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Numerical Modeling of Dam Failure Phenomenon Using Software and Finite Difference Method

Somaye Soleymani[1], Hamidreza Golkar*[2], Hamzee Yazd[3], Mojtaba Tavousi[4]

[1]*Department of water engineering, Ferdows Branch, Islamic Azad University, Ferdows, Iran;*
[2]*Department of Planning, Regional Water Company of Khorasan Razavi , Mashhad , iran*
[3]*Department of water engineering, Ferdows Branch, Islamic Azad University, Ferdows, Iran*
[4]*Department of water engineering, Ferdows Branch, Islamic Azad University, Ferdows, Iran*

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[*]*Corresponding Author. E-mail: golkar.hr@ferdowsiau.ac.ir*

Abstract
Dams are of manmade engineering structures that are directly related to water and its destructive forces. Despite all remarkable advantages of these structures, they exposed to threat of destruction and failure. This causes increasing risk potential at dam downstream. This paper studied possible failure phenomenon of Bidvaz dam, Esfarayen, through Mike 11 software and using definite difference method by the help of aerial photographs, topographical maps and AutoCAD and global mapper software. Moreover, regarding the crack (breach) created in the aforementioned dam, the break and failure place is the balance of the created breach. The research results and peak discharge of dam failure following 3 hours and 34 minutes and 26 seconds of dam failure seen around Esfarayen city. Furthermore, level of peak discharge of dam failure in city surrounding was 4861.9 m^3/s indicating the 2.47% reduction in failure peak discharge from dam to city area. This percentage seems reasonable according to the high steep river as well as river low width resulted by flood plains.

*Keywords***:** Dam failure, peak discharge, numerical modeling, Mike 11 software

1. Introduction
According to large financial and life damages of dam failure, this phenomenon considered as one of the most devastating disasters. In spite of all advantages of dams in flood controlling, fisheries, supplying of drinking water and agriculture, power generation, and increased quality of water resources, they are exposed to threat, destruction and failure like all manmade structures. This increased risk potential at dams downstream. Studies show that dam failure can create larger discharge even heavier than the predictable raining at dams' downstream. Predicting the flood wave (flood water) resulted from dam failure and the time of approaching to the site downstream is of critical hydraulic issues.
In this research, according to reports of instrumentations and expert observing of dam downstream, an approximate one meter deposition seen at upstream dam roof, which increasingly intensified up to where the issue of Bidvaz dam failure turned into a critical, strategic issue. Therefore, this paper studies failure phenomenon of this dam and the resulted flood zoning and its consequences. Several solutions of dam failure introduced and a numerical model presented through finite difference method. The numerical model numerically solves unsteady streams equations in one-dimension.
The research seeks for achieving the objectives such as studying failure phenomenon of Bidvaz Dam, Esfarayen, and its consequences, studying flood depth, velocity, and maximum discharge at the entrance of Esfarayen city. Thus, the following questions raised to meet the aforementioned goals: where is the hole (crack) placed and at which level it occurred? What is the maximum discharge crossing failure section? When the peak discharge

resulted from dam failure approaches to Esfarayen and in what volume? What is the maximum water depth around Esfarayen city and what is the width of the 25-year flow in this area?

2. Research history and related works

In general, damages caused by dam failure flood are not only limited to flood depth, but also depend on other factors such as flow rate. One proper methods of considering speed parameter is using risk matrix method. Du plessis (2000), Adriaans (2001), Stephenson (2002), Ross (2003), Fatorrelli et al (2003), and Vrouwenvelder et al (2003) studied the effect of flow rate on dam failure consequences [2]. They suggested various risk matrices by integrating speed parameter with any parameters of flood depth, probability of dam failure and or the percentage of flood damages.

Wood et al [7] experimentally and numerically studied dam failure flows in canals with 90 degree bend. She conducted this study with 2-dimensional equations (SWE).

Aureli et al [8] experimentally and numerically evaluated hydrodynamic force and load imposing on dam and resulting in to failure through using two- and three-dimensional mathematical models.

Kocaman et al [9], in a study, investigated dam failure caused by effective shocking waves on a vertical wall through empirical testing and CFD simulation based on VOF.

Abbasi and Ismaili [10], in modeling dam hydraulic failure resulted from sudden flood by Fluent software, examined dam failure phenomenon in one-dimension condition and two states of flood and failure and failure with no flood through using Fluent software. Studies demonstrated that the effect of applied network on results is much more important than time step.

Bani hashemi et al [11], in the 2-dimension dam failure model based on Fred and Mac Cormac, showed that combining the two methods of Fred and Mac Cormac with second order accuracy and adding artificial viscosity is able to model dam gradual failure and resolving of shock flow despite its simplicity.

Abareshi et al [12], in "risks of Taragh dam failure", studied Taragh concrete dam failure in the vicinity of Mashhad city by using MIKE 11 software in which areas at the risk of flooding identified by combining flood speed and depth maps through risk matrix; further, each area risk rate determined, too.

Toufani and Ahmadi [13], in numerical modeling of dam gradual failure, presented modeling method of dam gradual erosion by an initial gap on the crown to start the erosion. Finally, model results compared with the results of a laboratory sample and a field test. The provided charts represent a relatively good consistence.

Seiyfi zade et al [14] studied PolRud dam failure caused by passing and flood routing in downstream through using BREACH GUI model. Flood routing performed using HEC-RAS model in order to determine dam failure effects.

Mohammadnezad et al [15] simulated dam failure and the resulted wave diffusion through using limited volume numerical method in right two dimensions.

Montazeri naming et al [16], in simulating the wave of Maronik Dam failure, introduced a one and two-dimension numerical model for simulating dam failure flooding.

Godariz et al [17] presented function of earth dam failure risk analysis due to body overflow regarding uncertainty of some input parameters.

Numerical models are known as one of the strong means of evaluating dam failure flood. Now, there are various models of studying dam failure phenomenon. Table 1 shows developed numerical models of dam failure around the world.

2.1.Studying findings of dam failure, following results obtained:

The highest dam failures belonged to small dams.

The highest failures belonged to the new constructed dams. It is interesting that 70% of dam failures occurred within the first ten years of utilization.

The major failure actor in concrete dams was foundation.

Table 1: Dam failure numerical models

Raw	Model	Owner
1	DAM BRK	USA/National weather service
2	SMPDBK	USA/National weather service
3	BOSS DAMBRK	BOSS International
4	HAESTED DAMBRK	HAESTED METHOD
5	UKDAMBRK	Binnie&Partner
6	DWAFDAMBRK	Department of water affair and forestry Pretoria
7	HEC Program	USA-COE
8	LATIS	Tams
9	DBK1	IWHR-China
10	DBK2	IWRH-china
11	TVDDAM	Royal Institute of technology-Stockholm
12	RUBAR3	Cemagref
13	RUBAR20	Cemagref
14	CASTOR	Cemagref
15	SOBEK	Delft hydraulics
16	DELFT2D	Delft hydraulics
17	DYX10	Reiter consulting engineers
18	DYNET	ANU-Reiter
19	RECAS	Enel centro di ricerca idraulica
20	DYNET	ANU –rieter
21	MIKE	DHI
22	FLORIS	ETH zurich
23	TELEMAC	EDF

In earthen (Embankment) and rockfilldams, the first failure factor was dams' overflow, erosion of dam body, and weak foundations.

Dam overflow along with internal erosion reported as the major failure factor in building dams.

Saint-Venant equations and MIKE numerical model

One-dimension equations of unsteady flow in open channels referred as equations. The equations consisted of two individual equations of continuity and momentum.

