Nile Basin Initiative
Eastern Nile Subsidiary Action Program
Eastern Nile Planning Model (ENPM) Project
Strengthening Knowledge Base and Modeling Capacity at ENTRO and EN Universities

Final Report

SPECIAL TECHNICAL STUDY:
DAM BREAK ANALYSIS FOR SELECTED CASCADE OF DAMS ON THE BLUE NILE

Civil Engineering Department

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List of Abbreviation

AMSL	Above Mean Sea Level
BCM	Billion Cubic Meter
ENPM	Eastern Nile Planning Model
ENSAP	Eastern Nile Subsidiary Action Program
ENTRO	Eastern Nile Technical Regional Office
GERD	Grand Ethiopian Renaissance Dam
GIS	Geographical Information System
IDEN	Integrated Development of the Eastern Nile
PMF	Probable Maximum Flood
Mm3	Million meter cube
MSL	Mean Sea Level
NBI	Nile Basin Initiative
NGO	Non Governmental Organizations
USBR	United States Bureau of Reclamation
USGS	United States Geological Survey
WMO	World Meteorological organization

Executive Summary

The ENPM project is among the seven projects identified within IDEN, ENSAP's first project. The objectives of the ENPM project is to strengthen the capacities in improved decision support modeling, to identify optimum water related investment and interventions and evaluate them from regional context. The project has three components: (a) development of knowledge Base for the EN; (b) Modeling system; (c) Institutional strengthening and capacity building component.

The ENPM is focused on improving the knowledge base and analytical capacity at ENTRO to better support regional and national activities relating to water resources planning and management through Eastern Nile Universities. As part of this effort ENTRO commissioned a special studies addressing dam breach analysis for selected cascade of dams on the Blue Nile sub-basin.

Nile basin, in particular Blue Nile sub-basin, has great potential for multipurpose dams which immensely benefits the Nile communities. Nile River has a number of major dams including High Aswan Dam (HAD), Rosaries, Sennar, Grand Ethiopian Renaissance Dam (GERD) which is under construction, Merowe dams, and upper Atbara dams (under construction). It is an accepted practice to have knowledge of flood hazard potential and possible emergency action plans in case of probable dam breaches (single or cascade) to downstream communities whose livelihood is dependent on the Nile River.

The main objective of this study is to identify, analyze, and assess impacts of dam breach in the Blue Nile cascade using appropriate modeling tools. This study covered three dams including GERD, Rosaries and Sennar dams in the Blue Nile sub-basin

Generally dam breach modelling can be carried out by either i) scaled physical hydraulic models, or ii) mathematical simulation using computer. As compared to physical model cost effective approach is to use mathematical model. Mathematical modelling of dam breach floods can be carried out by either one-dimensional analysis or two dimensional analysis. In the present study due to limitation of time in constructing, calibrating and verifying breach-hydraulic model, a one dimensional unsteady analysis which gives information about flood discharge and water levels, variation of these with time is adopted.

Using one dimensional hydraulic model (based on HEC-RAS software) three step approach is used to simulate dam break inundation:

1) Determining breach size and discharge
2) Routing the breach discharge downstream, and
3) Determining flood depths at possible hazard sites.

The study produced the following useful products:

- Extended Blue Nile river cross section upstream (from Mandaya dam site to Rosaries dam) over ENTRO-RTi hydraulic model which was done from Rosaries to Khartoum, although the accuracy is limited

- Terrain models for channel and floodplain of the Blue Nile extended from Mandaya dam site to Rosaries dam

- Geo-referenced Blue Nile hydraulic model and dam breach model configuration in HEC-RAS environment for 3 dams (GERD, Rosaries and Sennar)

The breach hydraulic model has shown that peak breach discharge downstream of GERD is about 27,203 m^3/s which is comparable to peak value of PMF at GERD / Rosaries. The flood peak attenuate to 17,411 d/s of Rosaries, to 16,855 d/s Sennar and to 14, 580 at Khartoum. It takes about 250 hours (10 day) for the peak flood to arrive at Khartoum. It is to be noted that estimate of the peak hydrographs at different locations are constrained by the difficult in having reliable cross section, Manning roughness of channel and flood plain topography.

For Scenario II, in case of only Rosaries and Sennar breaches, the peak breach flood peak is 17,411 m^3/s and 13,350 m^3/s d/s of Sennar.

For Scenario III, in case of all three dam breaches in dry periods (peak inflow 2100 m^3/s) the breach flood magnitude are reduced from 27,203 m^3/s to 24,641 m^3/s d/s GEDR and the peak flood d/s Rosaries is 17,381 m^3/s and at Sennar is 15,932 m^3/s reaching 13,500 m^3/s at Khartoum.

Estimate of the area of flooding, depth and velocity in flood plain become grossly unreliable as the cross section extension in the present study was based on 90x90 DEM and no other survey measurement was possible in this scope of study. Due to the above reason, all attempts made to produce realistic flood extent, depth and velocity map were unsuccessful. For more reliable works, acquisition of accurate topographic map including all assets (with vertical accuracy about 0.5 m) of Blue Nile reach in about 40 km band is recommended.

Initial model results should be seen considering the following limitations of the model:

Survey and Terrain data:

River surveying was limited to specific pilot reaches of the Blue Nile as was done by RTi. Floodplain areas were characterized using a 90-meter DEM which gives inaccurate results in estimating depth of flood in the floodplain. The river is subject to continuous changes in channel geometry due to its high sediment load.

Hydraulic Modelling:

As was also discussed by RTi, model results in Khartoum are sensitive to the downstream boundary condition, about which there is some uncertainty.

River and flood plain Manning roughness coefficient estimates are very approximate which will create uncertainty on flood depth, magnitude and velocity.

Breach Modelling:

Estimate of breach width and progression are approximate due to inadequacy of existing knowledge to model breach phenomena.

In conclusion, the result of this report shall be considered very preliminary, due to limitation of the extended detailed survey data (topography and property) and complexity of modelling river and flood plains, and uncertain breaching process for more than 900- km long stretch in short period.

1. Background

The Nile Basin Initiative

The Nile Basin Initiative (NBI) is a partnership of the riparian states of the Nile: Burundi, Democratic Republic of Congo, Egypt, Ethiopia, Kenya, Rwanda, Sudan, Tanzania and Uganda**.** The NBI seeks to develop the river in a cooperative manner, share substantial socio-economic benefits, and promote regional peace and security. The NBI launched with a participatory dialogue process among the riparian that resulted in a shared vision: "to achieve sustainable socioeconomic development through the equitable utilization of, and benefit from, the common Nile basin water resources." The discourse also gave birth to a strategic action program to translate its vision into concrete activities and projects.

NBI's Strategic Action Program

The NBI's Strategic Action Program is made up of two complementary components: the basin-wide Shared Vision Program, to build confidence and capacity across the basin; and subsidiary action programs, to initiate concrete investment and action on the ground at sub-basin levels. The programs are reinforcing in nature. The Shared Vision Program lays the foundation for unlocking the development potential of the Nile by building regional institutions, capacity, and trust. This can be realized through the investment-oriented subsidiary action programs, currently under preparation in the Eastern Nile and the Nile Equatorial Lakes regions.

Eastern Nile Subsidiary Action Program (ENSAP)

The Eastern Nile region is in the countries of Egypt, Sudan and Ethiopia and encompasses the sub-basins of the Baro-Akobo-Sobat, the Blue Nile, the Tekezé-Setit-Atbarah, portions of the White Nile in Sudan, and the main Nile. The Eastern Nile countries are pursuing cooperative development at the sub-basin level through the investment-oriented Eastern Nile Subsidiary Action Program (ENSAP).

ENSAP seeks to realize the NBI shared vision for the Eastern Nile region, and is aimed at poverty reduction, economic growth, and the environmental degradation reversal throughout the region. Towards this end, the Eastern Nile countries have identified their first joint project, the Integrated Development of the Eastern Nile (IDEN). IDEN consists of a series of sub-projects addressing issues related to flood preparedness and early warning; modeling in the Eastern Nile, power development and interconnection; irrigation and drainage; watershed management; and multi-purpose water resources development.

The Eastern Nile Technical Regional Office (ENTRO) is a technical regional body supporting the implementation of ENSAP. Established in 2002 and located in Addis Ababa, Ethiopia, ENTRO is responsible for providing administrative, financial management, and logistical support in the implementation and management of ENSAP. In general, ENTRO's core functions are: ENSAP coordination and integration; project preparation; financial management; communications and outreach; training; monitoring and evaluation; information exchange; and serving as the secretariat for ENSAP organizations.

Eastern Nile Planning Model (ENPM) Project

The ENPM project is among the seven projects identified within IDEN, ENSAP's first project. The objectives of the ENPM project is to strengthen the capacities in improved decision support modeling, to identify optimum water related investment and interventions and evaluate them from regional context. The project has three components: (a) development of knowledge Base for the EN; (b) Modeling system; (c) Institutional strengthening and capacity building component.

The ENPM is focused on improving the knowledge base and analytical capacity at ENTRO to better support regional and national activities relating to water resources planning and management. To this effect, ENTRO intends to employ the services of a consultant to assist ENTRO in helping the Eastern Nile region develop a more integrated set of tools for flood forecasting and management. The revised implementation plan of the project strongly depends on the involvement of the main universities in the EN basin (Cairo University from Egypt, University of Khartoum from Sudan, Addis Ababa University from Ethiopia and Juba University from South Sudan) in achieving the project objectives. To support the above mentioned goals two special technical studies of relevance to the universities and ENTRO were selected and given to each university.

Addis Ababa University took the lead in carrying out two technical special studies by the title 1- Dam Breach Analysis for selected cascade of dams on the Blue Nile, and 2- Surface-GW balance for conjunctive use in the Tana Beles sub-basin. This paper will address the first special study i.e. Dam Breach Analysis for cascade of dams on the upper Blue Nile river.

2. Rationale and Need for the Study

Dams are an important part of this nation's infrastructure, providing flood control, water supply, irrigation, hydropower, navigation, and recreation benefits. Despite their many beneficial uses and value, dams also present risks, of low probability, to property and life due to their potential to fail and cause catastrophic flooding. To mitigate these risks, dam owners and regulators carefully analyze and inspect dams to identify potential failure modes and protect against them. Since no program for preventing failure can ever be certain, and because the potential for loadings exceeding design limits can never be eliminated, another essential part of risk mitigation is simulating potential failures and planning for them. These plans can include public education programs, development of warning systems and procedures, and development of effective evacuation procedures.

Dam breaches can be caused by overtopping a dam due to insufficient spillway capacity during large inflows to the reservoir, by seepage or piping through the dam, along internal conduits or foundation, embankment slope slides, earthquake damage and liquefaction of earthen dams from earthquakes, or landslide-generated waves within the reservoir. Hydraulics, hydrodynamics, hydrology, sediment transport mechanics, and geotechnical aspects are all involved in breach formation and eventual dam breach. The large number of breach cases of all types of dams is found to be earth dam breaches. Though it is a rare case, concrete gravity dams tend to have a partial breach as one or more of the monolith sections formed during the dam construction are damaged by the escaping water.

Blue Nile sub-basin has great potential for multipurpose dam. In addition to the existing major dams such as HAD and Rosaries and Sennar, Grand Ethiopian Renaissance Dam (GERD) is under construction, and heightening of Rosaries dam and Merowi dams (both completed). It is essential to have knowledge of flood hazard potential and possible emergency action plans during potential dam breaches (single or cascade) to downstream communities whose livelihood is dependent on the Nile River.

3. Objectives of the Study

The main objective of this study is to identify, analyze, and assess impacts of dam breach in the Blue Nile cascade using the one dimensional HEC-RAS model. The study covers dams in Blue Nile sub-basin including GERD, Rosaries and Sennar dams.

Specific objectives are:

- Evaluate the extent of flooding (water depth and area extent – leading to flood hazard zones) in case of dam breach (single or cascade) for each of the above mentioned dams.
- Assess preliminary property damage that will be associated with such flooding

4. Extent of the study

The study area encompasses the Blue Nile stretch from the downstream of Lake Tana Khartoum (Figure 1)

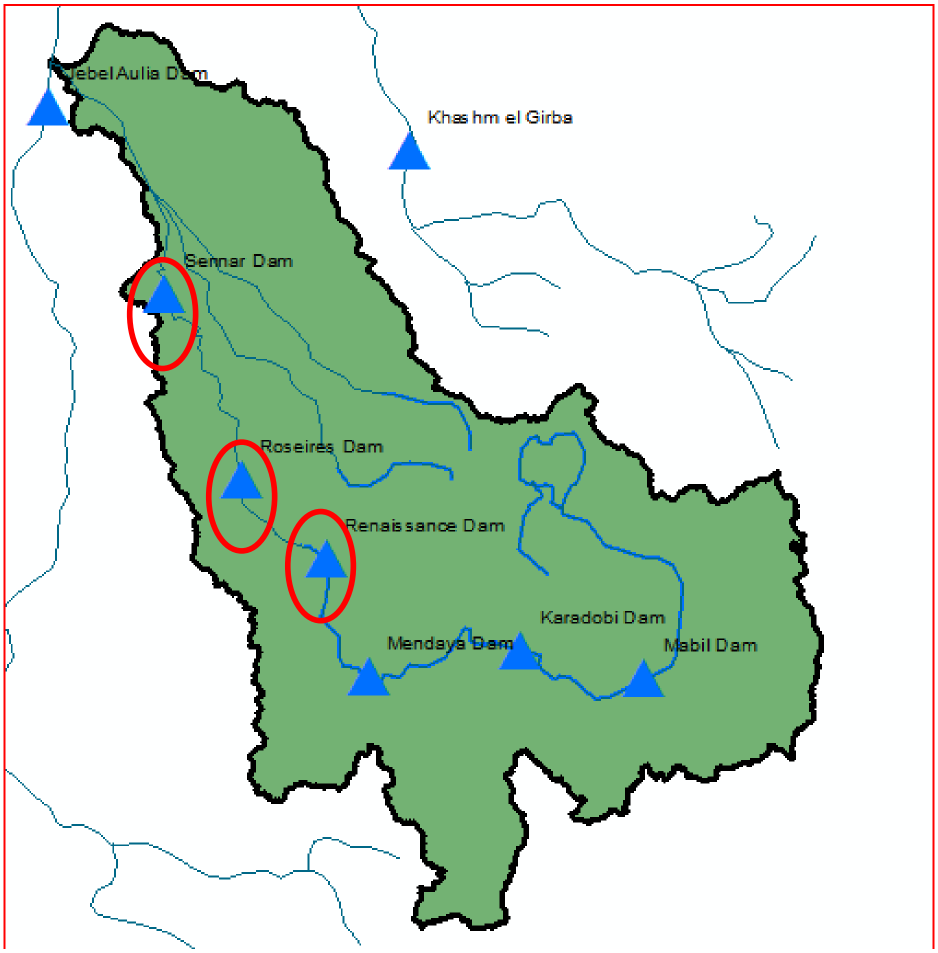


Figure 1: Study area (Blue Nile river and major dams)

Topographic characterization of seven reaches along the Blue Nile for about 900 km long is given in Table 1. Annex I typical reaches feature from Mandaya to Khartoum.

