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# Virtual Failure Analysis of the Çınarcık Dam†

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**ABSTRACT**

*In the present study, analyses of fictitious failure of a dam under a set of pre-defined scenarios, within the extent of a case study, has been performed. The subject of the study is the Çınarcık Dam, located within Bursa Province of Turkey. The failure of the dam is assumed to be triggered when certain critical criteria are exceeded. Hence, the analyses focus on the aftermath of the failure and strive to anticipate the level of inundation with respect to time downstream of the dam itself. For the purpose of the analyses, the FLDWAV software developed by the National Weather Service of USA is used to spatially and temporally predict the flow profiles, water surface elevations and discharges occurring downstream of the Çınarcık Dam under the defined set of scenarios. Based on these analyses, indicative inundation maps and settlements under risk were identified, and also some suggested pre-event measures that may be taken in advance will be addressed.*

***Keywords***: *FLDWAV, Dam Failures, Breach, Numerical Simulation, Çınarcık Dam*

## 1. INTRODUCTION

It is obvious that potential dam failures may result in great disasters despite many benefits they have for the society. There may be many reasons for dam failures [1], among them the floods occurring in river basins near existing dams, triggered by intensive rain is the most responsible one. When the flood hydrograph entering a dam reservoir reaches a peak value of unusual magnitude, the amount of water exceeding the capacity of the dam reservoir should be diverted downstream of the dam. If a spillway built for just that function was not designed for that kind of magnitude, excess water may spill over the dam crest. In case this happens, a breach may form in the dam body in minutes or hours depending on the type of material used in the dam body. As this breach gets larger and larger in time, the enormous amount of water stored in the reservoir upstream of the dam may start its motion as an uncontrolled flood wave downstream of the dam. A flood caused by a dam failure may occur in a much bigger magnitude compared to those floods generated by rain or snow melt. In the downstream river bed, the fast moving flood wave with its great power having the potential to destroy whatever comes in its way, may provoke deadly consequences should there be residential areas on its course.

According to a report presented by ICOLD (International Commission on Large Dams, ICOLD) in 1973, 38% of the dam failures occur either by water flowing over a dam crest

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due to insufficiently designed spillway or the destruction of the spillway itself by the large magnitudes of heavy precipitation. And 33% of the dam failures is caused by piping or leaks in the dam body. On the other hand, 23% of the dam failures are attributed to foundation problems, unstable slopes, landslides, earthquake initiated liquefaction or large waves generated on the reservoir surfaces.

Dam failures like Buffalo Creek Coal-Waste Dam, Teton Dam, Toccoa Dam and Laurel Run Dam that occurred in the USA in the 70's, resulted in attracting a great deal of attention on the dam-failure subject. According to an inventory report prepared in the USA in 1975, in which about 50 000 small and large dams were studied, as many as 20 000 dams have been found to be in a situation where they may cause both financial as well as human losses should any one of them fail, [2]. Table-1 shows the seriousness of the situation very clearly with some relevant data of observed dam failures, [3].

***Table 1.*** *Dam Failure Record in the U.S., [3]*

| **Name of the Dam** | **Date and Time of Failure** | **Height (m)** | **Volume Released ($m^3$)** | **No. of Deaths** | **Economic Damage (in million USD$)** |
|---|---|---|---|---|---|
| Williamsburg | 16.05.1874, 7:20 | 13 | 378,681 | 138 | Unknown |
| South Fork | 31.05.1889, 15:10 | 22 | 14,185,124 | 2,209 | Unknown |
| Grove | 22.02.1890, 02.00 | 34 | 74,000,000 | 85 | Unknown |
| Austin | 30.09.1911, 14.00 | 15 | 1,050,000 | 78 | 14 |
| St. Francis | 12.03.1928, Midnight | 57 | 46,873,000 | 420 | 14 |
| Castlewood | 2.08.1933 Midnight | 21 | 6,167450 | 2 | 2 |
| Baldwin Hills | 14.12.1963, 15:38 | 20 | 863,442 | 5 | 11 |
| Buffalo Creek | 26.02.1972, 08.00 | 14 | 498,330 | 125 | 50 |
| Black Hills | 9.06.1972, 23.00 | 6 | 863442 | - | 160 |
| Teton | 05.06.1976, 11:57 | 93 | 308,372,250 | 11 | 400 |
| Kelly Barnes | 6.11.1977, 01:20 | 12 | 770,100 | 39 | 3 |
| Lawn Lake | 15.07.1982, 05:30 | 8 | 831,371 | 3 | 31 |
| Timber Lake | 22.06.1995, 23.00 | 10 | 1,787,325 | 2 | 0 |

