

MULTI-FAILURE ANALYSIS OF SEYHAN DAM AND ÇATALAN DAM

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MULTI-FAILURE ANALYSIS OF SEYHAN DAM AND ÇATALAN DAM

by

Can Kaya

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ABSTRACT

MULTI-FAILURE ANALYSIS OF SEYHAN DAM AND ÇATALAN DAM

Flooding risk due to a dam break has always been taken into consideration for the risk evaluation studies because of its serious potential damages to community and property located at downstream of the dam. In the scope of this thesis, the multi-failure dam break analysis of Seyhan Dam and Çatalan Dam has been carried out. The failures of the dams are not analyzed in terms of structural limits but it is assumed that the failure scenarios are based on some certain critical criteria that are exceeded. Because of their close proximity to Adana City, which is one of the most crowded cities in Turkey, Seyhan Dam and Çatalan dam have a key position to conduct this thesis work.

The main goal of this study is to forecast the flood and its impacts due to multiple dam failures of Seyhan Dam and Çatalan Dam under specified circumstances and dam failure scenarios within the HEC-RAS computer software analysis program. Another objective of this study is to determine the inundated areas and the flood features affect the corresponding regions. Based on the flood routing analysis results and evaluations, the recommendations are presented in order to implement them to the areas under risk.

ÖZET

SEYHAN VE ÇATALAN BARAJLARI ÇOKLU YIKILMA ANALİZİ

Baraj bölgelerinde yaşayan toplumlara ve maddiyata ciddi zararlarından dolayı, baraj yıkılmalarına bağlı sel ve taşkın durumları her zaman risk değerlendirme çalışmalarında ele alınmıştır. Bu tez çalışması kapsamında Seyhan ve Çatalan Barajlarının çoklu yıkılma analizi incelenmiştir. Baraj yıkılma analizleri yapısal olarak ele alınmamıştır fakat yıkılmaların belirli şartları ve limit değerleri aştığında gerçekleştiği varsayılmıştır. Türkiye'nin en kalabalık şehirlerinden biri olan Adana'ya olan yakınlığından dolayı, Seyhan ve Çatalan Barajları bu tez çalışmasının yürütülmesinde önemli bir rol oynamıştır.

Bu tez çalışmasının amacı; HEC-RAS bilgisayar programı yardımıyla, belirlenen şartlarda ve yıkılma senaryolarına bağlı olarak, Seyhan ve Çatalan barajlarının çoklu yıkılma mekanizmaları sonucu oluşan sel ve taşkınların ve ayrıca bu taşkınların bölgede yaratacağı etkinin belirlenmesidir. Bir başka amaç ise, taşkın bölgelerinin ve bu taşkınların özelliklerinin belirlenmesidir. Analiz sonuçlarına ve incelemelere bağlı olarak, risk bölgelerinde uygulamalarına geçilebilmesi amacıyla belirli önermeler sunulmuştur.

LIST OF SYMBOLS

a	The velocity head weighting coefficient alpha
b	Average width of fully formed breach in earthen dams
C	Crest width of dam (ft)
FAT_{BASE}	Base fatality rate
F_{age}	Age risk factor
F_{ev}	Ease of evacuation factor
F_h	Health risk factor
F_{mt}	Material risk factor
F_{st}	Story risk factor
F_{sv}	Flood severity factor
F_{war}	Warning factor
h	Flood depth (structures are exposed to) (m)
H_b	Depth of breach (ft) from dam crest to base elevation of breach
H_w	Height of water over the base elevation of the breach (ft)
g	Acceleration of gravity
LOL_i	Loss of life at a particular location “ i ” downstream of the dam
PAR_i	Population at risk at a particular location “ i ” downstream of the dam
Q_{safe}	Mean annual discharge or safe channel capacity at the same site (m^3/s)
Q_{df}	Discharge at a particular site caused by dam failure (m^3/s)
t	Time
t_f	Time of failure
τ	The breach formation time (hours)
V	Velocity
W_b	Width of breach (ft) at base elevation of breach

W_{df}	Maximum width of flooding caused by dam failure at the same site (m)
V_m	Volume of material in breach (yds ³) which is eroded
V_w	Volume of water stored in the reservoir (acre-ft) at the water surface elevation under consideration
Z_1	Slope ($Z_1:1$) of upstream face of dam
Z_2	Slope ($Z_2:1$) of downstream face of dam

LIST OF ACRONYMS / ABBREVIATIONS

ARS	Agricultural Research Service
AVHRR	Advanced Very High Resolution Radiometer
BFF	Breach Formation Factor
DSI	Devlet Su İşleri (General Directorate of State Hydraulic Works)
EAP	Emergency Action Plan
FEMA	Federal Emergency Management Agency
GUI	Graphical User Interface
HEPP	Hydroelectric Power Plant
ICOLD	International Commission on Large Dams
LFC	Large Format Camera
PAR	Population at Risk
USGS	United States Geological Survey
WCD	World Commission on Dams
WSE	Water Surface Elevation

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1. INTRODUCTION

Construction of dams has been a long-established practice with the oldest known dam, the Sadd el-Kafara near Cairo, Egypt, being built between 2950 and 2750 B.C. [1]. Building a dam gathers up different branches of engineering disciplines because of its sophisticated features.

The National Dam Safety Act of 2006 (P.L. 109-460) [2] defines a dam as: Any artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material, for the purpose of storage or control of water, that:

- is 25 feet (7.62 meters) or more in height from:
 - (i) the natural bed of the stream channel or watercourse measured at the downstream toe of the barrier; or
 - (ii) if the barrier is not across a stream channel or watercourse, from the lowest elevation of the outside limit of the barrier; to the maximum water storage elevation; or
- has an impounding capacity for maximum storage elevation of 50 acre-feet (61674 cubic meter) or more.

This definition does not apply to any such barrier that:

- is a levee; or
- is a barrier described above that
 - (i) is 6 feet (1.83 meters) or less in height regardless of storage capacity; or
 - (ii) has a storage capacity at the maximum water storage elevation that is 15 acre-feet (18502.2 cubic meter) or less regardless of height; unless the barrier, because of the location of the barrier or another physical characteristic of the barrier, is likely to pose a significant threat to human life or property if the barrier fails.

According to the International Commission on Large Dams (ICOLD) definition; dams having a height of more than 15 m or between 5m to 15 m with a reservoir volume of 3 hm³ or more, are called as large dams. According to ICOLD standards, there are 591

large dams out of totally 625 different sizes of dams in Turkey as of January 2006 (Figure 1.1). Out of these 591 dams, 579 dams are built by DSI and 222 large dams are built within the framework of large-scale water projects, while the remaining 369 dams are built within the framework of smaller-scale water projects. The total reservoir capacity of these 222 large dams is about 140 km³ [3].

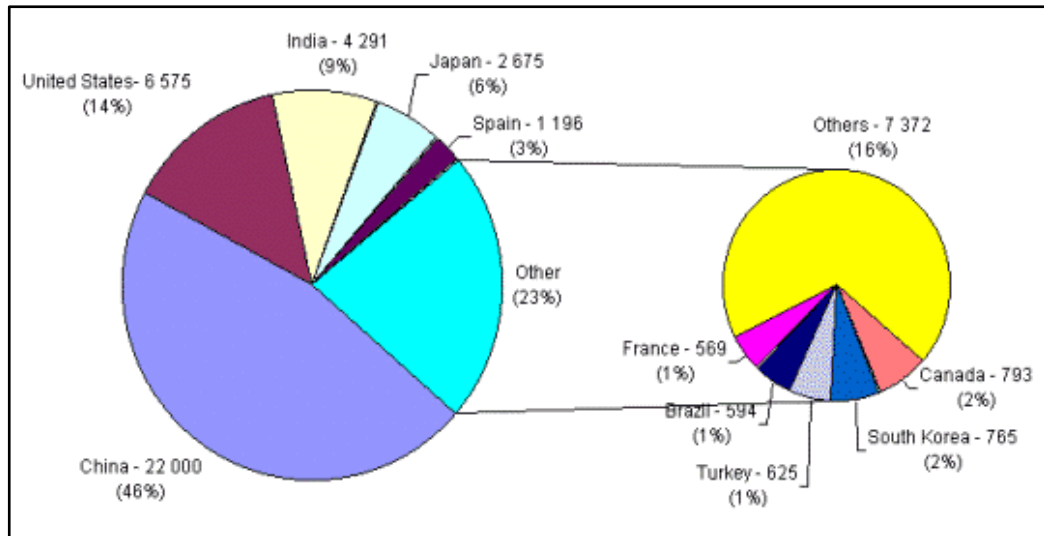


Figure 1.1. World Population of Dams by Country According to WCD Estimates, Based on ICOLD and Other Sources [4].

Throughout the history dams have provided many profits for people. There are many types according to their construction purposes. In the Engineering Manual 1110-2-1420 written by U.S. Army Corps of Engineering [5] the purposes of dams are listed as follows;

- Reservoir Purposes
- Flood Control
- Navigation
- Hydroelectric Power
- Irrigation
- Municipal and Industrial Water Supply
- Water Quality
- Fish and Wild Life
- Recreation
- Water Management Goals and Objectives

Besides having many advantages in constructing a dam, in case of its failure some certain types of damages is inevitable. Although there are some methods developed in order to reduce or eliminate the life loss by taking some specific precautions in case of a dam failure, it may sometimes cause very harsh structural and environmental damages. The reason why it may cause very heavy damages to environment is the fact that the outflow hydrograph due to failure is most likely very high compared to maximum design flood. Therefore it also explains the importance of prediction the potential hazards of dam failures and conducting dam break analysis.

1.1. Objective of the Study

The objective of the study is to forecast the flood and its impacts due to multiple dam failures of Adana Seyhan Dam and Çatalan Dam under different circumstances and dam failure scenarios within the HEC-RAS computer software analysis program. Another objective of this study is to determine the inundated areas and the flood features affect the corresponding regions. In addition to these two main objectives, the recommendations according to analysis results and evaluations are presented in order to implement them to the areas under risk.

In the analysis of the dam failures there are two main objectives. These are the prediction of the outflow hydrograph of the reservoir and the routing of this outflow hydrograph through the downstream of the dam to determine the possible outcome of the flood due to dam break. The prediction of the outflow hydrograph of the reservoir is vital especially for the population located close to the dam.

Dam Safety Guidelines written by Washington State Department of Ecology state that the reports discuss the findings from a dam break inundation analysis should address the following issues and list the pertinent parameters selected [6].

Dam Break Flood:

- The reservoir level and assumed inflow at the time of the hypothetical failure,
- The method of estimating/selecting the breaching dimensions and characteristics for the assumed mode of failure,

- The magnitude of the estimated dam break peak discharge at the dam site and the attenuation of the flood peak discharge as it propagates through the downstream valley.