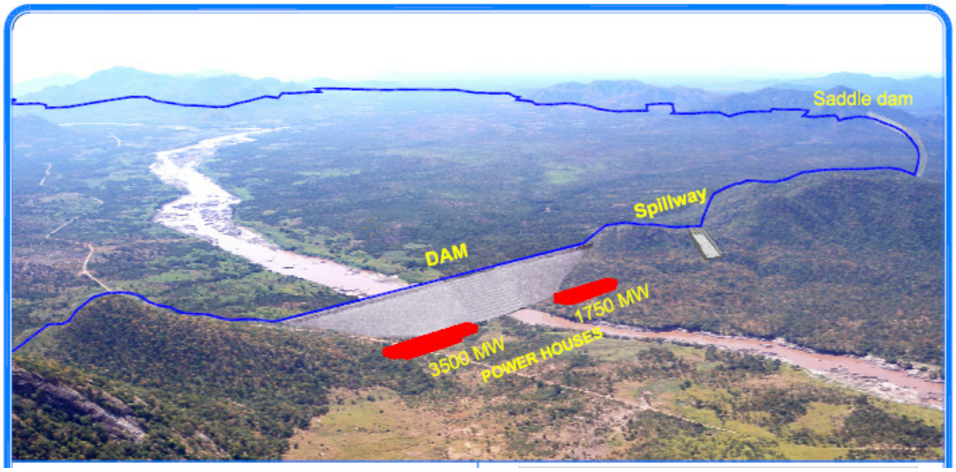
Table 1: Blue Nile reach topographic characterization

Reach	Length	Description
Ethiopia to Roseires Dam	107 km	Incised in rock and shows a minimum of lateral erosion and deposition with very minor flood plains.
Roseires to Singa	203 km	Drops 24 m and has a meandering nature in a clay plain characterized by lateral erosion and deposition, forming flood plains on the inner curves and bank erosion in the outer curves.
Singa to Sennar	90 km	Meandering channel characterized by pronounced shifting from west to the east with many remnants of ox-bow lakes and abandoned channels.
Sennar to Hag Abdalla	108 km	Characterized by many bends and ox-bow lakes at shorter distances. In this portion, the Dinder River joins the Blue Nile.
Sennar to Wad Medani	66 km	Characterized by many meanders and severe gully erosion forming Kerib land on both banks. The Rahad River joins the Blue Nile at Wad Medani.
Wad Medani to Hasahisa	47 km	Characterized by moderate meandering and both towns are located in pronounced bends.
Hasahisa to Khartoum	157 km	Characterized by wider meander bends.

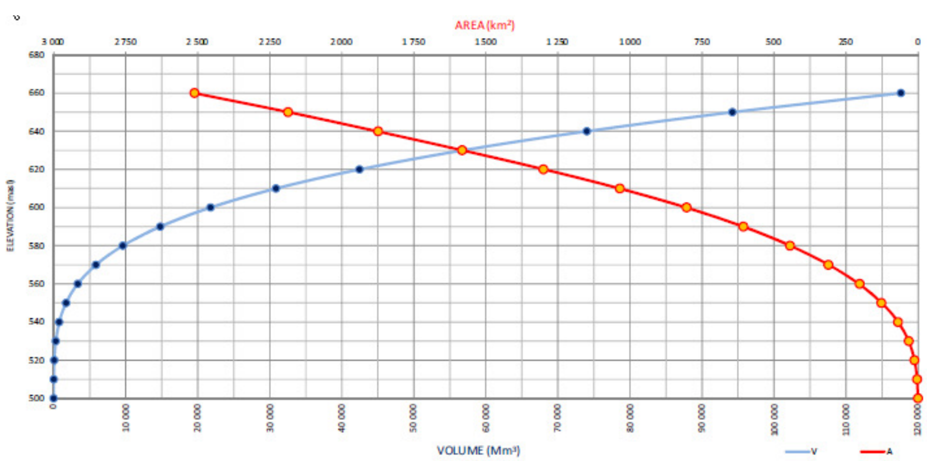
5. Dam Characteristics

Salient characteristics of GERD, Rosaries and Sennar dams are given below.

GERD – **145 m RCC dam and 50 m rockfill saddle dam**

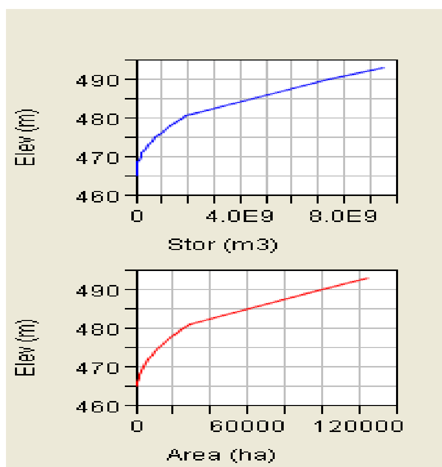


Reservoir Characteristics	
FSL (masl)	640
MOL (masl)	610
Storage @ FSL (MCM)	62930
Active Storage (MCM)	37780
Surface Area at FSL (Km^2)	1680
Extent Reservoir US (Km)	200
Evaporation Losses (MCM/Yr)	1679.72



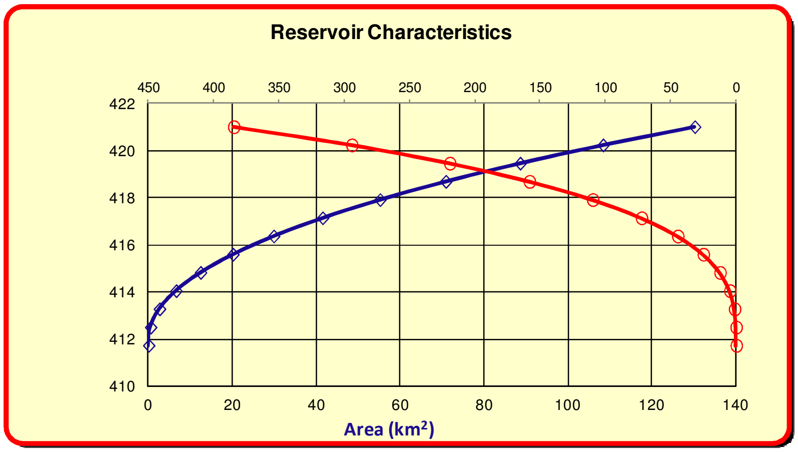
	SILL EL. (m a.s.l.)	DESIGN HEAD (m)	DESIGN HEAD ELEVATION (m asl)	
GATED SPILLWAY	524.9	15.1	640	Normal operating level
UNGATED SPILLWAY	640	2.5	642.5	Approx water level in the reservoir for RP=10000 y flood
UNGATED EMERGENCY SPILLWAY	642			

Rosaries dam



Spillway Characteristics	
Spillway Type	Radial Gates
Spillway Crest Elevation 9masl)	463.7
No. of Gate Bays	10
Spillway Length/Width (m)	10
Height of Gates (m)	12
Spillway Capacity at FSL (m^3/s)	18,750

Sennar dam



Reservoir Characteristics	
FSL (masl)	421
MOL (masl)	417
Storage @ FSL (MCM)	480
Active Storage (MCM)	413
Surface Area at FSL (Km^2)	152
Evaporation Losses (MCM/Yr)	224.79

6. Methodology and Literature Review

6.1 Dam-break flood risk Assessment

Dam-break risk assessment defines the magnitude of the flood hazard that may occur due to a dam failure, estimates its main consequences and evaluates its significance. To assess this type of risk, it is generally necessary to undertake integration between the dam reliability analysis, in order to evaluate the probability of dam failure and numerical dam-break flood simulations, in order to estimate the potential damages. Predicting the effects through flood simulation allows the identification of flood prone areas, the flood path and magnitude and aims to assess valley vulnerabilities as well as losses and damages. However, determining the consequences of dam failure is a difficult task.

There is a range of risk evaluation guidelines (WCD, USBR, ICOLD, ANCOLD, etc.) and criteria that can be used to assess the risk of a dam. These risk guidelines vary in how probability and risk are defined and assessed. Some guidelines are based upon the probability of an outcome occurring, whereas others are based upon the cumulative probability distribution for a type of outcome. For example, the ANCOLD (2003) Guidelines on Risk Assessment recommend that the assessment of tolerable risk is based upon an assessment of probability of loss of life for the person or group which is most at risk (individual risk) and the societal risk criteria and the USBR (1999) publication is based upon the total probability of failure. For this study, an appropriate guideline will be used for the risk assessment study. Dam safety guidelines dictate that the consequences of failure should be based on the incremental consequences of failure. This means that the consequences of a dam failure during a large flood event should not include the damages that would have resulted from the flood event regardless of the dam failure.

A GIS can be an extremely useful tool in many aspects of dam failure consequence assessment. Using GIS techniques and modelling software, preliminary effects and damages of a dam-break flood will be investigated. The following steps are necessary:

- Determine the extent of flood damage based on the flood maps at the time of flood event
- Determine the affected spatial areas, and delineate risk zones and scale of damage

Demographic and socio-economic data of the downstream community together with the results of dam-break simulation are planned to produce risk maps in a GIS environment. Downstream community information must include the location, number and nature of buildings and other places of occupation in the failure impact zone. In this short period, however, what we can do is to use secondary data / map from Sudan along the Blue Nile. The major sources of data is RTi recent study. This information may be obtained from maps, persons with local knowledge. A recent aerial photograph, if available, also provides useful information on the location of downstream infrastructures.

6.2 Causes of dam breach

Dams have been playing a vital role in the development of any country by meeting the water demand for domestic use, irrigation, power generation, flood protection etc. There are about 45,000 large dams in the world. Lemperiere (1993) concluded that, today's dams are ten times safer than fifty years ago, but population numbers downstream of dams have increased by a factor of twenty and are continuing to grow and such a growth of population is making the economical, technical criteria in determining the yardsticks of sustainable development. About 5% of the dams have been failing due to several factors like floods, landslides, earthquakes, deterioration of the foundation, poor quality of construction and act of war etc (Figure 2).

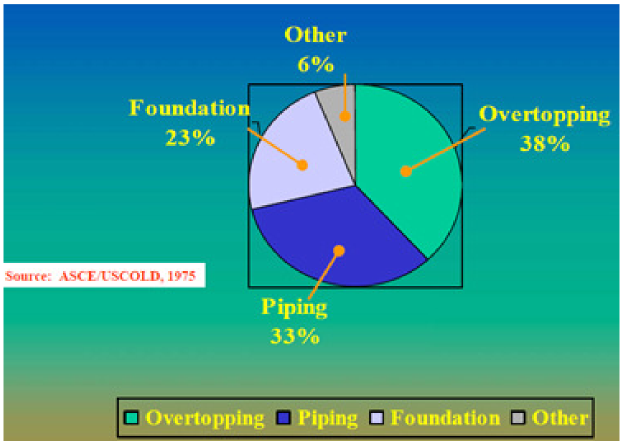


Figure 2: Causes of dam breach

It is to be noted that in reality, overtopping and piping failures (71%) are not applicable to the planned dams in Blue Nile gorge are RCC / Concrete type.

Overtopping Failure

Water level in the reservoir is raised above the top of the dam and over flow past the spillway section. The Overtopping may be due to any of following reasons:
- Actual flood flows reaching the reservoir may be much more than the design flood while the spillway is of inadequate capacity to discharge the flows.
- Insufficient free board with the result that strong winds and surface wave action are sufficient to cause spillage.
- Faulty or non-operation of spillway gate and other operating equipments at the time of peak flood may result in heading up of excessive flow in the reservoir resulting in overtopping or dam bursting.

Piping Failure

Though seepage through the dam is inevitable, any excessive and uncontrolled seepage through the dam section and its foundation may lead to piping or sloughing causing failure of the dam. Piping can occur in the embankment, through the foundation and from the embankment into the foundation.

6.3 Dam Break Modelling Approach

Generally dam break modelling can be carried out by either i) scaled physical hydraulic models, or ii) mathematical simulation using computer. A modern tool to deal with this problem is the mathematical model, which is most cost effective and reasonably solves the governing flow equations of continuity and momentum by computer simulation. Mathematical modelling of dam breach floods can be carried out by either one-dimensional analysis or two dimensional analysis. In one dimensional analysis, the information about the magnitude of flood, i.e., discharge and water levels, variation of these with time and velocity of flow through breach can be estimated in the direction of flow. In the case of two dimensional analysis, the additional information about the inundated area, variation of surface elevation and velocities in two dimension can also be assessed.