Preliminary steps of a similar study [4] were taken by the General Directorate of the State Water Works in Turkey (DSI) in 2002 by drawing up of a classification of some dams according to several risk criteria. The dams were located within the regional directorates of DSI whether they were in operation or in the conception stage. In that study, the dams that were considered were either greater than 15 m in height or their heights ranged between 5 m and 15 m but each had a reservoir volume greater than $3hm^3$. Once the study was completed for those dams for which the possibilities of potential serious problems were investigated, in case they would fail, it was found that 42 % of them carried the highest risk, and 43% carried high risk. In other words, 85% of those dams considered in the study fell in the high or highest risk category. Consequently, later in 2005, Dam Safety Unit was established within the Department of Dams and Hydropower Plants [4].

As of today, a great number of dams have been built in our country for flood control, potable water supply, and irrigation and energy production purposes. Once all of the hydropower potential has been exploited in the country, 526 Hydropower Plants will have been built [5]. Nearly all of those dams are in the large dam classification.

On the other hand, it is possible to evaluate the risks and take measures early before undesired events take place for any existing dam in operation or any dam not built yet. Prior to the unavoidable failure, people's lives may be saved and economic losses minimized within the limited time available.


***Figure 1***. *Satellite View of the Çınarcık Dam and Settlements Downstream of the Dam*

The Çınarcık dam located within the borders of Bursa province, which is the subject of the present study, has been made to fail in numerical simulations under several scenarios and the hydrodynamics of the potential flood wave was studied and the risks evaluated as it advanced downstream of the dam on the river [6]. Çınarcık dam was built on the Orhaneli creek, which is one of the two creeks branching into the Mustafakemalpasa creek. The other creek is the Emet creek. The aerial distance between the dam and the Mustafakemalpasa settlement is about 30 km. Figure 1.

## 2. TECHNICAL SPECIFICATIONS OF ÇINARCIK DAM

One of the functions of the Çınarcık dam is energy production; it is feeding the turbines of the Uluabat Hydropower Plant through a tunnel 11270 m long. The installed power of the plant is 120 MW. Table 2 shows some important properties of the dam.

***Table 2.*** *Technical Specifications of Çınarcık Dam*

| | |
|---|---|
| **1. Dam Body** | |
| Type | Rock-Fill |
| Height from Thalweg | 123.00 m. |
| Height from Foundation | 125.00 m. |
| Crest Elevation | 333.00 m. |
| Thalweg Elevation | 210.00 m. |
| Crest Length (Exclusive of spillway length) | 325.00 m. |
| Crest Width | 12.00 m. |
| **2. Reservoir** | |
| Maximum Water Elevation | 330.00 m. |
| Maximum Operating Water Elevation | 330.00 m. |
| Minimum Operating Water Elevation | 304.75 m. |
| **3. Spillway** | |
| Type | Front Inlet, Radial Gated |
| Capacity | 5191.80 $m^3/s$ |
| Gate Dimensions | 10.00 m x 14.00 m (5 gates) |
| Spillway Crest Elevation | 316.00 m. |
| Spillway Crest Length | 60.00 m. |
| **4. Bottom Outlet** | |
| Purpose | Irrigation Water Supply, and Discharge of Reservoir Water at Emergency |
| Capacity at Maximum Water Elevation | 71.78 $m^3/s$ |
| **5. Uluabat Power Plant Tunnel** | |
| Tunnel Capacity | 48.89 $m^3/s$ |
| Turbine Design Discharge | 48.90 $m^3/s$ |

**3. NUMERICAL MODEL USED IN SIMULATIONS (FLDWAV)**

FLDWAV is a generalized flood routing model for unsteady flow simulation. The main equations used in the model are complete one dimensional Saint-Venant equations of unsteady flow. These equations are solved in a computation domain that includes dams, bridges, and other structures, and considers the relevant boundary condition equations related to them. Moreover, the external boundary conditions at the upstream and downstream locations are taken into account. The systems of equations are solved by an iterative, nonlinear, weighed-four-point implicit finite-difference method, [7]. These equations account for the contraction, expansion in the channel and hydraulic losses in the bends. In addition, lateral flow joining into the main channel or leaving off the main channel may be considered. Saint-Venant conservation of mass and momentum equations are provided, respectively below.