Almost important phenomena of hydraulic flow in waterways have unsteady flow. In such condition, the flow depth or speed changes in term of tome. Flow condition at flood time is one instances of unsteady flow. Gradual, unsteady flows well stated by flow one-dimension differential equations in the form of Saint-Venant. The equations are as follows.

$$\begin{cases} \dfrac{\partial Q}{\partial x} + \dfrac{\partial A}{\partial t} = q_L \\ \dfrac{1}{g}\dfrac{\partial v}{\partial t} + \dfrac{\partial v}{\partial x}\left(\dfrac{v^2}{2g}\right) + \dfrac{\partial y}{\partial x} + S_f = 0 \end{cases} \tag{1-3}$$

The following questions answered by the aid of Mike software: in which areas the flood occurred and what is water balance exceed level at flooding? What are long-term environmental effects of changes in pollution load?

Where sediments deposit in the river and how are the river general morphological changes? What is the maximum pollutants concentration in some particular points following urban or industrial pollutants entrance?

Of the existing models of Mike software, hydrodynamic model (HD) module as the main core of software is the basis of almost other modules including flood prediction (FF), diffusion and transfer (AD), water quality (WQ) and sediment transfer (ST).

MIKE 11 hydrodynamic module implicitly solves continuity and momentum equations by using numerical method of Abbott-Ionescu and finite difference method in unsteady state and conducts the calculations of unsteady flow at estuaries, lakes, and reservoirs.

Solving Saint-Venant equations by finite differences method requires all unknown of a selected time step determined in the previous time step. Unsteady mode equations may not be solved for the first time step and must be identified with primary requirements for the model.

Flow characteristics measured according to the provided border conditions and based on the permanent flow equations. The estimations are later used as the primary requirements. It suggested that if the information is not adequate, the primary requirements of initiating the calculations assigned to the program itself.

Two internal and external border conditions are defined in MIKE 11. Internal border conditions are indeed internal nodes and the start and end points of any river branches. Further, the points with hydraulic structures or lateral flow also defined as an internal border point. In addition to internal border points, external borders also selected for network measurements. These points are indeed system input and output points, which can be in one of the following forms:

Fixed h or Q values	h(x0,t) = h	Q(x0,t)=Q
h and Q values changing with time	h(x0,t) = h(t)	Q(x0,t)=Q(t)
The relation between h an Q like height- hydraulic conductivitycurve		h=f(Q) or Q=h(x0,t)
Where Q and h are the known values of h and Q		

2.2.Introducing understudied area and dam

Bidvaz Dam is located at 19 km of northeast of Esfarayen City at the longitude of 57° and 37′ and northern latitude of 37° and 05′. The dam is accessed through the main road connecting Esfarayen City to Ardoghan village. The nearest village to dam site is Denj village situated at about one km of dam axis around dam reservoir area. Figures 1 and 2 illustrate understudied area.

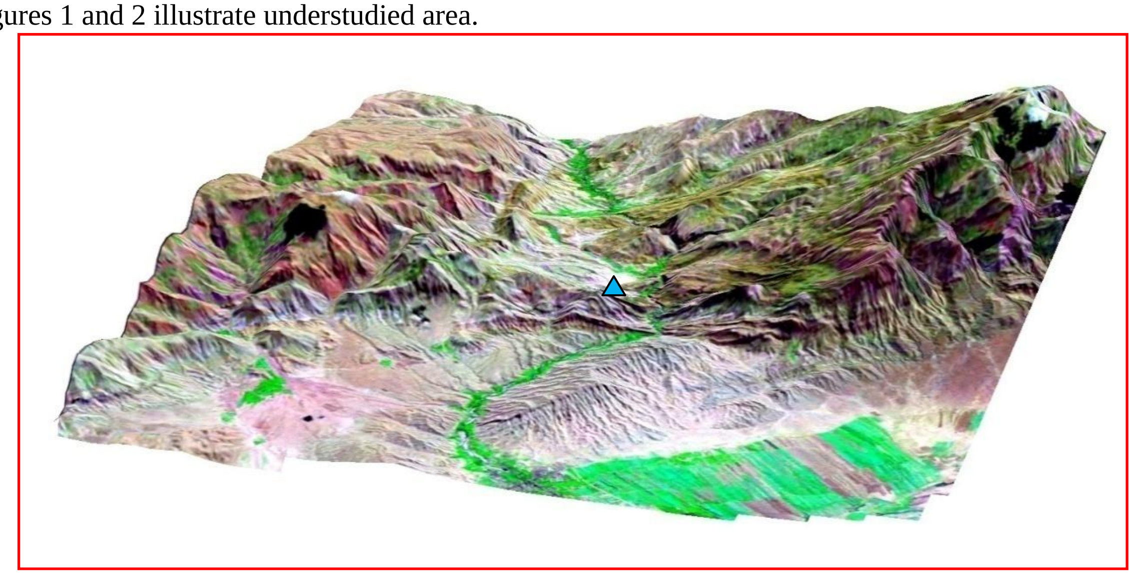

Figure 1: Dam site and understudied area

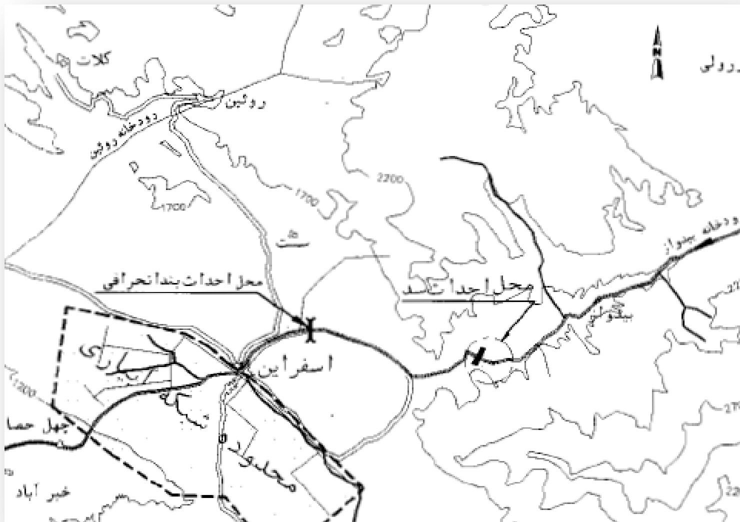

Figure 2: Dam site and understudied area

In term of river system, waterway path at the dam site axis originated from Shah Jahan Mountains in eastern Alborz mountain range; it initially flows in eastern-western direction and redirected along west south-east north once it joined to Parchin branch. It receives water flow of several sub branches and enters into dam axis passing Ardoghan, Denj and Hassan Abad Ab Pakhsh; finally, it entered to Esfarayen City and ended to downstream in Jovin River. Table 2 illustrates general characteristics of catchment area at dam axis site.