Using simulation approach, there are three phases in a dam break inundation analysis namely:

(1) Determining breach size and discharge
(2) Routing the breach discharge downstream, and
(3) Determining flood depths at possible hazard sites.

Breach Size and Parameters

Breach initiates at a certain point on the top of the earthen or rock-fill dam due to overtopping. The breach widens and deepens and results in increasing flood flow through the breach. The predominant mechanism of breaching for earthen or rock–fill-dam is by erosion of embankment material by the flow of water over the dam. When the breach stems from overtopping, excessive shear stresses on the surface induced by water flow, initiates erosion process. Erosion will begin when local shear exceeds a critical value, after which earthen dam material is set in motion. The formation and duration of breach depending on the height of the dam, the material used for the dam construction, compaction of material, quantity and duration of flood flow. Overtopping breaches are usually either rectangular or trapezoidal in shape (Figure 3). The duration of breach is usually few minutes to few hours.

To carry out a dam break routing simulation, breach parameters must be estimated and provided as inputs to the dam-break inundation mapping simulation models (Tony L. Wahl, 2001). Dam breach parameters describe the following: Size or dimensions of the dam breach, Time it takes for the breach to form, Bottom elevation, Bottom width, side slope, location of the breach, time of breach development and time pattern of the breach development (linearity degree of breach formation) Initiation of the breach.

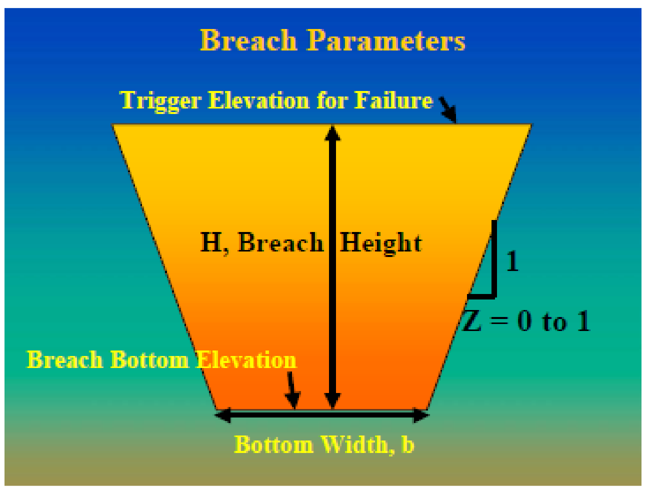


Figure 3: Breach parameters

A hypothetical breach can be characterized with four parameters:(a) breach height hb, (b) final breach bottom width b or average breach width Br, (c) sideslopes z and (d) and the total time for breach formation τfailure .These parameters can be estimated using empirical relationships that were derived from documented case studies of dam failures. The results from these methods were used to generate a matrix of probable breaching parameters that can be compared to determine the most critical conditions. The selected condition can be applied to the HEC-RAS model for the development of risk hazards inundation maps as a results of dam breach.

If the breach parameters are known, the breach discharge can be determined using hydraulic principles. However, unless a major structural weakness and obvious breach condition are known, determining the breach parameters must be based on experience and engineering judgment.

6.4 Methods of Prediction Breach Parameters

Coleman S.E, Andrews.D.P and Webby M.G (2002)

Coleman S.E, Andrews.D.P and Webby M.G carried out experimental studies on overtopping breaching of non-cohesive homogeneous embankments. The non-dimensional equations for prediction of breach pattern, from their studies are given as:

$$L_b^* = L_b/Hs = 16\,(h_b^*)^{1.5} \tag{3}$$

$$H_b^* = (2.30\,t_* + 1)^{-1} \tag{4}$$

Where

$L_{b*} = L_b / H_s$,

$h_{b*} = h_b / H_s$,

$H_b^* = H_b / H_s$,

$t^* = gt^2/H_s x10^6$

L_b= length of breach crest,
H_b = Height of breach crest above foundation,
g= acceleration due to gravity,
t = time of breach.

Froehlich (1995)

Froehlich (1987) developed relations for breach width, side slope, and time of failure based on 43 case studies of dam failures. In a follow up paper (Froehlich, 1995) this work was expanded to include a total of 63 dam failures. The empirical equations for breach width and time of failure from the latter paper are listed below:

$$B_r = 0.183\,K_O V_w^{\;0.32} h_{bm}^{\;0.19} \tag{5}$$

$$\tau_{failure} = 0.0025 V_w^{\;0.53} h_{bm}^{\;-0.90} \tag{6}$$

The Froehlich equations include empirical coefficients Ko that differentiate between a piping (Ko =1.0) breach and an overtopping breach (Ko =1.4).

MacDonald and Langridge-Monopolis (1984)

MacDonald and Langridge-Monopolis (1984) considered 42 case studies of dam failure, 30 of which were classified as "earth-fill dams," and developed predictive equations for the volume (V_{Eroded}) of earth removed during a breach and the time ($\tau_{failure}$) of breach formation. The equations are shown below:

$$V_{er} = 0.0261\left(V_w h_w\right)^{0.769} \quad \text{Erodible materials} \tag{7}$$

$$V_{er} = 0.00348\left(V_w h_w\right)^{0.852} \quad \text{Resist materials} \tag{8}$$

$$\tau_{failure} = 0.179\left(V_w\right)^{0.364} \tag{9}$$

$$b = \frac{V_{er} - h_d^{\,2}\left(0.5T_d + \frac{0.5h_d Z_3}{3}\right)}{h_d\left(C + \frac{h_d Z_3}{2}\right)} \tag{10}$$

Where
b = bottom width of the breach, in feet
V_{er} = volume of eroded material, in cubic feet
h_d =height of dam, in feet
C = crest width of dam in feet
Z1 = slope of the upstream face of the dam, in horizontal units per 1 vertical unit
Z2 = slope of the downstream face of the dam, in horizontal units per 1 vertical unit
Z3 = Z1 + Z2 *the side slope of the breach opening, in horizontal units per 1 vertical unit (0.5 as recommended by MacDonald and Langridge-Monopolis)

Von Thun and Gillette (1990)

Von Thun and Gillette (1990) used the data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure. They also developed upper and lower bound prediction equations for breach formation time based on plots of breach formation time versus depth of water above the breach invert. The lower bound is for erosion resistant material and the upper bound is for easily erodible material. The predictive equations are listed below:

$$B_r = 2.5h_w + C_b \tag{11}$$

$$\tau_{failure} = 0.02h_w + 0.25 \quad \text{Erodible materials} \tag{12}$$

$$\tau_{failure} = 0.015h_w \quad \text{Resist materials} \tag{13}$$

Von Thun & Gillette parameters also do not distinguish between a piping and overtopping breaches.

Table 2: C_b is a function of reservoir storage (Wahl, 1998)

Reservoir Size (m^3)	C_b (m)
<1.23 x 10^6	6.1
1.23 x 10^6 to 6.17 x 10^6	18.3
16.17 x 10^6 to 1.23 x 10^7	42.7
>1.23 x 10^7	54.9

Bureau of Reclamation (1982)

$$B_r = 3h_w \quad (14)$$

$$\tau_{failure} = 0.011h_w \quad (15)$$

As per the UK Dam Break Guidelines and U.S. Federal Energy Regulatory Commission (FERC) Guidelines, in the case of concrete gravity dams, the breach width should be taken 0.2-0.5 times the crest length of the dam. The breach development time for gravity dam should be about 0.2 hour. The breach depth can been taken corresponding to the relatively weaker locations in the dam such as galleries, sluices etc.

It is to be noted that the parameters describing a breach are typically taken to be the breach depth, width, side slope angle and formation time. Breach depth is usually taken to be the dam height and some argue that the breach side slope angle should be taken as vertical for most cases, so breach width and breach formation time are the two parameters of most interest. Numerous investigators have developed regression models to predict these two parameters. Wahl (1998) reviewed the methods available at that time and Wahl (2004) and Froehlich (2008) have considered the uncertainty of breach parameter estimates and found them to be very significant, especially the time parameter. The review by Wahl (2004) found that the best methods of breach width prediction (Reclamation 1988; Von Thun & Gillette 1990; Froehlich 1995b) had uncertainties of about ±1/3 order of magnitude, and the best predictions of breach time (Froehlich 1995b) had uncertainties of about ±2/3 order of magnitude.

6.5 Breach Formation and Progression

The breach is the opening formed in the dam as it fails. The actual failure mechanics are not well understood for either earthen or concrete dams. In previous attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed completely and instantaneously .The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. These assumptions are somewhat appropriate for concrete arch-type dams, but they are not appropriate for earthen dams and concrete gravity-type dams.

The terminal breach shape is specified by a parameter (Z) identifying the side slope of the breach, i.e., 1 vertical: z horizontal slope, and parameter (b) which is the terminal width of the bottom of the breach. The range of side slope parameter Z value is: $0 < Z < 2$. Its value depends on the angle of repose of the compacted and wetted materials through which the breach develops. Rectangular, triangular, or trapezoidal shapes may be specified in this way using various combinations of values for Z and b. For example, $Z = 0$ and $b > 0$ produces a rectangular shaped breach. Similarly, $Z > 0$ and $b = 0$ yields a triangular-shaped breach. The terminal breach width (*b*) is related to the average width of the breach (B_r), the breach depth (*hd*), and breach side slope (*Z*) such that:

$$b = B_r - 0.5zh_d \quad (16)$$

The bottom elevation of the breach is simulated as a function of time (t) according to the following relationship:

$$h_b = h_d - (h_d - h_{bm})\left(\frac{t_b}{\tau}\right)^{\rho} \qquad \text{If } 0 < t_b < \tau \quad (17)$$

Where:
h_{bm} = final elevation of the breach bottom
t_b = the elapsed time since start of breach
ρ = the breach nonlinearity formation exponent

τ= full breach formation time

Note that the nonlinearity formation exponent (ρ) ranges from 1 to 4, with ρ = 1 corresponding to a linear breach rate and ρ = 2 corresponding to a nonlinear quadratic breach rate. Note that generally a linear breach formation rate is assumed.
The instantaneous bottom width (bi) of the breach is given by the following relationship:

$$b_i = b\left(\frac{t_b}{\tau}\right)^{\rho} \qquad \text{If} \quad 0 < t_b < \tau \qquad (18)$$

During the simulation of a dam failure, the actual breach formation commences when the reservoir water surface elevation (h) exceeds a specified value, hf. This feature permits the simulation of an overtopping of a dam in which the breach does not form until a sufficient amount of water is flowing over the crest of the dam. A piping failure may be simulated by specifying the initial centerline elevation of the pipe.

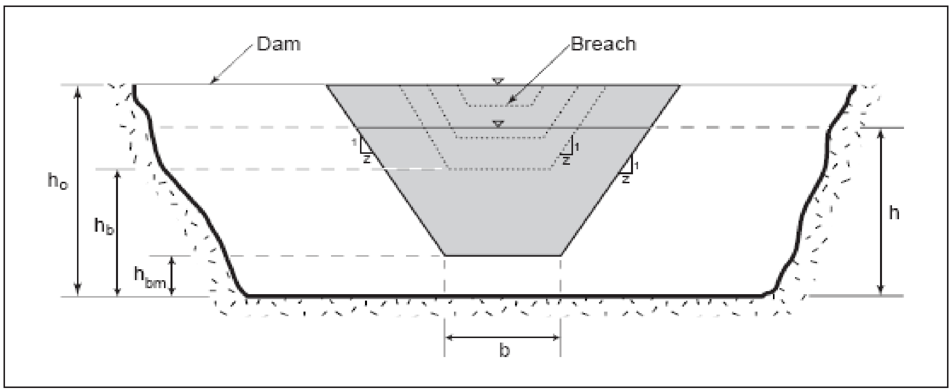


Figure 4: **Front view of dam showing formation of overtopping breach**

6.6 Methods of Estimating the Peak Breach Outflow Discharge

In addition to prediction of breach parameters, many investigators have proposed simplified methods for predicting peak outflow (Q_p) from a breached dam. These methods are valuable for reconnaissance-level work and for checking the reasonability of dam-break outflow hydrographs developed from estimated breach parameters. This study considers the relations by: USBR envelope curve equation, Hagen (1982), MacDonald and Langridge-Monopolis (1984), Froehlich (1995a) and
All of these methods are straightforward regression relations that predict peak outflow as a function of dam and/or reservoir parameters, such as V_w, h_d & h_w, with the relations developed from analyses of case study data from real dam failures.

Empirical Equations

USBR envelope curve equation

$$Q_p = 75 h_w^{1.85} \tag{19}$$

where:

Qp = peak discharge at the dam, in cubic feet per second, and
hw = depth of the water behind the dam at time of breaching, in feet.

Hagen (1982)

$$Q_p = 0.54(S.h_d)^{0.5} \quad \textit{In which "S" is the reservoir storage at time of failure} \tag{20}$$

MacDonald and Langridge-Monopolis (1984)

$$Q_p = 1.15(V_w h_w)^{0.412} \tag{21}$$

Froehlich (1995)

$$Q_p = 0.607\left(V_w^{0.295} h_w^{1.24}\right) \tag{22}$$

ICODS (Interagency Committee on Dam Safety)

Another peak breach discharge equation based on historical data that also includes storage behind the dam at the time of breach has been developed by the Subcommittee on Emergency Action Planning of ICODS (Interagency Committee on Dam Safety):

$$Q = 370(HS)^{0.5} \tag{23}$$

where:

Q = peak discharge at the dam, in cubic feet per second
H = height of water in the reservoir measured from streambed, in feet, and
S = reservoir storage capacity cubic feet

6.7 Water Elevation-Discharge Relationship

Empirical water elevation-discharge relations such as broad crested weir flow can be utilized for simulating rapidly varying flow in order to develop a breach outflow hydrograph *(D.L. Fread, 1988)* in the locations such as a dam, bridge, or waterfall (short rapids) along a waterway where the St. Venant equations are not applicable, the flow is rapidly varied rather than gradually varied.

$$Q_b = K_S\left(1.7 b_i (h_w - h_b)^{1.5} + 1.35 z (h_w - h_b)^{2.5}\right) \tag{24}$$

Where:
Q_b = breach outflow.
b_i = the instantaneous breach bottom width.
h_b = the elevation of the breach bottom.
h_w = the elevation of the water surface just upstream of the structure.
z = the side slope of the breach.