Conservation of mass:

$$\frac{\partial Q}{\partial x} + \frac{\partial s_{co}(A + A_o)}{\partial t} - q = 0 \tag{3.1}$$

Conservation of momentum:

$$\frac{\partial (s_m Q)}{\partial t} + \frac{\partial (\beta Q^2/A)}{\partial x} + g A \left(\frac{\partial h}{\partial x} + S_f + S_e + S_i\right) + L + W_f B = 0 \tag{3.2}$$

Where; Q is the discharge, h; water surface elevation with respect to the mean sea level, A; active cross sectional area, $A_o$ ; inactive cross sectional area, $s_{co}$ and $s_m$ ; sinuosity factors, t; time, x; flow direction, q; lateral inflow or outflow per lineal distance along the channel, β; momentum correction coefficient, g; gravitational acceleration , $S_f$ ;frictional slope for main channel and flood plains, $S_e$ ; slope for expansion and contraction, $S_i$ ; additional friction slope associated with internal viscous dissipation of non-Newtonian fluids such as mud/debris flows, B; active river cross section width, $W_f$ ; effect of wind resistance on the surface of flow.

**4. ARTIFICIAL DAM-BREAK SCENARIOS**

Two groups of dam-failure scenarios were used to investigate the potential risks in the settlements nearby the Orhaneli and Mustafakemalpasa creeks, to be imposed by a probable flood generated by a prospective failure of the Çınarcık dam. They are the failures triggered by the overspill at the dam crest and the failures caused by piping within the dam body. Sub scenarios were tried out for each group of failures consisting of various failure parameters. In this paper, one of the failure scenarios, the Scenario W4, which is used in the winter season, will be presented. This scenario is one of the 10 scenarios in which it is assumed that water spilling over the dam crest causes the dam to fail. In Scenario W4, the failure of the dam was investigated under the effect of the catastrophic flood hydrograph entering the dam reservoir. This hydrograph is shown in Figure 2. The details of the scenario will be given in the section where the breach formation generated by overspill at the dam crest, to be explained.

