depth at the nearest gage in or near the watershed. Thus, it assigns a weight to each gage in proportion to the area of the watershed that is closest to that gage.
- Isohyetal. In this method contour lines of equal precipitation are estimated from the point measurements. The mean annual precipitation is estimated by finding the average precipitation depth between each pair of contours (rather than precipitation at individual gages), and weighting these depths by the fraction of total area enclosed by the pair of contours.

Evaporation and Transpiration

HEC-HMS omits any detailed accounting of evaporation and transpiration, as these are insignificant during a flood. However, with the HEC-HMS soil-moisture accounting (SMA) model, it is possible to analyze watershed response for longer precipitation records that is records that include both periods of rainfall and periods without rainfall. During periods without rainfall, the watershed moisture state continues to change, as water moves and is stored. Evaporation and transpiration are critical components of this movement.

Evaporation, as modeled in HEC-HMS, includes vaporization of water directly from the soil and vegetative surface, and transpiration through plant leaves. This volume of evaporation and transpiration combined is estimated as an average volume. The evaporation and transpiration are combined and collectively referred to as evapotranspiration (ET) in the SMA model and in the meteorological input to the program. In this input, monthly-varying ET values are specified, along with an ET coefficient. The potential ET rate for all time periods within the month is computed as the product of the monthly value and the coefficient HEC (2000).

Computing Runoff

HEC-HMS computes runoff volume by computing the volume of water that is intercepted infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation. It assumes that a directly-connected impervious surface in a watershed is that portion of the watershed for which all contributing precipitation runs off, with no infiltration, evaporation, or other volume losses, while precipitation on the pervious surfaces is subject to losses.

With each model, precipitation loss is found for each computation time interval, and is subtracted from the mean areal precipitation depth for that interval. The remaining depth is referred to as precipitation excess, and it is considered to be uniformly distributed over a watershed area, thus a volume of runoff.

HEC-HMS allows the modeler to choose between numerous infiltration loss parameterizations HEC (2000). The various methods that can be used include; initial and constant, HEC exponential, SCS curve number, gridded curve number, Holtan method, Soil Moisture Accounting and Green and Ampt.

SMA model

The SMA model conceptually can be represented as in figure 2-6 below. It represents water movement in and above the soil. The five layers include canopy interception, surface depression storage, soil, upper groundwater, and lower groundwater. The water balance component operates like a vertical stack of horinzotal layers. Evaporation occurs from the top layer at the potential rate. When rainfall exceeds the evaporation, the excess rainfall contributes to the generated runoff.

Given precipitation and potential evapotranspiration (ET), the model computes basin surface runoff, groundwater flow, losses due to ET, and deep percolation over the entire basin. As shown in the diagram 2-6 precipitation first fills canopy interception before through fall. The water at surface depressions, soil tension zone and canopy is available for evapotranspiration. From the soil there occurs infiltration and further on percolation to groundwater.

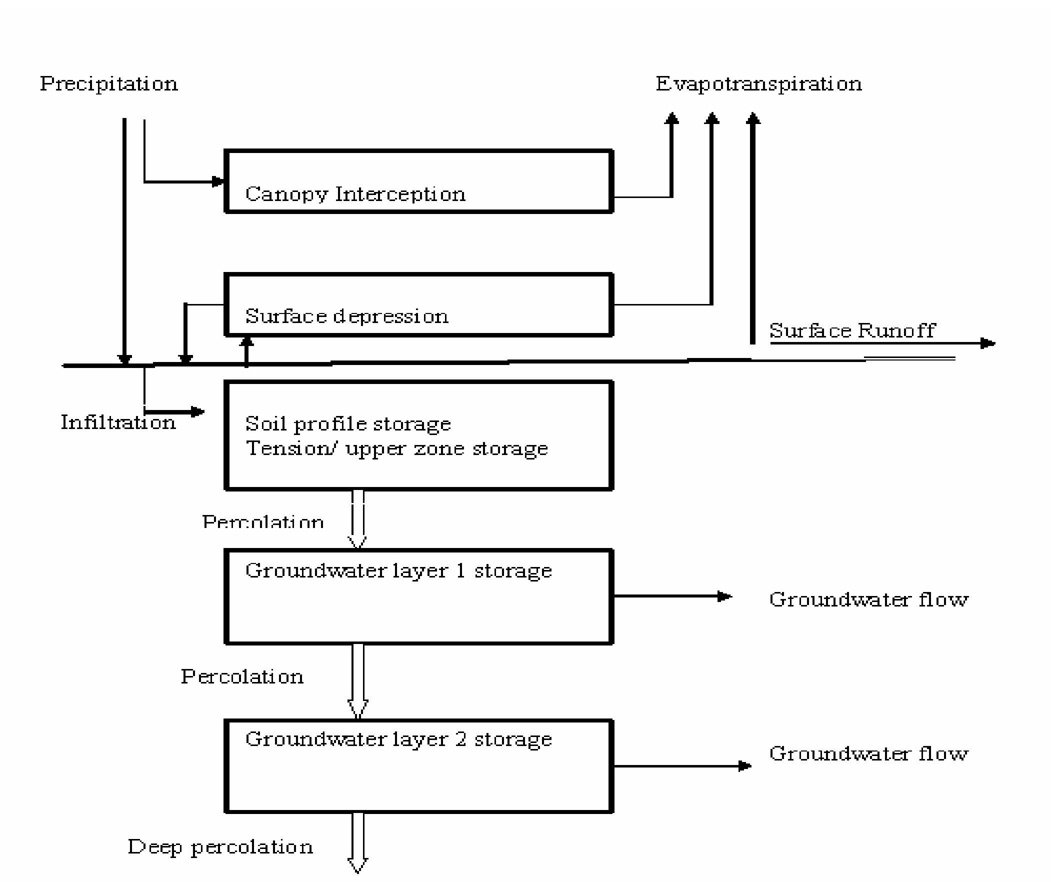

Figure 2-6 : A figure showing Soil Moisture Accounting schematisation

Kinematic wave transform

Several methods are available for surface runoff computations which include the UH methods of Clark TC & R (1945), Snyder (1983), the Soil Conservation Service (SCS, 1984, 1986), and Kinematic wave overland flow. The computation of excess precipitation to runoff will be accomplished using Kinematic wave method.

In transforming precipitation to runoff the kinematic wave was adopted, in which the watershed and its channels are conceptualized in two modules i.e. overland and channel flow. The watershed is represented as two plane surfaces (permeable and impermeable) over which water runs until it reaches the channel. The water then flows down the channel to the outlet.

Overland-flow model.

The overland model is based on the fundamental equations of open channel flow: the momentum equation and the continuity equation. Flow over the plane surfaces is primarily one-dimensional flow .The energy gradient is estimated with Manning's equation. The overland flow module conveys flow from upstream sub watersheds to the main channel thus collecting water from overland flow planes to sub-collectors to collectors to the mail channel.

Kinematic wave Reach Routing

Flood routing in HEC-HMS can be modeled using Muskingum method, Modified Puls method, Kinematic, and Muskingum –Cunge method. The St. Venant equations or the dynamic equations; the momentum equation and the continuity equation of open channel flow form the basis of the HEC-HMS routing models. The momentum equation equates the sum of gravitational force, pressure force, and friction force to the product of fluid mass and acceleration.

The kinematic wave routing method approximates the full unsteady flow equations by ignoring inertial and pressure forces. It also is assumed that the energy slope is equal to the bed slope. This method is best suited to fairly steep streams. The total length and the average slope of each reach element were entered. The Manning's n roughness coefficient was provided as the average value and was modified by calibration.

Baseflow

Baseflow takes into account normal flow through a channel or the effects of groundwater. HEC-HMS offers two methods for baseflow calculation, recession, and constant monthly. The recession method is an exponential decay function of a defined starting baseflow. For the constant monthly method a constant value of each month is given. No baseflow is an option and this is used in simple hydrologic models over short periods or highly urbanized basins with channels, baseflow can be neglected.

Applicability and limitations of HEC-HMS

The following are the applicability and limitations of HEC-HMS.

- Backwater effects: These effects can result from tidal fluctuations, significant tributary inflows, dams, bridges, culverts, and channel constrictions and restrictions. However none of the routing models that are included in HEC-HMS can simulate channel flow well if the downstream conditions have a significant impact on upstream flows. Computations move from upstream watersheds and channels to those downstream. Thus downstream conditions are not yet known when routing computations begin.

- Floodplain storage: HEC-HMS can not account for translation and attenuation of a flood wave. To analyze the transition from main channel to overbank flows, the model must account for varying conveyance between the main channel and the overbank areas
- Interaction of channel slope and hydrograph characteristics: When channel slopes lessen, assumptions made to develop many of the models included in HEC-HMS are being violated: momentum-equation terms that were omitted are more important if the channel slope is small. For example, the simplification for the Kinematic-wave model is appropriate only if the channel slope exceeds 0.002.
- Occurrence of subcritical and supercritical flow: During a flood, flow may shift between subcritical and supercritical regimes, thus this should be identified and treated separately.

HEC-HMS Calibration

Calibration uses observed hydro-meteorological data in a systematic search for parameters that yield the best fit of the computed results to the observed runoff. To compare a computed hydrograph to an observed hydrograph, HEC-HMS computes an index of the goodness-of-fit: Sum of absolute errors, Sum of squared residuals, Percent error in peak, Peak-weighted root mean square error. The search methods that are used include Univariate-gradient Search Algorithm and Nelder and Mead Algorithm.

2.4.3 1D Hydraulic modelling using HEC-RAS

River Analysis System, HEC-RAS calculates one-dimensional steady and unsteady flow. It can model a single river reach, a dendritic river system, or a full network (looped system) of stream channels (unlimited number of river reaches can be modeled).

HEC-RAS often can be used in association with HEC-HMS for determining flood flows and flood elevations in a given Catchment. It is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. Other special features include optimization of flow splits, automatic roughness calibration, and multiple-opening bridge and culvert analysis.

In flow data, boundary conditions for different flow conditions should be selected depending on whether it is subcritical, supercritical, or mixed flow regime as this determines whether to use upstream, downstream or both boundary conditions. However for flood analysis both boundary conditions are used, with the downstream boundary conditions playing an important role especially in areas with backwater effects.

The following equations 2-12 and 2-13 are solved for subcritical flow in HEC-RAS.

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = ws_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \qquad \textbf{2-12}$$

$$h_e = L\overline{S}_f + C\left(\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right) \qquad \textbf{2-13}$$

Where
WS_1 and WS_2 are downstream and upstream water surface elevations at ends of the reach respectively.
V1, V2 are mean velocities
h_e =energy head loss
L =reach length
Sf =friction slope for the reach
C contraction or expansion loss coefficient

The discharge-weighted reach length L is computed by weighting lengths in the left overbank, channel and the right overbank with their respective flows at the end of the reach. A representative friction slope in HEC-RAS is given as equation 2-14:

$$\overline{S}_f = \left(\frac{Q_1+Q_2}{K_1+K_2}\right)^2 \qquad \textbf{2-14}$$

Where, K1, K2 represent the conveyance at the beginning and at the end of the reach. Conveyance is defined from Manning's equation as given in equation 2-15:

$$K = \frac{1.49}{n} AR^{2/3} \qquad \textbf{2-15}$$

The total conveyance for the cross section is obtained by summing the conveyance from the left overbank, right overbank and the main channel.The energy or velocity coefficient α is obtained with the equation 2-16 given as:

$$\propto = \left(\frac{A_T^2}{K_T^3}\right)\left(\frac{K_{LOB}^3}{A_{LOB}^2} + \frac{K_{CH}^3}{A_{CH}^2} + \frac{K_{ROB}^3}{A_{ROB}^2}\right) \qquad \textbf{2-16}$$

The distance weighted reach length, L, is calculated as;

$$L = \frac{L_{lob}\overline{Q}_{lob} + L_{ch}\overline{Q}_{ch} + L_{rob}\overline{Q}_{rob}}{\overline{Q}_{lob} + \overline{Q}_{ch} + \overline{Q}_{rob}} \qquad \textbf{2-17}$$

T is the cross-sectional total,
LOB is the left overbank,
CH is the channel,
ROB is the right overbank.

L_{lob}, L_{rob}, L_{ch} are cross section reach lengths specified for flow in the left overbank, main channel, and right overbank respectively.

$\overline{Q}_{ch}$ $\overline{Q}_{rob}$ $\overline{Q}_{lob}$ are arithmetic average of flows between cross sections for the channel, right bank and left overbank respectively.

Cross section subdivision for conveyance calculations.

HEC-RAS subdivides the cross section based on the roughness break point i.e. the locations that the value of manning n, changes. For each subdivision the conveyance is computed based on the Manning equation 2-15. The discharge is then given by the multiplication of the conveyance with the friction slope as shown in equation 2-18.

$$Q = KS_f^{1/2} \tag{2-18}$$

The total conveyance is obtained by summing up the channel, left and right overbank subdivided conveyances.

The friction loss is computed as the product of the friction slope ($\overline{S}_f$) and the reach length (L).
The friction slope is given as;

$$S_f = \left(\frac{Q}{K}\right)^2 \tag{2-19}$$

However, alternative expressions for representation of reach friction slope are given, which can be average conveyance equation, average friction slope equation, geometric mean friction slope, and harmonic mean friction slope equation. Further, depending on the flow regime and the profile type e.g. M1, S1, equations can be selected appropriately.

Contraction and expansion loss evaluation
Contraction and expansion in HEC-RAS is computed by the equation 2-20 below.

$$h_{ce} = C\left|\frac{\alpha_1 V_1^2}{2g} - \frac{\alpha_2 V_2^2}{2g}\right| \tag{2-20}$$

Where: C is the contraction or expansion coefficient.
This occurs whenever there is a change in velocity.

Unsteady flow routing.
The unsteady flow routing is governed by the continuity and momentum laws see equation 2-4 and 2-5. When the water in the river rises and inundates the floodplain, the depths may increase such that the floodplain starts to convey also. Due to this the change of discharge with time becomes very important and then the flow has to be considered as unsteady.

Implicit Finite Difference Scheme
The four point scheme is used in solving one –dimensional unsteady flow equations. The space and derivates and function values are evaluated at an interior point, $(n+\theta)\Delta t$. The values at $(n+1)\Delta t$ enter into all terms in the equation. Implicit schemes are unconditionally stable for 0.5<θ<=1.0, for θ=0.5 it is conditionally stable and unstable θ<0.5 .Other factors that can contribute to non-stability of the solution scheme include; abrupt in changes of slope, characteristics of flood wave, hydraulic structures, and dramatic changes in the cross-sectional properties. Due to these factors, model applications should always be accompanied by sensitivity study,

to check for accuracy and stability of the solution with various time and distance intervals.

2.4.4 2D Hydraulic modelling using SOBEK

Introduction

SOBEK Rural -Overland Flow Module developed by WL | Delft Hydraulics was used for this study, which is designed to simulate the progression of flood waters and the depth of flooding in an area along a water body which can either be a river, lake or a canal. It calculates water depths, water level velocities, and volumes in the flooded area, which is represented by a two-dimensional grid. These results can be used for further analysis in studies on flood Risk analysis, Disaster management, Evacuation planning, and Flood damage assessment among others.

SOBEK-Overland Flow integrates the one-dimensional (1D) modelling package SOBEK-Flow with the two-dimensional (2D) hydrodynamic prediction package known as Delft-FLS (Delft Flooding System). Both packages have successfully been applied in many studies around the world. This chapter highlight some of the technical details of SOBEK.

SOBEK Flow Module

SOBEK Flow module is a typical 1D package for flow channel problems. It is based on one dimensional De Saint Venant flow equations i.e. momentum equation and continuity equation explained in equation 2-21 and 2-22:

Continuity equation

$$\frac{\partial Q}{\partial x}+\frac{\partial A_f}{\partial t}=q_{lat} \qquad \textbf{2-21}$$

Momentum equation

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q^2}{A_f}\right)+gA_f\frac{\partial h}{\partial x}+\frac{gQ|Q|}{C^2RA_f}-W_f\frac{\tau_{wi}}{\rho_w}=0 \qquad \textbf{2-22}$$

SOBEK Overland Flow Module

Overland Flow Module in SOBEK is a two dimensional hydrodynamic simulation package based on fully 2D shallow water Equations. The two dimensional shallow water equations are presented in the following forms, first the 2D continuity equation as shown in equation 2-23 and the 2D momentum equations in the X and Y directions as shown in equation 2-24 and 2-25.

2D continuity equation:

$$\frac{\partial \xi}{\partial t} + \frac{\partial (uh)}{\partial t} + \frac{\partial (vh)}{\partial y} = 0 \qquad \textbf{2-23}$$

Momentum equation in X-direction

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial \xi}{\partial x} + g\frac{u|V|}{C^2 h} + au|u| = 0 \qquad \textbf{2-24}$$

Momentum equation in Y-direction

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial \xi}{\partial y} + g\frac{v|V|}{C^2 h} + av|v| = 0 \qquad \textbf{2-25}$$

u = velocity in x-direction (m/s)
v = velocity in y-direction (m/s)
V = velocity: $V = u_2 + v_2$
ζ = water level above plane of reference (m)
C = Chezy coefficient (m½/s)
h = total water depth: $h = \zeta + d$ (m)
d = depth below plane of reference (m)
a = wall friction coefficient (1/m)

These equations consist of acceleration terms, the horizontal pressure gradient terms, advective terms, bottom friction terms and wall friction terms. These equations are non-linear and are a subset of the well-known shallow water equations that describe water motion for which vertical accelerations are small compared to horizontal accelerations.

As opposed to the shallow water equations, the described equations do not incorporate the turbulent stress terms, accounting for the sub grid transfer of momentum in between grid cells.

The wall friction terms have been introduced to account for the added resistance that is caused by vertical obstacles, like houses or trees. The wall friction coefficient is based on the average number and diameter of the obstacles per unit area and the average obstacle drag coefficient (C_d coefficient).

Numerical scheme

Both the SOBEK Flow and SOBEK 2D module are based on the same numerical scheme, known as Delft scheme (or Stelling scheme). It has the following features:

1. Work with complete 1D De Saint Venant equation/ 2D shallow water equations, including transient flow phenomena and backwater profiles;
2. Has an automatic drying and flooding procedure that is 100% mass conserved;
3. Uses a so-called minimum degree algorithm with an iterative simulation technique which is highly efficient in case of large networks and long time series;

4. The used procedure guarantees a solution which in certain flow conditions reduce the time step temporarily by a time step estimation procedure to avoid numerical instability;

Coupling of 1D and 2D Model

Implicit coupling of 1D and 2D schematisations is based upon the concept that both are defined on separate computational layers. The 2D layer is described on the basis of a rectangular computational grid where flow over DEM-defined topographies are adjusted accordingly especially on the floodplains where obstacles to flow such as dikes are observed.

All sub-grid conveyance objects, such as channels, including remnants of dead river branches and similar local depressions, and all kinds of hydraulic structures are described on the 1D schematisation layer. Subsequently, 1D and 2D schematisations will be linked to each other via water level compatibility at selected computational nodes, see Figure 2.7 a.

The computational domain is split into a 1D network, with general volumes of arbitrary shapes, and a 2D system with rectangular computational cells. The 1D network and 2D system are implicitly coupled and solved simultaneously based upon the momentum balance and the conservation of mass between separate computational layers.

For the momentum balance the 1D and the 2D system remain strictly separated. That means that velocities or discharges belong either to the 1D part or to the 2D part. For the conservation of mass, being a scalar quantity, the appropriate 1D and 2D volumes are combined so that they share the same water level, see Figure 2.7 b.

Both the 1D and the 2D computational layers have finite difference formulations for volume and momentum equations, based upon the staggered grid approach. In other words, the finite volume approach is applied, the momentum volumes are different from the mass volumes, and there exists no interaction between the 1D and the 2D momentum volumes. This means that vertical velocities and shear stress interaction between 1D flow and 2D flow are neglected.

For each momentum volume the following law is applied that states that the Rate of change of momentum, transport of momentum, integrated hydrostatic pressure and friction losses are equal to zero.

The interaction between the 1D and the 2D part takes place via mutual volumes. For mutual 1D/2D mass volumes the following equation 2-26 is solved:

$$\frac{dV_{i,j}(\xi)}{dt} + \Delta y\left((uh)_{i,j} - (uh)_{i-1,j}\right) - \Delta x\left((vh)_{i,j} - (vh)_{i,j-1}\right) + \sum_{l=k_{i,j}^{1}}^{l=k_{i,j_k}^{i,j}} (Q_n)_l = 0 \qquad \textbf{2-26}$$

where:
V = combined 1D/2D volume;

u = velocity in x direction;
v = velocity in y direction;
h = total water height above 2D bottom;

ζ = water level above plane of reference (the same for 1D and 2D);

Δx = 2D grid size in x (or i) direction;

Δy = 2D grid size in y (or j) direction;

Q_n = discharge in the direction normal to the mass volume faces;
i, j, l, K, L integer numbers for nodal point numbering.

$$\frac{dV_{i,j}(\xi)}{dt} + \Delta y\left((uh)_{i,j} - (uh)_{i-1,j}\right) - \Delta x\left((vh)_{i,j} - (vh)_{i,j-1}\right) + Q_{K+1} - Q_K = 0 \qquad \textbf{Equation 2-27}$$

The numerical implementation is such that in the vicinity of steep gradients proper shock conditions are being fulfilled, both for 1D and 2D volumes (Stelling et al., 1998).

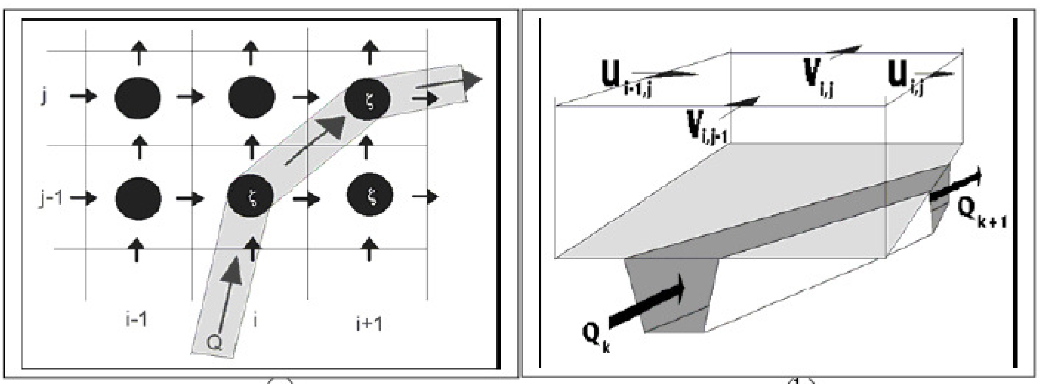

Figure 2-7 :Schematisation of SOBEK 1D-2D hydraulic model (a) COMBINED 1D-2D staggered grid (b) combined finite mass volume for 1D-2D computations

After discretisation in time by the "θ method" the velocities are eliminated by substitution of the momentum equations into the continuity equation. The resulting system is linear for purely 2D volumes, but if a 1D part is involved the equation might be non-linear with respect to the volume $V(\zeta)$. This is solved by Newton iteration. The method used for the solution is a combination of the minimum degree algorithm and the pre-conditioned conjugate gradient.

Storage

The storage that is used in the continuity equation of the SOBEK-Flow-module is related to the connection nodes and calculation pints. The storage at a node is equal to the node storage plus the storage of half of the reach segment that are connected to the node. Each calculation point has as its storage half of the reach segments on either side of the calculation point. Figure 2-8 shows side view of the storage in the nodes typical for a open channel system.

The storage above surface level (open channel systems) is normally given as a storage width. This width multiplied with the length of the reach segment gives the storage area.

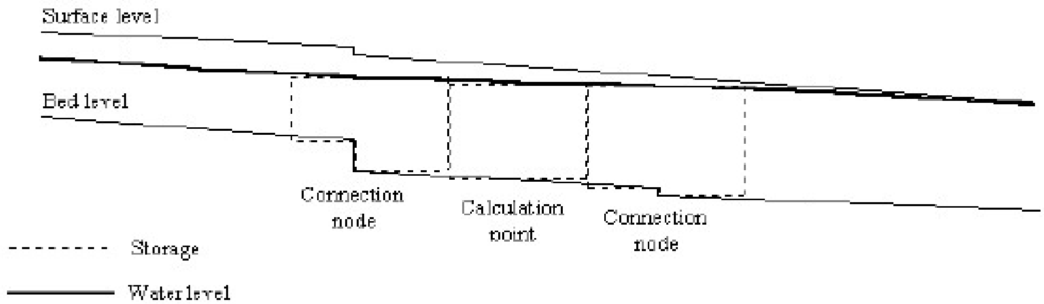

Figure 2-8 :Storage in an open channel system in SOBEK (after WL|DELFT Hydraulics)

Bed Resistance and conveyance

The SOBEK-Flow-module uses the Chezy bed friction value in solving the water flow equations. Conveyance is a quantity which represents the discharge capacity of a river for every water level. It combines the values for friction and hydraulic radius into one parameter, which is calculated internally in SOBEK to solve the hydrodynamic equations.

The formula for conveyance is written as:

$$K_i = A_i C_i \sqrt{R_i} \qquad \textbf{2-28}$$

Where:
K = conveyance of the subsection under the applying water depth and friction
i = the subsection
A = wetted area within the sub section under the applying water depth
C = Chezy friction value under the applying water depth
R = Hydraulic radius under the applying water depth

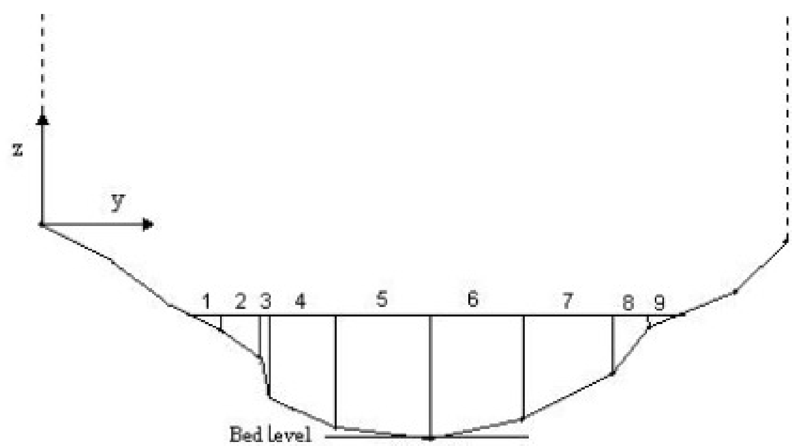

Figure 2-9 :Subsections for conveyance calculation in SOBEK (after WL|Delft Hydraulics).

Total conveyance of such cross section (see figure 2-9) is then calculated using equation 2-29:

$$K = \sum_{i=1}^{n} K_i \qquad \textbf{2-29}$$

Where:
K = the total conveyance of the cross section
K_i = the conveyance of subsection i
i = the number of a subsection (counted from y=0)
n = the number of subsections
All individual conveyances are summed up to calculate a total conveyance of the cross section for each given water level.

2.7 Efficiency Criteria Test

The evaluation of hydrologic model performance can be easily reported through comparisons of simulated and observed variables. Frequently, comparisons are made between simulated and measured stream flow at the catchment points where there are measurements at the outlet points or these comparisons they can be within the catchment as long as there is observed variables. The performance of a model needs to be evaluated so as to be able to provide a quantitative estimate of the model‘s ability to reproduce historic and future watershed behavior, to provide a means for evaluating improvements to the modeling approach through adjustment of model parameter values, model structural modifications, the inclusion of additional observational information, and representation of important spatial and temporal characteristics of the watershed and finally to compare current modeling efforts with previous study results.

The process of assessing the performance of a hydrologic model requires the hydrologist to make subjective and/or objective estimates of the ―closeness of the simulated behaviour of the model to observations (typically of stream flow) made within the watershed.
The most fundamental approach to assessing model performance in terms of behaviours is through visual inspection of the simulated and observed hydrographs. In this approach, a hydrologist may formulate subjective assessments of the model behaviour that are generally related to the systematic (e.g., over- or under prediction) and dynamic (e.g., timing, rising limb, falling limb, and base flow) behaviour of the model Efficiency criteria are defined as mathematical measures of how well a model simulation fits the available observations (Beven, 2001). In general, many efficiency criteria contain a summation of the error term (difference between the simulated and the observed variable at each time step) normalized by a measure of the variability in the observations. To avoid the cancelling of errors of opposite sign, the summation of the absolute or squared errors is often used for many efficiency criteria.
There are a number of efficiency criteria of measures, however in this study coefficient of determination were used in the evaluation of the efficiency of the model prediction.

2.7.1 Coefficient of determination r2

The coefficient of determination r2 is defined as the squared value of the coefficient of correlation according to Bravais-Pearson. It is calculated as:

$$r^2 = \left(\frac{\sum_{i=1}^{n} \left(O_i - \overline{P}\right)\left(O_i - \overline{P}\right)}{\sqrt{\sum_{i=1}^{n} \left(O_i - \overline{P}\right)^2} \sqrt{\sum_{i=1}^{n} \left(O_i - \overline{P}\right)^2}} \right)^2 \qquad \textbf{2-30}$$

Where, O is observed and P predicted values.

The term r2 can also be expressed as the squared ratio between the covariance and the multiplied standard deviations of the observed and predicted values. Therefore it estimates the combined dispersion against the single dispersion of the observed and predicted series. The range of r2 lies between 0 and 1 which describes how much of the observed dispersion is explained by the prediction. A value of zero means no correlation at all whereas a value of 1 means that the dispersion of the prediction is equal to that of the observation.