Inundation Analysis:

- The travel time of the flood wave to various locations in the downstream valley,
- An inundation map depicting the areal extent of flooding,
- Representative channel/valley cross-sections depicting flow depth and typical flow velocities.

Downstream Hazard Classification:

- A general description of the valley and level of development downstream of the dam,
- The method used to determine the Downstream Hazard Classification.

2. DAM FAILURES

The Engineering Manual 1110-2-1420 by U.S. Army Corps of Engineers (1997) [5] lists the causes of dam failures as follows;

- Overtopping of a dam due to insufficient spillway capacity during large inflows to the reservoir,
- Seepage or piping through the dam or along internal conduits,
- Slope embankment slides,
- Earthquake damage and liquefaction of earthen dams from earthquakes, or landslide-generated waves within the reservoir.

HEC Research Document 13 (1980) [7] lists the prominent causes as follows;

- Earthquake
- Landslide
- Extreme storm
- Piping
- Equipment malfunction
- Structural damage
- Foundation failure
- Sabotage

In addition to this given list in the HEC Research Document 13 (1980), another important cause for dam failures is mis-operation. Davis (2001) [8] defines mis-operation as:

- Mis-operation of a dam or its appurtenant works is the sudden accidental and/or non-scheduled operation of a water retaining element of a dam that releases stored water to the downstream channel in an uncontrolled manner. Mis-operation also includes the deliberate release of floodwater because of an emergency situation, but without the issuance of a timely evacuation warning to the downstream interests. Mis-operation also includes the inability to operate a gate in an emergency, a

condition that could lead to overtopping of the dam and potential breach. Misoperation does not include structural failure of the dam.

The International Commission on Large Dams compiled a survey of dams more than 15 m in height that failed between 1900 and 1973 [9]. The three main causes of failure were overtopping (inadequate spillway), foundation defects, and piping. The major cause of failure of concrete dams was foundation failure (53 percent), and the major cause of failure of fill or embankment dams was piping and seepage (38 percent). Piping and seepage failures did not occur with any concrete dam. For all dams built, failures by overtopping due to an inadequate spillway (34 percent), foundation defects (30 percent) and piping and seepage (28 percent) have about the same rate of occurrence (Figure 2.1). These data clearly indicate that the greatest risks of a dam failing originate from ignorance of the magnitude and frequency of extreme floods, and uncertainties of the geologic setting [10].

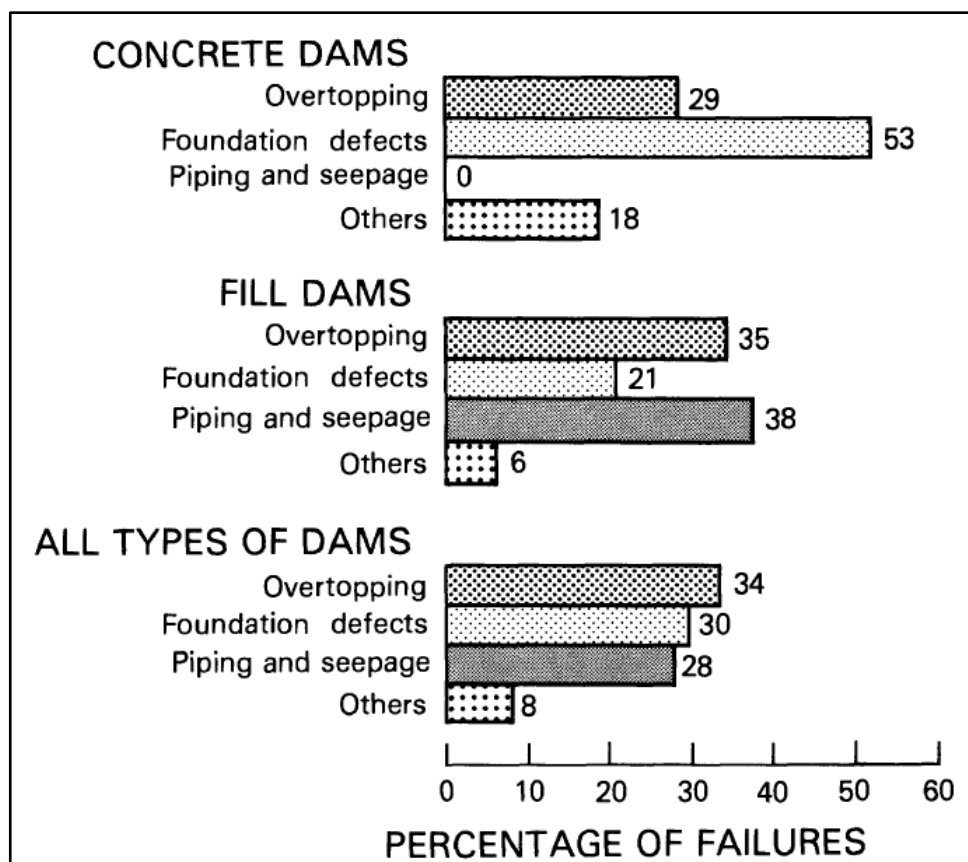


Figure 2.1. Causes of Failures for Different Dam Types Higher Than 15 m [9].

There are many reasons for dam failures. Some of these reasons are well explained in detail in literatures whereas some of them cannot be elaborated because of their high ambiguities. In the Federal guidelines prepared by FEMA [11] these reasons are listed in five main failure modes. These are Hydrologic, Geologic, Structural, Seismic, and Human influenced failure modes (Table 2.1).

Table 2.1. Typical Dam Failure Modes [11].

Failure Mode	Examples of dam failures
Hydrologic	Overtopping due to: • Inadequate spillway design • Blocked spillway • Loss of freeboard due to embankment settlement or erosion • Structural overstressing of dam components
	Surface Erosion due to: • High velocity water • Wave action
Geologic	Piping and internal erosion caused by: • Internal cracking hydraulic fracture, or differential settlement • Inadequate filters • Outlet pipeline failure • Pipes through the embankment formed by roots or animal/insect burrows
	Slope instability and hydraulic fracturing: • Load exceeds sliding resistance at base or joints of structure

Table 2.1. Typical Dam Failure Modes [11] (cont.).

Failure Mode	Examples of dam failures
Structural	Concrete dam: Failure of critical structural components Embankment dam: Failure of the upstream or downstream face
Seismic	Earthquakes/ground movement; also liquefiable foundations or embankment materials
Human influenced or caused	Misoperation: • Sudden rise in reservoir level causes flow through transverse cracks in embankment • Incidents including gate failures, power interruption etc.
	Terrorist activities: • Purposeful misoperation of the dam • Impact of object that removes part of the dam crest

According to statistics, dams of significant size fail in the United States at an average rate of more than one per year [12]. In the world, during the last 100 years, there have been about 200 significant failures of constructed dams, resulting in the death of more than 11,100 people. More than 60 percent of this loss of life occurred in three failures alone: Vaiont, Italy, 1963 (2,600), South Fork (Johnstown) Pennsylvania, U.S.A., 1889 (2,200), and Machhu II, India, 1974 (2,000 +). Major causes of constructed- dam failures are overtopping due to an inadequate spillway (34 percent), foundation defects (30 percent), and piping and seepage (28 percent). An order-of magnitude estimate of risk of dam failure is approximately 10^{-4} per dam-year. In the United States for the last 2 decades, the annual loss of life from dam failures has been about 14. The average number of fatalities per dam failure is 19 times greater when there is inadequate or no warning [10]. Some examples for dam failures with their death toll and economic damages are given in Table 2.2 [13].

Table 2.2. Human and Economic Consequences of Dam Failure by W. Graham [13].

Dam	Date and Time of Failure	Height (ft)	Volume Released (ac-ft)	Deaths	Damage
Williamsburg Dam, MA (Mill RiverDam)	May 16, 1874 at 7:20 a.m.	43	307	138	
South Fork Dam, PA, (Johnstown Dam) Walnut	May 31, 1889 at 3:10 p.m.	72	11500	2209	
Grove Dam, AZ,	22 February 1890 at 2 a.m.	110	60000	About 85	
Austin Dam, PA	September 30, 1911 at 2 p.m.	50	850	78	
St. Francis Dam, CA	March 12-13, 1928 at midnight	188	38000	420	\$14 m
Castlewood Dam, CO	August 2-3, 1933 at midnight	70	5,000	2	\$2 m
Baldwin HillsDam, CA	December 14, 1963 at 3:38 p.m.	66	700	5	\$11 m
BuffaloCreek, WV(Coal WasteDam)	February 26, 1972 at 8 a.m.	46	404	125	\$50 m
Black Hills Flood, SD (Canyon Lake Dam)	June 9, 1972 at about 11 p.m.	20	700	???	\$160 m
Teton Dam, ID	June 5, 1976 at 11:57 a.m.	305	250,000	11	\$400 m
Kelly Barnes Dam, GA	November 6, 1977 at 1:20 a.m.	40	630	39	\$3 m

Table 2.2. Human and Economic Consequences of Dam Failure by W. Graham [13] (cont.).

Dam	Date and Time of Failure	Height (ft)	Volume Released (ac-ft)	Deaths	Damage
Lawn Lake Dam, CO	July 15, 1982 at 5:30 a.m.	26	674	3	\$31 m
Timber Lake Dam, VA	June 22, 1995 at 11 p.m.	33	1,449	2	\$0 m

A study conducted by Middlebrooks [14] into the causes of 220 earth dam failures during the period 1850-1950 summarizes observed causal agents and their frequency of occurrence. It is interesting to note that 50 percent of the failures catalogued by Middlebrooks occurred within the first five years and that 19 percent failed upon first filling (Table 2.3) [6].

Table 2.3. Dam Failures and Ages of Dams at Time of Failure [6].

NUMBER OF YEARS AFTER COMPLETION	CAUSE OF FAILURE (%)				TOTAL %
	OVERTOPPING	CONDUIT LEAKAGE	SEEPAGE	SLIDES	
0-1	9	23	16	29	19%
1-5	17	50	34	24	31%
5-10	9	9	13	12	11%
10-20	30	9	13	12	16%
20-50	32	9	24	23	22%
50-100	3	0	0	0	1%

3. SEYHAN DAM AND ÇATALAN DAM

Seyhan Dam and Çatalan Dam which are in the scope of this thesis are located on the Seyhan River. The accentuated point in dam break analysis is the multiple failure consequences of these two dams. Whereas main goal in examination of multi-failure dam analysis is to reveal the correlations between the dams and how the result of inundation or flood pattern changes with respect to change in multi interaction of failures. It is also another concern to observe how failure of Çatalan Dam which is near to the upstream river of Seyhan Dam, change the storage area and the way this accumulated water storage force overtopping failure.