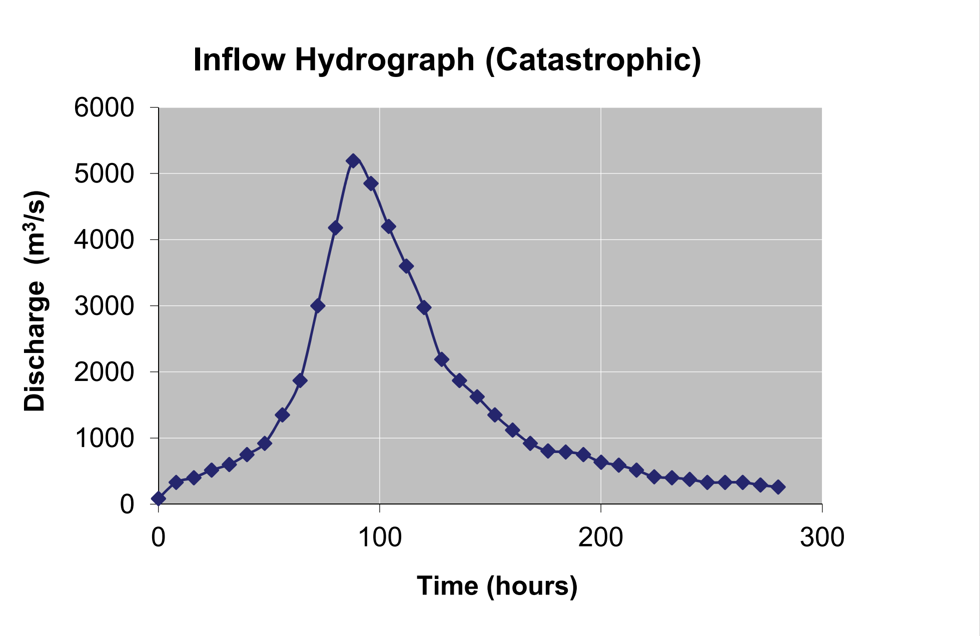


***Figure 2**. Catastrophic Inflow Hydrograph to Çınarcık Dam Reservoir*

**4.1 Definition and Importance of Breach Parameters**

Geometrical shape of a breach during the simulations of dam-break failures triggered by the water spilling over the dam crest may be assumed to be rectangular, triangular and trapezoidal. Temporal evolution of a breach from the crest to the toe of the dam is assumed to occur with the use of one of those geometrical shapes. This assumption, in general, is compatible with the observations. The discharge passing the breach at any instant is computed by the broad-crested weir equation. A breach developed completely is defined by several parameters. For instance, the parameter z shown in Figure 3 indicates the measure of the side slope in the horizontal plane. The vertical measure of the side slope is taken as unit distance. For the horizontal measure of the side slope, values of z may be selected between 0 and 2. In selecting this value, factors such as the internal friction of the dam-fill and the degree of compaction of the fill layers may play a role. The base width of the trapezoidal breach when developed fully is shown by the symbol, b. Using different combinations of b and z parameters, a breach of triangular, rectangular and trapezoidal shapes may be obtained. For instance, a rectangular breach may be formed with z=0 and b>0, while a triangular breach may result with z>0 and b=0 or a trapezoidal breach may be obtained with z>0 and b>0. As seen in Figure 3, breach width starts its evolution at a point at the dam crest and progresses linearly, then it completes its evolution at the ultimate base width, b during the failure duration of $\tau$. Breach base height from the dam toe is shown by $h_{bm}$, and $h_o$ shows the height of the breach base from the toe at an instant during the breach evolution, and h shows water surface elevation.

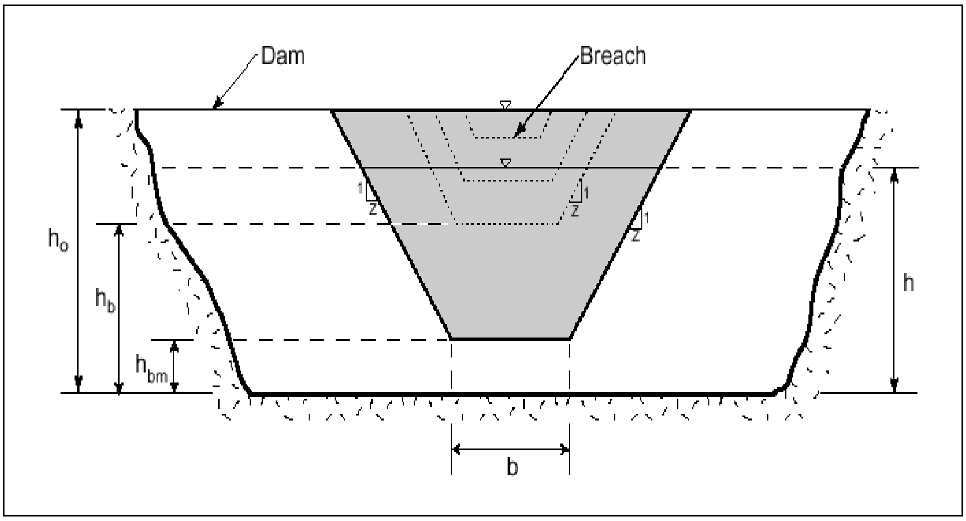


***Figure 3***. *Geometrical Definition of Breach Parameters*

Breach shape for earth-fill and rock-fill dams, in general, is assumed to be trapezoidal. Breach geometry is defined by parameters like breach height, average breach width, and side slopes. Later, the discharge passing through the breach is computed by the broad-crested weir equation. The time of failure in the FLDWAV model is defined as follows. It is the time that elapses between the instant when breaching starts on the upstream face of the dam and the instant the breach evolution is completed. In case of the situations where the failure occurs due to water spill over the dam crest, the beginning of the dam failure time is assumed to be the instant when the fill on the downstream face of the dam is washed away and the breach in the dam body moves across the dam crest width and reaches the upstream face of the dam. Moreover, breach initiation time is given with the purpose to warn people to vacate the region. Breach initiation time begins, when the amount of water large enough to make one think that people should be warned to vacate the region, flows for the first time over the crest or through the dam body and ends when breach formation begins. There is not much information to guide us in guessing the breach initiation time. Because of this reason, breach initiation time is not used in the FLDWAV model.