2.7.2 Root mean square error

The Root mean square error RMSE was used also in the calibration of the observed and the measured values. The formula is as shown in equation 2-31 below.

$$\text{RMSE} = \sqrt{\frac{\sum_{i=1}^{n} (y_i - \overline{y}_i)^2}{n}} \qquad \textbf{2-31}$$

3. Study Area

Timis Bega basin is located in south-west of Romania in Banat province, it lies between latitude 44°30' and 46°, and longitude 20°20' and 22°40'.The climate is temperate continental with influences from the Mediterranean basin and also under the Carpathian Mountains protection, east and north which diminish the climate influence of Eastern Europe Plain during the cold seasons. Figure 3-1 shows the

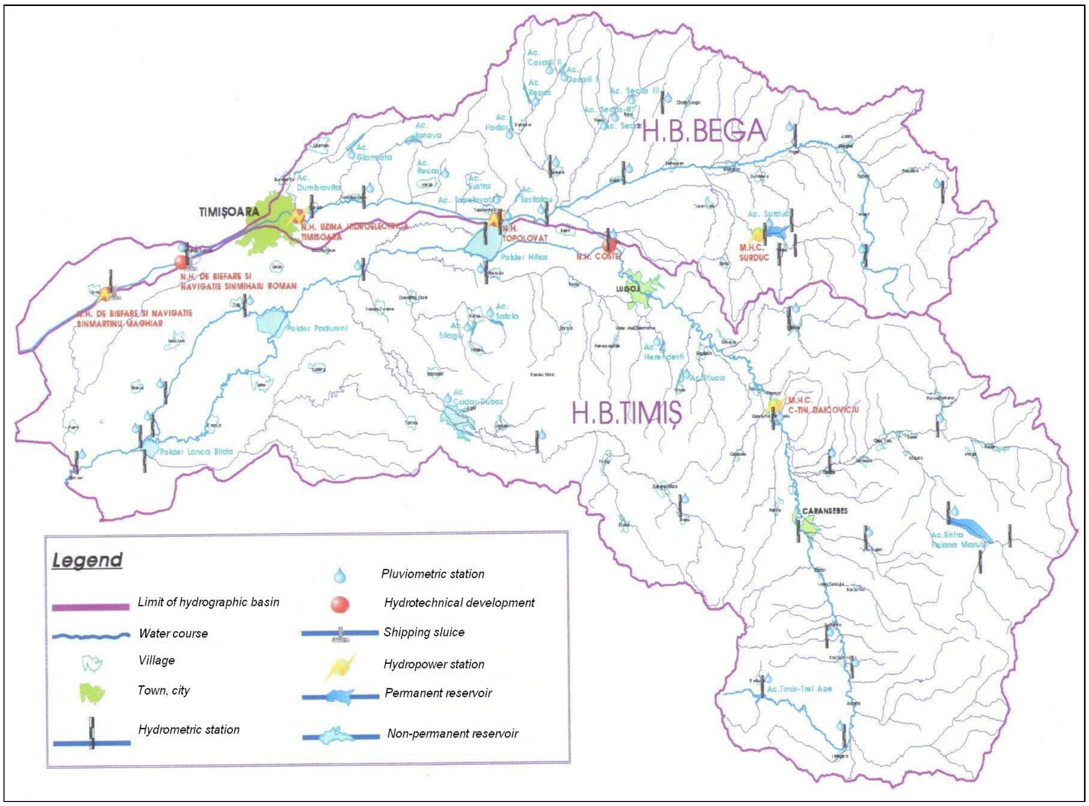

Figure 3-1 :The Timiş and Bega Rivers hydrographic basins

3.1 Climate

The climate is temperate continental with influences from the Mediterranean basin, it is of moderate humid continental type, exposed to predominant northerly cold winds in the winter and moderate westerly winds from the Atlantic in the summer. At altitudes higher than 1000 amsl, in the Banat Mountains, the intersection of the two influencing zones generates heavy snowfalls.

January, which is the coldest month average temperatures range from -4°C to 0° C. During the summer, the highest temperatures are recorded in the Danube Valley 24°C, in July. Temperatures decrease toward the high elevations in the northwest and toward the southeast, where the Black Sea exerts a moderating influence. Precipitation decreases from west to east and from the mountains to the plains, with an annual average of between 1000 mm and 1250 mm in the mountains and about 380mm in the delta.

3.2 Description of major rivers

The Banat catchment is formed by several subbasins however, Bega and Timiş basins will be considered, see fig 3-1. The main rivers in this basin Timis and Bega River discharge water in Serbia beyond Romanian borders. Due to this the Romanian authorities are restricted to flowing conditions downstream the border.

3.2.1 Bega River

The river starts at the junction headwaters of Bega Luncanilor and Bega Poieni. After starting of to the north, the river bends to the west at Coşava, finally it enters the low Banat plains. There, it begins to spill over, so the Bega canal was constructed tracking Bega Veche, which is also channeled for 97 km, as parallel waterway for 114 km, before the two being connected northeast of Zrenjanin, Serbia. The Bega canal runs through Timişoara and continues to the south-west, enters Serbia near the village of Hetin. It has a draining area of 2,878 km².

3.2.2 Timis River

The Timiş River is 359 km long rising in the Semenic Mountains, southern Carpathian Mountains, Caraş-Severin County, Romania. It flows through the Banat region and flows into the Danube near Pančevo, in northern Serbia. The drainage area covers 13,085 km² (in Romania 8,085 km², in Serbia 5,000 km²). After entering Banat, the river becomes slow and meandering, causing floods in rainy years. Especially devastating were the floods of 2005, when the villages Boka and Jaša Tomić which are in Serbia were badly damaged.

Figure 3-2 : A photo showing 2005 flood in Jaša Tomić, in Serbia.

3.3 Land use pattern

The area has population concentrated in cities, towns and villages. Among these, Timisoara city, capital of the Timis County which has 300,000 inhabitants represents the largest urban agglomeration in the west of the country. As a consequence, the protection of this center during the high waters periods as well as the ensuring of the minimum water requirements during the droughty periods represented priority objectives along time. The villages of 500 – 1000 inhabitants in the Banat Plain are situated from one and other at about 10 to 15 km. The whole plain, having cernozioms and brown soil as superficial layers, was and still is aimed to agriculture, the main crops being the ones of corn and wheat.

3.4 Flood mitigation measures in the area

Ion and Nicoară 2006 discussed the technical measures initiated with more than 250 years ago in Banat catchment with the aim to relief the low lands of this area from the effect of reiterated floods. As it is pointed out, flood regulation has been achieved through the regulation of some watercourses sectors by correcting their path; dykes; some permanent and non-permanent reservoirs on the watercourses or aside them (polders); the drying of swamps; the development of a drying canals network, endowed with pumping stations; and the accomplishment of a double connection between Timiş and Bega Rivers, consisting of hydrotechnical joints and canals that allow a water flow gravitationally. Water flowing form Timiş to Bega ensures minimum water supply of Timişoara city and from Bega to Timiş ensures flood protection of Timişoara in case of a high-water on Bega.

3.5 Flood issues in the area

The flood mitigation measures in the area have not been able to control and manage floods sufficiently. Various major flooding events have been recorded in the last century causing dike breaches and consequently catastrophic floods, there were floods recorded at Lugoj and Cebza in 1912, 1966, at Grăniceri and at Crai Nou two breaches in the right bank dike 2000, in 2005 there was failure of three dykes; one along Bega and two breaches along Timis downstream of Timişoara, that resulted to the flood of about 25,000 ha spilling a water volume of about 300 - 350 million m3 which led to severe damages as explained in Ion and Nicoară (2006) paper.

3.6 Current Flood forecasting and warning

The current flood forecasting and warning is based on empirical models which show relationships of the flow at a downstream point to that at the upstream station. In managing of the flooding crisis in Banat in the year 2005, with specific concern on the Timiş and Bega Rivers basins shows that there were three significant time periods, with warnings provided in advance.

4. Research Methodology and Approach

4.1 Introduction

This section explains the methodological issues and approach that were used in this research. It focuses primarily on providing an in depth explanation of the tools and techniques that were used in the research process.

4.2 Literature Study

The study further continued with a review of literature and previous studies that had been undertaken on the Timis and Bega basins, as well as reviews on projects in which integration of HEC-HMS and HEC-RAS were reviewed to give a good insight in the performance of the coupled hydrologic-hydraulic model as well of 1D2D models and also to understand their limitations.

4.3 Data Collection

The data required for the study was collected from various sources. Most of the rainfall gage measurements, river flow, cross sections and level data were acquired from previous studies in the basin. The satellite image data which was used for the catchment delineation was provided as a 30mx 30m from the NASA Shuttle Radar Topography Mission (SRTM).

4.4 Software tools and modelling software

The following software tools were used at various stages of the research: HEC-HMS, HEC-RAS, GIS for pre and post processing, SOBEK 1D-2D. These are discussed in section 2.4 in detail.

4.5 Data Analysis and Processing

The data that was required for this study were:

- Digital elevation map (DEM) for the Timis Bega basin
- Long time series of evaporation, precipitation, river level and discharge data
- River characteristics, sections, cross-sections, longitudinal profile and discharge

4.5.1 Data Screening

The data screening consisted mainly of three steps

- A rough screening of the rain, flow and level data and computation of totals for the years.
- Plot of these total according time steps of months and year to be able to note any trends or discontinuities

4.6 GIS Analysis

The catchment analysis involved the following;

- Pre-processing to remove extremes for hydrological analysis
- Filing in of sinks
- Stream and watershed delineation
- Plotting of rain and stream gauge stations

A more detailed description of all the procedures involved are provided in the Annex.

4.7 Models development

The first phase was to develop hydrologic model (HEC-HMS) of the Timis-Bega catchments. The second phase was the development of a one-dimensional hydraulic model of the major rivers and coupling it with HEC-HMS hydrologic model. The third phase of the model development entailed a refinement of the second phase with coupling and integrating of the HEC-HMS (0D) HEC-RAS (1D) and 1D-2D SOBEK model. These models were calibrated and checked for sensitivity. Figure 4-1 shows the coverage of the various models as used in this study. The rainfall runoff model HEC-HMS was used in the whole catchment, the 1D hydraulic model HEC-RAS was used in the main channel starting at Balint and Lugoj gauging stations for Bega and Timis river respectively upto their outlets. The SOBEK 1D-2D was used on the floodplain starting at Remetea and Brod gauging stations.

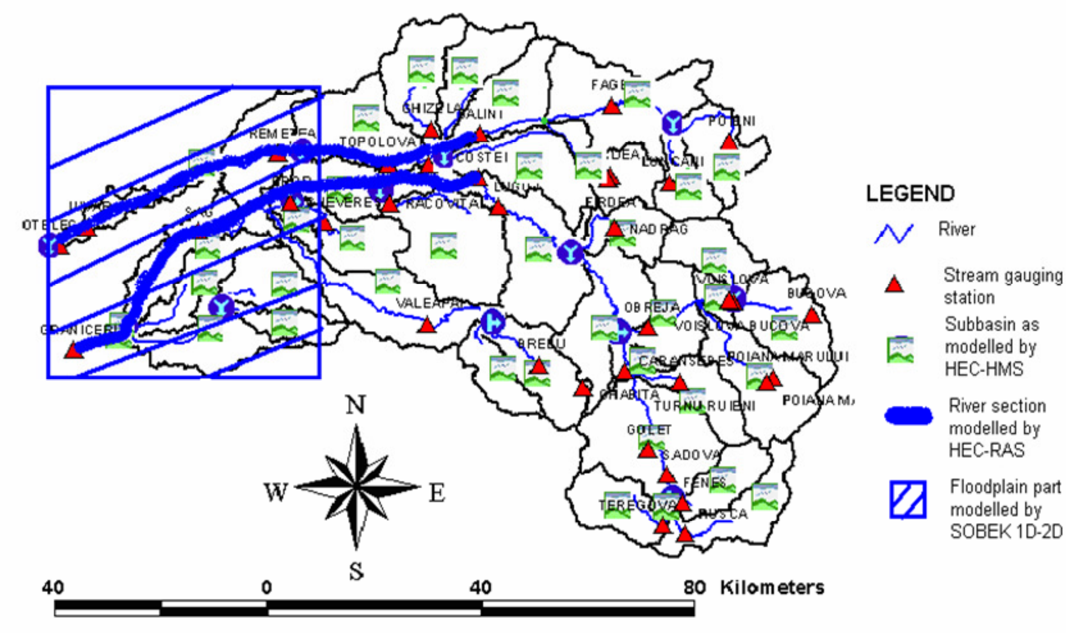

Figure 4-1: A figure showing the interlinkage of different models.

The figure 4-2 shows the flowchart of the methodological process as explained in this section.

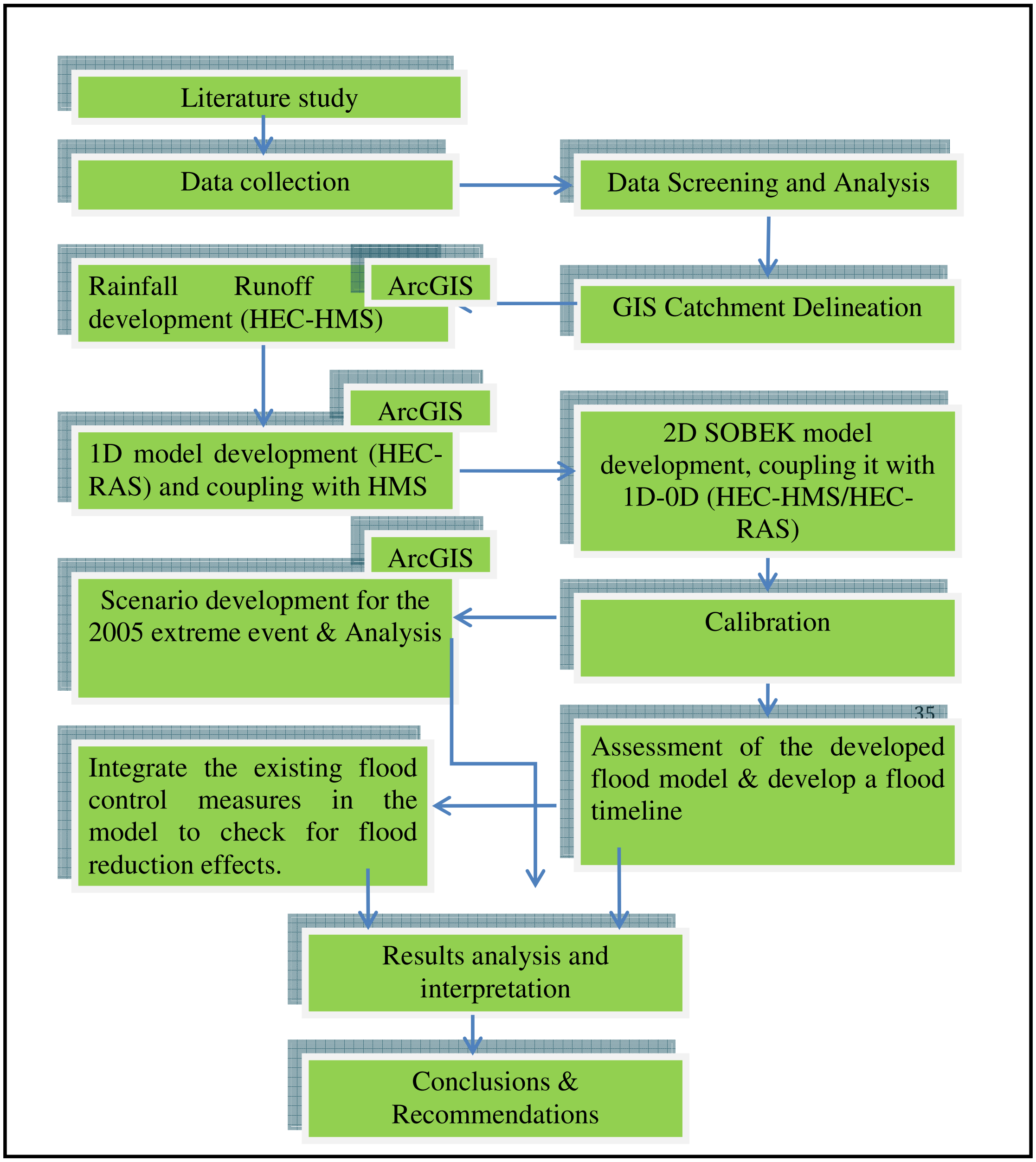

Figure 4-2 : A figure showing a flowchart of the methodology process.

4.8 Analysis of various scenarios and proposed measures

The integrated model was used to assess the different scenarios that were developed. The various scenarios that were analyzed included:

i. The base case which was the 2005 flood event, with no mitigation measure applied.
ii. The 2005 flood event with a mitigation measure of dyke breaching after the arrival of the flood peak.
iii. The 2005 flood event with a mitigation measure of dyke breaching at the time when the flood event started.
iv. The 2005 flood event with mitigation measures of storage and dyke breaching.

5. Results and Discussions

5.1 Data used

Rainfall and discharge data for 14 stations was provided, and the gauging stations are shown in figure 5-1. Rainfall was provided as 12 hour measurement value while discharge was provided as daily values. Table A-1 in the Annex shows the various data that was used in this study.

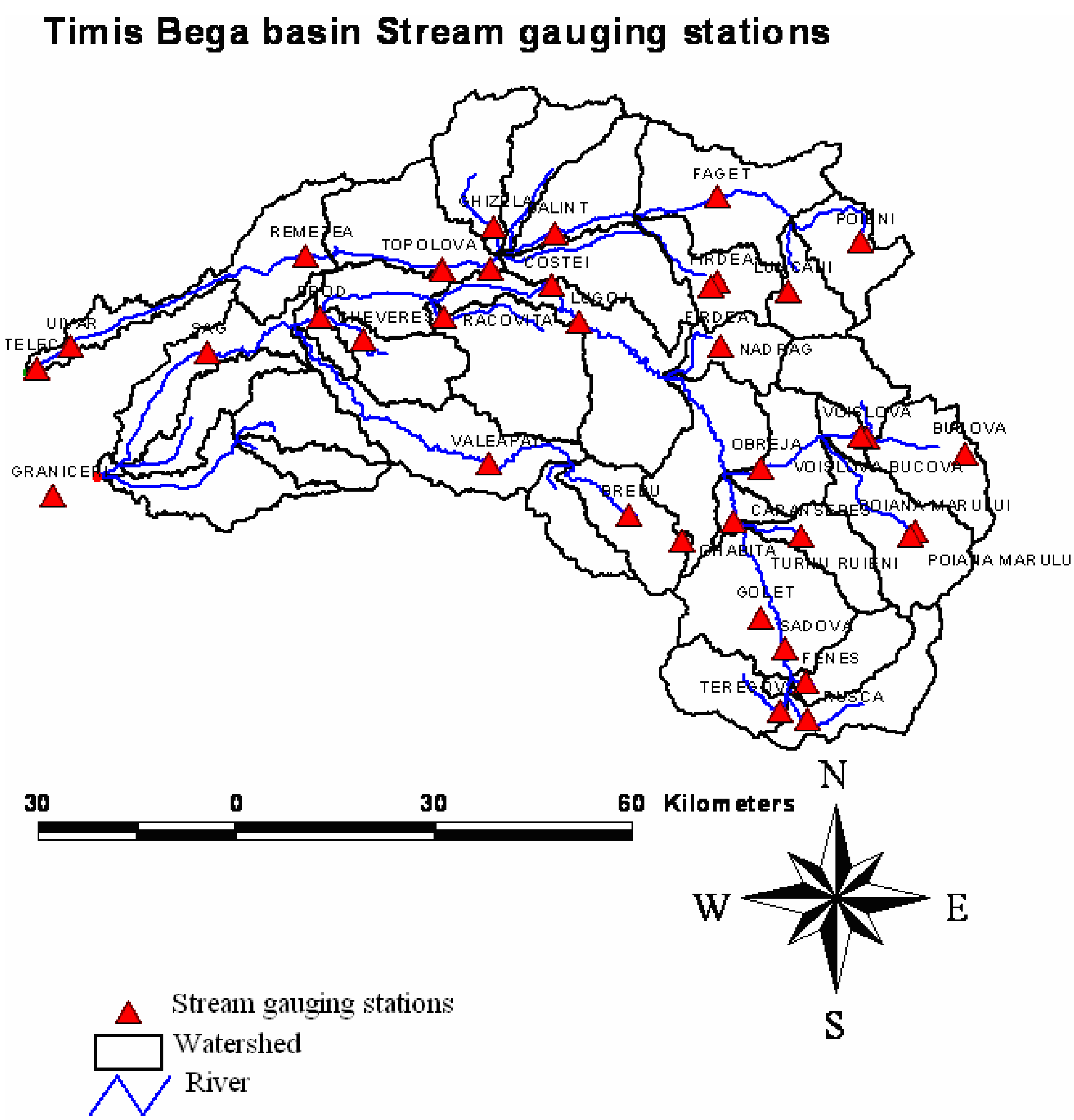

Figure 5-1 : A figure showing Timis Bega basin gauging stations.

5.2 Timis Bega Catchment Delineation and Extractions

5.2.1 Catchment Delineation and Extractions

The detailed procedure for catchment delineation is described in the annex and extractions are shown by figures A-1 to A-4. Below is the delineated catchment of the Timis-Bega basin that was obtained from the DEM with an overlay of the discharge stations.

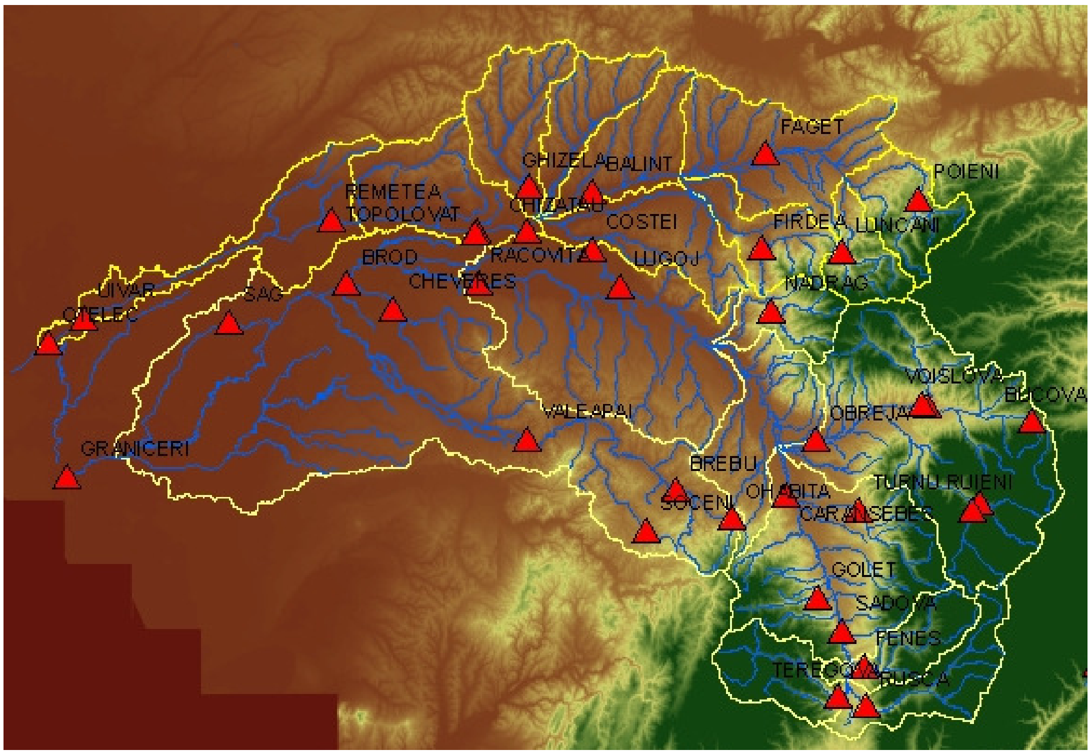

Figure 5-2 : A figure showing delineated Timis Bega basin

The delineated catchment fitted well with what was reported in literature as derived from topographical maps as shown figure 3-1. The figure 5-1 shows the various Subbasins within Timis Bega basin. The stream gages in the Timis-Bega catchment and as well table 1 shows the associated areas.

Table 1: Timis Bega subbasins with their codes and associated areas.

River	Gauging station	CODE No.	Basin Area (km2)
Timis	Teregova	53105	167
Rece	Rusca	53205	163
Fenes	Fenes	53305	125
Ungauged basin	-		105
Timis	**Sadova**	**53110**	**560**
Goloet	Golet	55105	41
Sebes	Tunu Ruieni	53405	122
Ungauged basin	-		349
Timis	**Caransebes**	**53115**	**1072**
Bistra	Voislova Bucovei	53505	232
Bistra	Voislova	53510	404
Bistra Marului	Poiana Marului	53605	79
Sucu	Poiana Marului	53705	77
Bistra	Obreja	53515	659
Nadrag	Nadrag	53805	35
Ungauged basin			736
Timis	**Lugoj**	**53125**	**2706**
Sasa	Poieni	50205	80
Bega	Luncani	50105	73.5
Bega	Faget	50110	474
Bega	Balint	50115	1064
Gladna	Firdea	50301	57

5.3 Data Screening

The flow data was obtained from the Ministry of Water Resources in Romania. A rough screening of data with a computation of the annual totals for the hydrological year was performed. Location of discharge stations are shown in figure 5-1 below.

Using rating curves (Q-h relation) the water levels are converted into discharge. Figure 5-3 and 5-4 show the rating curves for Lugoj and Balint station that were used in converting water levels into discharge. The measured discharge from the gages for each subbasin was used for calibration.

The precipitation data that was provided has been plotted and is shown in figure A-6 in the appendix.

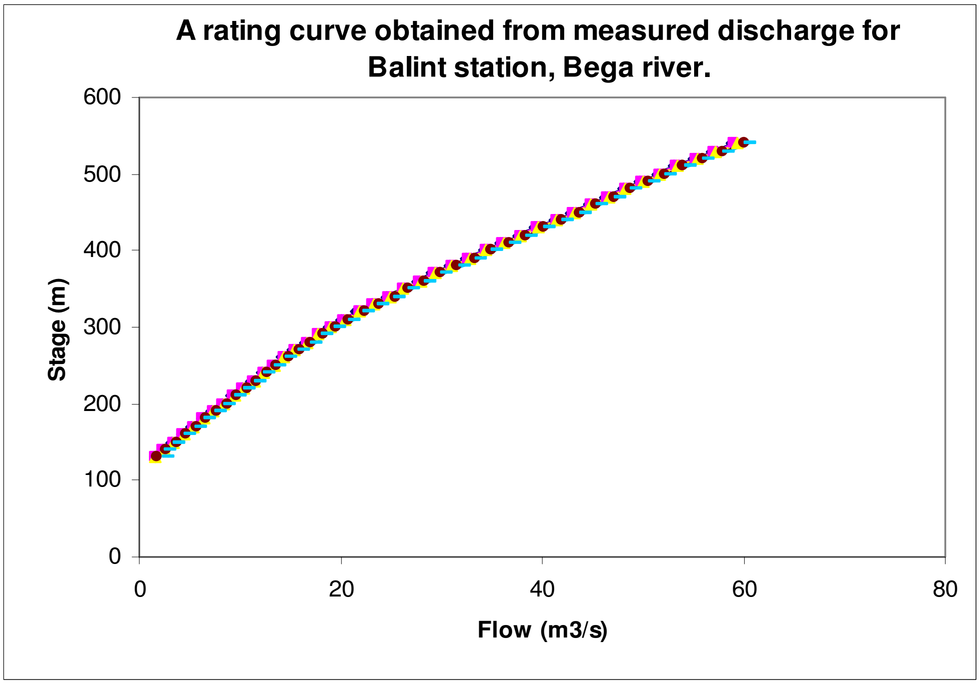

Figure 5-3 : A figure showing Balint station measured rating curve

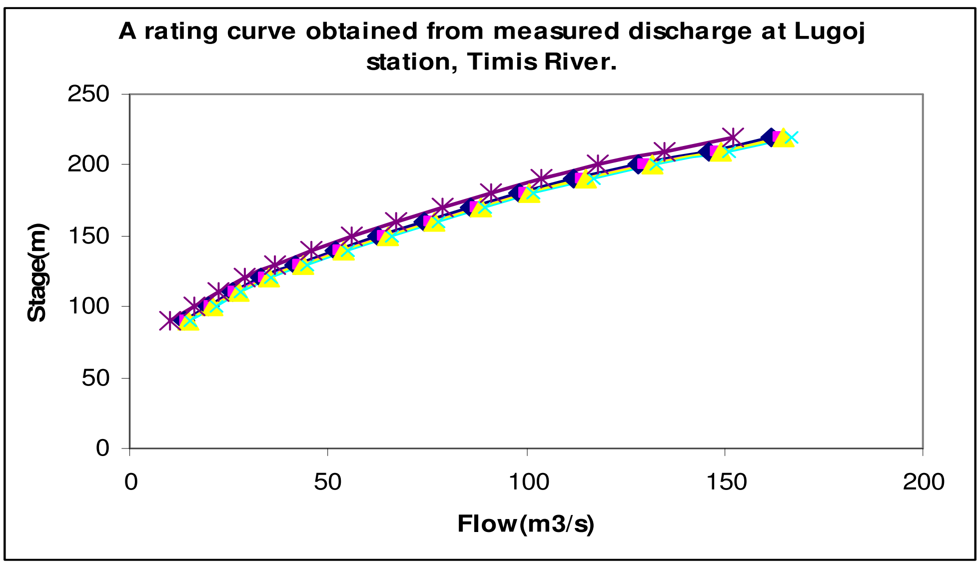

Figure 5-4 :A figure showing Lugoj station measured rating curve.

The measured rainfall and discharge in the Timis-Bega basin that was obtained for the various stations were tested on their reliability. In doing this a correlation was run for all the stations against each other and the results are as shown below in table 2 and 3 for Timis and Bega River respectively.

A correlation was taken between the discharge stations to see how well they relate. This was done for the discharge since the rating curves (water level versus discharge measurements) were provided. The figure 5-5 and 5-6 below shows the discharge as was obtained from the rating curves for Timis and Bega Rivers. All the stations show a good correlation except for Raul timis teregova.

Table 2 :Correlation coefficient for discharge in all stream guaging stations in Timis River

Gaging stations	Rusca	Raul timis teregova	Raul timis teregova gata	Sadova	Golet	Caransebes	Lugoj
Rusca	1						
Raul timis teregova	0.147108	1					
Raul timis teregova gata	0.45102	0.16285	1				
Sadova	0.913852	0.181035	0.605042	1			
Golet	0.540406	0.157361	0.665248	0.637371	1		
Caransebes	0.782342	0.197217	0.508572	0.798566	0.640297	1	
Lugoj	0.729313	0.195153	0.623286	0.85767	0.614009	0.801401	1

The measured discharge was plotted in time for Timis river for the various gage stations and the discharge can be seen to increase downstream, see figure 5-5. The most downstream gage, Lugoj has the highest discharge with tributaries like Raul timis Teregova contributing lowest due to its small catchment area.