Table 2: Dam catchment area characteristics at dam axis site

River name	Bidvaz	Basin (km^2)	518
Basin area (km^2)	117	Equivalent rectangle length (km)	47.6
Equivalent rectangle width (km)	10.9	Basin average slope	3.75
Basin maximum height (m)	3084	Basin minimum height (m)	1300
Basin average height (m)	2117	The main channel length (km)	57.5

The constructed dam consists of various and heterogeneous sectors. The non-permeable section of dam main body is composed of an upstream-oriented clay core. The materials used here has low permeability around 10^{-7} Cm/sec placed at the 20 cm Shotcrete layer implemented on stone floor and modified alluvial deposit. The clay core at 1533 level (balance) has 5 m width and rightlycontinues up to 1519 level with 1 horizontal to 10 right slope. The width of clay core at this level is 7.8 m. At this level, the upstream slope of clay core of 1 horizontal to 1.4 right and downstream slope of 1 horizontal to 1.2 right extends up to stone or alluvial foundation level in any section. The maximum sectional width of clay core on alluvium equals 20.2. The clay core utmost height from river bed disregarding screen heightis 61 m. Due to unavailable adequate materials in the vicinity, the wide clay core excluded. However, designing inclined core and concentrating on filter granular designing, diameter as well as the recommended materials for the core, defensive predictions determined against the crack of different factors. The materials used in dam clay core were supplied by fine-grain materials in Denj village located at least 1 km from dam site. The amount of materials passing sieve 200 is in the range of 50-98% and the amount of clay grains at diameter less than 0.002 mm is within 18-20%. Material fluidity limit is from 20 to 38 and the pasty factor is in the range of 6-21. Experiments of preset sources demonstrate the divergence feature of these materials. Table 3 shows dam general characteristics and Figure 3 represents dam main components.

Table 3: Dam general characteristics

Dam reservoir total volume	52.9’000’000 m^3	Dead volume	21’000’000 m^3
Useful volume	31.9’000’000 m^3	Water level normal balance	1530 m from sea level
Maximum level of sedimentation	1506.2 from sea level	Annual average inflow to the reservoir	50.5’000’0000 m^3
Average flow of middle sub-basin	8.7’000’000 m^3	Drinking water	5’000’000 m^3
Water allocated to water rights	3’000’000 m^3	Net cultivated area	4900 hec
Average annual water supply in agriculture	31.3’000’000 m^3	Average evaporation losses of lake surface	2’000’000 m^3

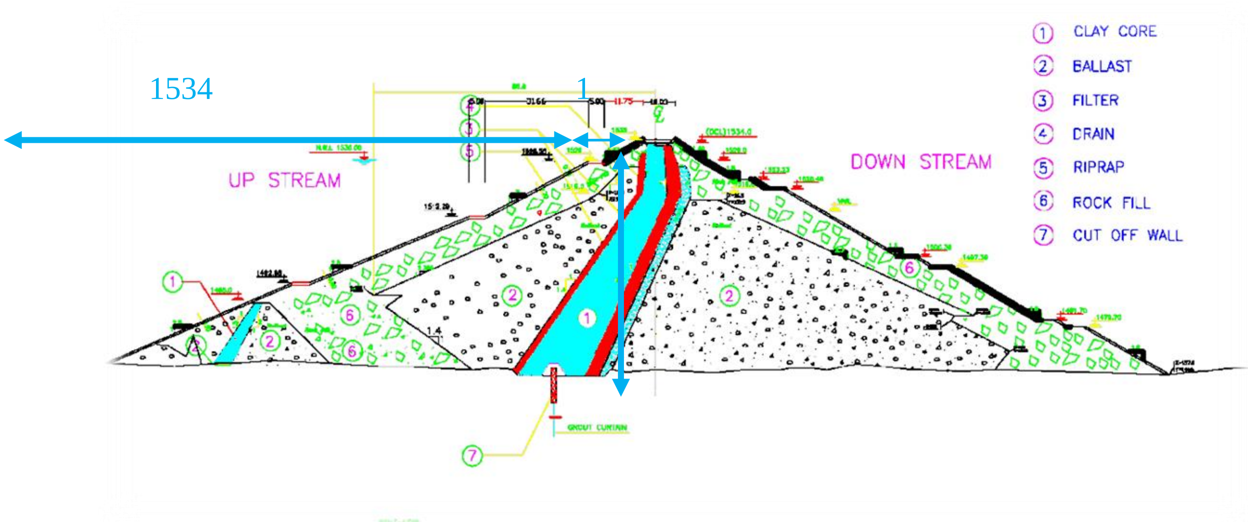

Figure 3: Dam main elements (components)

According to conducted experiments, soil mechanical features are as follows:
$C_{uu} = 0.4\ Kg/cm^2 \emptyset_{uu} = 0^{\circ}$
$C_{cu} = 0.4\ Kg/cm^2 \emptyset_{uu} = 21^{\circ}$
$C^{'} = 0.4\ Kg/cm^2 \emptyset_{uu} = 22^{\circ}$
The relationships of surface-volume- height in dam reservoir are provided in Table 4 regarding conducted studies.

Table 4: Data of surface-volume-height curve in dam reservoir

Sea level (m)		1468	1485.5	1491.75	1492.75	1498	1504.25	1510.5	1516.75	1523	1529.25	1530
Prior to sedimentation	Surface (Hec)	0	2.66	4.21	4.59	6.59	9.47	12.68	15.63	19.11	22.2	22.6
	(Volume (million m^3)	0\	1.6	3.7	4.3	7.2	12.1	19.1	28	38.8	51.7	52.9

2.3. Dam failure numerical modeling through MIKE 11 software

The data required in numerical model includes the geometry of waterways network and rivers in plan, cross-sections features, border conditions and hydrodynamic data. Plan of river geometry applied by using satellite images and the digital file received from Aster satellite images and scrutinized by data of field mapping in this area. Figure 4 shows the digital file used in this area for determining river course. The model required data provided as follows:

- Preparing and collecting required data and statistics including maps, aerial images, digital files, and hydrologic information of dam catchment basin
- Studying reports of dam phases 1 and 2
- Providing geometry of basin related waterways and rivers network in plan using satellite images and the digital file received from Aster satellite images and scrutinized by data of field mapping in this area (Figure 5).
- Characterizing cross-sections: cross-sections offered based on filed mapping file in understudied area. Totally, 1334 cross sections entered in numerical model in understudied area. Figure 6 represents some created cross sections in software context.
- Characterizing border conditions: in border condition at the entrance, the constant flow rate of 100 m^3/s used with dam failure onset. In n output boundary condition, the riverdischarge-scale curve used given the steady flow at this cross section and according to numerical model capabilities. Figure 7 shows the used discharge-scale curve at downstream output border condition of Esfarayen City.
- Providing hydrodynamic data
- Applying MIKE 11 software and inputting data to software
- Creating basic files: RIVER NETWORK, CROSS SECTIONS, SIMULATION, BONDARY CONDITION, NEW HYDRODYNAMIC MODULDE OF MIKE ZERO software
- Data required in numerical model include geometry of waterways network and rivers in plan, characteristics of cross sections, border conditions and hydrodynamic data.