The location and the positions of dams with respect to each other can be seen from the map (Figure 3.1). As it seen from the figure, the distance between two dams is quite small with respect to each other. The distance in between the dams are roughly 20 kilometers. That means propagation of downstream flood mechanism due to failure of Çatalan Dam would be rather sudden and sever, because there is no enough distance to attenuate the effect of flooding. In other words the energy dissipation of downstream flood due to failure of Çatalan Dam would be rather small. The detail of the flood patterns and their effects are examined in evaluation of the results section.

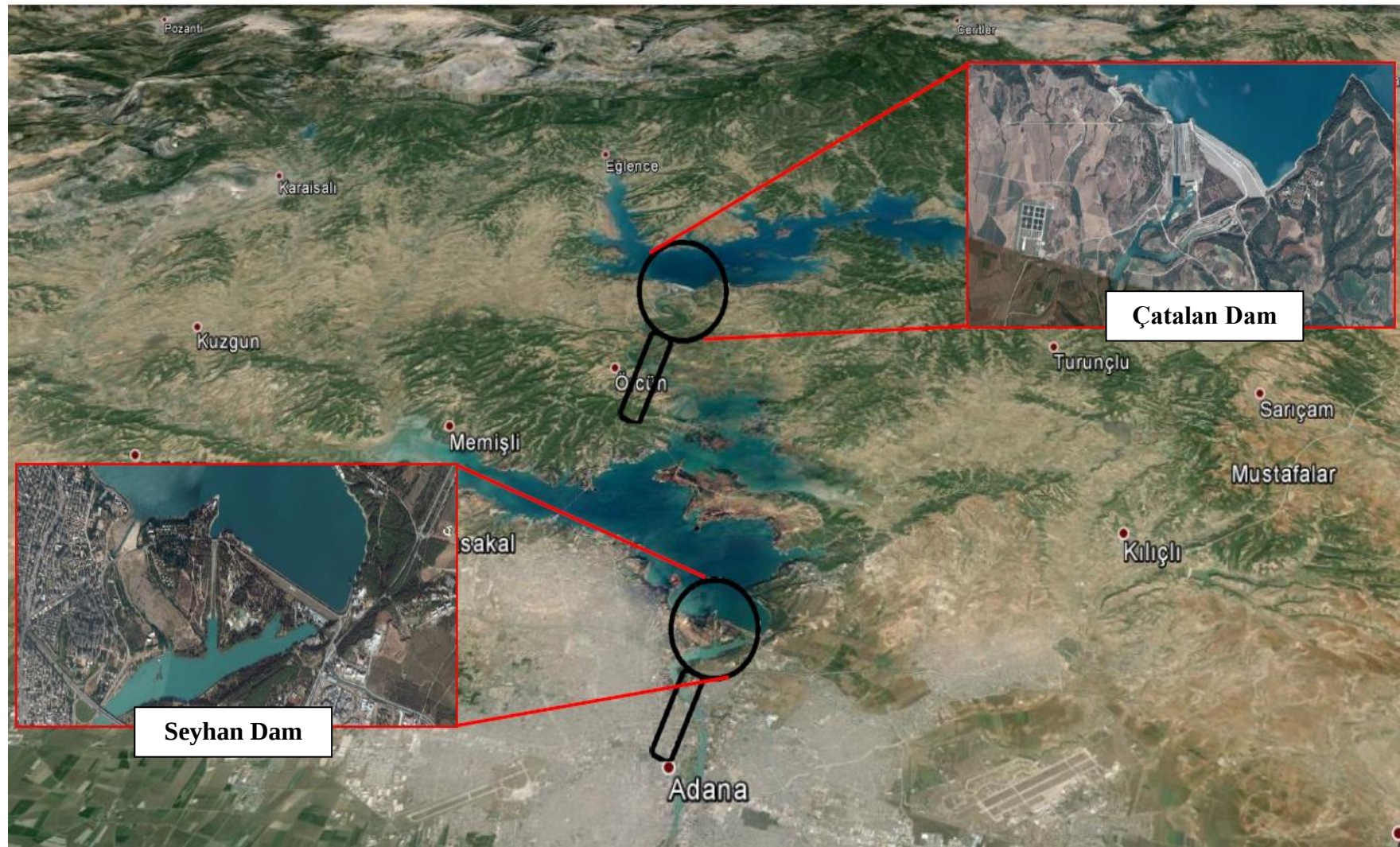


Figure 3.1. Site Map of Çatalan Dam and Seyhan Dam.

3.1. Seyhan Dam

Seyhan Dam is one of the biggest dams in Turkey. The construction of the dam has been started in 1953 by the endorsement of Prime Minister Adnan Menderes and the project manager of the dam was Süleyman Demirel latter prime minister and 9th president of Turkey. The type of dam is earth fill dam with height of 53.2 m. Total reservoir capacity of the dam is 799 ha³. The normal water level reservoir area is measured as 63.04 km². The dam was constructed on the Seyhan River in 1956 in order to provide electricity and flood protection. The dam was designed according to 100 years return period floods. Seyhan River is formed by confluence of Zamantı River and Göksu River. The total catchment area of the Seyhan River basin is approximately 20000 km².

A feasibility study for the Seyhan Dam was carried out from 1949 to 1951, and the dam was constructed from 1953 to 1956. The dam spillway has a capacity of 2500 m³/s; the power plant design discharge is 231m³/s and the total amount of water used for irrigation is 33m³/s [15].

In the coastal area of the Seyhan River basin, precipitation is about 800 mm annually. At higher elevations, it increases to 1000 mm, and in the northern parts of the basin the corresponding value diminishes to 400 mm. Much of the precipitation, 50% of the annual total, falls between December and March. The mean annual precipitation of the basin is 590 mm. According to the climatic features of Turkey, the precipitation falling in winter over the areas above 1000 m elevation is generally in the form of snow, and produces snow cover on the ground. Snowmelt starts around the end of winter or at the beginning of spring, when air temperatures rise [15].

The flood occurred in 1980 was the largest flood causing considerable damages to Adana City. This flood demolished 21 buildings and also caused considerable level of damages to 76 buildings. This flood was accepted as close to 100 years return period flood in the area. Therefore in order to increase the capacity of the food protection, it was decided to construct Çatalan Dam at the upstream of Seyhan Dam in order to provide 500 years return period flood criteria.

3.2. Çatalan Dam

Çatalan Dam is located on Seyhan River at approximately 1 kilometer downstream of Çatalan Town. Construction of Çatalan Dam was started in 1982 and completed in 1997. The dam was constructed on the Seyhan River in order to provide electricity, flood protection, and irrigation. The type of the dam is earth fill. Total body volume of the dam is 14.50 hm^3 and the height of the dam is 70 meters. The capacity of the reservoir of the dam is 1629 hm^3 and its surface area is 69.25 km^2 at normal water level. Total length of the crest is 894 meters and width of the crest is 12 meters. There are 2 diversion tunnels located on left side with dimensions of 8,50 m in diameter and 1777 m in length. The annual energy production of the dam is 596 GWh and total irrigation area which is provided by the dam is 1870 ha.

The location of the dam is at the upstream of Seyhan Dam. The main purpose of construction of the dam is to improve the flood control capacity at the Seyhan River. The dam is approximately 22 km north of Adana City.

4. METHODOLOGY

In the scope of this thesis work, by using HEC-RAS analysis program, the multiple failures of Seyhan Dam and Çatalan Dam with respect to predetermined failure cases and conditions will be investigated. HEC-RAS software program, model features, external data, and internal data implemented into the model will be discussed in detail in the following sections.

4.1. Dam Break Flood Prediction Tools

There are many dam break flood analysis programs available in order to predict the possible outcome of the flood and its features. By improving the methods and capabilities of these software programs, it has been brought to mind how these programs are reliable. Therefore, to prove their high consistency and feasibility, there have been some studies conducted. For instance, one of the laboratory studies conducted by Bellos, 2004, is an experimental study of the transmission of the wave in converging-diverging laboratory flume. The measurements of the flow depths were performed indirectly and were located at 10 cross-sections of the channel. Comparisons between actual measurements and numerical computations showed a satisfying agreement [16].

Dam break flood analysis programs use different methods. U.S. Bureau of Reclamation grouped the analysis methods into four categories: 1) Physically based methods; 2) Parametric models; 3) Predictor equations; 4) Comparative analysis [17]. These analysis methods conclude that the differences in the interpretation and approximation of river geometry are the primary source of the differences in the model simulations, although the numerical algorithms for solution of the St. Venant Equations are similar [18].

According to their types and application areas, Federal Emergency Management Agency (FEMA), categorize the peak breach discharge prediction and downstream routing of breach hydrograph analysis programs under three different titles in terms of their levels. The table below provides guidance to determine the levels for analysis for dam failure

inundation modeling and mapping. This table is arranged similarly to some State-developed tiered analysis structures, providing a logical combination of methods to perform an analysis [11].

Table 4.1. Tiered Approach Dam Breach Inundation Mapping for use in EAPs [11].

Tier Level	Applicable to	Breach Parameter Prediction	Peak Breach Discharge Prediction	Downstream Routing of Breach Hydrograph
Tier 1 –Basic level Screening and Simple Analysis	• Low-hazard potential / small size • First level screening for significant- or high-hazard dams	Empirical Equations	Simplified Models (SMPDBK, GeoDam-BREACH, or Technical Release [TR]-66) or HEC-HMS	GeoDam-BREACH, SMPDBK, DSAT,1D HEC-RAS Steady State, or HEC-HMS Hydrologic Routing
Tier 2 – Intermediate	• Significant-hazard potential / intermediate size • High-hazard dams with limited population at risk	Empirical Equations	HEC-HMS or HEC-RAS Unsteady Model	HEC-RAS (Steady or Unsteady Modeling) 1-D or 2-D models
Tier 3 – Advanced	• High-hazard potential / large size dams with sufficient population at risk to justify advanced analyses	Empirical Equations, NWS BREACH, or WinDAM	HEC-RAS Unsteady Model	HEC-RAS Unsteady Model or 2-D models

As table provided by FEMA, within the scope of this thesis work as a dam break flood prediction tool, HEC-RAS unsteady flow simulation program is used. HEC-RAS is designed to perform one-dimensional hydraulic calculations for both natural and

constructed channels. Unsteady flow simulation component of HEC-RAS is capable of simulating one-dimensional unsteady flow through a full network of open channels. The unsteady flow component can perform subcritical and mixed flow regime calculations.