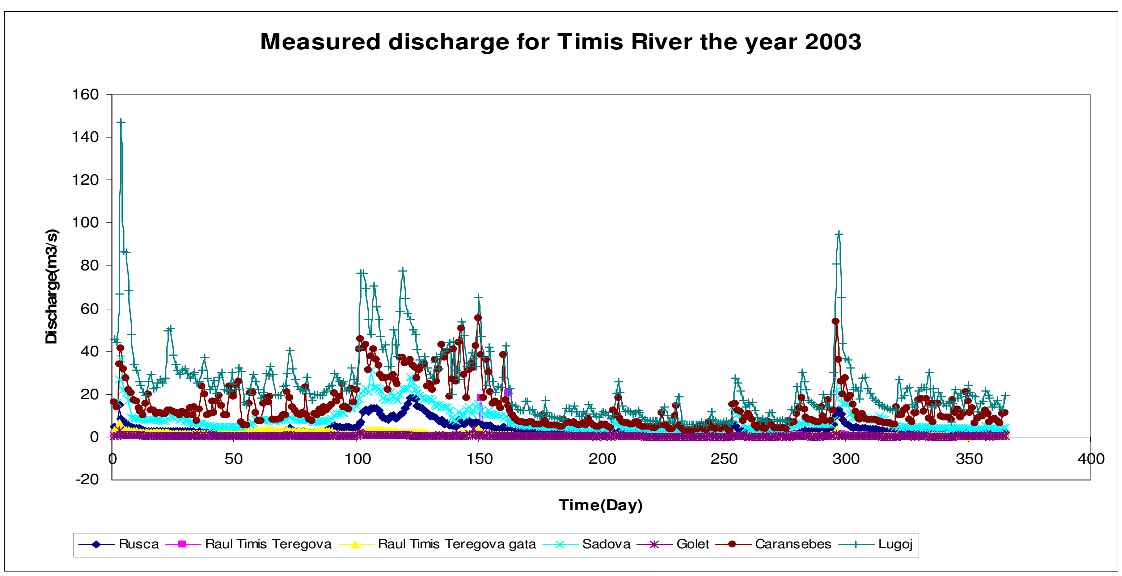

Figure 5-5 :A figure showing the measured discharge for Timis River at various stations.

Bega river had four gauging stations with data and the gaging stations have a high correlation except for the most upstream Poieni station.

Table 3 : Correlation for discharge at various gauging stations on Bega River

	Poieni	*Luncani*	*Faget*	*Balint*
Poieni	1			
Luncani	0.458506	1		
Faget	0.301968	0.771436	1	
Balint	0.156977	0.66065	0.836803	1

The measured discharge for the Bega River was plotted the stations show a good correlation with discharge increasing downstream except for Poieni station in the period of April-May, where its high than the downstream discharge, this is shown in figure 5-6. The data for Poieni was not well recorded for the period April-May this was checked and corrected.

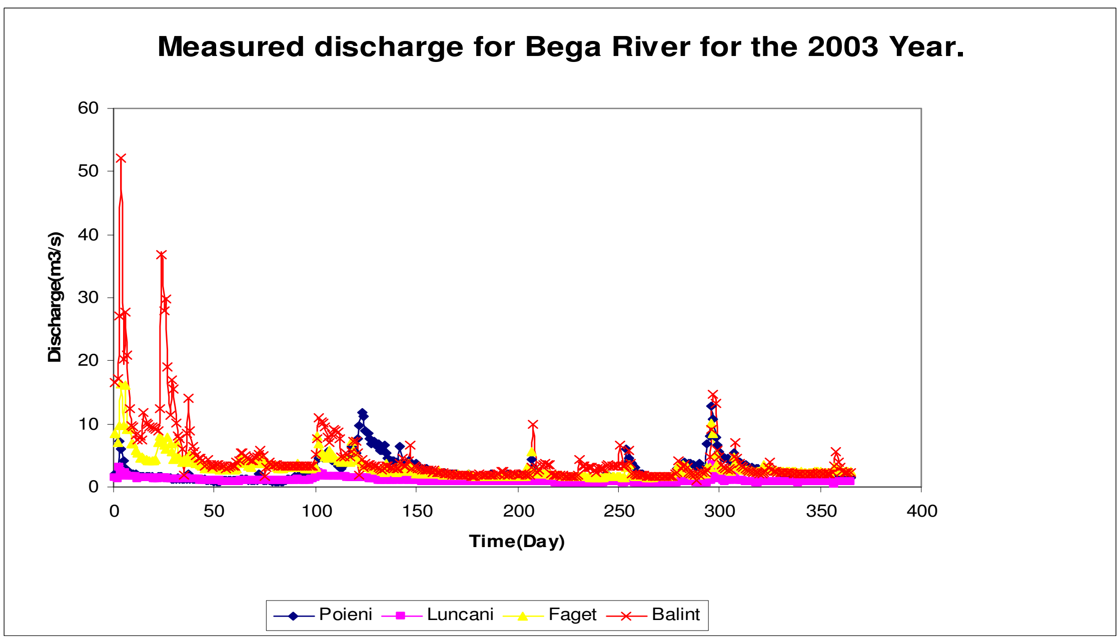

Figure 5-6 :A figure showing measured discharge for Bega river at various stations.

5.4 Application of Rainfall-Runoff model HEC-HMS on Timis Bega basin

5.4.1 Model setup

The Timis_Bega basin was modelled using HEC-HMS developed by US Army Corps of Engineers. The delineation of sub catchments from a digital elevation model, generation of watershed characteristics and stream network were processed by use HECGeo-HMS which uses ArcView GIS extension. The river network was schematized from the upstream station Sadova to the downstream station Graniceri for the Timis River, and Poieni to Remetea for the Bega River, which in all covers a distance of about 275 km, figure5-7 and 5-8 from ARCGIS.

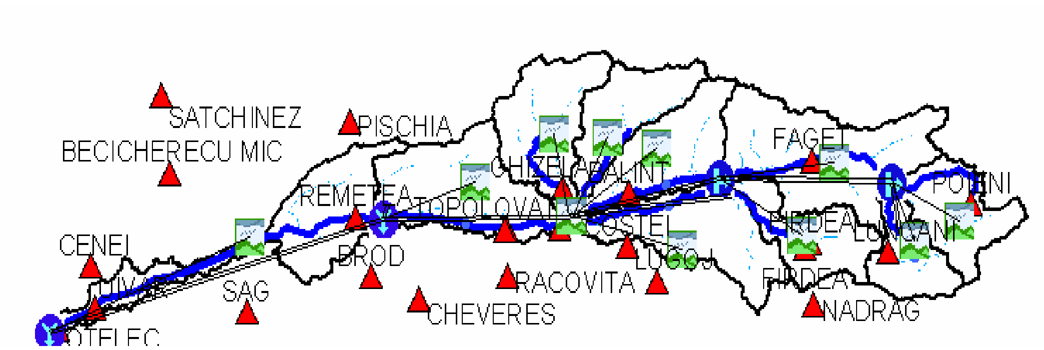

Figure 5-7 : A figure showing Bega River with its subbasins and their interconnectivity.

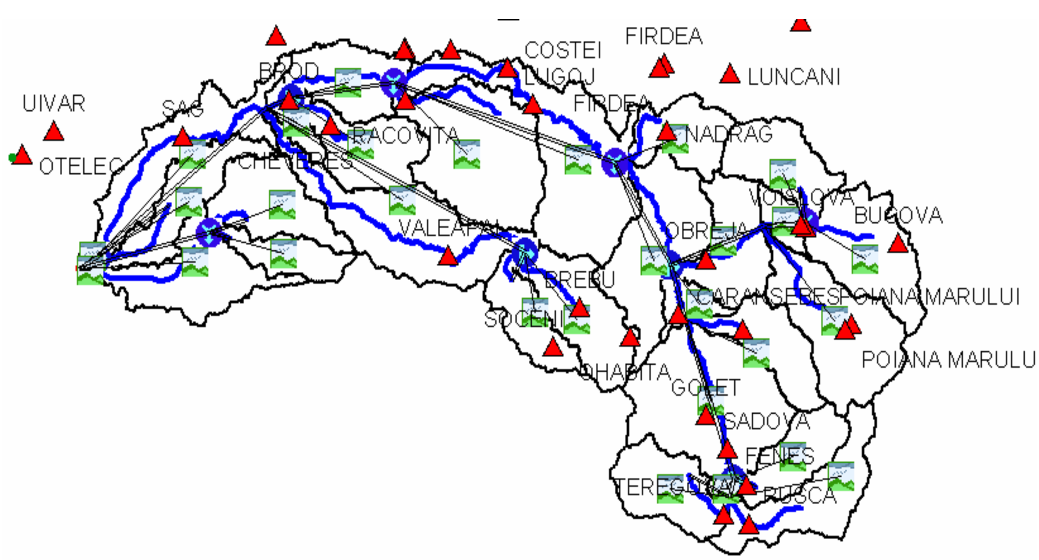

Figure 5-8 : A figure showing Timis River with its subbasins and their interconnectivity.

In setting up the HEC-HMS model each sub-basin is linked to a schematization point such that the subbasins are given the HMS icon and the channels are also shown as straight lines connecting upstream to downstream, this is shown in both figure 5-7 and figure 5-8.

5.4.2 The Basin Model

In setting up the hydrological model each subbasin was represented as an element with various characteristics. The subbasins were connected by junctions, and the channels were modelled as reaches, where water can be routed downstream. Figure 5-7 and 5-8 shows the interconnectivity of the various subbasins for Bega and Timis basins respectively from HEC-GeoHMS. A Mapfile was used as background but is not used for computation. Sub basins produce discharge hydrograph at the outlet of their respective areas.

Each subbasin was given same characteristics, thus lumping the model parameters at the subbasin level. The subbasin characteristics that are required in the model include precipitation, evaporations, physical characteristics like slope, manning roughness, channel length. The spatial distribution and contribution of the precipitation by the measuring gages to the various subbasins was provided by the Romanian water hydrology department as the gage weights.

Evaporation was computed using the Blaney and Criddle method which is a temperature based method used in computing evapotranspiration. The soil moisture accounting method was used to compute the losses in the catchment given precipitation and potential evapotranspiration. The runoff was routed downstream using Kinematic wave thus the subbasin length, slope and roughness coefficient were provided.

Figure 5-9 shows the model schematization as the various elements were introduced in the HEC-HMS.

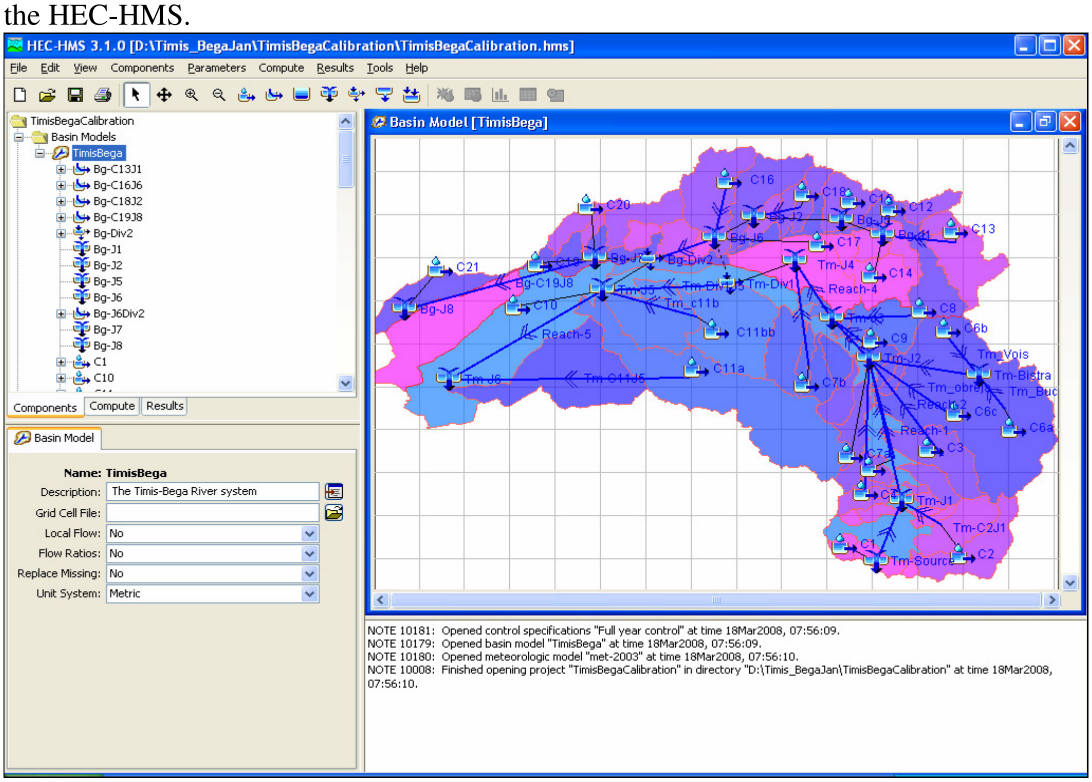

Figure 5-9: A figure showing the basin model for the Timis-Bega catchment

In computing losses with the SMA model, the initial conditions of canopy, soil, surface, and groundwater are given into the SMA as the percentage part full of water at the beginning of the simulation. The storage capacity of each component i.e. canopy, soil, surface and groundwater are provided into the model.

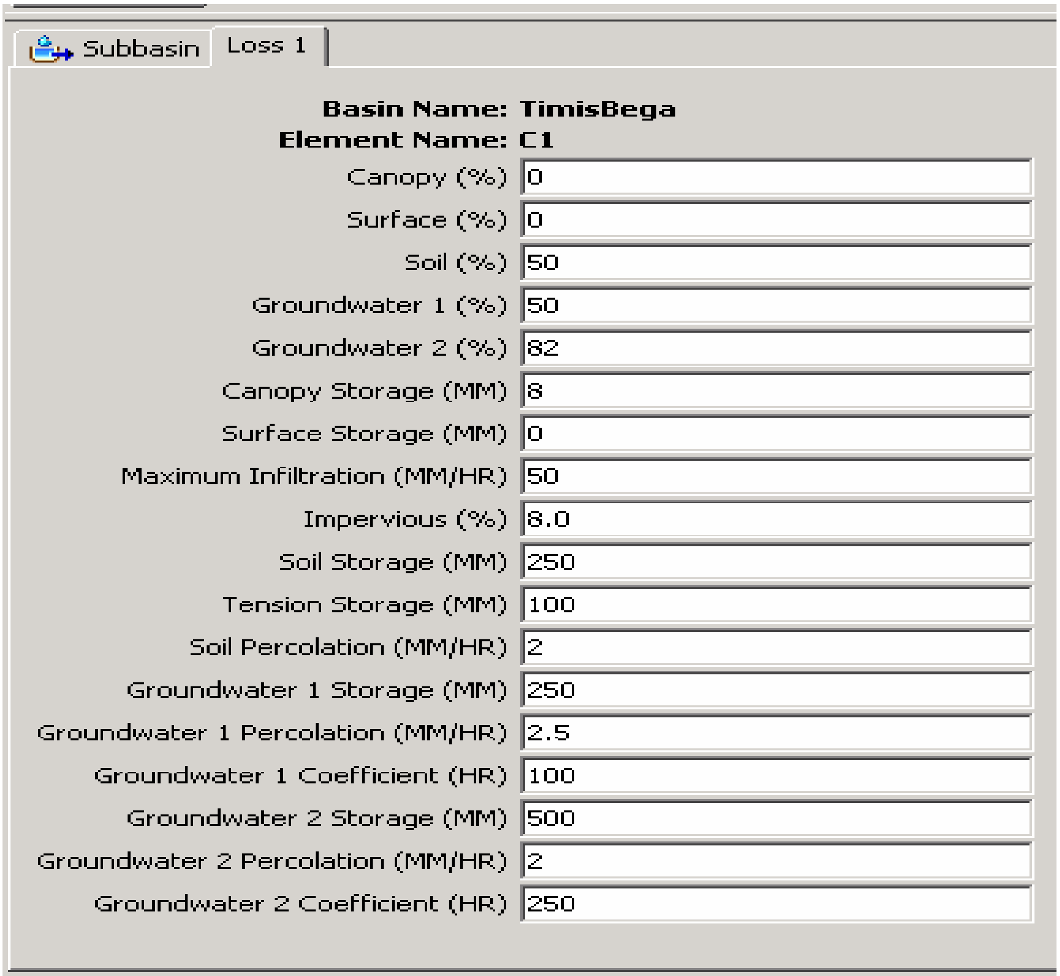

Figure 5-10: A figure showing the parameters for the Soil Moisture Accounting Model for C1.

The storage capacity represents the maximum amount of water that can be held for that component. Canopy storage represents the maximum amount of water that can be held on leaves before through fall to the surface begins and the surface storage represents the maximum amount of water that can pond on the soil surface before surface runoff begins. Groundwater storage and coefficient also have to be entered for the upper and lower groundwater layers. In the model the percentage full, storage capacity, coefficients were provided for each subbasin as shown in figure 5-10 for one of the subbasins C1.

Once the model has computed losses from precipitation, the resulting runoff has to be routed through the catchment from the most remote part of the catchment to the outlet. This was achieved by use of kinematic wave which the watershed and its channels are conceptualized in two modules, overland and channel flow. The watershed was represented as one plane surface (permeable) over which water runs until it reaches the channel. The channel cross sections, slope and roughness of the plane and channel were provided into the model.

In introducing baseflow into the model, the constant monthly baseflow method was used to specify a constant baseflow for each month of the year.

5.4.3 The meteorologic Model

The meteorologic model is used to introduce the meteorological inputs into the model. It requires precipitation data, potential evapotranspiration and Snowmelt. In this study the snowmelt part was not used, though the catchment is highly influenced by snowmelt in the Winter months and spring when temperatures increase. Measured precipitation was provided into the model and the gage weighting method was used to compute the areal distribution.

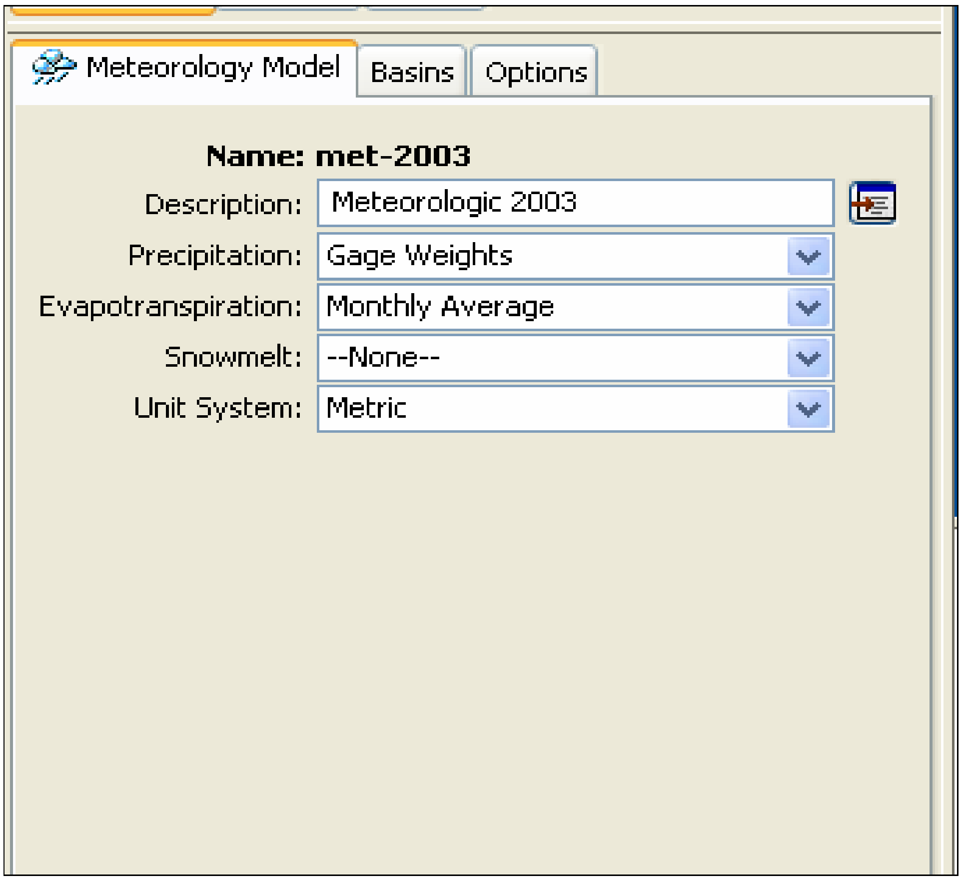

Figure 5-11: A figure showing the elements required for the Meteorological model

The potential evapotranspiration in the catchment was computed using the Blaney-Criddle method, which uses mean air temperature as the main input.

$$ET_{B\&C} = K\ p\ (0.457\ Ta + 8.13) \qquad 5.1$$

Where

$ET_{B\&C}$ Potential evapotranspiration in mm/month
K Crop coefficient
P Monthly percentage of day light hours in the year

T_a Mean monthly air temperature (24 hours means) in ^0C

P is the monthly percentage of day light hours in the year was computed using the maximum possible sunshine hours tables taking latitude 46 North. The monthly percentage was obtained from adding the total sunshine hours in all the months, and dividing each month with the total. A factor of 0.85 was applied since the maximum possible hours were used.

K, the crop coefficient was taken as varying with the season, high K values during summer and low values during winter and also there is a variation on the types of crops. Ta, the mean air temperature was computed from the data that was given from Romanian Waters. The potential ET was computed as shown in table 4 below. Figure A-7 in the annex shows the values as introduced in the model.

Table 4 : A table showing the potential ET as computed by Blaney & Criddle method.

Blaney & Criddle	ET= K p (0.457Ta +8.13)				
Month	**n**	**P**	**K**	**Ta**	**ET potential**
Jan	9.1	5.269	0.2	-2	22.81
Feb	10.4	6.022	0.2	0.4	30.03
Mar	11.9	6.89	0.2	5.4	43.81
Apr	13.5	7.817	0.25	11.8	63.42
May	14.9	8.627	0.6	16.9	82.06
Jun	15.7	9.091	0.6	20.4	95.19
Jul	15.4	8.917	0.75	22.2	97.78
Aug	14.2	8.222	0.72	21.75	89.14
Sep	12.6	7.296	0.6	17.85	71.3
Oct	10.9	6.311	0.4	11.85	51.29
Nov	9.5	5.501	0.3	5.8	35.58
Dec	8.7	5.037	0.2	0.6	25.4
	146.8				**707.81**

5.4.4 Control Specifications

The control specifications contain all the timing information for the model i.e. the starting time and date, and the stop time and date, and the computation time of the simulation. Figure 5-12 shows an example that was used.

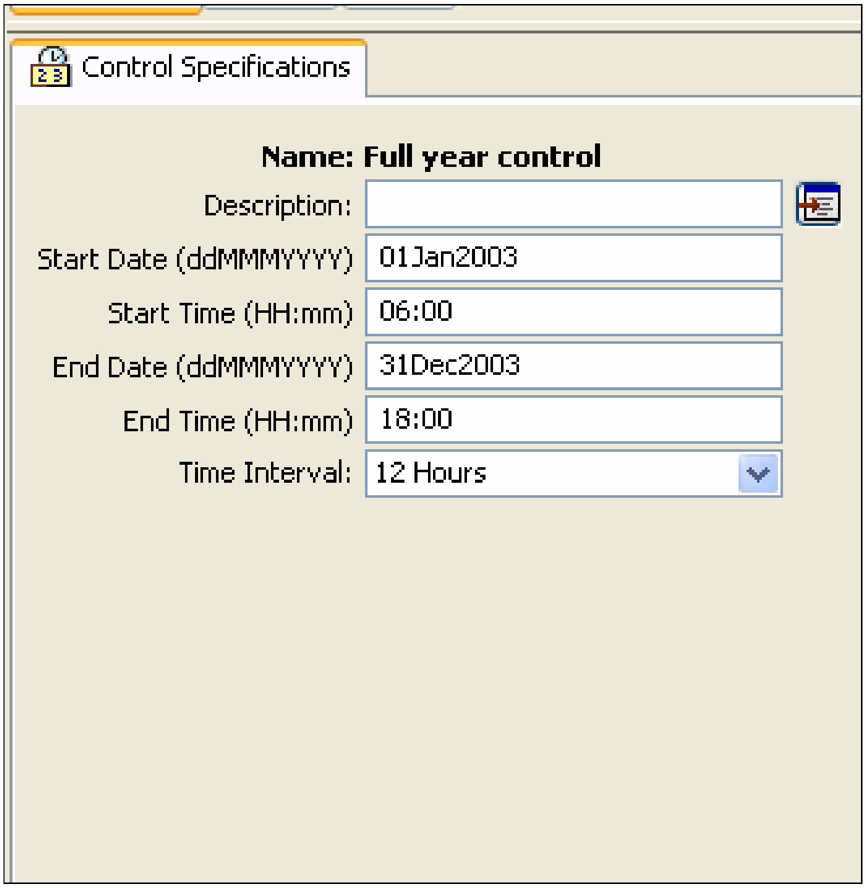

Figure 5-12: A figure showing the Control Specifications used for the simulations

Different hydrologic simulations were run and results in terms of graphs which show time versus flow, and summary tables which show peak flows with their corresponding time series gave the outflows and inflows for each time step. The resulting hydrographs from the catchments were used in the 1D hydrodynamic model.

5.4.5 Model Calibration

In calibrating the rainfall-runoff model, two stations within the basin were used where observed flows were provided. These stations include Lugoj for Timis River and Balint station for Bega River. Downstream of these stations there are other subbasins, diversion and the main outlets of both rivers.

The model was calibrated for the year 2003. The parameter calibration for the hydrodynamic part was the Manning's roughness coefficients. For the runoff of the sub-catchments, the soil parameters were adjusted until a best fit was found. However, calibration resulted in a hydrograph that was somehow similar in shape as the observed hydrograph, the flow values in the dry periods were lower than those observed. Figure A-8 in the annex shows optimization results during the calibration.

Figure 5-13 shows a graph of the observed and modelled results at the Lugoj station. The simulated and observed hydrograph peak was realised on 24 october at 0600 as 87.2 m3/s and 94.70 m3/s respectively. The total outflow for the simulated was 224.43

mm and the observed outflow was 268.84 mm. This means that the peak discharge and the total outflow was computed with less than 5% error.

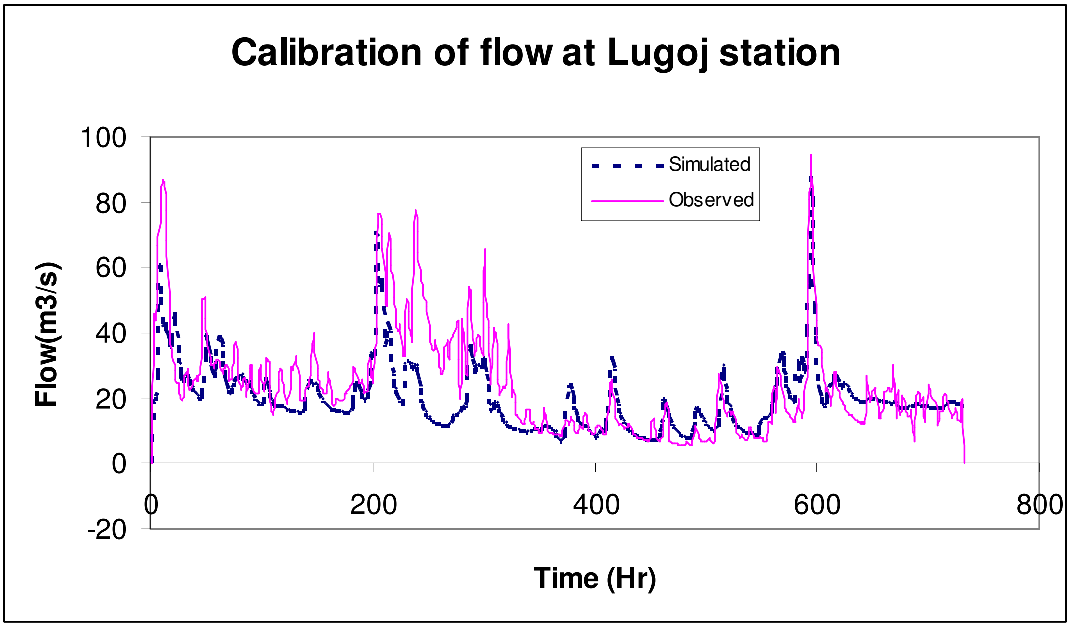

Figure 5-13: A figure showing the calibration results for Lugoj station

A correlation coefficient of 0.55 was obtained as shown in figure 5-14. However, the period April-May was not well predicted, the observed hydrograph shows perturbations that were not shown in the precipitation.

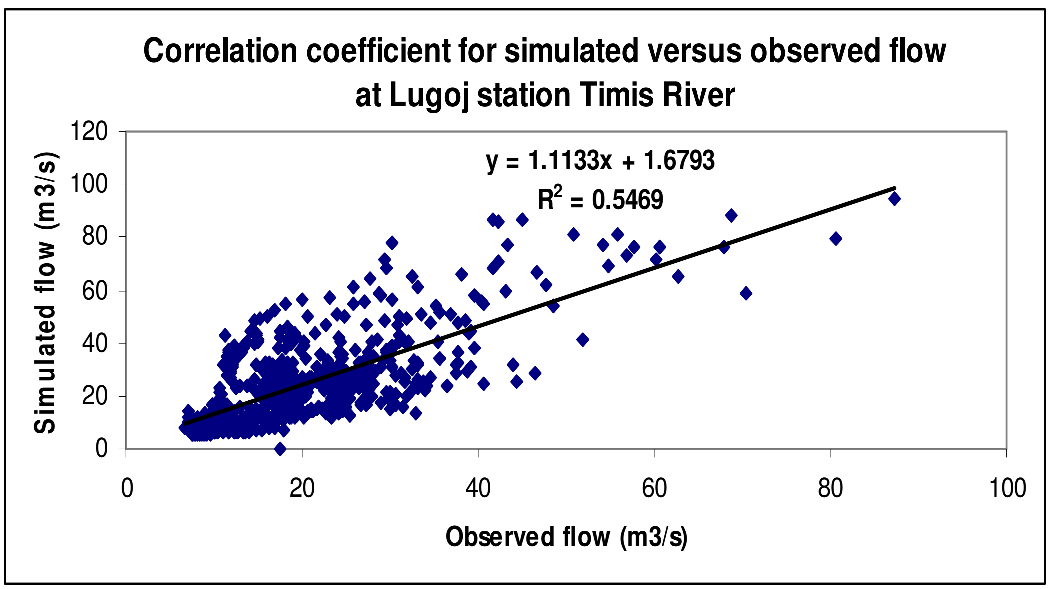

Figure 5-14: A figure showing the correlation coefficient of simulated and observed values in Timis River

For the Bega River Figure 5-15 shows a graph of the observed and simulated results at the Balint station. The simulated and observed hydrograph peak was realised on 04 January at 0600 as 45.3 m3/s and 52.10 m3/s respectively. The total outflow for the simulated was 164.20 mm and the observed outflow was 148.46 mm. This shows that the peak discharge and the total outflow were computed with less than 10% error.