Equation (24) has presented by D.L.Fread (1988) DAMBRK model as well as BREACH model (1984), and used by hydrologic engineering center software HEC-1.

6.8 Dam Break Flood Routing

A simple routing procedure is based on using a "decay rate" which are determined from historical dam breaches:

$$Q_x = 10(\log[75D^{1.85}] - aX), \text{ if } S/D > 40 \quad (25)$$

$$Q_x = 10(\log [370 (DS)0.5] - aX), \text{ if } S/D < 40 \quad (26)$$

where:
Q_x = peak discharge at mile X, in cubic feet per second.
S = storage, for the reservoir at crest of dam, in acre-feet,
D = depth of water behind the dam as measured from crest of dam to streambed, in feet,
a = 0.01 for reservoir storage > 1,500 acre-feet,
a = 0.04 for storage between 800 and 1,500 acre-feet, and
a = 0.1 for storage < 800 acre-feet.

One dimensional analysis is generally accepted, when valley is long and narrow and the flood wave characteristics over a large distance from the dam are of main interest. On the other hand, when the valley widens considerably downstream of dam and large area is likely to be flooded, two dimensional analysis is necessary (HEC-RAS Manual).

Moreda (2010) Compared to FLDWAV (which is a combination of DWOPER and DAMBREAK models, HEC-RAS offers a much more user friendly GUI, better documentation, training, and the option to specify more detailed cross-section and hydraulic structure information.

Both models uses two basic equations which are the continuity and momentum equations respectively as shown below.

$$\frac{\partial Q}{\partial x} + \frac{\partial s_{co}(A + A_o)}{\partial x} - q = 0 \quad (26)$$

$$\frac{\partial (S_m Q)}{\partial t} + \frac{\partial (\beta Q^2 / A)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_f + S_e + S_i\right) + L - W_f B = 0$$

Where:
Q = the flow,
H = the water-surface elevation,
A = the active cross-sectional area of flow,
Ao = the inactive off channel storage area,
S_{co} and S_m = sinuosity factors,
X = longitudinal distance along the rive channel,
T = the time,
q = the lateral inflow or outflow,
β = the momentum coefficient for velocity distribution,
g = the acceleration due to gravity,
S_f = the channel flood-plain boundary friction slope,
Se = the expansion-contraction slope,
Si = addition friction slope for viscous fluids and debris flow,
B = active low top width and
W_f = the effect of wind resistance on the surface of flow.

Among the slopes that are listed in the above equation S_f is dominant.

HEC-RAS unsteady flow model solves essentially the same equations as FLDWAV, except that the current version of HEC-RAS does not include the wind effects term W_f, the expansion and contraction slope term (S_e), or the viscous fluid term (S_i) and is simplified to:

$$\frac{\partial Q}{\partial t}+\frac{\partial QV}{\partial x}+gA\left(\frac{\partial z}{\partial x}+S_f\right)=0 \tag{27}$$

Where:

- Q = the flow, V= the velocity
- Z= the elevation of water surface
- A = the active cross-sectional area of flow,
- A_T = the inactive off channel storage area,
- X = longitudinal distance along the river channel,
- t = the time,
- q = the lateral inflow or outflow,
- g = the acceleration due to gravity,
- S_f = the channel flood-plain boundary friction slope,

Similar to FLDWAV, HEC-RAS uses a four-point implicit finite difference scheme to generate a set of finite difference equations. However, HEC-RAS solves the finite difference equations using linearization and a sparse matrix linear algebra solver while FLDWAV applies a Newton -Raphson iteration technique to solve the nonlinear equations. HEC-RAS program estimate breach flow using input data shown in the template (Figure 5).

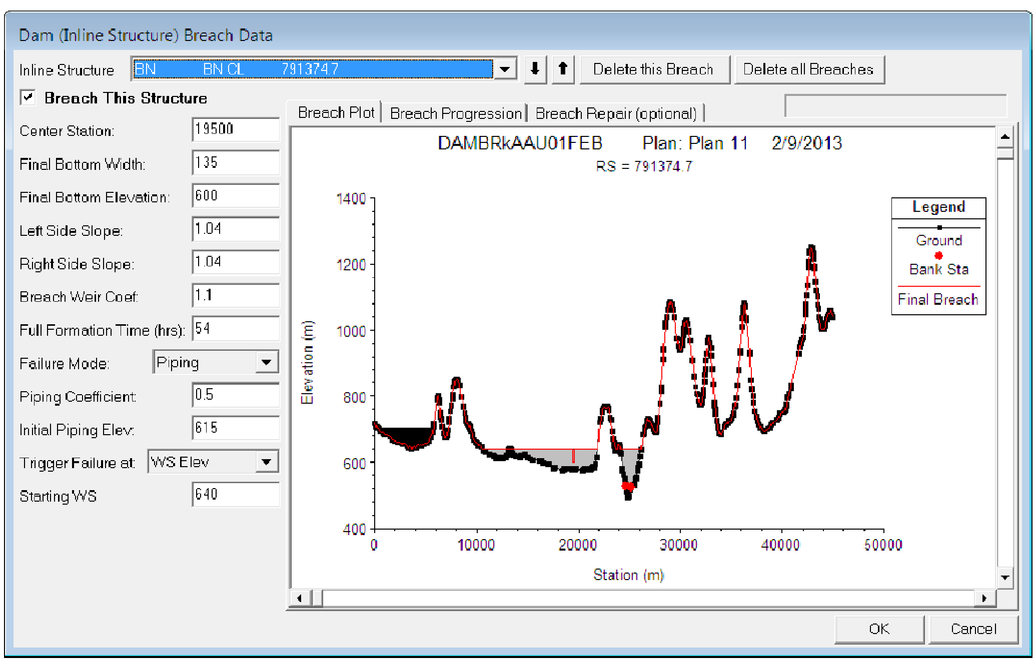


Figure 5: Sample Template used in HEC_RAS program to input breach data

Once the breach discharge is estimated downstream routing is done using the continuity and momentum equations described above.

7. Hydraulic Model Development

7.1 General Methodology

HEC-RAS version 4.1 is used for the dam breach analysis for three dams in the Blue Nile sub-basin. One-dimensional unsteady flow routing model option has been utilized to model breach which capable of integrating complex channels and structures under very dynamic hydrologic conditions. HEC-RAS also has the capability of modeling dam breach events under a wide range of scenarios. Cross sections, stream centerlines, and other geometric features of the stream were extracted from GIS using HEC-GeoRAS and ArcGIS based plat from.

7.2 Development of the HEC-RAS Model

Channel and floodplain information about the Blue Nile reach needed by the HEC-RAS model was extracted from aerial imagery data using HEC-GeoRAS. HECGeoRAS is an ArcGIS tool that creates geometric input for HEC-RAS and performs inundation mapping from HEC-RAS output. Additional data not extracted by HEC-GeoRAS from GIS data was defined in the HEC-RAS model, such as the dimension of Inline lateral structures (dams). HEC-GeoRAS utilizes user-defined GIS layers to extract information from the terrain data. For example, a GIS layer of cross-section cut lines is needed so that HEC-GeoRAS knows where to extract cross-section information from the terrain data. Table 3 contains a list of the GIS layers that were created, by digitizing lines and polygons, as well as a brief description of importance.

Table 3: Description of GIS layers created to extract information for hydraulic modeling.

GIS Layer	Description
Stream Centerline	The stream network is comprised of stream centerlines for the Blue Nile Stream starting from the upstream Mandaya area up to Khartoum. Stream centerlines identify the connectivity between different streams for flood routing and are used to establish the river and reach name for cross sections and hydraulic structures within HEC-RAS.
Cross-Section Cut Lines	Cross-section cut lines establish the location and extent for extracting station-elevation data from the terrain model for the channel cross- sections. Cross-sections were added at locations in order to capture changes in the channel and floodplain geometry as well as define an adequate floodplain boundary for mapping purposes. For this study a cross section width of 80km has been adopted.
Flow Path Centerlines	Flow path centerlines define the center-of-mass of flow in the left overbank, channel, and right overbank areas. Flow path centerlines are used to compute the lengths between adjacent cross sections.
Inline Structures	Inline structure cut lines define the location on the stream and lateral extent of man-made and natural structures that act as dams and/or weirs. In this model three online structures to represent Sennar , Rosaries and GERD dams has been incorporated.
Land Use	In a previous consultancy of flood risk mapping by RTI (River side technology) and UNESCO CWR (2010) for the Blue Nile polygons were created to define areas of similar land use. Manning's n roughness coefficients were then associated with each land use type based on field observations. HEC-GeoRAS overlays the land use polygons with the cross-section cut lines to determine the horizontal distribution of Manning's n for each cross-section. This was only used to estimate the left and right overbank Manning's n values. The channel n values were estimated separately. This work was used and further extended upstream beyond the border to cover the upstream reservoir areas.

7.2.1 Cross-Section Elevation Data

For generating the cross sections a hybrid DEM was created incorporating Hydroshed 90 m DEM data and surveyed cross sections along the routes of the Blue Nile River. More than 30 survey cross sections were merged with the DEM to create the new DEM which is 40 KM wide on the left and right banks each (Figure 6). In total 1080 cross-sections have been used.

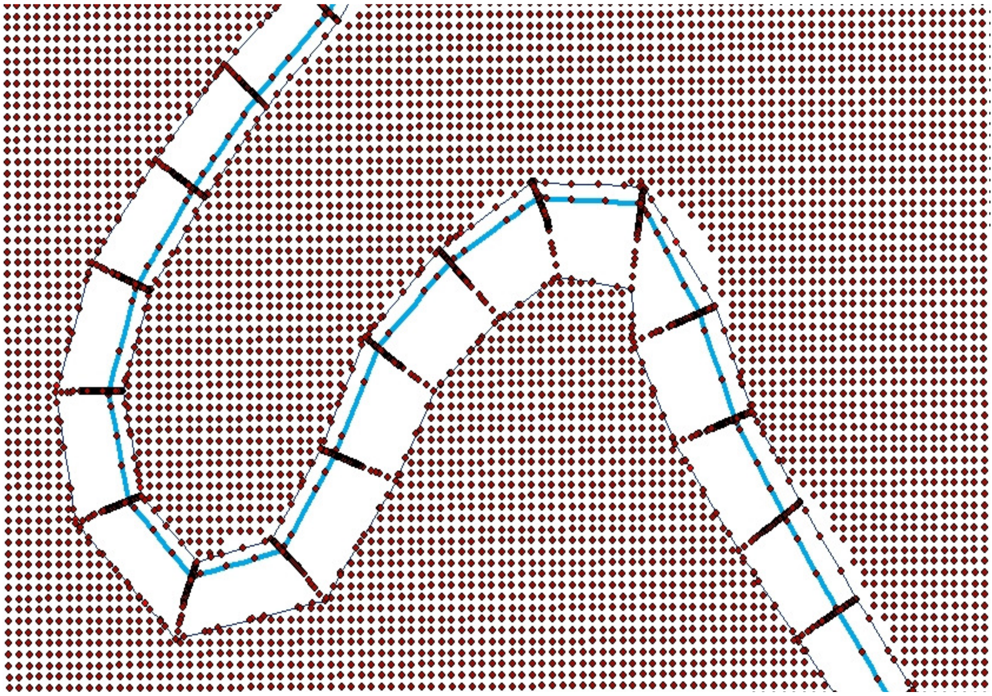


Figure 6 TIN Nodes And Tin lines used to create the Hybrid DEM

Several cross section surveys carried out under the flood risk mapping by RTI (River Side Technology) and UNESCO CWR (2010) were used for the DEM and also building the structures. Some of the cross sections are plotted below (Figure 7 and 8)

