Numerical Simulation Analysis Using

ISIS & HEC-RAS

A thesis submitted in partial fulfilment of requirements for the

Degree of

Masters of Science

In

Civil & Structural Engineering

By: Belal Almassri

UoB No: 10022691

Supervisor: Dr. Songdong Shao

School of Engineering Design and Technology

Civil Engineering Department

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STATEMENT OF ORIGINALITY

I understand that all my project work must be my own unaided work. If I make use of material from any other source I must clearly identify it as such in any interviews, reports or examinations. I understand that my reports must be written unaided in my own words, apart from any quoted material which I must identify clearly in the correct manner. I understand that the work which I shall present for assessment must be work carried out by myself only during the project period which has not been previously prepared. Where any such previous work is made use of in the project, I shall make this clear in any interviews, reports or examinations. I understand that violation of these conditions may result in a mark of zero for the component or components of assessed work affected.

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ACKNOWLEDGEMENTS

I want to express my sincere gratitude to my supervisor **Dr. Songdong Shao** for his valuable, professional supervising and guiding my research. In particular, I have appreciated that he let me to follow my own path during my research project.

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May God Bless You All

ABSTRACT

Numerical Modelling is one of the most information-dependent researches compared to other sectors. Computers have been used to improve the efficiency of management in most of the engineering projects including the disaster recovery processes. Treating information efficiently is the key to maintain a high degree of the security for all of the engineering projects.

This research presents the numerical analysis of different simulation cases related to different types of modelling tests using two of the leading softwares in the river modelling field which are the ISIS and the HEC-RAS, these cases present two of the dam failure tests, the simulation of the river flood, the dry bed river and the sinusoidal oscillation case. This research gives also fully understanding steps for the two softwares from the starting up of the programs and finishing with the results and the discussions. The research also highlights a comparison of the results of this research and other results from previous researches then it highlights a comparison between the ISIS and the HEC-RAS in order to decide which one is more efficient, faster and more reliable than the other.

From the simulation analysis done in this research it presents some more advantages in the HEC-RAS which they are not exist in the ISIS however the ISIS can be more efficient and gives more reliable results if a different type of simulation cases were applied or different size of data has been inserted into the programs so for these reasons this research suggests more investigations to be done in the future studies before carrying on the simulation processes with different types of cases and large amounts of data in order to decide which software is more efficient than the other.

Keywords: ISIS; HEC-RAS; Steady State Analysis; Unsteady State Analysis; Upstream Boundary Condition; Downstream Boundary Condition; Flow Hydrograph; Stage Hydrograph.

Table of Contents

List of Abbreviations:

EDF: Electricite de France.

EPRI: Electric Power Research Institute.

FERC: Federal Energy Regulatory Commission's.

HEC-RAS: Hydrologic Engineering Centres River Analysis System.

ICOLD: International Commission on Large Dams.

NRCS: Natural Resources Conservation Service.

PMF: Probable Maximum Flood.

PRA: Portfolio Risk Assessment Methodology.

RAM-D: Risk Assessment Methodology of Dams.

List of Tables:

List of Figures:

Chapter 1: Introduction

1.1 General Introduction

The rapid development of computer and communication technologies provides the potential for inducing big changes in the use of data, information and knowledge in the civil engineering projects industry. In particular, engineering software technologies and the Internet make it increasingly possible to provide a good management for the engineering projects in the field. Some firms have begun to adopt the new technologies believing that it would improve the productivity and the safety of their projects. The main topic which is being discussed here in this research is based on the software technologies ISIS and HEC-RAS in the fluid dynamics field which used in so many companies throughout discussing the dam failure cases, river flooding issues and simulating the rivers reactions in the flood cases in order to maintain high level of the security in the engineering projects.

1.2 Research problem

Although there have been some researches into dam break analysis using ISIS and HEC-RAS, no work has been done on a significant scale to investigate the ISIS and the HEC-RAS applications in so many different cases of the dam failures and the river flood simulations. Some research discussed specific cases of previous dam failures and made comparisons between the analysis of the dam failures which come from the ISIS or HEC-RAS and the other methods of analysis but no one made a simulation analysis for the two softwares on a large scale and applied so many cases of the dam break analysis and the river simulation analysis into the ISIS and the HEC-RAS at the same time. Greater attention needs to be paid to the interaction of the ISIS or the HEC-RAS application with other numerical or physical modelling methods in order to watch the difference, the advantages and disadvantages of the ISIS and the HEC-RAS applications.

1.3 Research aims

The first aim of this research is to understand the usage of the HEC-RAS and the ISIS applications in the numerical simulation of the flow analysis and show how much can these softwares help in the simulation process of the dam failures and the flood in the rectangular channels, the other main aim of this research is to know which is the best software between the ISIS and the HEC-RAS, which one is faster, friendlier and gives most reliable results.

1.4 Research layout

The research project consists of five chapters. Figure 1.1 shows the relationship between the chapters. The first chapter gives a general introduction and a sample of the layout of the research. The other chapters will be divided as the following:

Chapter two is the literature review that covers the following areas:

1. River floods events and factors causing floods.
   - River flooding factors.
   - Maximum floods records around the world.
   - Sample of the flood hazard assessment (United Kingdom)
2. Dam failure background which includes the following points:
   - Definitions and concepts of the dams.
   - Historical background of the dam failure.
   - Factors which causing the dam failure.
   - Risk study assessment methodologies to reduce the dam failure likelihood.
3. Benefits of the numerical modeling.
4. Selection of the software and its user guide which include the following points:
   - Starting ISIS and basic concepts.
   - Starting HEC-RAS and basic concepts.

Chapter three explains the methodology which covers the following areas:
   - Dam failure simulation cases selection.
   - Other simulation cases selection.
   - The methodology of the analysis of the simulation cases.
   - Reliability check of the results of the analysis.

Chapter four which is the biggest chapter in this research illustrates the procedure of different five simulation cases and the findings of the simulation analysis of each case using both the ISIS and the HEC-RAS and then conducting comparisons between the two softwares in order to tell which is the best. Chapter five is the last chapter; it includes the conclusions and recommendations of the researcher.

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   │ Comparisons of Results│
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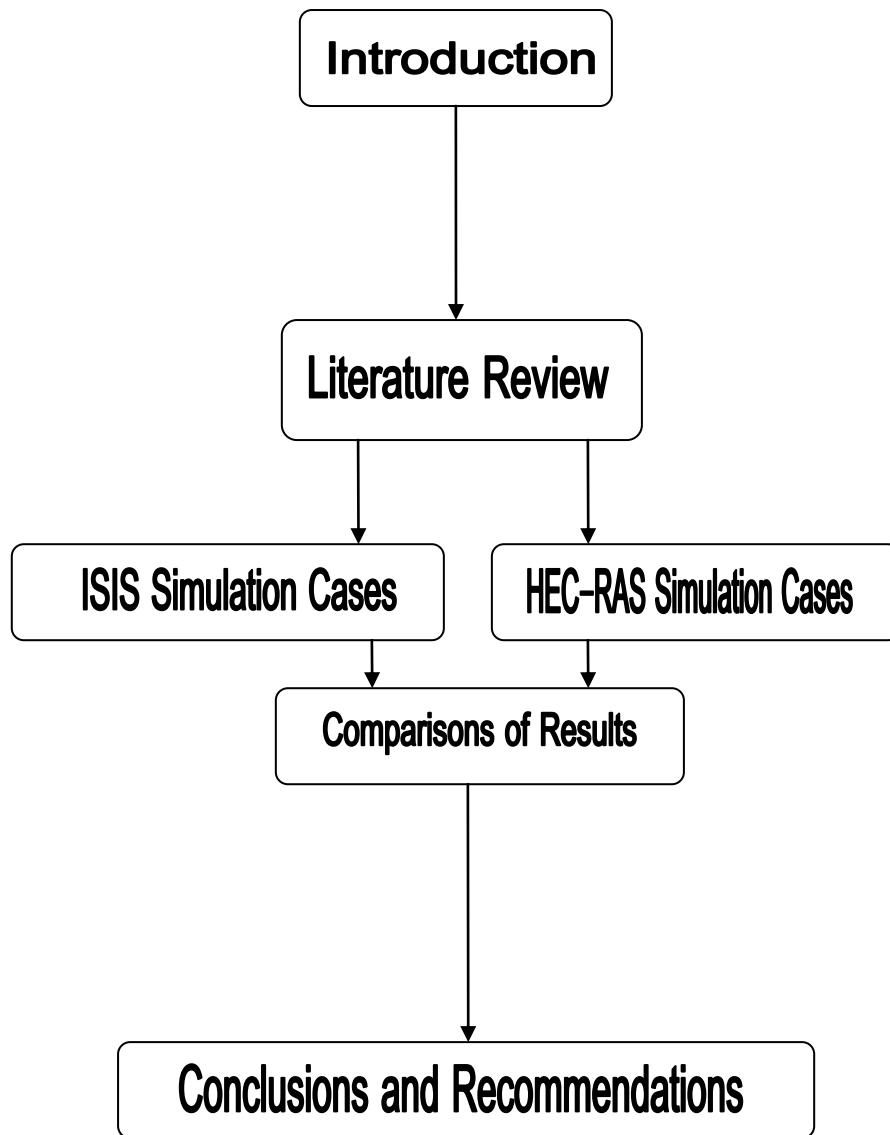
Figure 1.1 Research Layout

Chapter 2: Literature Review

2.1 River flood events and factors causing floods

In the last few decades so many researches and case studies have been done in different countries across the world in order to understand the potential danger of a sudden river flood and try to minimize the likelihood of river floods throughout doing flood risk assessment, trying to treat the factors of flooding and study the previous river flooding records. In this part a brief discussion has been done on the factors causing river flood, the historical records of famous flood records around the world and an example of how the flood hazard is being assessed in one of the main developed countries in the world which is the United Kingdom.

2.1.1 River flooding factors

Floods are considered as a part of the hydrological cycle and it happens due to heavy rains which may happen at any time during the year and it is not related to winter only. When huge amounts of rain fall over short period of time which called flash floods and happen in small or steep areas, one of the characteristics of the flash floods is the short duration which makes it very dangerous because it has a little warning than any other type of floods so that it causes the most hazard damage to the environment and the lives of people. When the heavy rains are companied with heavy storms then the worst flood damage will take a place which called with the tidal or the spring floods which is happening on very short period of time as well and contains large height of the waves of water and it considered to be as danger as the flash floods, other types of floods as the plain floods which happen in the large rivers and take longer duration of time than the flash floods so it is considered to be less danger than the first two types (Samuels, 2003).

Other reasons which cause the river flooding as listed in (Loh, 2011) are listed below:

1. Inadequate drainage system or failure of the drainage improvements.
2. Inadequate river capacity or tidal backwater effect.
3. The construction of bridges and culverts blocks the Reach Rivers and decrease the capacity of the rivers.

2.1.2 Maximum floods records around the world

When the maximum floods take a place the most dangerous disasters will happen so for that reason so many researchers have studied the maximum floods records around the world, as 141 rivers were selected around the world with maximum floods carry a flow of water equals or greater than 10,000 m^3/s it has been found that China had the most dangerous floods percentage around the world with 32.14% which means 45 rivers between the 141 rivers had the maximum floods in the world and the maximum floods in Asia which had the highest percentage of the maximum floods through the all continents with 54.61%. The second highest country in the world is the USA with 12.86% which means that 18 rivers between the 141 had the maximum floods in the world. Around 80% of the maximum floods around the world relate usually to the solar activity and it is related especially to the relative solar number R where it has been found that the maximum floods occur when the value of R is around 24.73 or around 110.2 (Yongquan, 1993)

2.1.3 Sample of the flood hazard assessment (United Kingdom)

The effective flood management must be covered by an operational process which identify the locations which are exposed to different types and different degrees of flooding, the hazard mapping is an effective way which can manage the flood hazard through developing the long term strategies of the flood management which is the current used method as a flood hazard assessment in the UK which is responsible for identifying the areas where flood problems can take a place (Brown and Damery, 2002). Flood hazard assessment in the UK is a changeable matter because it is related to the developments of the agriculture and the relationship between the agriculture future and the flood hazard management in the UK is more than any other country, one more thing affects on the hazard management which is the changing of the governmental policies (Penning-Rowsell and Handmer, 1988).

2.2 Dam failure background

2.2.1 Definitions and meanings

The dam is that structure which blocks or changes the direction of water through a river or any other stream flow, the main purpose of the dam is to collect and store the water in an area so that the water can be used in many different purposes like irrigation, house usage or energy generation. There are different types of dams according to their shape, height and the material which used in the construction of the dam. According to the shape of the dams the dams are divided into three types: small, large and major dams (Nuera, 2005). The ICOLD defined the large dams as those dams having a height of 15 meters from the foundation or the height is between 5 and 15 meters but the reservoir capacity is more than 3 million cubic meters (Coface, 2003). The other two types of dams according to their shape defined by The Asian development bank which defined the major dams as those dams having height more than 150 meters and the small dams which having a height less than 3 meters.

(NRCS, 2003) mentioned that the dams are built in order to maintain two different major purposes; the first one is to encounter the water and make flood management for the river stream or any other water stream. The other type of dams is the diversion dam which is a structure built in order to divert all or part of the river or water stream then that water stream can be controlled effectively and used for different purposes, these purposes mentioned in (Nuera, 2005) as the following:

1. Saddle dam: the saddle dam is built at the edge of the water stream in order to protect the land next to that water stream from flood risk.
2. Dry dam: the dry dam is very important in the heavy rainy areas so the flow of the river or the water stream is very big.
3. Overflow dam: the overflow dam is used to increase the level of water inside the river then different sizes of weirs can be used at different levels in order to generate electricity.
4. Check dam: the check dam is that type of dams used to ensure that the soil erosion not take place and try to reduce the velocity of water flow in the river.

According to the material used to build the dams, the dams have been divided into two major types, the first one is the embankment dam and the other one is the masonry dam. The embankment dam is built out of fill materials, those materials are less dense than concrete so

it is expected that the embankment dam will need more material of loose rock or earth or a combination.

The embankment dam has been divided into three different types of dams regarding the fill material used in the construction of the dam as the following:

1. Rock-fill embankment: filled with loose rocks covered with a crest of concrete to prevent the water seepage inside the fill, that crest can be made from any other material prevent the passage of the water inside the fill.
2. Earth-fill embankment: made from well-compacted soil layers (more than 50% of the fill material consists of soil) covered with waterproof layer as the rock-fill embankment, sometimes a clay layer inserted inside the fill or outside the fill on the upstream face.
3. Zoned embankment: used in modern dams which the core is made from water resistant material and then covered with rock or soil in order to prevent the water to go through the dam material. So that different degrees of permeability will be exist in these types of dams.

On the other hand the masonry dam consists of stone, brick or concrete joined together with mortar which is a mixture of sand, cement powder and water, the masonry dam has been divided into three different types of dams which are the Arch dam, the Gravity dam and the Buttress dam (NH Department of Environmental Services, 2006).

1. Arch dam: the arch dam is made of concrete or stone-brick material has the shape of curve across the river, the main design purpose of the arch dam is to move the water pressure onto the narrow walls of the river. This type of dams doesn't depend on its self-weight as the following type so it is not expected to be heavy. The arch dam can be formed into eggshell shape or multiple arch shapes.
2. Gravity dam: this type of dams depends on its own weight to hold the water pressure of the reservoir, the gravity dam is much heavier in weight than the arch dam and it is made from concrete because concrete is known of its water resistance, heavy weight and easy formation into so many different shapes.
3. Buttress dam (hollow dams): the buttress dam consists of concrete wall supported with buttresses placed on the downstream face and made from steel bars, the buttress dam used usually in the wide rivers like the gravity type of dams and not like the arch

dams which used in the narrow type of dams (NH Department of Environmental Services, 2006).

The previous types of dams are considered the most common types of dams according to the material used in the construction process, the following figure summarize the different types of dams according to different kind of divisions.

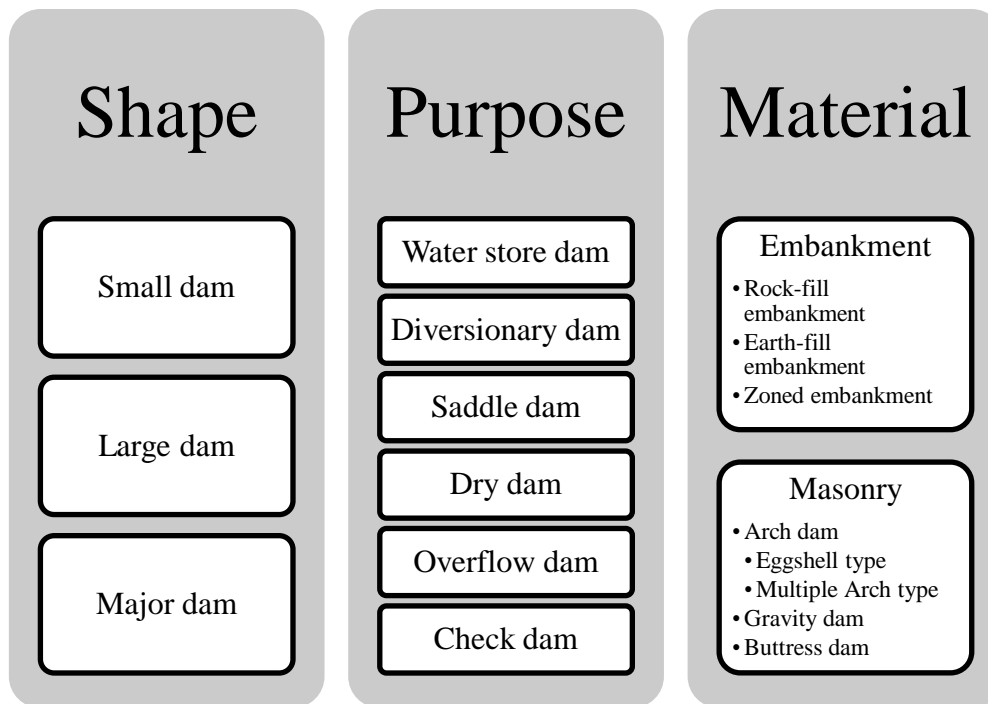| Shape	Purpose	Material
Small dam	Water store dam	**Embankment** • Rock-fill embankment • Earth-fill embankment • Zoned embankment
Large dam	Diversionary dam	

	Saddle dam	**Masonry**
		• Arch dam
		  • Eggshell type
		  • Multiple Arch type
		• Gravity dam
		• Buttress dam

Major dam	Dry dam	
	Overflow dam	
	Check dam	

Figure 2. 1 Dam Classification

2.2.2 Historical background of dam failures

One of the oldest manmade structures on the earth are the dams which have been used for the first time in the Roman Empire in the first century in order to encounter the river water near the Mediterranean Sea (Nuera, 2005). The construction of dams for many different purposes continued until the modern age when the fluid mechanics science has become deeper and more professional even though the failures and the breaks of the dams still one of the important issues which concern the engineers and the designers all over the world. There are many different causes of the dam failure, one third of those causes globally are because of the Overtopping, and another one third of the dam failures globally are because of the foundation defects, other reasons like piping, seepage and inadequate materials has took a place also along the previous years (Probe international, 2008)

One of the dramatic historical dam failure cases which happened in 1959 in the Malpasset dam in France and killed 500 people just after few weeks of the crack notice, the main reason of the crack happened is the tectonic fault which has been realized to be exist after the crack took a place, the existence of the tectonic plate under the downstream face built non uniform pressure under the dam and when the heavy rains came, the level of the reservoir had increased and helped in the Malpasset dam break (Ahmed and Morris, 1999).

Another famous dam break happened in the kolnbrein in Austria in 1979, the cracks and the seepage of the water were noticed after the second partial filling, they tried to drain the water at that time but that wasn't applicable and the problem still exists. In 1989 the engineers decided to support the dam with buttresses from the downstream side in order to hold the dam and prevent the failure but the repair cost was about 190 million $ and that was a case studied and simulated by the engineers again and again in order to prevent some similar cases in the future (Robinson et al., 2002).

Another two important historical cases in the dam failures issue were the Fontana dam in USA and El Atazar Dam in Spain, the major problem of Fontana dam was the chemical reaction between the aggregate and the cement inside the dam and that resulted in a significant cracks. About El Atazar dam break the main reason behind that was the different deformation between the left and the right side of the dam and that resulted in a significant settlement in the foundations (Robinson et al., 2002).

A table illustrates some of the other historical dam failures cases happened is shown below with the dates, names and the causes of the failures:

Table 2. 1 historical dams failures (Chanson, 2009)

Dam name	Date	Cause of failure
Blackbrook dam, Uk	1799	Dam settlement and insufficient capacity of the reservoir.
Hebra dam, Algeria	1881	Insufficient capacity of the reservoir leads to the overturning and the heavy rains increased the level of the reservoir.
Teton dam, USA	1976	Piping in the dam.
Glashütte dam, Germany	1953	Overtopping during a very large flood and insufficient reservoir capacity.
Tous dam, Spain	1977	Electrical problem followed by overtopping
Dale Dyke, USA	1863	Poor construction work

2.2.3 Factors of dam failures

As mentioned before that the dam failures causes are considered one of the important subjects which being studied by the engineers all over the world during the last years in order to ensure the dam safety and to prevent the damage for the people or the structures to take a place. So many researches have been produced for the dam failures in order to find the main reasons behind the break of the dams and how to repair it. In this part a list of the major reasons of the dam failures will be discussed as the following (Probe International, 2008):

- Overtopping: as mentioned before that the reason of overtopping is considered to be behind one third of the dam failures globally, the overtopping happened when the level of the reservoir reach its limit and exceed this limit so the capacity of the water inside the dam exceeds its limit. This reason happened when the spillway became insufficient or the settlement of the dam crest exceeds its limit. The heavy rains usually plays the facilitator of the dam failures because when the heavy rains come it is expected to increase the capacity of the reservoir and then the level of the water inside the reservoir exceeds the limit and so the failure take a place. (Probe International, 2008).

- Failure of the dam foundations: another one third of the all dam failures happened due to the large defects of the foundations supporting the dam. The major reason behind that is a design mistake when the weight of the dam hasn't taken into consideration very well then a differential settlement will occur underneath the dam that will result in a non uniform pressure, this pressure will cause instability and landslide under the dam so at that time if any seepage for water happened or any land shake took a place it is expected that will dramatically compromise the structure. (Probe International, 2008).

- Piping and internal erosion of soil: about one fifth of the all dam's failures happened due to the piping and seepage problems and that is considered very famous in the embankment type of dams which considered semi-permeable. The seepage of water through the dam reduce the strength of the dam and result in large cracks and settlements through the dam, those cracks will play as water pipes through the dam which will make the dam weaker and weaker and will compromise the dam in any instant. (Probe International, 2008).

- Other reasons: inadequate maintenance, insufficient in the materials used in the construction and/or improper design for the dam. (Federal Emergency Management Agency, 2006).
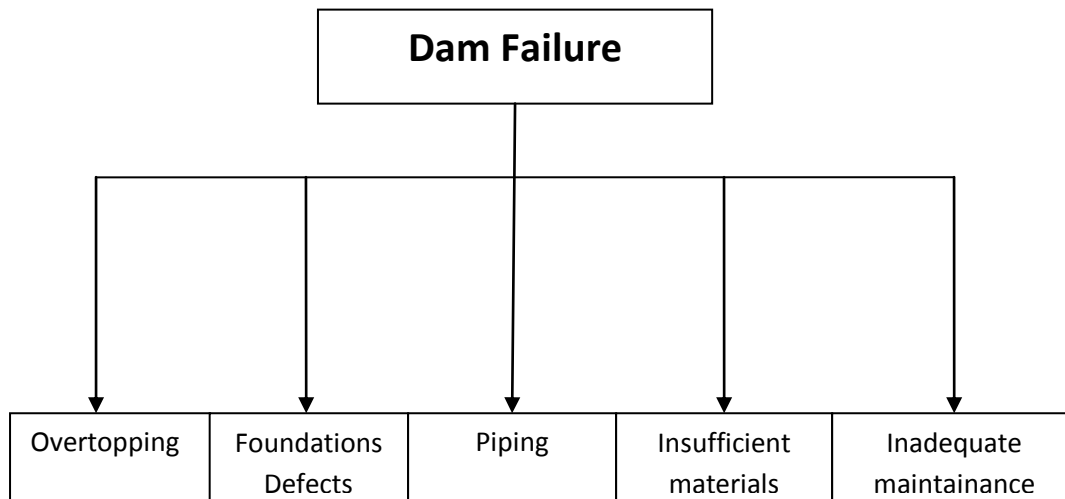
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                        ┌─────────────────┐
                        │   Dam Failure   │
                        └─────────────────┘
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┌────────────┬────────────┬────────┬────────────┬────────────┐
| Overtopping | Foundations | Piping | Insufficient | Inadequate |
| | Defects | | materials | maintainance |
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Figure 2. 2 Dam Failure Causes

2.2.4 Risk study assessment

The risk study assessment of the dam failures should pass number of levels and points in order to allocate the risk and try to reduce that risk at the end and try to increase the safety of the dam; those stages are summarized here as the following (Hill et al, 2003):

- Estimating the probabilities of the dam failures:

There are so many available methods for estimating the probabilities of the dam safety risk analysis, one of those methods is the historical performance method which study a similar cases and dams as the dam which being studied and analysed then try to assume that this dam will behave as the same old dams in the future, the historical method is considered to be applicable for the initial stages of the risk assessment because it is not detailed and the analysis of the dam safety can't be relied only on it. Other method for estimating the probability is the event tree method which simulates and models the dam failure process from the beginning until the end with all the details of the dam and the foundation and can be connected sometimes to the historical performance data in order to give a check for the results of the assessment. (Hill et al, 2003).

- Combining probabilities:

In this stage there are two different cases should be discussed, the first one is the common cause failure modes which can happen at the same time at any single section in the dam should be studied and then it should be assumed that the failure of the dam will happen due to

a combination of those probabilities, the second case is the study of the single failure cause probability at different number of sections in the dam. (Hill et al, 2003).

- Apply the Uni-Modal bound theorem:

This theorem produce a method of how the probabilities of the dam failure causes at every section inside the dam which they are not mutually exclusive can be modified and adjusted in order to reflect the actual physical condition of the dam in on the modelling process. (Hill et al, 2003).

- Risk Assessment methodology:

There are some methods and procedures consider the dam safety and try to make methodologies in order to reduce the risk of the dam failures, one of those methodologies is the RAM-D risk assessment methodology which was invented in order to evaluate the security level of the dams, the RAM-D risk assessment methodology based on the following equation (Harrald et al, 2004):

Risk = (Likelihood of accident) x (Consequence) x (1-System Effectiveness)

This risk methodology designed especially for the dam owners and the security managers, away from the previous equation, the RAM-D use different types of equations and worksheets in order to calculate the risk of the dam failure. To sum up the most important objective of the RAM-D is to understand and identify the most critical risks which are considered to be undesired then try to analyse the potential hazards of those risks and finally suggest and recommend some steps to increase the emergency and security level. (Harrald et al, 2004).

Other risk methodologies like the Portfolio Risk Assessment Methodology (PRA) which produced by team of engineers, safety managers and decision makers, the risk assessment steps include identification of the hazards then make an engineering assessment after that a decision making will take a place and finally the prioritization and the recommendations will be provided by the team who responsible for the methodology. (Harrald et al, 2004).

Another assessment tool for the dam safety has been produced by the Electric Power Research Institute (EPRI) which works with the U.S. Federal Energy Regulatory Commission's (FERC) Division of Dam Safety and Inspections in order to produce a

hydropower security assessment tool which will be used after that in the studies of the dam safety related to the FERC. The tool aims at producing a security-planning program which will facilitate the self-assessment of the dam and will produce applicable results which meets the standards regulations of the dam safety for so many different purposes. (Harrald et al, 2004).
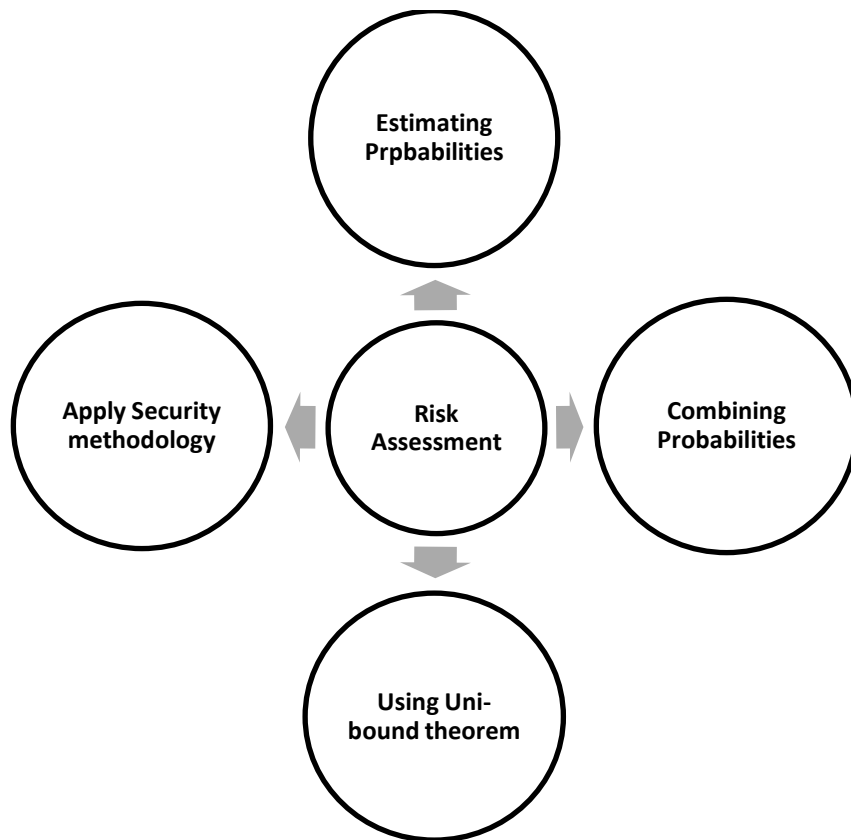

Figure 2. 3 Dams Risk Assessment

2.3 Benefits of numerical model

The numerical model of the rivers is considered one of the most important branches in the hydraulics science because of the following reasons as mentioned in (Abdullah et al, 2009):

1. It performs a large testing operation for the hydraulic case and can evaluate the dam break analysis.

2. It also gives a good prediction of the inflow and outflow hydrograph for any required dam.

3. Calculate the magnitude of the flooding due to the dam and then that magnitude can be compared with the PMF inflow capacity and then the failure risk on a specific section of the dam can be determined.

4. An emergency response plans can be produced in order to prevent the failure of the dams.

5. Determine the flood hydrograph, peak discharge, flood wave propagation time and so on, this concerning with the type of the dam structures, mechanisms and size of the break.

There are different models have been designed in order to manage the water resources. Those models are ISIS, MIKE11, HEC-RAS, HYDRO-1D and Info Works RS (River Simulation). Those programs have been designed to analyse the dam break and consider so many practical hydraulic cases. Those programs can be used as empirical model, analytical model, parametrical model and numerical model. (Abdullah et al, 2009).

In general and to sum up, the failures of the dams can't be totally prevented by using the numerical modelling but the numerical modelling gives a complete methodology in order to reduce the failure risk. There are two ways to reduce the risk the first way is to decrease the probability of the dam break and the second way is to reduce the impact on it. Before the dam break occurs, the process of the flood forecasting should be completed at the fore stage.

2.4 Selection of the software and its user guide

There are wide range of available softwares can compute and predict the flooding but as mentioned before that the biggest concern in this research is to conduct the flow simulation process through the ISIS and the HEC-RAS and compare them together. In this part of this chapter a briefly description of each software, how it works and a simple user guide have been provided to this research.

2.4.1 Starting ISIS and basic concepts

2.4.1.1 Introduction to ISIS

ISIS is one of the worldwide famous software in river modelling which invented by consultants, professional engineers and public bodies from different countries. ISIS is considered as one of the important software in the decision making and strategic planning, it studies the flooding warning, the climate changes, and the development of the rivers areas. It considered as one of the most important packages which deal with flood risk mapping, flood prediction, surface water mapping and dam break analysis (Vasilyev, 2007).

The numerical brain of the ISIS software is based on the Preissmann Implicit Scheme or 4-point Box Scheme. In the implicit approach, three unknown future values *(j-1,n+1), (j,n+1), (j+1,n+1)* will be used with other three known present values *(j-1,n), (j,n), (j+1,n)*. All the unknown values will be linked and solved together using the matrix principle approach (Anderson, 1995).
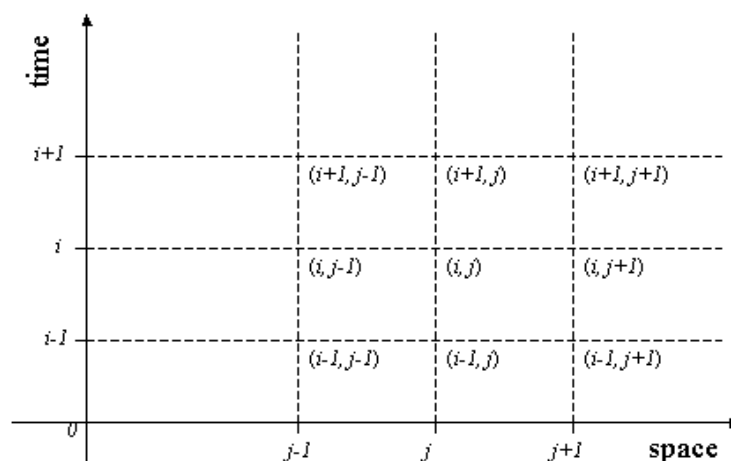

Figure 2. 4 Implicit method (Anderson, 1995)

The three known values with the three unknown values will be used in the following equation in order to form the matrix then after the matrix being solved the ISIS program can get the future values (Vasilyev, 2007).

$$\frac{T_i^{n+1} - T_i^n}{\Delta t} = \propto \frac{\frac{1}{2}(T_{i+1}^n - 2T_i^n + T_{i-1}^n) + \frac{1}{2}(T_{i+1}^{n+1} - 2T_i^{n+1} + T_{i-1}^{n+1})}{(\Delta x)^2}$$

$$\begin{bmatrix} -B & A & 0 & 0 & 0 \\ A & -B & A & 0 & 0 \\ 0 & A & -B & A & 0 \\ 0 & 0 & A & -B & A \\ 0 & 0 & 0 & A & -B \end{bmatrix} \begin{bmatrix} T_2^{n-1} \\ T_3^{n-1} \\ T_4^{n-1} \\ T_5^{n-1} \\ T_6^{n-1} \end{bmatrix} = \begin{bmatrix} K_2^{\cdot} \\ K_3 \\ K_4 \\ K_5 \\ K_6^{\cdot} \end{bmatrix}$$

Figure 2. 5 Implicit method matrix (Anderson, 1995)

The ISIS software gives the choice for the user either to run steady or unsteady simulations, if the user is doing steady run then he has the choice either to do a direct run or timestepping run, the direct choice is considered to be faster and more accurate than the second choice. If the user is doing unsteady run then he has the choice also to choose between two different types of runs, the first one is fixed timespan and the second one is the adaptive timespan. ISIS gives the ability to do one dimensional modelling for the structures and the channels, all the section including the weirs, the spills, the reservoirs and everything can be input into the ISIS in order to find the results which depends on the data input quality and the skill of the user not only the one dimensional modelling can be applied but also the ISIS gives the ability to do two dimensional modelling based on the topography of the area, the hydrological data, the roughness and the floodplain and channel locations (Vasilyev, 2007).

Finally all the results can be obtained from the ISIS software, all the water profiles and the tables including all the information. Output numerical data may be plotted by ISIS by Halcrow software or Microsoft Office Excel. The graphs of the river sections at different time intervals can be plotted, the maximum water level and the maximum reservoir capacity can be calculated then the output data can be analyzed and investigated considering different case in order to form a good decision about the case which being studied (Vasilyev, 2007).

2.4.1.2 Basic concepts of ISIS

In addition to the modelling of the flows and water levels in open channels, ISIS can be used for different types of structures as bridges, sluices, weirs, culverts and pumps which can be modelled easily in the simulation analysis process. The majority of the information in this section has been retrieved from the ISIS by Halcrow User Manual through the software's website. The main concept in the simulation process consists of three main points:

1. The upstream boundary condition which represents a flow of water entering the river.
2. The downstream boundary condition which represents the flow of water getting out from the river.
3. The river channel which will be represented by number of river sections (at least two sections).

The main interface of ISIS is the Network Properties Window, which opens automatically every time ISIS is opened. The Network Properties Window is used to insert the components of the river model such as river sections and weirs. This interface consists of multiple icons for different purposes some of them are related to the boundary conditions, the river cross sections, the junctions through the river mode and the different types of the structural units such as bridges, weirs, sluices and others as shown in the following figure:
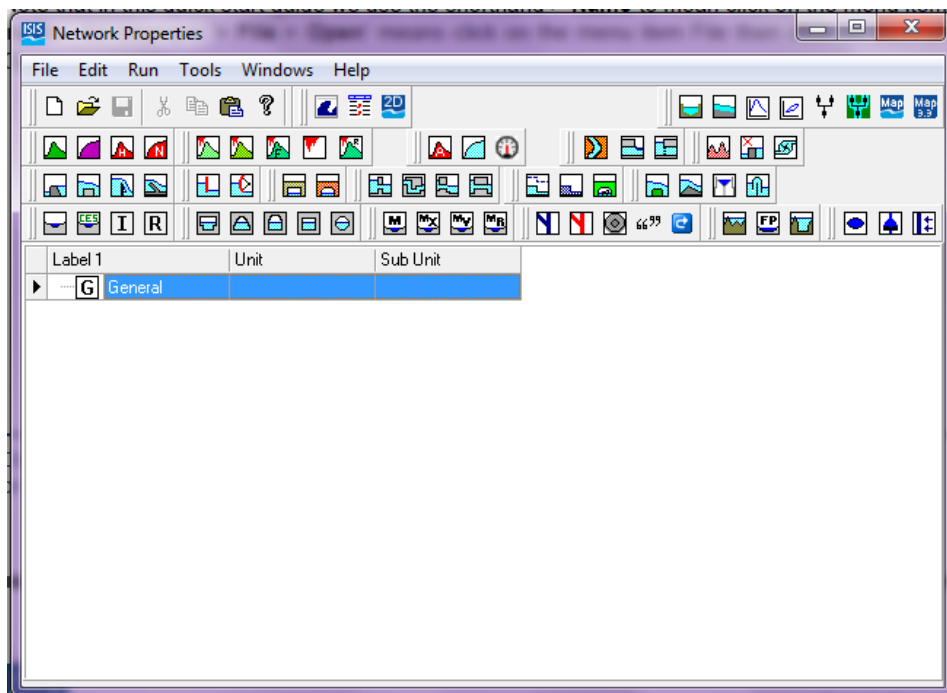
Figure 2. 6 ISIS main interface

ISIS deals with a large range of different types of files in order to control the model data and the results. The most important file types are listed below in the following table:

Table 2. 2 Files Types in ISIS

File type	Usage
.dat	Include the all of the model data and the initial conditions.
.zzs	Include the initial conditions only.
.ied	Include alternative boundaries (optional).
.ief	Include the simulation parameters such as the name, the location, the time step etc.
.zzn	Include the simulation results in binary format.
.ixy & .gxy	Include the visualiser and the GIS information.