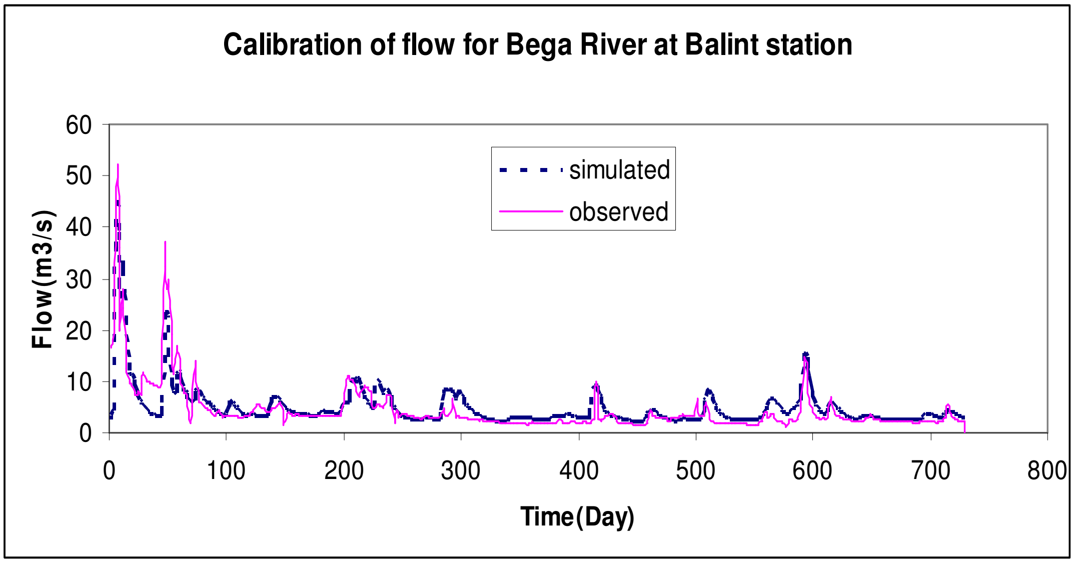

Figure 5-15: A figure showing calibration results for Balint station.

The observed versus simulated hydrographs for Balint station in Bega showed a high correlation of 0.7025, see figure 5-16.

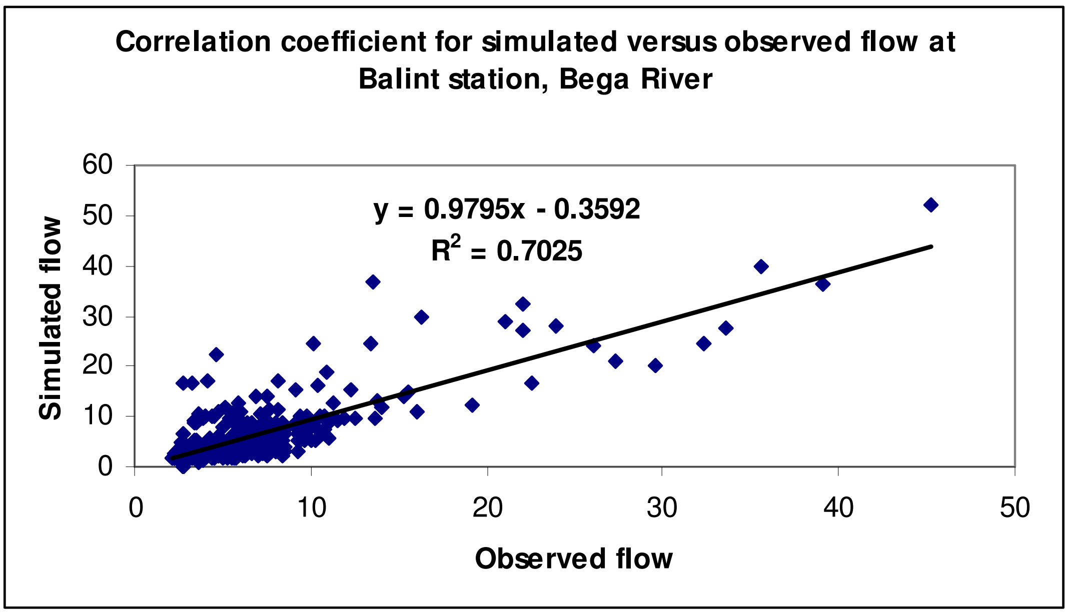

Figure 5-16: A figure showing the correlation coefficient for Balint station, Bega River.

The Root Mean Square Error (RMSE) was computed using the equation below.

For a perfect fit, $y_i = \overline{y_i}$ the RMSE = 0. So, the RMSE value ranges from 0 to infinity, with 0 corresponding to the ideal. The RMSE for Timis River and Bega River was obtained to be 11% and 2.7% respectively. The Bega river simulations were better than the Timis River, thus Bega had fewer dispersions.

5.4.6 Rainfall runoff model sensitivity

Catchment roughness

As shown in figure 5-17 the overall subbasin roughness referred to as plane roughness is a primary factor affecting the accuracy of the peak discharge. The roughness on the plane was varied and the peak discharge was observed to change, with a higher roughness resulting in a lower peak and lower roughness value resulting in a higher peak. The graph shows results obtained from Balint station when a roughness value of 0.4 and 0.56 were used uniformly on the catchment. When a roughness of 0.4 was used higher peaks resulted while 0.56 resulted to lower peaks this can be attributed to higher roughness increasing resistance in the catchment thus not allowing the runoff to flow out of the subbasin to contribute to the total discharge.

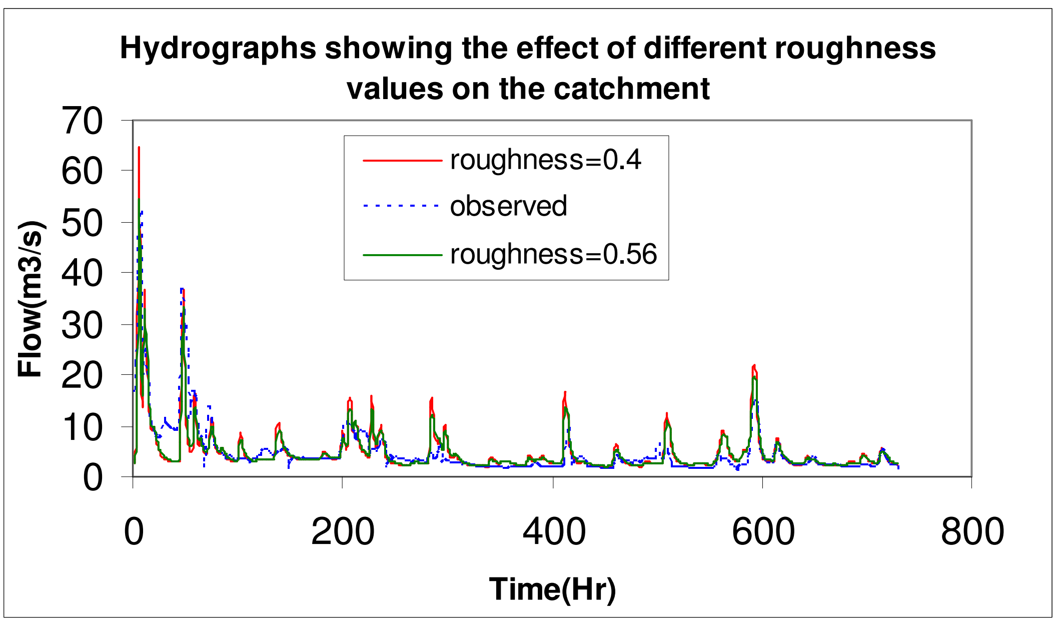

Figure 5-17: A figure showing the sensitivity of the Bega catchment to roughness values.

5.5 Model Integration of Rainfall Runoff-1D-2D

In transforming rainfall time series into a flood inundation map various interconnected components as shown in figure 5-18 were involved. HEC-DSSvue was used for data storage. The measured precipitation was provided into the data storage HEC-DSS, which was used as input into the HEC-HMS model. The system when provided with these rainfall time series it transforms them into runoff through hydrologic simulations in HEC-HMS. The streamflow obtained from the rainfall runoff model was fed into the hydraulic model HEC-RAS that computed water surface elevations. This computed discharge hydrographs were provided into the SOBEK1D-2D model that simulated the floodplain inundation.

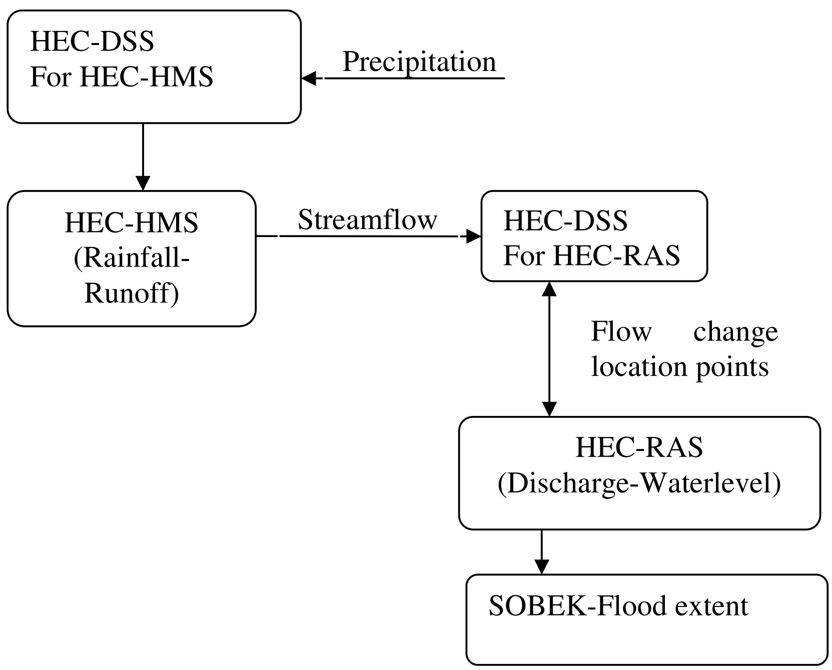

Figure 5-18: A flowchart showing the coupling of HEC-HMS, HEC-RAS and SOBEK 1D-2D.

The elements that were used to support the full integration include HEC-RAS, HEC-DSSvue and HEC-HMS model configurations.
Procedure

i. Input precipitation gage weights into HEC-DSS for HEC-HMS
ii. Executing HEC-HMS
iii. Transferring flow values of HEC-HMS into HEC-RAS by establishing connection points in HEC-DSS file (output of HMS) for HEC-RAS and updating related input files.
iv. Executing HEC-RAS

The modeling systems of the Hydrologic Engineering Center (HEC) use the HEC Data Storage System for time series, HEC-DSS (USACE, 1995), a data base specifically designed for water resources applications that uses a block of time sequential data, called pathnames, as the basic unit of storage and that stores the records in binary format files for access efficiency. A good handle on the HEC-DSS system was critical in this implementation to make available relevant time series records from the database time series tables to the HEC-HMS and HEC-RAS models and to allow the transfer of records. Figure 5-19 shows HEC-RAS model reading flow data from HEC-DSS which are outputs of HEC-HMS

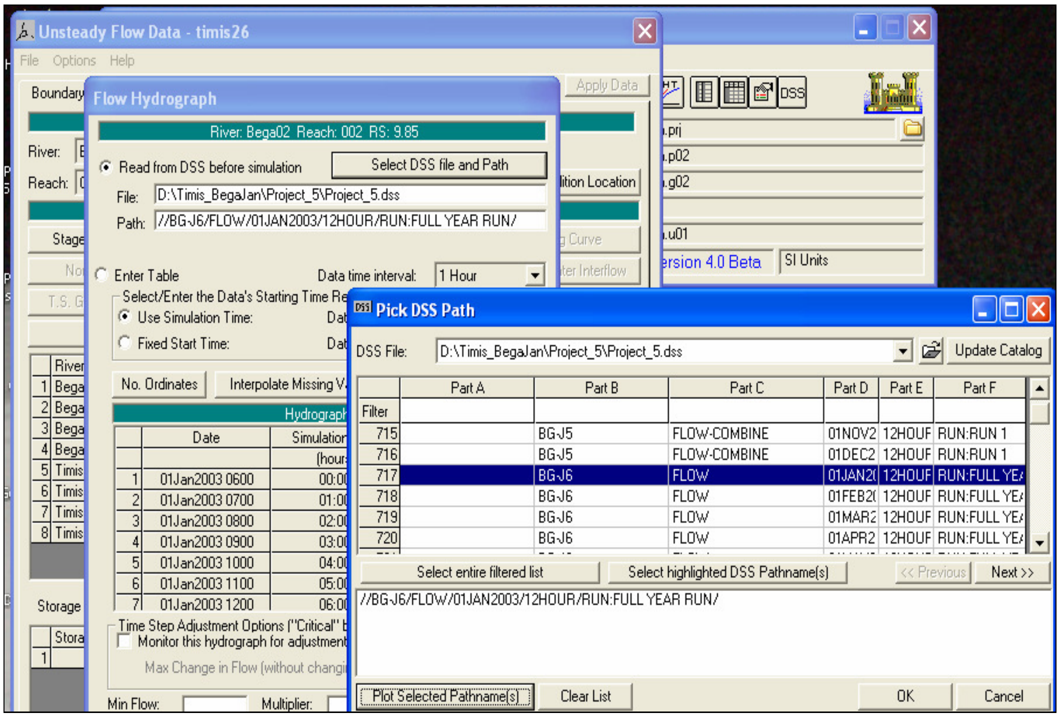

Figure 5-19: A figure showing HMS output file read from HEC DSS into HEC-RAS

A unique identifier (model codification) to support the connectivity between features in each model was needed to spatially relate features across the models. For HEC-HMS this code corresponds to a text string that is used as a feature which is also located in the "B Part" of the Pathnames in the HEC-DSS time series file catalog. In the same manner, in HEC-RAS, the connectivity is accomplished by knowing which cross section is associated to the previous hydrologic features (HMSCodes).

This implies knowing which river, reach and stationing the cross-section lies on. The geographic locations where the models are to exchange hydrologic information, to allow communication between models were identified. The spatial character of this integration requirement is model connectivity. After the modeling features were populated with corresponding model codes (HMSCode and RASCode in this case), a relationship class was needed between them to create a 1 to 1 association between. Not all cross sections in the hydraulic model represented points of information exchange. Figure 5-20 shows the cross sections that were used for information exchange between the rainfall runoff and 1D model.

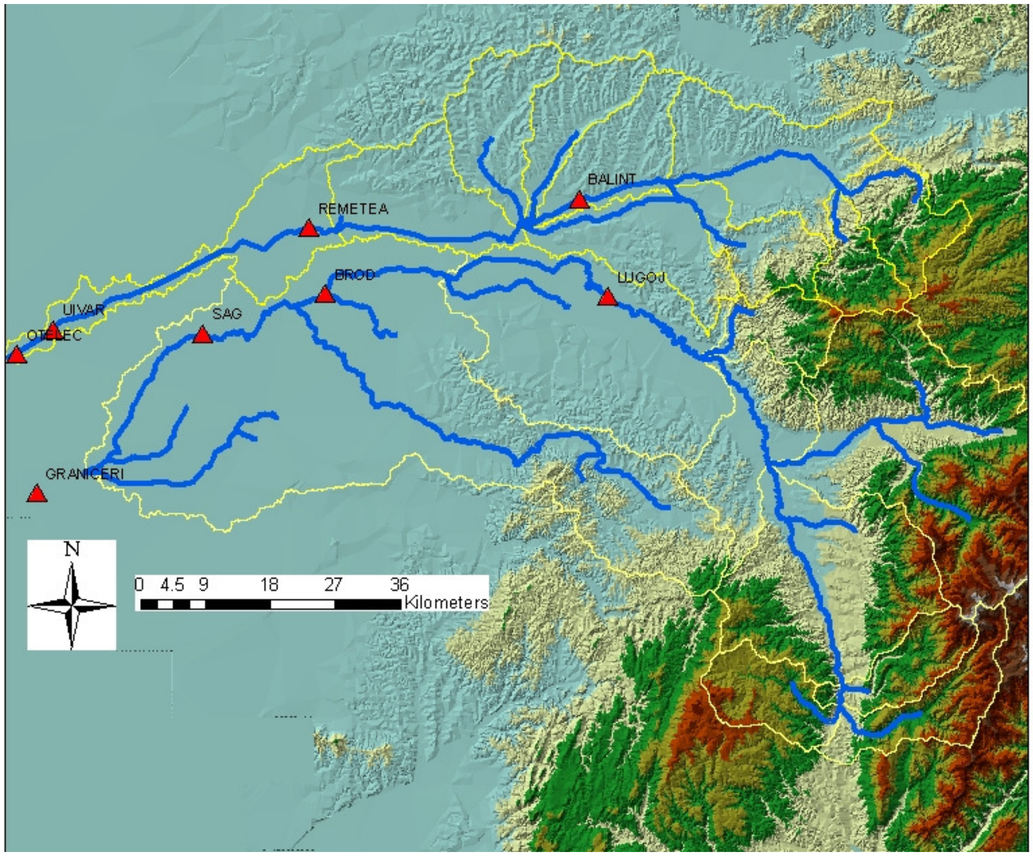

Figure 5-20: A figure showing the cross sections for connectivity of HMS and RAS.

The discharge hydrographs from the 1D hydraulic model were provided into the 2D SOBEK model as boundary conditions on the upstream part, and as well it gave an insight on the choice of the downstream boundary conditions. Model simulations were performed with the aim of providing key information about the river and overbank flows in Timis-Bega basins.

5.6 1D Hydraulic modeling using HEC-RAS.

5.6.1 Schematization of River Network

To schematize the river network, the river reach was introduced first, after this then the cross sections were provided to the model, example of cross sections that were provided are shown in figure 5-21 for Sag stations. The parameters are representative cross-sections for each subbasin, including left and right bank locations, roughness coefficients (Manning's n), and contraction and expansion coefficients. Roughness coefficients, which represent a surface's resistance to flow and are integral parameters for calculating water depth.

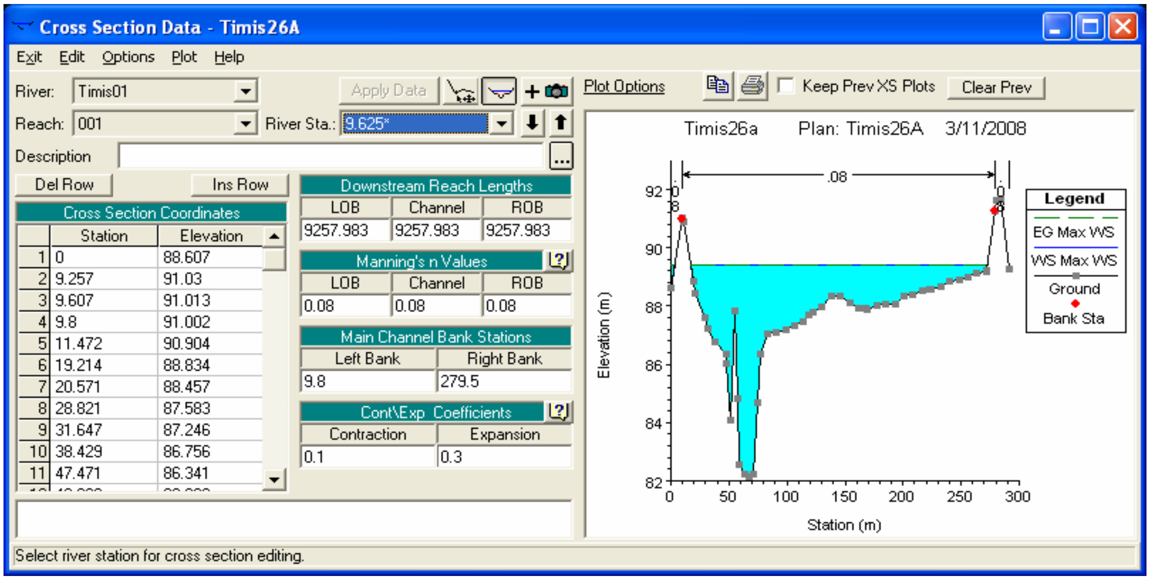

Figure 5-21: A figure showing cross- sectional data for Timis River at Sag station.

5.6.2 Flow data

Flow data was provided by HEC-HMS calibrated output hydrographs, the subbasins downstream of the calibrated stations were provided as lateral inflows into the model, this were catchment c11a,c19,c20,c21 and c11b. Boundary conditions for different flow conditions were taken as normal depth in the downstream part. For flood analysis downstream boundary conditions are very important and in this case both conditions were used. The initial conditions were provided as flow.

5.6.3 Boundary Conditions

Boundary conditions for the 1D channel were introduced as hydrographs from the rainfall runoff model. The most upstream BC was introduced as flow hydrograph and the downstream catchments that were joining the flow were introduced as lateral inflow. Downstream boundary conditions were introduced as normal water depth. The initial conditions were provided as flow. Figure 5-22 below shows the river reaches and the type of boundary conditions that were provided in HEC-RAS, all the flows were provided from HEC-HMS.

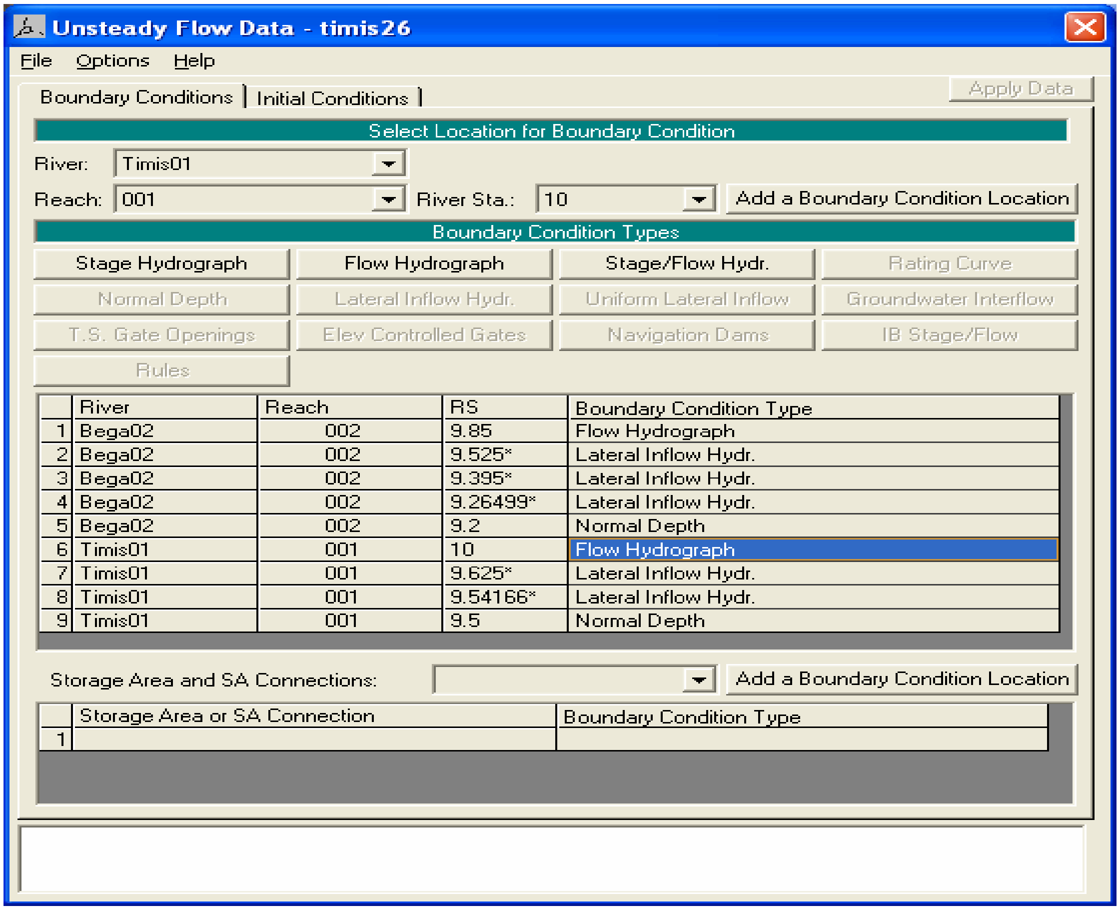

Figure 5-22: A figure showing boundary conditions as provided into HEC-RAS

The fig 5-23 below shows the schematic of the system as a plan view were introduced in HEC-RAS.

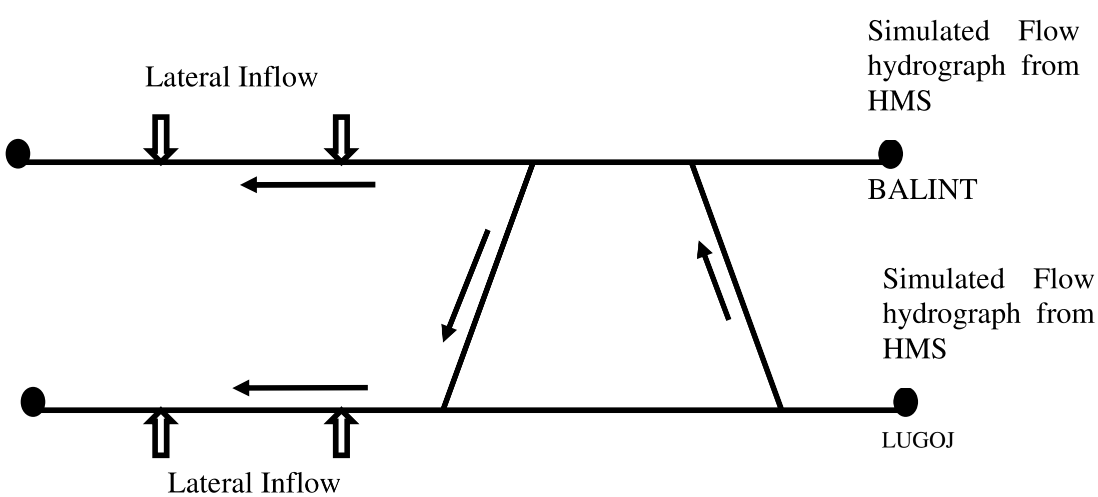

Figure 5-23: A figure showing plan view schematization of River network in HEC-RAS

5.6.4 Model Results

The model results from the 1D hydraulic model, shows that the higher the manning coefficient the peaks are attenuated as shown in fig 5-24 below various manning coefficients 0.15,0.12 and 0.08, were used in the model. However in the downstream part there were no available observed measurements so calibration could not be carried out.

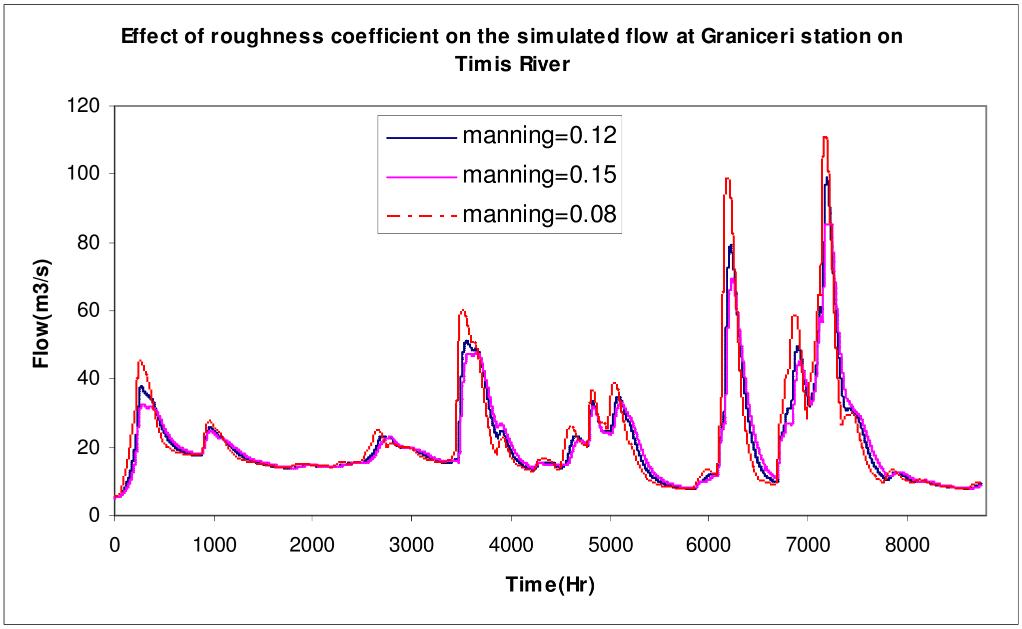

Figure 5-24: A figure showing HEC-RAS downstream discharge with roughness sensitivity

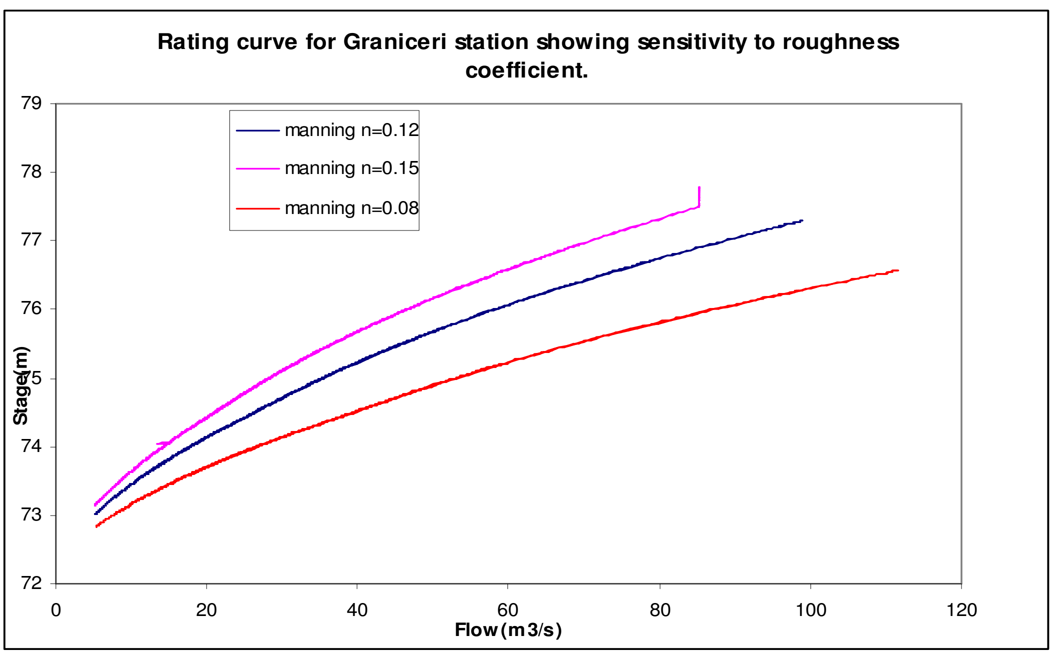

Figure 5-25: A figure showing a rating curve at Graniceri station, Timis River.