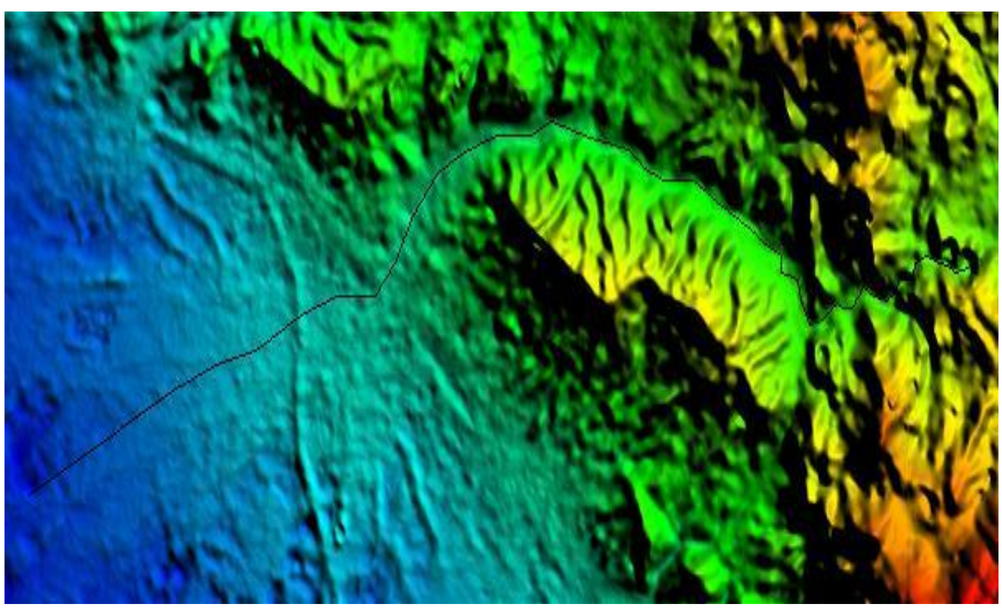

Figure 4: Digital file of dam area and downstream by USGS

Figure 5 represents the geometrical plan of waterways network modeled in MIKE software.

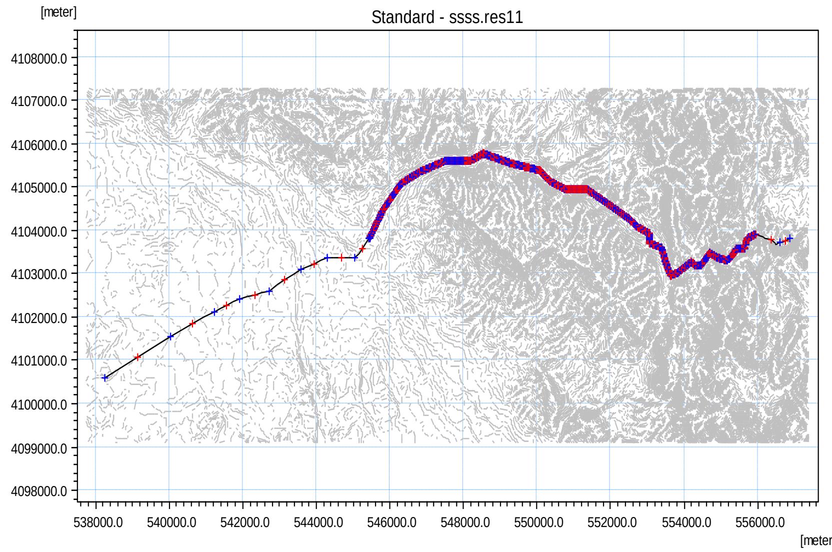

Figure 5: Waterway network and topography lines in understudied area

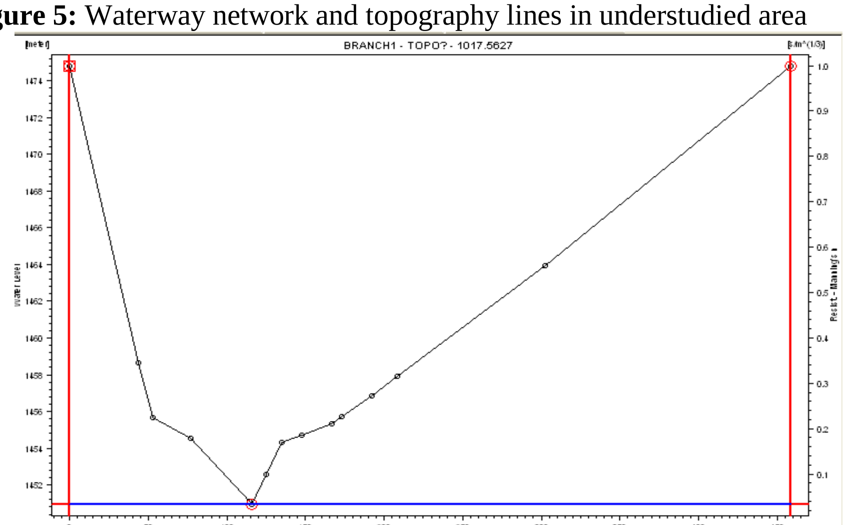

Figure 6: Cross section at 1017.5627 m distance from sample dam site

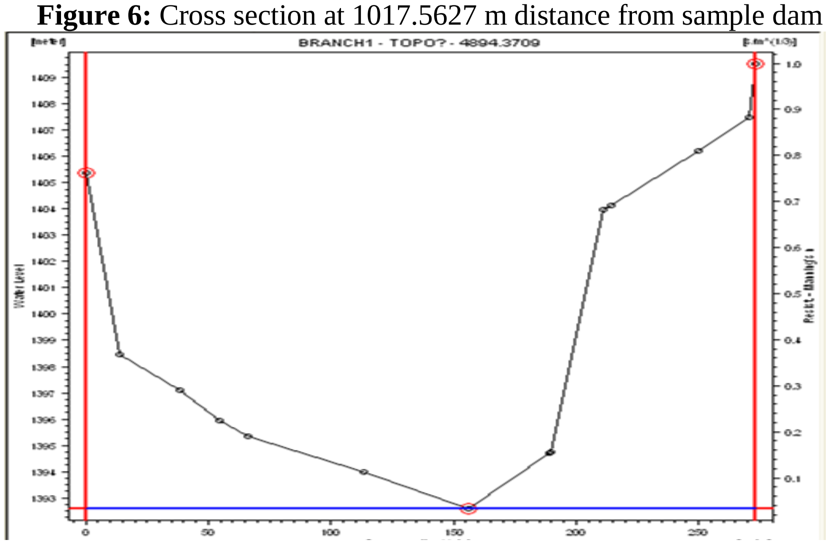

Figure 7: Cross section at distance of 4894.3709 m from sample dam site

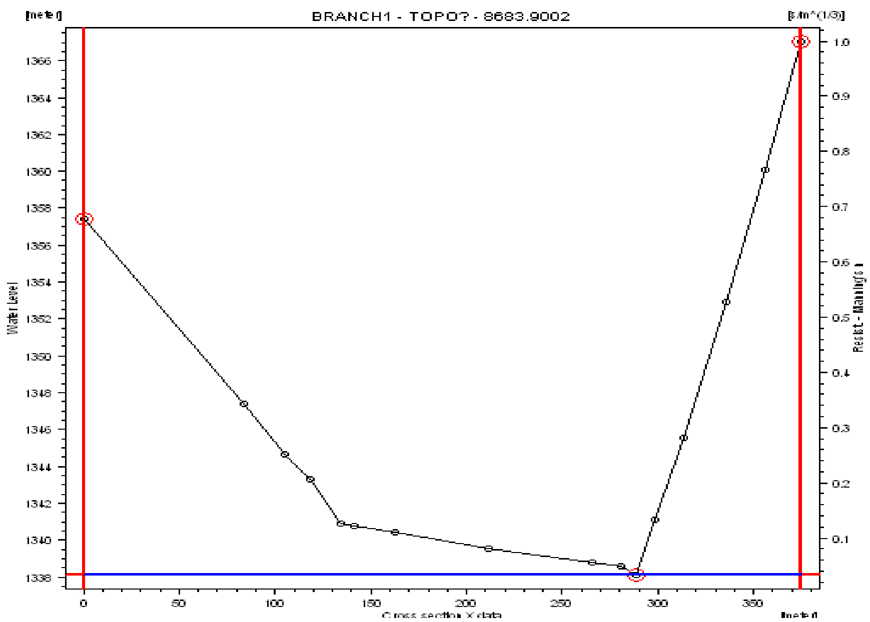

Figure 8: Cross section at 8683.9002 m distance from sample dam site

In input boundary condition, the steady flow rate of 100 m^3/s was used along with dam failure onset. In output boundary condition, river discharge-scale curve was used assuming the steady flow rate at this cross section and according to the used numerical model potentials. Figure 9 shows the discharge-scale curve used at model output boundary condition in downstream of Esfarayen city.