2.4.1.3 Design simple single river channel and view the results

In this part a simple river channel will be designed in order to explain the major steps which needed to design a river channel using the ISIS, most of the data used in this part with the help of the ISIS Free & the ISIS profession quick start guide. As mentioned in the previous part that there are three major considerations needed to design the river channel and to run the simulation process successfully those parameters are the upstream boundary, the downstream boundary and a minimum of 2 cross sections. The following points show the major steps needed to build the river channel and to run the simulation analysis:

- Begin a new blank model by clicking on >File >New in the network properties window or by clicking on the icon of New ▯ from the main window of icons.
- Define the upstream boundary condition by clicking on the icon of Flow Time Boundary ▲ then the node label editor has been appeared and the label has been assumed to be S1 as shown in the following figure:
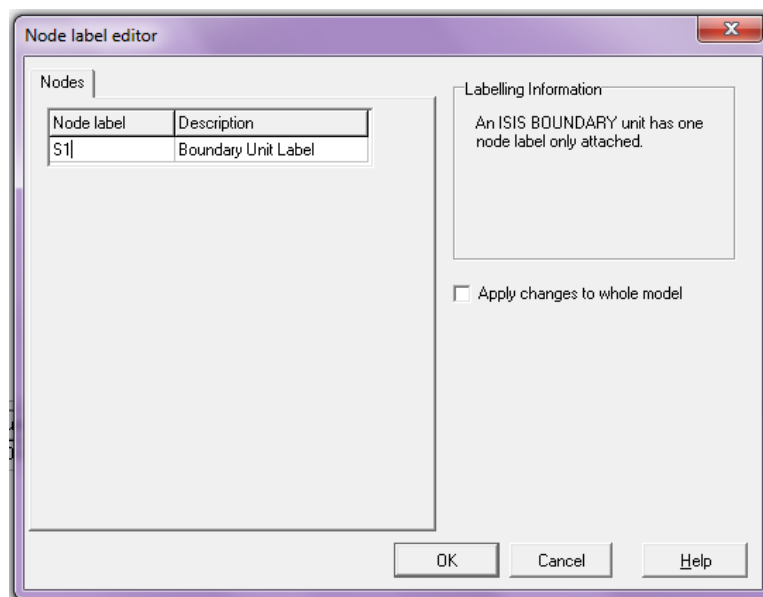

Figure 2. 7 Node Label Editor

- After the label of the upstream boundary has been inserted the data of the flow time table need to be inserted through double click on the S1 (QTBDY), it has been assumed that the peak flow is 20 m^3/s, the base flow is 10 20 m^3/s and the peak time is 12 hours as shown in the following figure:
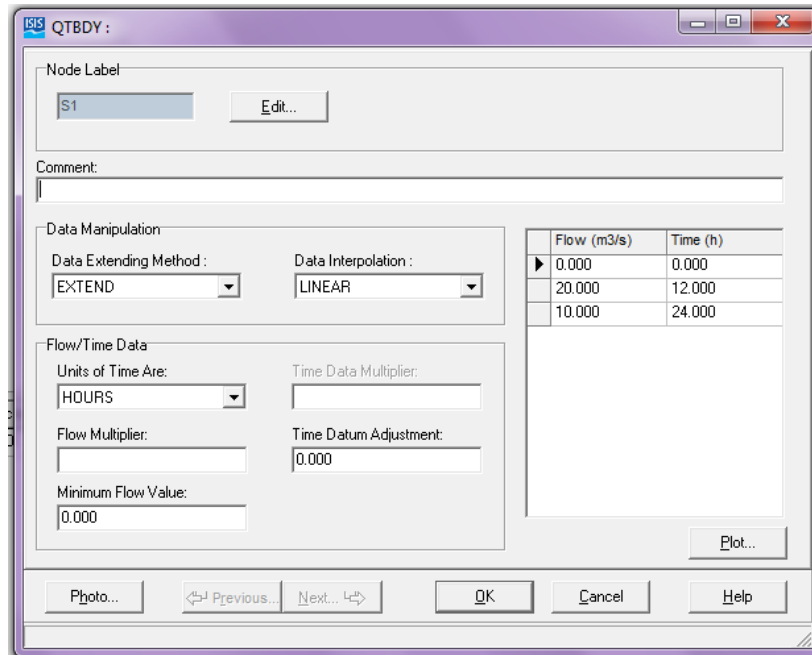

Figure 2. 8 Upstream boundary condition

- Now at least two cross sections have been inserted by clicking on the river section icon ⬇ then the node label editor has been appeared and in order to link the first cross section with the upstream boundary it has been labelled with the same label of the upstream boundary condition S1 as shown in the following figure:
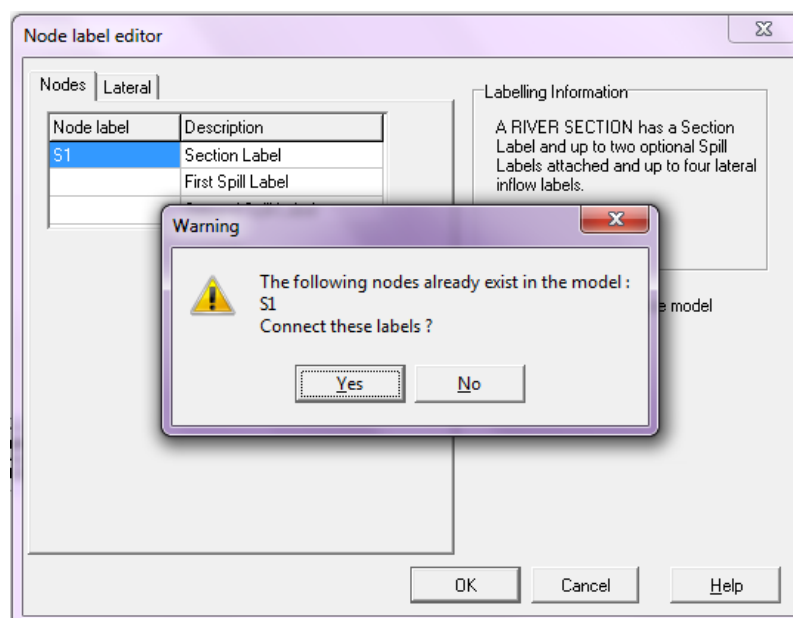
Figure 2. 9 Connect the labels (ISIS)

- After the cross section has been labelled the geometric details of the cross section need to be inserted by double clicking on the S1 (RIVER) then a new window which shown in the following figure and it has been assumed to be filled with the following data:

Figure 2. 10 River Section Details

- The shape of the river section can be checked easily by clicking on Plot in the previous figure, the following steps will be the same as the previous step but in the last river section it should be noticed that the distance to the next section should be set to be zero.

- The last major step in order to complete the channel design is the downstream boundary condition which must be satisfied by clicking on HTBDY boundary condition icon ![icon] which should be labelled with the same label of the last river section in order to be linked as one river channel and the details of the downstream boundary condition which include table of the stage head against the time values have been assumed in this sample example to be filled as in the following figure:
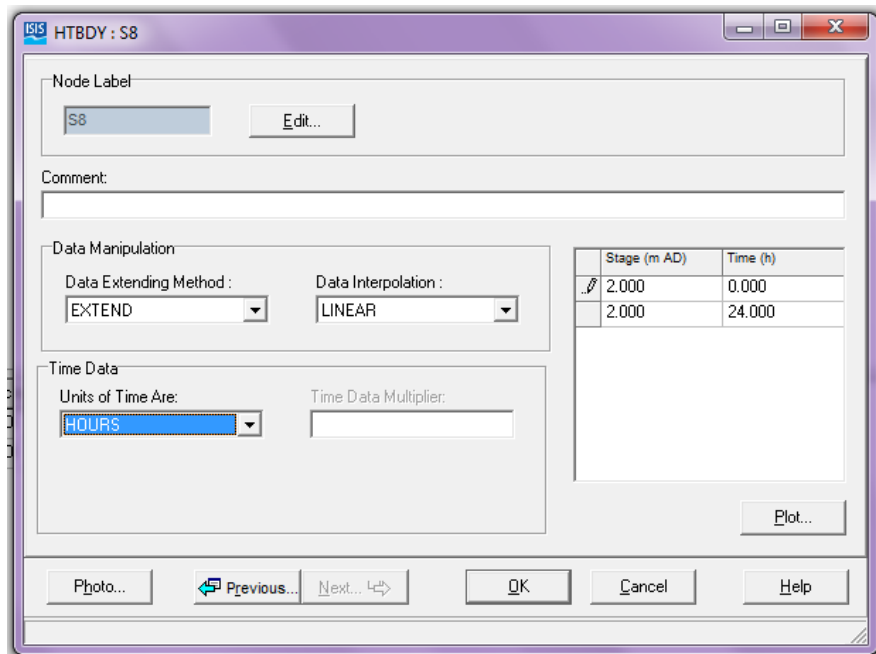
Figure 2. 11 Downstream boundary condition

- After completing the single channel the river channel can be visualised by clicking on the icon of the visualiser icon ⥩ and this icon will enable to see the all river channel with the all links as shown in the following figure:
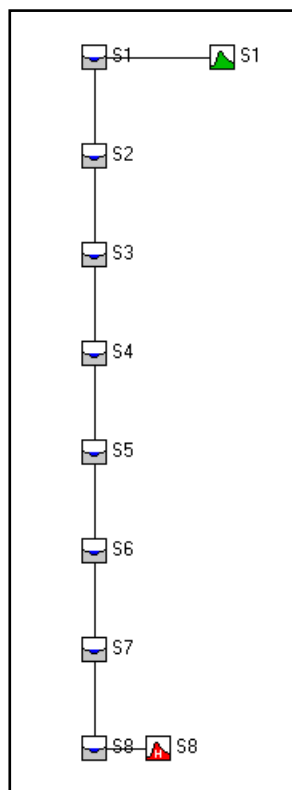

Figure 2. 12 ISIS Visuilizer

- Once all of the design criteria has been met the simulation analysis can be run at this time which will generate flow output results which can be checked at the end of the simulation process, by clicking on the simulation icon ![icon] a new window will appear which include 5 types of the simulation analysis's where the steady (direct) type should be run first in order to make the initial conditions ready to the simulations analysis, in the unsteady (Fixed timestep) and (Adaptive timestep) the finish time should be larger than the start time and if the timestep was very small the software will take more time to converge and the results will be more accurate.
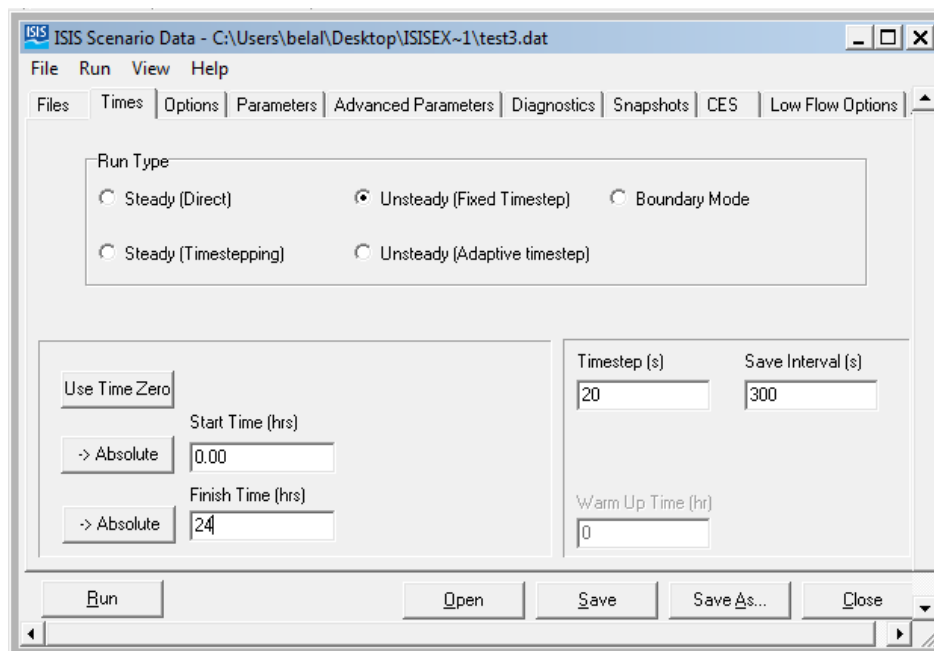

Figure 2. 13 Simulation Analysis (ISIS)

- The results of the steady flow analysis appeared once the simulation completed as word file contain the flow values, velocity values, Froude number and other parameters at different river sections, the results of the unsteady state analysis can be obtained by clicking on the tabular CSV command icon ![icon] which give the ability to obtain the values of velocity, flow, stage and other parameters for the unsteady flow analysis.

- Other results as the longitudinal profile for the river channel can be obtained by clicking on the longitudinal section icon ![icon], the plot of the time series can be drawn by clicking on the time series icon ![icon] as shown in the following figure:
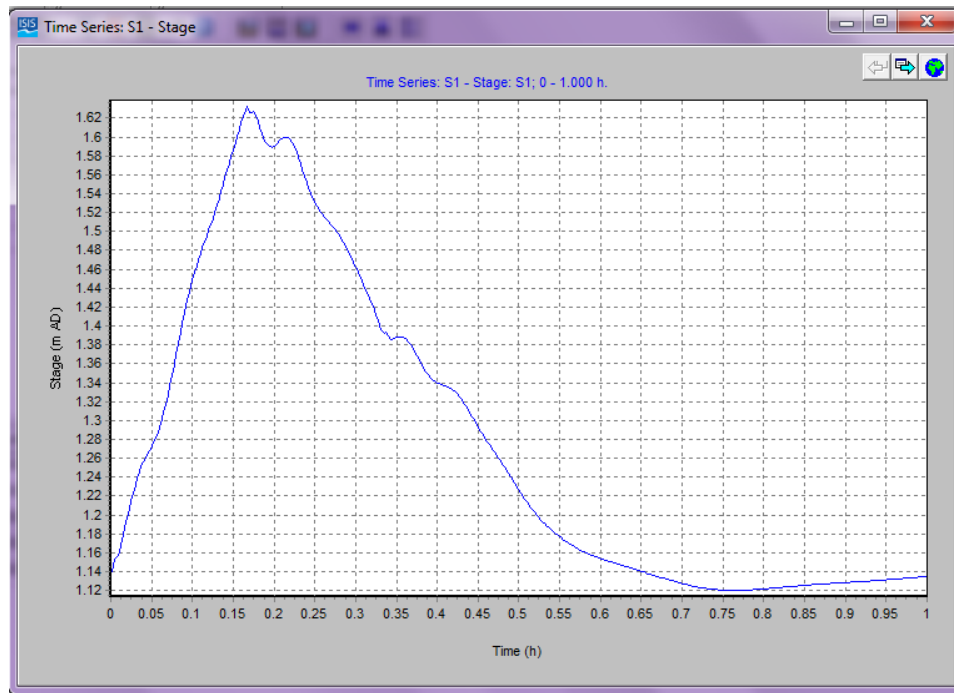

Figure 2. 14 Time Sereis Sample (ISIS)

2.4.1.4 Dam breaks analysis using ISIS application

ISIS software has been used to simulate the case of the Malpasset Dam break and the Toce River dam-break model, regarding the Malpasset Dam break a comparison has been made between the ISIS results and the physical model results by (Ahmed and Morris, 1999), they presented the numerical modelling of the Malpasset dam failure and they discussed the differences between the experimental data and the field data. They found a difficulty in simulating the dry be conditions of the river because of the 1-D package of the ISIS, they recommended to perform a further researches in the improvement of the 1-D models because of the time limitations of their test case so they didn't consider everything related to the dam failure, they recommended also a further checks to be established on the Malpasset dam like the effect of using a head loss at the double curvature of the valley and the time changing of the structure failure. Another test case has been made on the Toce river dam break by (Rosu and Ahmed, 1999), they made a comparison between the ISIS results and the physical model results, they found the same problem in the Malpasset dam break analysis using ISIS which is the inability to simulate the dry bed of the valley and another difficulty has been named in this test case which is the inability to achieve the overflow of the reservoir.

2.4.2 Starting HEC-RAS and basic concepts

2.4.2.1 Introduction to HEC-RAS

HEC-RAS is an integrated system of software which has been designed for different tasks and purposes as the graphical usage, the hydraulic analysis, management capabilities and reporting issues. This latest version of the HEC-RAS can perform one dimensional steady and unsteady flow calculations, the sediment transport calculations are not included in this version and it will be included in the next versions in the future. The water surface profiles are computed by performing the step method for every cross section in the river which is an iterative method solving the energy equation. The standard step method is considered to be the most applicable method of calculating the flow profiles because it can be fitted for different types of channels. This method based on dividing the channel into segments of known length and then finds the unknown depth at one end of the segment (Bentley, 2009).

2.4.2.2 Basic concepts of HEC-RAS

The HEC-RAS defines the bed of the river or the channel by a number of sections in the x-axis direction and elevations in the y-axis direction where the river station 10 is considered to be the upstream boundary station and the river station 0 is considered to be the downstream boundary station. Most of the information shown in this section has been retrieved from the HEC-RAS user's manual which has been downloaded from the US Army Corps of Engineers website. There are major steps should be provided in order to get the simulation analysis completed:

1. Drawing the river reach line which shows the direction of the flow from the upstream boundary to the downstream one.
2. Define the river station and as mentioned above that the upstream boundary is labelled with 10 and the downstream with 0.
3. At least two river cross sections are needed to define the geometric data for the reach river.
4. Finally and before running the flow analysis the initial conditions plus the boundary conditions must be defined.

The main interface of the HEC-RAS is shown in the following figure which contains so many icons related to different tasks and purposes, for example from this interface a new project can be created or an old one can be opened from File tap, the initial and the boundary conditions can be inserted into the project from the Edit tap, running the steady or the

unsteady flow analysis can be done from the Run tap and other options and views of the results for the simulation of the projects can be seen from View and Options taps.
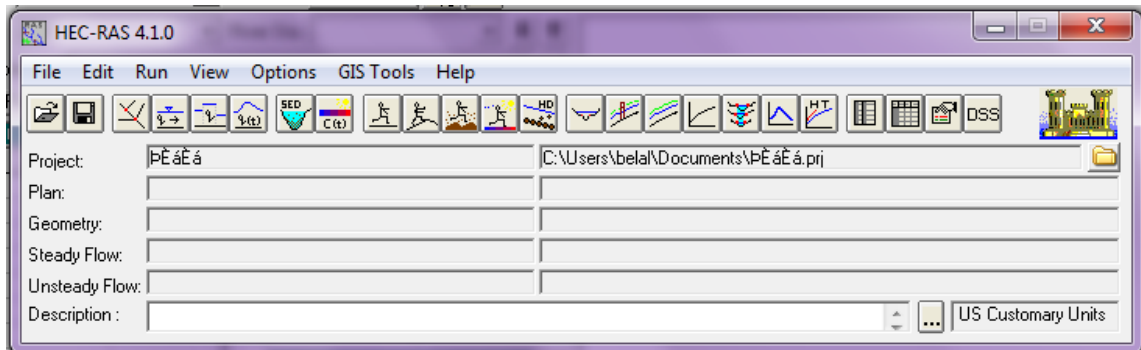

Figure 2. 15 HEC-RAS main interface

2.4.2.3 Design simple single river channel and view the results

In this part a simple river channel will be designed in order to explain the major steps which needed to design a river channel using the HEC-RAS, most of the data used in this part with the help of the HEC-RAS manual. The following points show the major steps needed to build the river channel and to run the simulation analysis:

- A new project can be created by going to >File >New project in the main interface of the software, after doing that the HEC-RAS will ask for the name of the new project as shown in the following figure:
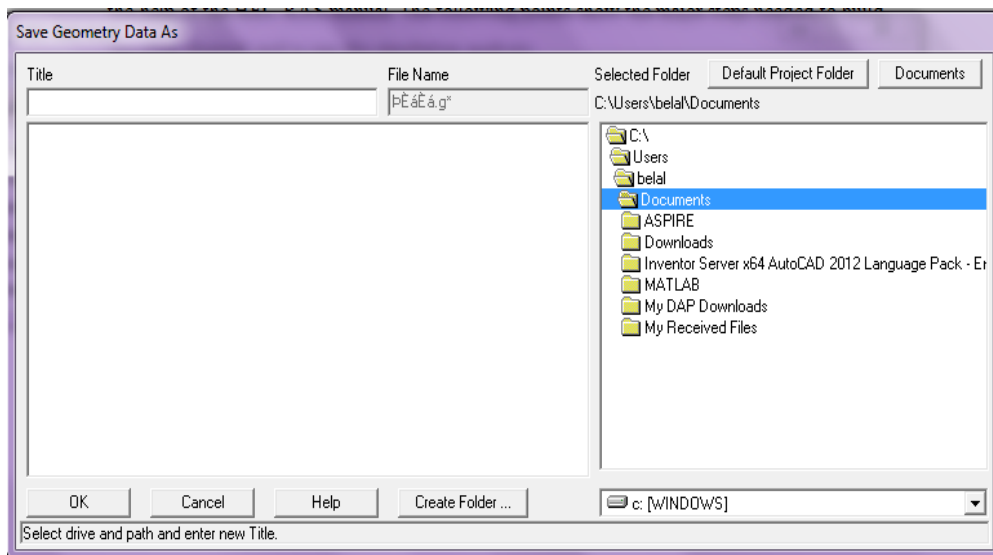
Figure 2. 16 Open a new project (HEC-RAS)

- After saving the new project then it is the time to enter the geometric data of the project by clicking on the icon of Edit/Enter geometric data as shown in the following figure:
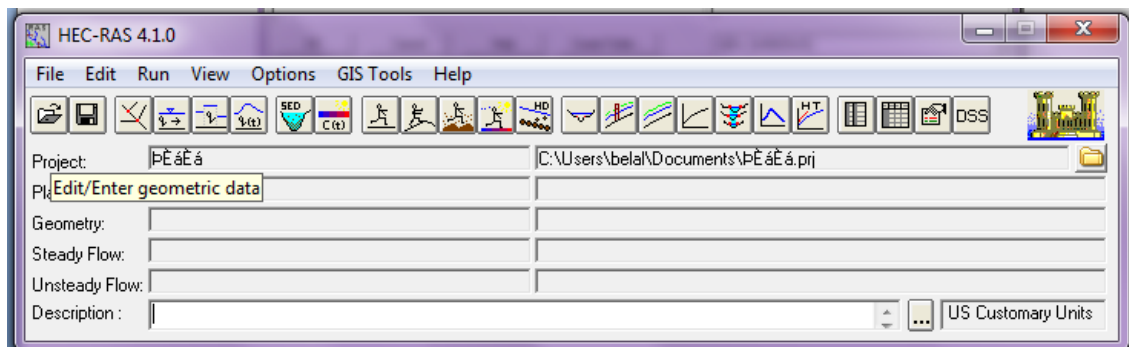
Figure 2. 17 Enter the geometric Data Button

- In the following step a new window will appear and the river reach needs to be drawn now by clicking on the icon of River Reach and by clicking on that icon and drawing the beginning and the end of the river the HEC-RAS will ask for the name of the river and the reach which can be named with any name as shown in the following figure:
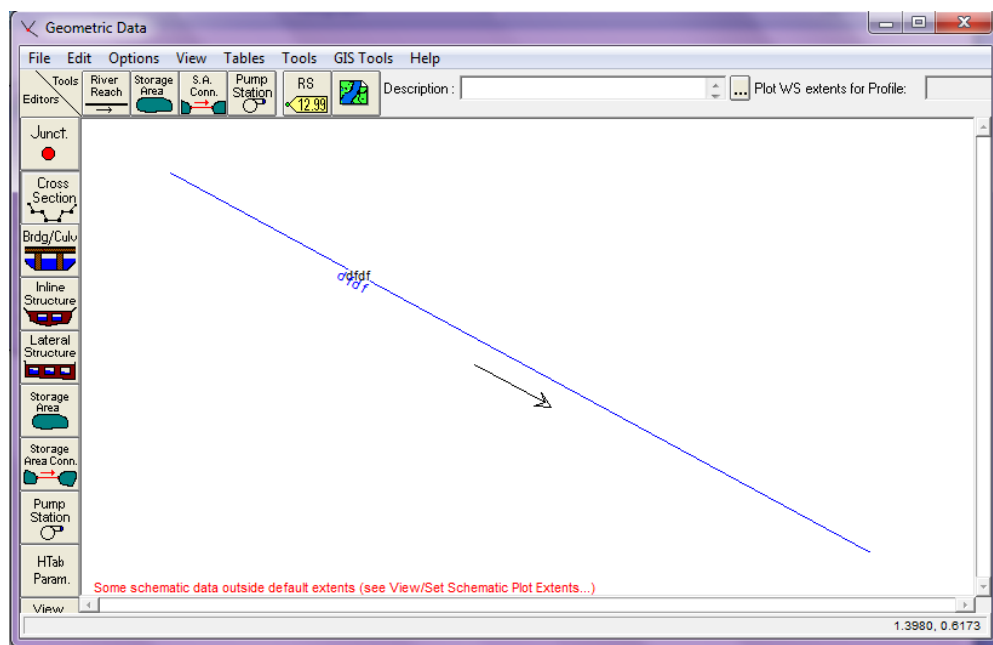
Figure 2. 18 Enter the geometric data window

- From the geometric data window by pressing on cross-section button to enter the details of the river cross section and from the new window appeared by going to >Options >Add a new cross section then the geometric details can be easily inserted into the new window where these details include the cross section coordinates (x-

direction and y-direction), manning values, the distance to the next section and the slope of the river bed. The following figure shows the cross section data window:
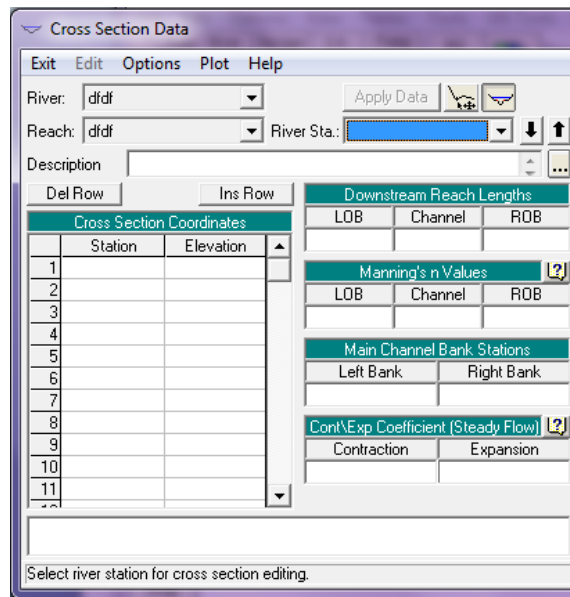
Figure 2. 19 River section details (HEC-RAS)

- Once >add new cross section pressed the HEC-RAS asks for the river station of the cross section which should be 10 for the upstream boundary and 0 for the downstream boundary. Since the same geometry data will be used in the following river sections HEC-RAS gives the ability to copy the details of the current cross section by pressing >Options >Copy current cross section, another important command gives the ability to interpolate the rest of cross sections which can save the time by going to >Tools >XS Interpolation as shown in the following figure:
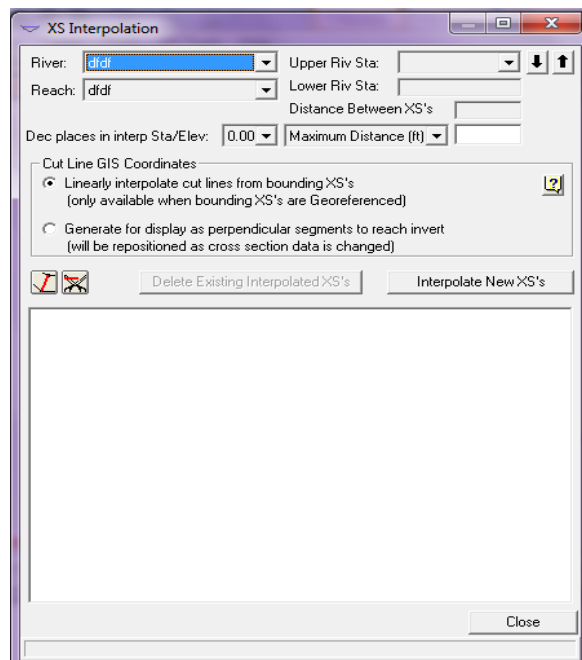
Figure 2. 20 XS Interpolation

28

- The last step before running the flow analysis is to define the initial conditions, the upstream and the downstream boundary conditions of the reach river by going to >Edit >Steady Flow Data and that if the flow analysis is steady state, if it is unsteady state it will be >Edit >Unsteady Flow Data. According to the available data the reach boundary conditions will be defined but the usual definition of the boundary conditions as the following:

1. The upstream boundary condition is being defined as flow hydrograph.
2. The downstream boundary condition is being defined as stage hydrograph.
3. The sluice gates structures are being defined as T.S. gate openings.

- The steady flow analysis and the unsteady flow analysis can be run by going to >Run >Steady flow analysis >Compute or >Run >Unsteady flow analysis >Compute. The starting, the ending time and the time step for the unsteady flow analysis should be inserted into the new opened window as shown in the following figure:
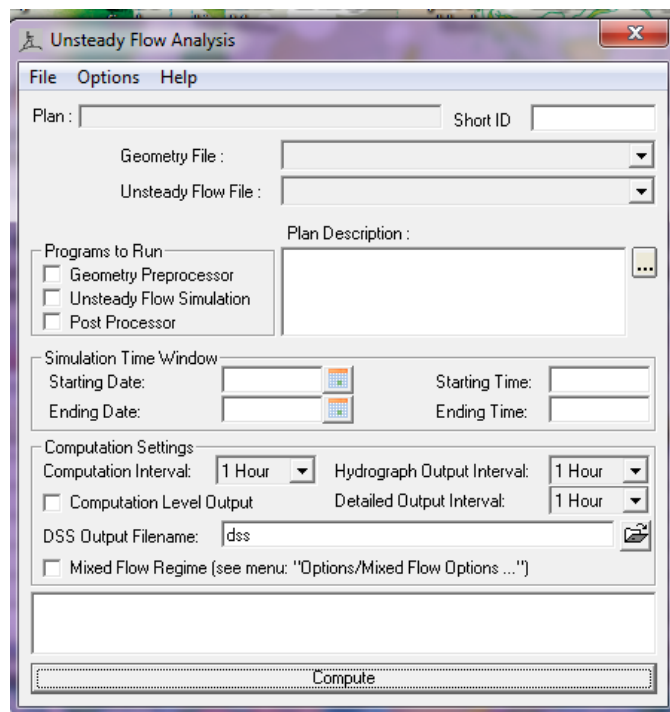
Figure 2. 21 Unsteady Analysis Window

- Finally the results of the steady flow analysis can be viewed by going to the icon of view summary output tables in the main window of the HEC-RAS, the unsteady flow analysis results can be viewed by going to >View >Unsteady flow time series plot which can show the results as either plots or tables of numbers.

Chapter 3: Methodology

3.1 Research Methodology

This chapter presents the research methodology concerning Numerical Simulation Analysis Using ISIS and HEC-RAS applications. The research aims to conduct different numerical simulation cases under different situations. In order to achieve the stated aim, the ISIS and the HEC-RAS applications were downloaded and so many cases were applied to the programs. Ideas on the type of each case have been asked and answered and the associated methods were generated and developed based on previous applications applied on the same topic presented in international journals. A draft version of the simulations was generated by the programs and then analyzed in order to correct the mistakes and regenerate those simulations again and again to reach to the final shape of the simulations. The two softwares system was developed in order to examine the research validity based on the identified objectives. After comparisons, comments and amendments, a final form of the simulations was produced as shown in chapter 4.

3.2 Simulation Cases Selection

Five simulation cases have been chosen to be conducted in both the ISIS and the HEC-RAS covering different types of numerical simulations:

1. Two of those cases are covering the dam break analysis topic when two different situations have been assumed the first one is simulating the dam failure as a sluice gate opening abruptly, the toce river dam break was the example which has been provided to this case and the second case represents one of the famous dam failures which is the Malpasset dam failure and the both cases were chosen according to previous published reports in international journals and once the simulation process finished the results have been compared to the reports results.

2. The third case has been chosen to simulate the sinusoidal oscillation, the tidal backwater effect has been simulated in the ISIS and the HEC-RAS and then compare together.

3. The fourth case shows how the dry river bed can be simulated in both applications and which is the simplest software in simulating the no flow condition and the change from the dry to the wet case.

4. The last simulation case shows how the flood routing for a rectangular channel can be simulated and this case has been chosen according to a previous report used the simplified dynamic model to describe the case; the results from both the SDM and the two applications have been compared together at the end.
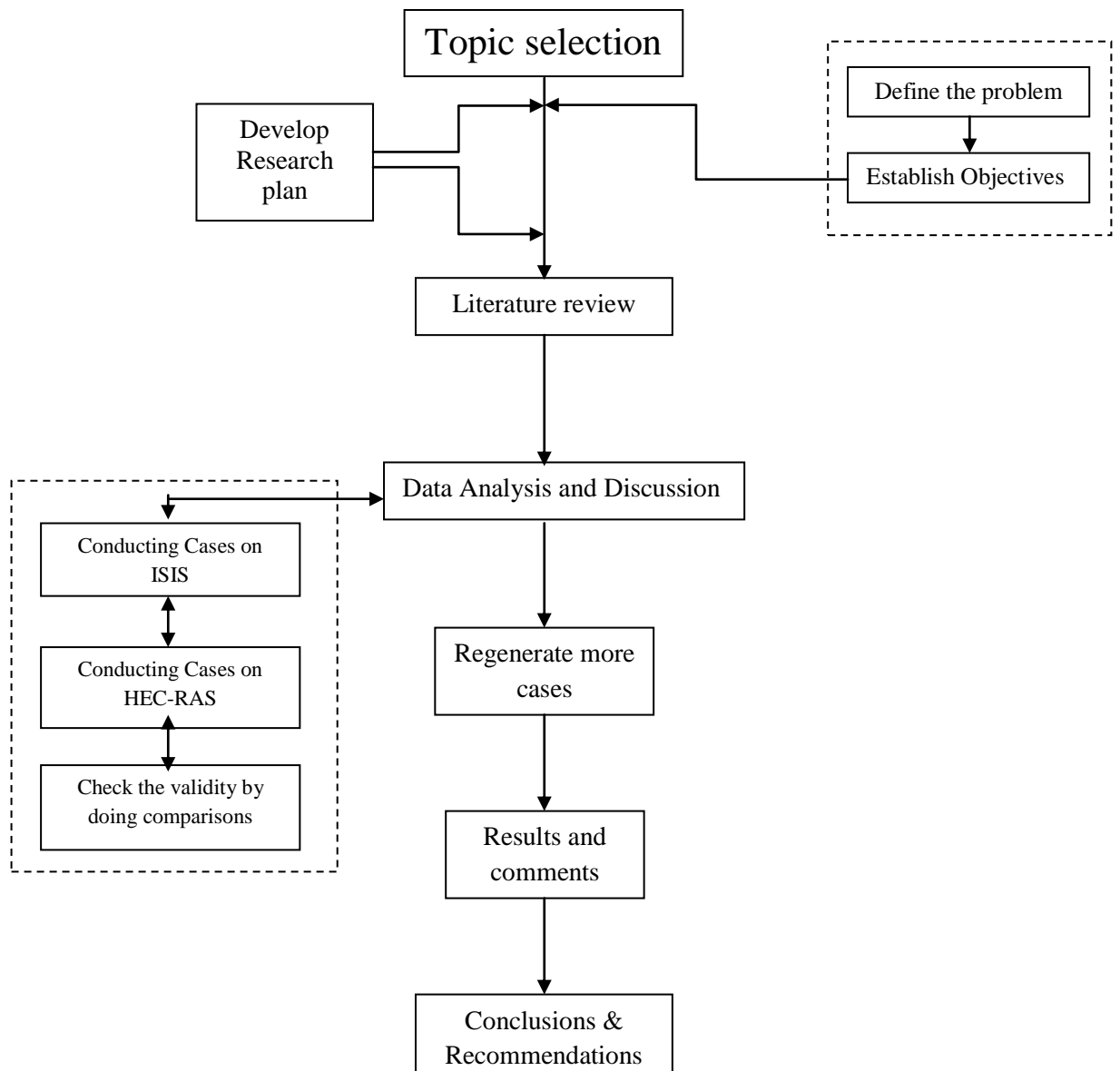
3.3 Research Methodology Flowchart

Figure3. 1 Flowchart of Methodology

3.4 Dual Software System Selection

The decision of the selection of the two softwares which have been used in this research has been made according to the following points:

- The ISIS and the HEC-RAS are considered to be the most popular softwares in the river modelling field in so many countries around the world and especially in the United Kingdom.

- The modern features which have been developed in the ISIS and the HEC-RAS as the model health checker, the results extractor, the 1D modelling, the 2D linked modelling and the friendly user interface all these productivity tools increased the degree of professionalism for the two softwares.

- Although there are main competitors to the ISIS and the HEC-RAS as the Mike 11 and the Infoworks RS the ISIS and the HEC-RAS are still considered to be the simplest and the most accurate softwares in the river modelling area.

3.5 Verification of the Simulation Results

The simulation results of the five cases have been double checked as shown in the following mechanism:

1. The first check by using the dual system software and compare the results together.
2. The second check by comparing the results from the softwares to the results of the previous reports.

Results obtained from the ISIS → Results Checked with the HEC-RAS → Results Checked with the reports results
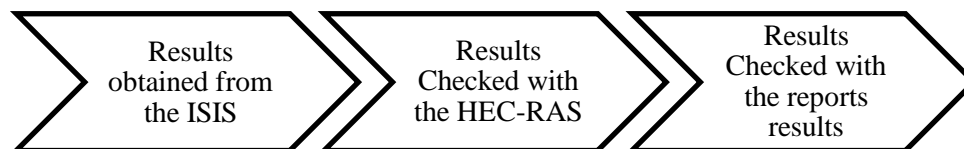
Figure3. 2 Process of the Verification of Results

3.6 Methodology of the Flow Analysis and Results

The flow analysis of each simulation case has passed two main stages the first one was the steady flow analysis and the second one was the unsteady state flow analysis, as mentioned before it is very necessary to conduct the steady flow analysis before the unsteady analysis in order to prepare the initial conditions for the flow analysis process. In each simulation case the results of the steady state flow analysis appeared once the simulation is completed but in the unsteady flow analysis it is necessary to extract the results using some commands in the softwares in order to get the following important results:

- The flow Hydrograph which include the flow values at each river section for every time step until the simulation time is finished.
- The stage Hydrograph which include the stage values against the simulation time values.
- Other results could be very useful as the velocity values and Froude number values.

Although the ISIS and the HEC-RAS give the ability to draw the flow and the stage hydrographs for each case the results have been extracted as tables and then moved to the Microsoft Excel in order to compare those values with other values extracted from other softwares or other reports.

Chapter 4: Simulation Analysis & Discussion

4.1 Case 1: Simulation of the dam breaks as a sluice gate opening abruptly through the dam

4.1.1 General description

The latest versions of the HEC-RAS model and the ISIS software include some new algorithms facilitate the modelling process of the overtopping and piping breaches. HEC-RAS and ISIS use hydraulic principles between the upstream and the downstream boundaries passing through the cross sections of the river. In this case in order to define the dam break process in both the ISIS software and the HEC-RAS model it has been considered that the dam crest as an inline weir and by increasing of the breach by the time the flow through the piping hole in the dam will be calculated as orifice flow and the flow through the breach will be considered as weir flow. This type of simulation is being used to simulate the overtopping and the piping failure in the dam breaks and in this scenario, the piping hole is simulated as a sluice gate. Sluice gates are available in three different forms:

1. Vertical gate
2. Radial (Tainter) gate
3. Drum gate

In this section the dam break is being studied by quickly opening the sluice gate which has been chosen to be vertical gate and then the water will go through this opening in the dam and there will still be difference between the upstream and the downstream water levels at different stages. Finally the water surface will be calculated in this test case in addition to the outflow hydrograph through number of steps in both softwares through the steady and the unsteady simulations and then the results with the comparisons between the two softwares have been produced in order to see the difference between the two softwares throughout the whole process.
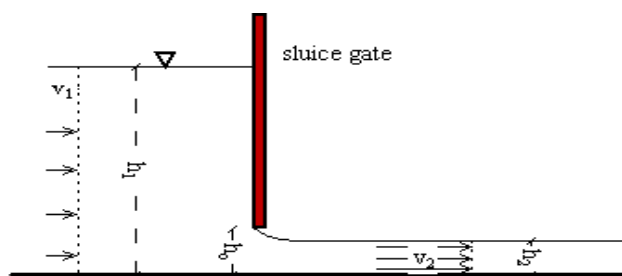
Figure 4.1 Sluice Gate Shape (www.engineeringtoolbox.com)

4.1.2 Test case 1 using the ISIS

In this section the test case 1 has been simulated through number of steps using the ISIS software.