A screenshot showing Timis river from a side view accounting for the backwater effect on a flow on the 27 october 2003 is shown in the figure 5-26. The corresponding cross-sections at Graniceri and sag for the same event are also shown as figure 5-27 a and 5-27 b.

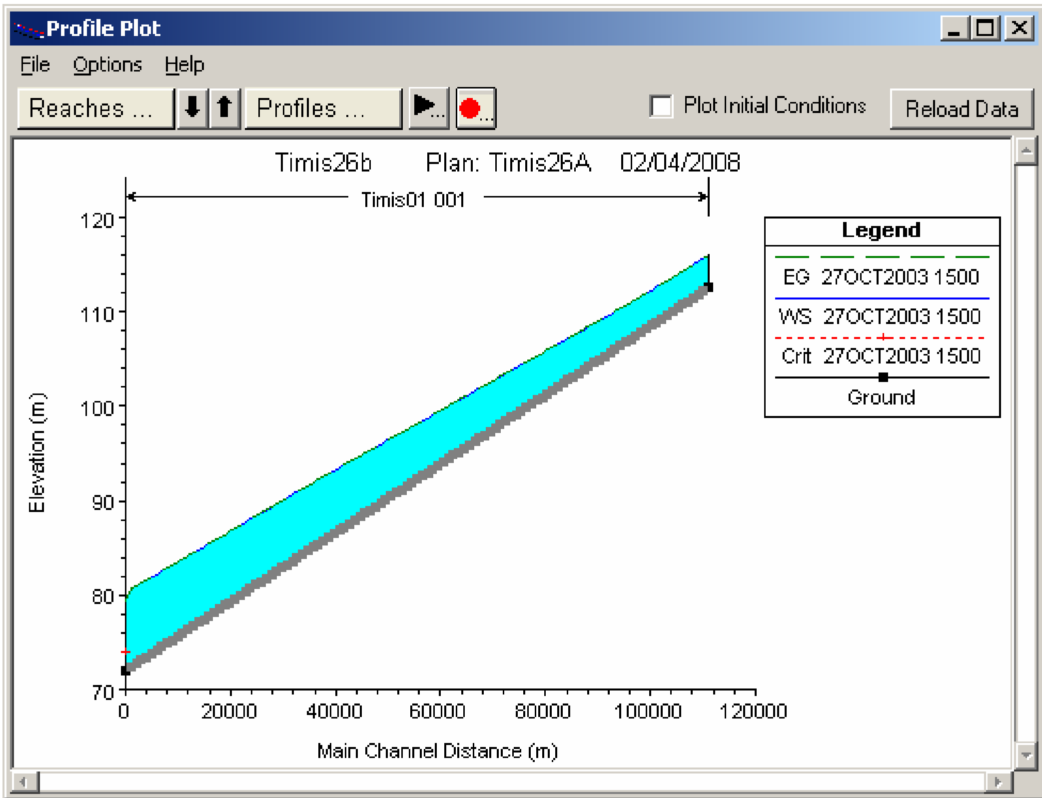

Figure 5-26: A figure showing backwater effect on Timis River.

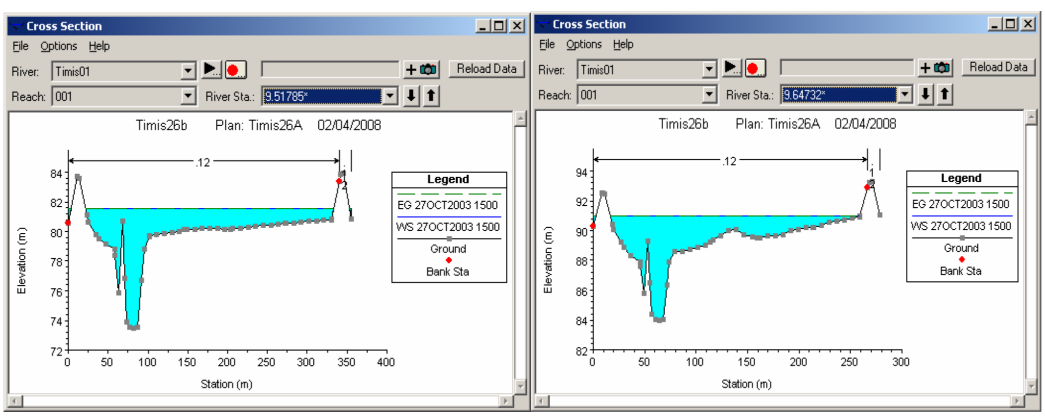

Figure 5-27: Cross sections corresponding to the side view of (a) Graneri (b)Sag

5.7 1D-2D Hydraulic modelling using SOBEK.

5.7.1 Model setup

In the 1D-2D modelling the river was schematized as a 1D channel and the floodplain was represented by the two dimensional grid. The flooding was allowed to occur from a 1D channel into the 2D grid. Two boundary nodes were introduced at the upstream and downstream of each river reach, with the course of the river changed according to the map by using the vector layer mode. A two dimensional boundary was introduced at the downstream side. The model schematization is as shown in figure 5-28 below.

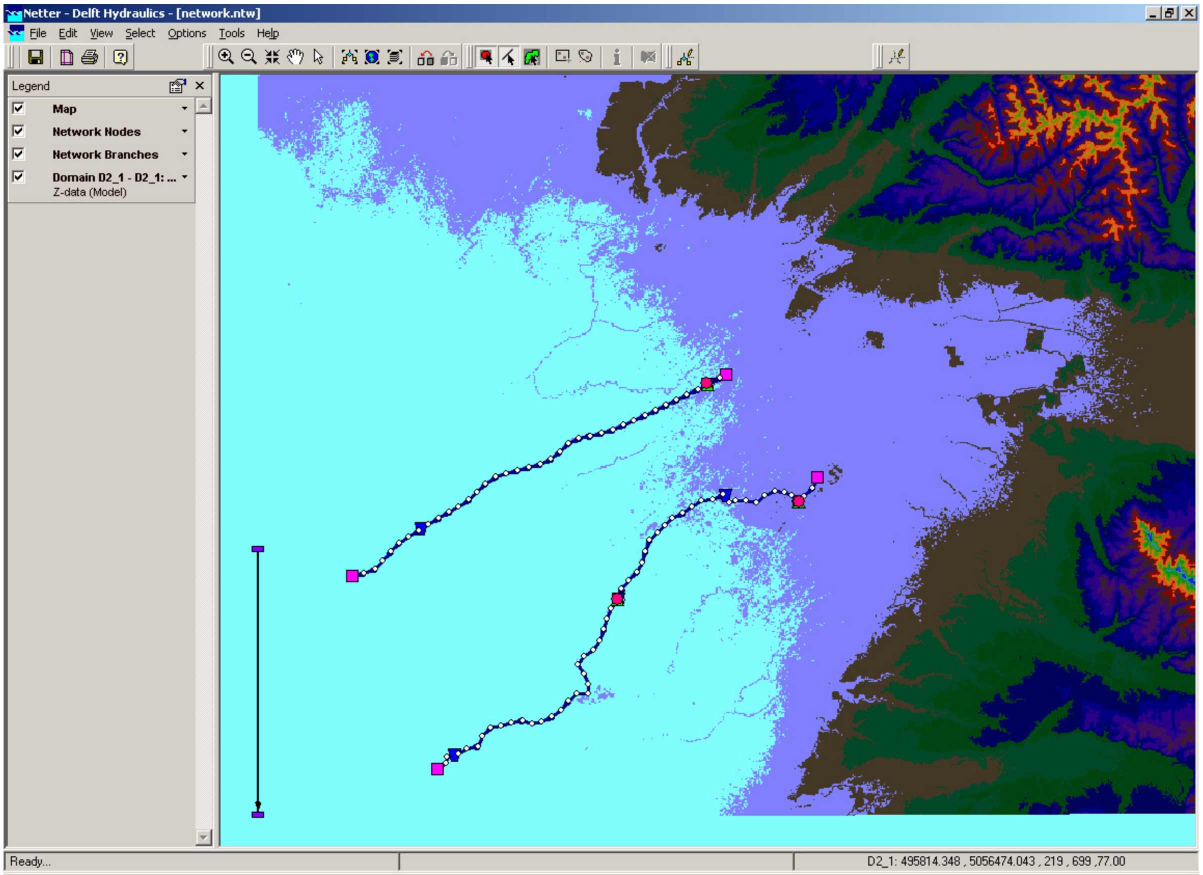

Figure 5-28: A figure showing SOBEK schematization of 1D-2D components

Flooding was allowed to the 2D from 1D using a dummy branch which has a weir with a control structure. As shown in the figure 5-29 below. The weir crest level was lowered with time so as to model the dike breach. Figure 5-30 below shows values that were used for the weir controller.

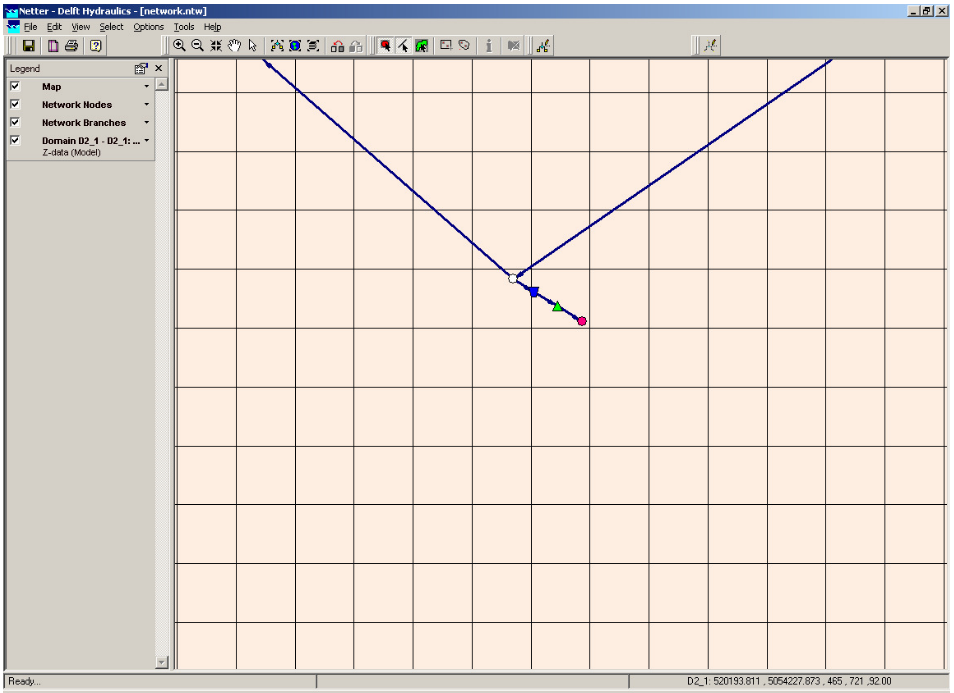

Figure 5-29: A figure showing Schematisation of dummy branch with weir

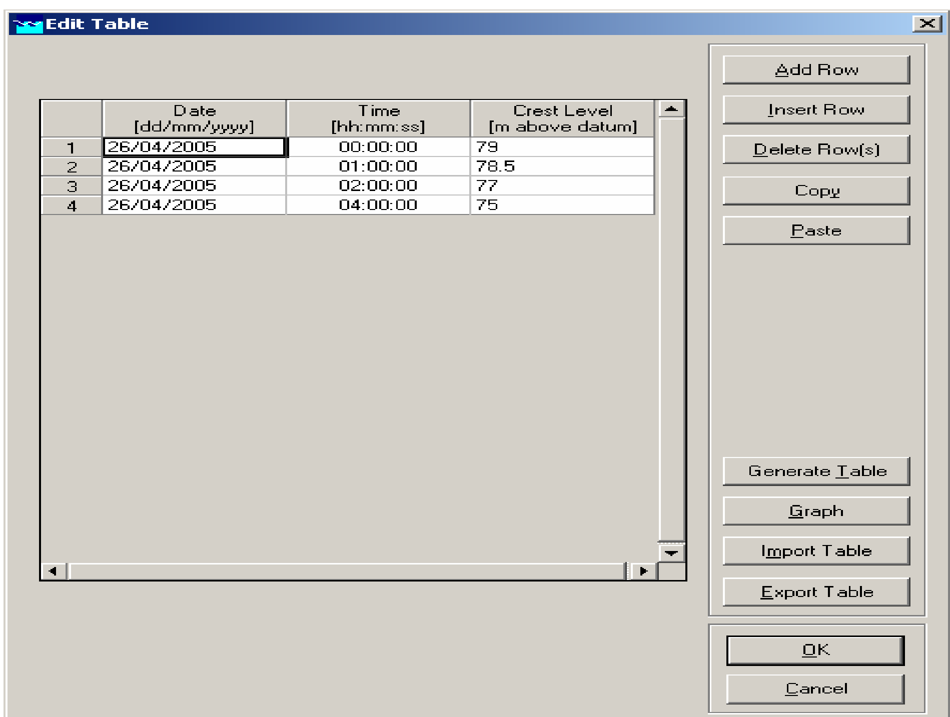

Figure 5-30: A figure showing Weir crest level controller

5.7.2 Boundary conditions

Boundary conditions for the Sobek 2D were obtained from the 1D hydraulic model HEC-RAS. For the downstream the boundary conditions were given as the water level and the upstream conditions were given as a flow hydrograph. The downstream water level was provided as the boundary condition both for Timis River and Bega River. The upstream input hydrographs were given as shown in the figure 5-31 below.

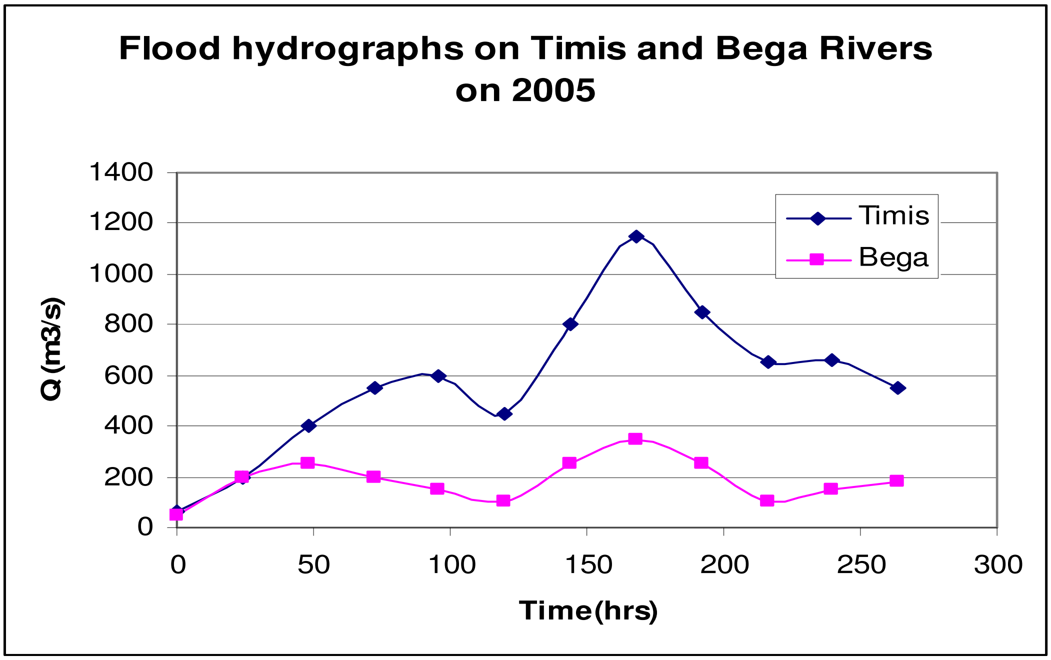

Figure 5-31: A figure showing flood hydrographs for the 2005 flood event.

5.7.3 1D/2D Model result and sensitivity

The model simulations for the 2005 flood event are as shown in figure 5-32. The flood affected areas were 22,631 ha and this compared well with the observed affected areas at the time of the event which was estimated to be 25 000 ha. This has been discussed in chapter 6. The water depths as simulated were upto 2.5m, see figure 5-33. At the dyke breaches the water depths varied greatly during the simulations. The velocities were observed to be higher at the start of the simulation and fairly constant. The flood was generated from extreme rainfall events so the velocities were expected to be lower, except for the area where there was a dyke breach. The points where there was dyke breach the velocities were higher upto 0.7 m/s. For the normal overflows the velocities were 0.15 m/s, see figure 5-34.

In checking the sensitivity of the model one of the scenarios was assessed by varying the friction coefficient for the DEM, 2D grid. The Manning roughness for the DEM was taken as 0.12, 0.15 and 0.08. The lower the manning roughness coefficient the higher the velocities and the flood extent increase downstream, as shown in figure 5-32. This means that resistance to the flow decreases and allows the flow to move with higher velocities. While when the roughness coefficient is higher the resistance is increased and the flow velocity is decreased.

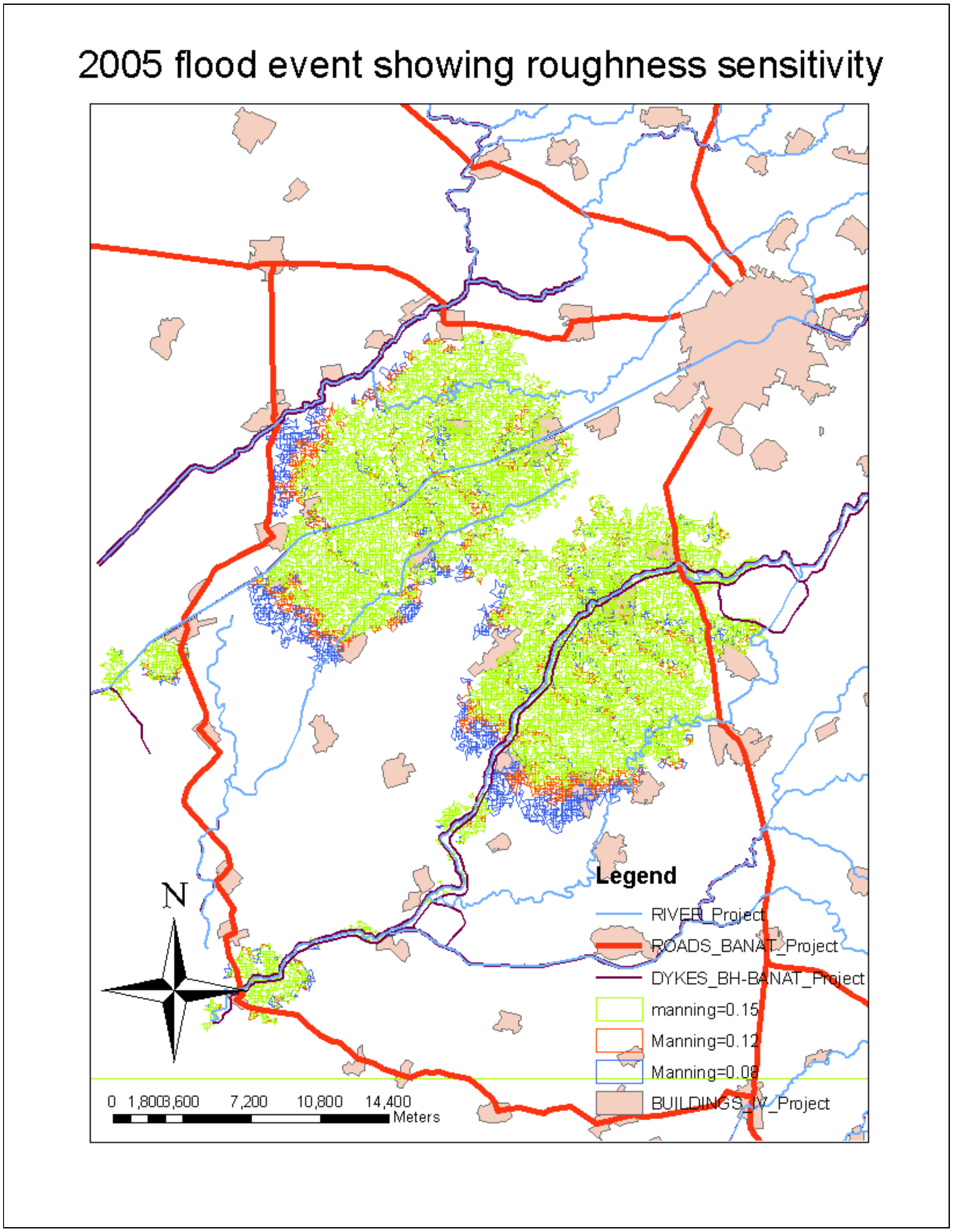

Figure 5-32: A figure showing modeled flood extent of the 2005 event for different roughness values.

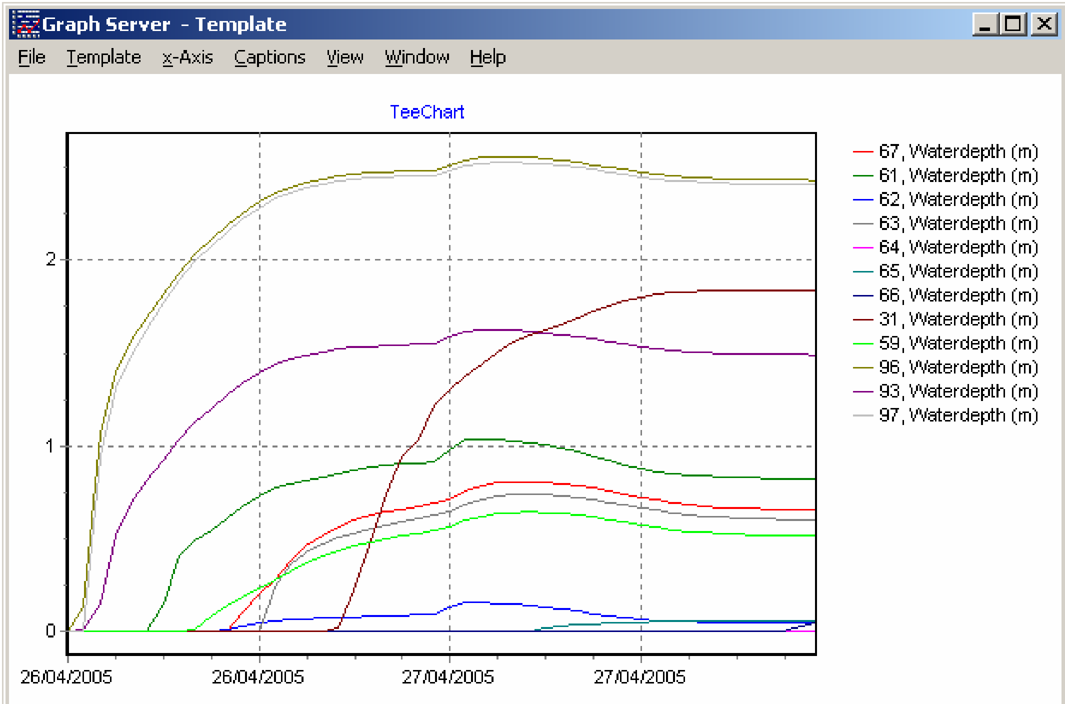

Figure 5-33: A figure showing the waterdepth for the 2005 flood event

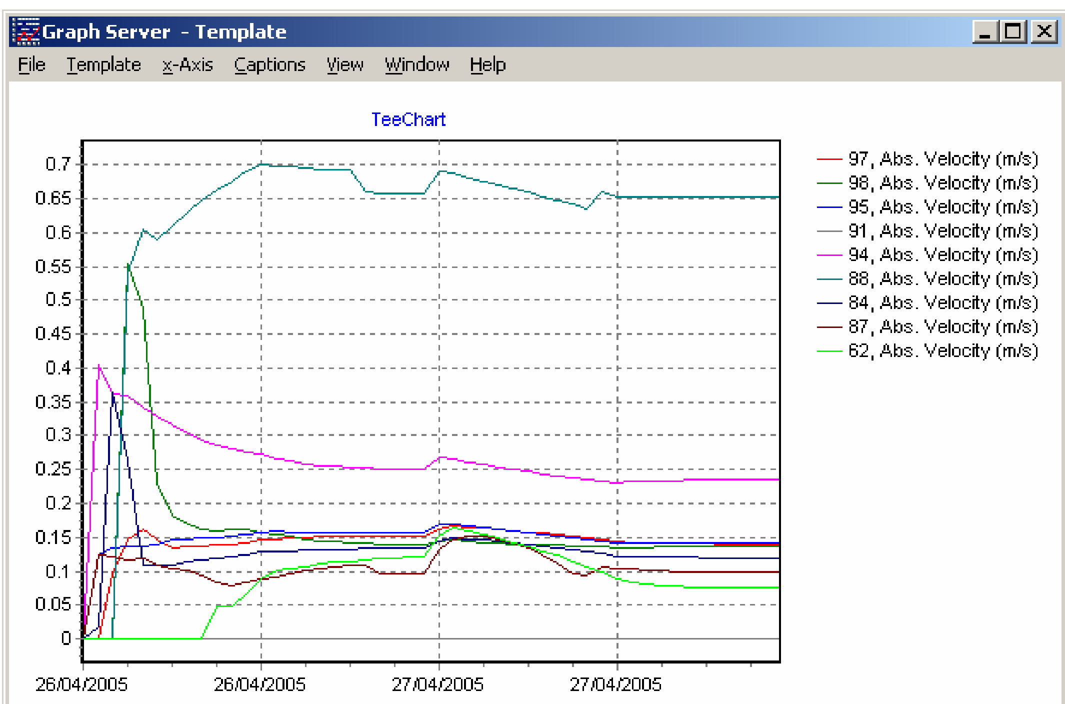

Figure 5-34: A figure showing the velocities for the 2005 flood event

5.7.4 Scenario Development

The flooded or inundated area from the Timis and Bega rivers was modelled and mapped out. The section of the flooded part of the river stretches from the gauging station Sag to Graniceri for Timis and Remetea to Otelec in Bega. The total length of this section is about 120Km. The area in between these rivers is a highly populated area with high economic value. The aim of the downstream 1D/2D modeling was to simulate various flooding extents and propose different methods of managing the flood. The following scenarios of mitigation measures were considered:

i. Base case with no mitigation measures
ii. Dyke breaching after arrival of the peak.
iii. Dyke breaching at the start of the flood event.
iv. Combined dyke breaching and storage.

Before an intentional dyke breach is made hydrological forecasts or measurements have to be done to realise that there is a flood event that might occur. Again, different authorities and people have to be involved directly or indirectly to give consent and/or be affected by the decision. Most importantly is when to take the decision of breaching and thus the flood time line becomes very important. Basically the flood time line from forecasts to the time the critical threshold is exceeded, needs to be determined so that decisions are not taken too early or too late, to ensure effectiveness. The whole of this process has been explained taking into account the key factors in decision making for flood management by intentional dyke breaching in chapter 6. Also in chapter 6 the 1D/2D model results of the different dyke scenarios will be presented.

6. Decision Support Tools for Flood Management: Application to Intentional Dyke Breach.

6.1 General

A decision support tool for flood management could involve the complete flow of information from obtaining the measurements in the field for the flood event or obtaining precipitation forecasts using them in a rainfall runoff model so as to transform the precipitation into discharge and routing the water through the river system as a one dimensional model to show the waterlevel at the key areas and finally showing the inundation of the floodplain using the two dimensional hydraulic model. Control and mitigation strategies can be applied and assessed to check for their effectiveness. The figure 6-1 below shows a flowchart for these processes.

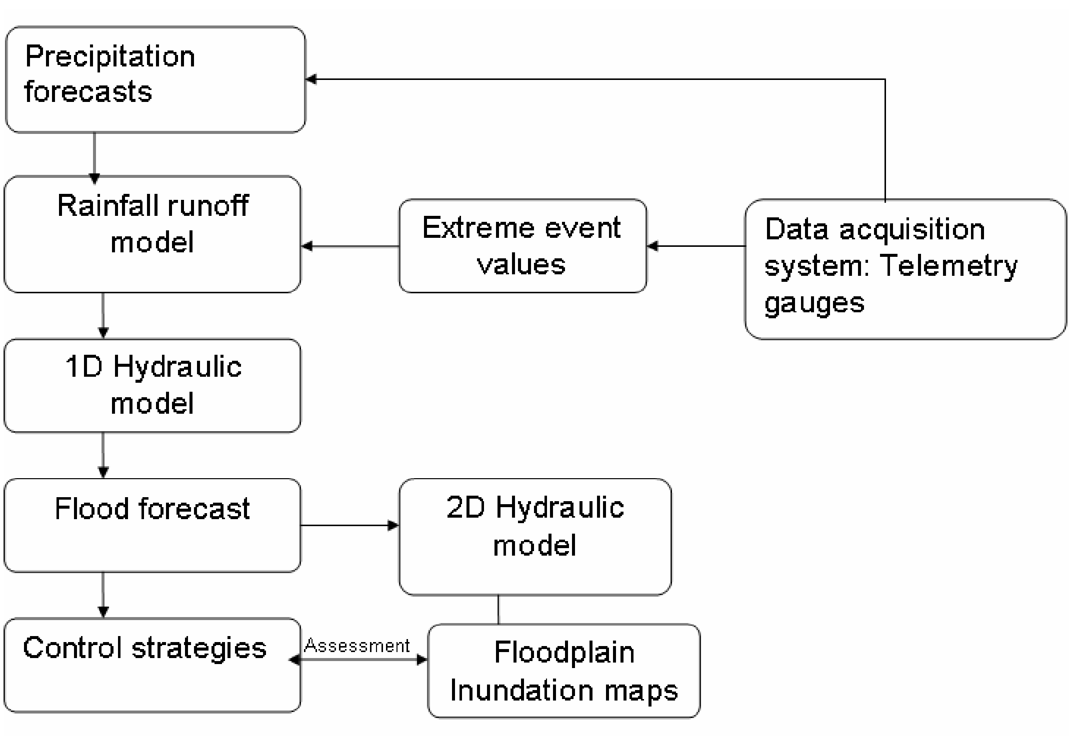

Figure 6-1: A flowchart showing the various components in a decision support tool for flood management.