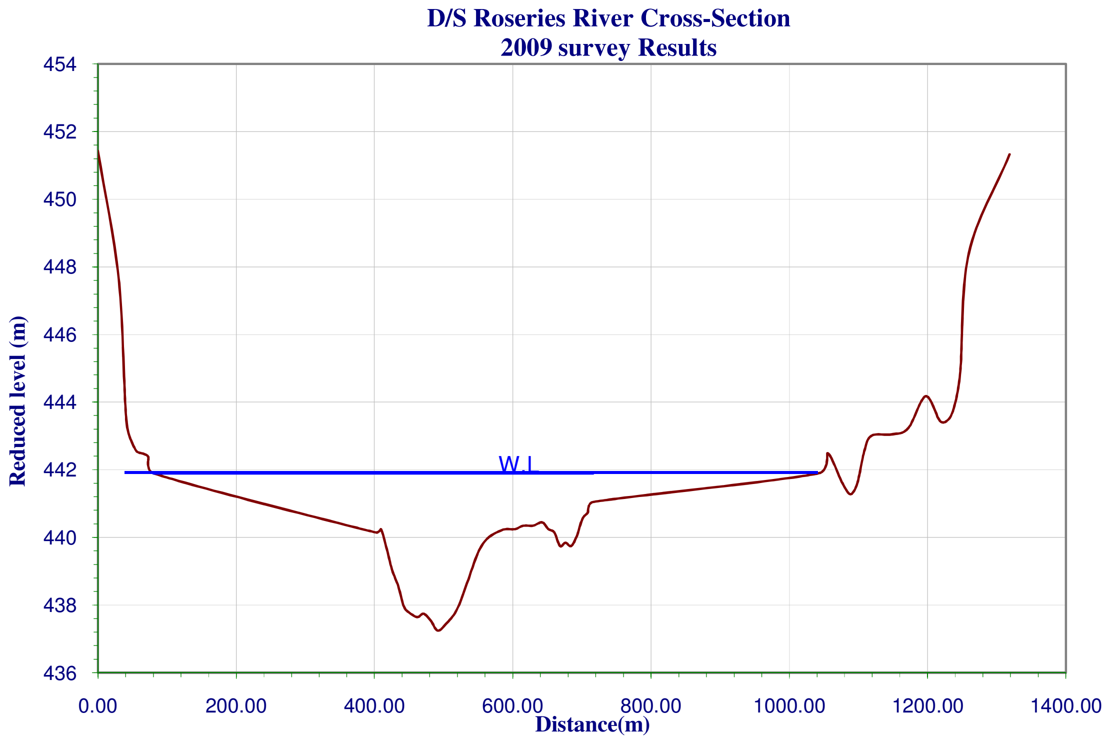


Figure 7 D/S Rosaries River Cross-Section

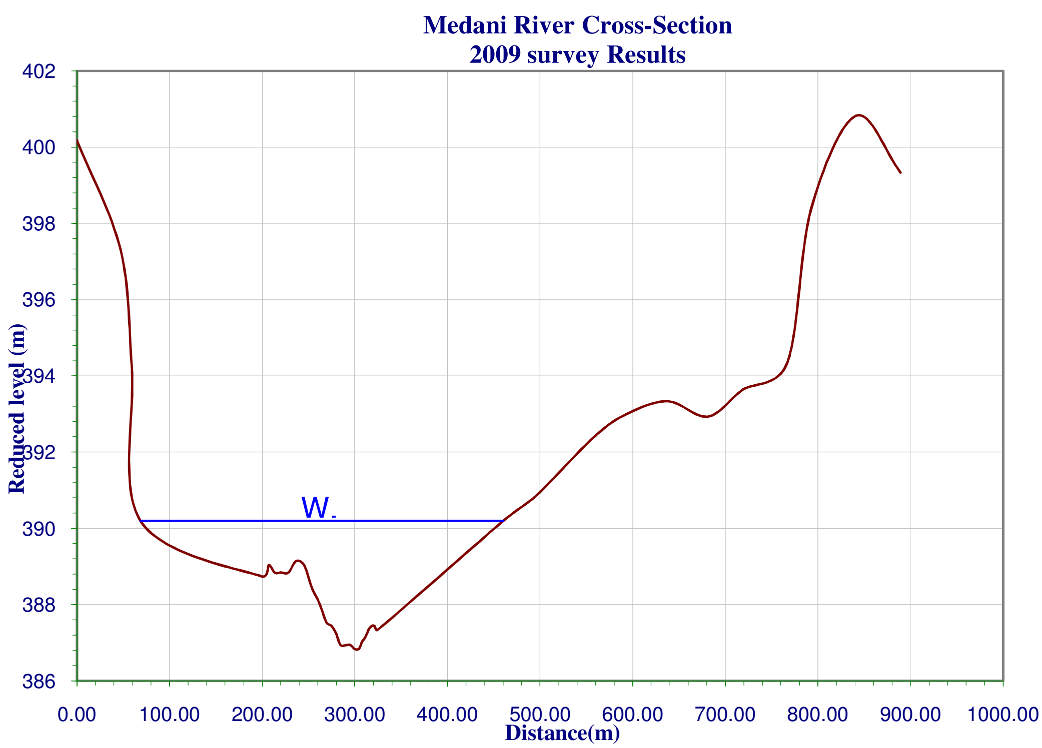


Figure 8 Medani River Cross-Section

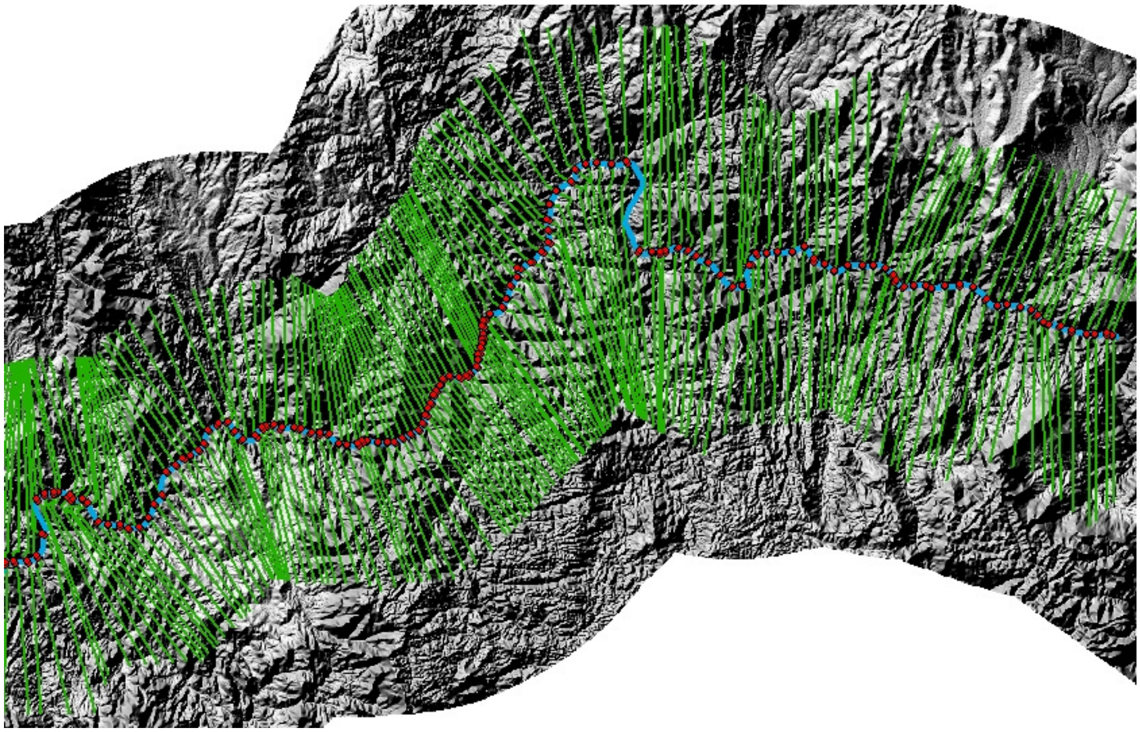


Figure 9 Cross sections overlayed on the hybrid raster data with bank points

7.2.2 Roughness Values

Manning's n values were used in the model to define roughness for each cross section. The n values are assigned based on land-use characteristics for common areas throughout the watershed. In general, each land-use characteristic was given an n-value based on published values for similar conditions (Chow, 1959), and on engineering judgment. For this study ENTRO-RTi calibrated n values used for the Blue Nile reach (from Eddeim to Khartoum) which varies in order of 0.04.

8. Scenarios setting

When setting breach scenarios, it is important to understand the type of dams and their quality of construction, and their location regarding the seismic zone. The heightening of Rosaries dam (central Cocnret and left and right side rockfill dam) nearly after 40 years indicates the good quality of construction at that time and now heightening being done according to the latest technology (design and construction). GERD dam (central RCC and left side rockfill dam) is being constructed following strict international standard and with state of the art design and construction method. These three dam are also located in least seismic impact zone. It is also known that the three dams have been equipped with sufficient spillway to pass PMF without overtopping.

Considering the above condition, failure mode considered in this study is through piping of the rockfill dam part of GERD, Rosaries and Sennar (earthfill). Further it is assumed that breach occurs in a sunny day while the incoming peak flood at GERD is about 10,400 m^3/s with return period of 50 years while all reservoirs is full.

Dam Breach modelling exercise will be done based for three dam configuration cases using HEC-RAS model considering the following three scenarios:

Scenario 1: Sennar breach other operate normally

Scenario 2: Scenario (1) + Rosaries dam breach (earthfill part)

Scenario 3: Scenario (2) + GERD Breach (Rockfill dam)

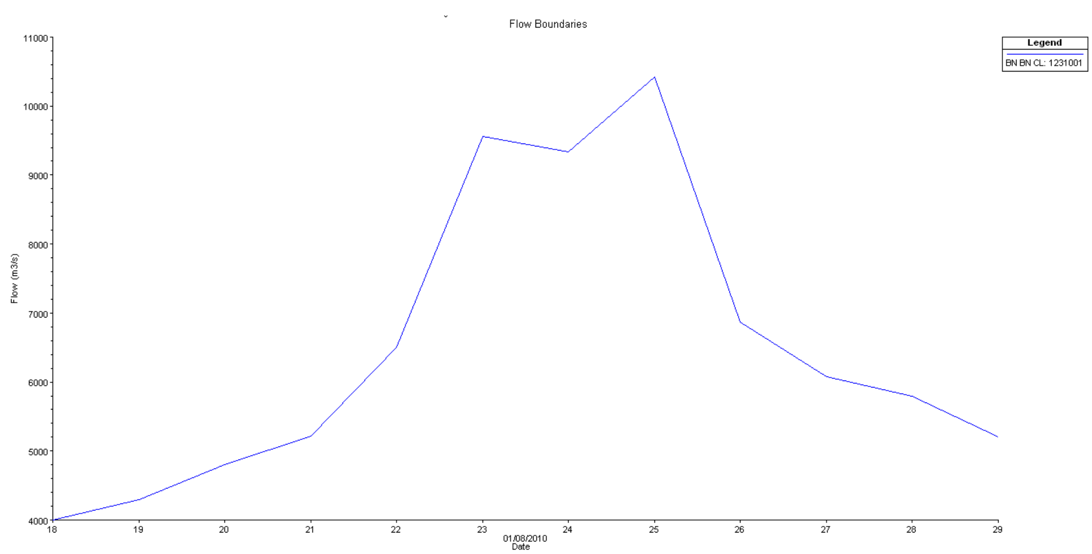


Figure 10: 50-year flood hydrograph entering into GERD during breach time

The flood affected zones will be assessed from d/s GERD to Khartoum Town. Existing HEC-RAS model established for flood early warning system hydraulic modeling of flood) made by ENTRO-RTI has been enhanced to include breach modelling and modified to be applicable to GERD-Khartoum Blue Nile reach by extending the existing cross-section limit (10 km band) to about 40 km band. Moreover the routing reach has been extended up to Mandaya dam site.

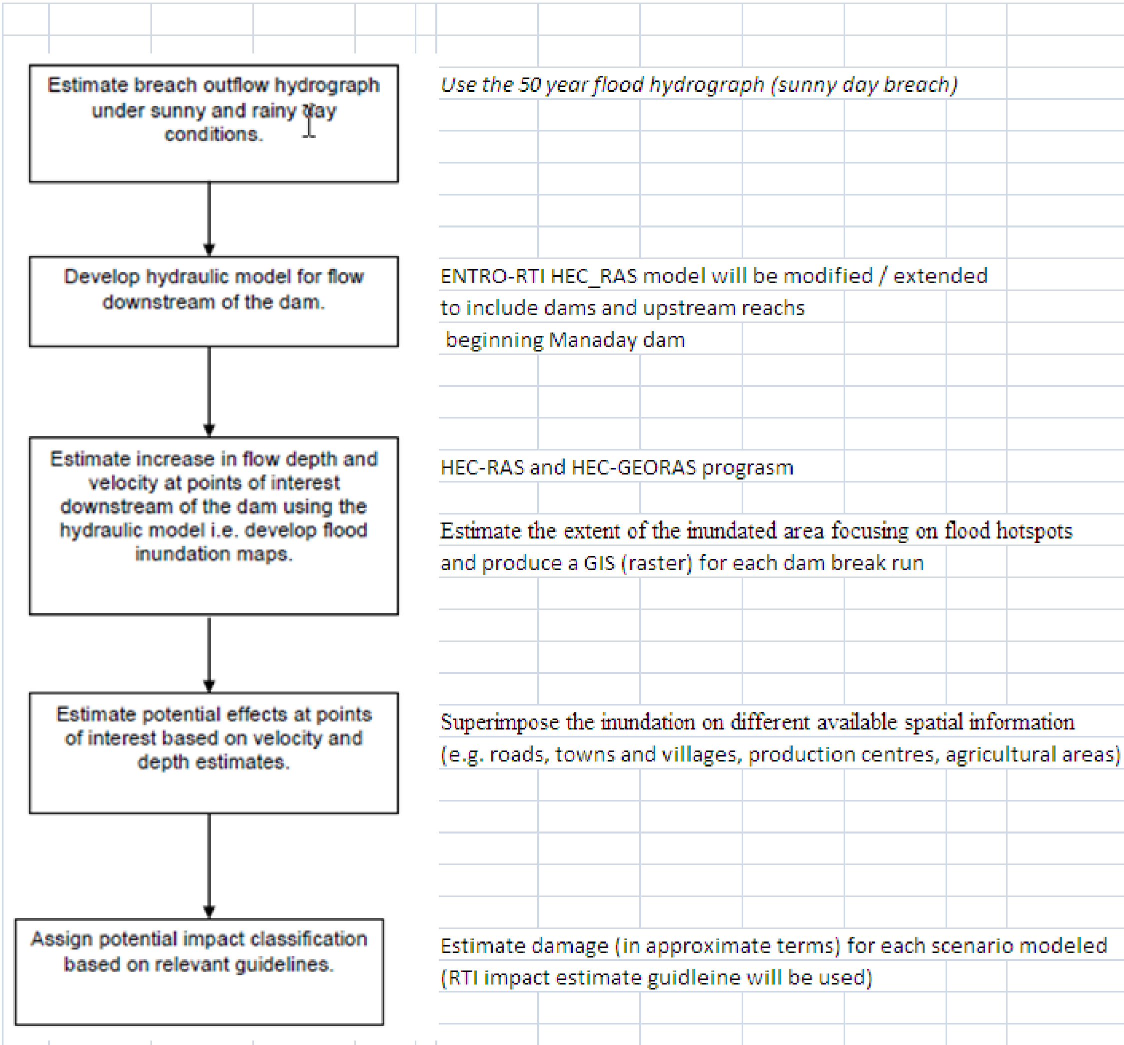


Figure 11: Work flow chart

9. Predictions of breach parameters

In order to simulate a dam failure through the HEC-RAS environment, this requires a set of dam breaching parameters. The parameters needed for the HEC-RAS dam breach model are breach shape, breach width, time to failure, pool elevation at time of failure, and breach side slope. To conduct this analysis, information about reservoirs storage volume and surface area and height of water were provided for each reservoir in the study area (Table 4). Table 5 and Table 6 summarize the estimated dam breaching parameters obtained from different prediction methods