- The upstream boundary has been defined by clicking on the Flow-Time Boundary picture button in the tools bar or select > Edit > Insert >Boundaries > Hydrographs > Flow/Time from the main menu and then the data has been filled into the window appeared as shown in figure 1:
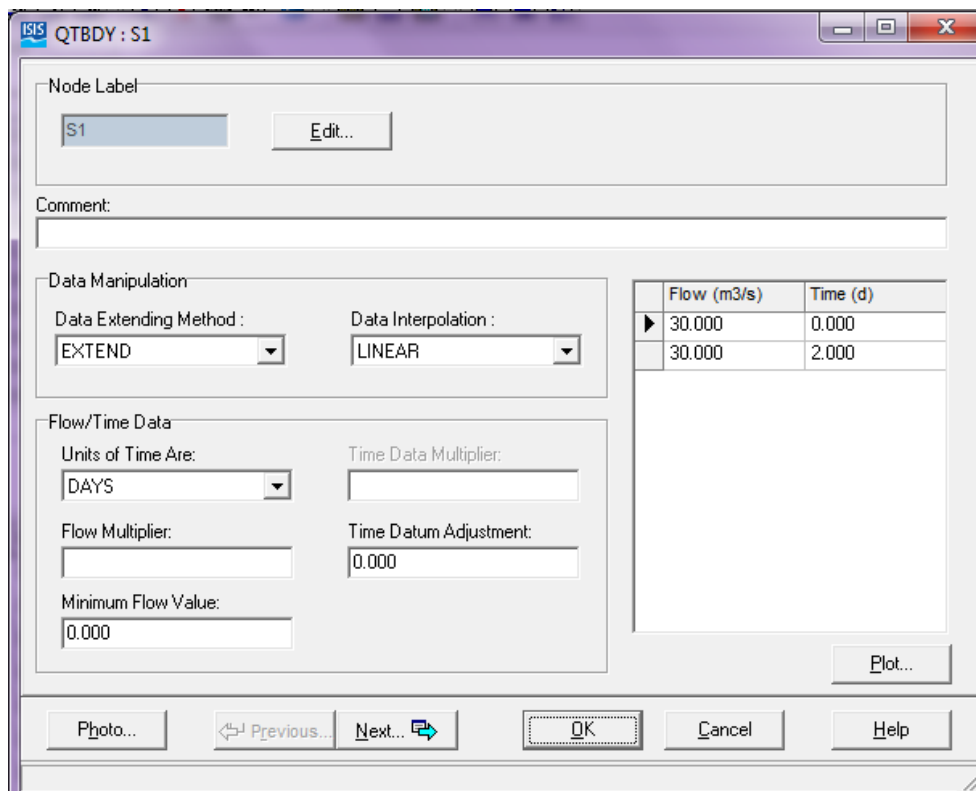
Figure 4.2 Upstream Boundary

- The river sections have been inserted using the picture button of the river section from the tools bar or by select > Edit > Insert > Channels > River > River Section from the main menu by entering the label of the cross section as S1 then the details of the river cross section which has been assumed to be as in the figure 2:
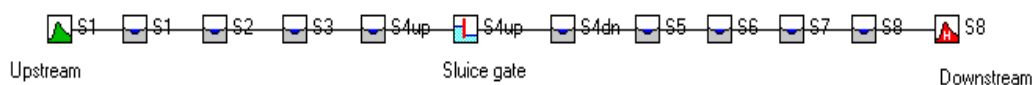
Figure 4.3 The River Profile for Test Case 1
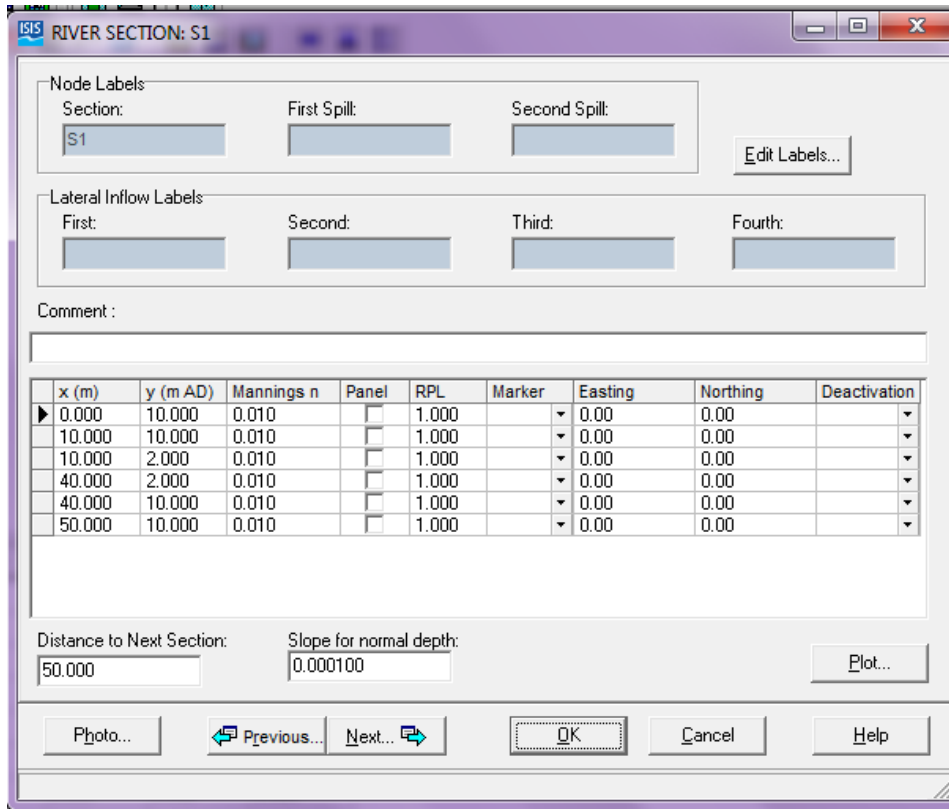
Figure 4.4 River Cross Section

- The second step has been repeated for S2, S3 and S4up. The only difference in S4up is the distance to the next section which is S4dn which has been inserted as zero as shown in the figure 3:
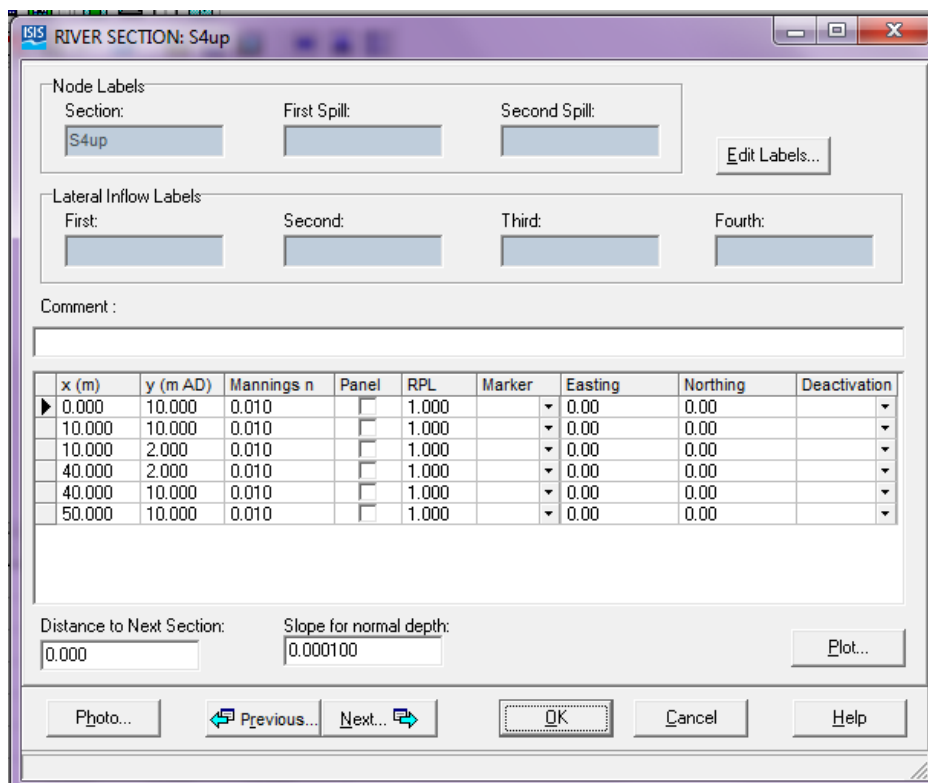
Figure 4.5 Section S4up

- Between sections S4up and S4dn the sluice gate has been inserted by clicking on the vertical sluice button picture in the tools bar then all assumed data of the weir and the gate has been inserted into the windows appeared which are shown in figures 4, 5 and 6. The dam data has been assumed to be as the following table 1. After the weir geometry has been edited the calculation method in the coefficients tap has been changed to variable mode:

Table 4. 1 Dam Information

Name	Value (meters)
Height of the gate (dam)	12
Elevation of the crest	2.1
Length of the weir	10
Breadth of the weir	30
Opening of the gate	5
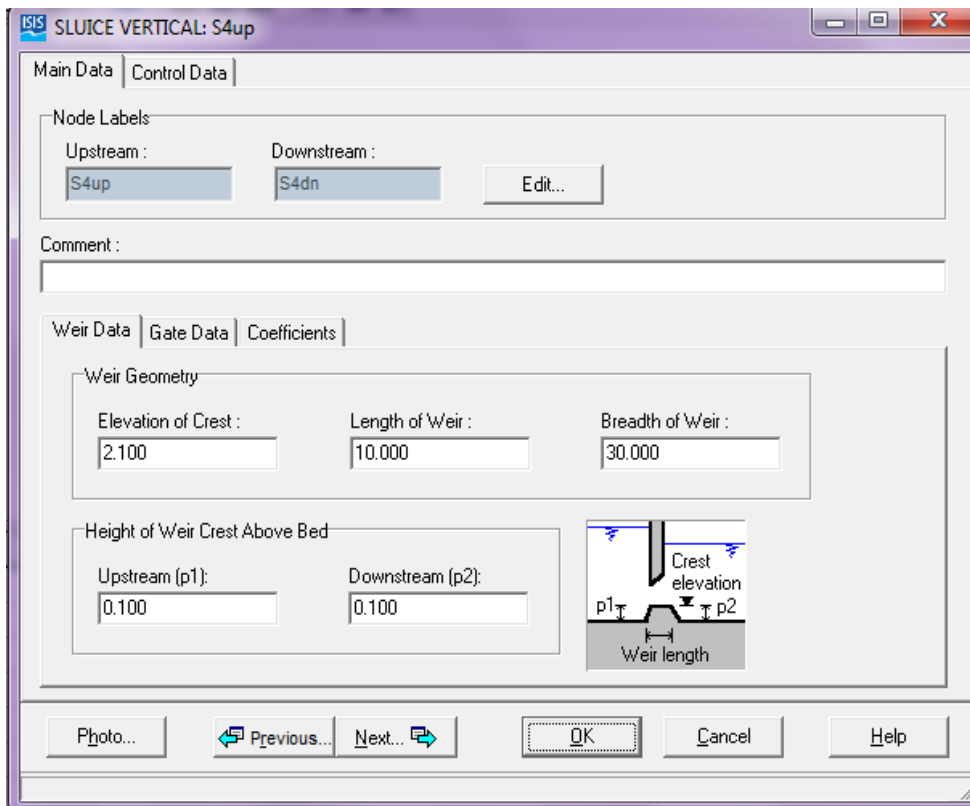
Figure 4.5 Weir data
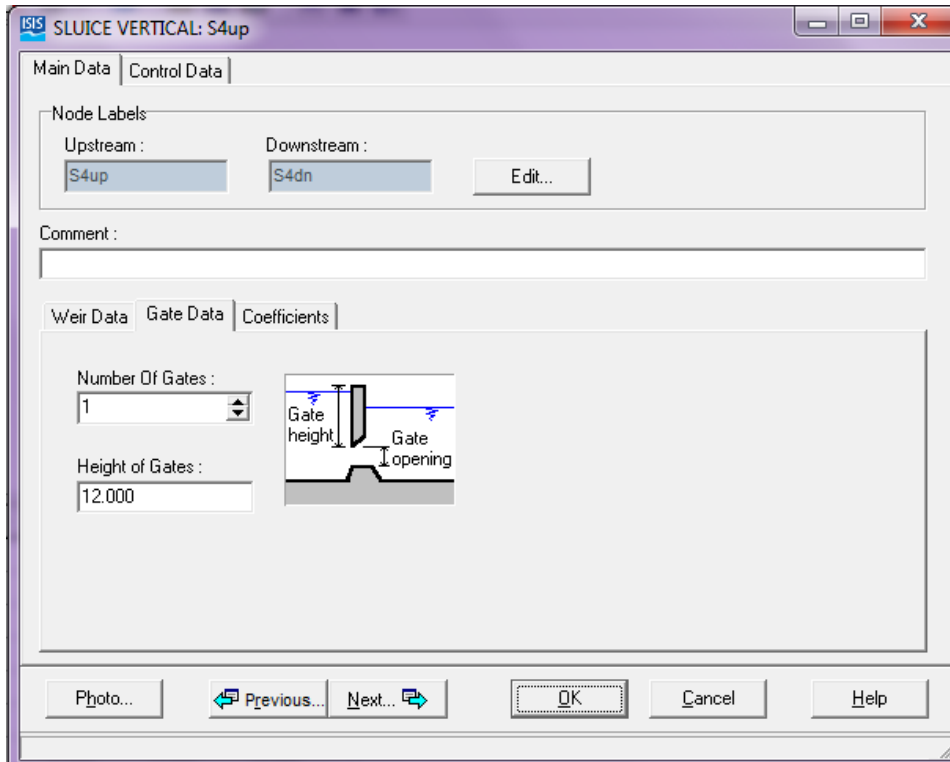
Figure 4.6 Gate Data
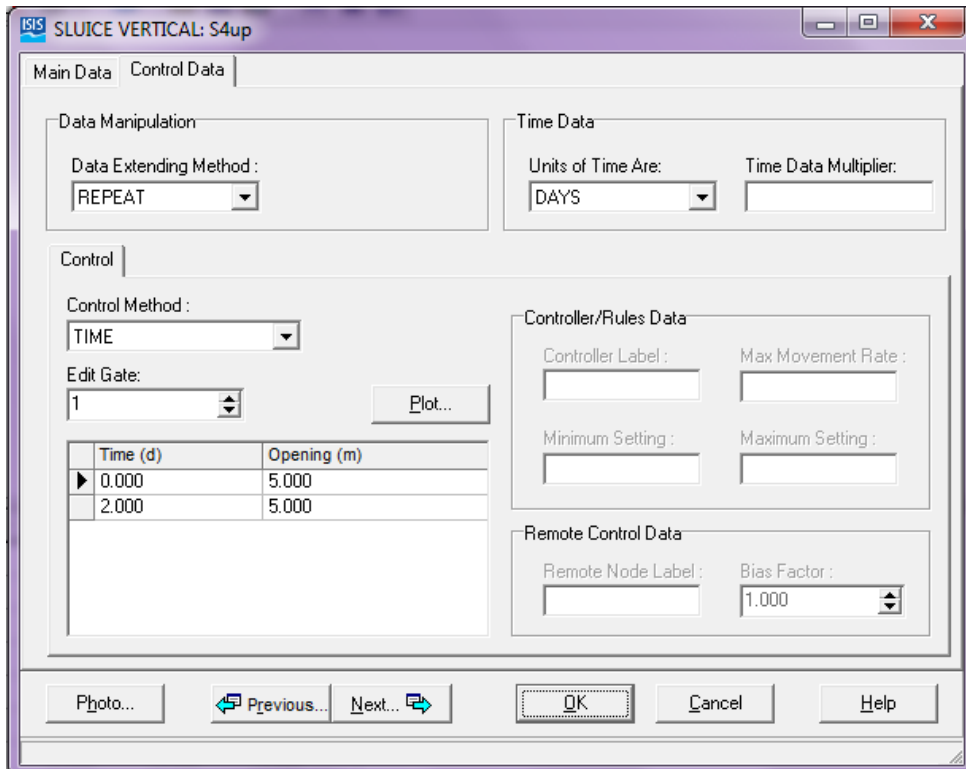
Figure 4.7 Gate opening data

- The rest of the river sections S4dn, S5, S6, S7 and S8 have been inserted with the same previous way as shown in figure 7 and the only difference in S8 is the distance to the next station which has been inserted as zero because it is the last section of the river profile. Then the downstream boundary of the river has been defined by clicking on the Head-Time Boundary button picture in the tools bar as shown in figure 8:
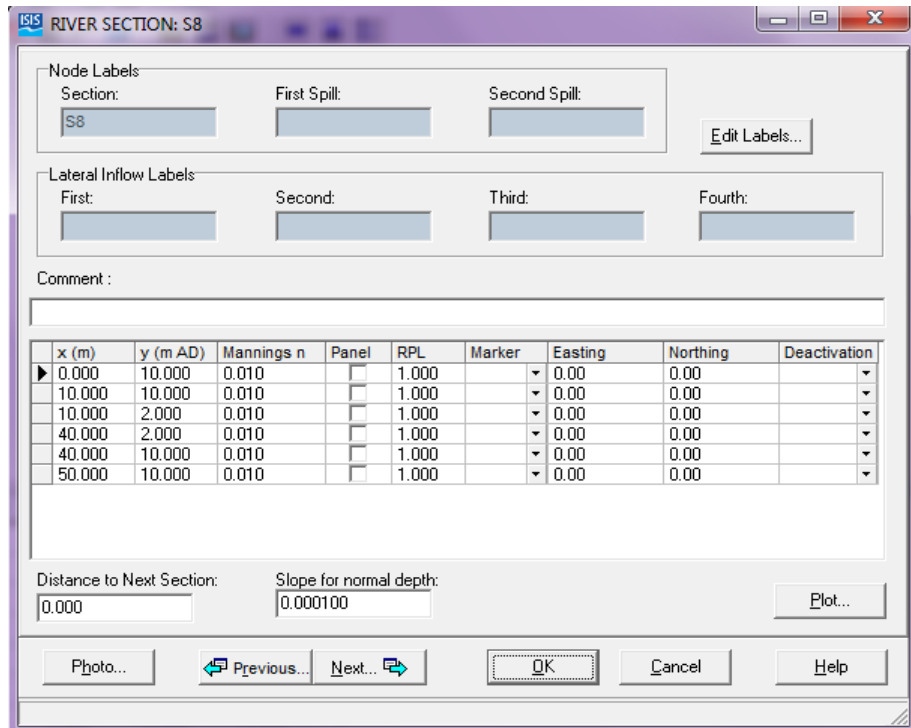
Figure 4.8 The last River Section
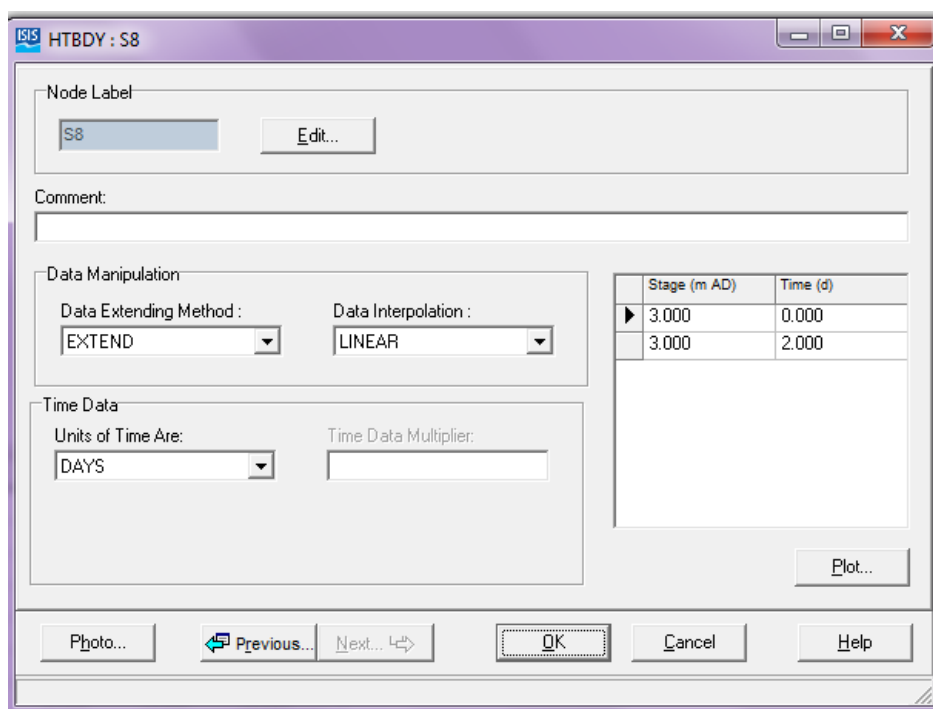
Figure 4.9 The Downstream Boundary

- The steady state simulation flow has been run by clicking on Run Flow Simulation button picture in the tools bar and it has been found that the free gate flow (UMODE) is 5 and the opening of the gate (USTATE) is 5 as shown in the following figures 10 and 11:
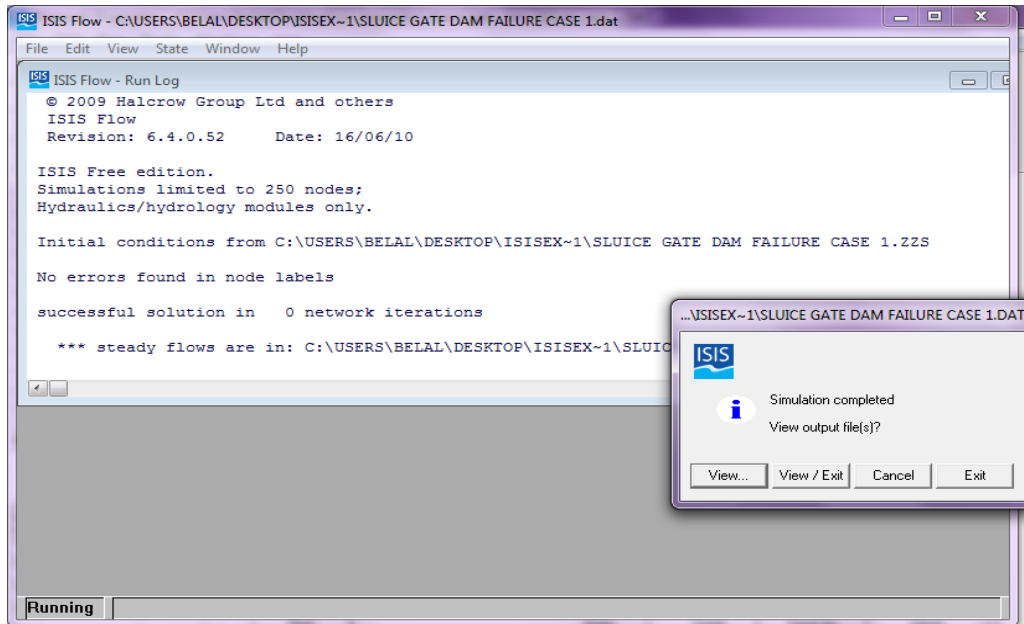
Figure 4.10 The Steady State Flow Simulation

Figure 4.11 Steady Simulation Results

- The unsteady state simulation flow has been run by clicking on Run Flow Simulation button picture in the tools bar and then select the unsteady (fixed time step), and if the time step changed to 3 and save interval into 60 and the finish time at 24 hrs, it has been found that flow stage decreases at the gate break location (Section 4) and the water volume below 3.003 meters can be neglected because 3.003 meters is considered as the minimum stage level and the 3.05 is the maximum stage record at the beginning of the river.
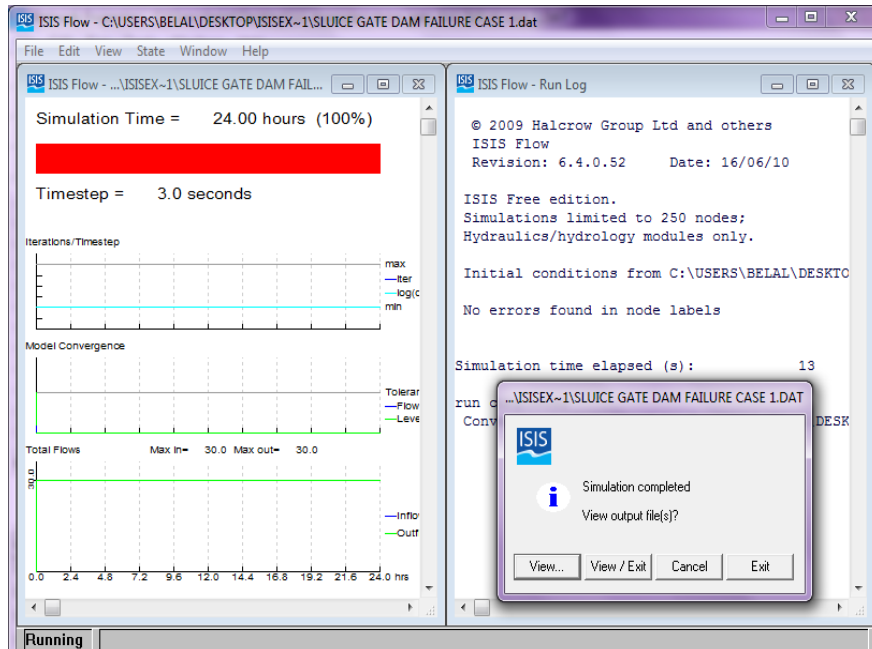
Figure 4.12 Unsteady simulation completed
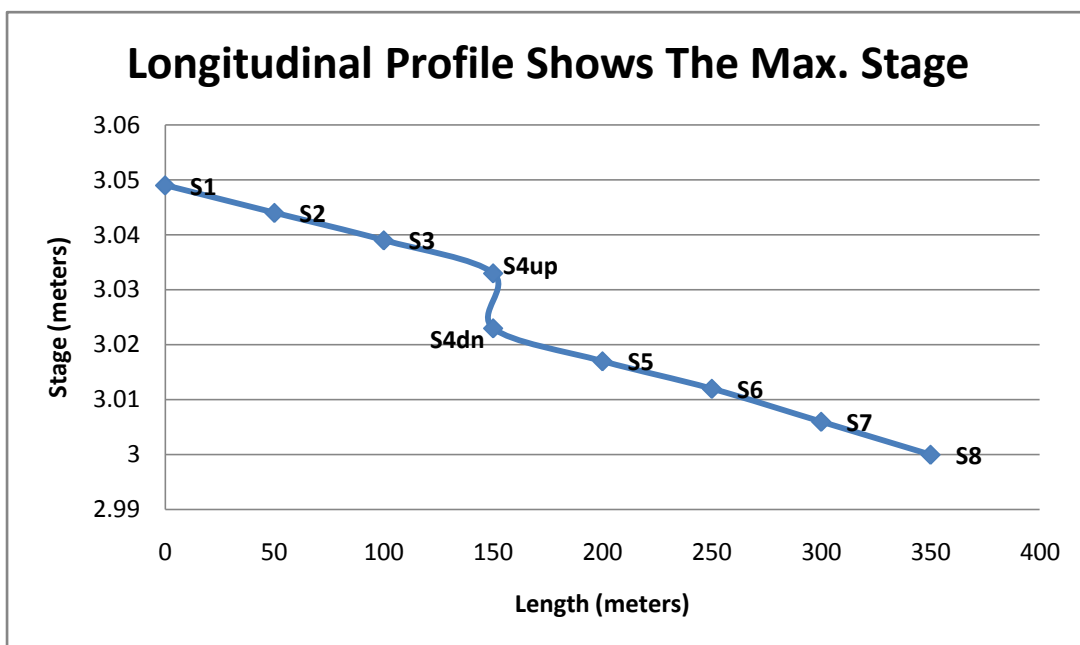
Figure 4.13 Longitudinal Profile

41

4.1.3 Toce river dam break test using ISIS

The test case 1 has been repeated according to the research paper on the Toce valley dam break analysis so the data of the upstream boundary conditions has been changed to be similar to that case by changing the values of the flow with time to be similar to the following values:

Table 4. 2 Upstream boundary conditions

Flow (m^3/s)	Time (s)
0.001	0.0000
0.001	15.000
0.050	17.000
0.100	20.000
0.150	22.000
0.200	23.000
0.228	25.000
0.220	27.000
0.225	29.000
0.180	30.000
0.150	50.000
0.100	100.00
0.060	180.00
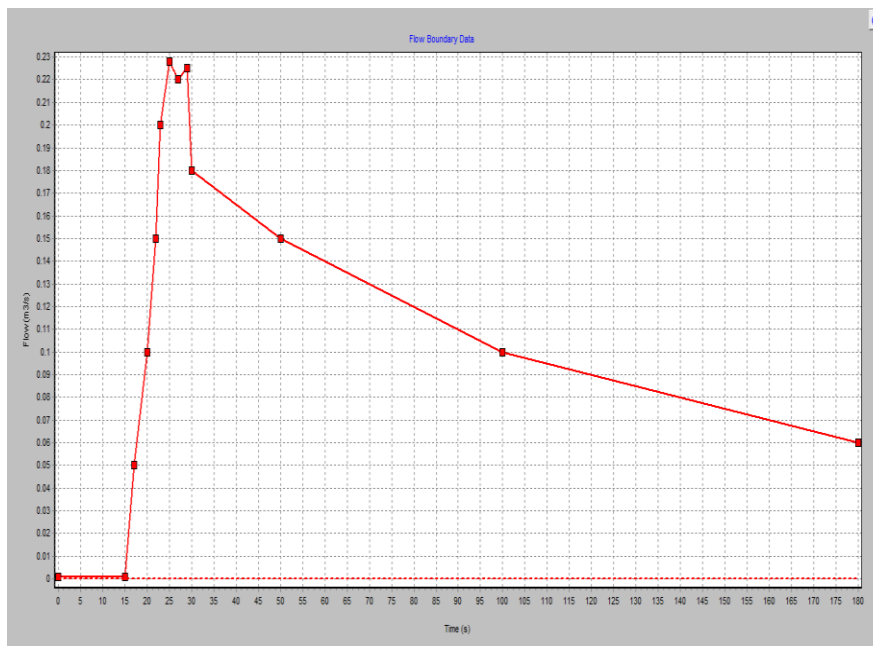
Figure 4.14 Flow Vs Time

By running the steady state simulation first the following results have been appeared, the free gate flow (UMODE) is 4 and the opening of the gate (USTATE) is 0.25 as shown in the following figure:

```
                                    FILE=SLUICE GATE DAM    ISIS VER= 6.4.0.52
    results from the direct method at time    0.0000 hours
label12     ?      flow     stage    froude   velocity     umode     ustate        z
S1          y      0.001    7.250    0.000     0.000       0.000      0.000     2.000
S2          y      0.001    7.250    0.000     0.000       0.000      0.000     2.000
S3          y      0.001    7.250    0.000     0.000       0.000      0.000     2.000
S4up        y      0.001    7.250    0.000     0.000       4.000      0.250     2.000
S4dn        y      0.001    3.000    0.000     0.000       0.000      0.000     2.000
S5          y      0.001    3.000    0.000     0.000       0.000      0.000     2.000
S6          y      0.001    3.000    0.000     0.000       0.000      0.000     2.000
S7          y      0.001    3.000    0.000     0.000       0.000      0.000     2.000
S8          y      0.001    3.000    0.000     0.000       0.000      0.000     2.000
```

Figure 4.15 steady state results

The main purpose of the steady state simulation is to make the boundary conditions ready to the simulation process so for that reason the steady state simulation should be run first, when the unsteady (fixed time step) has been run by selecting the time step to be 3 and the interval to be 60 so the simulation process completed successfully and the longitudinal section of the river through the river sections from S1 to S8 has been found to be in the following shape:
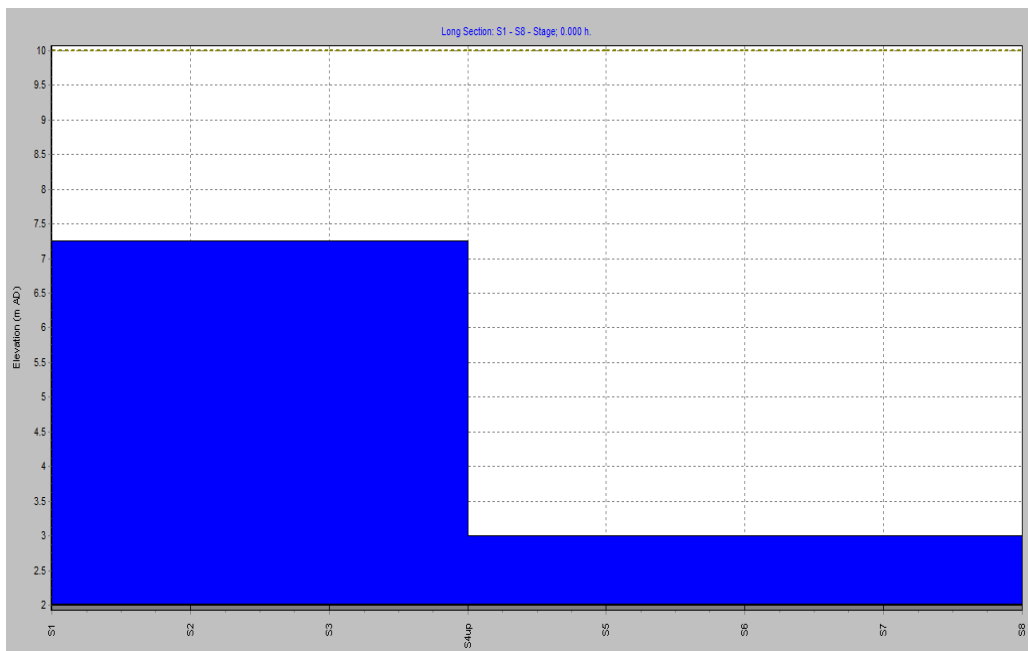
Figure 4.16 Unsteady state (fixed time step) results

At the river section S4dn if the time series plot has been drawn in the unsteady (fixed time step), the plot will be as in the following figure:
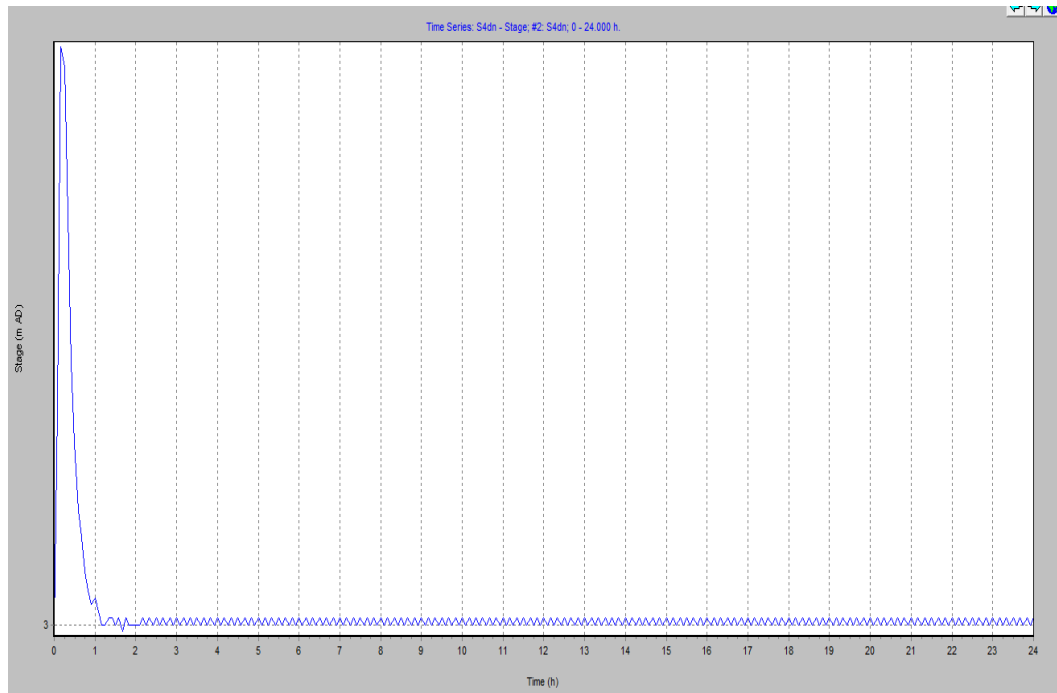
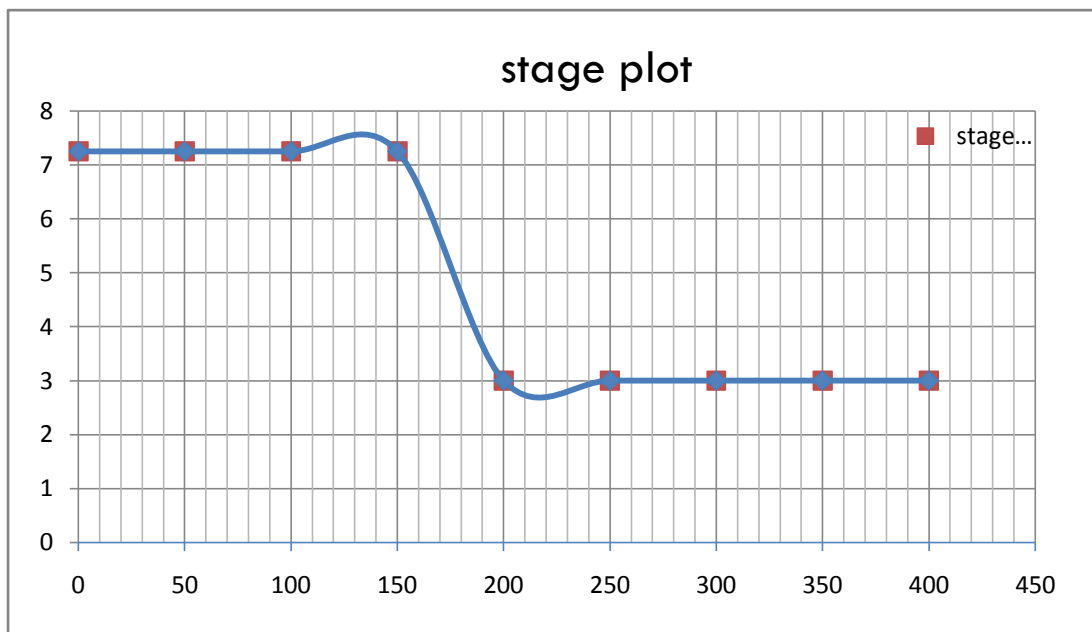**Figure 4.17 Time series for unsteady adaptive time step**

Figure 4.18 Stage plot for test case 1

As shown in the figure above that the stage plot of the eight sections, by changing the distance it has been found that the stage is changing but the inflection point is between 120 and 220 meters.

44

4.1.4 Test case 1 using the HEC-RAS

In this section the test case 1 has been simulated through number of steps using the HEC-RAS software.