4.1.1. HEC-RAS Analysis System

HEC-RAS is one of the most applied hydraulic models in dam break flood analysis. HEC-RAS is a hydraulic model developed by the US Army Corps of Engineers Hydrologic Engineering Centre. The first version of HEC-RAS was developed in 1990 and evolved from a steady flow model called HEC-2, first developed in 1966 (HEC, 2006). As computer capabilities improved, the HEC-2 software was converted to the windows-based HEC-RAS software to better assist hydraulic modeling with a graphical user interface (GUI) [19]. HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking environment. The system is comprised of a GUI, separate analysis components, data storage and management capabilities, graphics and reporting facilities [20].

The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction and expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situation where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (i.e., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions) [20]. The HEC-RAS model was initially used for calculating water surface profiles for one dimensional steady state flow. The results from the model have been applied to flood management and flood insurance studies throughout the USA [19].

HEC-RAS is currently capable of performing one-dimensional water surface profile calculation for steady gradually varied flow in natural or constructed channels. Subcritical, supercritical and mixed regime water surface profile can be calculated. Along with the unsteady and steady flow options, the HEC-RAS model also provides the following capabilities [19]:

- modeling of open channel networks and single rivers (both unsteady and steady flow options)
- analysis of bridges, weirs, and culverts (unsteady and steady flow options)
- modeling storage areas, navigation dams, tunnels, pumping stations, and levee failures (unsteady flow option only)
- Handling of subcritical, supercritical, and mixed-flow regimes

4.1.1.1. Steady Flow Water Surface Profiles. Water surface profiles are computed from one cross section to the next by solving the Energy equation with an iterative procedure called the standard step method. The energy equation is written as follows [20]:

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e \quad (4.1)$$

Where Z_1 and Z_2 are elevation of the main channel inverts, Y_1 and Y_2 are depth of the water at cross sections, V_1 and V_2 are average velocities (total discharge / total flow area), a_1 and a_2 are velocity weighting coefficients, g is gravitational acceleration, h_e is energy head loss.

The energy head loss (h_e) between two cross sections is comprised of friction losses and contraction or expansion losses. The equation for the energy head loss is as follows [20];

$$h_e = L \bar{S}_f + C \left| \frac{a_2 V_2^2}{2g} - \frac{a_1 V_1^2}{2g} \right| \quad (4.2)$$

Where L is discharge weighted reach length, $\bar{S}_f$ is representative friction slope between two sections, C is expansion or contraction loss coefficient.

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

$$L = \frac{L_{lob} \bar{Q}_{lob} + L_{ch} \bar{Q}_{ch} + L_{rob} \bar{Q}_{rob}}{\bar{Q}_{lob} + \bar{Q}_{ch} + \bar{Q}_{rob}} \quad (4.3)$$

Where L_{lob} , L_{ch} , L_{rob} are cross section reach lengths specified for flow in the left overbank, main channel, and right overbank respectively, $\bar{Q}_{lob}$, $\bar{Q}_{ch}$, $\bar{Q}_{rob}$ are arithmetic average of the flows between sections for the left overbank, main channel, and right overbank respectively.

For the calculation of discharge in the channel, the method used by HEC-RAS is to subdivide flows into units for which the velocity is uniformly distributed (Figure 4.1). The conveyance is calculated for each of subdivision from the Manning's equation;

$$Q = KS_f^{1/2} \quad (4.4)$$

$$K = AR^{2/3}/n \quad (4.5)$$

Where K is conveyance for subdivision, n is Manning's roughness coefficient for subdivision, A is flow area for subdivision, and R is hydraulic radius for subdivision (area /wetted perimeter).

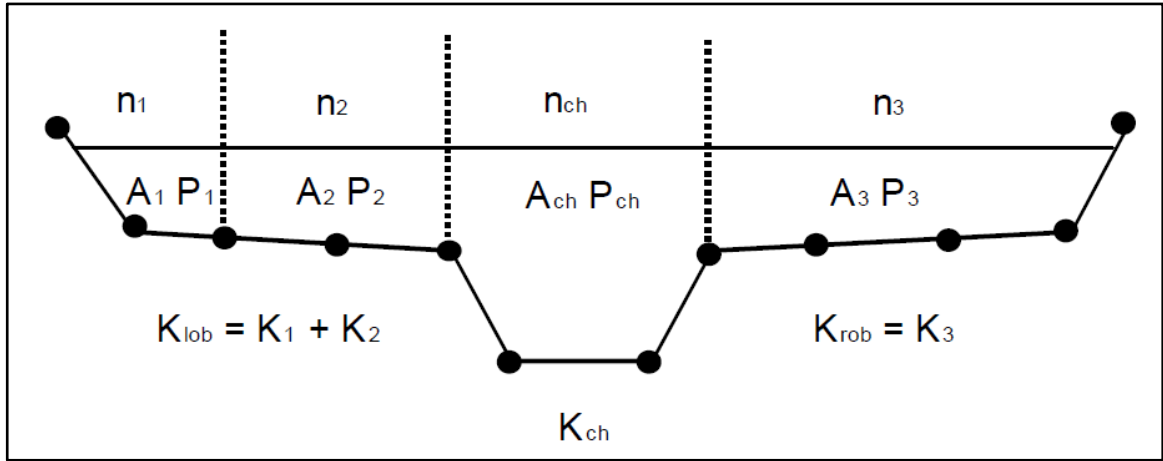


Figure 4.1. HEC-RAS Default Conveyance Subdivision Method [20].

For a given cross section of a river the mean energy is obtained by calculating the flow weighted energy of three divided subsections of the river cross sections. These three subsections are left overbank, main channel, and right overbank (Figure 4.2).

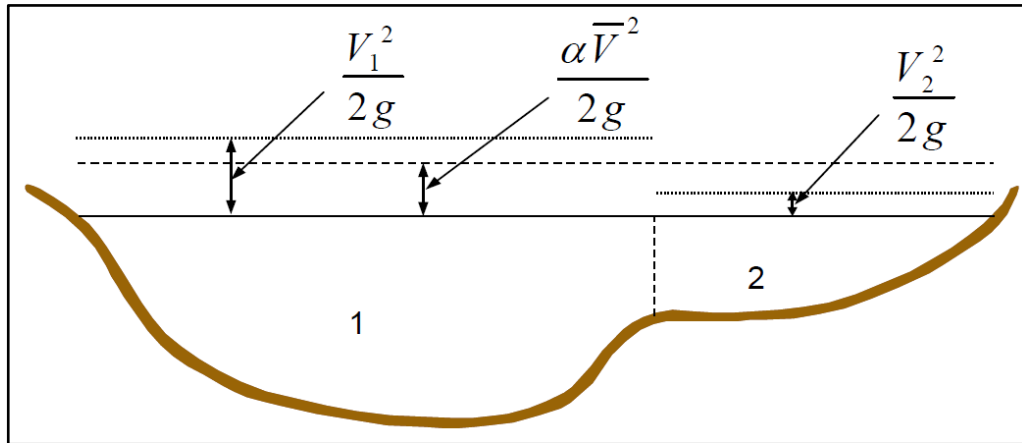


Figure 4.2. Method of Mean Energy Calculation [20].

Where V_1 is mean velocity for subsection 1, V_2 is mean velocity for subsection 2.

In order to calculate the mean kinetic energy the velocity head weighting coefficient alpha should be obtained. It is calculated as follows:

$$a \frac{\bar{V}^2}{2g} = \frac{Q_1 \frac{V_1^2}{2g} + Q_2 \frac{V_2^2}{2g}}{Q_1 + Q_2} \quad (4.6)$$

In general:

$$a = \frac{Q_1 V_1^2 + Q_2 V_2^2 + \dots + Q_N V_N^2}{Q \bar{V}^2} \quad (4.7)$$

Generally in steady flow analysis there are some main assumptions included in the calculations. These are listed as follows [20]:

- Flow is steady and the properties in each direction in the flow do not change with time.
- Flow is gradually changed.
- Flow is one dimensional. In other words the velocity components in directions other than the direction of flow are not accounted for.
- River channels have mild slopes.

4.1.1.2. Unsteady Flow Routing. When the river is rising water moves laterally away from the channel, inundating the floodplain and filling available storage areas. As the depth increases, the floodplain begins to convey water downstream generally along a shorter path than that of the main channel. When the river stage is falling, water moves toward the channel from the overbank supplementing the flow in the main channel. Because the primary direction of flow is oriented along the channel, this two-dimensional flow field can often be accurately approximated by one-dimensional representation. Off-channel ponding areas can be modeled with storage areas that exchange water with the channel. Flow in the overbank can be approximated as flow through a separate channel [20].

This channel/floodplain problem has been addressed in many different ways. A common approach is to ignore overbank conveyance entirely, assuming that the overbank is used only for storage. This assumption may be suitable for large streams such as the Mississippi River where the channel is confined by levees and the remaining floodplain is either heavily vegetated or an off-channel storage area. Fread (1976) and Smith (1978) approached this problem by dividing the system into two separate channels and writing continuity and momentum equations for each channel. To simplify the problem they assumed a horizontal water surface at each cross section normal to the channel and the floodplain was negligible and that the discharge was distributed according to conveyance, i.e. [20]:

$$Q_c = \Phi q \quad (4.8)$$

Where Q_c is flow in channel and q is total flow.

$$\Phi = K_c / (K_c + K_f) \quad (4.9)$$

Where K_c is conveyance in the channel and K_f is conveyance in the floodplain.

With these assumptions, the one-dimensional equations of motions can be combined into a single set:

$$\frac{\partial A}{\partial t} + \frac{\partial(\Phi Q)}{\partial X_c} + \frac{\partial[(1-\Phi)Q]}{\partial X_f} = 0 \quad (4.10)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial(\Phi^2 Q^2 / A_c)}{\partial X_c} + \frac{\partial[(1-\Phi)^2 Q^2 / A_f]}{\partial X_f} + gA_c \left[\frac{\partial Z}{\partial X_c} + S_{fc} \right] + gA_f \left[\frac{\partial Z}{\partial X_f} + S_{ff} \right] = 0 \quad (4.11)$$

In which the subscripts c and f refer to the channel and floodplain, respectively. These equations were approximated using implicit finite differences, and solved numerically using the Newton-Raphson iteration technique. The model was successful and produced the desired effects in the test problems [20].

The most successful and accepted procedure for solving the one-dimensional unsteady flow equations is the four-point implicit scheme, also known as the box scheme (Figure 4.3) [20].