The components of this decision support tool have been developed in this research for Timis and Bega catchments. In the following sections the tools are used to discuss important steps in deciding on intentional dyke breach for flood mitigation.
The section below explains the importance of forecasting in anticipating the flood, and shows the flood time line for Timis Bega basin based on the model simulations from the rainfall runoff model which showed the catchment response. In the final part SOBEK 1D/2D simulations of different dyke breach scenario's are presented. The floodplain inundation results show the effects of the mitigation strategies.

6.2 Hydrological forecasts and decision making

Introduction

Forecasting refers to the prediction of the behaviour of a system, and choosing the most efficient way to get the most reliable forecast of the observed variables. Decision making refers to the actions affecting the whole the system based on the information provided. Thus forecasting aids in decision making.

Hydrological forecasts have uncertainties from different sources such as measurement errors and modelling errors and the predicted variable can only be considered as 'most probable'. This means that the value is within a given range with a certain probability, see figure 6-2.

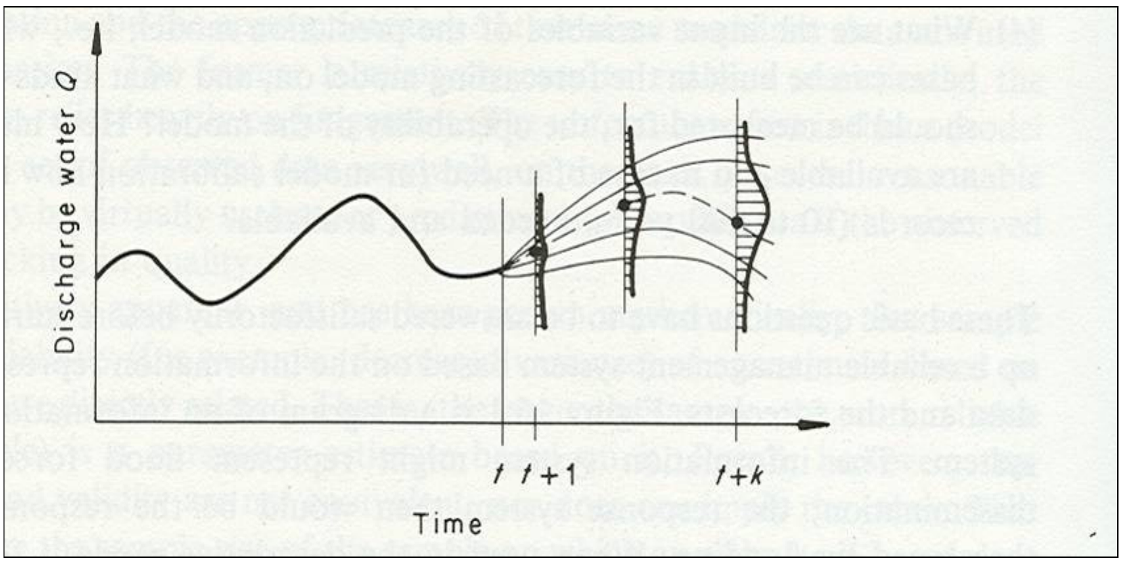

Figure 6-2: A figure showing error propagation in forecasts (Anderson & Burt, 1985)

The main problem between forecasting and decision-making is the probabilistic nature of the values that are forecasted. The decision makers would like to have a unique value without any probabilistic type of content. If this was to be provided then it means the whole forecast period is not considered, or that a method needs to be found with which the decision makers can make a decision, while taking the uncertainty in the forecast into account. As in fact there are methods to deal with decision making under uncertainty and these are used in forecasting practice.

River flow control has two objectives flood control and conservation control. Decision makers are faced hard with decisions when deciding about division of water resources among different users in low flows and even harder in case of a flood. Thus a good basis for decision making requires reliable, up to date information, which means good data collection and reliable hydrological forecasts.

During a flood event the main question that arises is that will the water level reach the critical threshold, such as the top of the levees or not. If the water goes higher then the question of evacuating the inhabitants from the area arises and more action is still required.

River flood control involves various measures for flood protection, ranging from operation of reservoirs to the breaching of dykes, if the flood still lasts. However a cost benefit analysis needs to be carried out to investigate the losses and advantages achieved. This cost-benefit analysis can be used to establish at what probability of exceedence an action needs to be taken – and thus allows the use of probabilistic forecasts. Autoregressive models can be used on the precipitation and the effect of uncertainties can be accounted for in the discharge. However, in this study a simple example is shown below to show the importance of a forecast especially on extreme events.

In an attempt to show that forecasting can be of great help in indicating that there is a potential flood, the flood peak for 2005 in Timis Bega basin was taken to be the perfect information thus was given P=1.0 and taking the uncertainties that can be expected in a good forecast such that the forecast is 95% or 110%. The flood peak was realised to be in a range of 1000 m3/s and 1300 m3/s, as compared to the threshold for the Timis River that is 140m 3/s, this already indicates a potential flood. This shows that with a good forecast the decision maker can start making decisions earlier from the forecast time for the anticipated flood. Figure 6-3 shows the Timis River peak for 2005 flood event and also the probabilistic discharges accounting for the 95% and 110% uncertainty. Again, this means that for extreme events not unless the forecast is worst like predicting it will not rain and it rains and vice versa, the flood peak can be indicated with probabilistic discharges because the discharge is too high as compared to the threshold such that a very poor forecast is the only one that cannot predict the flood. The effect of the forecast could be a taking a decision that is way too high or too low.

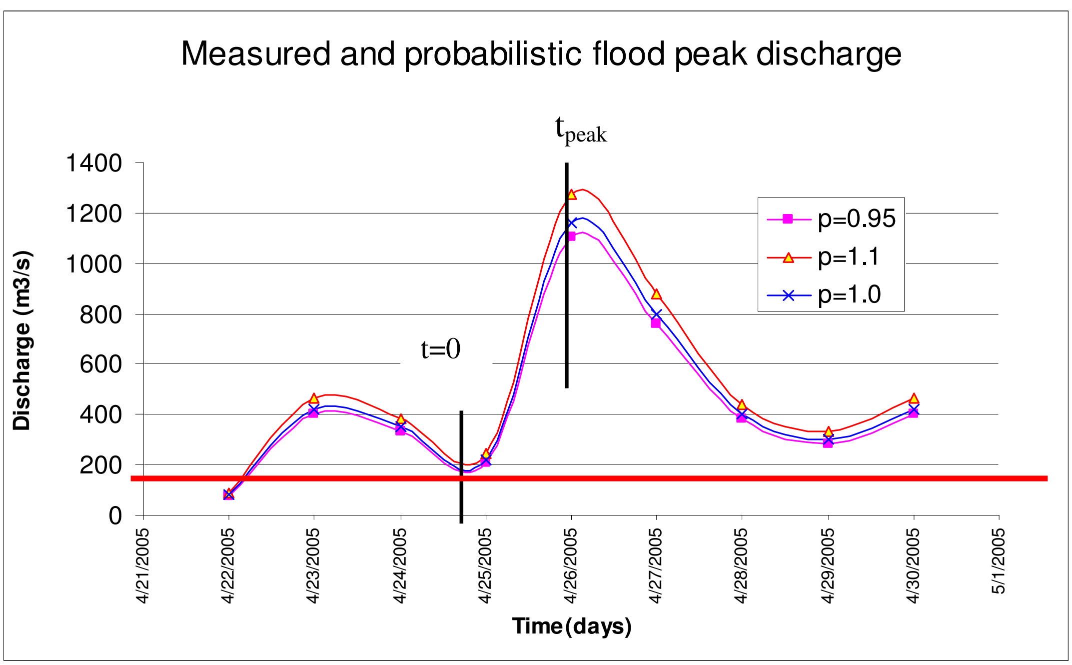

Figure 6-3: A figure showing measured and probabilistic discharges for a flood event

6.3 Hydrological response time and Flood timeline for Timis-Bega catchment.

When a flood occurs time is very important, if the flood is anticipated ahead in time lives and property can be saved by taking actions in the proper time. The flood timeline as shown in fig 6-3 below illustrates how this happens from precipitation either measured or forecasted to realising of the flood. The arrows from the top in this figure represent important stages of the flood in its occurrence. The first stage is the beginning of the precipitation that causes the flood, and the last is exceedence of a water level threshold at which property is damaged and lives are lost or sustain injuries.

The system approach to controlling a flood is based on the following five elements:

i. Data collection
ii. Model forecasts (Precipitation forecast, Transfer, Hydrological forecast)
iii. Information compilation (judgemental forecasts on the hydrological and environmental variables)
iv. Decision
v. Flood control measure (Implementation of the action).

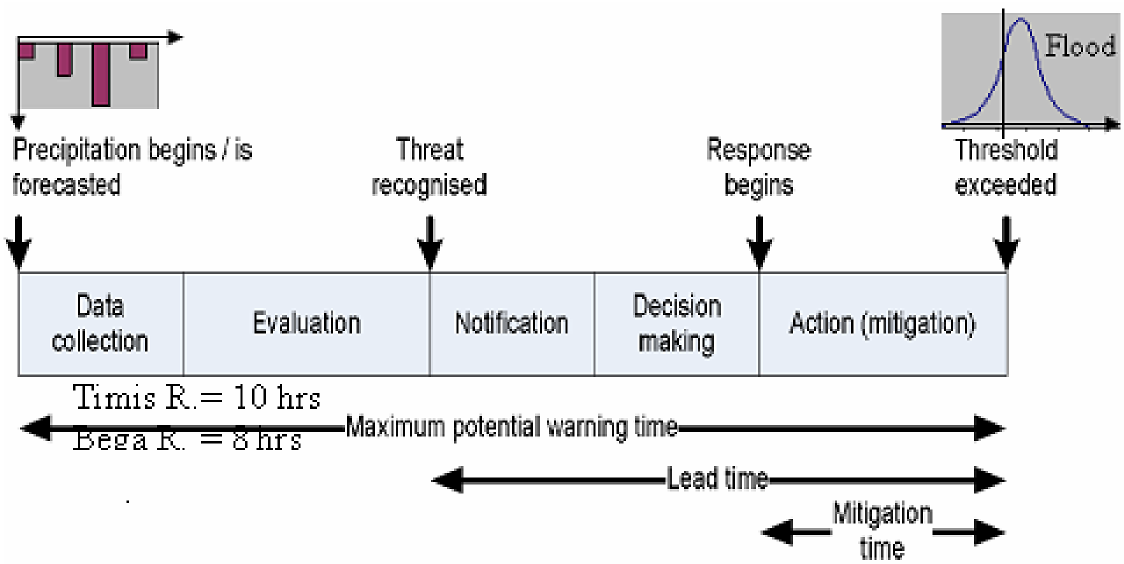

Figure 6-4: A figure showing the Flood timeline for a precipitation driven flood (from Verkade, 2007)

Actions can be taken between these flood stages to mitigate the damage to property and/or loss of life. The time between initiation of precipitation and threshold exceedence is the maximum potential warning time. This is the maximum time that is available for action. However, the maximum time varies from storm to storm and location to location within a watershed. In the Timis Bega the time it took for the 2005 flood event to arrive at the most downstream outlet on Timis R. was 8 hours and for the Bega R. was 6hrs. This time was established from the rainfall-runoff model as the time between the start of the rainfall event and the time of arrival of the peak discharge however due to the presence of dykes and restricted flow downstream, it took 2 hours for the water to rise above the dykes and start to overflow and finally dyke breaching.

From figure 6-3 the flood timeline first involves time that is required to detect the event, where hydro-meteorological data is collected and transmitted from sensors or gages in the field to a central site to be examined. This time is labelled Data collection, however this varies since in Timis Bega most stations are not automatic it required field collection of the rainfall measurements from the gauging stations. However precipitation forecasts were used in advance which gave some insight on what was expected.

The second stage as shown in figure 6-3 is the time required for Evaluation, of which requires application of knowledge to the data to create information necessary to recognize a potential or actual flood threat. For the Timis Bega catchment empirical models that relate water level at an upstream level to the downstream were used, where a flood is detected when water level gets to some level. For this study the integrated model was used for evaluation which entailed hydrological modelling and routing of the water through the river to obtain water levels at key points which was compared with bank elevation to determine if the level is near the threshold for overflow.

If a threat is recognized after evaluation relevant information is has to be provided to emergency responders and this is referred to as Notification time in Fig. 6-3. Once notified, the responders consult plans and policy and procedure manuals to make a decision about their response. They may, in turn, notify the public, who also will take time to respond to the threat.

However the authorities knowing the consequences expected from flood risk maps may want to take action to reduce the flooding effects to areas of high economic areas. With these the public or the areas to be notified maybe different as compared to if there was no action taken. The time required for different decisions to be made is referred to as Decision making in Fig. 6-3.

Finally before the water-level threshold is exceeded mitigation measures can be taken to reduce the flooding effects and this is referred to as Action mitigation time shown in Fig. 6-3. Since highly economic areas were to be affected by the flood, actions were taken so as to reduce the impact to these areas. Thus actions that would protect people and property from the rising water could include intentional breaching of dykes to areas that are of low economic values, and protect areas of high economic values. This was attempted in this study and it was realised that the areas that were to be affected could be reduced very much with intentional breaching just at the start of the rainfalls.

6.4 Flood mitigation by intentional dyke breach

The 2005 flood event was modelled taking into account the existing conditions as the base case and in other cases introducing various mitigation measures. The results are as shown below.

6.4.1 Base case

The 2005 flood event was modelled taking into account the breaching of three dykes on the Timis and Bega rivers (one on Bega and two on Timis River. The flood event was first modelled with no flood mitigation measures taken into account. The total inflow flood volume as shown in figure 6-4 was upto 300M m^3 and the outflow volume as was computed was 6.5M m^3, this means all the remaining volume inundated the floodplain.

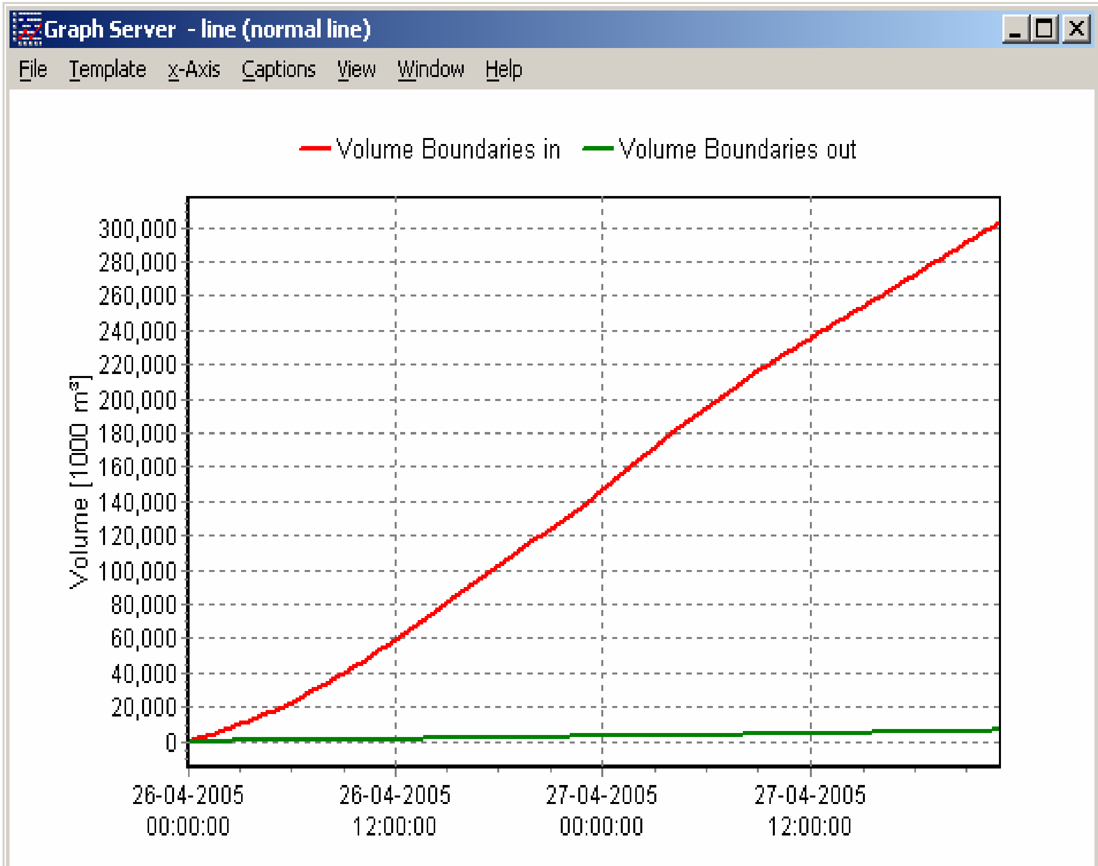

Figure 6-5: A figure showing the total volume of the flood as simulated into and out of the model

Figure 6-5 shows the flood map for Timis-Bega basin for 2005 flood event, it shows that an area of 22,631 ha was affected by the flood. The flood first overtops the the dykes and overflows then finally the dyke breach occur. The area in between Bega and Timis rivers was highly affected.

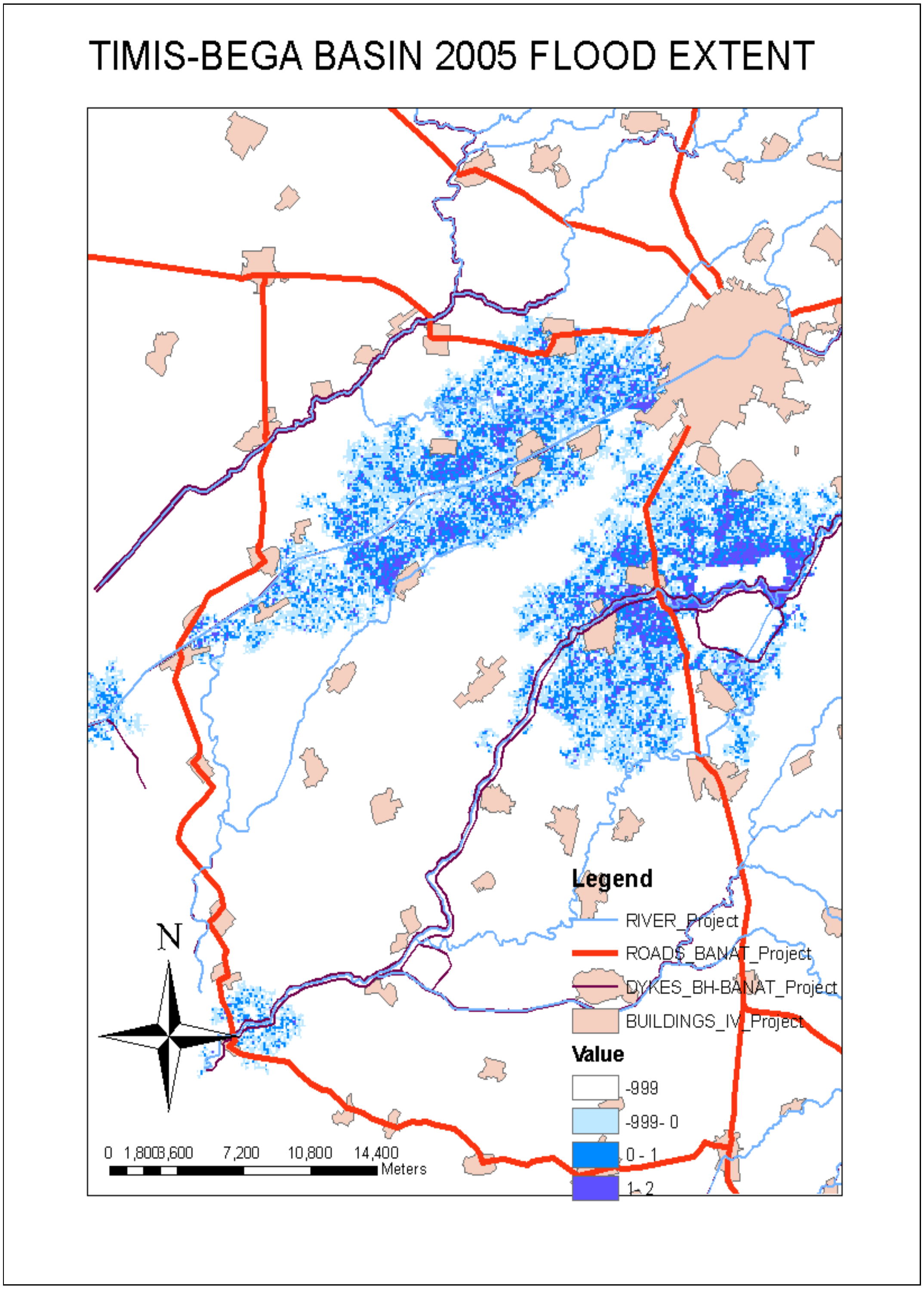

Figure 6-6: A figure showing the flooding extent of the 2005 flood event.

The total area that was affected by the flood was 22 631 ha, with 54% covered by a waterdepth of less than 1m, 33% of the area was covered by a waterdepth between 1 and 2 m, and the rest was covered by waterdepth that was more than 2m, see figure 6-6.

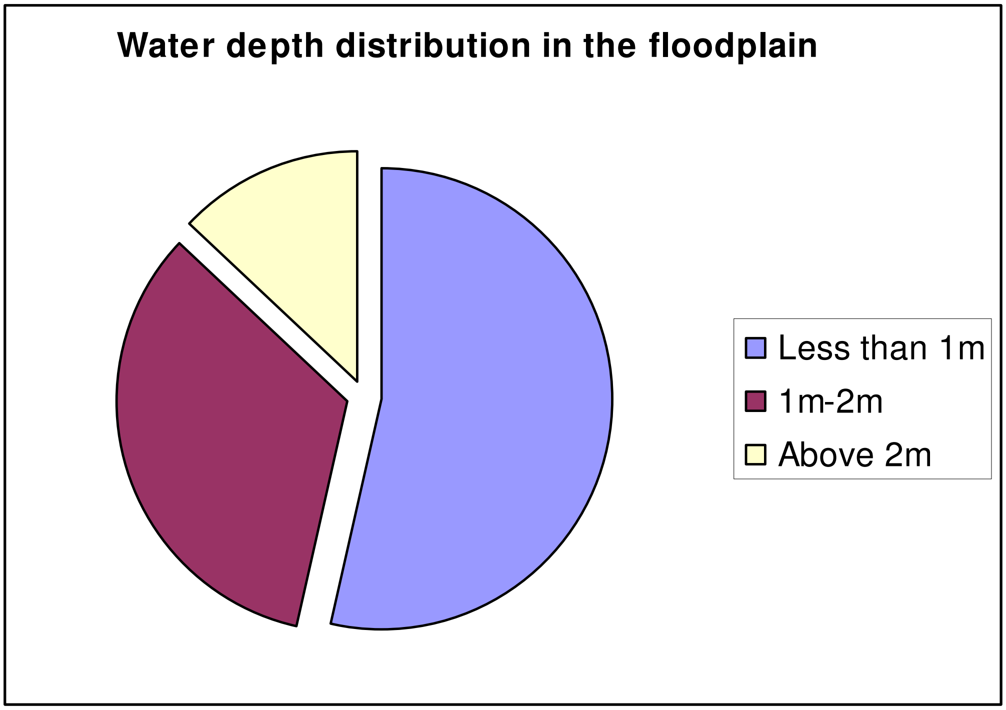

Figure 6-7: A graph showing the waterdepth distribution in the floodplain.

The total area that was actually covered during the flooding event was estimated to be 25000 hactares and the areas along the river due to overflow both sides (both left and right side of the rivers) were affected by the flooding due to overflow. However the area in between the two rivers Timis and Bega were most affected due to the major dyke breaches occured into the area. The model was able to show the areas that were affected by the flood, the total area is 22 631 ha which means that the model under predicted well with an error of about 10%.

6.4.2 Dyke breach after arrival of the flood peak

Figure 6-7 shows the 2005 flood map after intentional breaching was done after arrival of the peak Timis-Bega catchment, it is realised that the area that is affected is 54% of the base case. However taking a mitigation measure of dyke breaching reduced the flooding effects since the total area that was affected was 12,390 ha. This is shown in figure 6-7; however the area in between Timis and Bega was still affected.

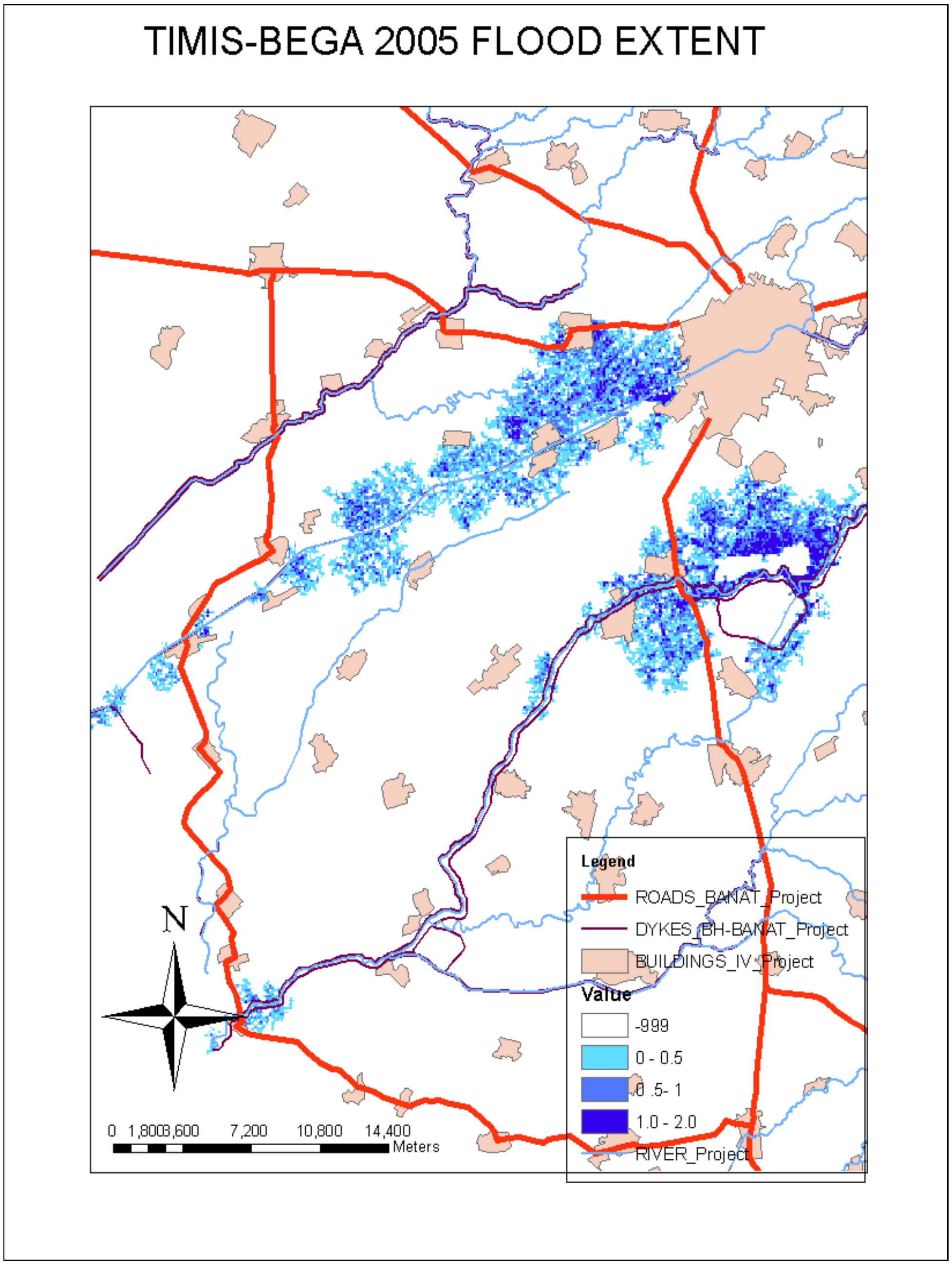

Figure 6-8: A figure showing a map of the flood extent with dyke breach after peak arrival.

6.4.3 Dyke breach at the start of the flood event

The effect of dyke breaching at the start of the event can be seen in figure 6-9. Actually it shows that the flood extent can be reduced considerably to the Timisoara city and also the area in between the Timis and Bega rivers.

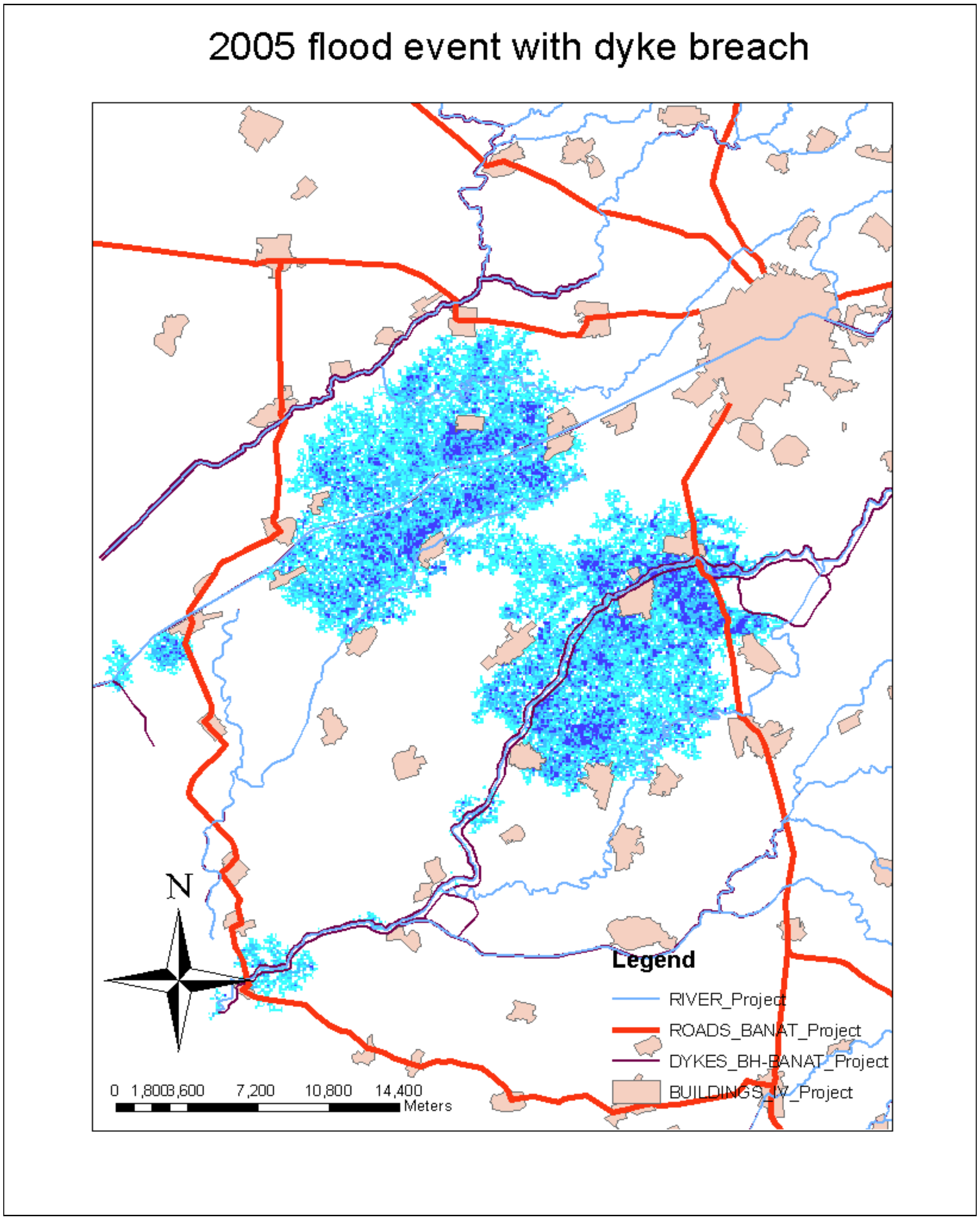

Figure 6-9: A figure showing a flood extent map with dyke breach at the start of the flood event.