- A new project has been started by opening the HEC-RAS window then from the file tap a new project can be started and titled then it can be saved as a file in any directory as shown in the following figure:
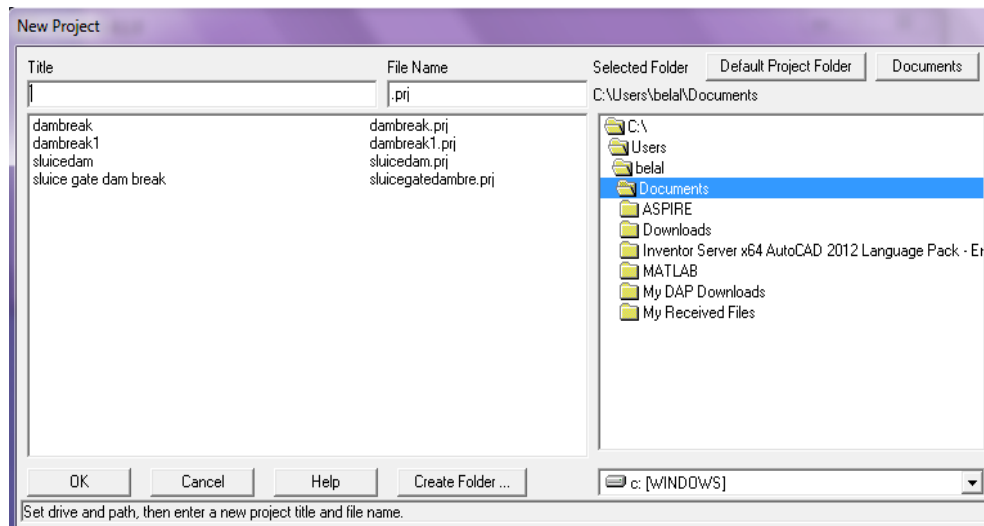
Figure 4.19 starting new project in the HEC-RAS

- After the new project has been created in the HEC-RAS main window, the geometric data of the new project needs to be entered by clicking on the geometric data button picture from the main menu in the software, a new window has been appeared from this window a new river reach has been defined by drawing the upstream and the downstream boundaries of the river.
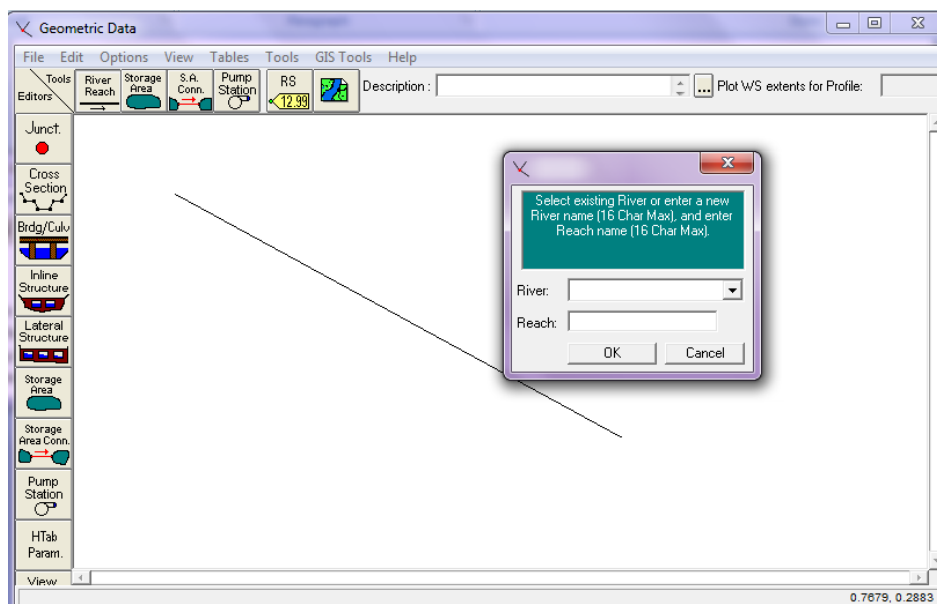
Figure 4.20 Entering the geometric data into the HEC-RAS

45

As shown in the previous figure that the software asks for the name of this river and reach which can be named as the same name of this project.

- The cross sections of the rivers have been inserted into the new project by pressing the cross section button picture from the left hand side in the new project window then a new window has been appeared and then the software asked for adding a new cross section in order to give the ability to put all the details of the cross sections of the river which have been assumed and inserted in there as shown in the following figure:
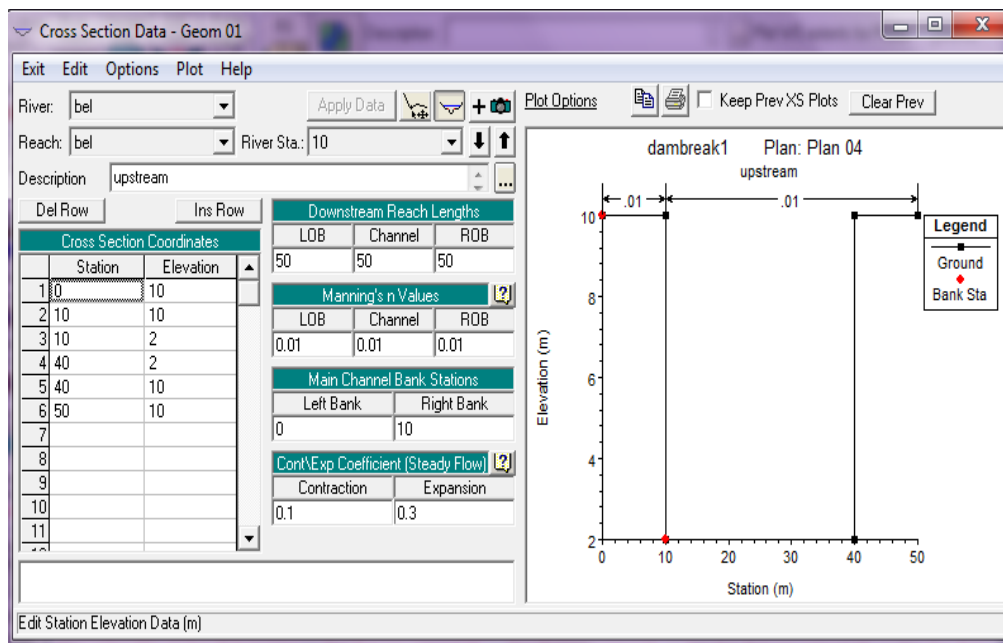
Figure 4.21 cross sections details

The previous cross section has been described to be the upstream river station which has been assumed to be 10 and with a distance equals 50 meters from the next cross section and the manning coefficient has been assumed to be 0.01 as shown in the figure above.

- The cross section at the downstream river station has been defined with the same way of the upstream but this time the downstream river station has been assumed to be 0 and the other parameters have been left to be the as the same in the upstream river station.

- The river cross sections between the upstream and the downstream river stations have been inserted into the project by using the interpolation tool which gives the ability to interpolate the distance between the upstream and the downstream boundaries because of the uniform river sections as shown in the following figures:
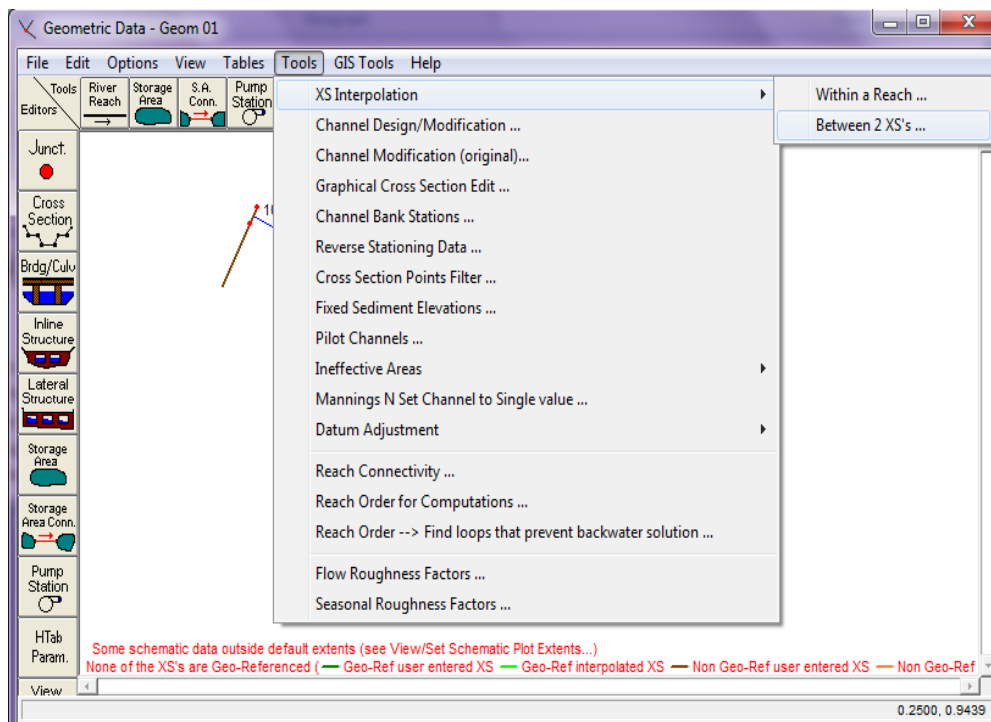
Figure 4.22 cross sections interpolation
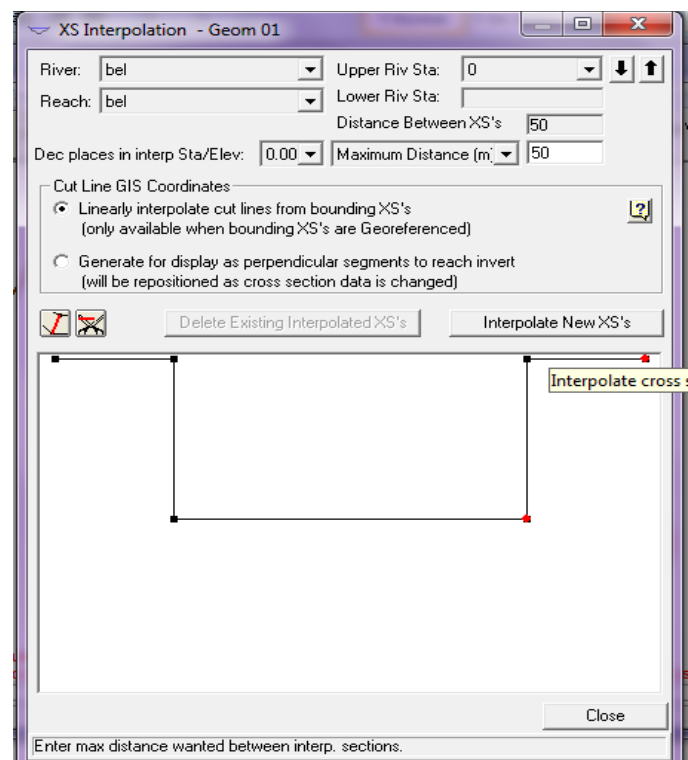
Figure 4.23 XS interpolation between river stations

The maximum interpolated distance has been assumed to be 50 meters and then the button "Interpolate New XS's" has been pressed to give the river channel with different stations between the upstream and the downstream boundaries as shown in figure 27.

- In this step the dam has been inserted as a vertical sluice gate between the fourth and the fifth sections of the river channel by clicking on the "Inline Structure" button picture from the left hand side tap then a new window has been appeared and press ass inline structure from the options menu, the river station of the inline structure has been assumed to be 4.5 (which is approximately the mid distance between the fourth and the fifth sections of the river channel), then the geometric information of the gate and the weir embankment has been added as shown in the following figures:
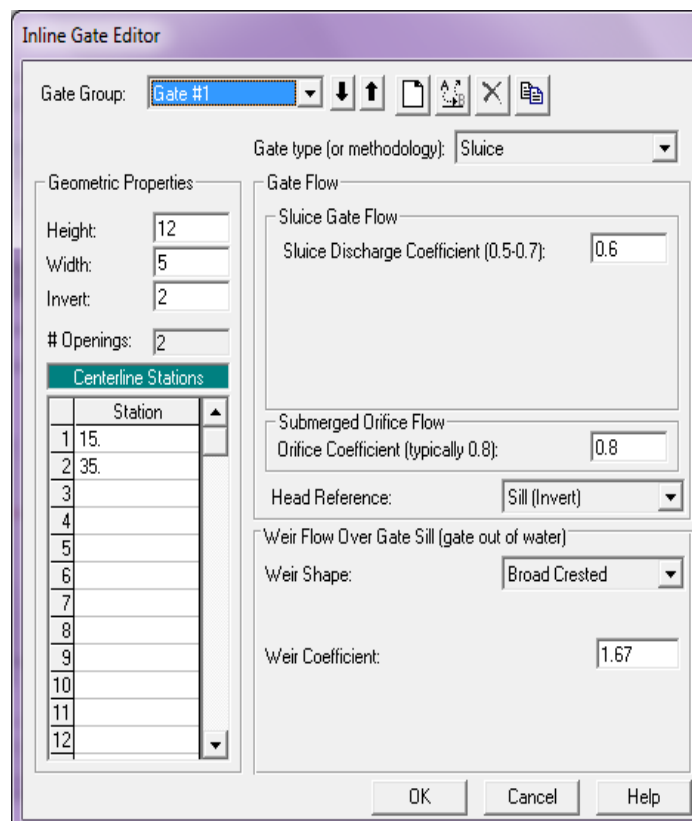
Figure 4.24 gate opening information

The number of gate openings has been assumed to be 2 times as shown in the figure above, the gate height has been assumed to be 12, the width is 5 and the invert is 2. It has been assumed two elevations coordinates for two different weir stations as shown in the following figure:
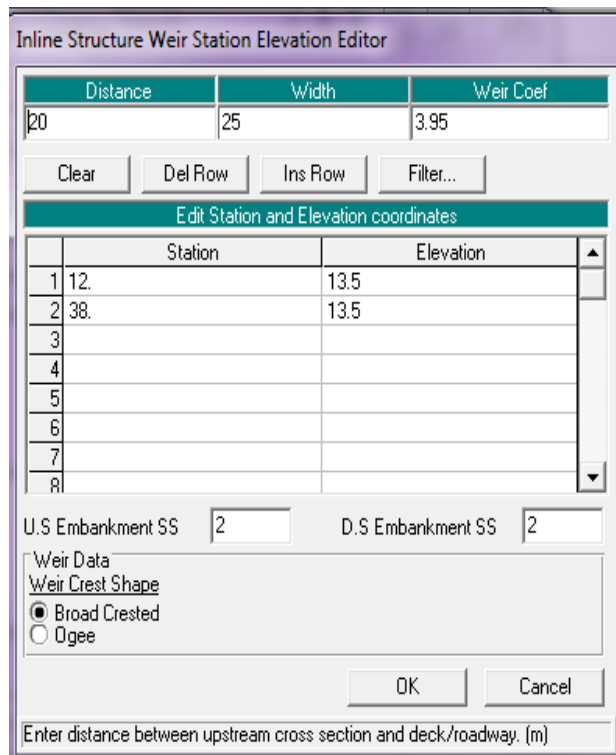
Figure 4.25 weir stations & elevations
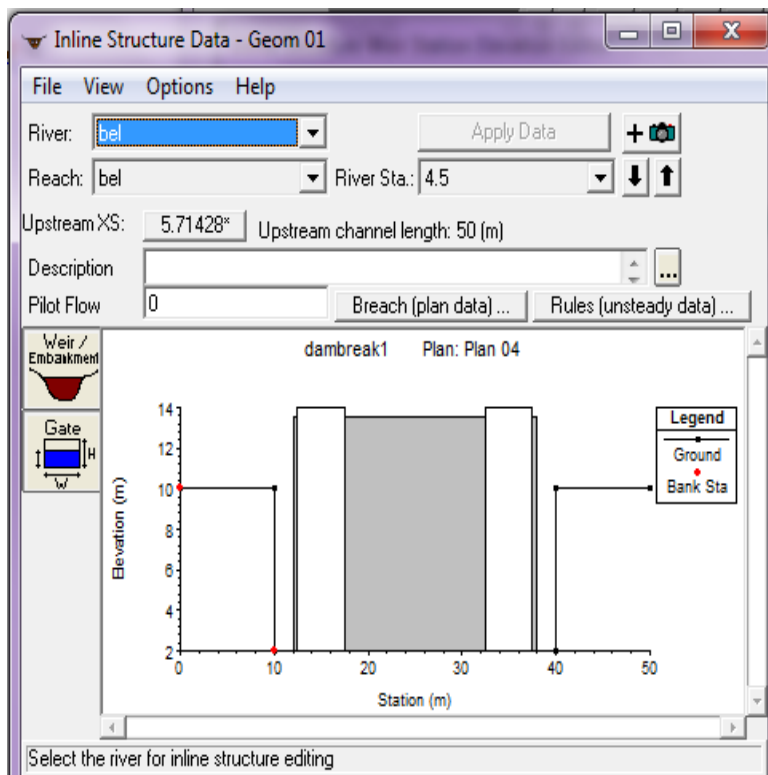
Figure 4.26 Inline Structure Data
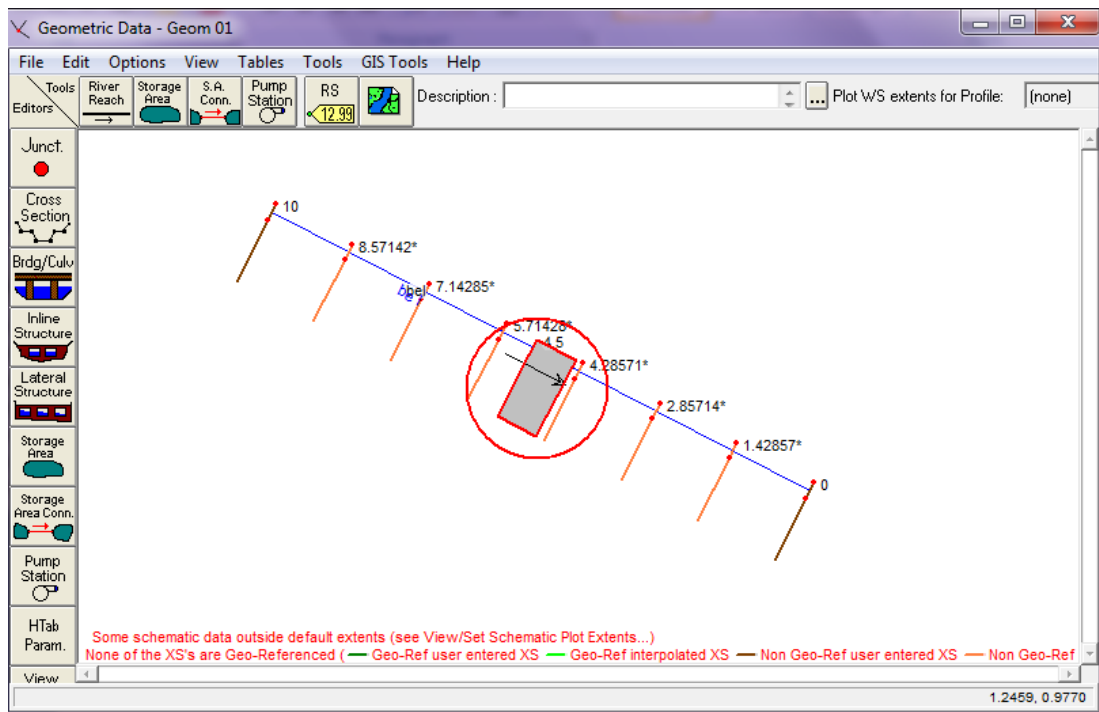
Figure 4.27 River channel profile

- In this step some more information has been added to the project before running the simulation process: The gate opening information: it has been added by pressing the "steady flow data" button from the edit menu then >options >gate openings, the open height has been assumed to be 5 meters as shown in the following figure:
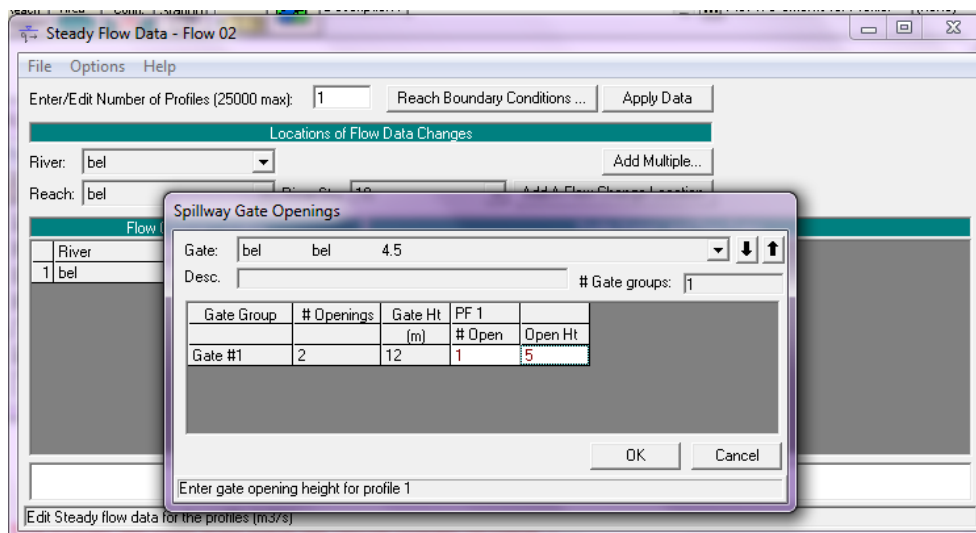

Figure 4.28 Gate openings information

- Before running the steady state and the unsteady state analysis the reach conditions for the upstream and the downstream should be defined, there are different ways to define the reach conditions as the following:

Steady state analysis:

For the upstream boundary and the downstream boundary conditions, they can be defined using one of the following methods (go to >edit >steady flow data):

a. Enter the rating curve (by inserting the changes in values of stage against the flow).

b. Enter the downstream slope for normal depth computation.

c. Set the condition to be a critical depth (unknown value).

d. Enter the water surface elevations for different values of flow.

Unsteady state analysis:

In the unsteady state analysis the upstream boundary can be defined using one of the followings parameters (go to >edit >unsteady flow data):

a. Stage hydrograph.

b. Flow hydrograph.

c. Stage/flow hydrograph.

The downstream boundary condition can be defined in the same parameters of the upstream boundary plus the rating curve and the normal depth. So from those parameters one of them is enough to define the boundary conditions which have been chosen to be the stage hydrograph in this test case
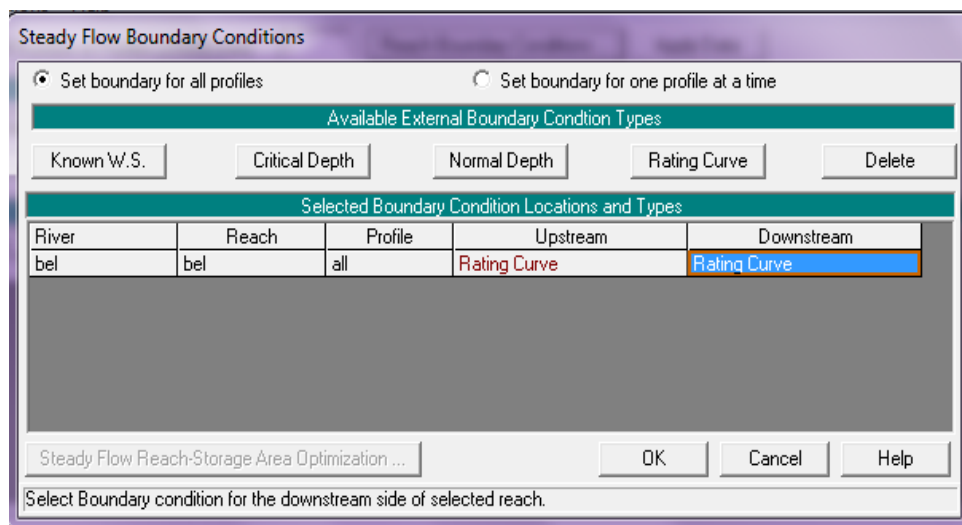
Figure 4.29 define the boundary conditions

- In this step the steady state analysis has been run and the information of the gate opening has been provided through the HEC-RAS tables from the main menu and it was as in the following figures:
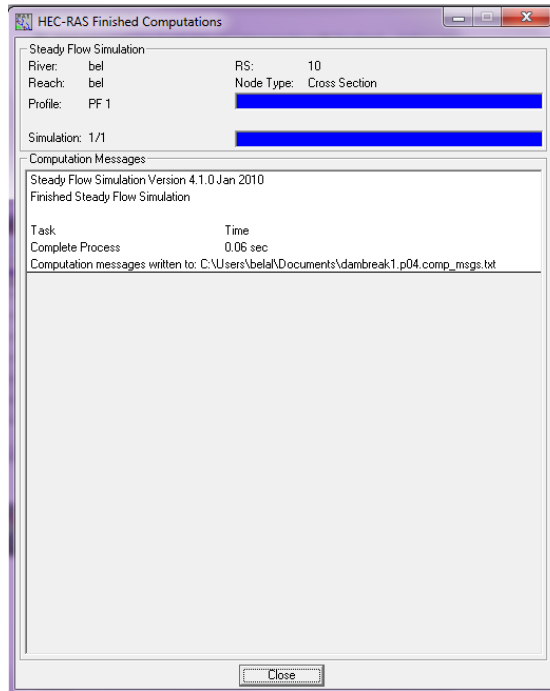
Figure 4.30 Steady state analysis complete

Figure 4.31 Results of the steady state analysis

From the results of the steady state analysis, it has been found that the flow of the river at the gate equals 10.37 m^3/s and the elevation of the gate is around 3.15 meters. The general profile of the river and the water surface profile have been shown in the following figure:
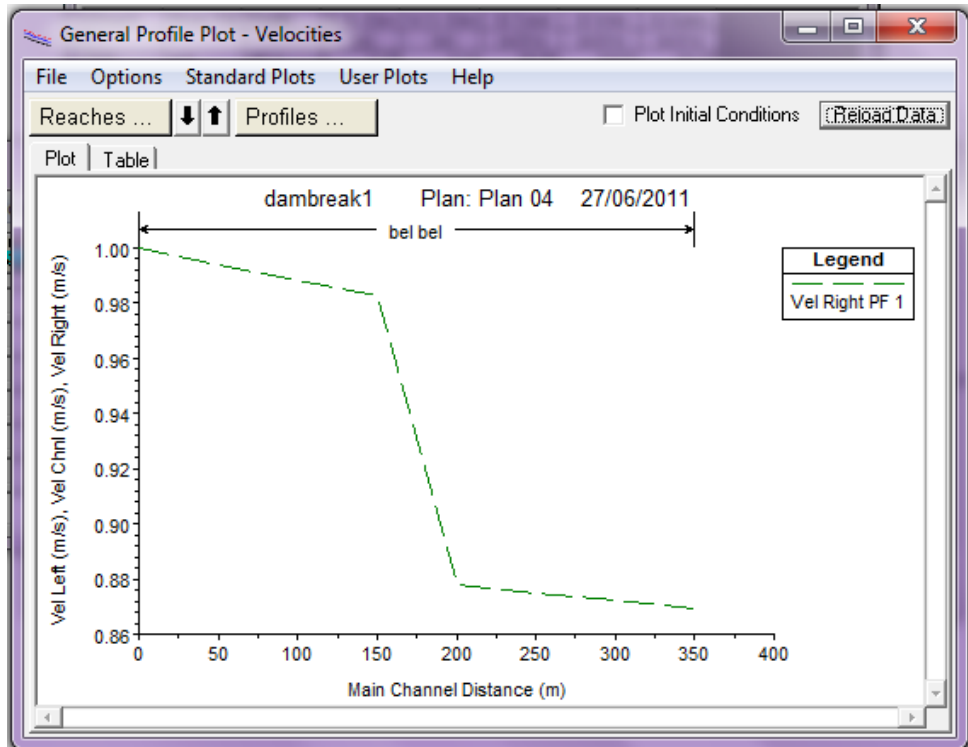
Figure 4.32 General profile plot

- The last step in this analysis is to run the unsteady state analysis in the HEC-RAS software then compare its results with the results obtained from the ISIS, the results have been grouped in the following graph to compare between the two softwares:
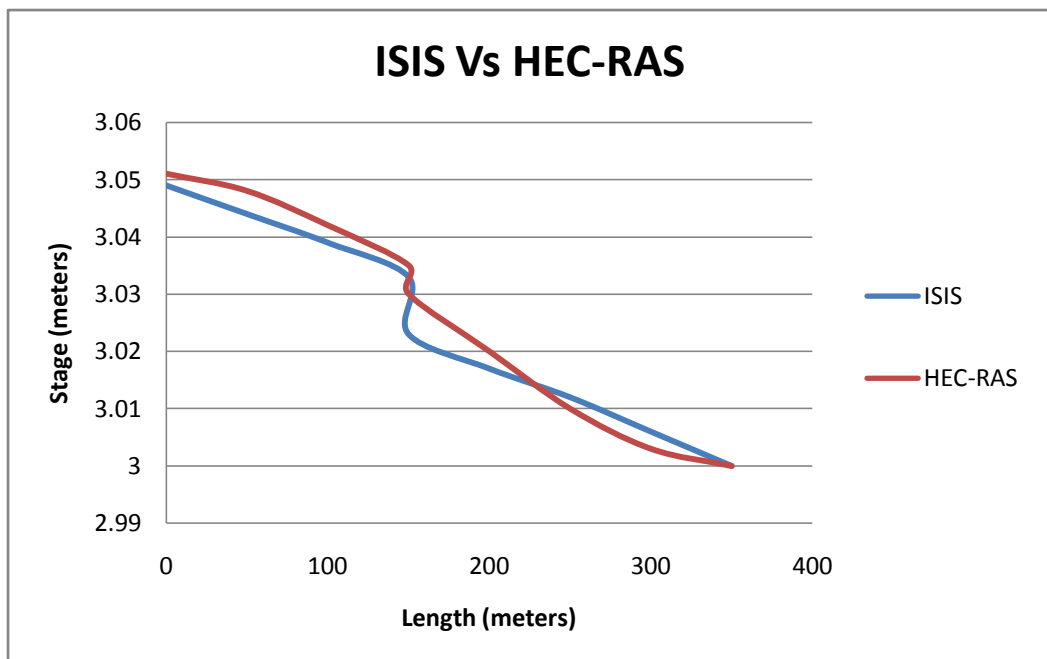
Figure 4.33 Stage Hydrograph (ISIS Vs HEC-RAS)

From the figure above it has been noticed that the HEC-RAS is higher than the ISIS stage values but both values still acceptable and close to each other.

4.2 Case2: Malpasset dam-break test

4.2.1 General description

The Malpasset dam is a double curvature arch dam type with a maximum height of 66.5 meters, crest length of 223 meters and reservoir capacity of 55 Mm3. The main purpose of the dam was the irrigation and the storage drinking water usage, the dam failed on the second of December 1959 and just a little part of the arch dam remained. The researches reasoned the failure of the dam to the high pore water pressure in the rock under the heavy rains happened during the construction process of the dam (Ahmed and Morris, 1999).

A comparison has been made between the ISIS numerical model results and the physical model results by (Ahmed and Morris, 1999) throughout simulating the Malpasset dam failure using the ISIS software by considering the dam failure as 3 sluice gates opening 10 seconds per each one and then drawing the outflow hydrograph with the hydrograph resulted from the physical model in one graph in order to watch the difference between the two curves. The journal run two simulations; the first one used the EDF hydrograph data and the second one used the computed hydrograph and in order to have this process done in the ISIS the two simulation processes have been done separately. The following graph shows the difference between the ISIS hydrograph and the EDF hydrograph where the ISIS hydrograph has been found to be longer than the physical one.
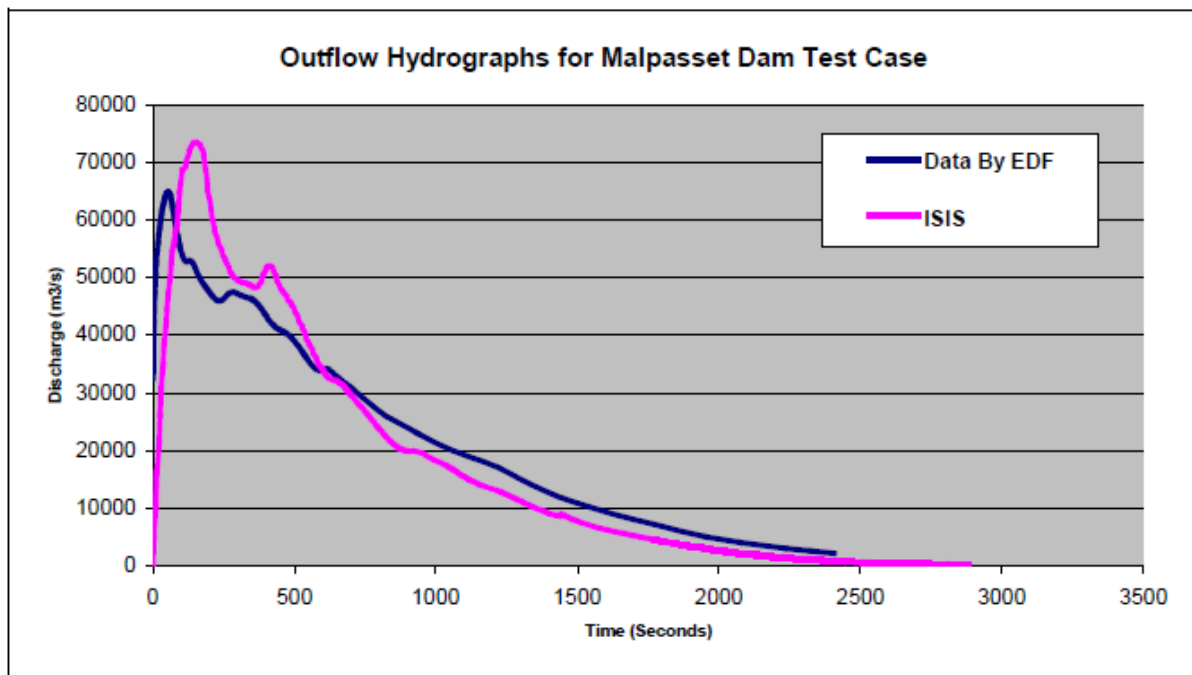
Figure 4.34 Outflow hydrograph, (Ahmed and Morris, 1997)

4.2.2 Test case 2 using ISIS

The simulation process steps of this test case are totally similar to the first test case with only some changes listed below:

- The upstream boundary condition was imposed to be small flow of 10 m^3/s which is the same as assumed in the journal and the downstream boundary condition to be a rating curve with fixed stage of 3 meters as shown in the following figures:
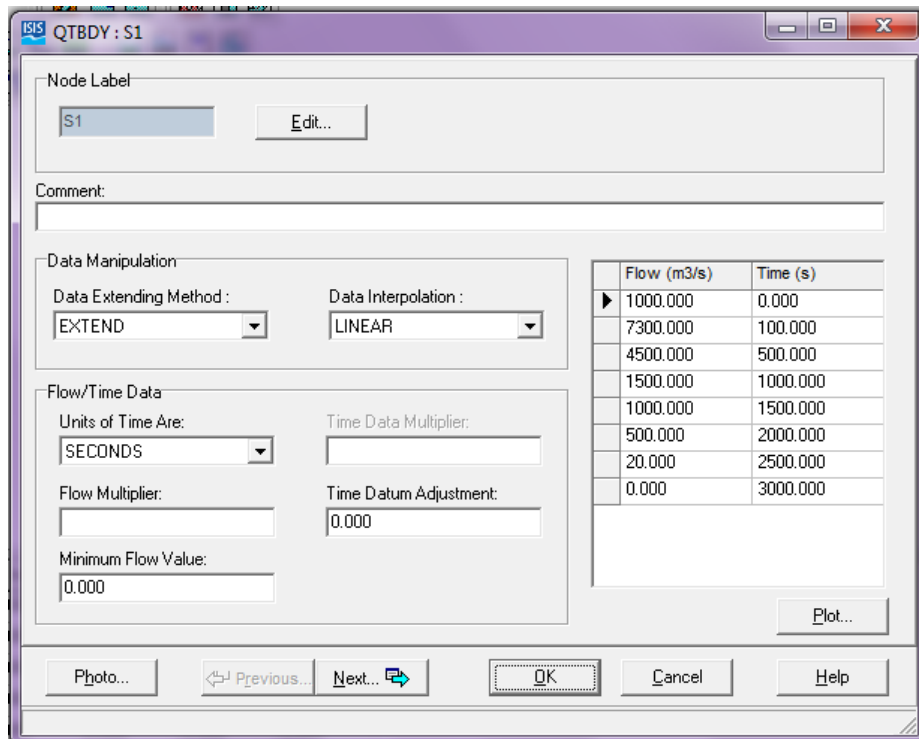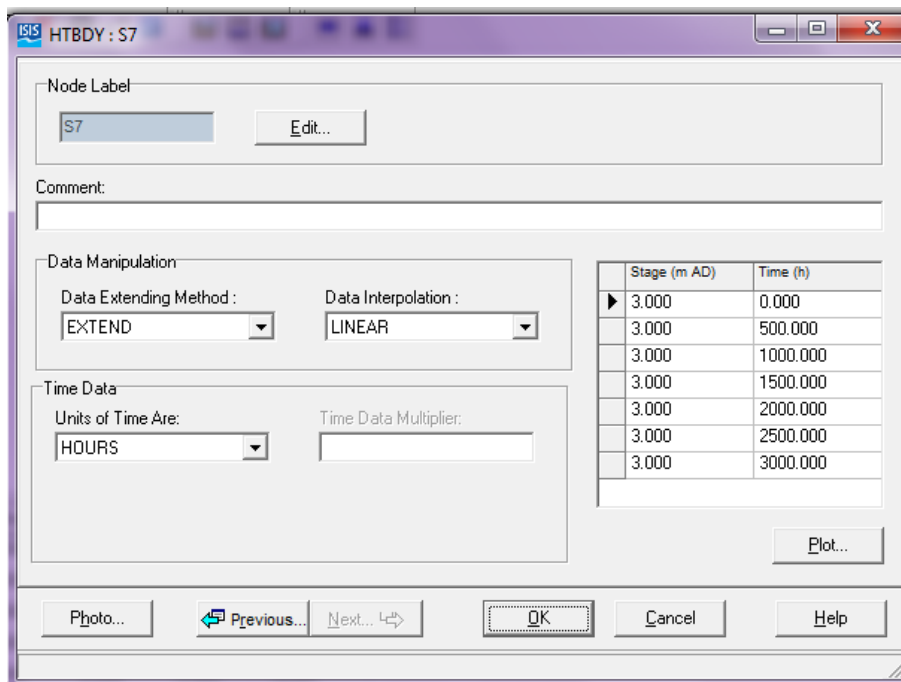

Figure 4.35 upstream boundary condition

Figure 4.36 Downstream boundary condition

55

- The dam break has been simulated using 3 radial sluice gates opening 10 seconds per each one so the total opening time is 30 seconds which is the same assumed time in the journal, the following information has been assumed to be the same as in the journal as well:

Table 4. 3 Gate data

Data	values
Height of the gates	9.7 meters
Radius of gates	15 meters
Height of the pivot	14.7 meters
Elevation of the crest	56.8 meters
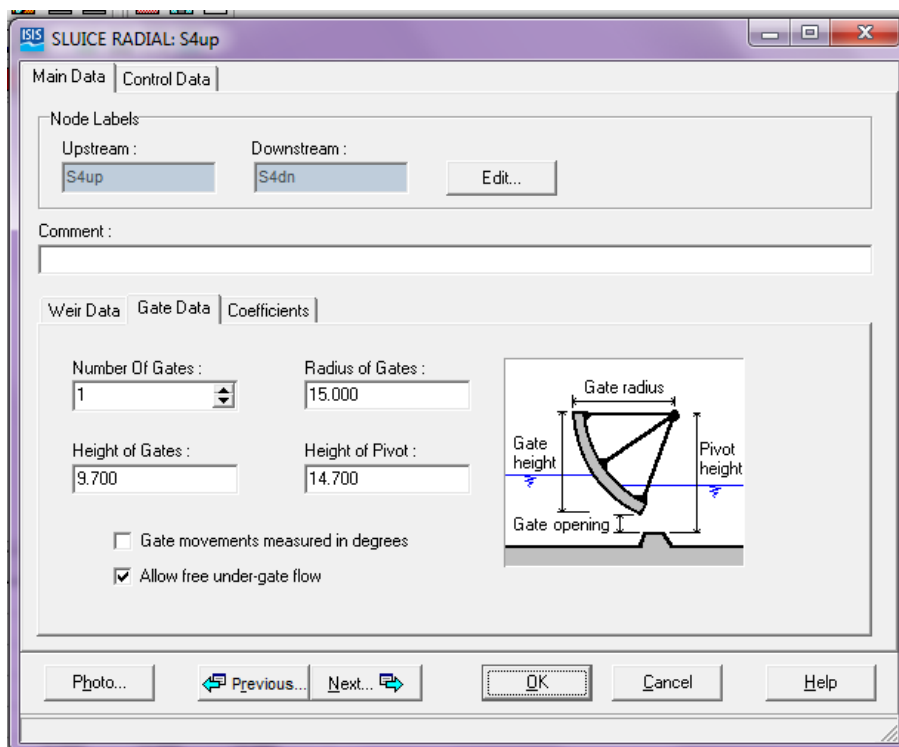
Figure 4.37 Sluice radial gate information

As mentioned before that the ISIS cannot run two different models at the same time so the two different models should be run separately then the results of the two models will be combined together in one Excel graph as shown in the following steps:

- The first step in the dam break analysis process is to run the steady state flow analysis and the results of that analysis have been found in the following form which shows the big difference in the velocity values before and after the dam location point :

```
                                        FILE=LAST.dat
ISIS VER= 6.4.0.52
 results from the direct method at time     0.0000 hours
label12       ?       flow     stage    froude   velocity      umode
ustate        z
S1           y   1000.000    73.185     0.008     0.227       0.000
0.000     0.000
S2           y   1000.000    73.185     0.008     0.227       0.000
0.000     0.000
S3           y   1000.000    73.185     0.008     0.227       0.000
0.000     0.000
S4up         y   1000.000    73.184     0.008     0.227       2.000
0.000    56.700
S4dn         y   1000.000     5.134     0.457     3.246       0.000
0.000     0.000
S5           y   1000.000     4.781     0.509     3.486       0.000
0.000     0.000
S6           y   1000.000     4.268     0.603     3.904       0.000
0.000     0.000
S7           y   1000.000     3.000     1.024     5.555       0.000
0.000     0.000
```

Figure 4.38 Steady state analysis results

- Secondly the unsteady fixed time step flow analysis has been run and the flow values for the river sections have been computed through the CSV tabular command in the ISIS and the following graph has been drawn using Excel:
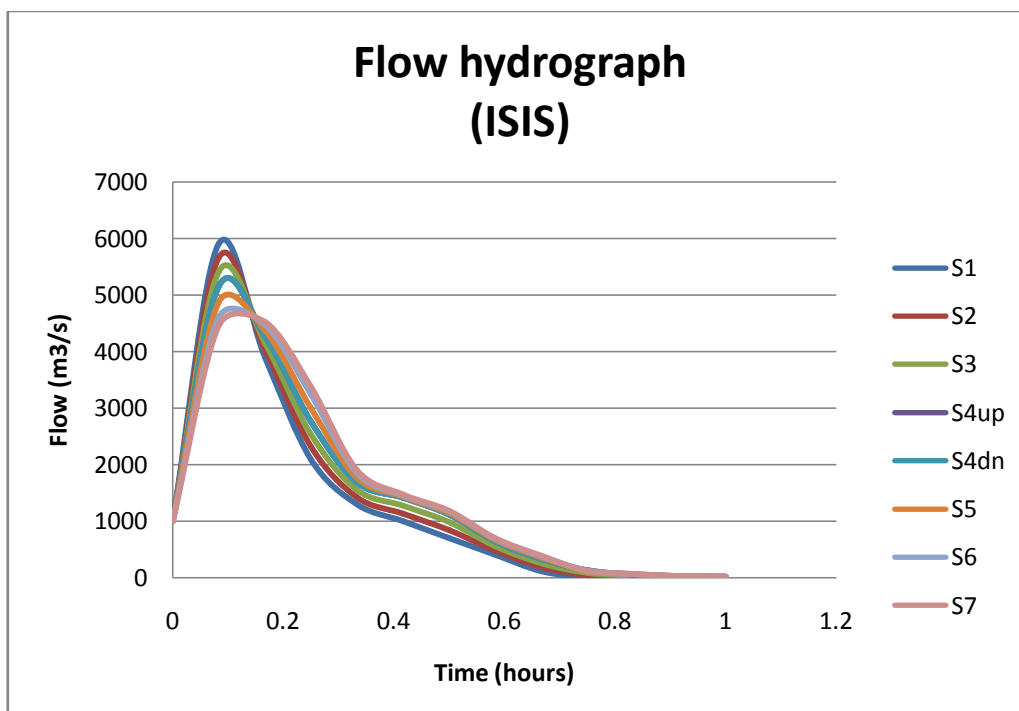
Figure 4.39 Flow hydrograph (ISIS)

As noticed from the previous flow hydrograph that the maximum flow point is around 6000 m^3/s and it has been reached after some time from the beginning of the simulation process then it decreased gradually until it reached zero by the end of the simulation time, it is very clear also that the highest flow hydrograph curve is at the first section then it decreased in the next following sections.