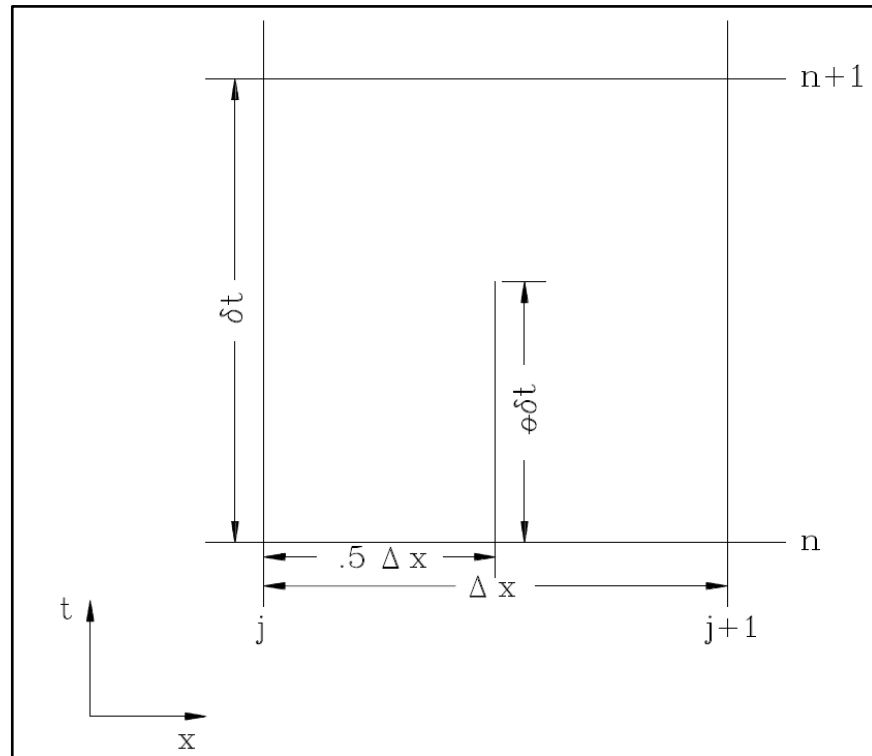


Figure 4.3. Finite Difference Cell [20].

The following notation is defined;

$$f_j = f_j^n \quad (4.12)$$

$$\Delta f_j = f_j^{n+1} - f_j^n \quad (4.13)$$

Time derivative;

$$\frac{\partial f}{\partial t} \approx \frac{\Delta f}{\Delta t} = \frac{0.5(\Delta f_{j+1} + \Delta f_j)}{\Delta t} \quad (4.14)$$

Space derivative;

$$\frac{\partial f}{\partial x} \approx \frac{\Delta f}{\Delta x} = \frac{(f_{j+1} - f_j) + \theta(\Delta f_{j+1} - \Delta f_j)}{\Delta x} \quad (4.15)$$

Function value;

$$f \approx \bar{f} = 0.5(f_j + f_{j+1}) + 0.5\theta(\Delta f_j + \Delta f_{j+1}) \quad (4.16)$$

The continuity equation can be written as;

$$\frac{\partial A}{\partial t} + \frac{\Delta S}{\Delta t} + \frac{\partial Q}{\partial x} - q_i = 0 \quad (4.17)$$

Where x is distance along the channel, t is time, Q is flow, A is cross-sectional area, S is storage from non-conveying portions of cross section and q_i is lateral inflow per unit distance.

The equation above can be written for the channel and the floodplain;

$$\frac{\partial Q_c}{\partial x_c} + \frac{\partial A_c}{\partial t} = q_f \quad (4.18)$$

$$\frac{\partial Q_f}{\partial x_f} + \frac{\partial A_f}{\partial t} + \frac{\partial S}{\partial t} = q_c + q_i \quad (4.19)$$

The subscripts c and f refer to the channel floodplain, respectively, q_i is the lateral inflow per unit length of floodplain, and q_c and q_f are the exchanges of water between the channel and the floodplain [20].

For continuity equation above equations can be approximated using implicit finite differences by applying time derivative, space derivative, and function value equations.

$$\frac{\Delta Q_c}{\Delta x_c} + \frac{\Delta A_c}{\Delta t} = \bar{q}_f \quad (4.20)$$

$$\frac{\Delta Q_f}{\Delta x_c} + \frac{\Delta A_c}{\Delta t} + \frac{\Delta S}{\Delta t} = \bar{q}_c + \bar{q}_i \quad (4.21)$$

Rearranging the above equations;

$$\Delta Q + \frac{\Delta A_c}{\Delta t} \Delta x_c + \frac{\Delta A_f}{\Delta t} \Delta x_f + \frac{\Delta S}{\Delta t} \Delta x_f - \bar{Q}_i = 0 \quad (4.22)$$

Where $\bar{Q}_i$ is average lateral inflow.

For momentum equation, it is stated that the rate of change in momentum is equal to the external forces acting on the system. For a single channel [20];

$$\frac{\partial Q}{\partial t} + \frac{\partial(VQ)}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0 \quad (4.23)$$

Where g is acceleration of gravity, S_f is friction slope and V is velocity

The equation above can be written for the channel and the floodplain;

$$\frac{\partial Q_c}{\partial t} + \frac{\partial(V_c Q_c)}{\partial x_c} + gA_c\left(\frac{\partial z}{\partial x_c} + S_{fc}\right) = M_f \quad (4.24)$$

$$\frac{\partial Q_f}{\partial t} + \frac{\partial(V_f Q_f)}{\partial x_f} + g A_f \left(\frac{\partial z}{\partial x_f} + S_{ff} \right) = M_c \quad (4.25)$$

Where M_c and M_f are the momentum fluxes per unit distance exchanged between the channel and floodplain, respectively [20].

Above equations can be approximated using implicit finite differences by applying time derivative, space derivative, and function value equations.

$$\frac{\Delta Q_c}{\Delta t} + \frac{\Delta(V_c Q_c)}{\Delta x_c} + g \bar{A}_c \left(\frac{\Delta z}{\Delta x_c} + \bar{S}_{fc} \right) = M_f \quad (4.26)$$

$$\frac{\Delta Q_f}{\Delta t} + \frac{\Delta(V_f Q_f)}{\Delta x_f} + g \bar{A}_f \left(\frac{\Delta z}{\Delta x_f} + \bar{S}_{ff} \right) = M_c \quad (4.27)$$

Note that;

$$\Delta x_c M_c = -\Delta x_f M_f \quad (4.28)$$

Adding and rearranging the above equations yields;

$$\begin{aligned} & \frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t} + \Delta(V_c Q_c) + \Delta(V_f Q_f) \\ & + g(A_c + A_f) \Delta z + g \bar{A}_c \bar{S}_{fc} \Delta x_c + g \bar{A}_f \bar{S}_{ff} \Delta x_f = 0 \end{aligned} \quad (4.29)$$

The final two terms define the friction force from the banks acting on the fluid. An equivalent force can be defined as;

$$g \bar{A} \bar{S}_f \Delta x_e = g \bar{A}_c \bar{S}_{fc} \Delta x_c + g \bar{A}_f \bar{S}_{ff} \Delta x_f \quad (4.30)$$

Where Δx_e is equivalent flow path and S_f is friction slope for the entire cross section

$$A = \bar{A}_c + \bar{A}_f \quad (4.31)$$

Now, the convective terms can be rewritten by defining a velocity distribution factor;

$$\beta = \frac{V_c^2 A_c + V_f^2 A_f}{V^2 A} = \frac{V_c Q_c + V_f Q_f}{QV} \quad (4.32)$$

$$\Delta(\beta VQ) = \Delta(V_c Q_c) + \Delta(V_f Q_f) \quad (4.33)$$

The final form of the momentum equation is;

$$\frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t \Delta x_e} + \frac{\Delta(\beta VQ)}{\Delta x_e} + g \bar{A} \left(\frac{\Delta z}{\Delta x_e} + \bar{S}_f \right) = 0 \quad (4.34)$$

For a differential distance, dx , the additional forces in the contraction produce a swell head of dh_l . This swell head is only related to the additional forces. The rate of energy loss can be expressed as a local slope [20];

$$S_h = \frac{dh_l}{dx} \quad (4.35)$$

Therefore the final form of the momentum equation can be expanded by this term;

$$\frac{\partial Q}{\partial t} + \frac{\partial(VQ)}{\partial x} + gA \left(\frac{\partial z}{\partial x} + S_f + S_h \right) = 0 \quad (4.36)$$

The result is inserted in the finite difference form of the momentum equation;

$$\frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t \Delta x_e} + \frac{\Delta(\beta VQ)}{\Delta x_e} + g \bar{A} \left(\frac{\Delta z}{\Delta x_e} + \bar{S}_f + \bar{S}_h \right) = 0 \quad (4.37)$$

At stream junctions, the momentum as well as the mass of the flow from a tributary enters the receiving stream. The entering momentum is given by [20];

$$M_l = \zeta \frac{Q_l V_l}{\Delta x} \quad (4.38)$$

Where Q_l is lateral inflow, V_l is average velocity of lateral inflow and ζ is fraction of the momentum entering the receiving stream.

The entering momentum is added to the right side of the final form of finite difference form of the momentum equation;

$$\frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t \Delta x_e} + \frac{\Delta(\beta V Q)}{\Delta x_e} + g \bar{A} \left(\frac{\Delta z}{\Delta x_e} + \bar{S}_f + \bar{S}_h \right) = \zeta \frac{Q_l V_l}{\Delta x_e} \quad (4.39)$$

This equation is only used at stream junctions in a dendritic model in HEC-RAS [20].

4.1.1.3. Boundary Conditions. To start the program to calculate, boundary conditions, which are define the initial water level at the end of the river, are needed. There are several options can be used in order to determine the boundary condition. The first option is critical depth. The critical depth boundary condition can be selected especially if there are control structures such as weir, gate, penstocks, dropt, etc. that controls and forces the critical depth. Another option for determining the boundary condition is rating curve. In this choice the water level is inserted from the given rating curve. Generally this case applies for control station where water levels and discharges are measured continually. In addition to these two options, normal depth method is also used in order to define the boundary condition. Normal depth is the most used boundary condition. In this option, the energy slope is used to calculate the normal depth by using Manning's equation. It is usually approximated by using channel slope or the water surface slope at the vicinity of the boundary station. In order to minimize the margin of error, it is highly recommended to increase the number of cross sections taken from the river.

4.1.1.4. Basic Data Requirements. There are five main data requirements in HEC-RAS. These are listed as follows;

- The River System Schematic:
 - (i) The river system schematic is required for any geometric data set within HEC-RAS system. The schematic defines how the various river reaches are connected, as well as establishing a naming convention for referencing all the

other data. The connectivity of reaches is very important in order for the model to understand how the computations should proceed from one reach to the next. The user is required to draw each reach from upstream to downstream, in what is considered to be the positive flow direction. The connecting of reaches is considered a junction. Junctions should only be established at locations where two or more streams come together or split apart. Junctions cannot be established with a single reach flowing into another single reach. These two reaches must be combined and defined as one reach. [20].