Table 4: Summary of Reservoir Characteristics

Inline Structure	**Reservoir volume at time of failure Billion m^3**	**Reservoir surface area (km^2)**	**Depth of water at time of failure / Rockfill / earthfill dam height exposed to piping failure (m)**
GERD	52	1,681	40
Rosaires Dam	6	578	29
Sennar Dam	0.4	150	16

Table 5: Summary of Average Breach width

Inline Structure	**MacDonald and Langridge-Monopolis 1984**	**Froehlich 1996**	**Von Thun and Gillette 1990**	**Bureau of Reclamation 1982**	**Remarks**
GERD	5259	1057	167	135	Renaissance Dam fails under the piping mode
Rosaires Dam	798	656	127	87	Piping model considered alone, when combined with GERD overtopping mode
Sennar Dam	145	241	94	48	Piping model considered alone, when combined with GERD overtopping mode

Table 6: Summary of Breach Timing in hours

Inline Structure	MacDonald and Langridge-Monopolis 1984	Froehlich 1996	Von Thun and Gillette 1990	Bureau of Reclamation 1982
GERD	16	54	1.2	1.5
Rosaires Dam	7	19	0.8	1.0
Sennar Dam	2	7	0.6	0.5

Results of breach parameters show that MacDonald and Langridge-Monopolis 1984 equation given the upper boundary of predicted values of the breach width (Br) for all cases of failure scenarios while lower values of Br were given by the Bureau of Reclamation 1982 equation. The predicted time of failure when using Froehlich 1996 method gives the longest time required for breach formation for all dam failure cases, while shortest values of τ were given by Bureau of Reclamation 1982 equation.

Results indicated that Reclamation and Von Thun and Gillette equations tend to produce the smallest values, this might due to the sensitivity to dam depth hd which is very small comparatively with the reservoir storage Vw in the case in Rosaires and Sennar cases. (Froehlich) and (MacDonald and Langridge-Monopolis) equations predicts significantly larger breach widths Br because the equations relates breach width to an exponential function of both the reservoir storage Vw and reservoir depth hd. Largest breach width values that produced by MacDonald and Langridge-Monopolis method is invalid at describing a dam breach particularly at Renaissance and Rosaires dam sites because the breach bottom width suggested by the method is far greater than the bottom width of the natural stream channel which is about 200 m. Longest values of time of failure were given by Froehlich can produce the most realistic case scenario in representing the erosion process through the earthfill / rockfill dam part. Therefore Froehlich is adopted to predict predicting of the time breach formation τ. In this study breach parameters estimated from USBR 1982 method are adopted.

10. Result and recommendations

There are useful outcomes of the study as indicated below:

- Extended Blue Nile river cross section upstream (from Mandaya dam site to Rosaries dam) over ENTRO-RTi hydraulic model which was done from Rosaries to Khartoum, although the accuracy is limited

- Terrain models for channel and floodplain of the Blue Nile extended from Mandaya dam site to Rosaries dam

- Geo-referenced Blue Nile hydraulic model and dam breach model configuration in HEC-RAS environment for 3 dams (GERD, Rosaries and Sennar)

Initial model results should be seen considering the following limitations of the model:

Survey and Terrain Modelling:

River surveying was limited to specific pilot reaches of the Blue Nile as was done by RTi. Floodplain areas were characterized using a 90-meter DEM which gives inaccurate results in estimating depth of flood in the floodplain. The river is subject to continuous changes in channel geometry due to its high sediment load.

Hydraulic Modelling:

As was also discussed by RTi, model results in Khartoum are sensitive to the downstream boundary condition, about which there is some uncertainty.

River and flood plain Manning roughness coefficient estimates are very approximate which will create uncertainty on flood depth, magnitude and velocity.

Breach Modelling:

Breach width and progress is approximate due to inadequacy of existing models to model breach phenomena.

10.1 Breach flood hydrograph propagation and water surface variation

Figure 12 summarise initial breach flow hydrographs (Scenario 3) at GERD, Rosaries, Senar and Khartoum. It is seen that peak breach discharge downstream of GERD is 27,203 m^3/s which is comparable to peak value of PMF at GERD / Rosaries. Figures 13 to 20 show watersurfae level in selected Blue Nile cross section and discharge along the reach. The flood peak attenuate to 17,411 d/s of Rosaries, to 16,855 d/s Sennar, to 14, 580 at Khartoum. It takes about 250 hours (10 day) for the peak flood to arrive at Khartoum. It is to be noted that estimate of the peak hydrographs at different locations are constrained by the difficult in having reliable cross section, Manning roughness of channel and over the flood plain. In case breach occurs in dry months the peak flood will drop by about 10,000 m^3/s.

For Scenario II, in case of only Rosaries and Sennar breaches, the peak breach flood peak is 17,411 m^3/s and 13,350 m^3/s d/s of Sennar (Figure 21).

For Scenario III, in case of all three dam breaches in dry periods (peak inflow 2100 m^3/s) the breach flood magnitude are reduced from 27,203 m^3/s to 24,641 m^3/s and the peak flood d/s Rosaries is 17,381 m^3/s and at Sennar is 15,932 m^3/s and about 13,500 m^3/s at Khartoum (Figure 22). In 1975 peak flood of 13,090 m^3/s was observed at Khartoum, where there was no significant regulation of flood upstream (Rosaries was not raised, no GERD).

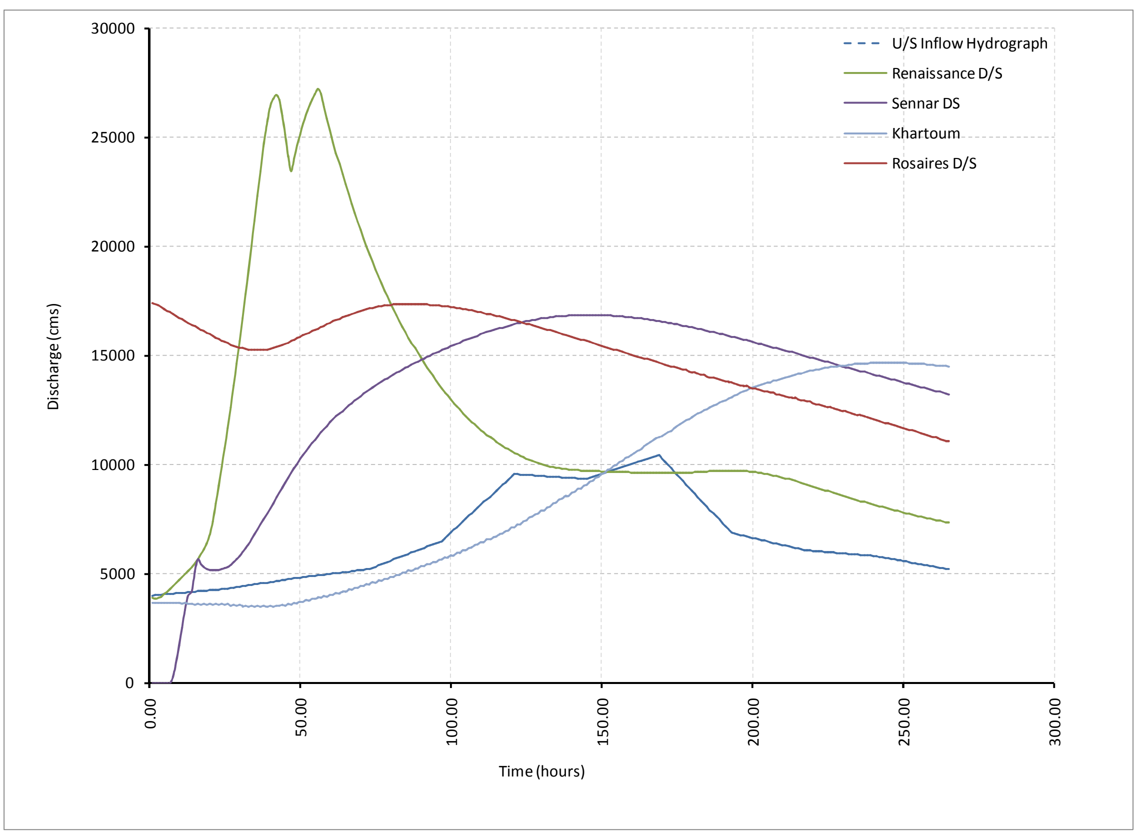


Figure 12: hydrographs (breaches and inflow) at various locations

(three dams breach case – Scenario 3)