The following graph shows the comparison between the outflow hydrograph obtained by the journal and the one obtained from this analysis in one graph which shows that there is a slight difference in the maximum flow point which has been found in the journal to be around 7000 m^3/s and this might happen due to rounding decimal or human error.
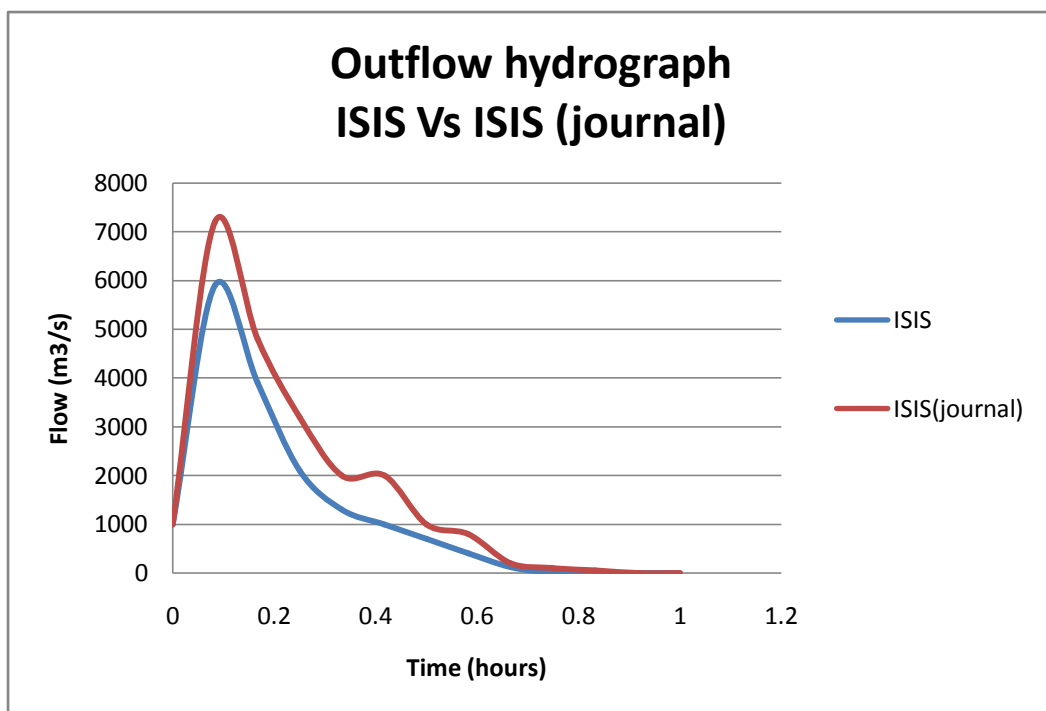
Figure 4.40 Outflow hydrograph comparison

The second graph shows the difference in the starting value of the flow which has been found around 5000 m^3/s and the other difference is the maximum flow point which has been found to be less than 7000 m^3/s with a little value:
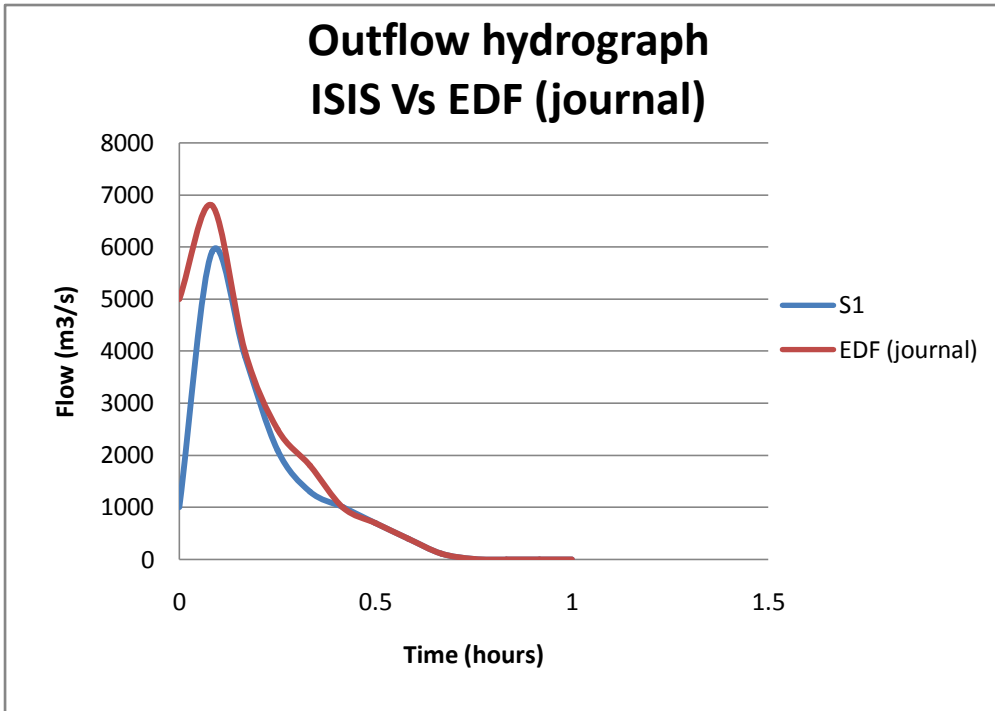
Figure 4.41 Outflow hydrograph ISIS Vs EDF

4.2.3 Test case 2 using HEC-RAS

The same procedure followed in the previous part has been applied here with some differenced related to the HEC-RAS; the major steps of the simulation process for this test case using the HEC-RAS software are listed below:

- The same gate information has been used to describe the Malpasset dam as a radial sluice gate with three openings, the following figures describe the shape of the weir, the information of the gate and the openings information:
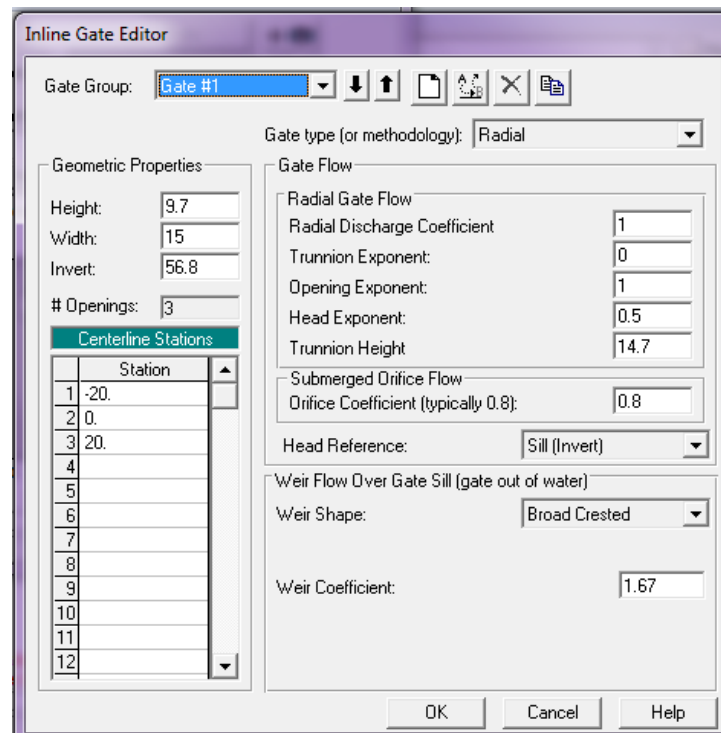
Figure 4.42 Gate information
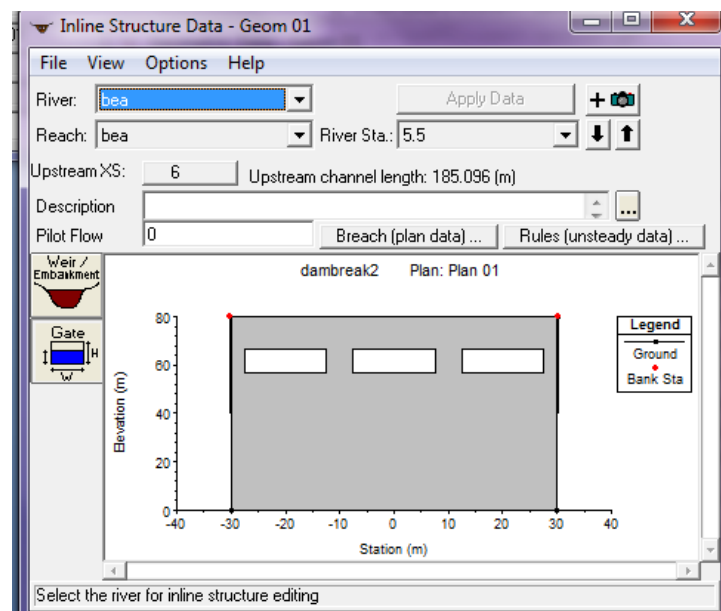
Figure 4.43 Weir embankment data

60

- The same number of river sections with the same information has been used also in this part to describe the river reach to be similar to the one used in the ISIS case and the one used in the journal:
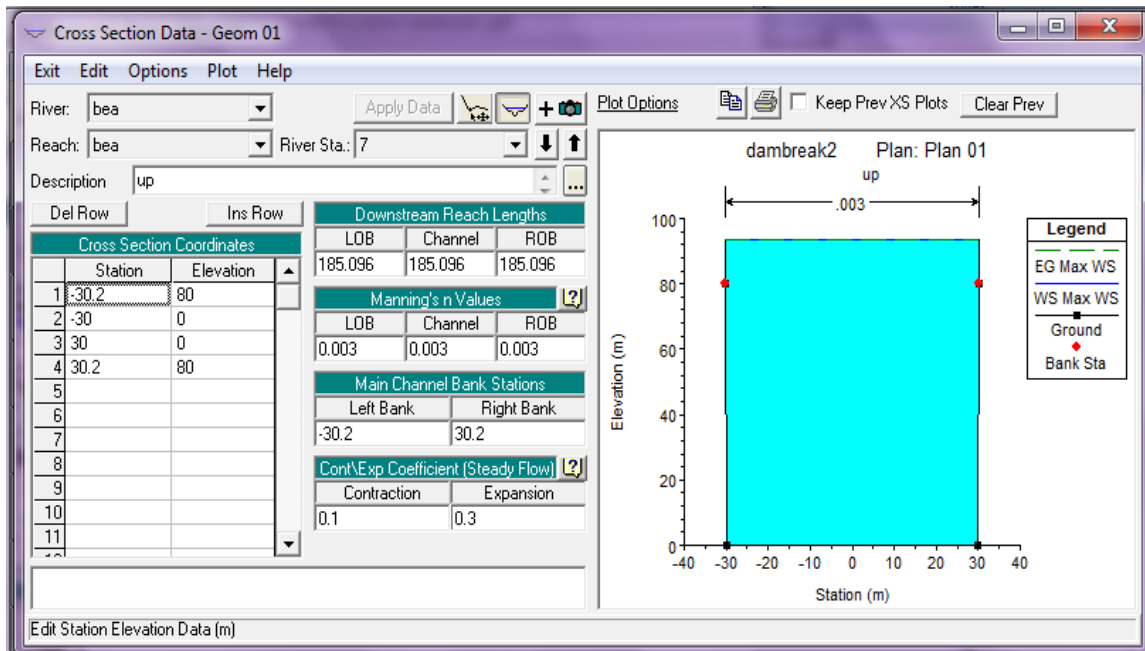
Figure 4.44 Cross section details

- The initial flow condition has been set to be 10 m^3/s , the upstream boundary condition has been defined as a flow hydrograph using the same details which used in the previous section, the downstream boundary condition has been defined as a stage hydrograph with the same data as well and the last boundary condition was the opening times of the sluice gate as shown in the following figure:
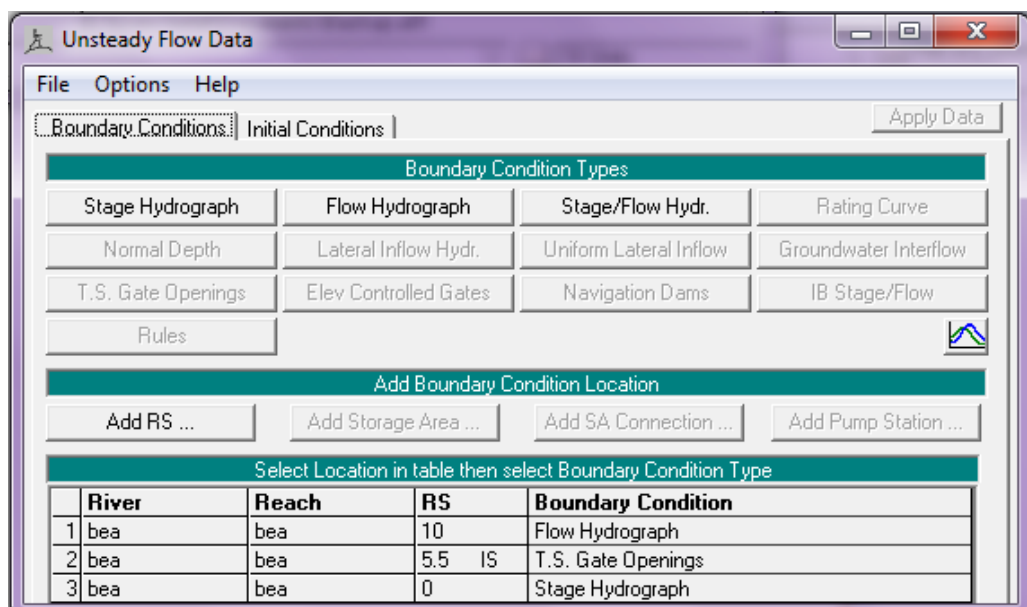
Figure 4.45 Unsteady flow boundary conditions

- The last step in this process was conducting the steady flow analysis followed by the unsteady one and then the flow hydrograph values can be obtained using the computation level output which give the spatial time series plot for each section separately, those flow hydrographs have been grouped altogether in one flow hydrograph as shown below:
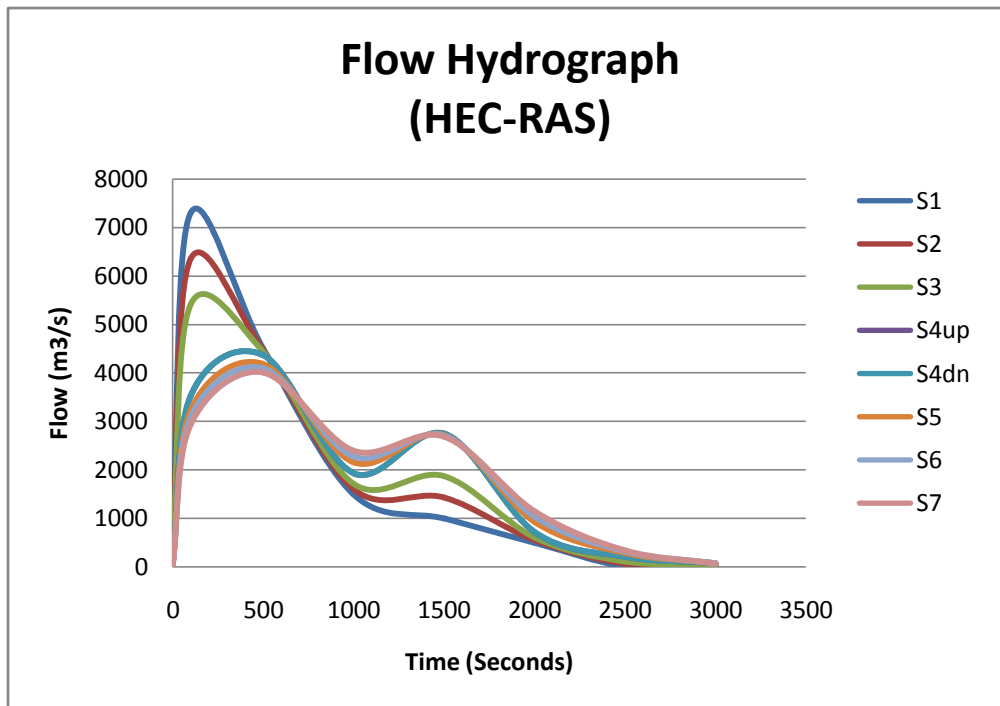
Figure 4.46 Flow hydrograph (HEC-RAS)

It has been noticed that the maximum flow value obtained by the HEC-RAS was around 7200 m^3/s and this value decreased in the following sections until it reached around 4000 m^3/s in the last section. The flow curve between 1000 seconds and 2000 seconds grew again until it reached a maximum local value which was $2800 m^3/s$ which describe some instability of the software at the last river sections.

4.2.4 Comparison between the results obtained from the ISIS, the HEC-RAS and the results obtained from the journal

The following outflow hydrograph includes the following curves:

1. The outflow hydrograph obtained from the ISIS.
2. The outflow hydrograph obtained from the HEC-RAS.
3. The outflow hydrograph obtained from the journal using the ISIS.
4. The outflow hydrograph obtained from the journal using the physical model (EDF).

It is obvious that the outflow hydrograph obtained from the HEC-RAS and the ISIS (journal) are coinciding on each other in most of the graph points and the dam failed when the discharge was 7200 m^3/s in both of them, as seen from the ISIS hydrograph that the dam failed when the discharge was 6000 m^3/s which is not the same of the ISIS (journal) as said before that the reason behind that could be the rounding decimal or a human error. The EDF hydrograph recorded 6800 m^3/s flow value when the dam failed which is still a quite closer to the value obtained from the HEC-RAS and the ISIS (journal).
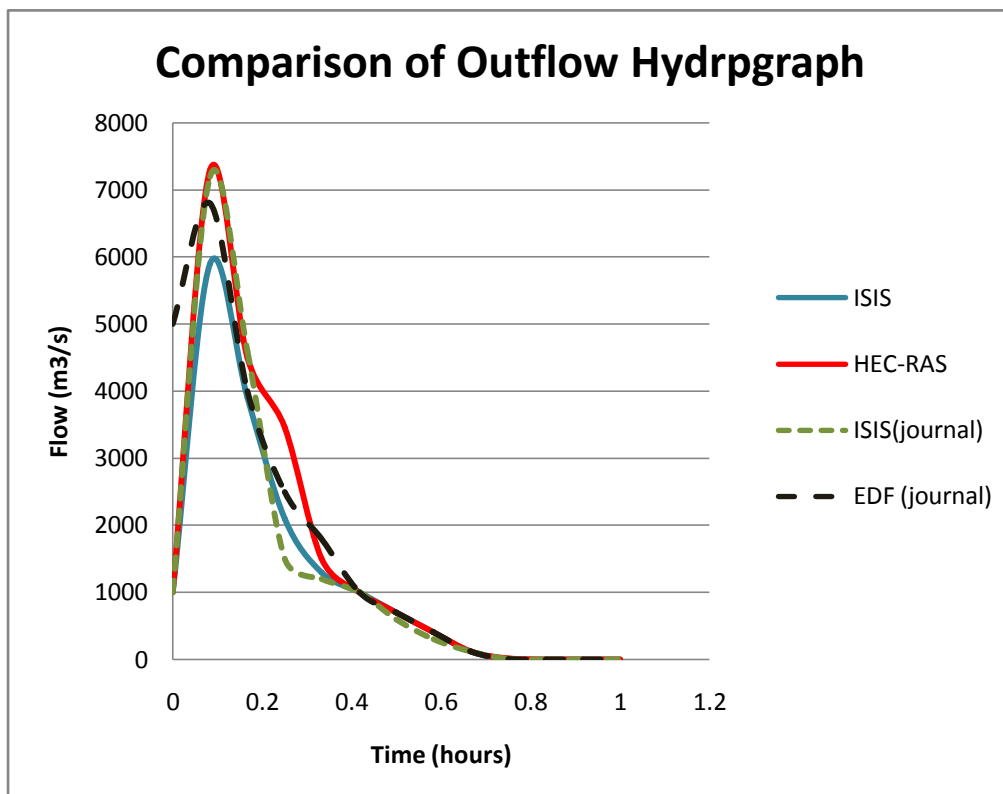
Figure 4.47 Comparison of outflow hydrograph

4.3 Case 3: Sinusoidal Oscillation

4.3.1 General description

The river discharge has been assumed to be constant value which is 2 m^3/s and the length between sections has been set to be 500 m, the constant discharge with the other data of the cross section have been used to create the tidal backwater effect in the ISIS and the HEC-RAS softwares, the tidal backwater effect will be simulated by changing the values of the downstream boundary condition periodically over the time, the results of the analysis for the river sections in this case will be in the wave oscillation form for all of the results components like the velocity, the stage, the flow and others (Loh, 2011). Other parameters have been assumed to be as in the following table:

Table 4. 4 Assumed parameters

Parameter	Value
Number of sections	8
Channel base width (meters)	5
Channel deep (meters)	5
Manning coefficient	0.0300
Slope for normal depth	0.0003
Simulation time (hrs)	48
Time step (secs)	60
Save interval (secs)	300

4.3.2 Test case 3 using ISIS

The previous data has been inserted into the ISIS as the same steps of the previous cases as shown below:

1. Define the upstream boundary condition by setting the flow discharge as $2\ m^3/s$ as shown in the following figure:
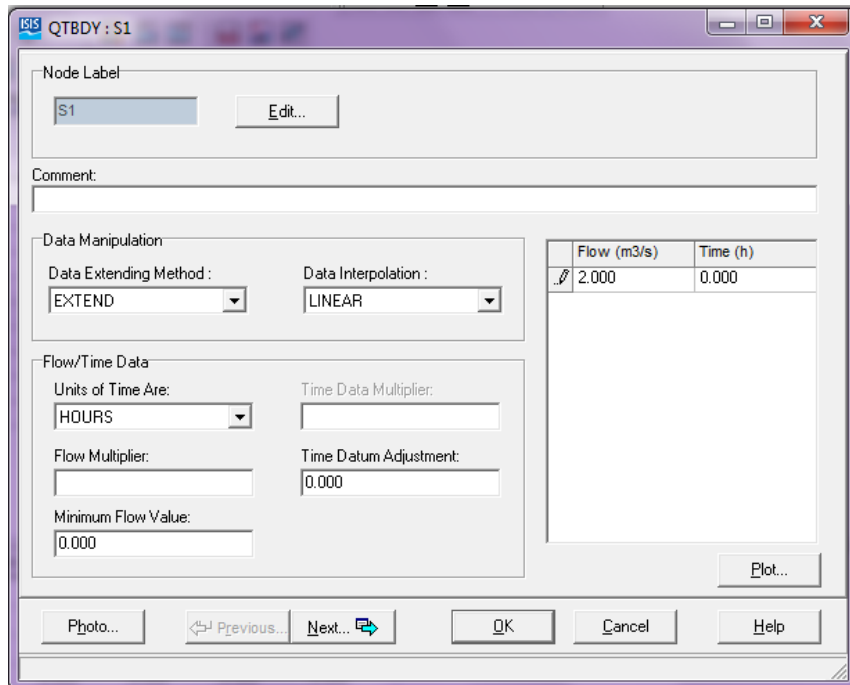
Figure 4.48 Upstream boundary condition

2. The 8 river cross sections details have been inserted to be as in the following:
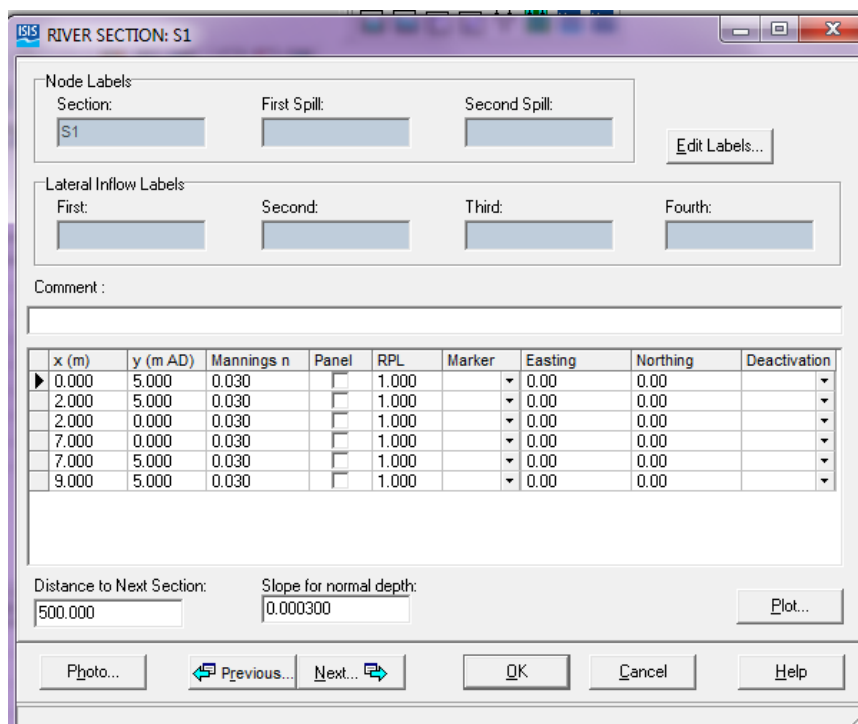
Figure 4.49 Cross section details

3. The details of the downstream boundary condition have been inserted to be oscillating over the time as shown in the following figure:
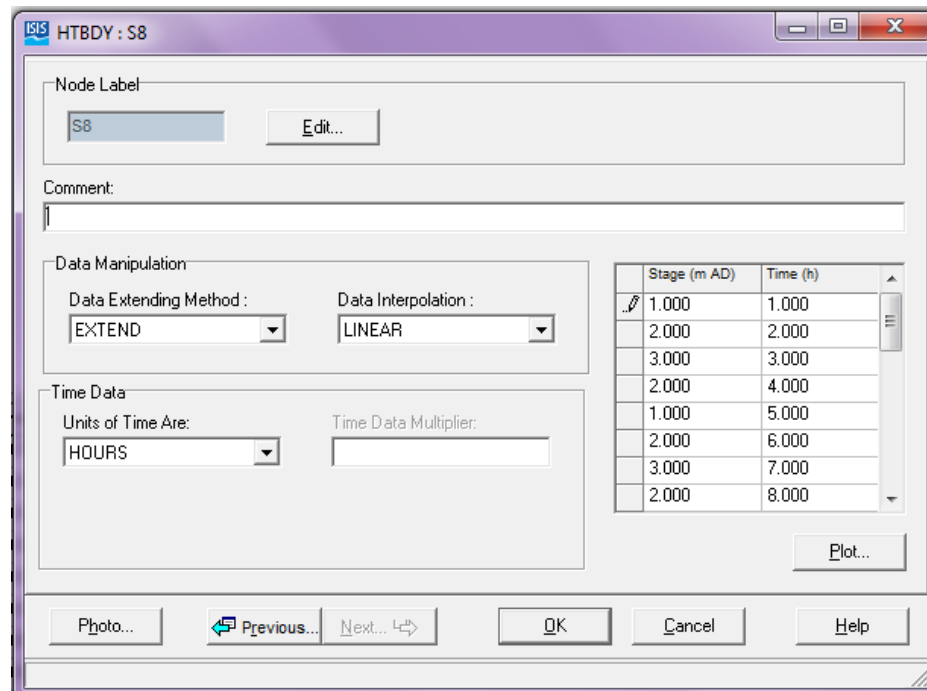
Figure 4.50 Downstream boundary condition

4. Conduct the steady state simulation first in order to allow the ISIS prepare the initial conditions, the steady state simulations results has been found to be as in the following figure:

```
                                    FILE=TEST3.dat              ISIS VER= 6.4.0.52
   results from the direct method at time      0.0000 hours
label12      ?      flow     stage    froude  velocity    umode     ustate         z
S1           y     2.000    1.435     0.074     0.279    0.000      0.000     0.000
S2           y     2.000    1.394     0.078     0.287    0.000      0.000     0.000
S3           y     2.000    1.348     0.082     0.297    0.000      0.000     0.000
S4           y     2.000    1.298     0.086     0.308    0.000      0.000     0.000
S5           y     2.000    1.242     0.092     0.322    0.000      0.000     0.000
S6           y     2.000    1.177     0.100     0.340    0.000      0.000     0.000
S7           y     2.000    1.099     0.111     0.364    0.000      0.000     0.000
S8           y     2.000    1.000     0.128     0.400    0.000      0.000     0.000
```

Figure 4.51 Steady state analysis results

5. Run the unsteady state (fixed timestep) simulation by setting the timestep to be 60 seconds, the save interval to be 300 seconds and the finish time to be 48 hours as shown in the following figure:
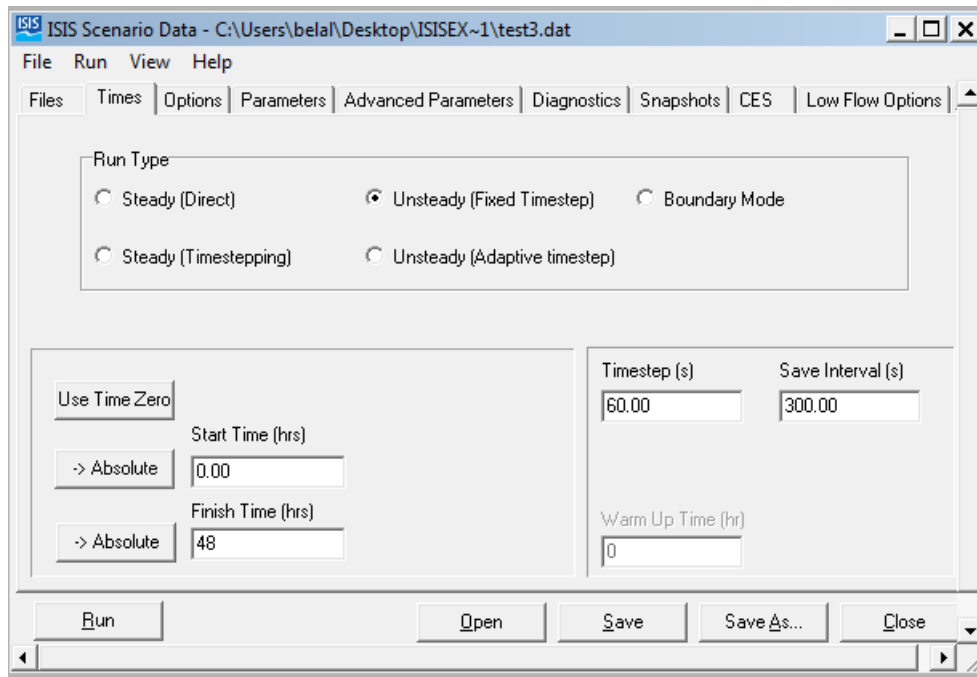
Figure 4.52 Unsteady state analysis