- Cross Section Geometry:

- (i) The general approach to laying out cross sections is to ensure that the cross sections are perpendicular to the flow lines. This requires an estimation of what the flow lines will look like in the overbank areas away from the main channel. One option is to draw a stream center line down the main channel along what is perceived to be the center of mass of flow. The same thing should be done for the left overbank and right overbank [20].
- (ii) Cross sections are required at representative locations throughout a stream reach and at locations where changes occur in discharge, slope, shape, or roughness, at locations where levees begin or end and at bridges or control structures such as weirs. Where abrupt changes occur, several cross sections should be used to describe the change regardless of the distance. Cross section spacing is also a function of stream size, slope, and the uniformity of cross section shape [20].
- (iii) In this thesis work in order to have more accurate cross sectional information, topographical maps of the dam regions that are obtained from Adana Municipal are used. The topographical maps of the regions are provided in NetCad format by the Municipal. The Netcad files are imported into ArcGIS program. As a final step, these ArcGIS formatted data are used in HEC-RAS. By using these ArcGIS data, HEC-RAS can be read exact cross sectional information.

- Reach Lengths:

- (i) The measured distances between cross sections are referred to as reach lengths. The reach lengths for the left overbank, right overbank and channel are specified on the cross section data editor. Channel reach lengths are typically measured along thalweg. Overbank reach lengths should be measured along the anticipated path of the center of mass of the overbank flow [20].

- Energy Loss Coefficients:

- (i) In HEC-RAS three types of loss coefficients are taken into account. These are Manning's coefficient n for friction loss, losses due to contraction or expansion coefficients, and bridge and culvert loss coefficients for the losses associated to weir shape, pier configuration, entrance and exit conditions, etc. [20].

- Stream Junction Data:

- (i) Stream junctions are defined as locations where two or more streams come together or split apart. In general, the cross sections that bound a junction should be placed as close together as possible. This will minimize the error in the calculation of energy losses across the junction. In HEC-RAS a junction can be modeled by either the energy equation or the momentum equation. The energy equation does not take into account the angle of any tributary coming in or leaving the main stream, while the momentum equation does. In most cases, the amount of energy loss due to angle of the tributary flow is not significant, and using the energy equation to model the junction is more than adequate [20].

4.2. Parameters Adopted In Use of Modeling

In constructing the numerical HEC-RAS model it is need to determine the external and internal data that specifies the properties and magnitude of the outputs. The external data of the model basically refer to the characteristics of the river, the dams, and the affected areas whereas the internal data of the model are initial conditions of the model and additionally the breach parameters of the dams.

4.2.1. Evaluation of External Data

The external data of the model are mainly extracted from the data requested from Adana Municipal that are consists of topographic maps of the considered areas and the features of the river. At the point where the information provided by the municipal is not adequate, various sources are used in order to determine the Manning roughness.

4.2.1.1. Cross Sectional Data. The cross sectional data of the model are obtained by using Arc-GIS model and RAS mapper feature of the HEC-RAS program. To acquire the cross sectional view first of all the topographic map of the terrain are transferred into the Arc-GIS software program. After this transformation the topographical map are installed the RAS mapper program of the HEC-RAS. Finally the cross sectional views are saved into HEC-RAS model by drawing section cuts at every certain intervals (Figure 4.4). At some locations, to reduce the uncertainties the distances between the cross sections are tried to be small as much as possible. Because the topographic elevations are already determined thanks to the map, the cross sectional details are plotted automatically, therefore there is no need to modify the cross sections manually. While constructing the cross sections of the river, it is vital to make section cuts extent through all the areas can be affected during any potential inundation due to dam break scenarios.

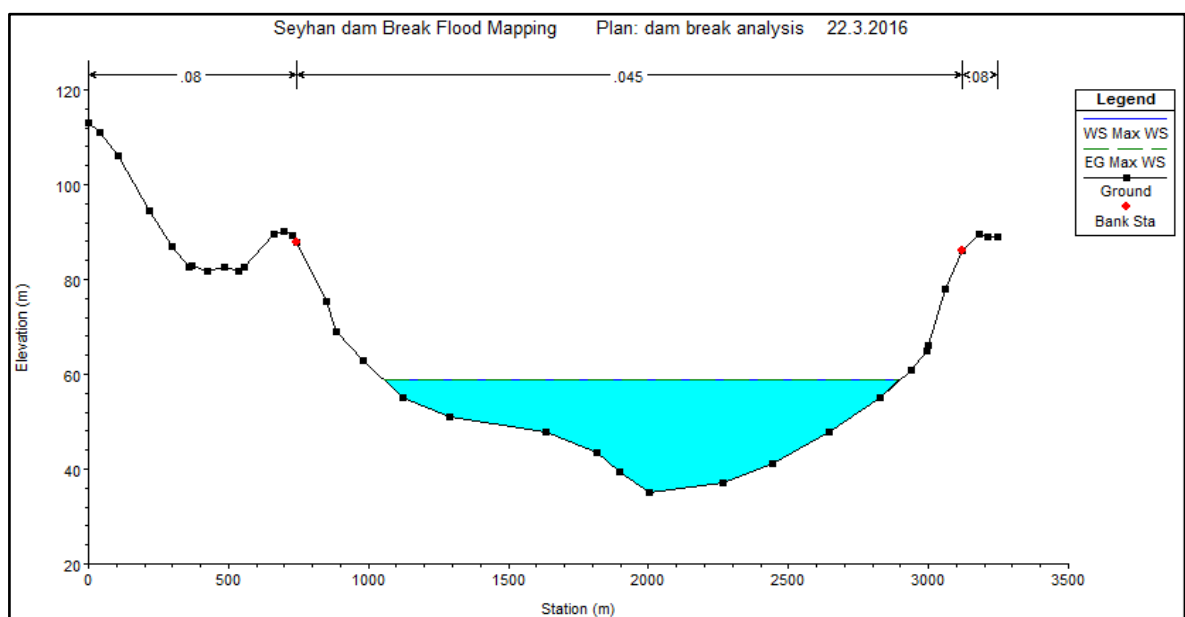


Figure 4.4. Example of a Cross Section.

4.2.1.2. Manning Roughness. Manning Roughness is the representation of resistance to flood in channels and flood plains. Values of roughness change with respect to many factors. The most important factors affect the value of roughness value are surface roughness of the bed material, cross section of the river, vegetation characteristics, and channel slope. Roughness coefficient, n value cannot represent resistance characteristics for the debris flow and mudflows accurately. In the scope of this thesis the roughness coefficient values are adopted from the open channel hydraulics book of Chow [53] for natural stream table and for flood plain. For the stream channel roughness coefficient is taken as 0.03 for the criteria of normal clean, straight, full stage, no rifts or deep pools, D-1a option from the table. Whereas for the left and right overbanks, roughness coefficient is taken as 0.045 for the criteria of maximum mature crops area, D-2b option from the table. (Table B.1).

In the study conducted by Bozkuş and Kasap, 1997 it is shown that the manning coefficient, n does not change the computed peak water surface elevations significantly. However it has a significant effect on the peak occurrence times. The reason for the increase in peak depths at the cross sections to some degree is the fact that a higher n value means more roughness, and hence more resistance to the flow. This, in turn, means less discharge passing a particular cross section. Consequently, this increases the water surface elevations at the stations and causes the peak depths to occur for longer times [21].

4.2.2. Evaluation of Internal Data

In HEC-RAS each reach has an upstream and a downstream boundary condition. Connections to junctions are considered internal boundary conditions. Internal boundary conditions are automatically listed in the list, based on how the river system is connected in the geometric data editor.

4.2.2.1. Initial Conditions. In the process of preparing HEC-RAS model, it is required to determine the initial conditions at all nodes constructed at the beginning of the modeling. These initial conditions can be determined by entering flow or stage data. The most preferred method to determine the initial condition is to enter flow data for every reach and then by performing a steady flow backwater analysis to compute water surface elevations. A second method can only be done if a previous run was made. This method

allows the user to write a file of flow and stage from a previous run, which can then be used as the initial conditions for a subsequent run. In addition to establish the initial conditions within the river system, the starting water surface elevation must be defined in any storage areas [20].

4.2.2.2. Breach Parameters. Simulation of embankment dam breach events and the resulting floods are crucial to characterizing and reducing threats due to potential dam failures. Development of effective emergency action plans requires accurate prediction of inundation levels and the time of flood wave arrival at a given location. If population centers are located well downstream of a dam, details of the breaching process have little effect on the result; travel time, attenuation, and other routing effects predominate. However, in a growing number of cases, the location of population centers near a dam makes accurate prediction of breach parameters (e.g., breach width, depth, rate of development) (Figure 4.5) crucial to the analysis. If breach parameters cannot be predicted with reasonable accuracy, increased conservatism with associated increased costs may be required [22]. However according to the researches, breach simulation and breach parameter includes the greatest uncertainty of all aspects of dam break flood forecasting [23].

The variability of the dam material properties causes a large uncertainty in the estimation of the peak arrival time, as it controls the dam breach speed and process. However, it has little influence on the flood inundation area and flood depth downstream, which are more determined by the lake volume [24].

Variation of breach parameters can affect peak discharge and inundation levels, as well as warning and evacuation time. The effect on peak discharges was examined by K. Singh and Snorrason (1984) [49]. For small reservoirs (those which experience significant drawdown before the breach is fully developed), changes in breach formation time can dramatically affect peak outflow. Variations of breach width can also produce large changes in peak outflow, especially for large reservoirs. Variations of breach height have a relatively small effect on peak outflow. Petrascheck and Sydler (1984) [50] also demonstrated the sensitivity of peak flow, inundation levels, and flood arrival time to changes in breach width and breach formation time. For locations near a dam, both

parameters can have a dramatic influence. For locations well downstream of a dam, timing of the flood wave peak can change significantly with changes in breach formation time, but peak discharge and inundation levels are insensitive to changes in breach parameters [22].