The magnitudes of the breach parameters used in one of the scenarios, W4, that resulted in the most adverse results in the failure simulations of the Çınarcık dam are as follows. The ultimate breach base width b=70 m, the final breach base height from the toe of the dam $h_{bm}$ = 113 m, breach formation time, 1.3 hrs, horizontal measure of the side slope z=1.27 and spillway gate opening, 2.5 m. It was assumed that the dam started to fail in an irrecoverable manner, when the water spilling over the dam crest elevation of 333.00 m reached the depth of 16 cm. In other words, the instant the water spilling over the dam crest reached the depth of 16 cm was accepted as the beginning of dam-failure. In addition, the amount of discharge coming from the Emet creek was taken as Q = 680 $m^3/s$.

## 5. CONCLUSIONS

Numerical model, FLDWAV, employed in the study computes the maximum water surface elevations, discharges and their occurrence times at pre-specified cross-sections. It also plots the discharge hydrographs at those cross-sections. Figure 4 shows the location of cross-sections used in the study.

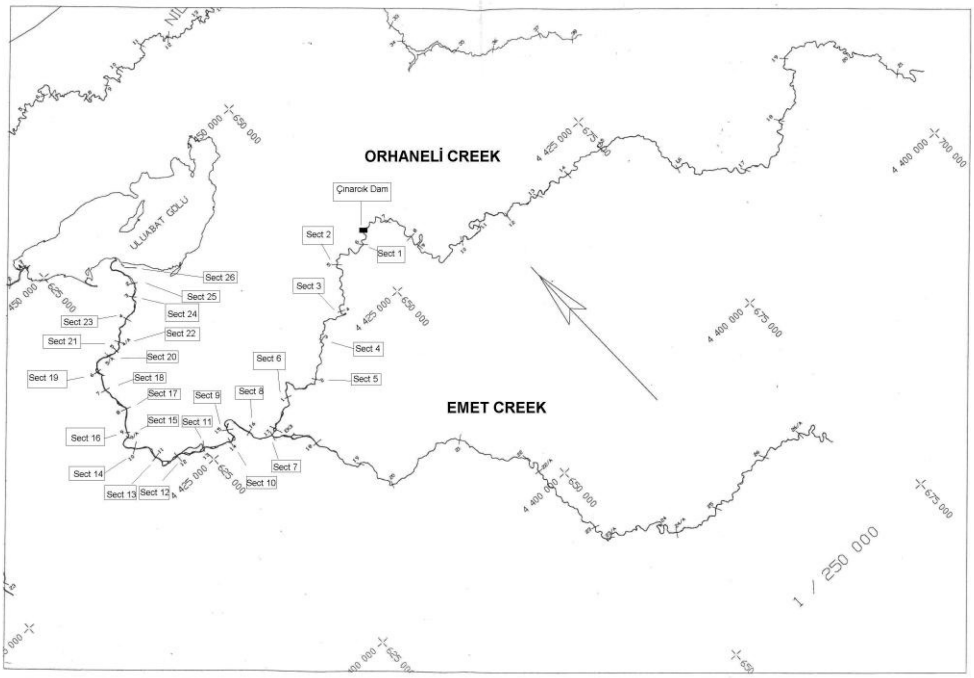


***Figure 4***. *Location of the Cross-Sections used in the Study*

In addition, plots showing water levels at cross-sections and the longitudinal profile of maximum discharge through the entire downstream reach may be obtained from the model. For instance, Figures 5 and 6, show the maximum water level and the time variation of water surface at km: 34.63, respectively. Figure 7 shows peak water surface profile over the entire reach.