6.4.4 Dyke breach and storage

The available total storage in the reservoirs in the Timis Bega catchment was evaluated to acertain to how much the flood volumes could be attenuated. The main permanent reservoirs in Timis-Bega catchment are Poiana Marului and Surduc. There are also 13 small non-permanent reservoirs. The location of these various reservoirs is presented in Figure 6-10. Poiana Marului Reservoir is located in the Bistra basin which is upstream of Timis River corresponding to catchment 6a, 6b and 6c in the rainfall runoff model, see figure 5-3. The Surduc Reservoir is located upstream of Bega river and has basin area of 135km^2, this corresponds to catchment 17 in the rainfall runoff model shown in figure 5-3.

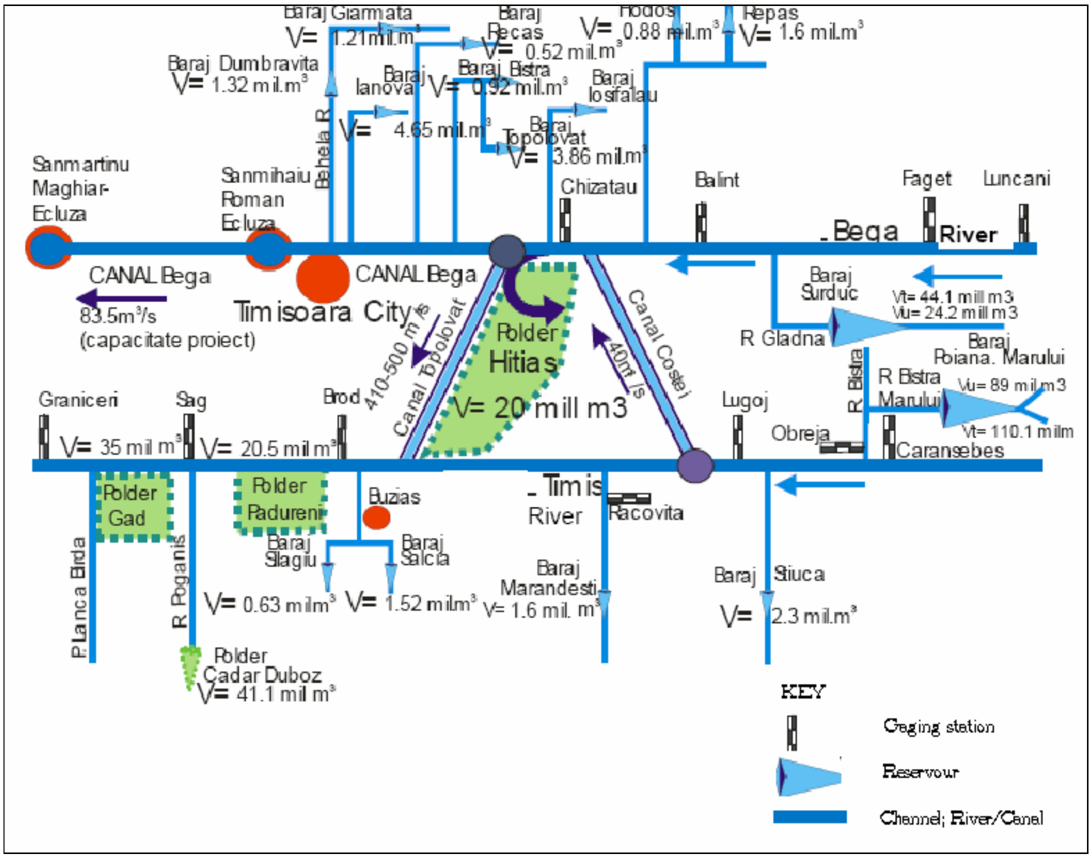

Figure 6-10: Schematisation of Tims-Bega showing all reservoirs in the catchment

The capacity for different reservoirs is given in table 5 below and it shows that the total capacity of all reservoirs is 284.676 Million m^3. According to the previous studies by Stănescu V. A&, Drobot R. on the 2005 flood event it was realised that the main permanent reservoirs Poiana Marului and Surduc retained a total volume of 25.9 M m3 (13.5 M m3 in Poiana Marului Reservoir and 12.4 M m3 in Surduc Reservoir).

The non-permanent reservoir Cadar Duboz Storage which has a capacity of 41.1 M m^3 on Pogonis River was very efficient in attenuating the high peaks of this river. Also the Padureni non-permanent storage stored 20 M m3 while Hitias non-permanent reservoir located at the confluence of diversion Canal Topolovat stored 5M m3, Nicoara I., (2005).

Table 5 :A table showing reservoirs in Timis-Bega with their capacities. Nicoara I., (2005).

BASIN	Permanent reservoirs Capacity (M m^3)		Polders Capacity (M m^3)		Non-permanent reservoirs Capacity (M m^3)
TIMIS	Poiana Marului Rusca Others	89.00 16.86 3.825	Paruden Gad	35.00 20.50	13.635
BEGA	Surduc Others	44.12 11.04	Hitias	20.00	8.896
	Total	**166.645**		**75.00**	**22.531**

In attenuating the 300M m^3 flood as for the 2005 flood event simulation, it required all reservoir capacities to be available for the flood volume to be managed. However these reservoirs especially the high capacity ones are multi purpose such that they are used for water supply, flood control and other functions.

Each of these reservoirs is located in a different subbasin, which means that every reservoir could be operated at a subbasin level so as to attenuate the flood volume before arriving in the main rivers Timis and Bega.

Taking Surduc reservoir that is located in Firdea which is C17 in the rainfall-runoff model. The contribution of this catchment into the 2005 flood was modelled and the hydrograph is as shown in figure 6-10, a flood peak of 350 m3/s could be attenuated by operating Surduc reservoir that has an area of 135 km2.

Surduc reservoir has a capacity of 44.1M m^3, flood control is only possible if the reservoir has some capacity available to hold the flood volume. However an effective way could be after measuring or forecasting the precipitation the peak runoff is anticipated and diverted into the reservoir. As show in figure 5-27 below case 1 if the reservoir could be operated to take upto 75% of all the flow through R. Gladna before joining Bega R a volume of 10 M m3 could be stored.

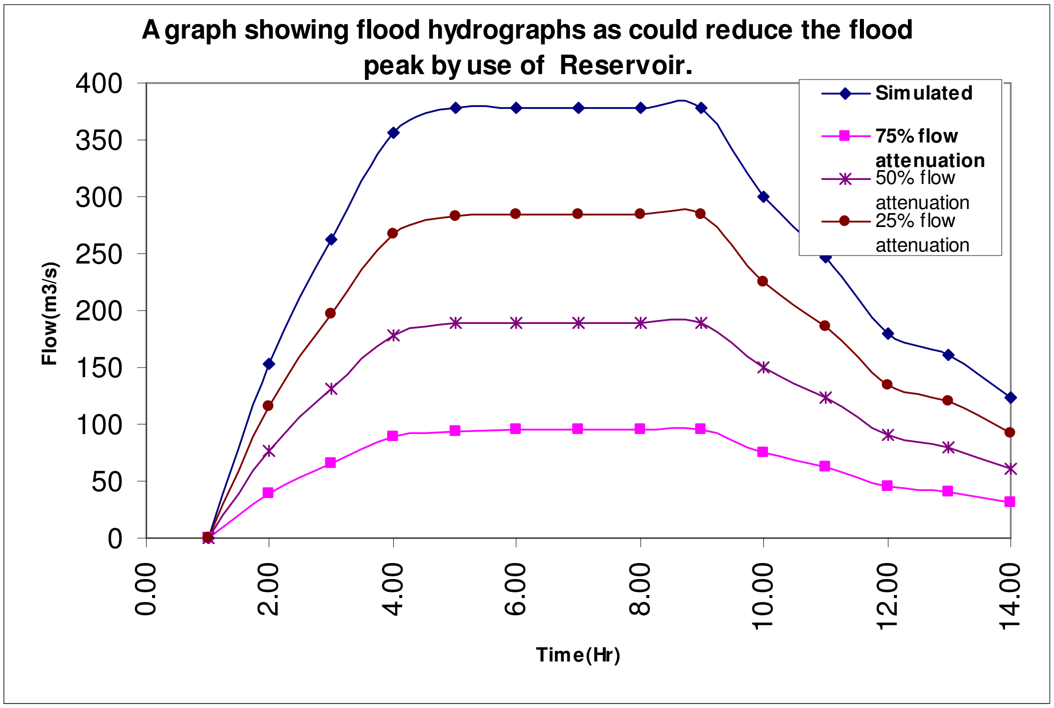

Figure 6-11: A graph showing the effect of storage in attenuating the flood peak by reducing the flow percentage at the start of the event.

In case 2 the river is allowed to flow during the peak time with one half of the total flow. While in case 3 the river flows with 75% of the flow, thus storing 25%. The table 6 below shows a summary of the total volume stored in the reservoir and volume through the river for all the cases. The total volume of runoff modelled from precipitation is 13.2M m^3. From table 12 it can be realised that the reservoir has a potential to attenuate the flood peak however this is possible if the storage volume is available before the peak arrives.

Table 6: A table showing various cases for reservoir attenuation of the peak

	Volume through the river(Million m3)	Volume stored in Reservoir (Million m3)
Case1: 25% through the river	3.3	9.9
Case 2: 50% through the river	6.6	6.6
Case3: 75% through the river	9.9	3.3

From the above table it can be seen that with more storage upstream the flood peak at the downstream can be attenuated.

Finally the reservoir storage was combined with dyke breach. A total of 25% of the flood peak volume as modelled which was 300M m^3 was stored in the reservoirs. Figure 6-12 shows the flooding extent after taking an action of intentional dyke breaching at the dyke along Timis River and Bega River and also storing 25% of the total flood volume. This means 75M m^3 were stored by the reservoirs however this was taken as the storage available from the polders since they are used for flood control. What can be realised is that the flood extent was reduced from reaching Timisoara city and also the flood to the area in between Timis and Bega was reduced. However communication networks especially from and to Timisoara were affected both when there is a mitigation measure and no mitigation measure. Even in case it is realised that the area in between Timis and Bega has o flood this is because of low elevations, thus overflows occur into this area first. However, the affected area was reduced significantly to 7,789 ha which is one third of the base case.

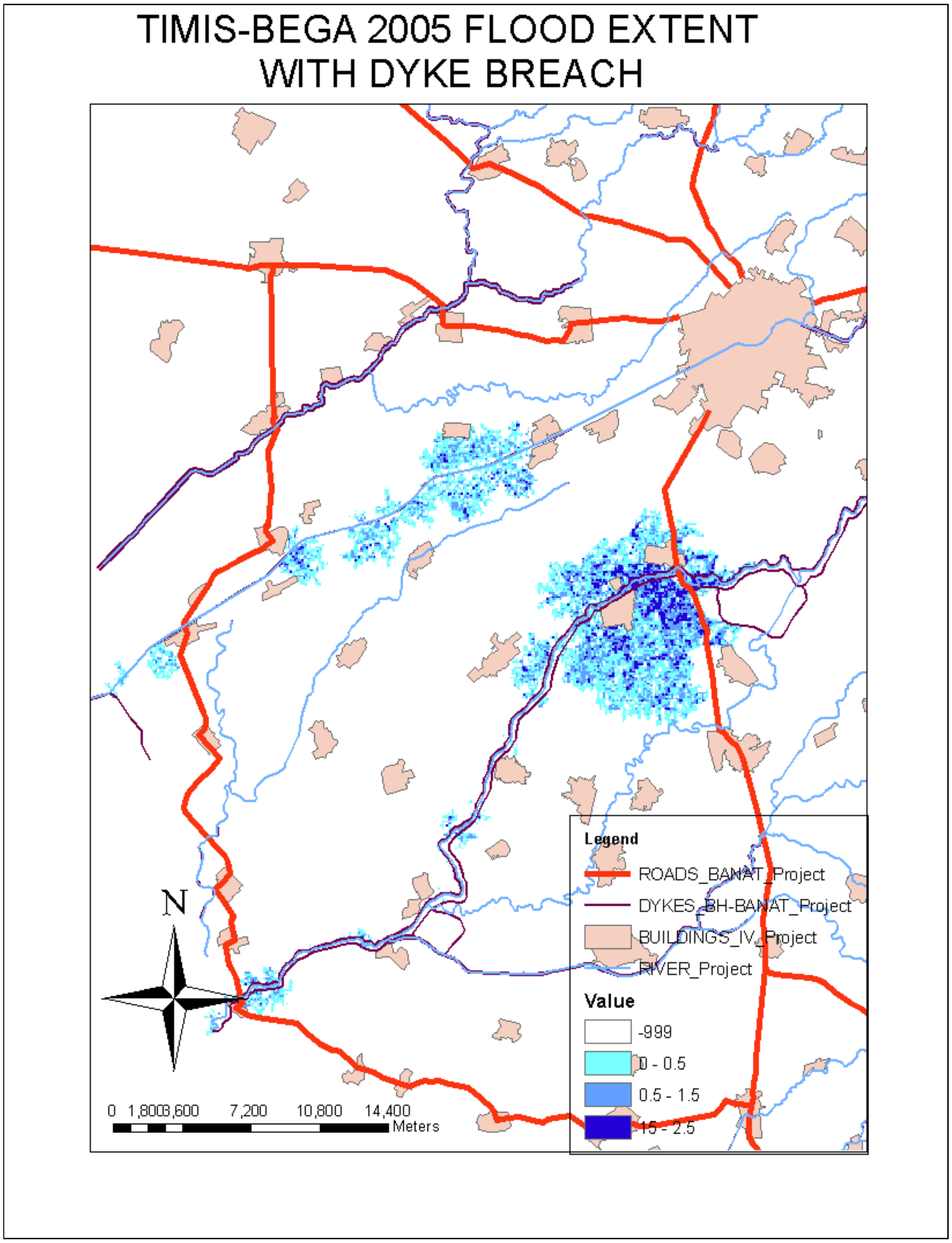

Figure 6-12: A figure showing a map with the flood extent when dyke breach and storage are provided as mitigation measures.

Figure 6-13 shows a chart that compares the flood affected areas in terms of waterdepth and how the various mitigation measures reduced the flooding extents. With intentional dyke breaching the affected areas were reduced in Dyke breach 1 to 12,390 ha and in Dyke breach 2 the affected area was 8,687 ha and finally dyke breaching with storage combined reduced the area to 7,789 ha. The most effective way to reduce the impacts of the flood as shown could be the combined dyke breach and storage of the water. Dyke breach 2 refers to breaching at the start of the event and 1 refers to breaching after the arrival of the flood peak.

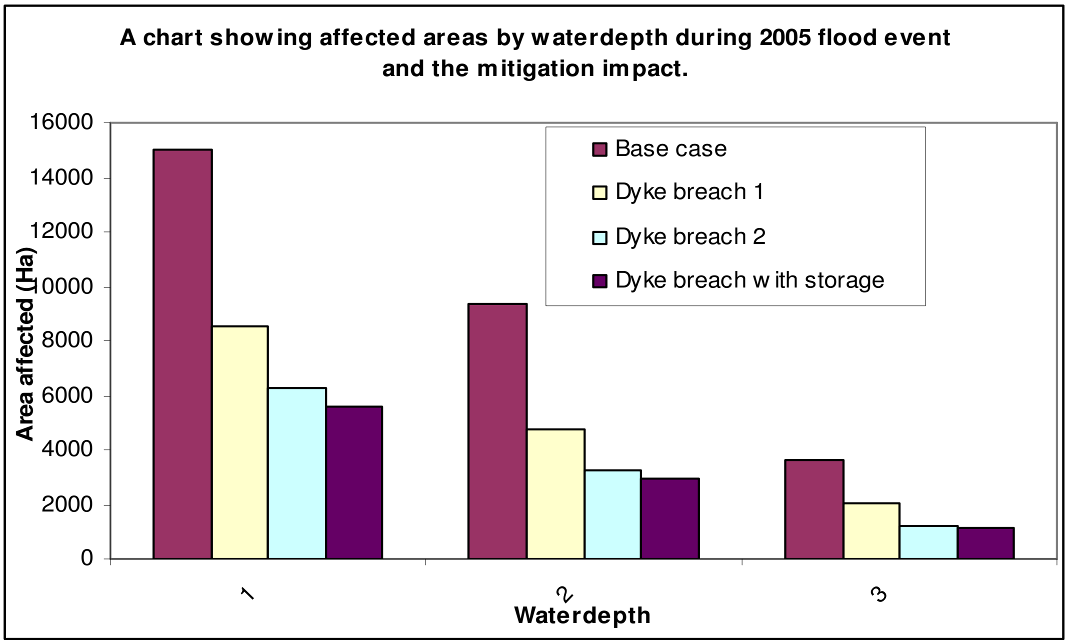

Figure 6-13: A figure showing the effect of mitigation measures in reducing the flooding extent.

The velocities are attenuated as the flood water flows downstream and as it moves to the floodplain. Figure A-9, A-10 in the Annex show the velocities as were obtained from the different roughness coefficients and it was realised that the velocities are higher when the roughness coefficient is low and vice versa.

7. Applicability of Research Study on Nzoia catchment in Kenya.

7.1 General

This study was carried out with the intention to investigate it's applicability in the Nzoia catchment in Kenya. Nzoia catchment is a tributary to Lake Victoria where the White Nile originates. Flood management is one of the key issues that are addressed in the Nile Basin. The Nile Basin Initiative looks forward to developing a decision support system in managing the water resources, thus this study explored the possibility of using integrated modelling for flood analysis and mitigation to demonstrate how modelling can support decision makers in flood management. An assessment on the applicability of this research to Nzoia catchment has been described in this chapter.

7.2 Location

Nzoia River basin lies in the western region of Kenya, it is bounded by latitudes 1^{o} 30'N and 0^{o} 30'S and Longititude 34^{o} E and 35^{o} 45'E. The Nzoia River rises from Cheranganyi hills and it flows into Lake Victoria with tributaries feeding it from Mount Elgon in the North. The basin covers an area of about 12709km2 and a total length of 334km upto its outfall into the lake. The downstream part is generally flat and swampy covering a total area of 25 km^2 with it is soils poorly drained and mainly of clay type. Figure 1, shows the location of Nzoia basin on the map of Kenya.

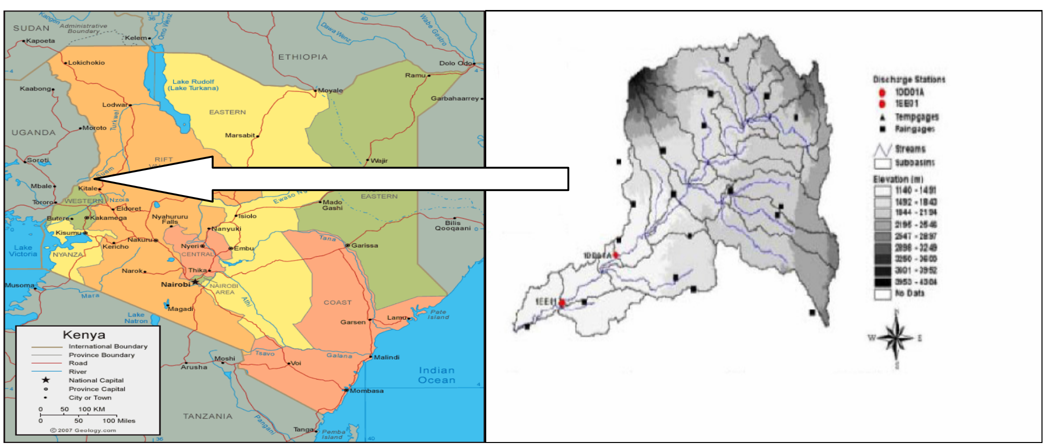

Figure 7-1: (a) A map of Kenya (b) Nzoia basin with various gaging sations. (Source:http://geology.com/world/kenya-satellite-image.shtml)

7.3 Main Land Use

Landuse varies throughout the whole basin but agriculture is most dominant. Livestock rearing and fish farming are common activities. The main crops that are grown in the area include cotton, maize and sugar-cane. However agro-economic conditions are generally poor throughout the area.

7.4 The Hydrology of River Nzoia Basin

The mean annual rainfall is between 1076 to 2235mm with a catchment average of 1424mm. (Githui, 2007).The rainfall trend represents two maxima and minima over the year. The First and Second maxima occur from April to May and July to November respectively. The highest river discharges occur between May and September while the lowest river discharges occur between January and March.

7.5 River Nzoia Floods

River Nzoia is characterized with flooding in its lower reaches. There is intense erosion in the upstream region due to deforestation. The deposition of this material eroded in the upstream takes place in the downstream due to the low gradient of the riverbed (Makhanu, 2005). The deposition of this material on the channel reduces its depth and consequently it is capacity and hindering the free flow of water. Dykes have been constructed over 32.8 km stretch (16.2km on the Southern side and 16.6km on the Northern side of the River) in the downstream of the river Nzoia to contain the flood problem, but the floodwater breaks them at weak points perennially. Flood disasters have occurred in 1945, 1948, 1951, 1961 – 1962, 1975, 1977, 1978, 1997 –1998 (El Nino rains), 2001, 2002 (Mango, 2003) and 2003. The frequency of the flood disaster has tremendously increased due to increase in the population exposure to the flood hazards as human settlements and crop farming encroach the river plains.

7.6 Application of integrated modelling for flood analysis on Nzoia catchment

In an attempt to assess floods in Nzoia catchment, integrated modelling can be used where catchment hydrological modelling is used for the subbasin routing, a 1D hydraulic model is used to rout water through the channel and consider the lake water tidal fluctuation (level fall and rise), and a two dimensional model to show the extent of flooding in the floodplain.

As in the case in Timis Bega the downstream floodplain is protected from floods by use of dykes, of which during floods they breach. In Nzoia River the dykes do breach annually due to floods, due to this similarity in the catchments. The Timis case study can be used a study and it can be applied on the Nzoia River.

A descriptive procedure has been explained on how to integrate these various models to show the floodplain flooding extents in section 5.5. However it is important to point out the main points that need to be considered and as well the data required.

In developing a rainfall runoff model the following data has to be provided: rainfall time series, evaporation, groundwater contributions, and infiltration. Land use maps also can be helpful identifying the various crops, crop coefficients and the roughness of the area. The model has to be provided with precipitation and potential evapotranspiration to compute losses and runoff. However in calibrating the model ‘soft values’ (these are values that can be field measurements) can be used so as to narrow the range of the optimisation algorithm, these results in easier and quicker calibration.

For the 1D model the data that has to be provided include river cross sections, river reach lengths, and roughness coefficients for the physical parameters. Boundary and initial conditions are also very important because the river system has to have the right upstream and downstream conditions which can be flow and stage hydrographs. Cross sections have to be provided at critical points especially at the confluence and if the river is straight with no major changes then the cross sections that are available can be used and interpolation can be done. In calibrating the 1D model the roughness coefficient can be used.

For the 2D model, since the flooding occurs from the river which is a 1D channel into the floodplain that is 2D then a Digital Elevation Model grid has to be provided, the following data have to be provided: the river cross sections reach lengths, DEM grid and land use maps. The roughness coefficient varies from the river to the floodplain and this has to be accounted for. In modelling a dyke breach a weir was used in this study where the controller was lowered, so this can be used especially where timing of the breach is done.

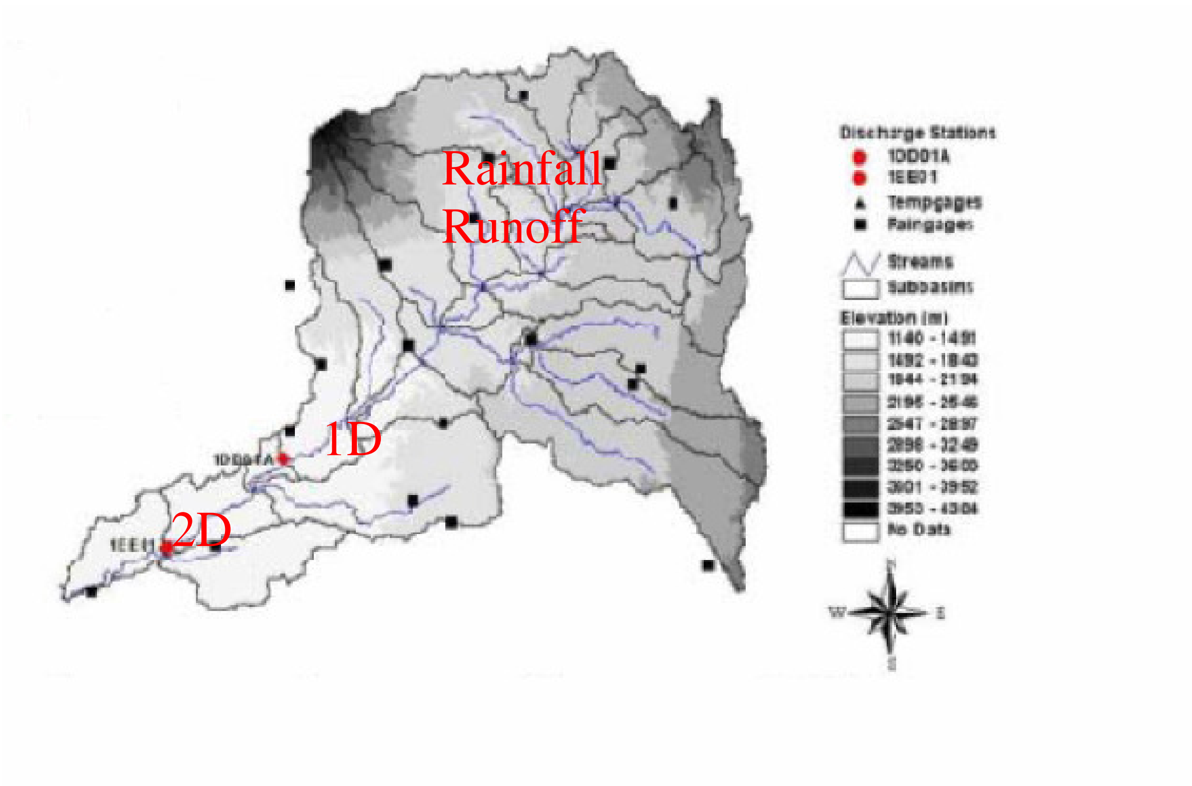

Figure 7-2: Figure showing Nzoia catchment with the interrelation of various models.

The Nzoia catchment can be modelled using the integrated approach amidst data scarcity. For the River network cross sections can be interpolated and free internet sources can be used for data filling and checking.

8. Conclusions and Recommendations

Conclusions

This research contributed to the attempt to understand how extreme flood events are generated, propagate and inundate rivers and their floodplains. This was applied in Timis Bega river basin and a descriptive explanation of the applicability on Nzoia river basin was provided.

The research work included the use of the rainfall runoff, 1D and 2D models to develop a integrated model that will serve as an evaluation tool for flood forecasting at key areas, show flooding extents and evaluate proposed flood mitigation measures in the lower part of the Timis-Bega river basin. The following conclusions were drawn from the analysis and results that were obtained from the research:

From the rainfall runoff model it was realized that in Timis-Bega the catchment is under the influence of both rainfall and snowmelt, where the snowmelt raises the soil moisture conditions and increases runoff. In calibrating the model it was observed that the period between March and May when the temperatures start to rise in the spring, the model was not able to predict this precisely. On the Timis River the calibration correlation coefficient was 0.55, but Bega River had a better correlation coefficient of 0.72. For the 2003 data that was used to calibrate the model the peak runoff as simulated for Timis was 87.20 m^3/s and 45.30 m^3/s for Bega and the observed values were 94.70 m^3/s and 52.10 m^3/s respectively. The Bega River had a total simulated outflow of 168.20 mm while the observed was 148.46 mm this means the model over predicted. For Timis river the total simulated outflow was 224.43 mm and the observed flow was 268.84 mm, which was underestimated. The root mean square error was obtained to be 11% and 2.7% for Timis and Bega rivers respectively.

Due to the major backwater effects downstream, the rainfall-runoff model was calibrated at gauging stations that are not influenced by the backwater, for Timis river this was Lugoj station and Bega this was Balint station. The catchments that were downstream of these stations were added as lateral flow into the 1D model. The model was able to account for the downstream conditions; however there was no data for the downstream to calibrate this model, although the roughness sensitivity was conducted. The results from the 1D model were used as the 2D upstream and downstream conditions. From the 2D model the inundated areas were identified for the flood event of 2005. The flood inundated area from the model was 22 631 ha and the assessment after the occurrence of the flood realized an area of 25 000 ha, the model prediction showed a 10% error.