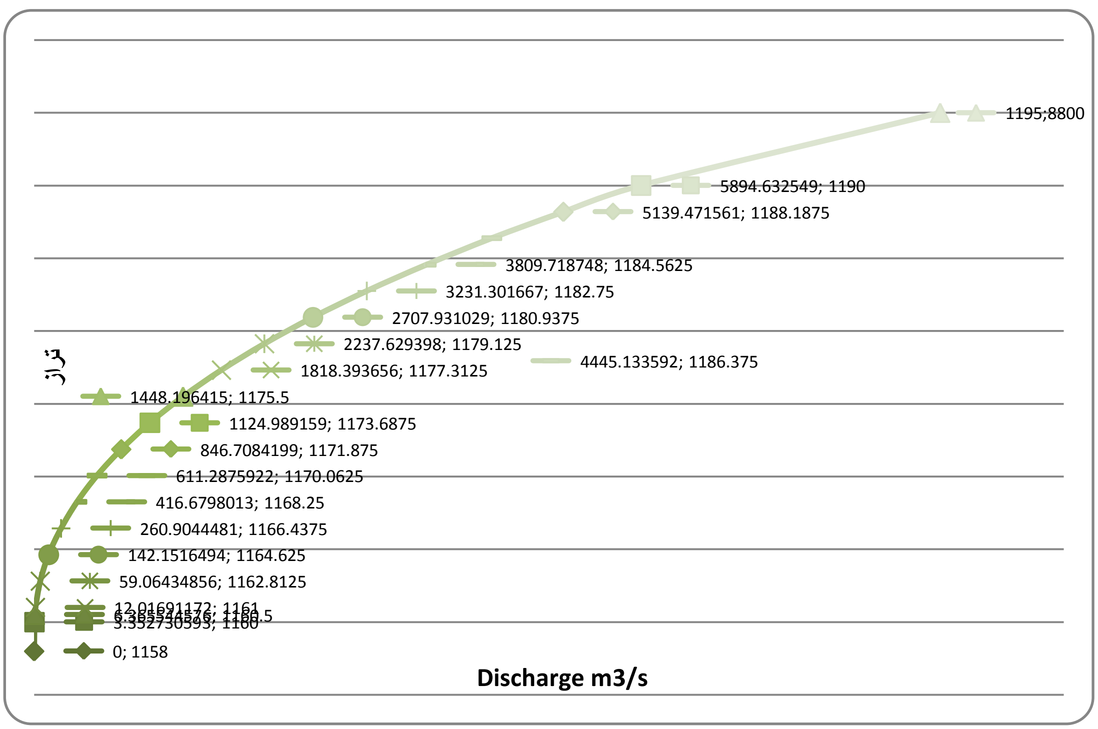

Figure 9: Discharge-scale curve at downstream section

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3. Results and discussion

Failure phenomenon modeling requires determining primary characteristics of tube formation in earthen dam body. In this regard, and according to lack of such information, different numerical scenarios with variable initial pipe diameter created and implemented. Figure 10 illustrates changes in peak discharge against pipe different initial diameter at the beginning of dam failure phenomenon.

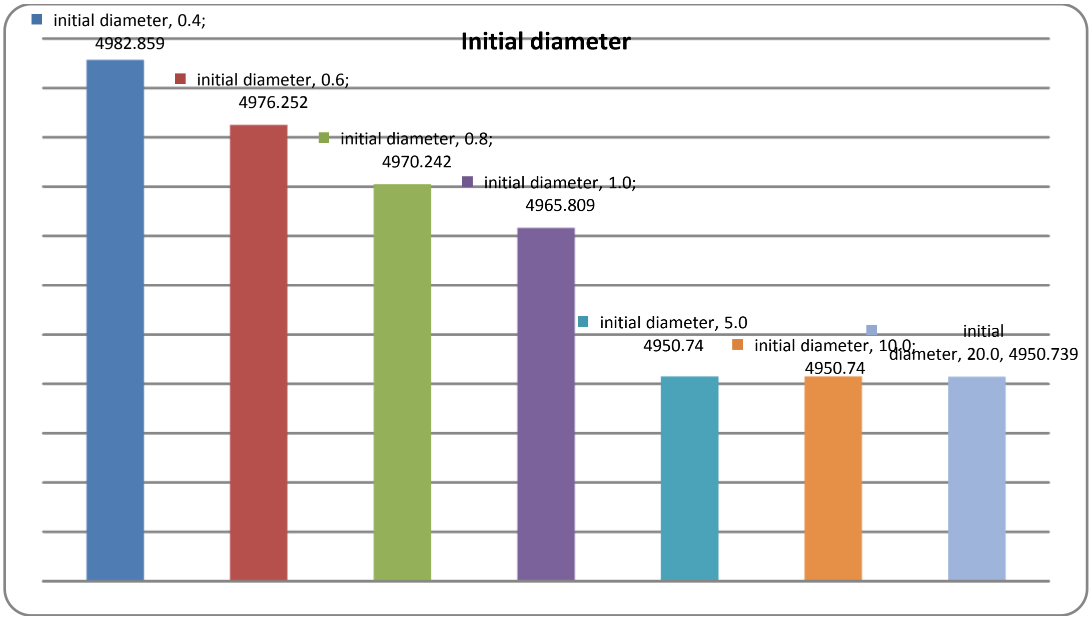

Figure 10: Studying the effect of various pipe diameters in determining maximum discharge of dam failure

Failure initial diameter measured based on trial and error. To do this, average width of gap was firstly measured based on equation 1; then, the model implemented in gap initial widths; and finally, gap width and average width are measured.

$$\mathbf{Bavg = 0.1803K0Vw\text{^}0.32hb\text{^}0.19} \qquad (1)$$

Where, Bavg is the gap average width (m), Cb is the function of reservoir storage, and K_0 is the flyover factor, S is the tank volume (m^3), tf is the time of gap development (hour), Vw gap high volume at the time of complete development (m^3), hd dam height (m), hw is gap high height at the time of canal development (m).
As seen in the figure, pipe increased initial diameter reduces the maximum peak discharge, which is attributed to the pipe ability in transferring more charge during failure phenomenon in the pipe larger initial diameters at the earthen dam body. Moreover, increasing pipe diameter up to 5 m higher has no effect on reducing flow peak discharge. It further assumes that the initial diameter of the pipe created in the body of earthen dam is 0.8 m and modeling scenarios conducted by this initial diameter. Figure 11 shows flow extending along understudied area influenced by dam failure with the initial pipe diameter of 0.8 m.

Figure 12 represents changes in water surface on dam axis in various times. As seen in the figure, water level reduces due to dam failure; furthermore, the level of 1530 selected as the normal level and it is assumed that this tank level has head potential at dam failure in upstream.

Figure 13 shows hydrographic changes of dam failure in different sections at about 2 km distances in 21-km range from dam downstream. As seen in the picture, dam failure maximum discharge at first section is 4985.44 m^3/s and 4125.7 m^3/s at the last section in 21142.30 km distances. This indicates that river capacity in reducing peak discharge is 17.2%.