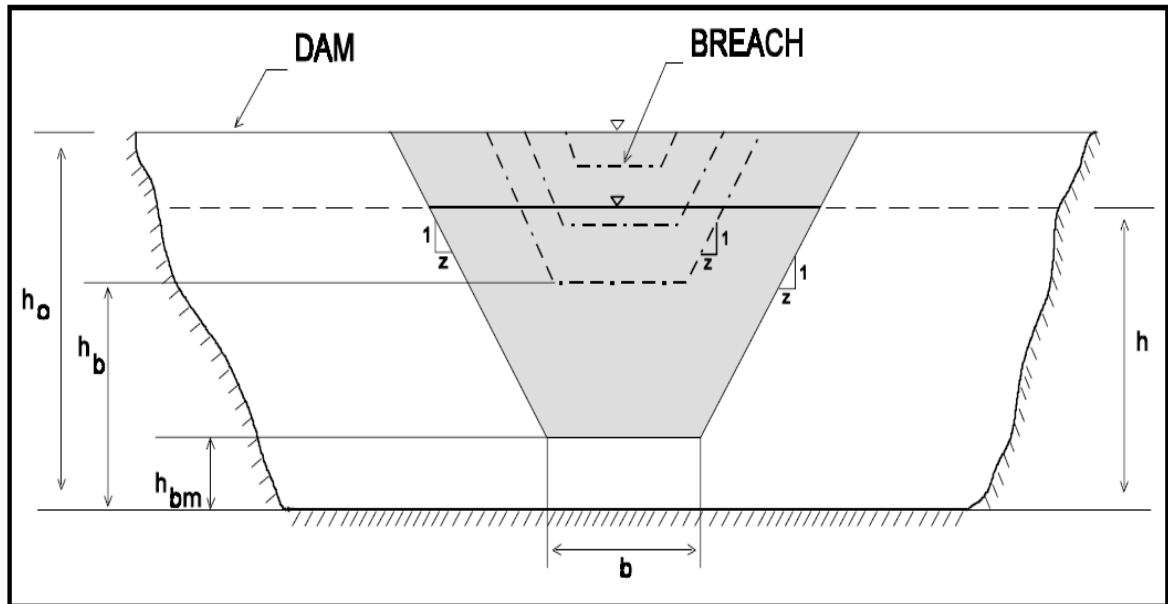


Figure 4.5. Dam Breach Formation [25].

Earthen dams which exceedingly outnumber all other types of dams do not tend to completely fail, nor do they fail instantaneously. The fully formed breach in earthen dams tends to have an average width in the range $(0.5 < b/H < 8)$. Breach widths for earthen dams are therefore usually much less than the total length of the dam as measured across the valley. The breach formation time is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is after the downstream face of the dam has eroded away and the resulting crevasse has eroded back across the width of the dam crest to reach the upstream face. This portion of the failure process could be thought of as the initiation time or time of failure (t_f). Time of failure (t_f) for overtopping initiated failures may be in the range of a few minutes to usually less than an hour, depending on the height of the dam, the type of materials, and the magnitude and duration of the overtopping flow of the escaping water [25]. Example of overtopping failure of an embankment laboratory dam model shown in Figure 4.6.

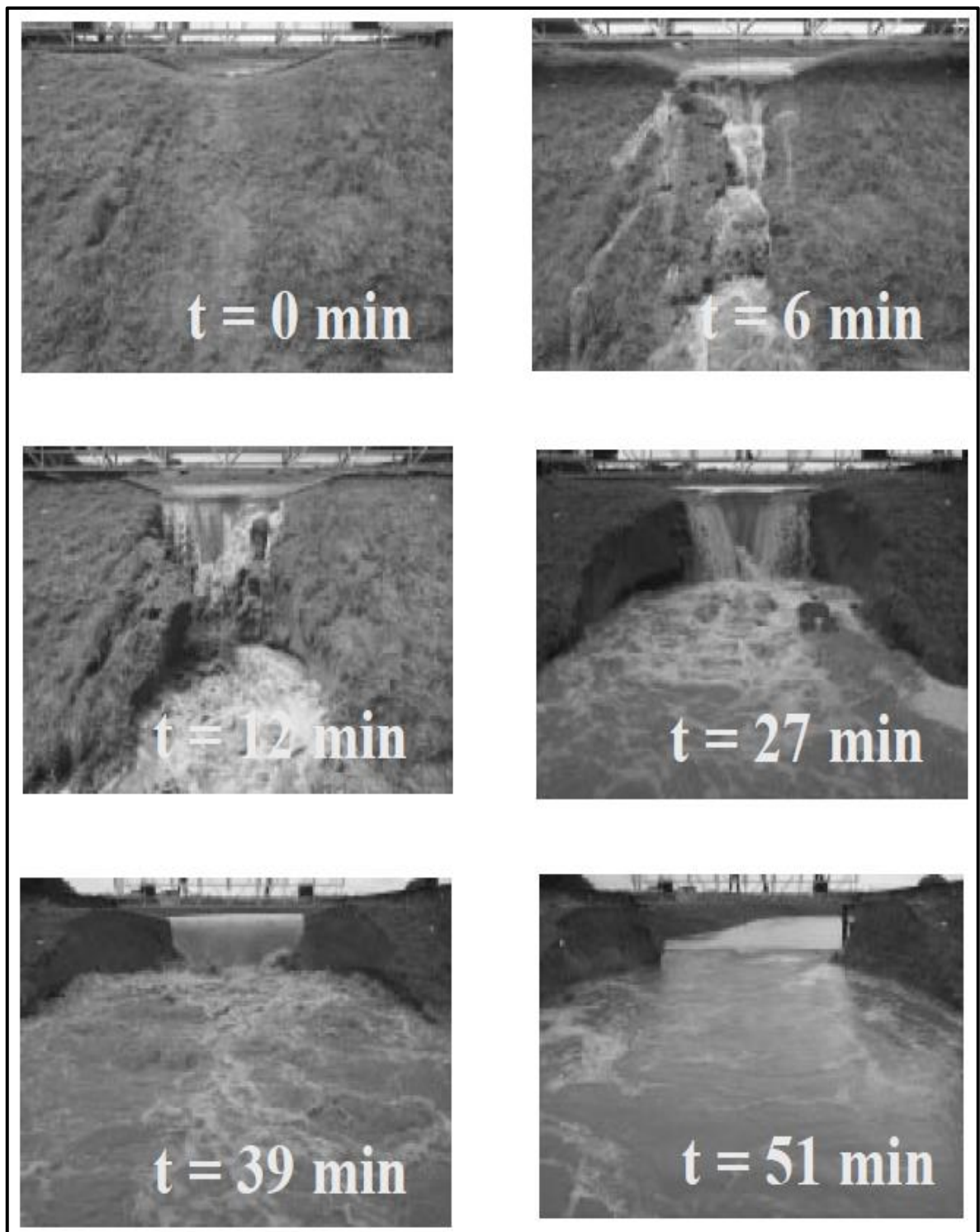


Figure 4.6. Time Series of an Embankment Breach Test of a Homogeneous Non Plastic Sandy Soil Conducted at the ARS Hydraulic Laboratory, Stillwater, OK. [27].

Table 4.2. Uncertainty Estimates of Breach Parameter and Peak Flow Prediction Equations. All Equations Use Metric Units (meters, m³, m³/s). Failure Times are Computed in Hours [26].

Equation	Number of Case Studies		Mean Prediction Error, $\bar{e}$ (log cycles)	Width of Uncertainty Band, $\pm 2S_e$ (log cycles)	Prediction interval around a hypothetical predicted value of 1.0
	Before outlier exclusion	After outlier exclusion			
BREACH WIDTH EQUATIONS					
<u>USBR (1988)</u> $\bar{B} = 3(h_w)$	80	70	-0.09	± 0.43	0.45 — 3.3
<u>MacDonald and Langridge-Monopolis (1984)</u> $V_{er} = 0.0261(V_w \cdot h_w)^{0.769}$ <i>earthfill</i> $V_{er} = 0.00348(V_w \cdot h_w)^{0.852}$ <i>non-earthfill (e.g., rockfill)</i>	60	58	-0.01	± 0.82	0.15 — 6.8
<u>Von Thun and Gillette (1990)</u> $\bar{B} = 2.5h_w + C_b$ <i>where C_b is a function of reservoir size</i>	78	70	+0.09	± 0.35	0.37 — 1.8
<u>Froehlich (1995b)</u> $\bar{B} = 0.1803K_o V_w^{0.32} h_b^{0.19}$ <i>where $K_o = 1.4$ for overtopping, 1.0 for piping</i>	77	75	+0.01	± 0.39	0.40 — 2.4
FAILURE TIME EQUATIONS					
<u>MacDonald and Langridge-Monopolis (1984)</u> $t_f = 0.0179(V_{er})^{0.364}$	37	35	-0.21	± 0.83	0.24 — 11.
<u>Von Thun and Gillette (1990)</u> $t_f = 0.015(h_w)$ <i>highly erodible</i> $t_f = 0.020(h_w) + 0.25$ <i>erosion resistant</i>	36	34	-0.64	± 0.95	0.49 — 40.
<u>Von Thun and Gillette (1990)</u> $t_f = \bar{B} / (4h_w)$ <i>highly erodible</i> $t_f = \bar{B} / (4h_w + 61)$ <i>erosion resistant</i>	36	35	-0.38	± 0.84	0.35 — 17.
<u>Froehlich (1995b)</u> $t_f = 0.00254(V_w)^{0.53} h_b^{-0.9}$	34	33	-0.22	± 0.64	0.38 — 7.3
<u>USBR (1988)</u> $t_f = 0.011(\bar{B})$	40	39	-0.40	± 1.02	0.24 — 27.
PEAK FLOW EQUATIONS					
<u>Kirkpatrick (1977)</u> $Q_p = 1.268(h_w + 0.3)^{2.5}$	38	34	-0.14	± 0.69	0.28 — 6.8
<u>SCS (1981)</u> $Q_p = 16.6(h_w)^{1.85}$	38	32	+0.13	± 0.50	0.23 — 2.4
<u>Hagen (1982)</u> $Q_p = 0.54(S \cdot h_d)^{0.5}$	31	30	+0.43	± 0.75	0.07 — 2.1
<u>Reclamation (1982)</u> $Q_p = 19.1(h_w)^{1.85}$ <i>envelope equation</i>	38	32	+0.19	± 0.50	0.20 — 2.1

The review by Wahl (2004) found that the best methods of breach width prediction (Reclamation 1988 [51]; Von Thun & Gillette 1990; Froehlich 1995b) had uncertainties of about $\pm 1/3$ order of magnitude, and the best predictions of breach time (Froehlich 1995b) had uncertainties of about $\pm 2/3$ order of magnitude [28]. Therefore in the scope of this

thesis work, in order to predict the dam breach parameters, Froehlich 1995b method is used. Comparison of observed breach widths with these three methods are shown in Figure 4.6.