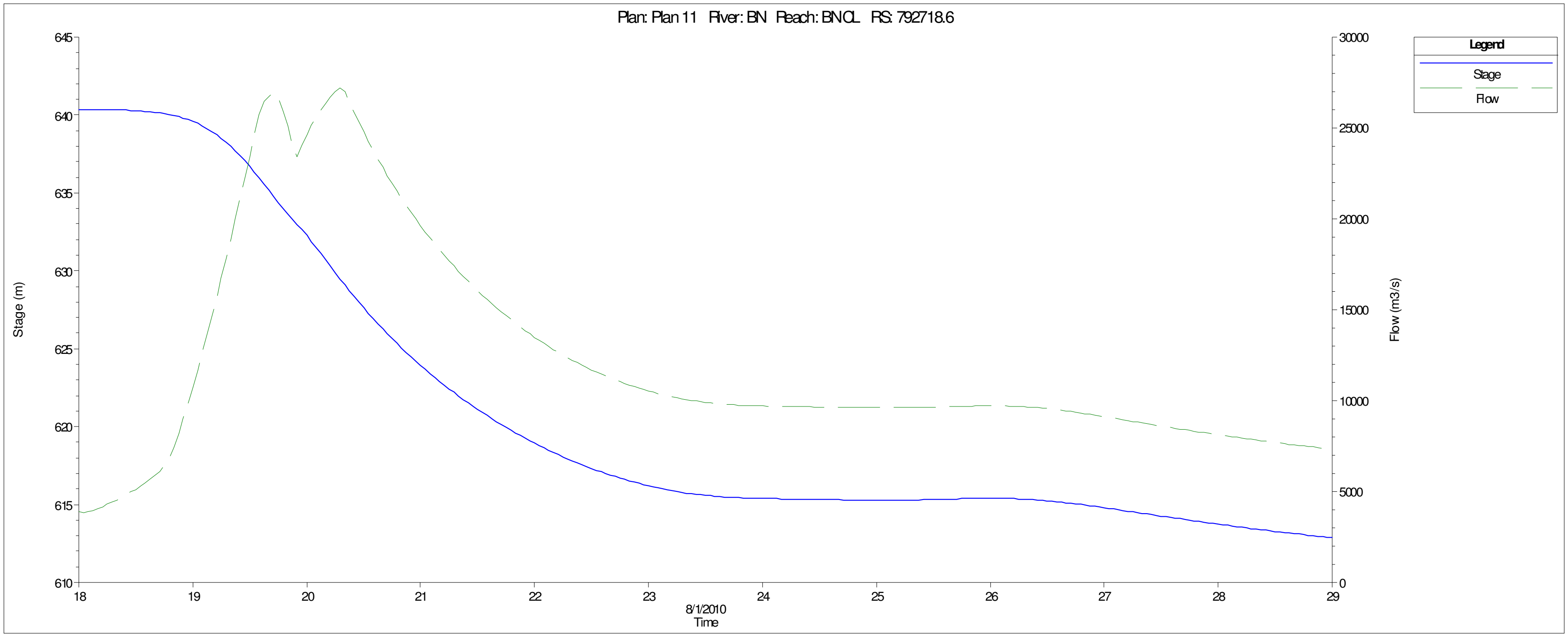


Figure 13: Breach hydrographs d/s of GERD

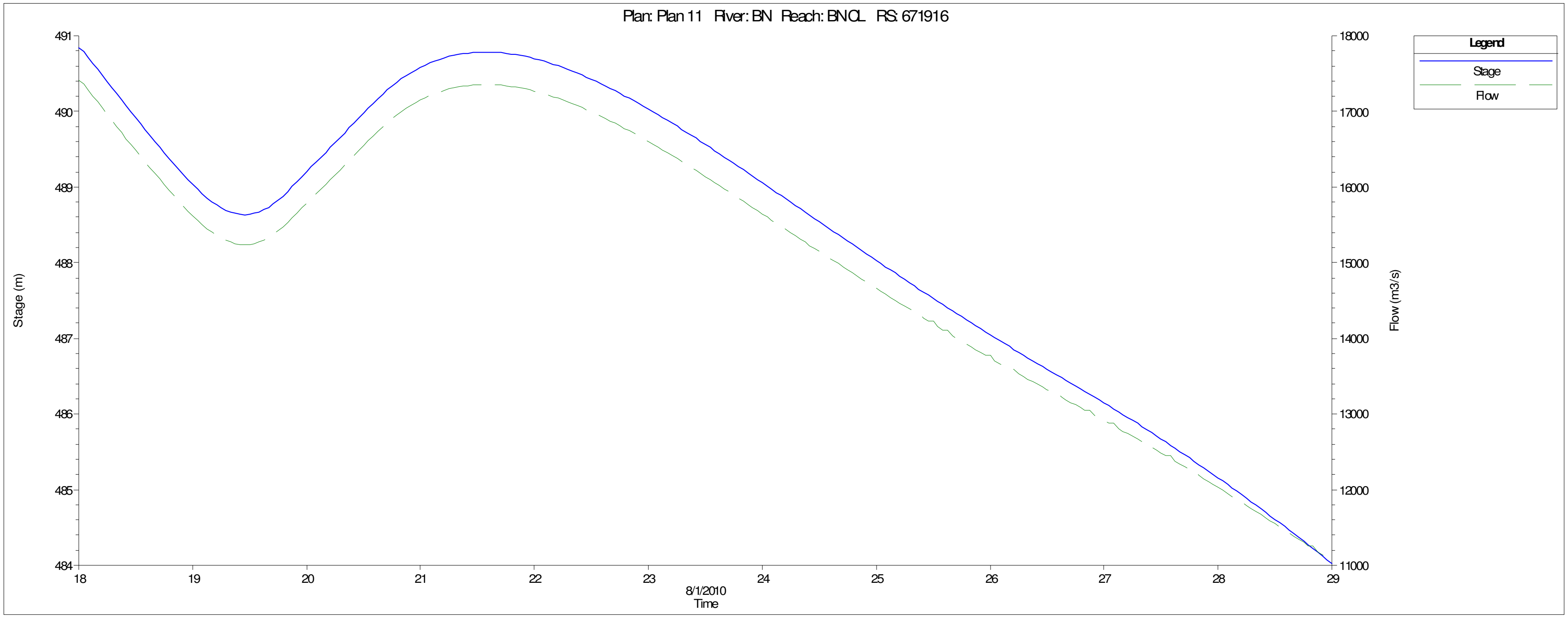


Figure 14: Breach hydrographs at d/s Rosaries

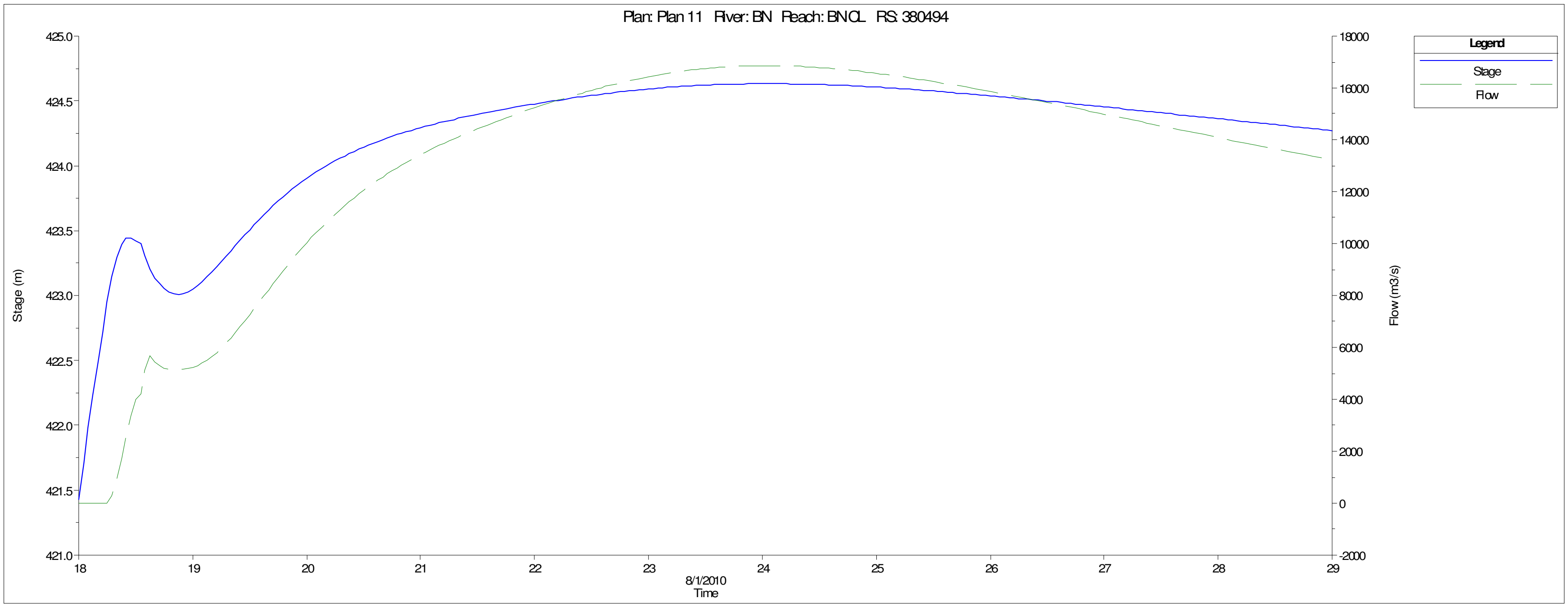


Figure 15: Breach hydrographs at d/s Sennar

Figure 15 through Figure 18 show HEC-RAS output of the flood water surface profile at different time along Blue Nile reach.