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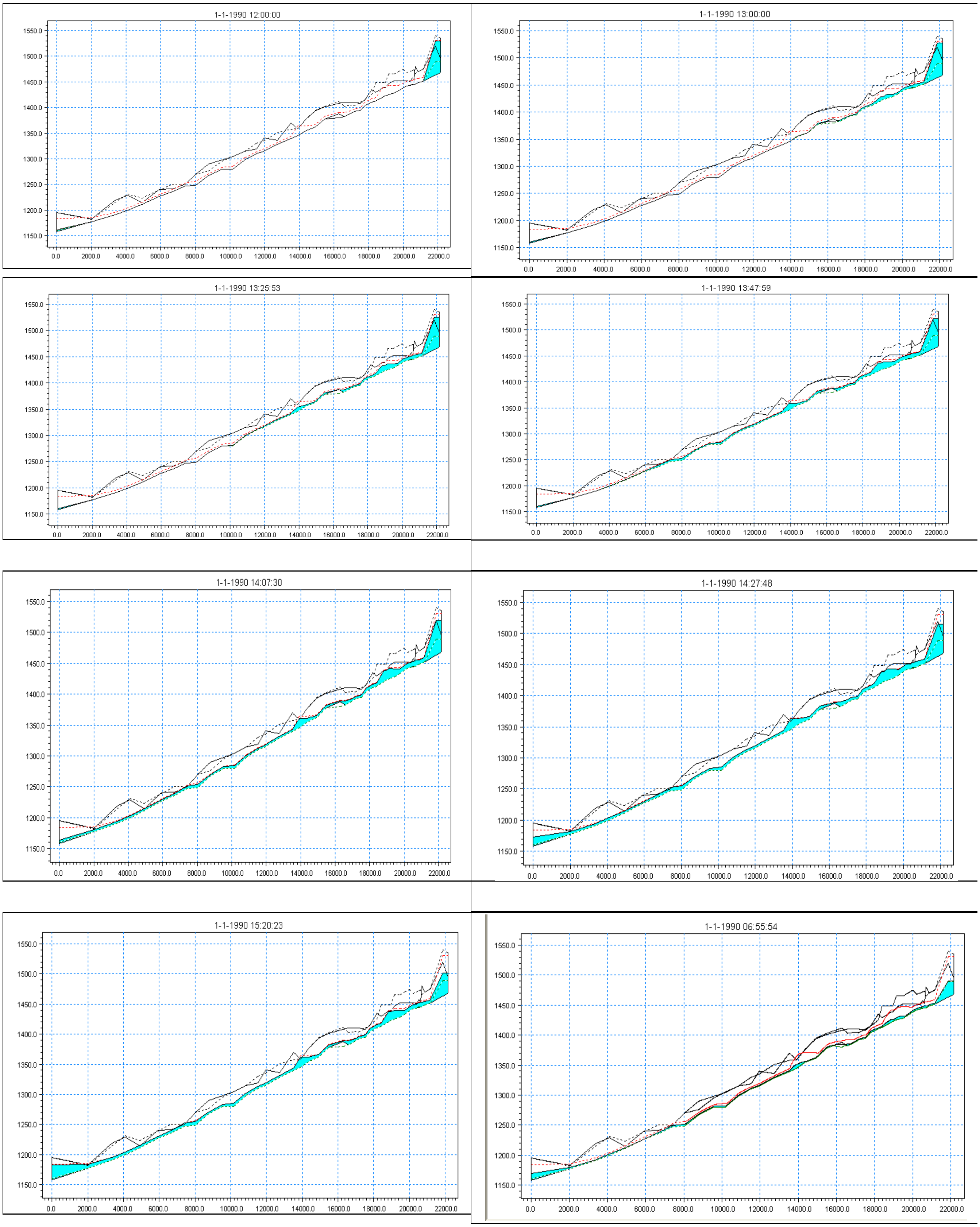