6. The long section of the river water surface has been found to be sinusoidal oscillated each different hour because of the continuos change of the river surface height each different hour as shown in the following figures:
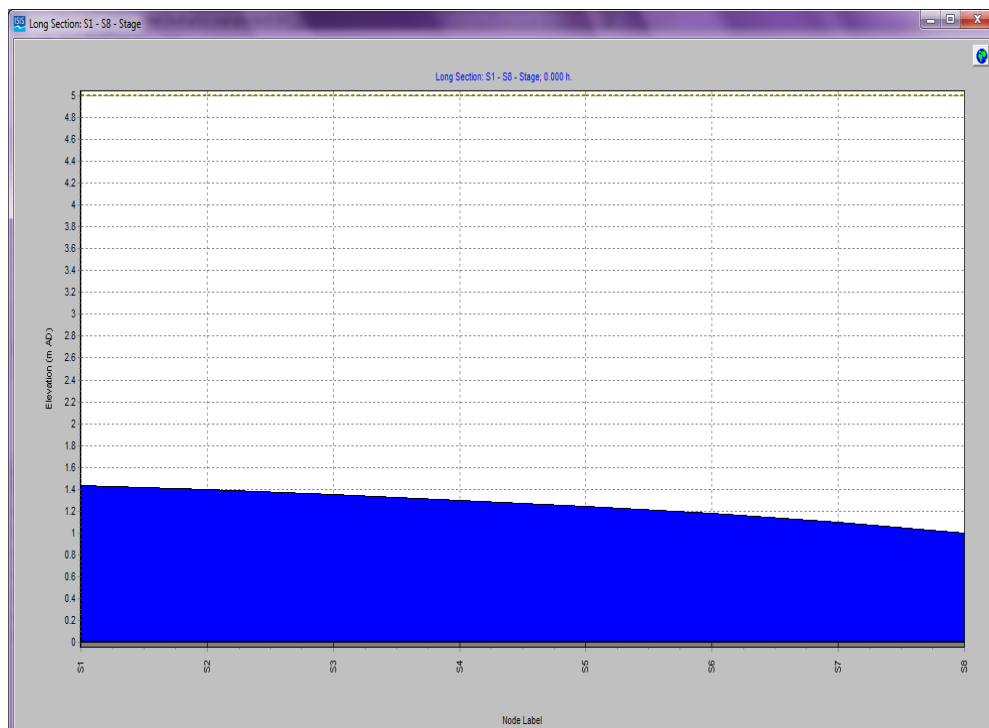
Figure 4.53 Longitudinal profile after 0 hours
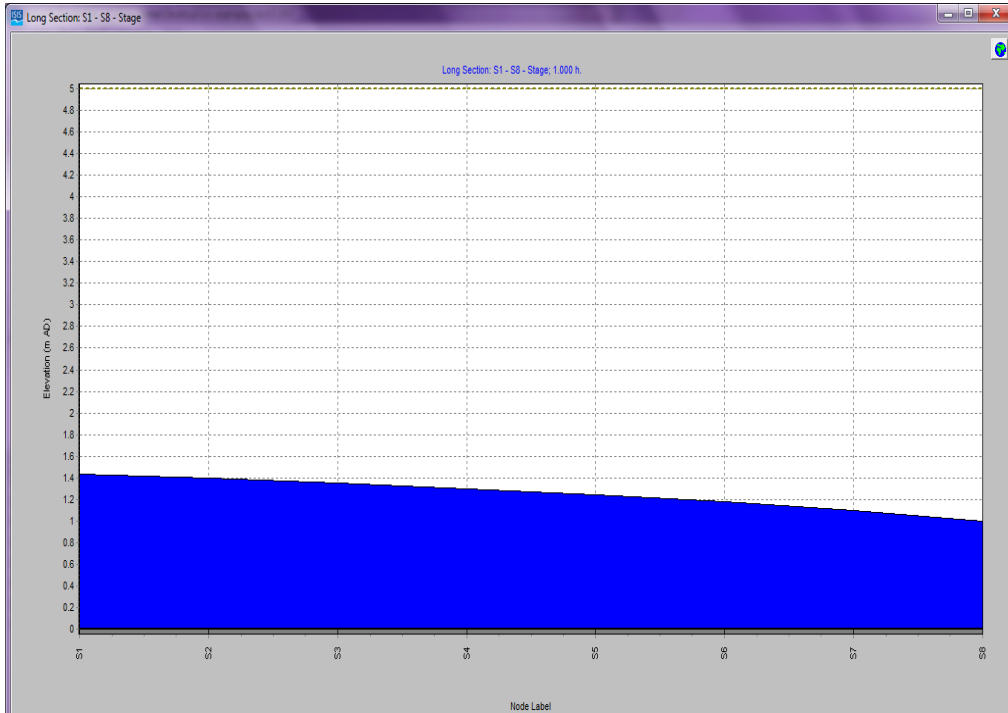
Figure 4.54 Longitudinal profile after 1 hours
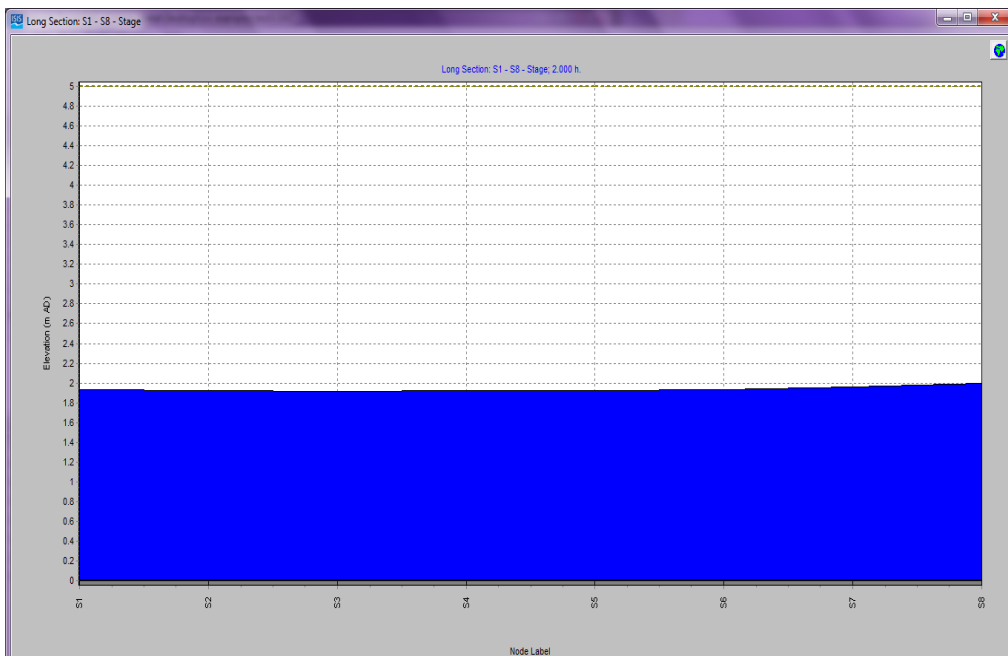
Figure 4.55 Longitudinal profile after 2 hours
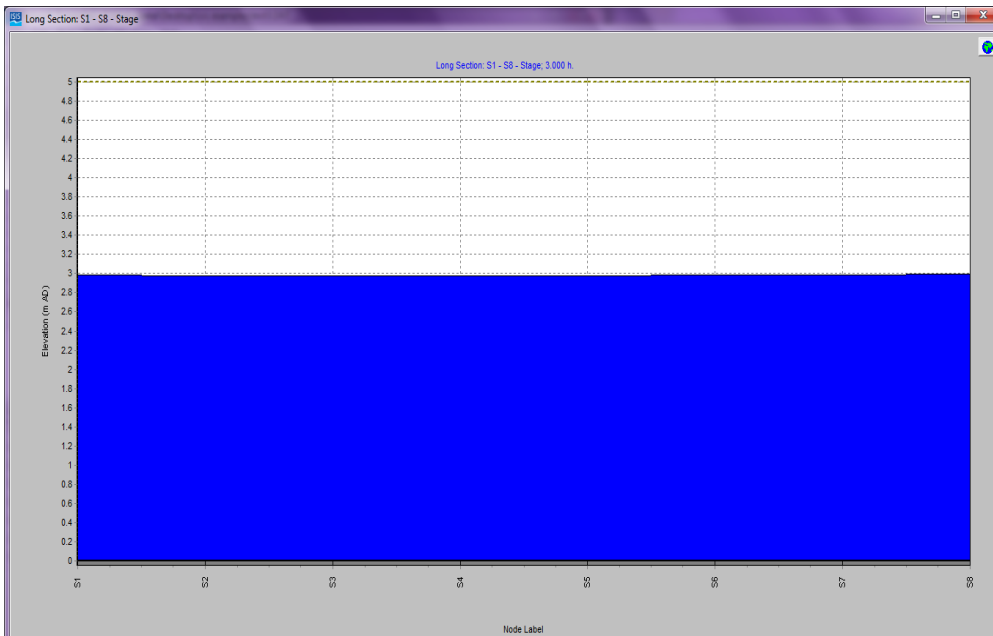
Figure 4.56 Longitudinal profile after 3 hours
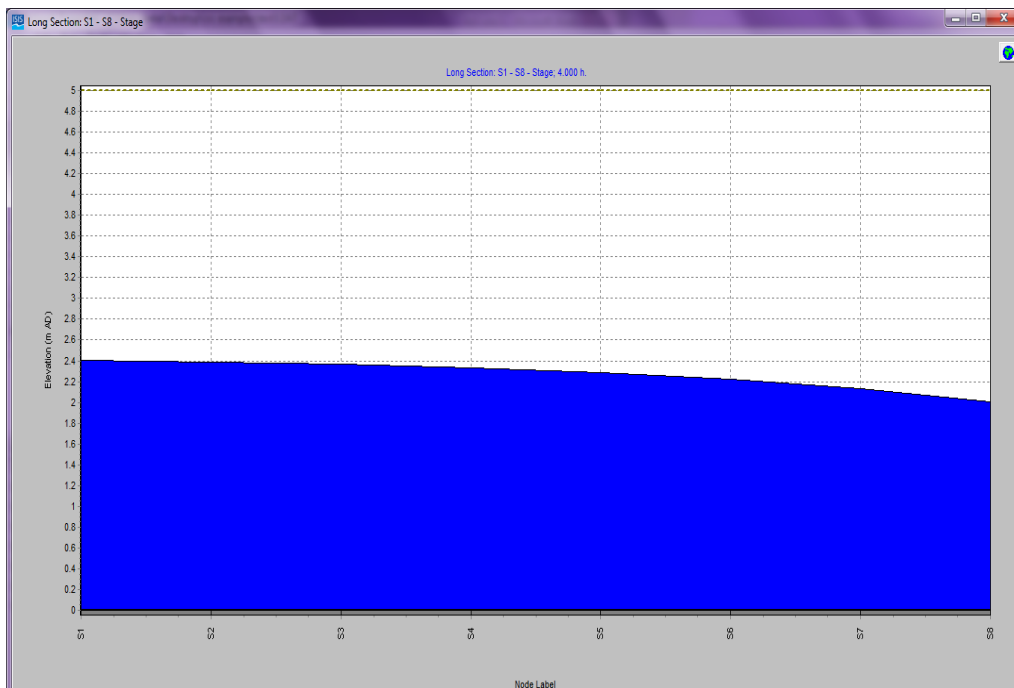
Figure 4.57 Longitudinal profile after 4 hours
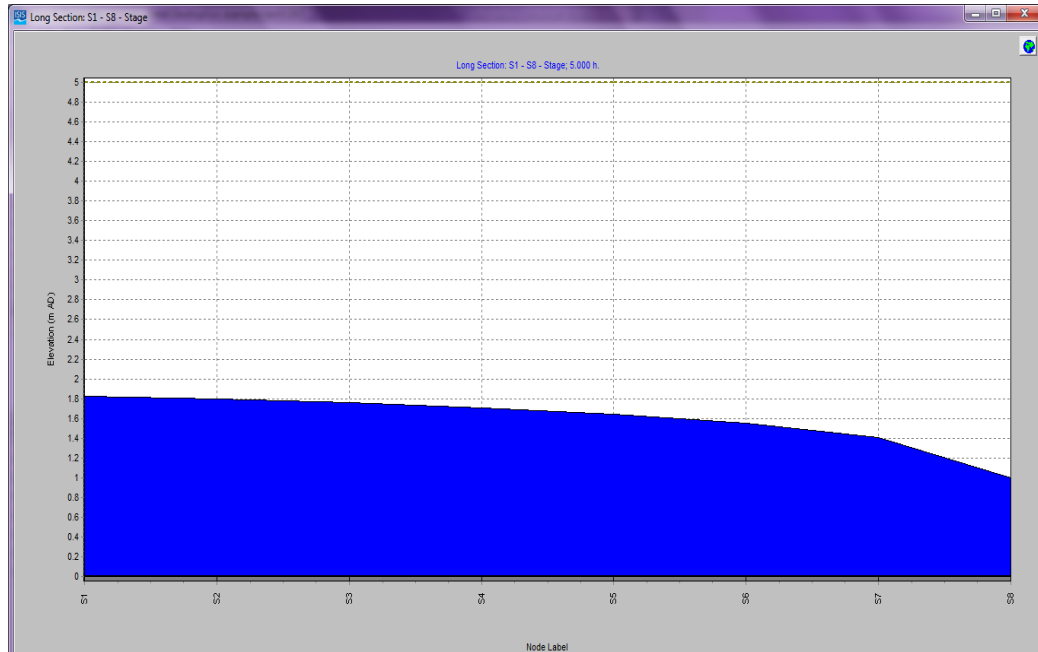
Figure 4.58 Longitudinal profile after 5 hours

7. The results of the unsteady state analysis have been taken through the CSV tabular command which gives all the variables including the velocity, the stage, the flow etc. So the stage and the velocity information for this case have been taken throughout this command the then plotted against the time in order to see the backwater tidal oscillations which have been found that those oscillations started from the last section S8 to the first one S1 from those following graphs it has been found that the more distance from the start of the river till the higher wave height it becomes for the velocity and the lower wave height it becomes for the stage and that proves the wave energy decreases as it travels due to the losses of the velocities and the friction forces from the channel bed and the river walls from the upstream boundary to the downstream boundary. The following figures show the wave oscillations for the stage and the velocity against the change of the time:
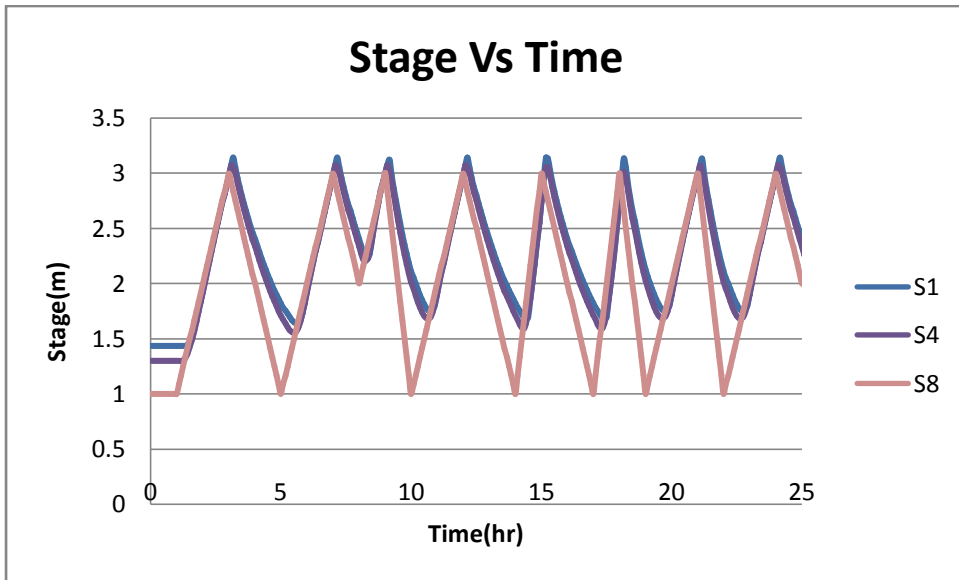
Figure 4.59 Stage hydrograph (ISIS)
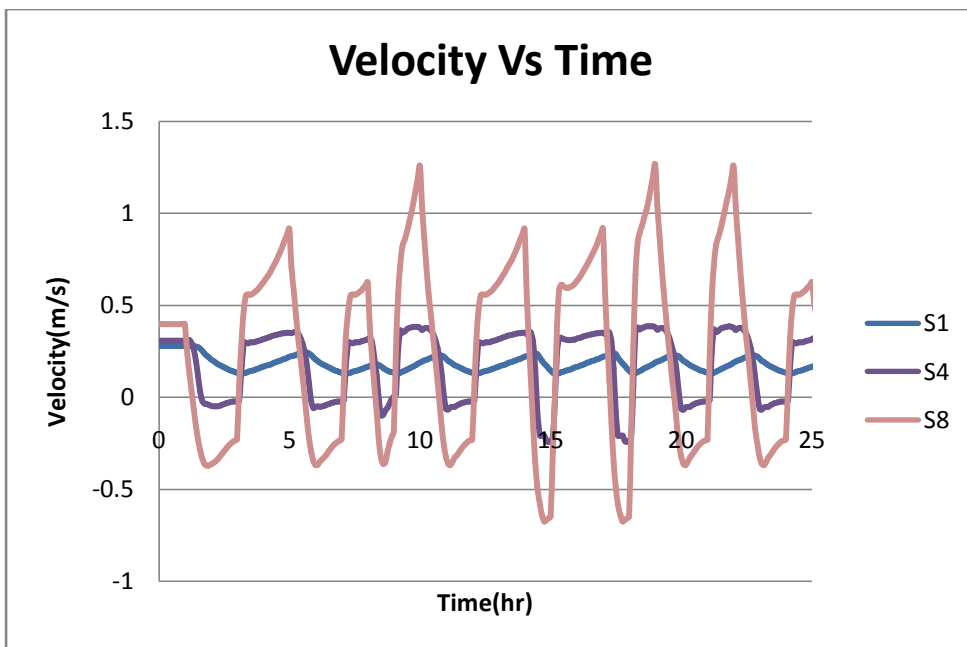
Figure 4.60 Velocity hydrograph (ISIS)

4.3.3 Test case 3 using HEC-RAS

- The river reach has been defined using 8 cross section starting from the river station 10 at the upstream boundary to the river station 0 at the downstream boundary as shown in the following figure:
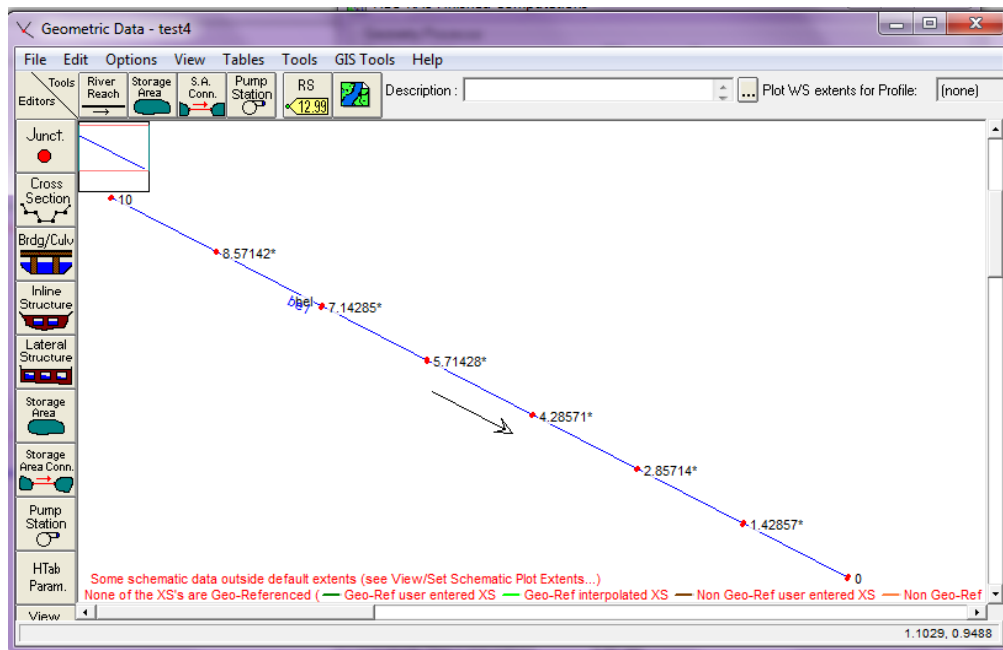
Figure 4.61 Reach river

- The cross section details have been assumed to be as the same information inserted in the ISIS for this test case where the distance between the cross sections is kept to be 500 meters and the manning coefficient to be 0.03 and any other details are shown in the following figure:
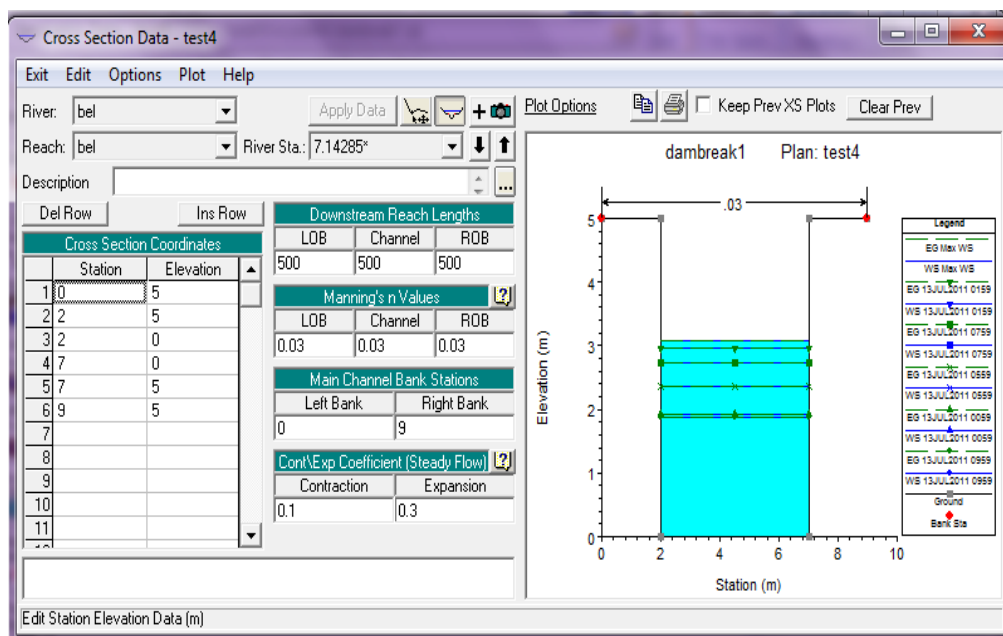
Figure 4.62 Cross section details

- In order to run the steady state simulation analysis a pre step has been done which is defining the steady flow data from: >edit >steady flow data, then the flow discharge set to be $2 \ m^3/s$ and the boundary conditions have been defined using the rating curve option for both the upstream and the downstream boundaries:
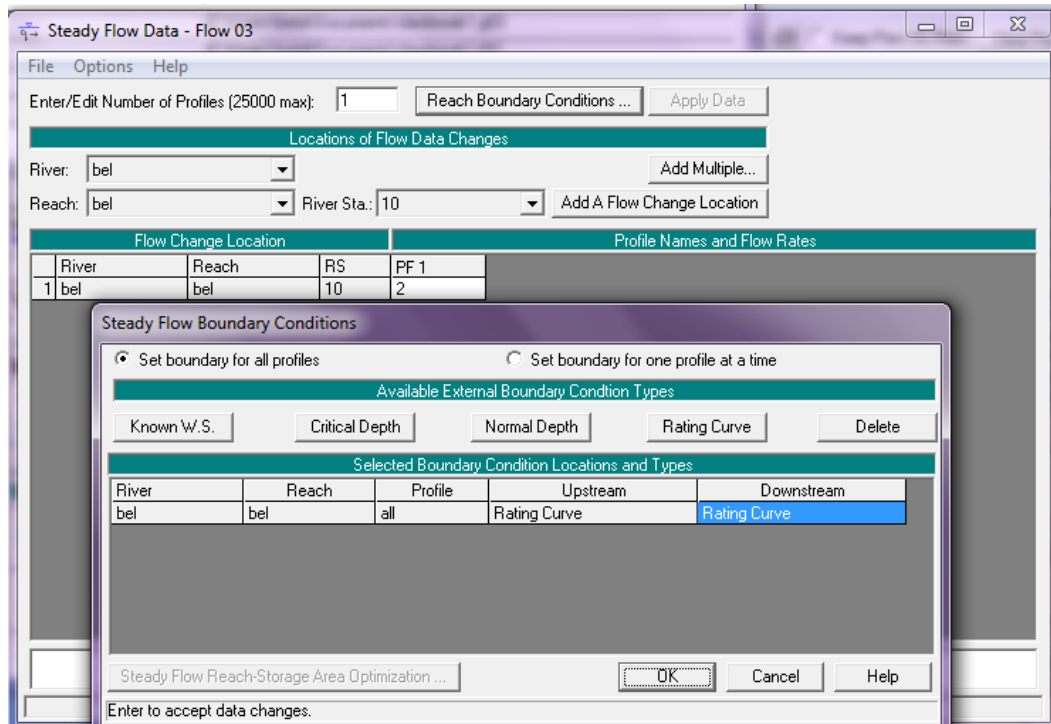
Figure 4.63 Steady flow boundary condition

The steady state analysis has been computed then the following results from the output summary tables for the steady state analysis have been extracted as shown below:

Figure 4.64 Steady state results

- The last step is to run the unsteady state analysis but before doing that step the unsteady state flow data needs to be defined using the same way of the steady state flow data but this time the only difference is how to show the sinusoidal case in the reach river using the same information used in the ISIS. The same information has been inserted into the downstream boundary condition as a stage hydrograph and for the upstream boundary condition as a flow hydrograph as shown below:
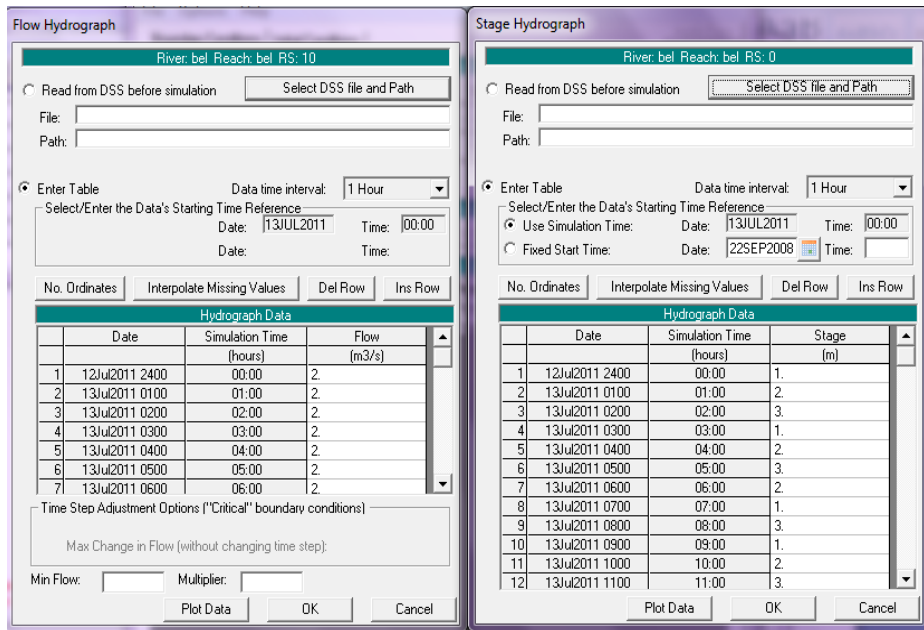

Figure 4.65 Unsteady state boundary conditions

- All the information needed from the unsteady analysis can be extracted through an important option in the following window which allows showing details about the stage and the flow at every section in the river. This option can be activated by tick the box in front of computation level output sentence as shown below:
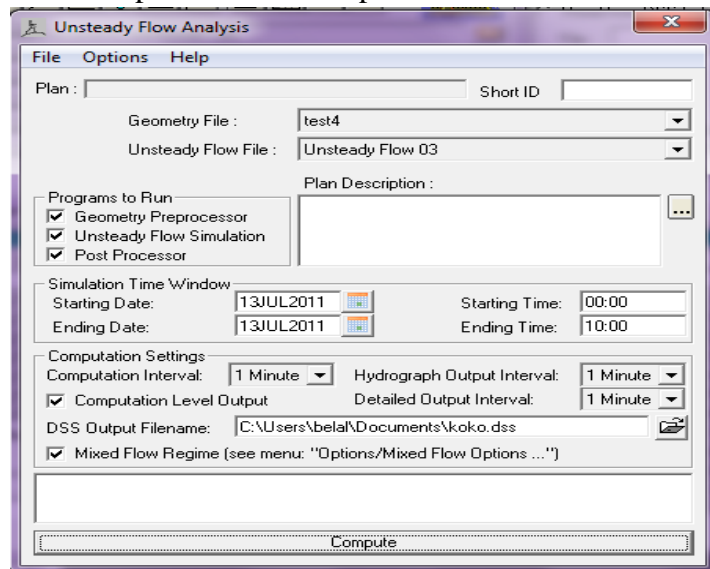

Figure 4.66 Unsteady flow analysis

74

As soon as the unsteady state analysis finished all the information needed to draw the graphs of the stage and the flow against time can be extracted from >view >unsteady state flow time series plot, so all the information needed has been extracted and moved to the Excel to draw the following graphs as shown below:
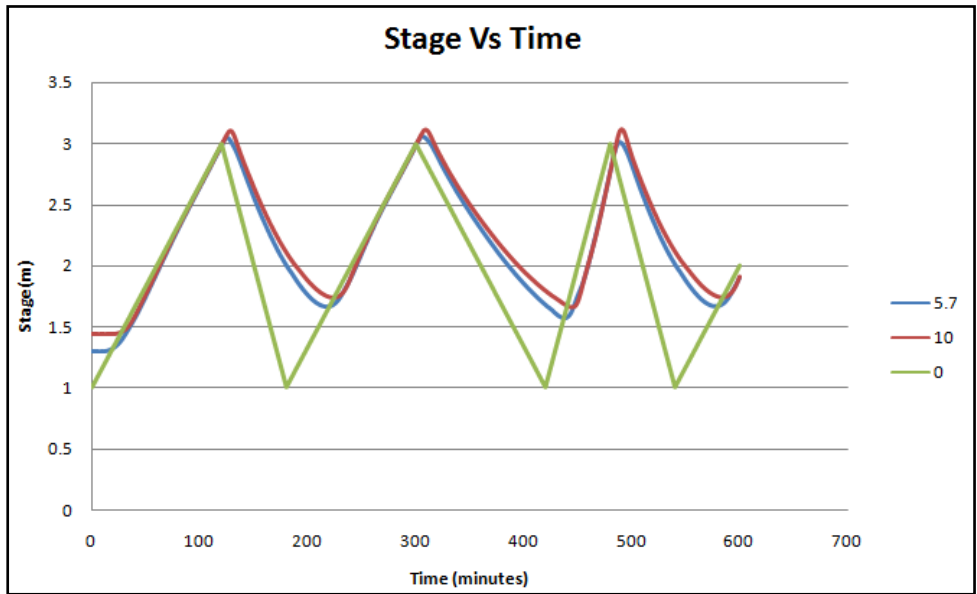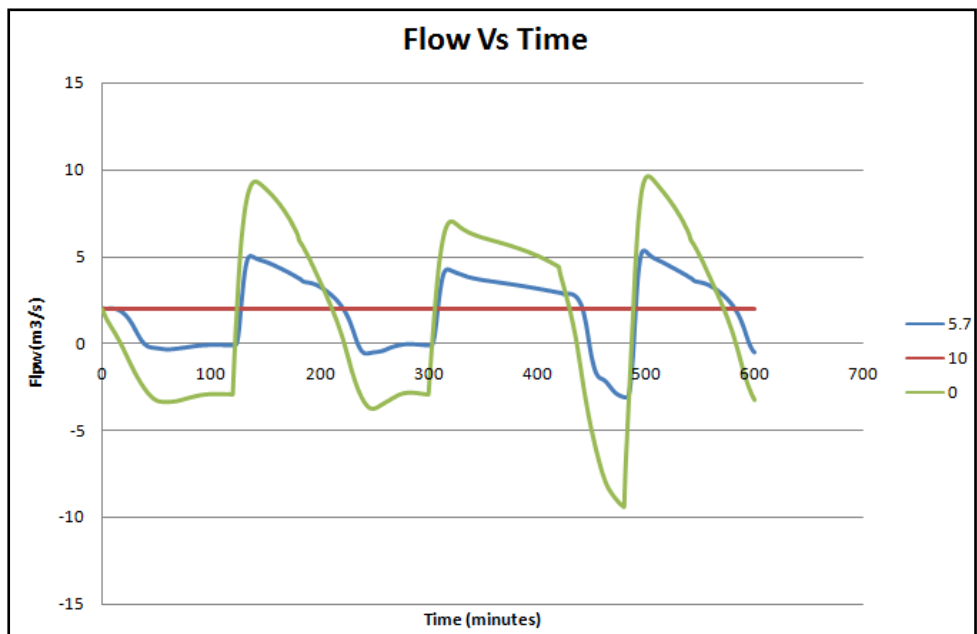

Figure 4.67 Stage hydrograph

Figure 4.68 Flow hydrograph

From the graphs above the backwater tidal oscillations are obviously exist in the stage and the flow graphs at different sections of the river and it has been noticed also the further from the beginning of the river reach the lower in the water surface and the more unstable in the flow of the water.

The following figure shows the water surface of the river at different times during the simulation process and it has been noticed that the largest level of water reached 3 meters after 2 hours from the beginning of the simulation process:
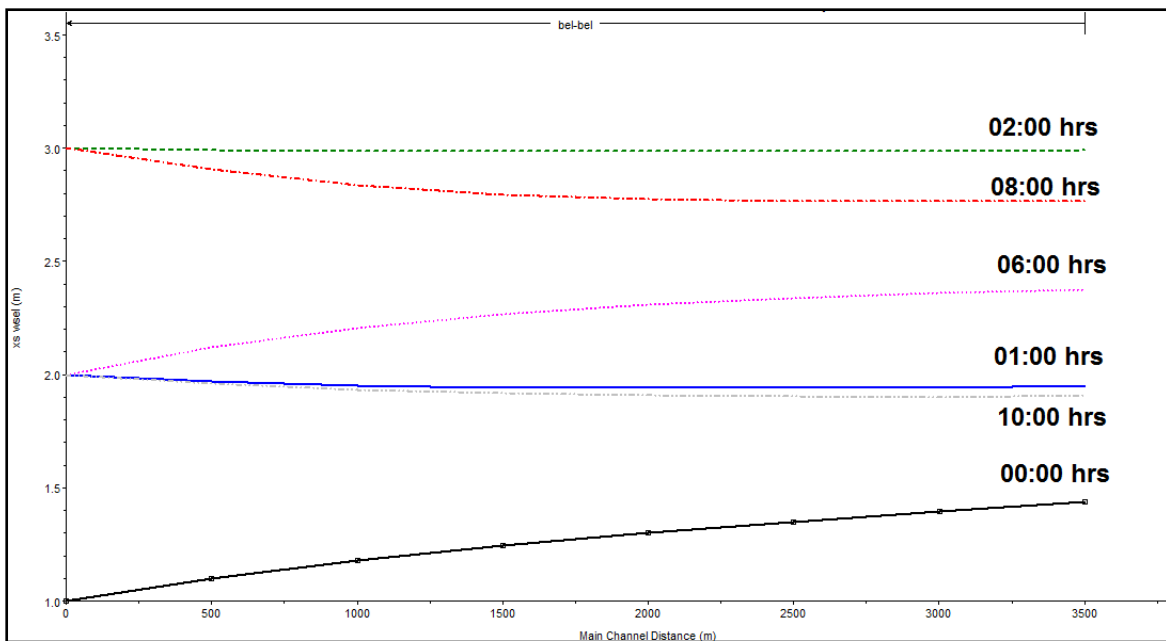
Figure 4.69 Longitudinal profile at different simulation times

4.3.4 Comparison between ISIS and HEC-RAS

The following graph shows the difference between the two softwares in the flow hydrograph where the ISIS has been found to be similar to the HEC-RAS at the first section of the river but at the other sections there is a slight difference between the two graphs as shown below, it is noticed that the HEC-RAS is quicker than the ISIS in reaching the maximum and minimum flow values:
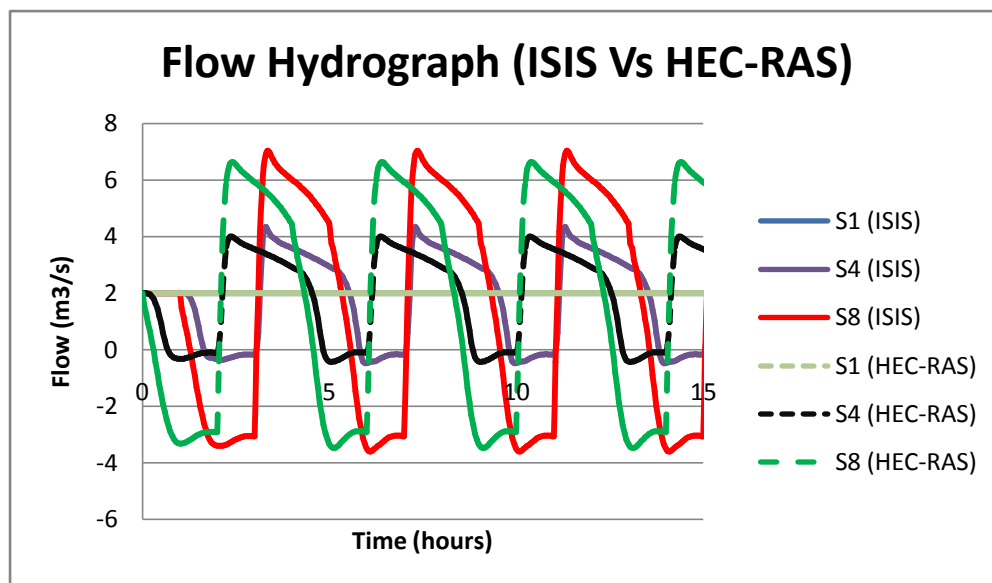

Figure 4.70 Outflow hydrograph comparison

The second graph shows the difference between the two softwares in the stage hydrograph where the ISIS has been found to the same as the HEC-RAS in the stage values with some difference in the timing between them:
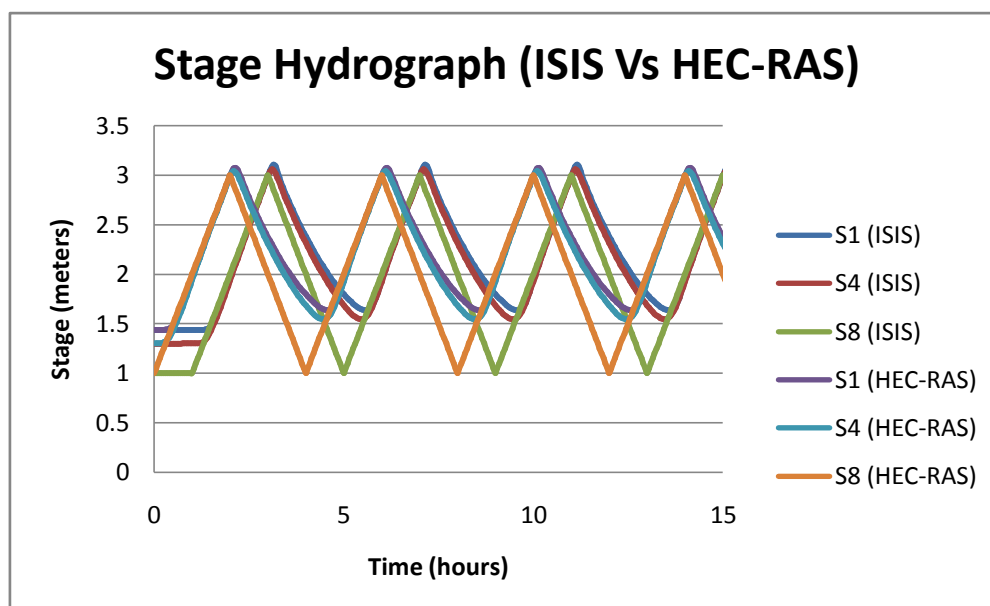

Figure 4.71 Stage hydrograph comparison

4.4 Case4: Dry River with constant flow discharge

4.4.1 General description

The main purpose of this test case is to simulate the dry river bed with no water flow then simulate a constant flow discharge through the river until it reaches the steady state flow, the height of the water in the river should be determined and plotted into a graph. This purpose can be simulated by making some assumptions into the initial condition of the river channel to be dry with no flow of water the river bed starts to change from the dry form to the wet form by increasing the time of the simulation until it reached the steady state case. The following assumptions have been made to be used in the case:

Table 4. 5 Assumed parameters

Parameter	value
Base flow (m3/s)	10
Distance between cross sections (m)	50
Cross section (m*m)	5*5
Slope of the channel	0.0001
Simulation time (hrs)	48
Time step (secs)	20
Save interval (secs)	300
Manning coefficient	0.03

4.4.2 Test case 4 using ISIS

- The first step to simulate the dry river bed is to define the constant flow discharge of the river in the upstream boundary condition to be $10 \, m^3/s$ after one small value which has been assumed to be $0.001 \, m^3/s$, that small value has been assumed instead of zero flow value as the ISIS can't run a simulation with zero flow value, the following figure shows the upstream boundary condition details:
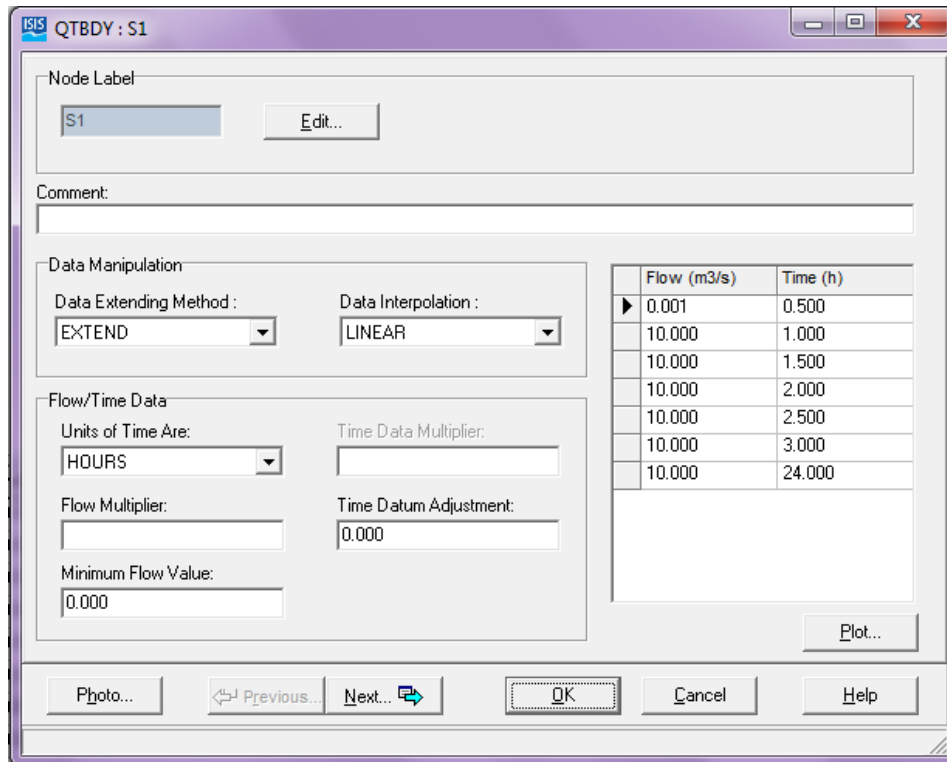
Figure 4.72 Upstream boundary condition

- The second step is to define 8 cross sections of the river (S1 to S8) with the details shown in the following figure:
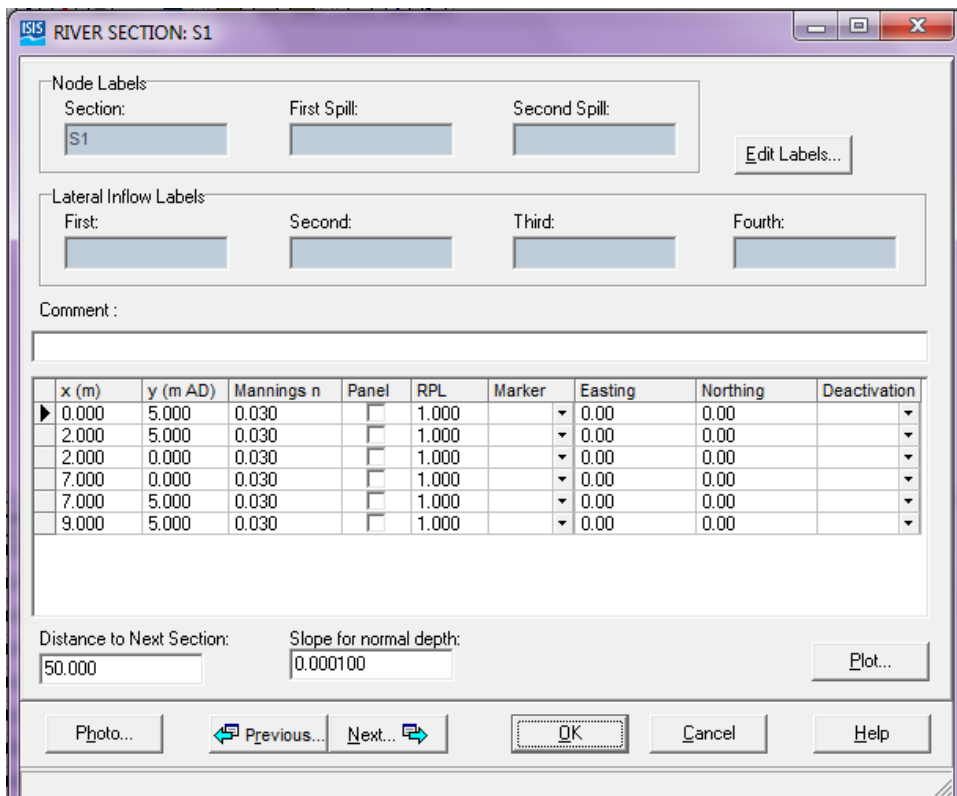