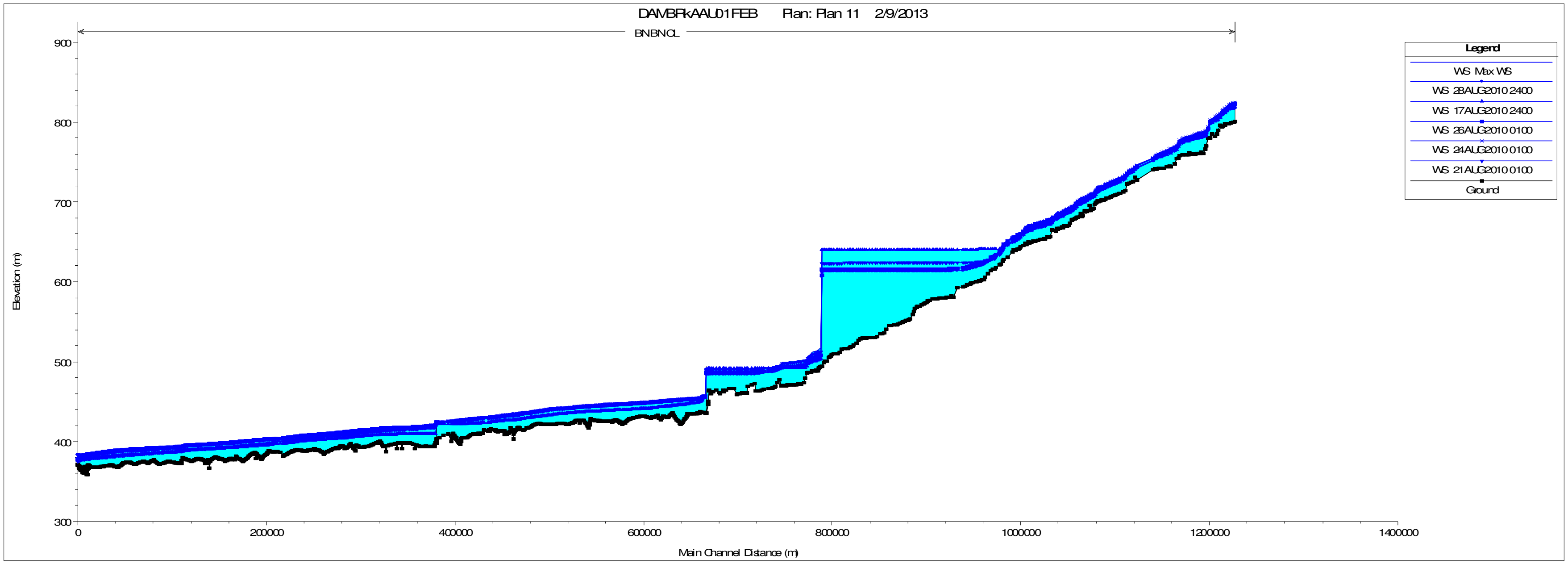


Figure 16: **Water surface profile during breach along Blue Nile reach**

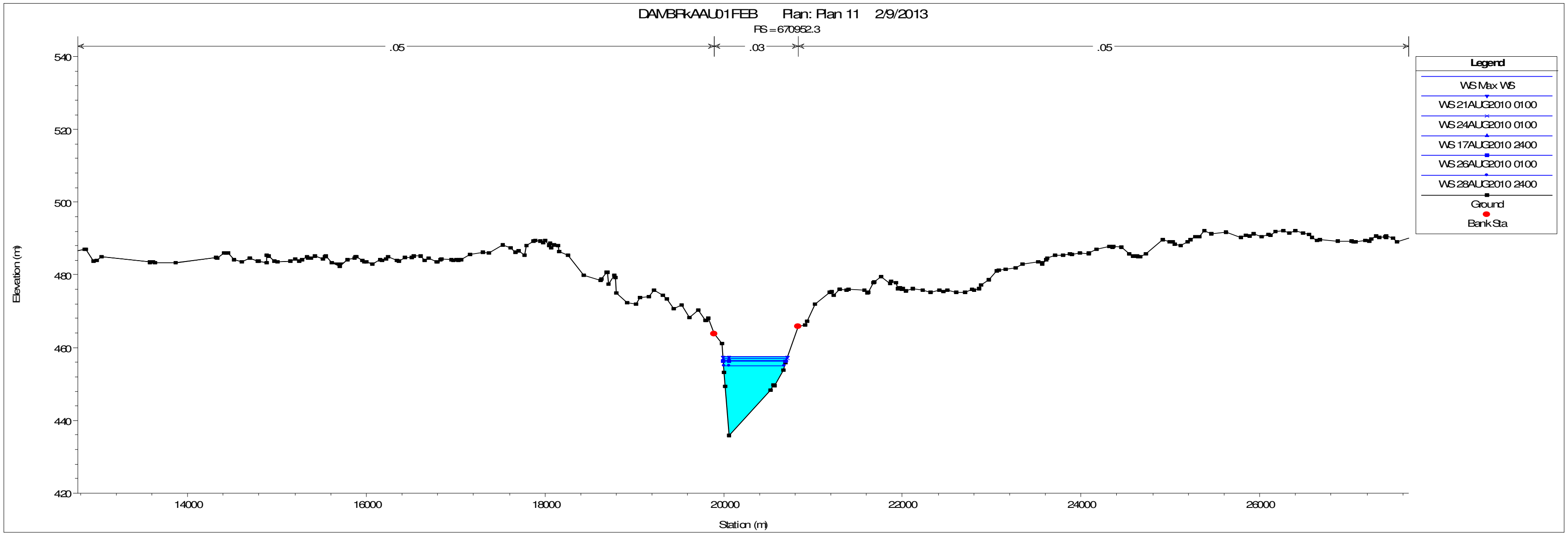


Figure 17: WS variation with time at d/s of Rosaries

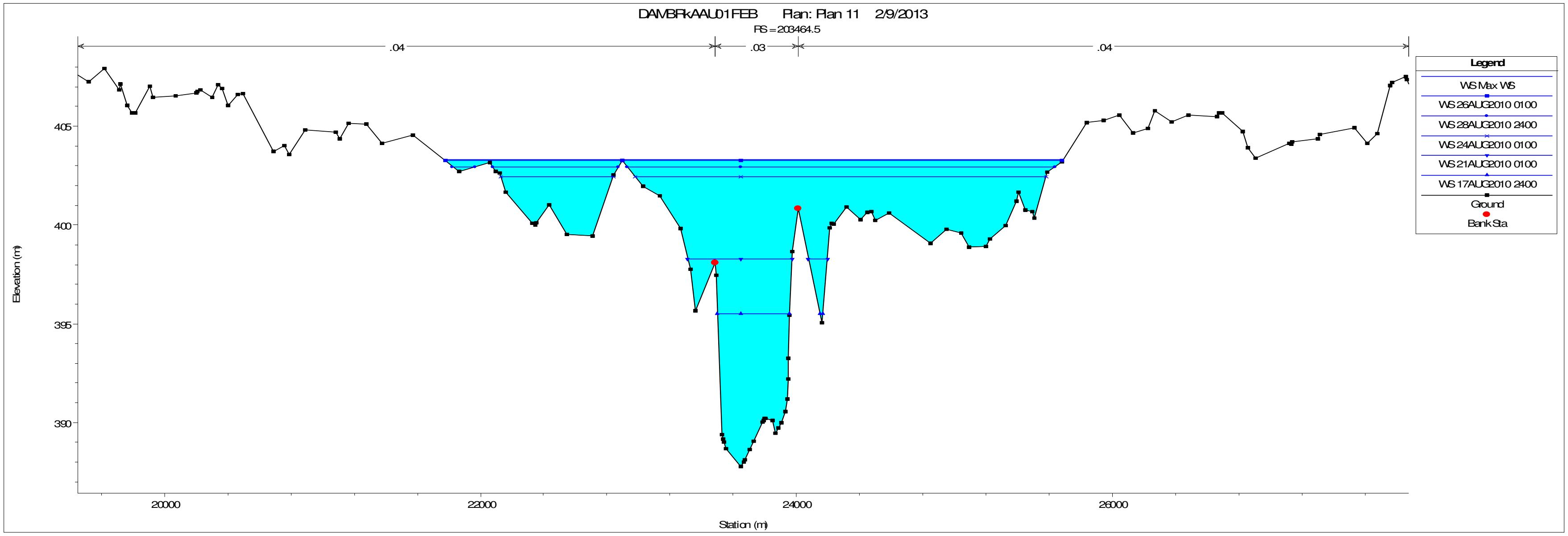


Figure 18: WS variation with time at Wad Medina

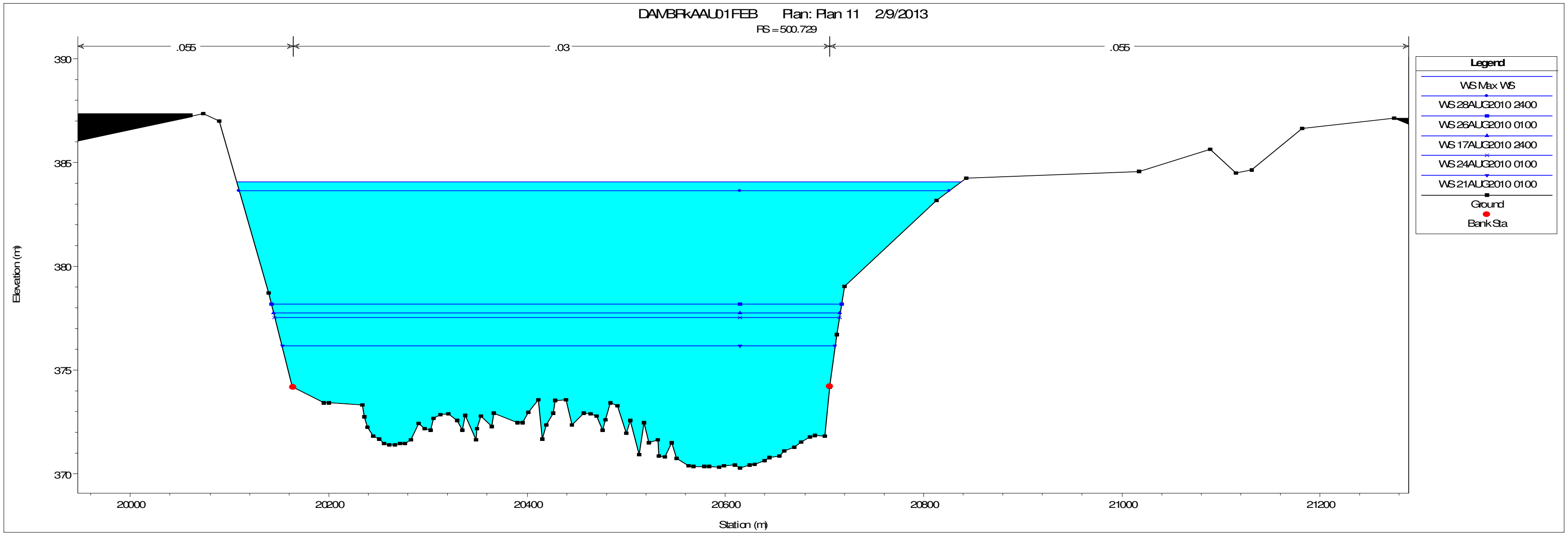


Figure 19: WS variation with time at Khartoum RS 500.729

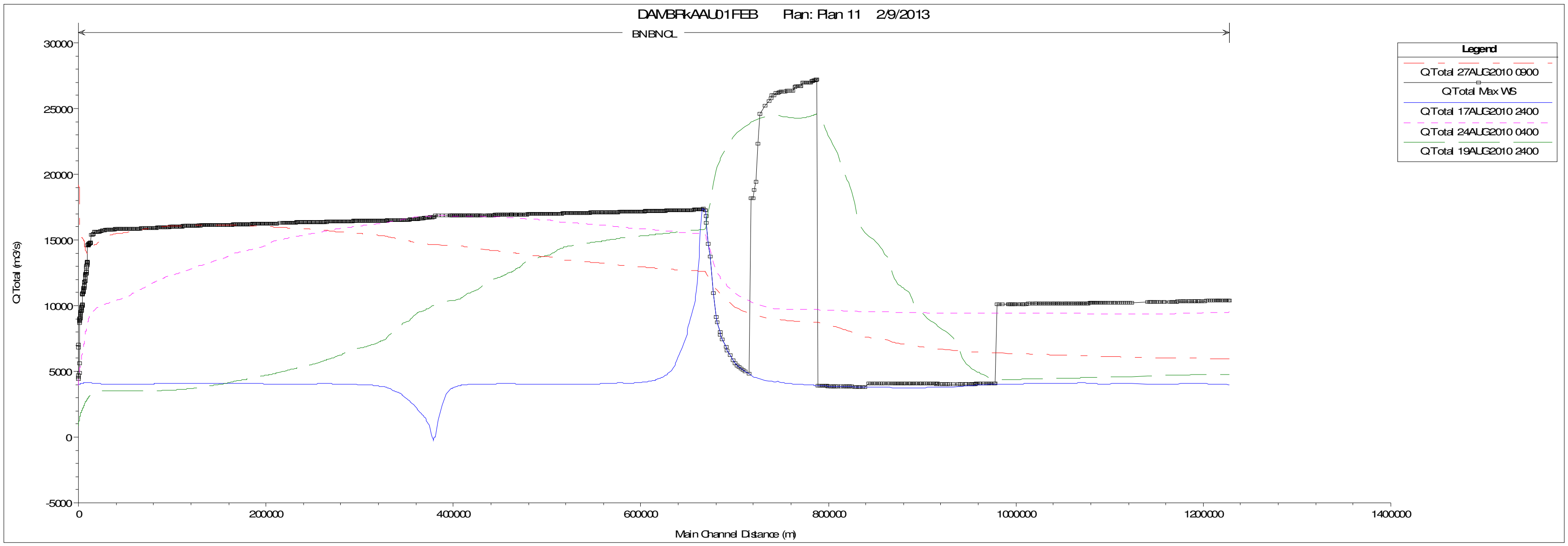


Figure 20: Maximum discharges along the Blue Nile reach during breach from Mandaya to Khartoum (RS 0) – Scenario III.

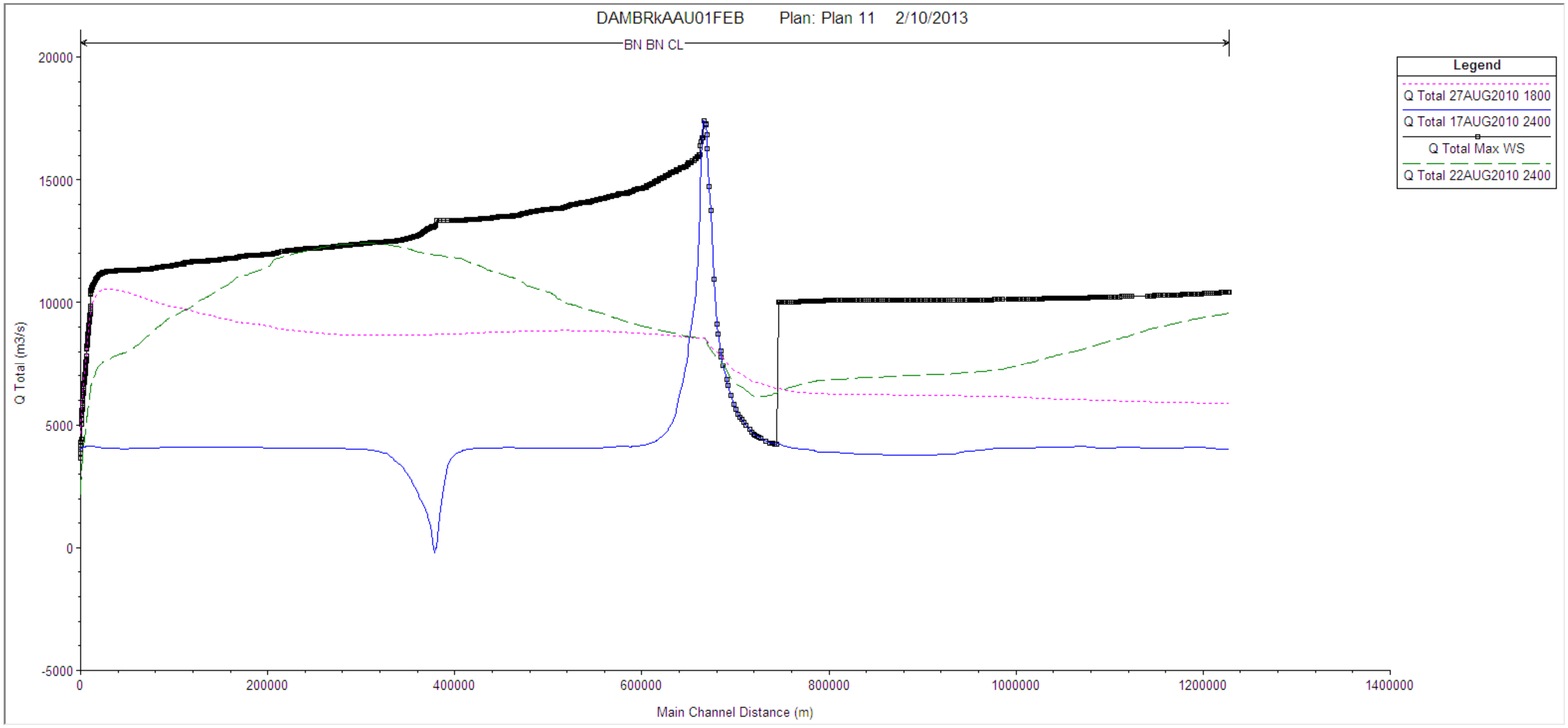


Figure 21: Maximum discharges along the Blue Nile reach during breach from Mandaya to Khartoum (RS 0) – Scenario II.

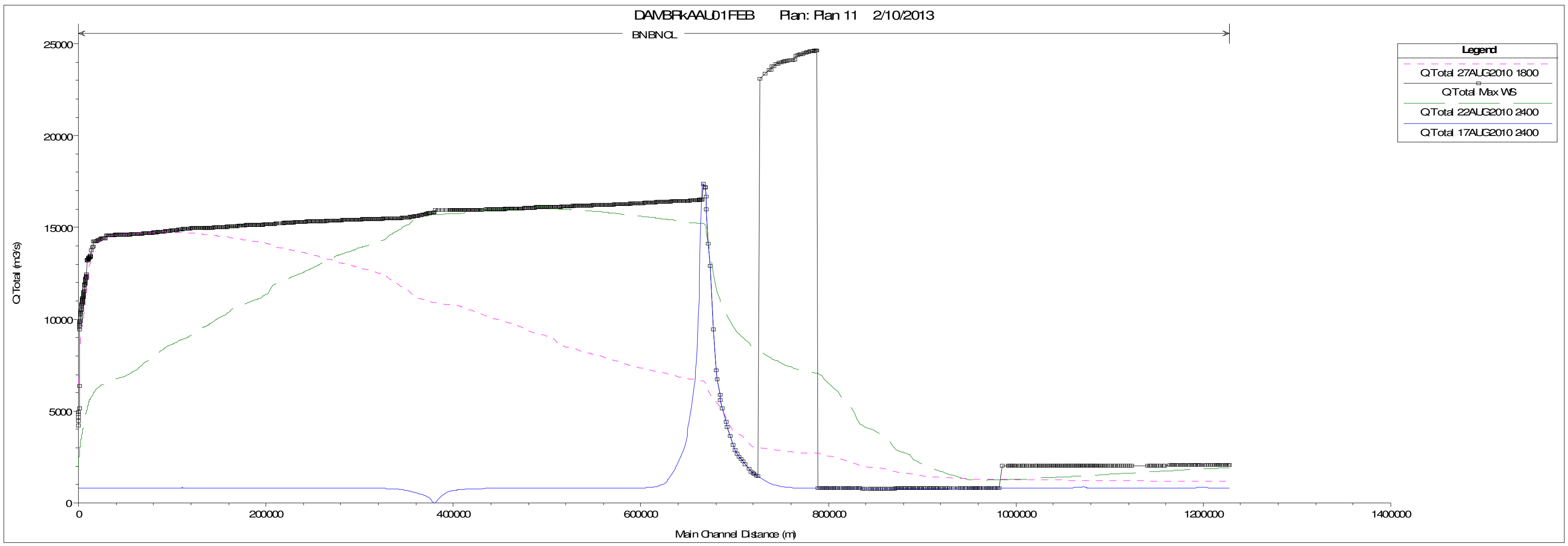


Figure 22: Maximum discharges along the Blue Nile reach during breach from Mandaya to Khartoum (RS 0) – Scenario III with breach occurring in dry period.

10.2 Flood extent, depth and velocity mapping

Estimate of the area of flooding, depth and velocity in flood plain become grossly unreliable as the cross section extension in the present study was based on 90x90 DEM and no other survey measurement was possible in this scope of study. Due to the above reason, all attempts made to produce realistic flood extent, depth and velocity map were unsuccessful. It is recommended that for more reliable works, acquisition of accurate topographic map including all assets (with vertical accuracy about 0.5 m) of Blue Nile reach in about 40 km band is recommended.

In conclusion, the result of this report shall be considered very preliminary, due to limitation of the extended detailed survey data (topography and property) and complexity of modelling river and flood plains with uncertain breaching process for more than 900- km long stretch in short period.

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Annex I- Typical reach from Mandaya to Khartoum along Blue Nile

Reach between Mandaya and Rosereis dam is characterized deep gorge with rock, bolder and gravel river bank and bed only widening when approaching the Sudanese boarder, with n value taken about 0.04. Riverine forest is also common.




The reach between Roseires and Sennar is characterized by clay plain soil and rain-fed agriculture with a meandering river, and is dominated by horticultural activities (banana plantations, Mangos, Gwafa and citrus). A value of 0.03 to 0.05 was cited in the literature for field crops. In this study, a value of 0.030 was used for the channel and 0.050 for the flood plain. For Sennar Dam Reservoir, the same n values as for the Eddiem to Roseires reach was used Rosaries-Nenar reach (RTi, 2010)

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The reach between Sennar and Medani is characterized by light brush, dark clay agriculture activities, grazing lands and some trees. Selected n value for the reach ranged from 0.030 to 0.060 and varied throughout the reach (RTi 2010)

The reach between Medani and Alnuba is characterized by a meandering channel with vertical accretion and formation of terraces, cultivation of fodder, and with sunut forest. n values from the literature for these conditions range from 0.03 to 0.05. A value of 0.30 was used in the channel and 0.040 in the floodplain for this reach (RTi 2010).