Figure 11: Changes in water surface due to dam failure along understudied area

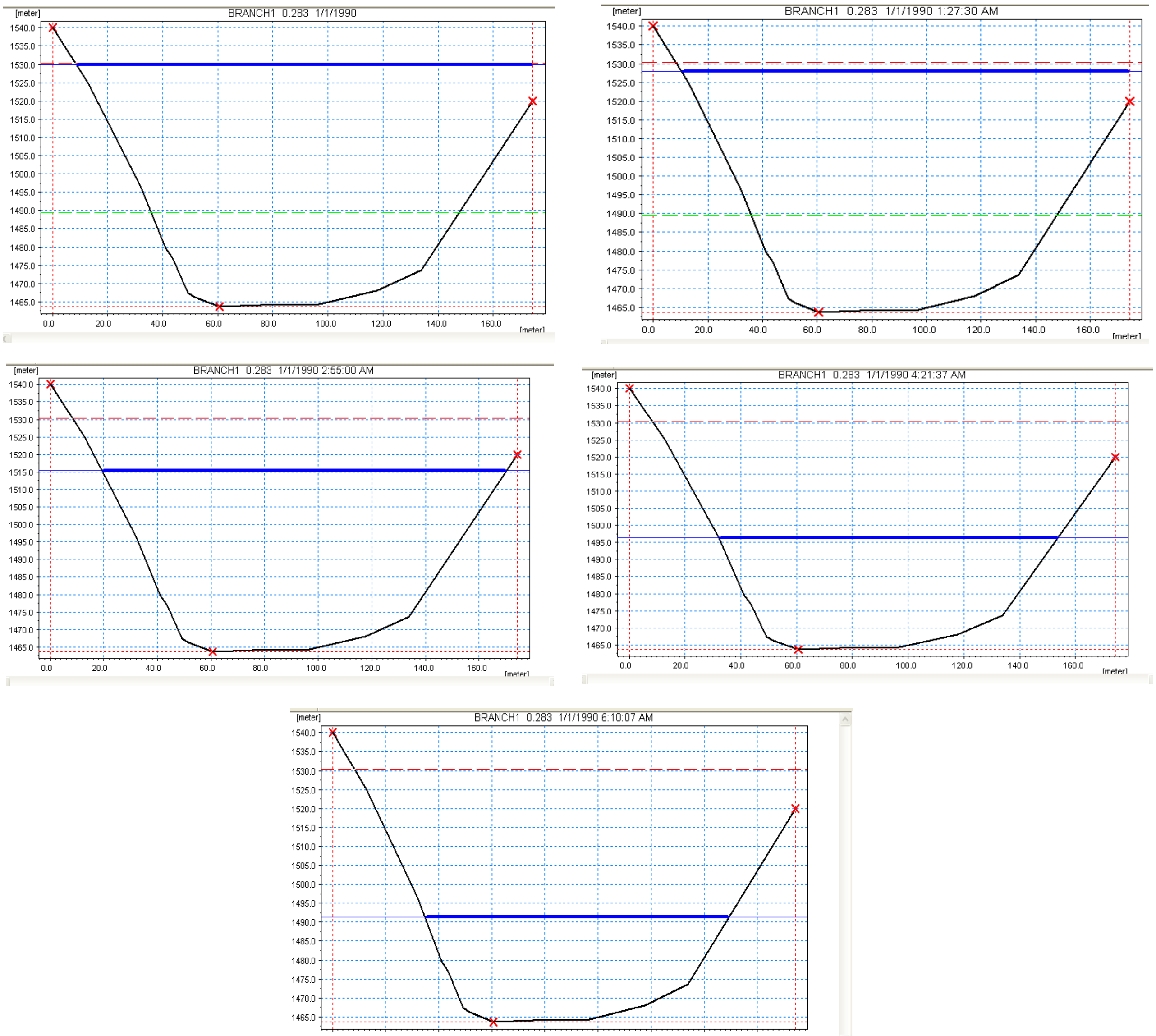


Figure 12: Changes of water surface in tank site during dam failure time

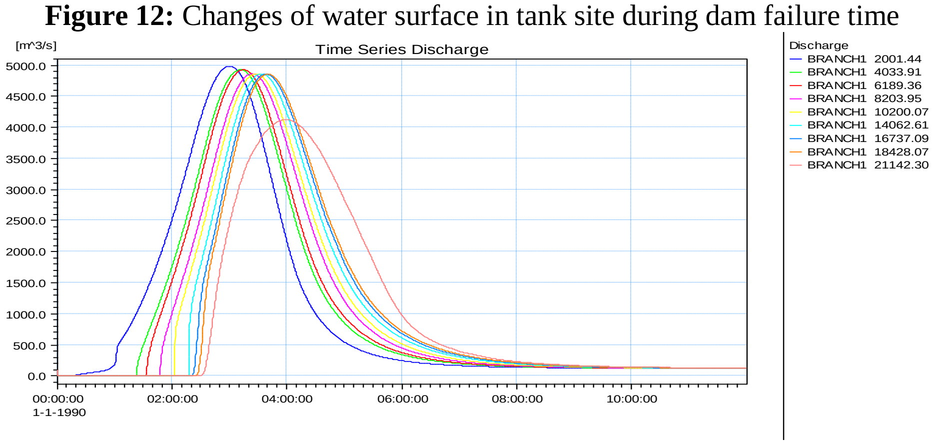


Figure 13: Chart of flood volume in different sections depending on time

Figure 14 represents Esfarayen city position comparing Bidvaz Dam and understudied area.

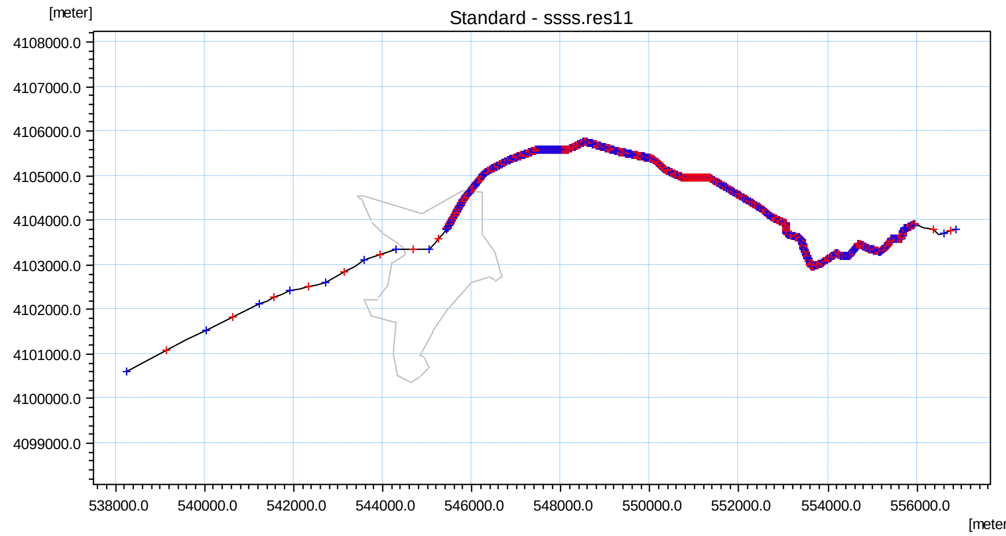


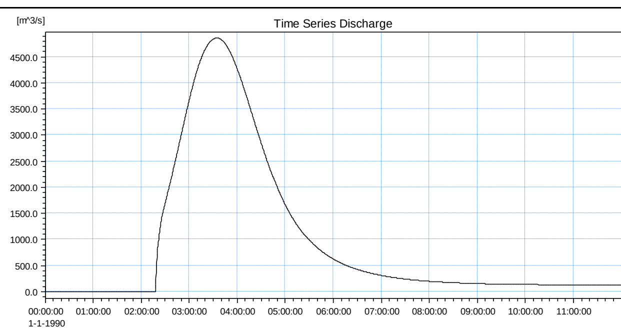
Figure 14: Dam position to Esfarayen City

Figure 15 shows hydrograph distribution of dam failure in different areas of Esfarayen City. As seen in figure, peak discharge of dam failure, in 14419.9 m chart at dam downstream, observed following 3 hours and 34 minutes and 26 seconds of dam failure in the surrounding of Esfarayen. Furthermore, the amount of peak discharge resulted by dam failure in urban area equals 4861.9 m^3/s demonstrating 2.47% decrease in failure peak discharge from dam site to city surrounding. This percent interpreted according to river high slope as well as river low width resulted by flood plain. As figure of changes in water level of Esfarayen City at the 14117 km in dam downstream reveals that maximum water depth in Esfarayen resulted from dam failure is 8.01 m. It is impossible to widen sections' width over dam failure in 1007 m due to topographical conditions.

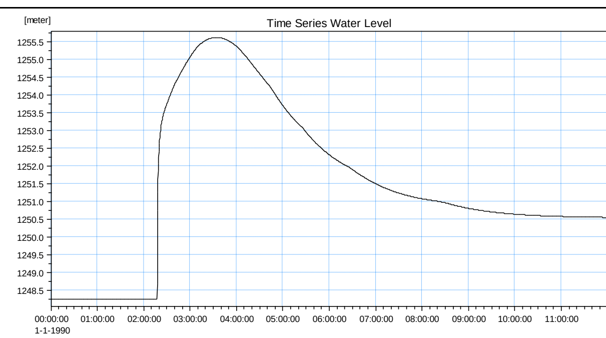
As last chart demonstrates, according to the area's topography conditions in downstream area of Esfarayen city, it is possible to higher enlarge flood flow in these areas.

Figure 16 shows changes in water level at dam downstream caused by dam failure phenomenon. As seen in the figure, dam failure event initially reduces water level by mild rate; next, by turning speed reaches to 1495; then, the speed of water level reduction decreases.

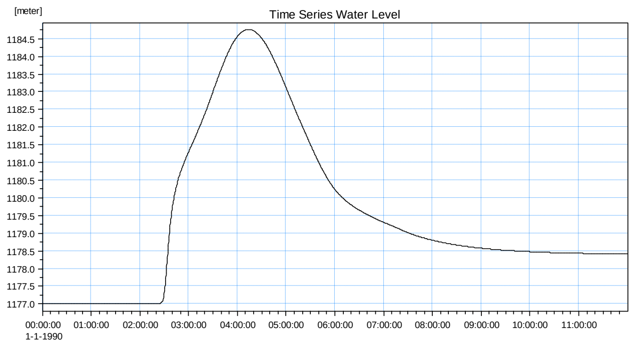
Figure 17 and 18 show the flooding area in dam downstream at different times. As clearly shows in the picture, river flooding area is largely influenced by river high slope in the upstream of Esfarayen city.