Figure 4.73 Cross section details

- Finally and before running the simulation analysis, the downstream boundary condition should be defined as a constant stage 1 meter over the change of the simulation time as shown in the figure:
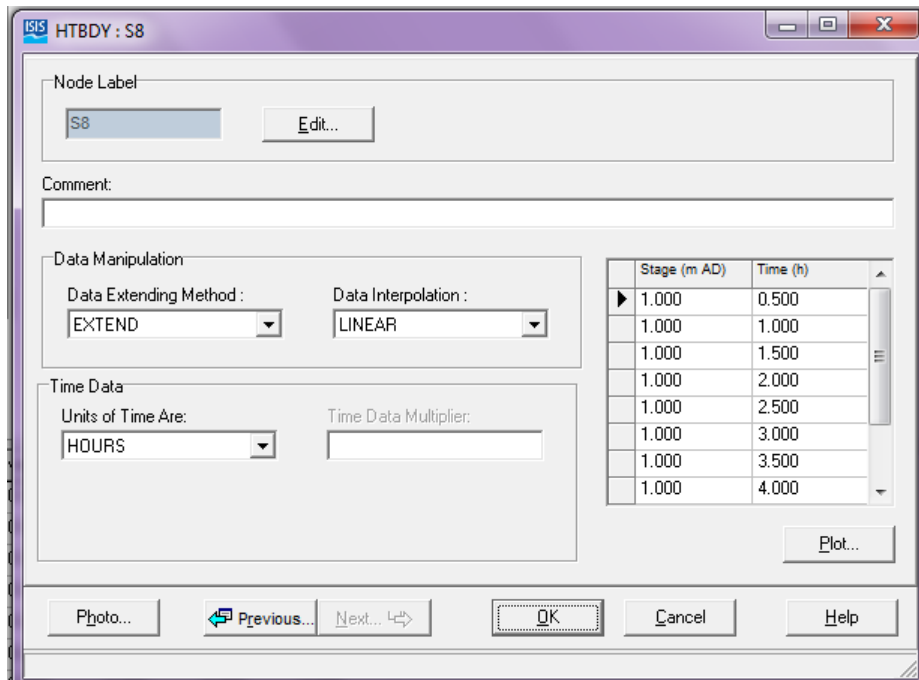

Figure 4.74 Downstream boundary condition

- The steady state simulation has been run, the results of the steady state analysis at zero hour time show the dry river bed simulation where the values of the velocity, the Froude number and the flow values are equal zero as shown in the figure:

Figure 4.75 Steady flow results after 0 hours

The results of the steady state simulation analysis after one hour of the simulation show the change of the river bed from the dry case to the wet case as the velocity values start to increase, the Froude number and the stage as well, the following figure shows the change from the dry to the wet:

```
|                                      FILE=TEST3.dat
ISIS VER= 6.4.0.52
 results from the direct method at time     1.0000 hours
label12        ?      flow      stage    froude   velocity     umode
ustate         z
S1             y     10.000     1.804    0.264     1.109       0.000
0.000      0.000
S2             y     10.000     1.745    0.277     1.146       0.000
0.000      0.000
S3             y     10.000     1.680    0.293     1.191       0.000
0.000      0.000
S4             y     10.000     1.605    0.314     1.246       0.000
0.000      0.000
S5             y     10.000     1.518    0.341     1.317       0.000
0.000      0.000
S6             y     10.000     1.411    0.381     1.417       0.000
0.000      0.000
S7             y     10.000     1.267    0.448     1.579       0.000
0.000      0.000
S8             y     10.000     1.000    0.639     2.000       0.000
0.000      0.000
```

Figure 4.76 Steady flow results after 1 hours

- In the last step of simulating the dry river bed is to run the unsteady state flow simulation fixed time step where the time step has been set to be 20 seconds and the simulation time assumed to be 12 hours:
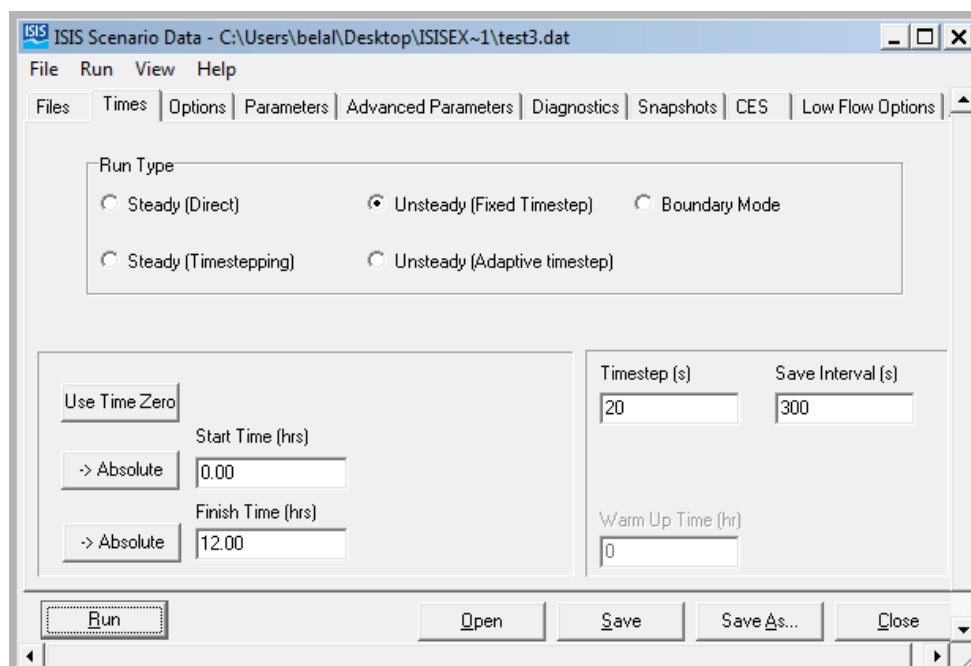

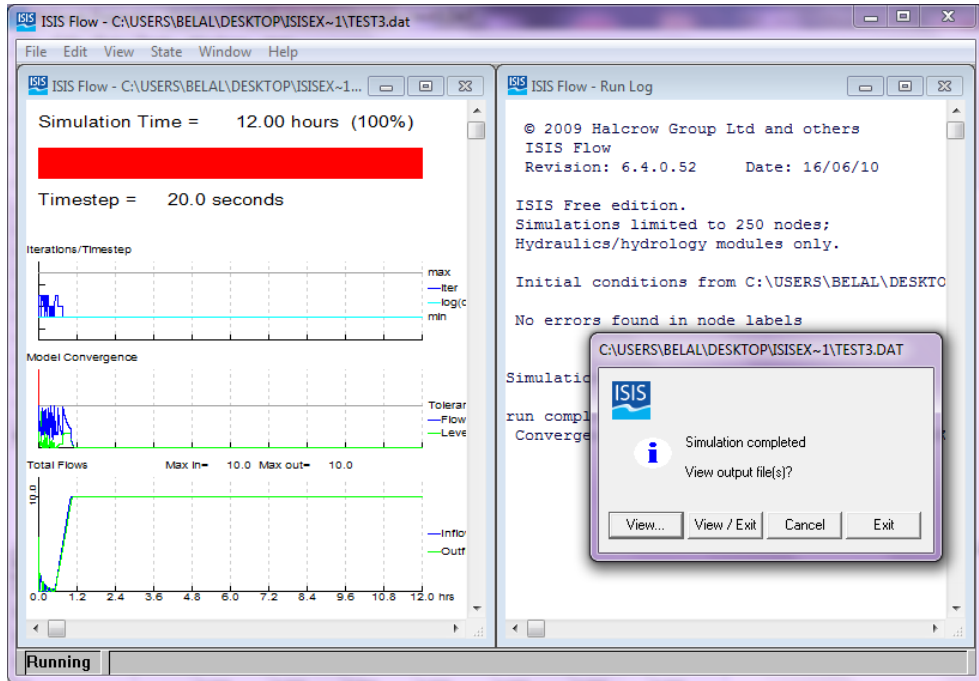**Figure 4.77 Unsteady flow analysis**

81

Figure 4.78 Unsteady state flow analysis completed

The following figures show the maximum surface of the water in the river which reaches 1.813 meters in section 1 and from the longitudinal river section it is obvious the increase of the height of the water by the time from section 8 to section 1 (from left to right):
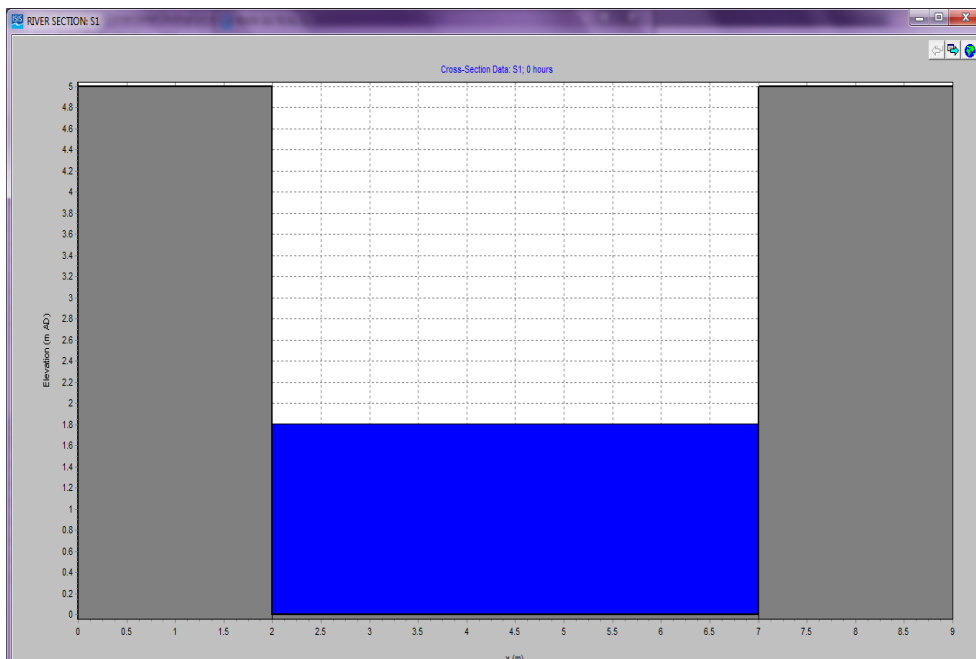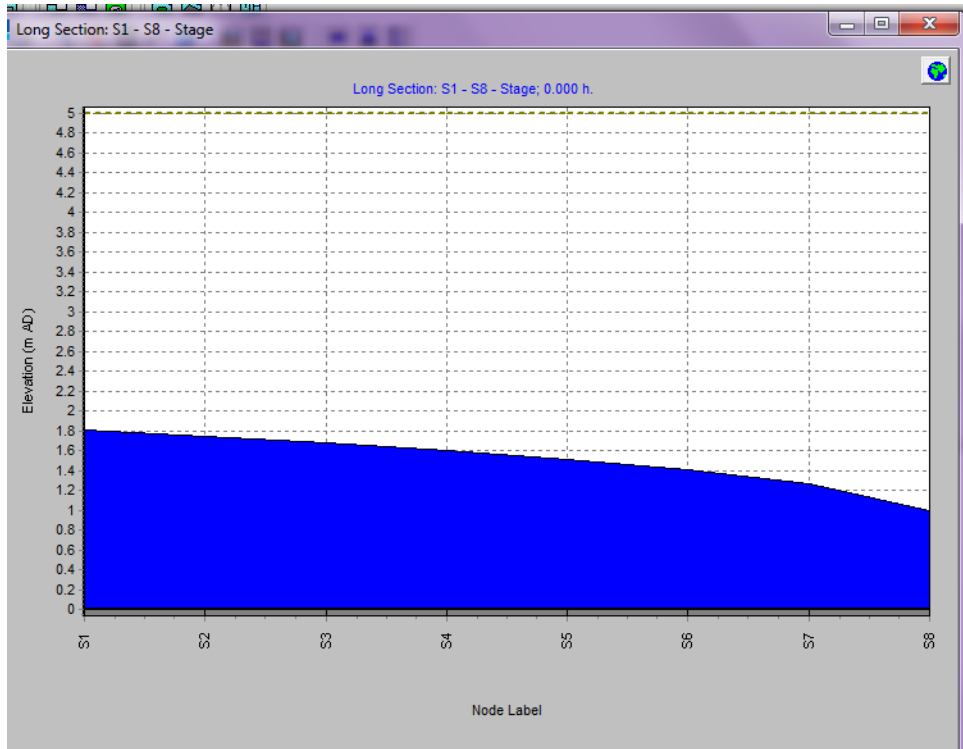

Figure 4.79 Cross section plot

Figure 4.80 Longitudinal profile after 0 hours

The values of the stage against time have been run through the tabular CSV command and then plotted to be as shown in the following figure which indicates the maximum height of the river at 1.8 meters and the more distance from the first section the lower in the stage against time:
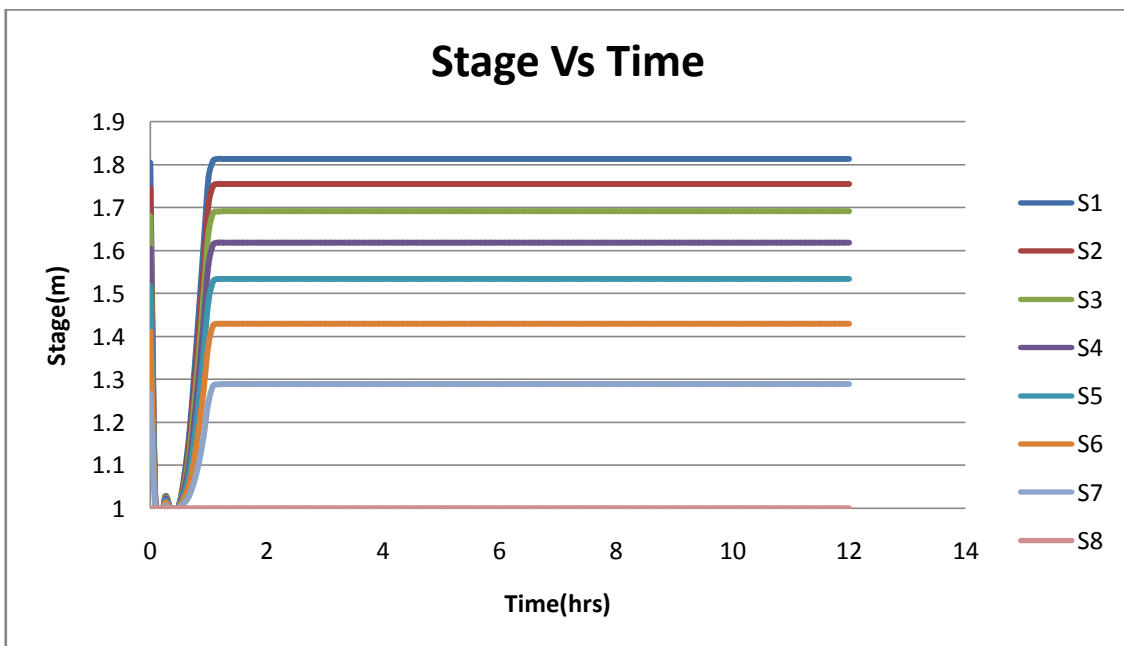

Figure 4.81 Stage hydrograph (ISIS)

The last figure shows the flow values against the time values which indicate the dry river bed at the beginning of the simulation time and then the increase of the flow and the height of the water surface until it reach the constant value for all of the river cross sections.
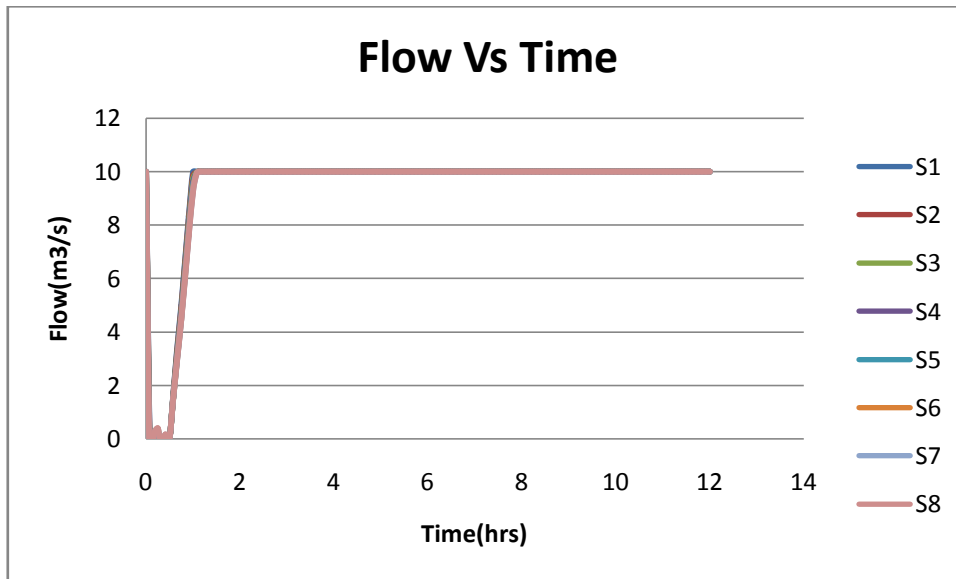

Figure 4.82 Flow hydrograph (ISIS)

4.4.3 Test case 4 using HEC-RAS

- The first steps are exactly the same of the previous case and the only difference in the boundaries of the reach river and they will affect the result only in the unsteady state analysis, so before running the steady state analysis, the boundaries of the reach river have been assumed using the rating curves then the value of the flow discharge has been assumed to be 0.001 then to be increased suddenly on the next hour of simulation to be 10 and this will reflect the change from dry case to wet case.

- In order to simulate the dry bed of a river through the HEC-RAS, the flow discharge of the water should be assumed to be very small value at the beginning of the simulation time then the change of the state from dry to wet will be through increasing the value of the discharge in the river over the time, the next graph show the dry bed condition in the steady state analysis where the values of the velocity and Froude number equals zero:

Figure 4.83 Steady flow results after 0 hours

The change from the dry to the wet condition is illustrated in the following figure where the values of the velocities started to increase over zero and the Froude numbers as well:

Figure 4.84 Steady flow analysis after 1 hours

- The maximum level of water has been found to be 3.5 meters as shown in the following figure where the level of water started to increase from 1 meter and stopped at 3.5 meters at the end of simulation time :
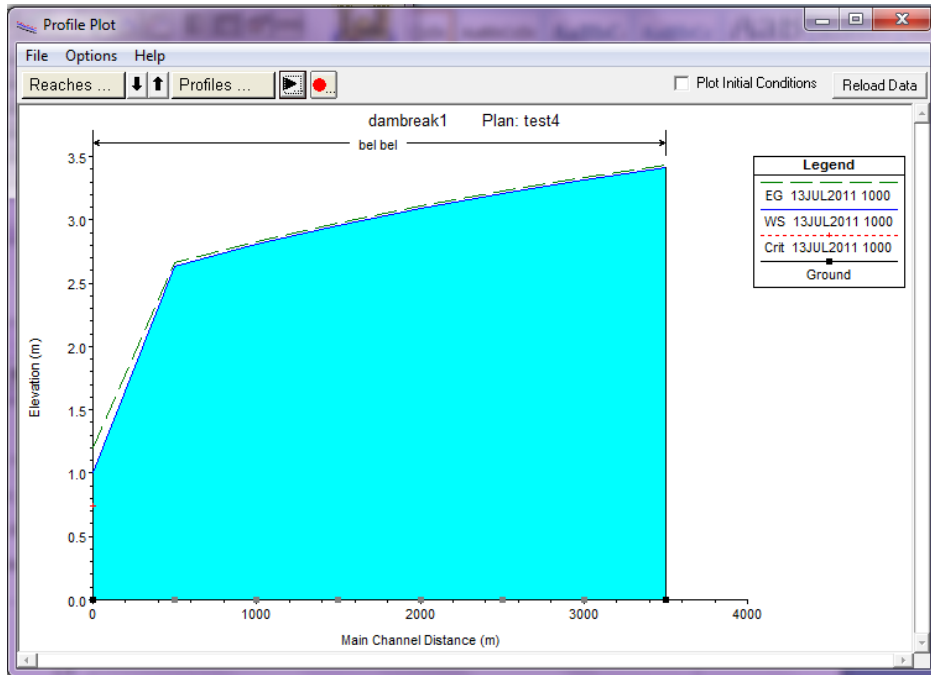
Figure 4.85 Profile plot

- The stage and flow hydrographs can be plotted either using the HEC-RAS or the Excel, the following figure shows the flow and the stage plots for the 10 station :
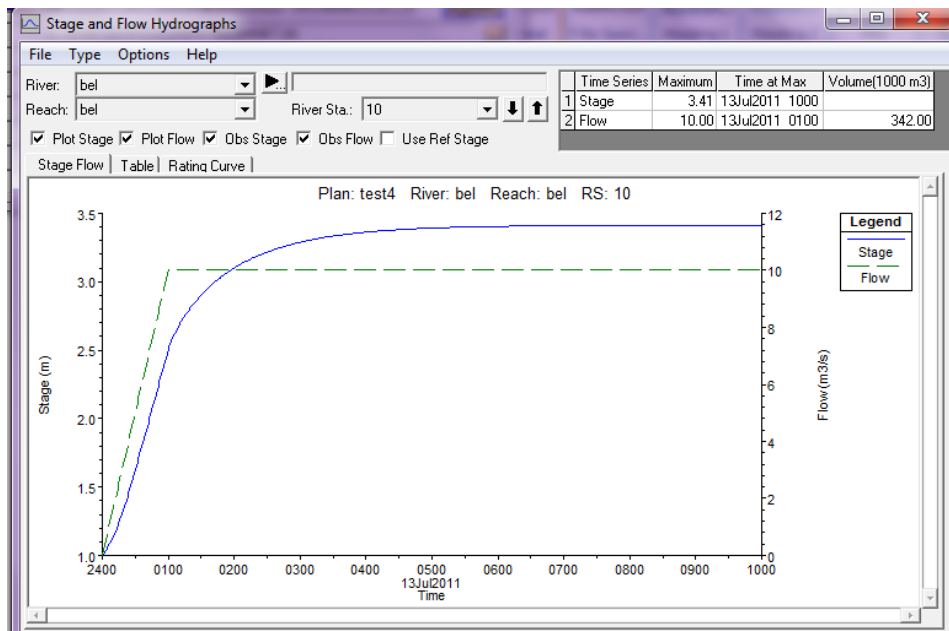
Figure 4.86 Stage Vs Flow (HEC-RAS)

- Finally the values of the stage and the flow can be extracted from the tables of the unsteady state spatial time series then those values can be transferred into Excel to plot the stage and the flow hydrograph for each river section altogether in one plot as shown in the following plots:
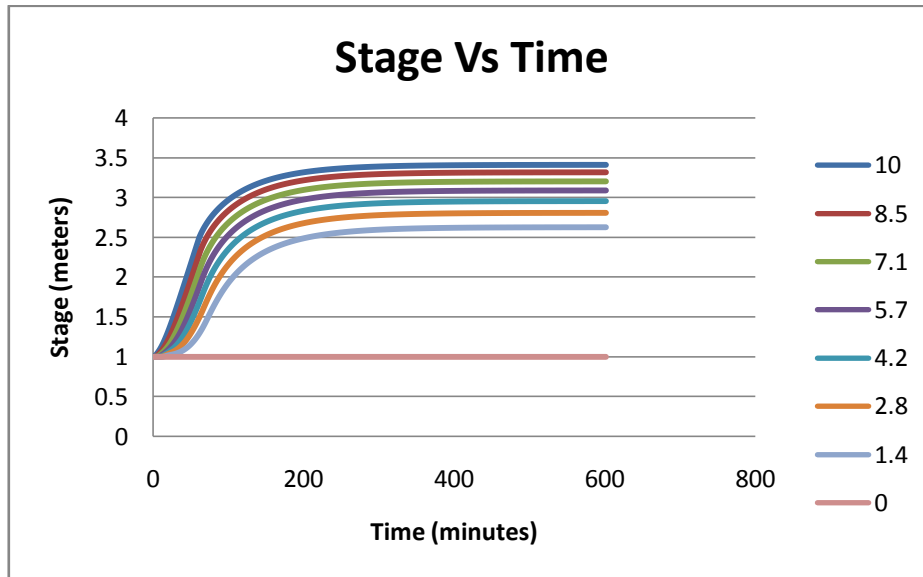
Figure 4.87 Stage hydrograph (HEC-RAS)

The previous plot show the decrease in the water level by moving away from the upstream boundary of the river until it reach the downstream boundary where the value of the stage is constant and equals 1 meter, the next graph shows the change of flow over time for all river sections and it is obvious that the flow at the beginning of the river (upstream boundary) is higher than the flow at the end of the river (downstream boundary):
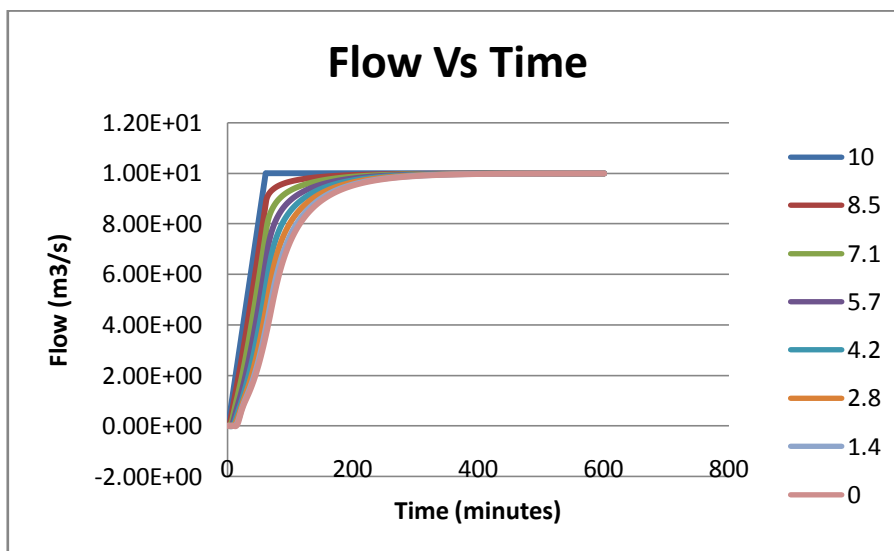
Figure 4.88 Flow hydrograph (HEC-RAS)

87

4.4.4 Comparison between ISIS and HEC-RAS

There is no big difference in simulating the dry bed river between the two softwares and the only difference is in the beginning of the simulation time where the hydrograph in ISIS takes some time to start increasing but in the HEC-RAS it started to increase at (0,0) point as shown in the following figures:
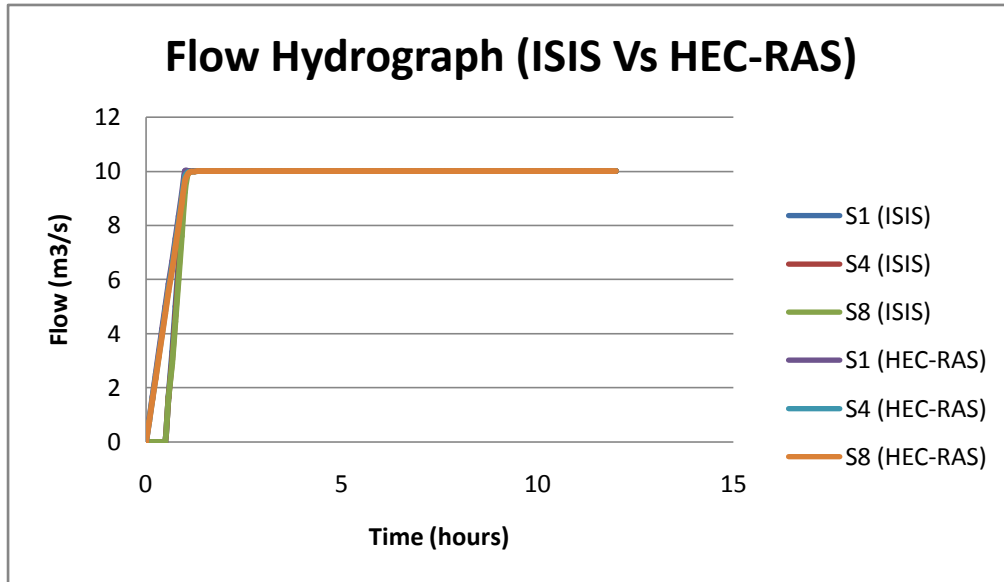
Figure 4.89 Flow hydrograph comparison
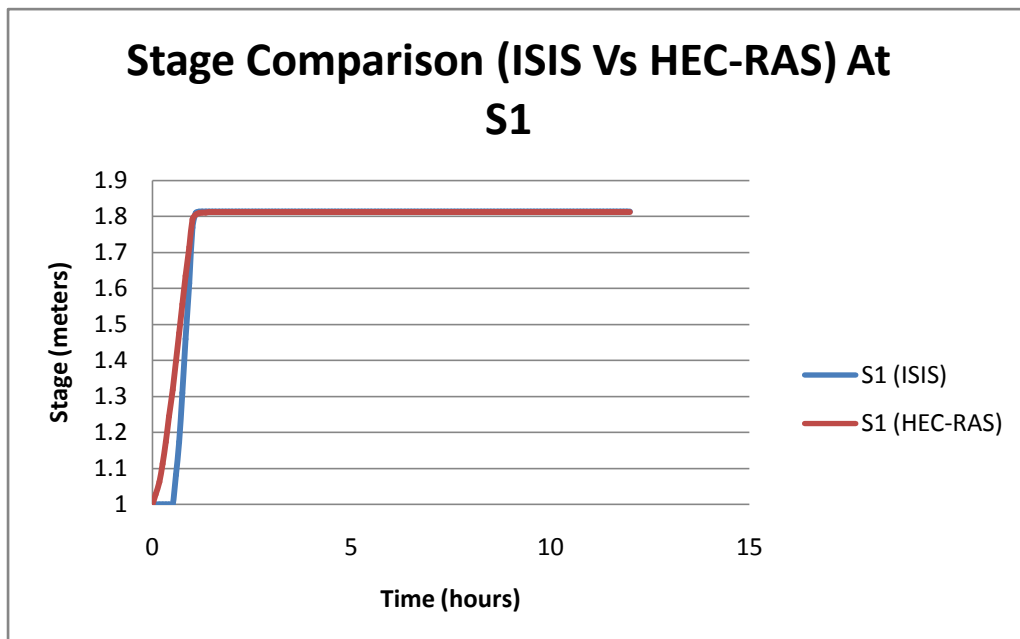
Figure 4.90 Stage hydrograph comparison

4.5 Case 5: Simplified dynamic model for flood routing in rectangular cross sections channels

4.5.1 General description

There are different types of methods and approaches can determine and estimate the floods and the change of flow over time for a particular stream, (Keskin and Aǵiralioǵlu, 1997) have derived a new momentum equation can estimate the flood routing for a rectangular channel; they called that model the simplified dynamic model. In addition to the simplified dynamic model the ISIS and the HEC-RAS have the ability to estimate the flood routing and find the variation of flow discharge over the change of time at different sections through the river stream. The following assumptions have been assumed to be the same as used in the journal of (Keskin and Aǵiralioǵlu, 1997); these assumptions have been inserted into ISIS and the HEC-RAS softwares in order to find the difference in the results between the simplified dynamic model, the ISIS and the HEC-RAS.

1. The total length of the channel equals 2000 meters.
2. The bottom width of channel equals 5 meters.
3. The depth of the channel equals 5 meters.
4. The bed slope of the river equals 0.0005.
5. Manning coefficient equals 0.0138.
6. The base flow has been assumed to be 3 m^3/s and the peak flow is to be $12 m^3/s$.

The cross section details have been assumed to be the same rectangular section used in the previous test case and the distances between the sections have been assumed as the same assumed by (Keskin and Aǵiralioǵlu, 1997); the following table show the detailed distances between the cross sections of the river:

Table 4. 6 Distances between sections

Section number	Distance to the next station
S1	50
S2	550
S3	600
S4	800
S5	0

4.5.2 Test case 5 using ISIS

- The first step is to define the upstream boundary condition to be as the same in the following figure which meets the design criteria for (Keskin and Aǵiralioǵlu, 1997):
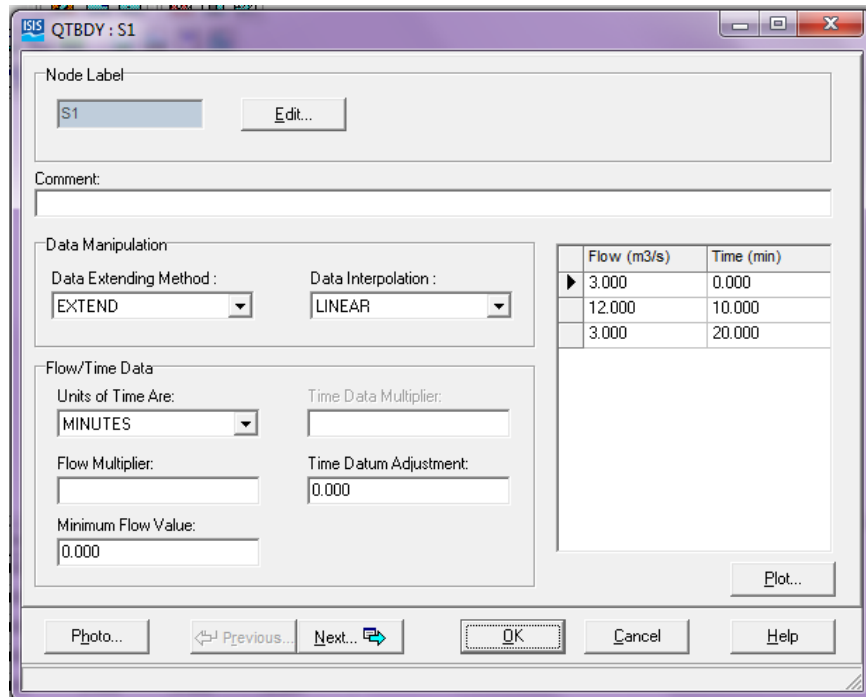
Figure 4.91 Upstream boundary condition

- The number of cross sections have been assumed to be 5 identical cross sections and their details have been assumed to be as in the following figure with the specified distances which have been used in the simplified dynamic model method:
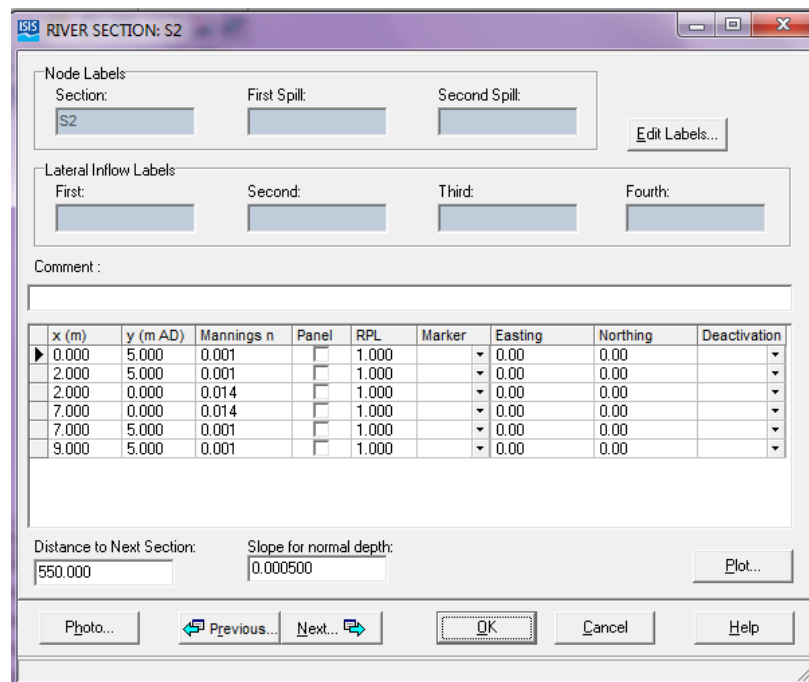
Figure 4.92 Cross section details

90

- The downstream boundary condition has been assumed to be a constant stage with 1 meter value over the change of time as shown in the following figure :
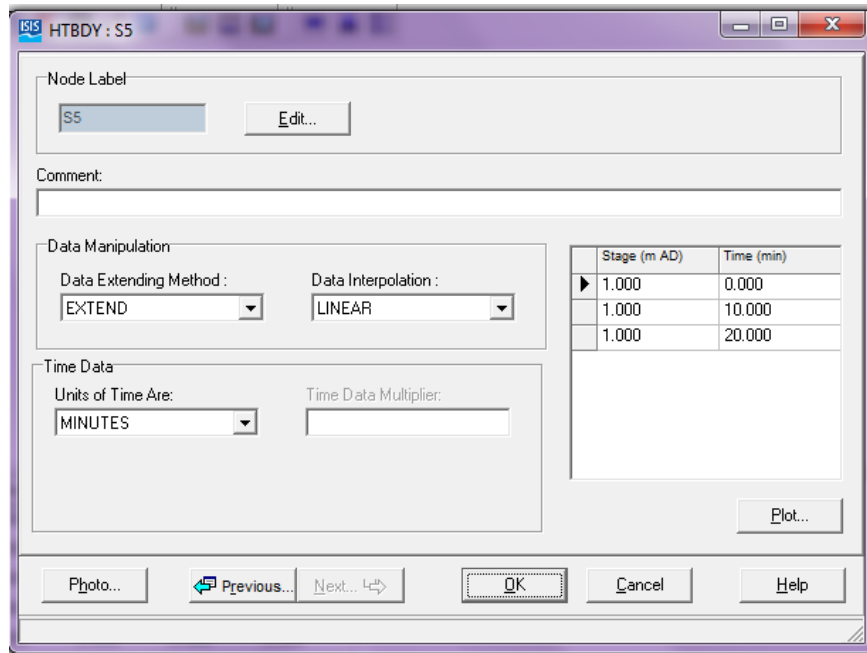
Figure 4.93 Downstream boundary condition

- The results of the steady state flow analysis are shown below and it shows the values of the stage which are around 1 meter according to the base flow 3 m^3/s :