Further, due to the inundation occurring in more economic areas mitigation measures were applied to reduce the flooding effects into these areas. Due to the 2005 flood occurred with major dyke breaches, intentional breaching was used as a mitigation measure, and breaching into areas that are considered to be of low economic value from the land use map, just before the flood peak arrived. It was observed that the flood extent was reduced considerably especially in between the Timis and Bega rivers and the Timisoara city which are the areas to be highly protected form the flood. However when both storage and dyke breaching are combined the areas that are affected by the flood are reduced by 62%.

The flood time line for Timis-Bega basin was determined from the 2005 flood showed that there were 10 hours in Timis R. and 8 hours available as the maximum possible warning time after the falling of the peak rainfall to the arrival of the runoff peak at the outlet and overtopping of the dykes. However it should be noted that this time varies depending on the antecedent conditions of the catchment. The time for mitigation could be taken as 4 hours when the rainfall was still on the rising limb.

The models sensitivity showed that the roughness coefficient is a key factor. The rainfall runoff model showed that a higher roughness coefficient on the catchment attenuates the peak, while a lower roughness coefficient increased the peak discharge. This was the same for the one dimensional hydraulic model which showed that the main channel peak discharge increased with decrease in roughness coefficient and vice versa. The two dimensional model showed sensitivity to the roughness, where a higher roughness in the floodplain reduces the flow velocities thus the extent of flooding is reduced and lower floodplain roughness showed increased velocities and increased flooding extents.

In assessing extreme flood events it is not easy to obtain the exact initial conditions which mean the model ends up having uncertainities. This was commented by Buchele (2000) and this study also agrees that reconstructing extreme events results in uncertainities due to lack of historical prevailing initial conditions.

In assessing this integrated model to function as a decision support system it was realized that it requires a general user interface that generally puts everything together such that all the models can be run and provide water levels at key sites. This was not considered in this study.

Flood modelling using the integrated approach for the Nzoia catchment can be done; with data scarcity, free data sources can be used. For the physical parameters like cross sections, interpolation can be used in case they are not sufficient. The integrated modelling approach can act as a decision support tool. Especially if forecasts are available, the models can be used to show the vulnerable areas, thus warnings and mitigation measures can be earlier planned for.

Recommendations

Based on this study the following recommendations can be considered in any further study related to the Timis Bega catchment:

1. The rainfall runoff model needs to account for the snow part since it plays a role in the hydrograph and maybe it needs to be quantified to what percentage the do rainfall and snow contribute.

2. More data is required to further calibrate and validate the rainfall runoff model.

3. A different model that accounts for restricted flow as a downstream boundary condition needs to be used to really account for the downstream restrictions.

4. A more elaborative land use map needs to be used to precisely identify areas that are affected by the flood.

5. A flood damage analysis needs to be carried out, to assess the effects of the flood so as to assist in decision making.

6. For the decision support system to function well it needs a user interface that integrates all the components and makes them run automatically, thus this can be carried out in another study.

7. Model uncertainties from such a cascaded model can be high and thus an assessment needs to be carried out.

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Annex

The following table A-1 shows data that was used for this study, it includes rainfall, waterlevel, cross-sections, temperature and DEM of 30 x30.

Table A-1: Data used for Timis-Bega catchment flood evaluation

Data type	Station	Period	Remarks
Water level	LUNCANI FAGET BALINT CHIZATAU TOPOLOVAT POIENI TOPOLOVAT FIRDEA GHIZELA RACOVITA TEREGOVA SADOVA CARANSEBES LUGOJ OBREJA CHEVERES RUSCA FENES GOLET BUCOVA VOISLOVA BUCOVA VOISLOVA	**2003**	**Rating curves were used to obtain discharge**
Rainfall	**Same as above**	**2003**	
Cross section	**At discharge stations**		
Temperature	**Min and max temperatures**		
Maps and DEM	**SRTM 30 x 30 DEM**		

Catchment Delineation

Catchment delineation was achieved using HEC-GeoHMS as an extension in ArcView 3.3. After Filling in sinks to the grid, the river network that was provided as shapefiles by the Romanian Water were used in generating an **AgreeDEM**.

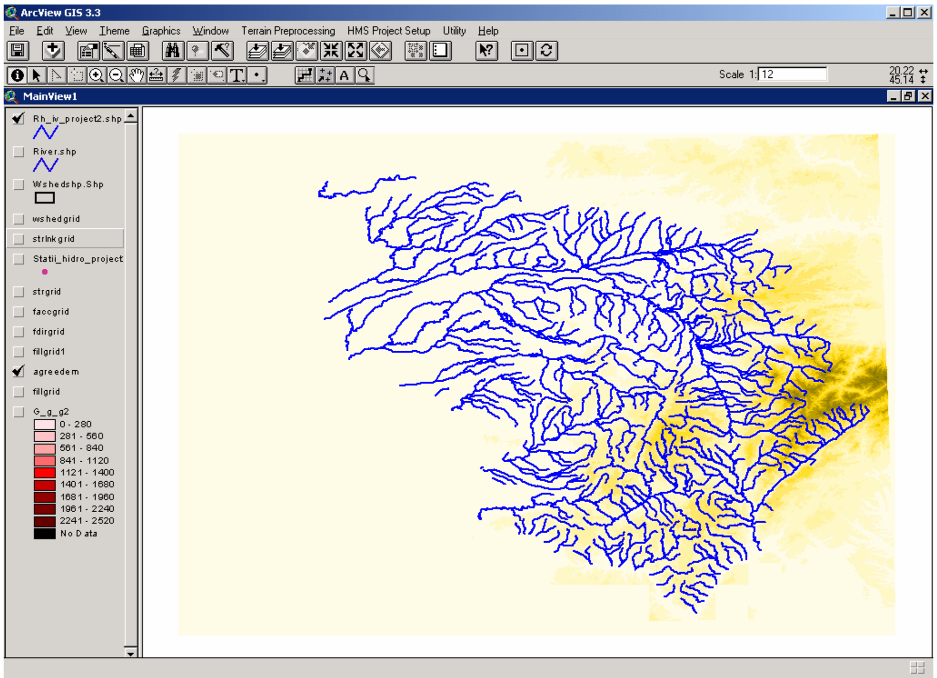


Figure A-1: A figure showing an Agreedem after filling in sinks in ArcView 3.3

After filling in the Sinks the ***Flow direction*** of the Dem will be determined.

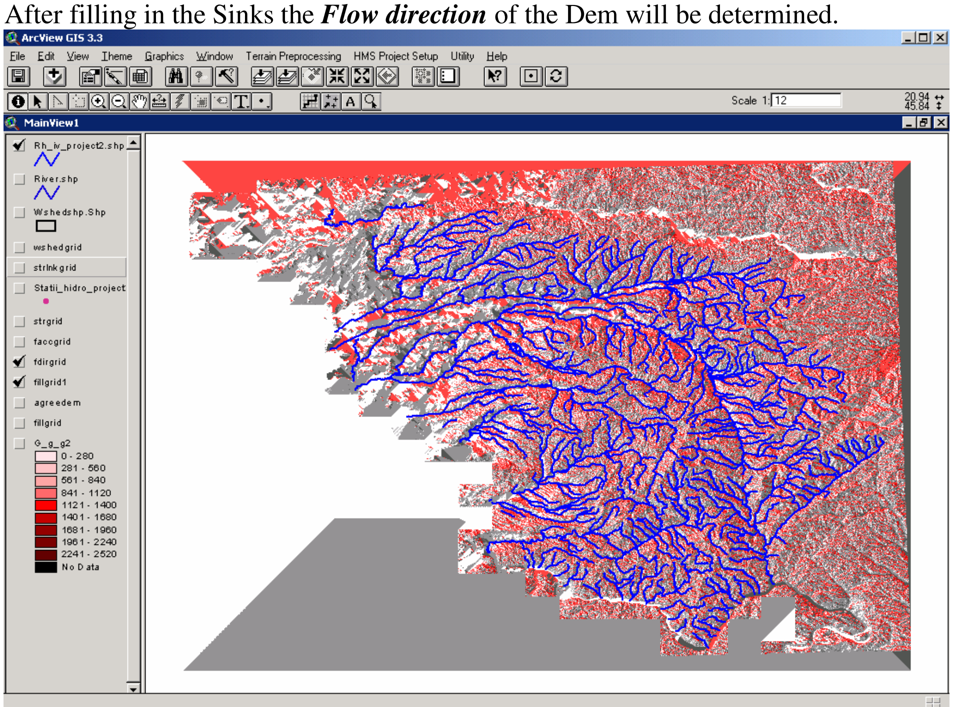


Figure A-2: A figure showing flow direction as computed in ArcView 3.3

The ***Flow Accumulation*** tool is also executed after the direction has been determined. This computes the associated flow accumulation grid that contains the accumulated number of cells upstream of a cell, for each cell in the input grid.

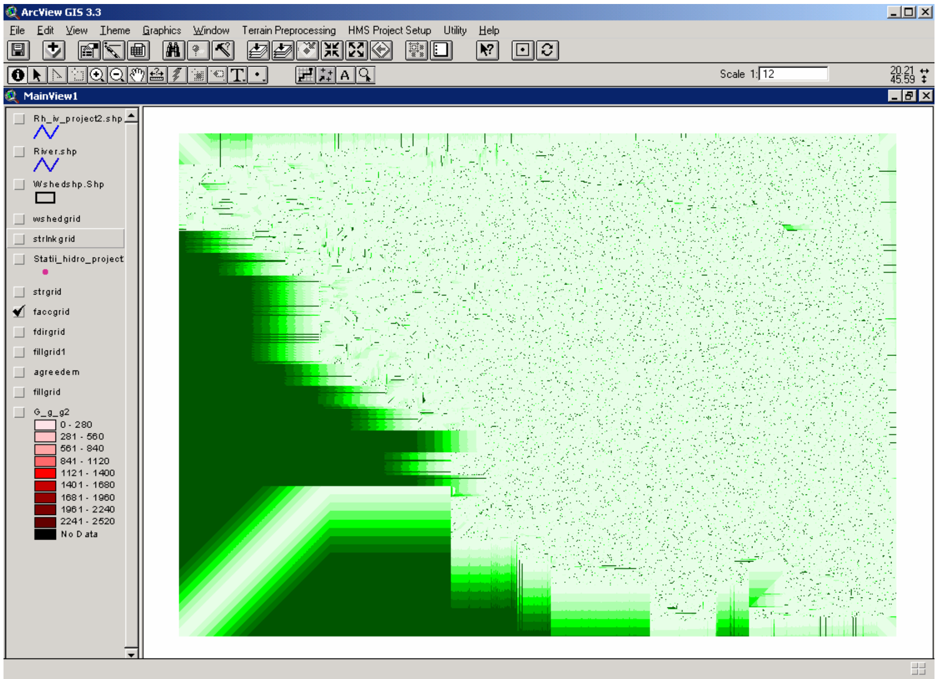


Figure A-3: A figure showing flow accumulation as computed in ArcView 3.3

The delineated catchment is as shown in figure A-4.

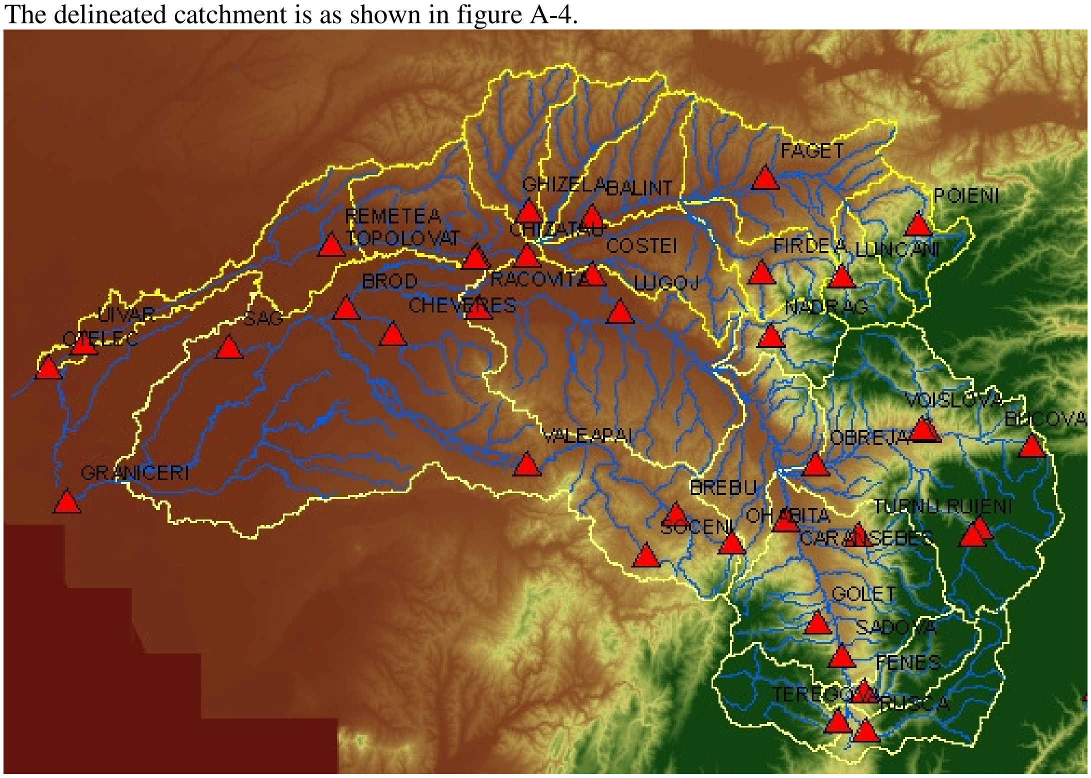


Figure A-4: A figure showing watershed grid as computed in ArcView 3.3

HEC-HMS DATA PROVIDED INTO THE MODEL

The model schematisation in HEC-HMS is as shown in the figure below.

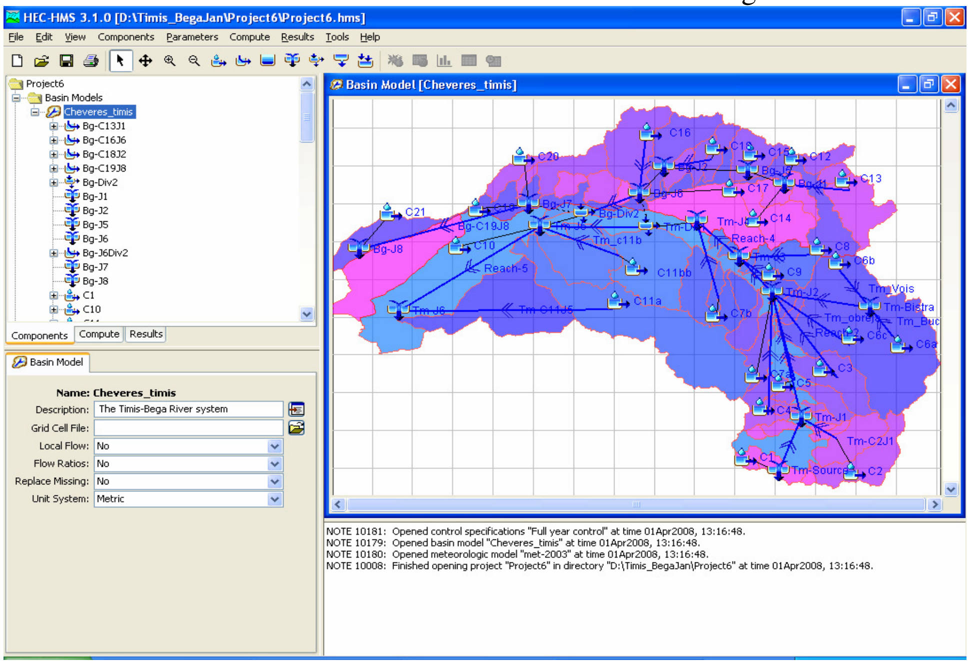


Figure A-5: A figure showing HEC-HMS model schematization

The precipitation data as was provided to the HEC-HMS model is as shown in the graph shown below from HEC-DSSvue.

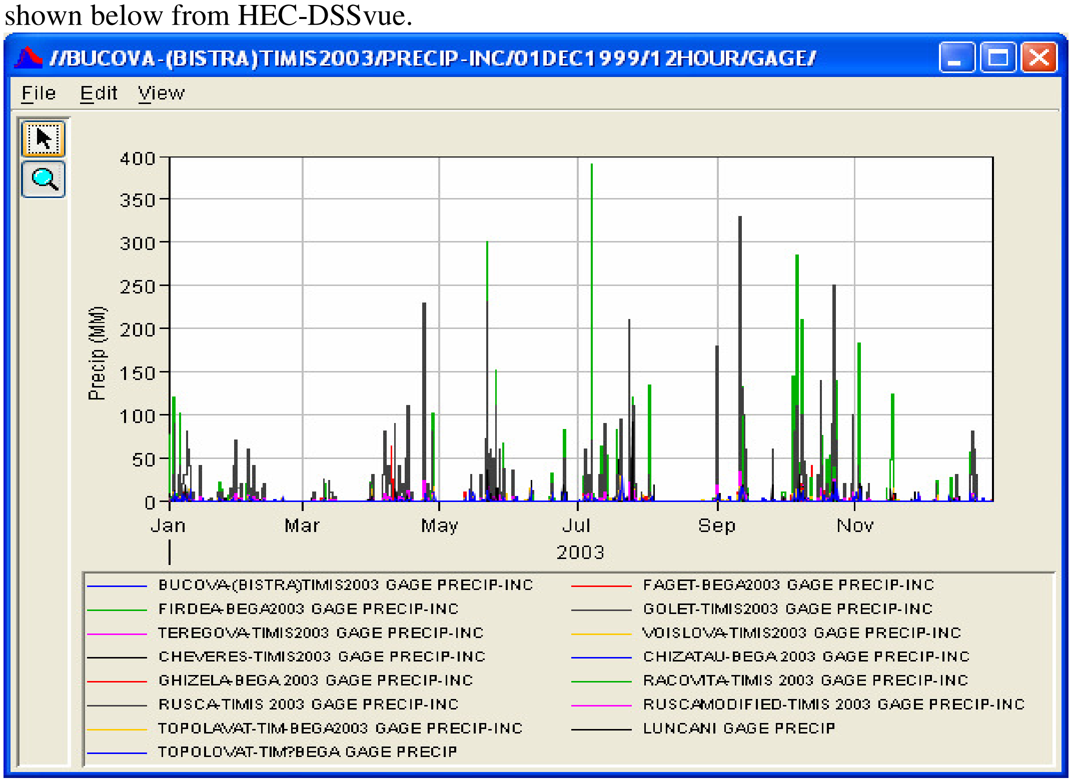
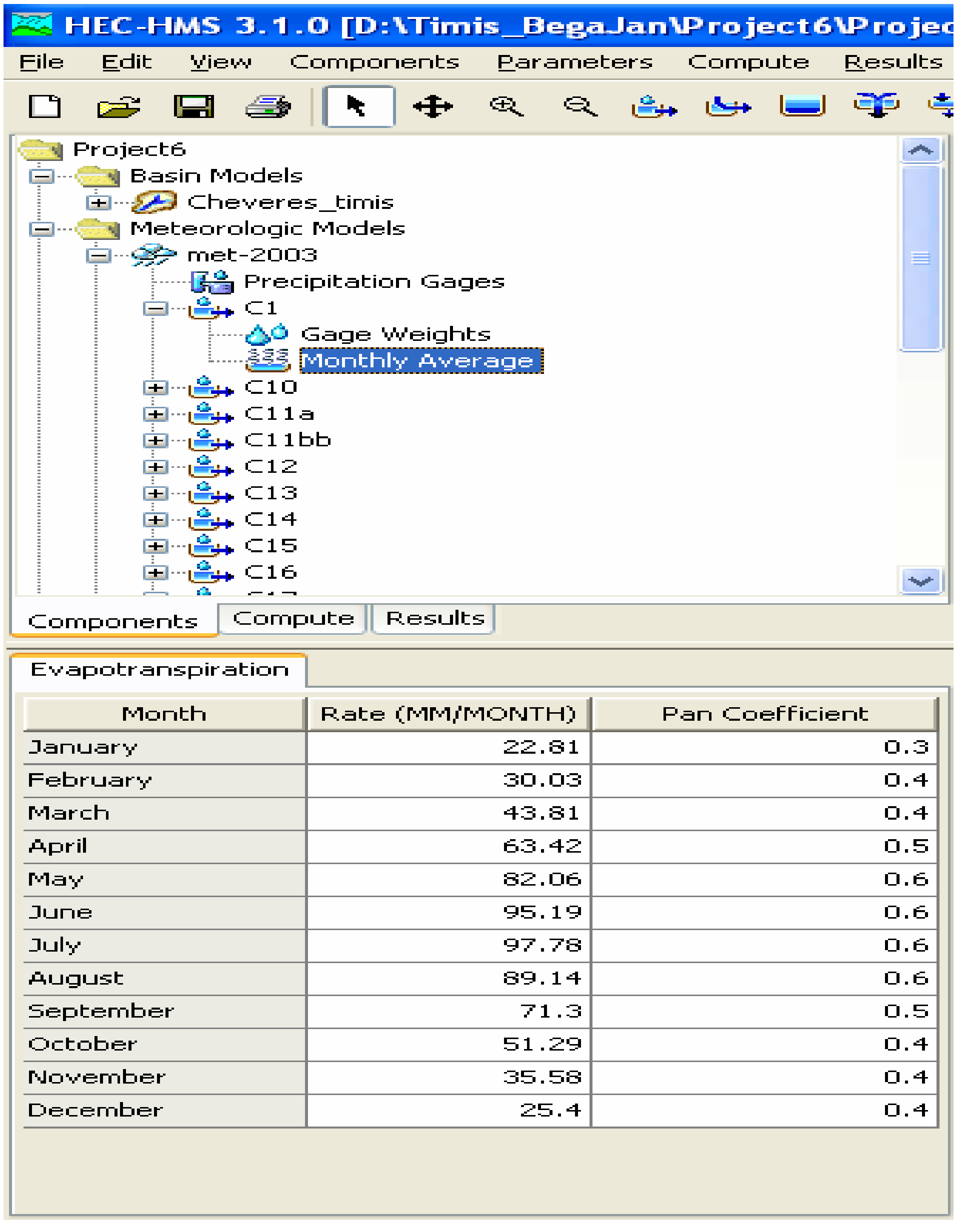


Figure A-6: A figure showing measured precipitation as introduced into HEC-HMS.

The potential EvapoTranspiration,ET as was provided into the model is shown below. This ET was provided as a constant to all the catchments.



Month	Rate (MM/MONTH)	Pan Coefficient
January	22.81	0.3
February	30.03	0.4
March	43.81	0.4
April	63.42	0.5
May	82.06	0.6
June	95.19	0.6
July	97.78	0.6
August	89.14	0.6
September	71.3	0.5
October	51.29	0.4
November	35.58	0.4
December	25.4	0.4

Figure A-7: A figure showing computed Evapotranspiration as introduced into HEC-HMS.

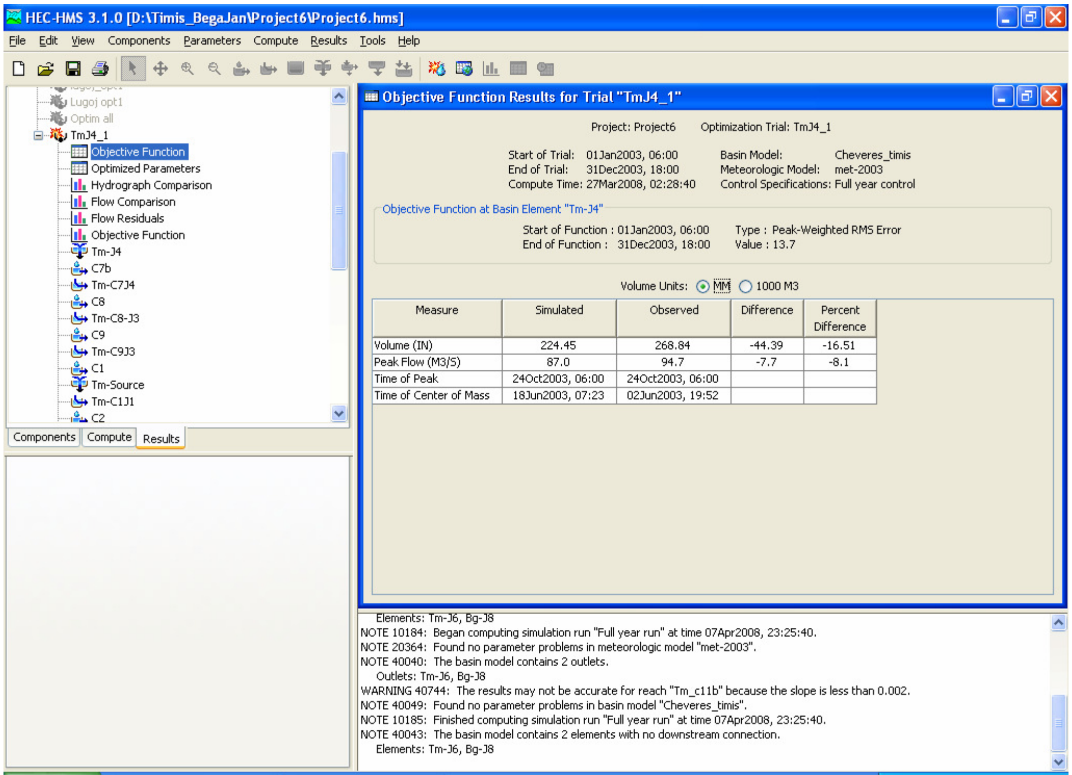


Figure A-8: A figure showing optimization results as obtained from HMS

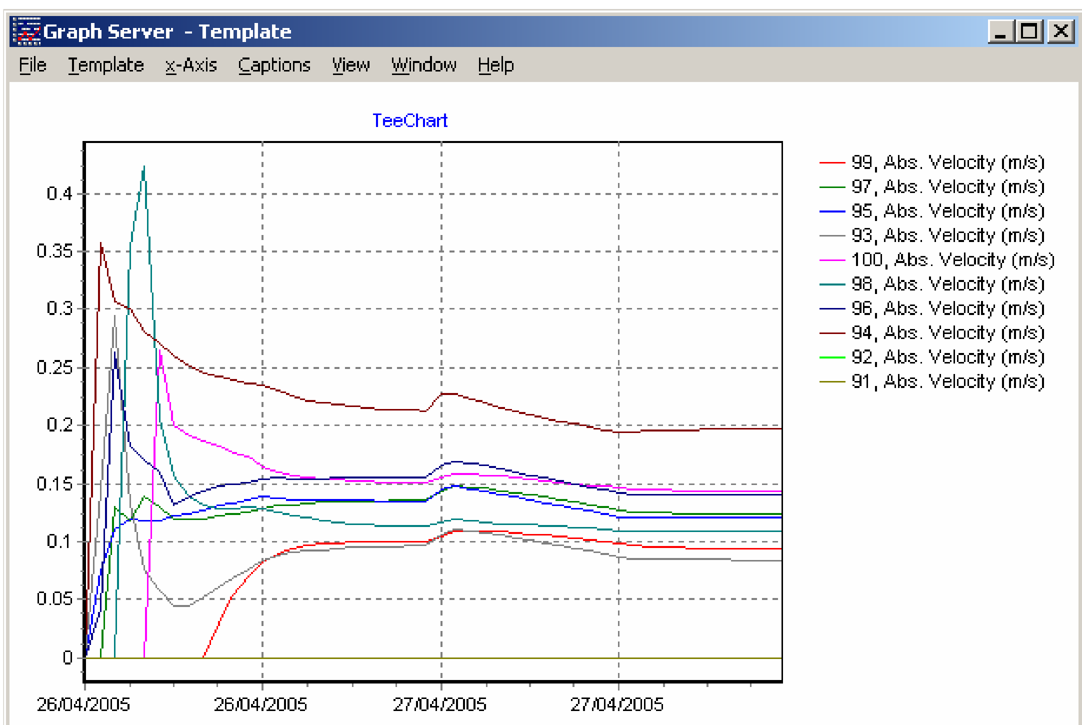


Figure A-9: A figure showing velocities at history points on the floodplain.

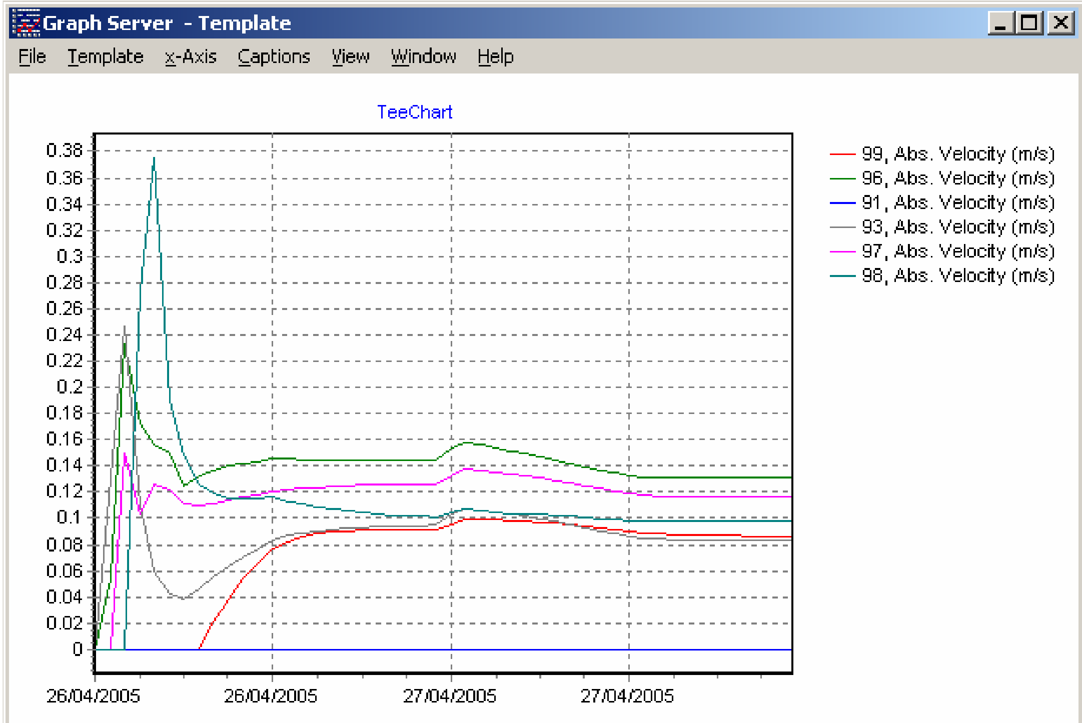


Figure 6-10: A figure showing velocities at history points on the floodplain.