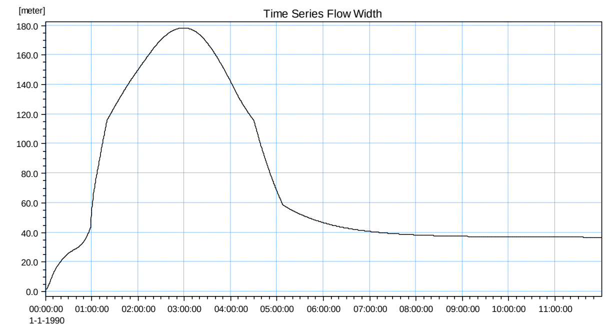
Changes in water depth over 14419.9 m from dam downstream in the area of Esfarayen



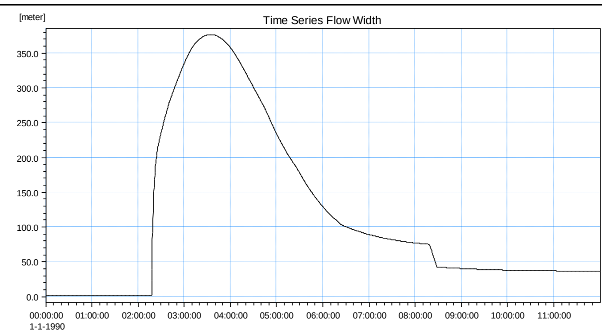
Changes in water depth over 14117 m from dam downstream in the area of Esfarayen



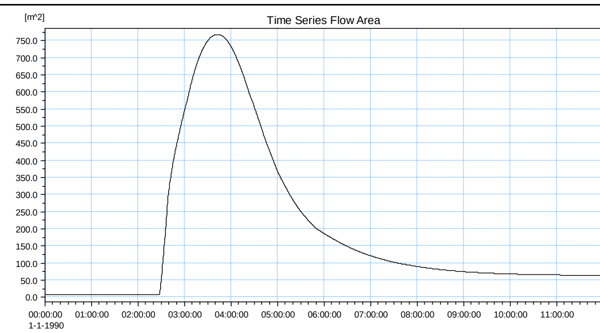
Changes in water level at Esfarayen city downstream



Changes in flow width of dam downstream and in upstream in Esfarayen city



Changes in flow width in the surrounding of Esfarayen city



Changes in flow surface at Esfarayen city downstream

Figure 15: Hydrograph distribution resulted from dam failure in different areas of Esfarayen city

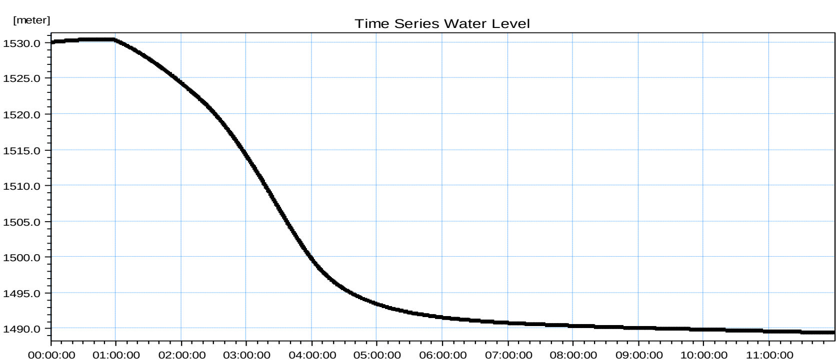


Figure 16: Reduced underground water level in the area of Bidvaz Dam

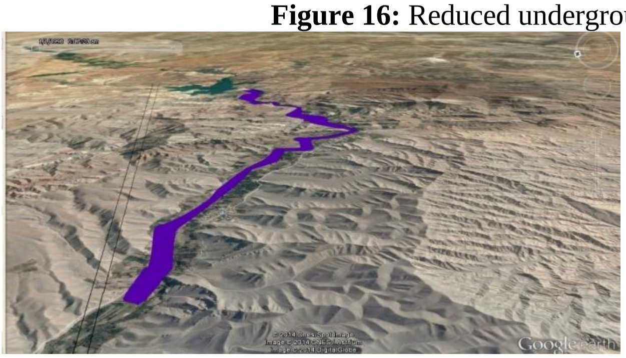


Figure 17: Distribution of flood area resulted from dam failure 1 hour and 47 minutes following dam failure onset

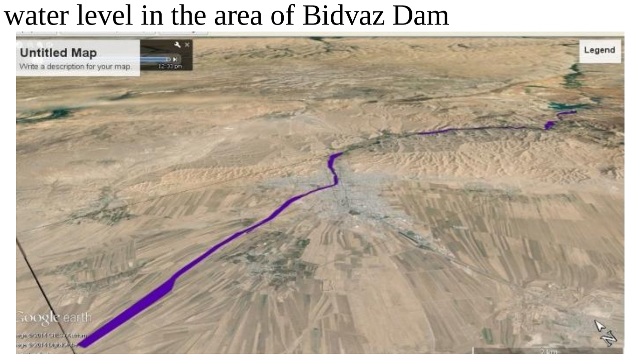


Figure 18: Distribution of flooding area resulted from dam failure 7 hours and 9 minutes after dam failure

Conclusion

- The major part of dam failures occurred during the first to tenth years of utilization. Dams' dewatering must be conducted in consistence with standards and instructions.
- Proper utilization and installing warning systems is necessary at dams' upstream and downstream.
- The defect of Bidvaz dam caused by imperfect studies and designing. There are lots of such earthen dams emerged in the country, which are threatened by failure. Thus, earthen dams require more attention regarding international criteria at the time of designing and utilization in order to decrease the risk of destructing these structures.
- The more the pipe maximum diameter in dam body, the larger decrease in dam failure maximum discharge. This may be related to the pipe ability in transferring more charge during failure phenomenon. Moreover, larger pipe diameter than 5 m has no effect on reducing flow peak discharge.
- Dam failure maximum discharge at the first section is 4985.44 m^3/s and 41257.7 m^3/s at the last section in 21142.30 m of dam downstream demonstrating the river capacity in reducing 17.2% of peak discharge.
- Peak discharge resulted from dam failure seen 3 hours, 34 minutes and 26 seconds following dam failure in the area of Esfarayen city. Further, dam failure peak discharge in city equals 4861.9 m^3/s, which shows 2.47% reduction in failure peak discharge from dam site to city. This amount is interpretable according to the river high slope as well as its low width caused by flood plain.
- Due to dam failure phenomenon, water level initially reduces in mild speed; next, the speed increases until the level approaches to 1495; then, the rate of water level reduction decreased.
- Maximum water depth, caused by dam failure, in Esfarayen city is 8.01 m. Furthermore, according to seabed limitation and privacy maps, the 25-year flow at this section equals 27.5 m.
- In Esfarayen city, the area of maximum flooding plain is almost 2 times more than its area in upstream, which is influenced by river topography. In addition, the flow wideness and distribution increases in downstream, too.

Recommendations

- Many instructive lessons learnt from the total failed dams, which each group related to effective factors. Applying the numbers of failed dams and analyzing failure effective factors can be good direction in dam construction management.
- Economic and technical analyses should be in a way that implement according to schedule so that they are safe with flood through less risk.
- According to the 380 m area of flood plain in the city, it recommended that appropriate land using such as parks, parking lots, or sport courts founded in order to maintain security level of downstream regions if any dam failure occurred.
- Establishing a flood alarming system at downstream as well as constructing peak discharge reduction strap floors at upstream in Esfarayen city may be effective in decreasing peak discharge.
- Regarding the identified problems of dam failure in the region, the primary priority at dam downstream is to create human security, which requires considering access levels in the area.
- The crisis priority and navigation maneuver in the region based on different organizations' convergence approach involved in flood issues with the suggested chart may lead to synergy and better crisis management.

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