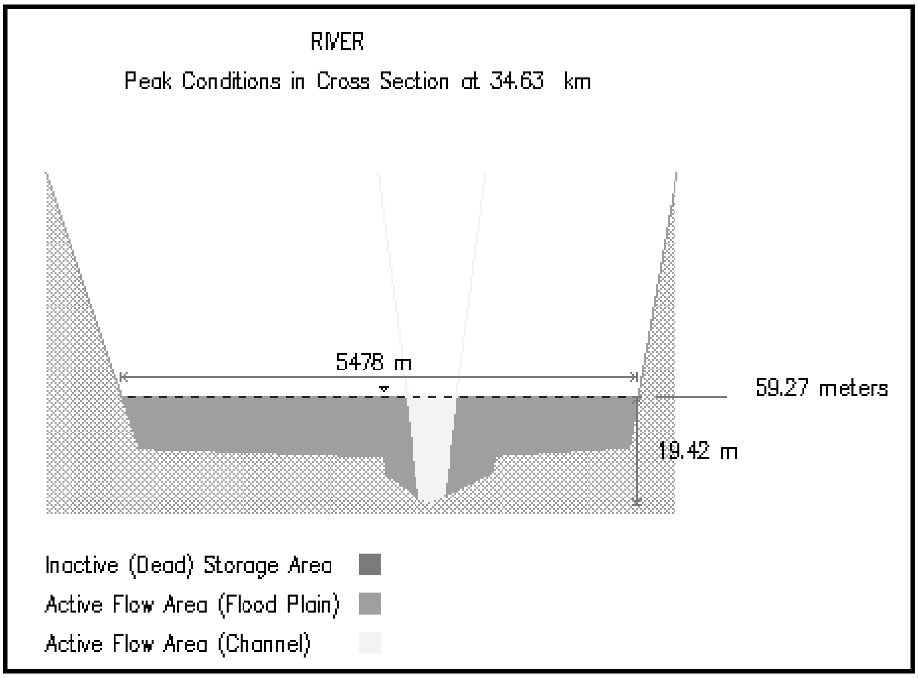


***Figure 5.*** *Maximum Water Surface Elevation at Cross-Section (km: 34.63)*

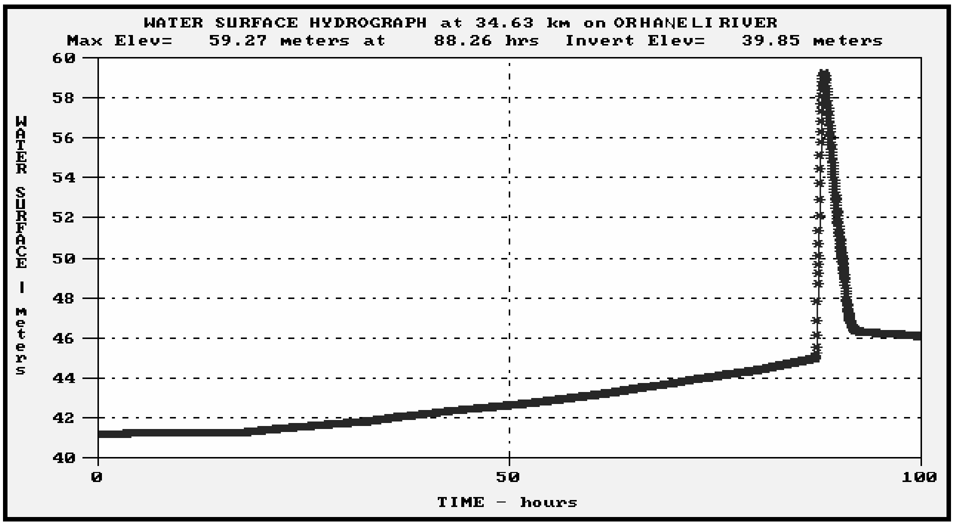


***Figure 6.*** *Water Surface Hydrograph at Cross-Section (km: 34.63)*

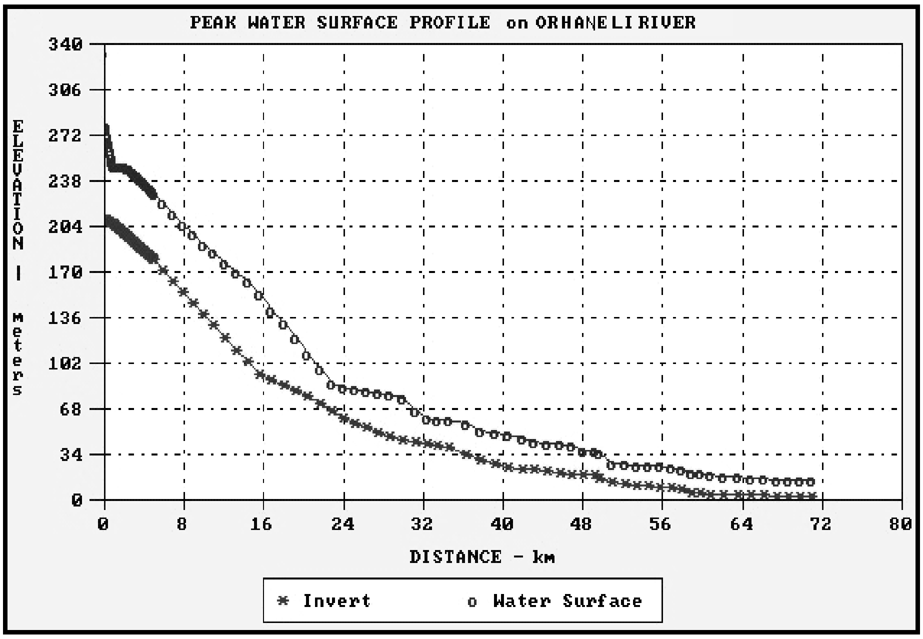


***Figure 7.*** *Peak Water Surface Profile on Orhaneli River*

According to Scenario W4, the maximum discharge in Orhaneli creek at km:34.63, was computed to be 77,115 $m^3/s$ and this discharge was reached in 80.8 hours after the dam failed. Moreover, the maximum water surface elevation was determined as 59.13 m 81.09 hours later. Similar computations have been performed for many other cross-sections along the river reach downstream of the dam, [6]. Table 3 shows some of the important quantities computed for three settlements.

***Table 3**. Some Important Quantities at Settlements for Scenario W4*

| Settlement Name | Settlement Elev. (m) | Distance to Dam (km) | Time of Max. WSEL (hrs) | Max. WSEL (m) | Peak Water Depth (m) | Water Surface Width at Peak Depth (m) | Time When WSEL Reaches Settlement Elevation (hours) |
|---|---|---|---|---|---|---|---|
| Karacalar Village | 90 | 20.50 | 80.35 | 108.53 | 30.36 | 1000 | 79.6 |
| Kestelek Village | 65 | 24.13 | 80.51 | 84.97 | 24.41 | 1829 | 68.6 |
| Çamandar Village | 55 | 28.63 | 97.64 | 78.82 | 34.33 | 1361 | 79.9 |

Figure 8 shows how the water surface varies with time, in the villages given in Table 3. When the results of the computations are analyzed, there is at least a duration of 70 hours to warn the people of Karacalar village (111 people) to move to a safe place. Similarly, it was computed that there is more than 60 hours for Kestelek (520 people) and Çamandar (193 people) villages to move to safety. Similar computations were made for all important residential areas located downstream. Based on some risk classifications, the number of potential losses in all of the villages was also determined. It is expected that the results of this study would constitute a database of some importance that the local administrators may use during or after a flood failure.