Figure 4.94 Steady flow results

- The last step is to run the unsteady fixed time step analysis with time step equals to 3 seconds and interval of time equals to 10 seconds as shown in the following graph:
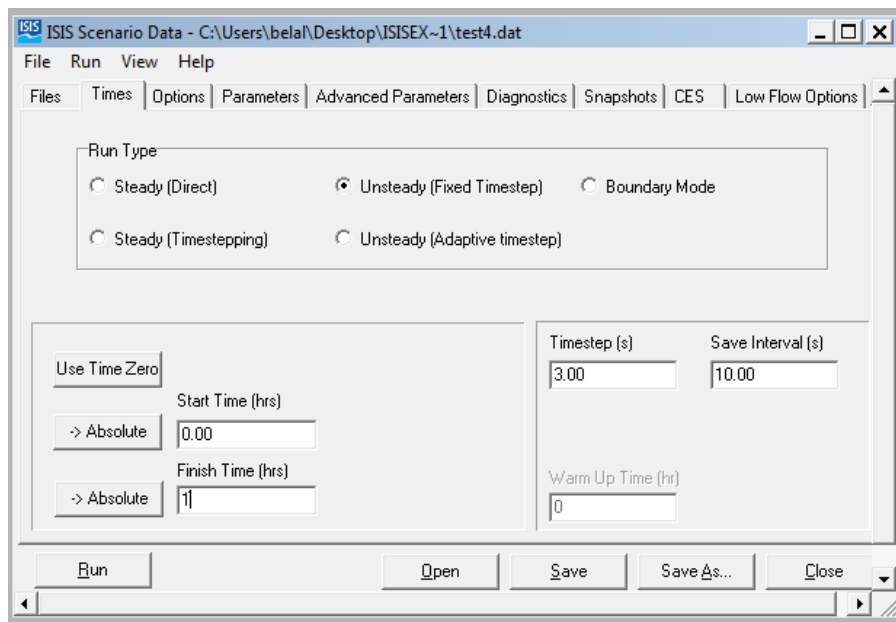
Figure 4.95 Unsteady state flow analysis

The unsteady state flow analysis has been completed and then the values of flow over time can be extracted through the CSV tabular command in the ISIS which gives the values of velocity, stage, flow and others in a table over the change of time:
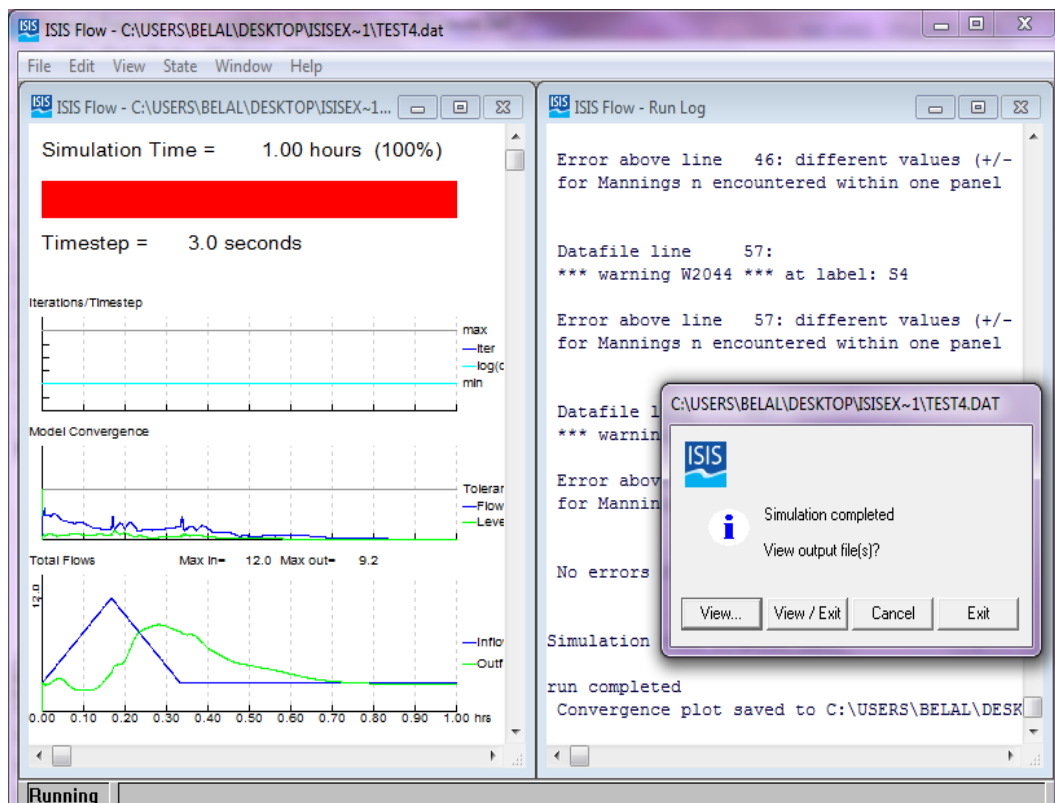
Figure 4.96 Unsteady flow analysis completed

92

The following figure shows the change of flow values over the time for the five different sections through the river, it is obvious that the highest value of flow exist in the first section and it becomes lower and lower at the following sections, the shape of the flow over time at the first two sections is stable triangle and it becomes more unstable at the following sections:
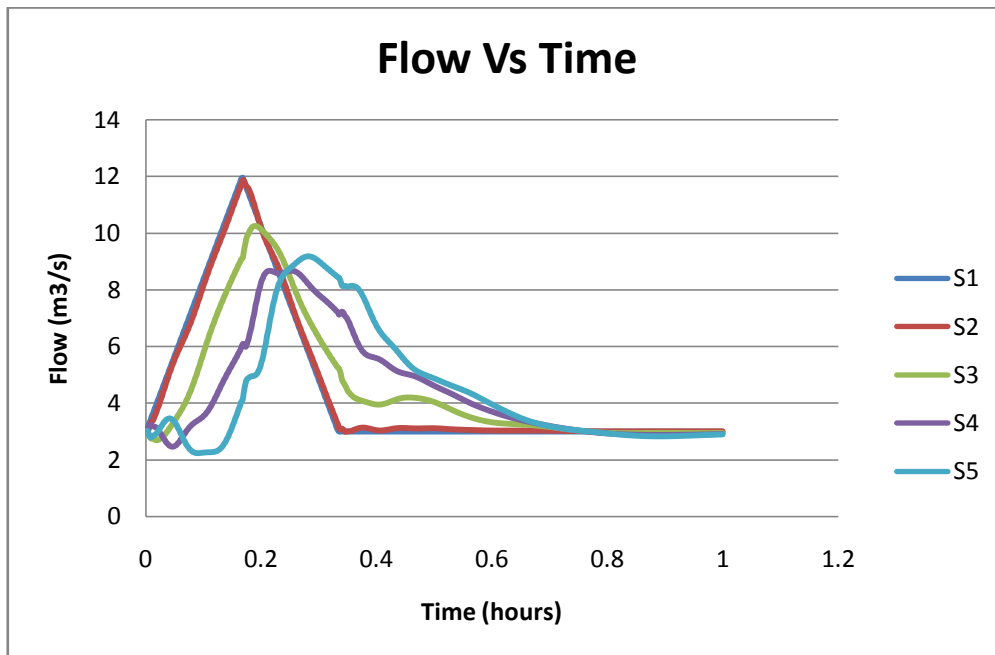
Figure 4.97 Flow hydrograph (ISIS)

The following figure shows the inflow hydrograph using the simplified dynamic model approach which has been found to be similar to the previous graph with some small changes in the stability and the smoothness of the last two curves for the last two sections:
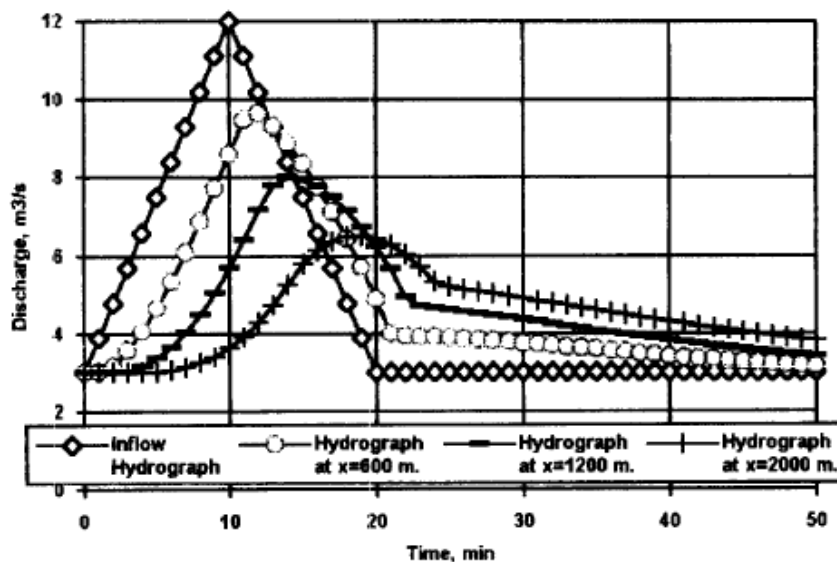
Fig. 1. Inflow hydrograph and computed outflow hydrographs at different distances.

Figure 4.98 Flow hydrograph, (Keskin and Aǵiralioǵlu, 1997)

93

The values of the inflow hydrograph at the first section using the simplified dynamic model approach and the ISIS have been drawn together in one graph in order to see the difference between the ISIS and the SD model:

1. The curve of the SD model hit the peak flow value which is 12 m^3/s but the curve of the ISIS can't hit that point but they are still very close to each other.
2. The ISIS is much quicker than the SD model in reaching the peak value and the base value as shown in the figure.
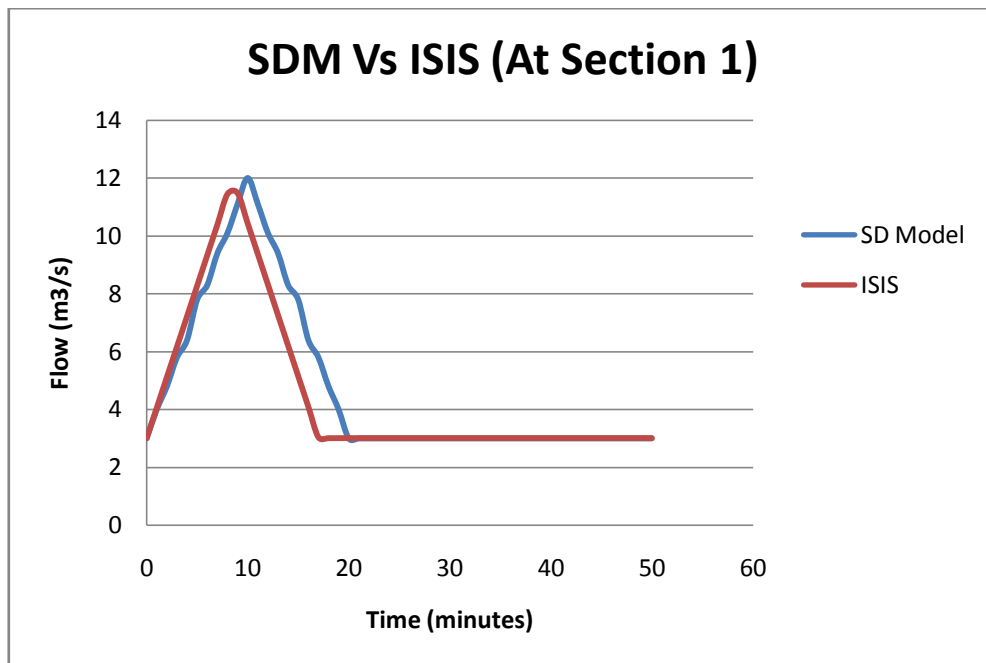
Figure 4.99 Flow hydrograph (ISIS Vs SDM)

4.5.3 Test case 5 using HEC-RAS

- Five sections have been inserted using the same details as used in the ISIS as shown in the following figure:
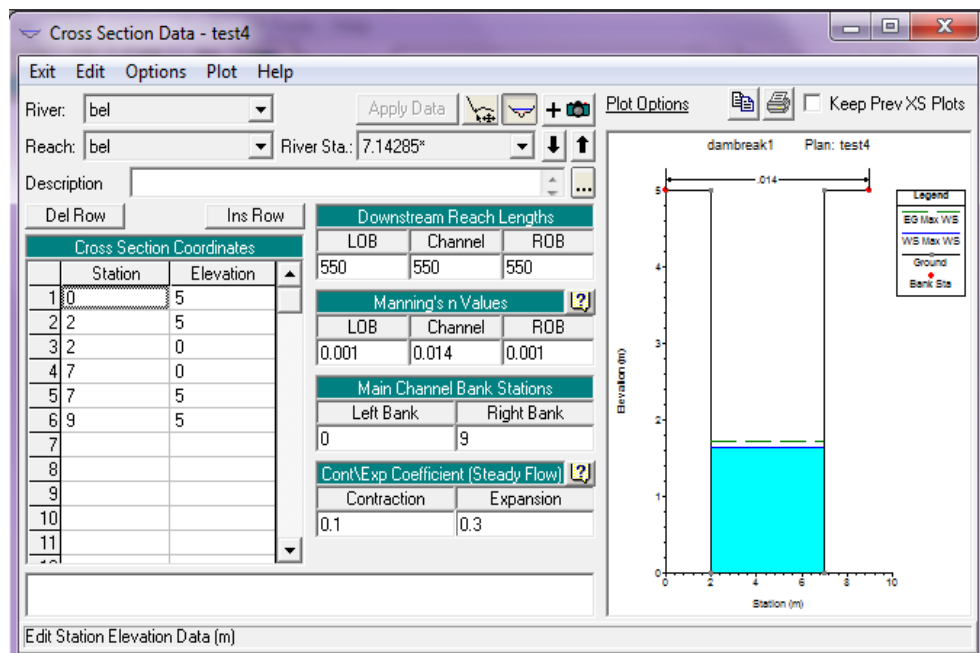
Figure 4.100 Cross section details

- Before running the simulation analysis, the upstream and the downstream boundaries have been defined using the same stage and flow hydrographs which have been used in the ISIS as shown in the following figures:
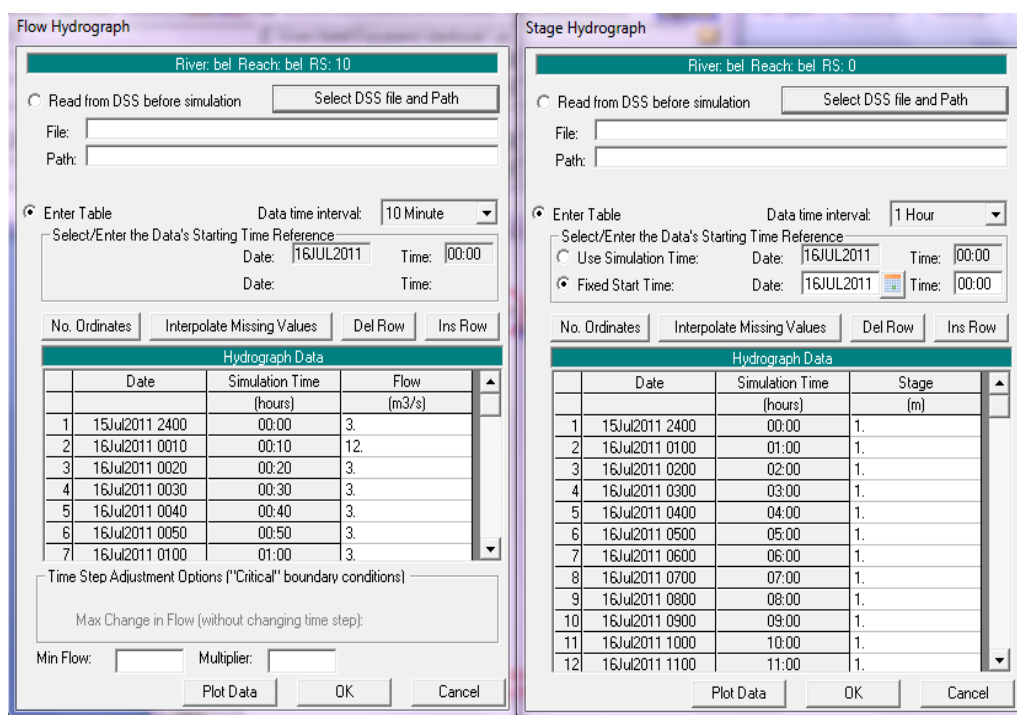
Figure 4.101 Unsteady flow boundary conditions

- The values of the flow discharge over the change in time have been found using the unsteady flow time series command in the HEC-RAS and then those values have been moved to the Excel to draw the flow hydrograph for the different river sections:
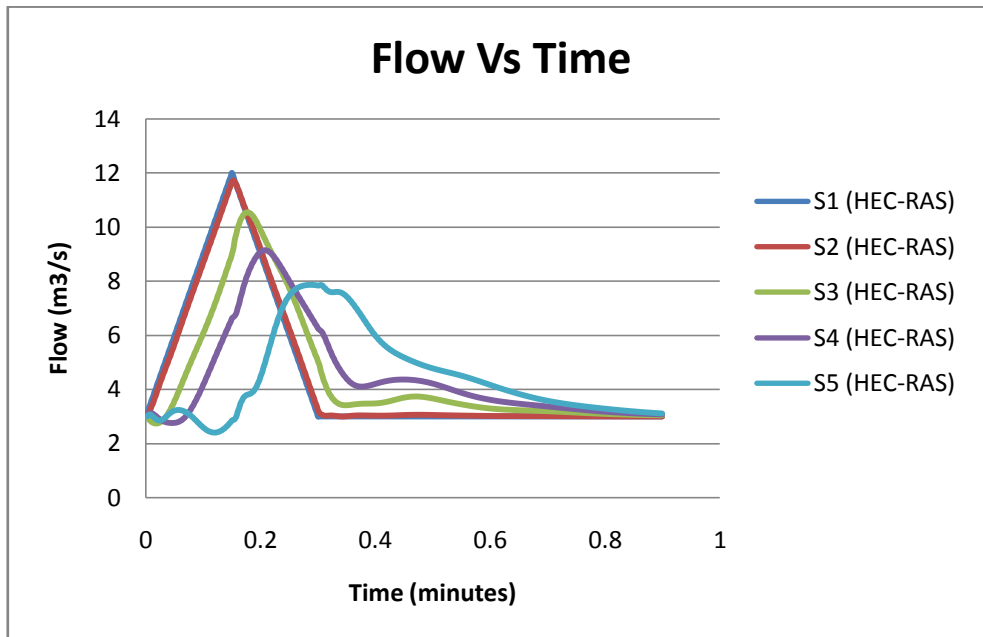
Figure 4.102 Flow hydrograph (HEC-RAS)

- The flow hydrograph of the ISIS and the HEC-RAS for S1, S3 and S5 have been drawn together in one graph and from the graph it can be noticed that the flow hydrograph of the HEC-RAS is quicker than the flow hydrograph of the ISIS and that means the HEC-RAS reaches the peak flow value before the ISIS. One other point can be noticed where the values of the flow hydrograph of the ISIS at the end of the river (at section 5) are higher than the values of the flow hydrograph of the HEC-RAS.
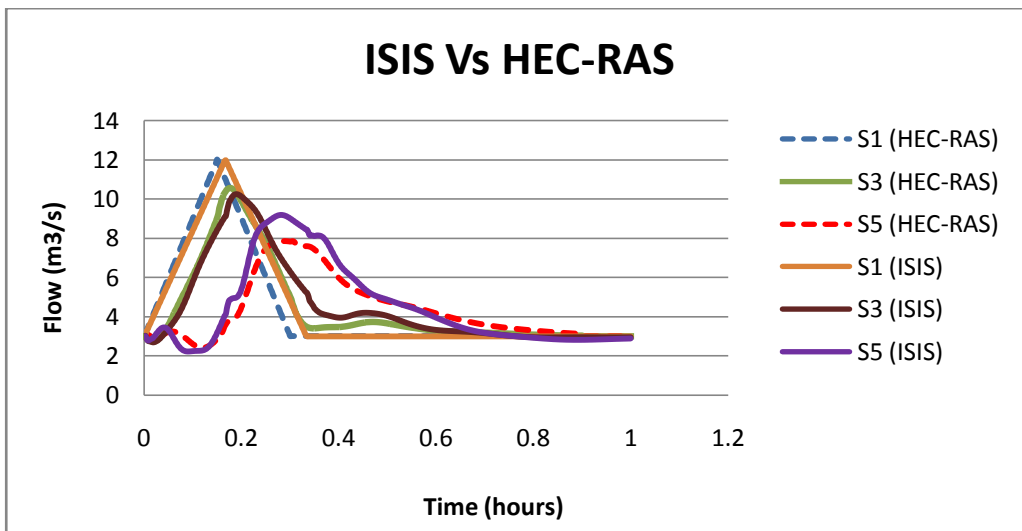
Figure 4.103 Flow hydrograph (ISIS Vs HEC-RAS)

96

The comparison of the flow hydrographs between ISIS, HEC-RAS and the simplified dynamic model can be noticed after drawing all of the flow hydrographs in one graph and in order to make the comparison very clear the graphs have been divided into 3 graphs each one describe the comparison at different section through the river:
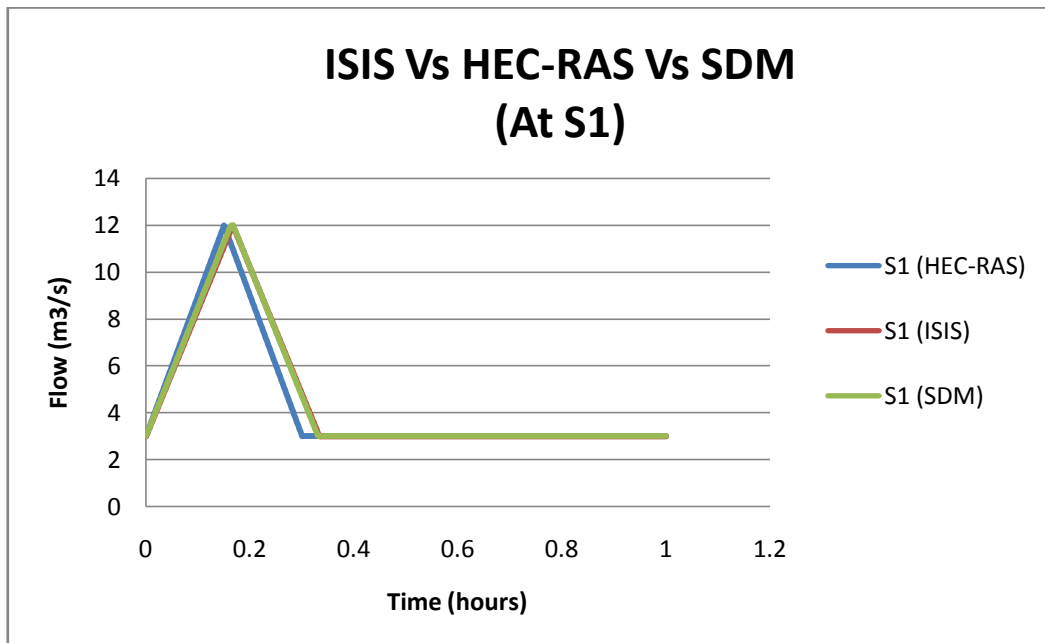
Figure 4.104 Flow hydrograph (ISIS Vs HEC-RAS Vs SDM) at S1
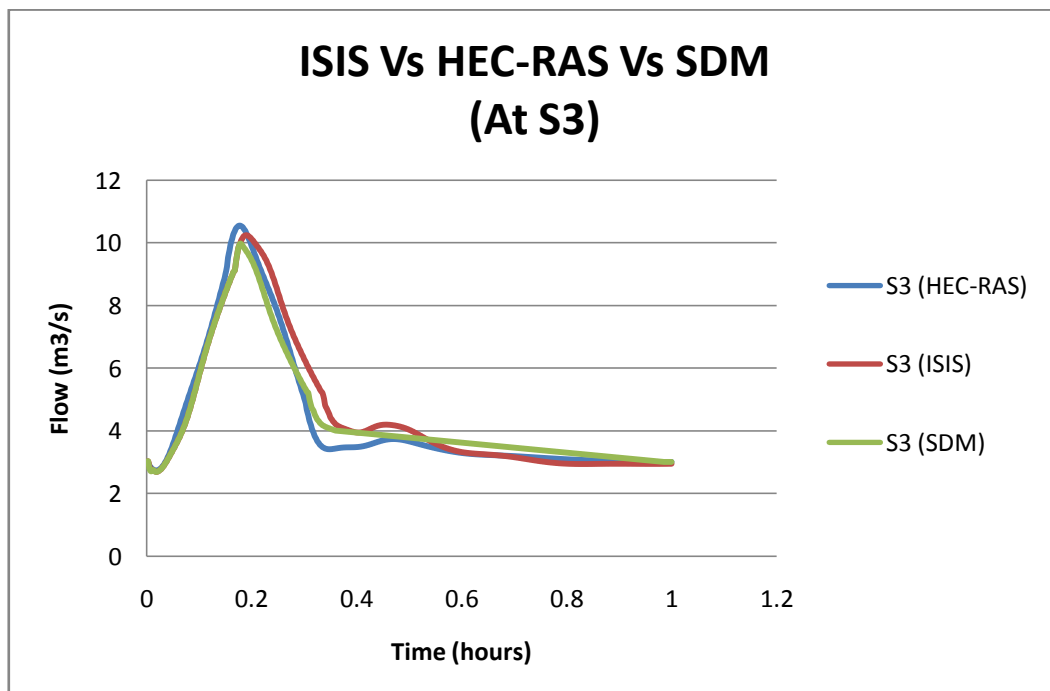
Figure 4.105 Flow hydrograph (ISIS Vs HEC-RAS Vs SDM) at S3
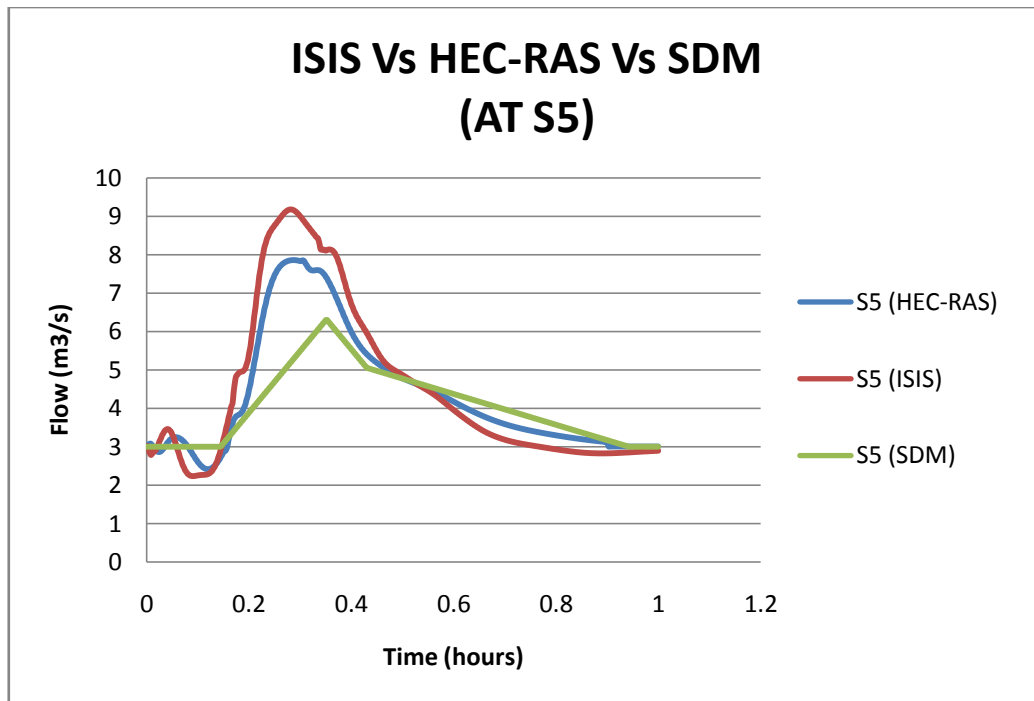
Figure 4.106 Flow hydrograph (ISIS Vs HEC-RAS Vs SDM) at S5

From the graphs above the following points can be noticed:

1. At the first section of the river the three methods are very close to each other even though the HEC-RAS is quicker than the other two methods.
2. At the third section S3, the values of the flow hydrograph for the HEC-RAS are the highest followed by the ISIS flow hydrograph values followed by the SDM method.
3. At the end of the river there is a clear instability in the ISIS and the HEC-RAS but in the SDM maintained its stability and the values are smoother than the other two methods.

4.6 Comparison highlight between ISIS and HEC-RAS

4.6.1 Comparison highlight between ISIS and HEC-RAS according to the cases observations

By looking at the comparisons which have been done between the two softwares during the 5 previous cases some major points can summarize the observed difference between the ISIS and the HEC-RAS, those differences are listed below in the following points:

- The ISIS and the HEC-RAS have resulted in the same stage hydrograph and the same output values for the first test case and the only difference between the two softwares were the two following points:
1. The HEC-RAS can reach higher maximum stage value than the ISIS.
2. The difference in stage values between the upstream and the downstream levels for the ISIS was larger than the one resulted from the HEC-RAS.
- In the comparison of the outflow hydrographs which have done in the second test case it has been found that the HEC-RAS recorded higher flow value than the ISIS at the time of the Malpasset dam break which was 7200 m^3/s but the ISIS recorded 6000 m^3/s , this means that the ISIS give a higher factor of safety than the HEC-RAS.
- In the third test case the two softwares have been noticed to be very close to each other and the only difference noticed is the time needed for the ISIS to reach to the maximum flow value and the same thing has been found in the stage hydrograph and from this point it can be concluded that the ISIS is slower than the HEC-RAS in most of the points.
- There is an obvious problem can be noticed in the ISIS when the dry bed river has been simulated because it needed some time to start the increasing in the flow and the stage hydrograph but the HEC-RAS started to increase at the (0,0) point and didn't need much simulation time to do that.
- There is no much difference between the two softwares in the last test case where the flow hydrographs at different river sections have been found to be similar to each other with a slight difference in the maximum flow value at the end of the river and some instability of the two softwares at the simulation of the end of the river.
- The HEC-RAS give the user the choice to enter the reach boundaries but in the ISIS it should be as a flow hydrograph or stage hydrograph only.

4.6.2 Comparison between ISIS and HEC-RAS according to the CPU time

The CPU required time to converge or to find the results in each software has been calculated for each simulation case and then showed in the following table:

Table 4. 7 CPU Elapsed Time

Case Number	ISIS		HEC-RAS	
	Steady	Unsteady	Steady	Unsteady
Case 1	00:00:01.16	00:00:02.63	00:00:00:00.31	00:00:01.96
Case 2	00:00:01.20	00:00:02.52	00:00:00:00.37	00:00:01.85
Case 3	00:00:01.10	00:00:02.78	00:00:00:00.17	00:00:02.05
Case 4	00:00:01.02	00:00:02.58	00:00:00:00.10	00:00:01.91
Case 5	00:00:01.10	00:00:02.03	00:00:00:00.25	00:00:01.73

From the table above it is very obvious that the CPU elapsed time by the HEC-RAS in both the steady and the unsteady flow analysis is much less than the CPU elapsed time by the ISIS and this is considered as an advantage to the HEC-RAS in the race between the two softwares. The following figure summarizes also the difference in the CPU elapsed time between the two softwares:
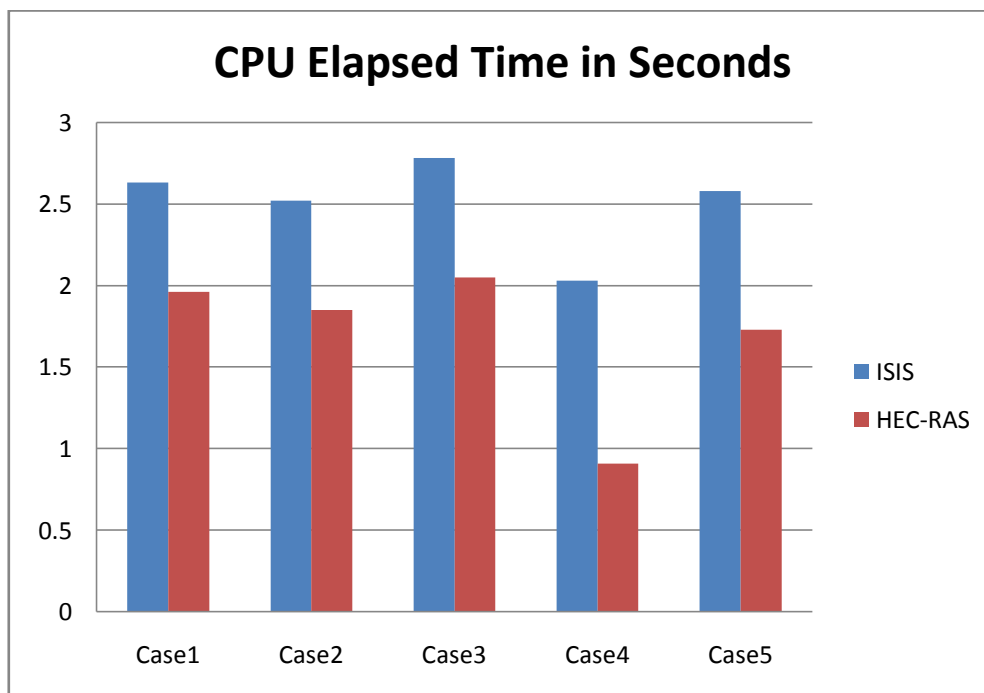

Figure 4.107 ISIS Vs HEC-RAS

Chapter 5: Conclusions, Recommendations and Future work

5.1 Conclusions

After five simulation cases have been done through the ISIS and the HEC-RAS, detailed comparisons have been conducted also so the best software can be recommended, there are many difficulties have been experienced from the simulation results while using the ISIS but on the other hand the HEC-RAS doesn't have these troubles, the following points conclude the differences between the two softwares:

- In simulating the dry river bed conditions the ISIS consumes some of the simulation time before starts to increase the flow and the stage values in both the flow and the stage hydrographs which will cause sort of replicating problem and this happens in the ISIS free edition because it is 1D limited package which doesn't have the ability to simulate the case of the dry river bed accurately on the other hand the HEC-RAS doesn't consume any time in simulating the dry river bed conditions and it starts to simulate the flow hydrograph from the first part of the second.

- In simulating the dam break test the ISIS gives a higher factor of safety than the HEC-RAS while the HEC-RAS gives values and results closer to the numerical physical models which have been done in previous reports.

- As mentioned before that the ISIS gives the ability to enter only one option to define the boundary conditions which are the stage and the flow hydrographs but the HEC-RAS gives more options to define that as the critical depth, the stage/flow hydrograph which gives the ability to insert the two values simultaneously.

- From the comparison of the CPU consumed time in both softwares it is very clear that the ISIS consumes more CPU time in order to find the results than the HEC-RAS.

Finally and from this research it has been found that both of the HEC-RAS and the ISIS are considered as very efficient softwares in the field of the numerical simulation analysis for the river floods and the dam break simulation tests but from the differences listed above it can be stated that the HEC-RAS is faster, more accurate, more reliable, more flexible and can simulate any test case without any difficulty however the ISIS could be similar in the processing speed yet a rather expanded range of values should be tried and tested at inputs to further validate a broad spectrum for the simulation outputs.

5.1 Recommendations and future work

In this part some points have been recommended to be applied and studied in the future studies:

- It should test both softwares ISIS and HEC-RAS with large amount of data input to study the processing time for the response that the simulations could give for these data and further validate which software could be more appropriate for the data type and ranges acquired through research.
- It should work out a complete investigation about the topics and collect more data to be inserted in the simulations so more reliable results will get out from the softwares.
- More improvements are needed in the next studies should be made on the simulations model and their limitations and try to solve the problems appeared in the previous studies such as the dry bed simulations in the 1D package.
- The future studies should propose an emergency response plans in the cases of dam breaks and river flooding.
- It should test which is the most suitable software for river flood simulations between the ISIS and the HEC-RAS throughout changing the types of the simulation cases and increase the size of each case.

REFERENCES

Abdullah R., Berhad S., Pengeluaran J. (2009). Dam Break Analysis: A Case Study at Durian Tunggal Dam, Mesyuarat Tahunan JTJAD Kali Ke 22 & Mesyuarat Exco JTJAD Bil 8 Sesi 2007-2010.

Ahmed M., Morris M. (1999), Malpasset Dam-Break Test Case A Comparison between the ISIS Numerical Model Results and the Field and Physical Model Data. ISIS, pp. 219-226.

Ahmed M., Morris M. (1999), Toce River Dam-Break Test Case Revisited. ISIS, pp. 207-218.

Anderson, J., (1995). Computational Fluid Dynamics. 1st Ed. London: McGraw-Hill Science/Engineering/Math.

Bentley Systems (2009) [online], Available on [http://docs.bentley.com/en/HMFlowMaster/FlowMasterHelp-06-37.html], accessed on 17/06/2011.

Brown J., Damery S. (2002), Managing Flood Risk in the UK: Towards an Integration of Social and Technical Perspectives. Transactions of the Institute of British Geographers, New Series, Vol. 27, No. 4, pp. 412-426.

Chanson H. (2009), Embankment Overflow Protections System and Earth Dam Spillways. in "Dams: Impact, Stability and Design", Nova Science Publishers, Hauppauge NY, USA, Ed. W.P. Hayes and M.C. Barnes, Chapter 4, pp. 101-132. ISBN 978-1-60692-618-5.

Coface (2003), Hydroelectric power stations and large dams. Environmental Guidelines, available on [www.adb.org/water/topics/dams/pdf/barragesgb.pdf], accessed on 12/06/2011.

Federal Emergency Management Agency (2006), "Why Dams Fail?", available on [www.fema.gov/hazard/damfailure/why.shtm], accessed on 17/06/2011.

Harrald J., Renda-Tanali I., Shaw G., Rubin, C. B., Yeletaysi, S. (2004). Review of Risk Based Prioritization Decision Making Methodologies for Dams. Risk Management. The George Washington University. Institute for Crisis, Disaster, and Risk Management.

Hill P., Bowles D., Jordan P., Nathan, R. (2003). Estimating Overall Risk of Dam Failure: Practical Considerations in Combining Failure Probabilities. Introduction Estimating Probabilities in Dam, ANCOLD Risk Workshop.

ISIS by Halcrow User Manual. [Online]. ISIS: Halcrow. Available on [http://www.isisuser.com/docs/manuals/isis/ISIS.htm], accessed on 12/06/2011.

Keskin, M. E., Ag, N. (1997), A simplified dynamic model for flood routing in rectangular channels, Journal of Hydrology, vol 202, issues (1-4), pp. 302-314, doi:10.1016/S0022-1694(97)00072-3.

Loh W. (2011), Application of ISIS in Modelling of River Hydraulics, University of Bradford.

NH (New Hampshire) Department of Environmental Services (2006), Types of Dams Common to New Hampshire. Environmental Fact Sheet, available on [http://des.nh.gov/organization/commissioner/pip/factsheets/db/documents/db-2.pdf] accessed on 15/06/2011.

NRCS (Natural Resources Conservation Service) (2003), Dam, Diversion, (No. Or Ft.) Code 348, pp. 1-2.

Nuera A. (2005), The Asian Development Bank And Dams. NGO Forum on ADB Guidebook Series, First Edition, pp. 3-10. Philippines.

Probe International (2008), "How Dams Fail?", available on [http://www.probeinternational.org/original-content/leading-causes-dam-failure], accessed on 20/06/2011.

Robinson R., Kerr J., Noschese F., Klein N., Belasen A. (2002), Cracking Dams. Syracuse University, available on [http://simscience.org/cracks/index.html], accessed on 11/06/2011.

Rosu C., Ahmed M. (1999), Toce River dam-break test case: A comparison between the ISIS numerical model and the Physical Model, ISIS, pp. 207-218.

Rowsell E., Handmer J. (1988), Flood Hazard Management in Britain: A Changing Scene The Geographical Journal, Vol. 154, No. 2, pp. 209-220.

Samuels P. (2003), Flood risk and flood forecasting – the state-of-the-art in EU research, EU-MEDIN on disaster research, available on [http://eprints.hrwallingford.co.uk/37/], accessed on 10/06/2011.

US Army Corps of Engineers, Institute for Water Resources Hydrologic Engineering. 2010. HEC-RAS River Analysis System Applications Guides. [online]. Available on [http://www.hec.usace.army.mil/software/legacysoftware/hec1/hec1-documentation.htm], accessed on 05/07/2011.

Vasilyev K. (2007), ISIS in River Modelling: A Practical Perspective, ISIS, available on [www.halcrow.com/isis], accessed on 20/06/2011.

Yongquan W. (1993), Solar activity and maximum floods in the world. Extreme Hydrological Events: Precipitation, Floods and Droughts. Proceedings of the Yokohama Symposium, IAHS Publ. no. 213.