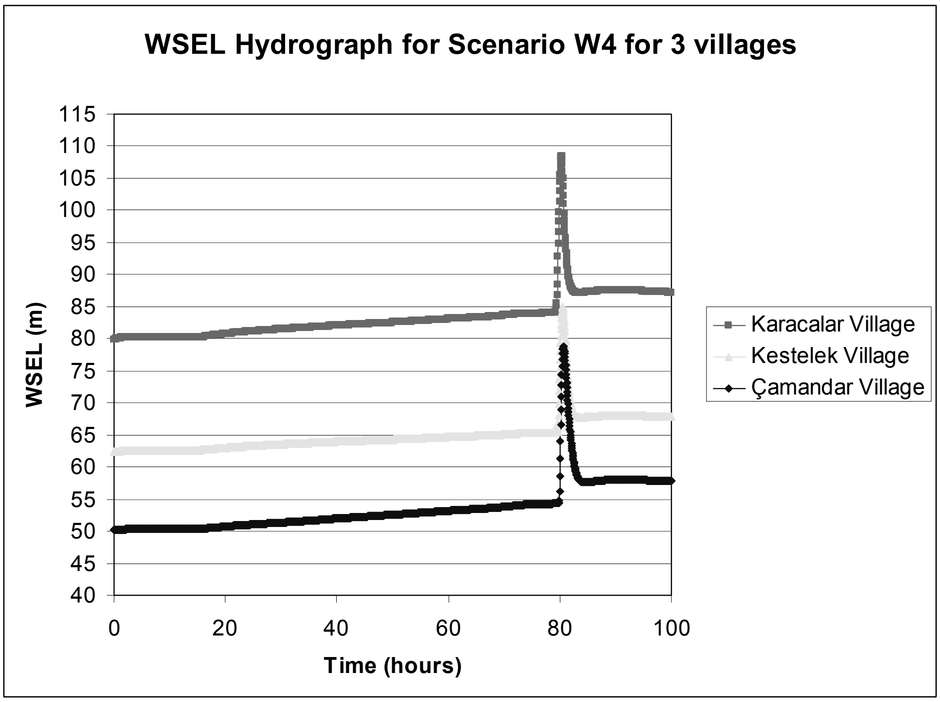


***Figure 8.*** *WSEL Hydrograph in Three Villages for Scenario W4*

The following recommendations are made for the local administrators in charge of public safety to apply in case of a flood caused by a dam failure, [1]

- Public officials should prepare emergency action plans (EAP) for dams that constitute important and high level danger.
- EAP's should be considered and updated every year. These works should include clearly defined methods in EAP's, and the names, telephone numbers, frequencies of wireless communication devices that belong to individuals or institutions to be contacted in case of emergency.

- They should make sure that definitions of existing communication devices and the relevant methods of warning to be used in EAP's be prepared.
- They should define the conditions including hydrological or nonhydrological, in which EAP's should be implemented, and prepare emergency response levels to be used, and the steps to be taken for each emergency response level based on hazard specifications.
- They should make sure that EAP's include as much as possible, inundation maps of potential areas subject to danger and the arrival times of the flood wave that local public officials would need, and tables that would have any other relevant information.
- EAP's should cover the list of important industries in the region according to predetermined priority criteria, and feasibility studies should be undertaken to decide which of those industries should be moved to safety.
- Local public officials and those people living in places of risk should be trained for vacating the region. These trainings are very useful since they will reveal the deficiencies or problems in the plans. Preliminary works should be undertaken on the subject of how the coordination will be ensured in case of emergency, by meeting with the directors of local TV and Radio stations.
- Diesel generators should be provided for emergency services, considering power loss of long durations that may happen.
- The General Directorate of State Water Works (DSI) should train some of its personnel so that they could prepare EAP's and perform dam-break analyses. Educational institutions domestic or abroad should be considered for this training. Among the options abroad, the following programs run by USBR, such as "Emergency Management Orientation Seminar", "Training Aids for Dam Safety" and "Dam Safety Training and Examinations and Public safety Assessments Around Dams" may be considered.

**Symbols**

| | |
|---|---|
| $A$ | Active cross-sectional area, $m^2$. |
| $A_o$ | Inactive (off-channel storage) cross-sectional area, $m^2$. |
| $B$ | Active river top width at water surface elevation, m. |
| $b$ | Terminal breach bottom width, m. |
| $g$ | Gravitational acceleration, $m/s^2$. |
| $h$ | Water surface elevation with respect to mean sea level, m. |
| $h_{bm}$ | Terminal breach height from dam foundation, m. |
| $h_o$ | Dam height from the foundation, m. |
| $L$ | Momentum effect of lateral flow |

| | |
|---|---|
| Q | Discharge, $m^3/s$. |
| q | Lateral inflow or outflow per lineal distance along the channel (inflow is positive and outflow is negative in sign), $m^2/s$. |
| $\mathbf{s_{co}}$, $\mathbf{s_m}$ | Sinuosity factors |
| $S_e$ | Expansion/contraction slope term in Saint-Venant equations |
| $S_f$ | Channel/floodplain boundary friction slope |
| $S_i$ | Additional friction slope associated with internal viscous dissipation of non-Newtonian fluids such as mud/debris flows. |
| t | Time, hour. |
| $W_f$ | Effect of wind resistance on the surface of flow. |
| WSEL | Water Surface ELevation, m. |
| x | Flow direction, m. |
| Z | Side slope of breach (Z horizontal:1 vertical) |
| β | Momentum coefficient for velocity distribution |

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