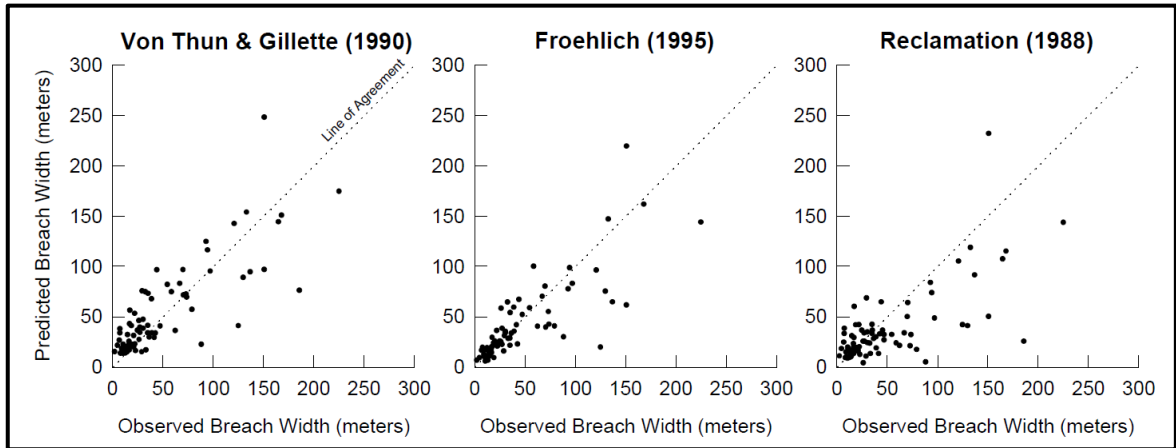


Figure 4.7. Predicted Breach Width versus Observed Breach Width [26].

In the Dam Safety Guidelines prepared by Washington State Department of Ecology, equations developed by MacDonald and Langridge-Monopolis [29]. These equations allow for failure analyses of embankment dams with varying geometry (slopes and crest widths). Specifically, the volume of material eroded from the breach is related to the Breach Formation Factor (*BFF*) [6]:

$$BFF = V_w (H_w) \quad (4.40)$$

Where V_w is volume of water stored in the reservoir (acre-ft) at the water surface elevation under consideration and H_w is height of water over the base elevation of the breach (ft).

Interpretation of the MacDonald and Langridge-Monopolis data suggests that estimates of the volume of material eroded from earthen dams comprised of cohesionless embankment materials may be taken to be:

$$V_m = 3.75 (BFF)^{0.77} \quad (4.41)$$

For erosion resistant embankment materials;

$$V_m = 2.50 (BFF)^{0.77} \quad (4.42)$$

Where V_m is volume of material in breach (yds³) eroded.

Therefore the base width of the breach can be computed as a function of the eroded volume of material as [6]:

$$W_b = \frac{27V_m - H_b^2 (CZ_b + H_b Z_b Z_3 / 3)}{H_b (C + H_b Z_3 / 2)} \quad (4.43)$$

Where W_b is width of breach (ft) at base elevation of breach, H_b is depth of breach (ft) from dam crest to base elevation of breach, C is crest width of dam (ft), Z_3 equals to $Z_1 + Z_2$, Z_1 is slope ($Z_1:1$) of upstream face of dam and Z_2 is slope ($Z_2:1$) of downstream face of dam.

For a rectangular breach, $Z_b = 0$, and Equation 3 reduces to

$$W_b = \frac{27V_m}{H_b (C + H_b Z_3 / 2)} \quad (4.44)$$

MacDonald and Monopolis [29] predicted the lower envelope of the breach development time in hours (τ) as a function of the volume of material (V_m) eroded during the breach [6]:

Breach Formation Time:

$$\text{Dams Composed of Cohesionless Materials} \quad \tau = 0.020 V_m^{0.36} \quad (4.45)$$

$$\text{Dams Composed of Erosion Resistant Materials} \quad \tau = 0.036 V_m^{0.36} \quad (4.46)$$

Where τ is the breach formation time (hours) and V_m is the volume of material (V_m) eroded during the breach (yds³).

4.2.2.3. Outflow Hydrograph. The outflow flood hydrograph from a dam failure is dependent upon many factors. The primary factors are the physical characteristics of the dam, the volume of the reservoir and the mode of failure. The parameters which control the magnitude of the peak discharge and the shape of the outflow hydrograph include: the breach dimensions; the manner and length of time for the breach to develop; the depth and volume of water stored in the reservoir; and the inflow to the reservoir at the time of failure. The shape and size of the breach and the elapsed time of development of the breach are in turn dependent upon the geometry of the dam, construction materials, and the causal agent for failure [6].

The peak outflow prediction studies according to three main features of dams are tabulated in Pierce et al., 2010 [30] (Table 4.3). These three main features are height of water behind the dam, storage of the dam, and together with height of water behind the dam and storage of the dam. The equations are derived with respect to these three aspects.

The most widely applied peak-flow prediction equations have been those of SCS (1981), MacDonald & Langridge-Monopolis (1984), Costa (1985), and Froehlich (1995a). An analysis by Wahl (2004) found the Froehlich (1995a) equation to have the lowest uncertainty of the peak flow prediction equations available at that time [28].

In the study of Fan X. et al., 2012 it is found that the outflow hydrograph is more sensitive to the grain size, the unit weight and the porosity (Fig. 4.8 A–C), while less sensitive to the internal friction angle and cohesion. Larger grain size has little effect on the peak discharge, but postpones the peak arrival time obviously, as it controls the sediment transport rate (Fig. 4.8 A). Lower values for the unit weight of the dam material lead to larger and faster outflow hydrograph peaks (Fig. 4.8 B). The same effect is obtained by increasing the porosity (Fig. 4.8 C) [24].

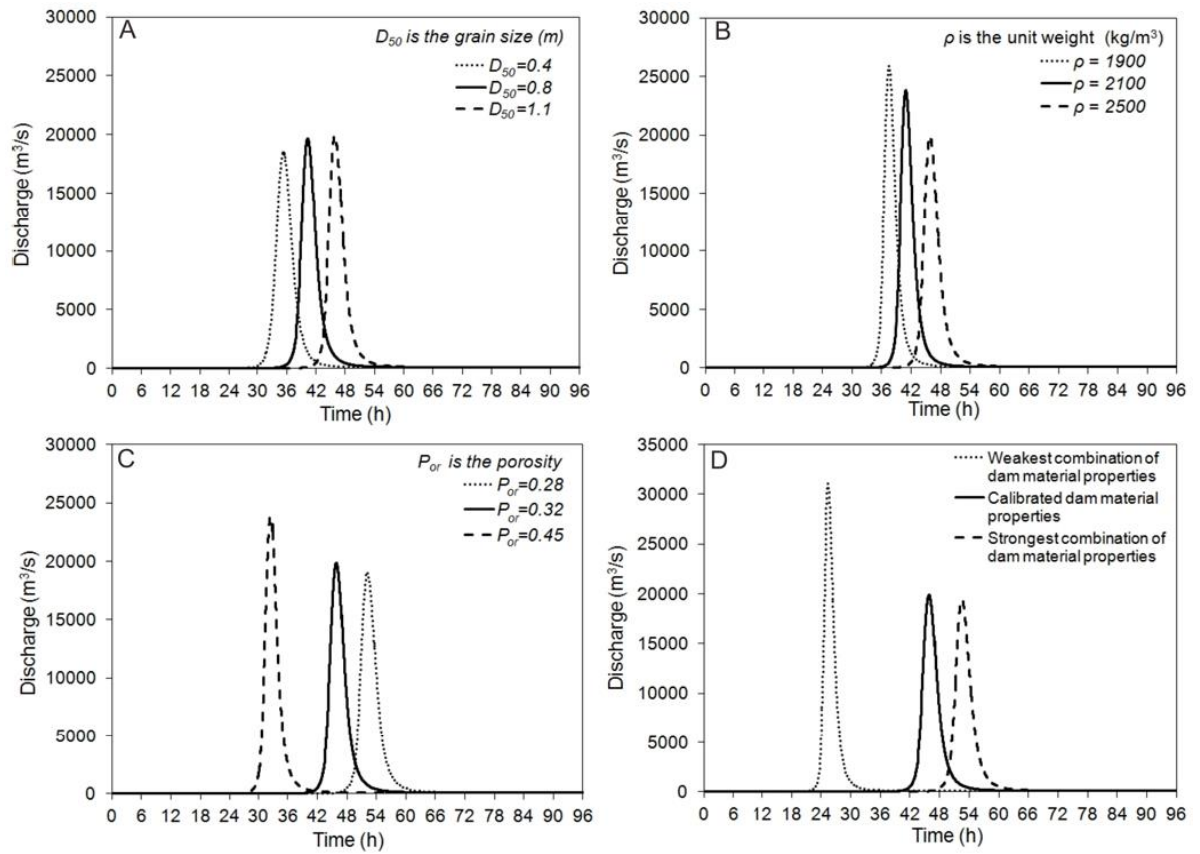


Figure 4.8. Sensitivity Analysis of the Outflow Hydrograph to the Material Properties of Tangjiashan Landslide Dam. (A: D_{50} Grain Size; B: Dam Material Unit Weight; and C: Porosity, and D: Uncertainty Analysis of the Model Outputs) [24].

5. SEYHAN AND ÇATALAN DAM BREAK ANALYSES

According to the guideline prepared by FEMA, dam breach inundation studies should continue to the point where adequate floodwater disposal is reached and the breach flood no longer poses a risk to life and property damage. The downstream extent of study should be established using the following criteria [11]:

- There are no habitable structures in the dam breach inundation zone, and anticipated future development in the floodplain is limited
- Dam breach flood flows are contained within a large downstream reservoir
- Dam breach flood flows are confined within the downstream channel or
- Dam breach flood flows enter a bay or ocean.

By taking into these considerations listed above, analysis domain is extended to the border of Gölbaşı at the west side of the region and to the Yukarı Çiçekli at the east side of the region.

In the final analysis, the parameters should be conservatively chosen after due consideration of the likely best estimates and how sensitive the final solution is to the parameters selected. To account for uncertainties in the analysis methods and site-specific conditions, 0.5 to 2.0 feet (0.15 to 0.61 meter) should be added to the computed dam break flood inundation elevations as a factor of safety. Lesser amounts may be warranted where shallow sheet flooding occurs and larger amounts would be appropriate where there is the potential for higher levels of debris build-up and sediment deposition [6].

After the dams and cross sectional information are entered into the model, critical locations are determined in order to provide extrapolated cross sections for more accurate and smooth analyses. The distance between the extrapolated cross sections are chosen to be 200 meters in order to have more detailed output. As shown from the profile view, measured cross sections and extrapolated cross sections are located one by one from upstream of Çatalan Dam to downstream of Seyhan Dam (Figure 5.1).

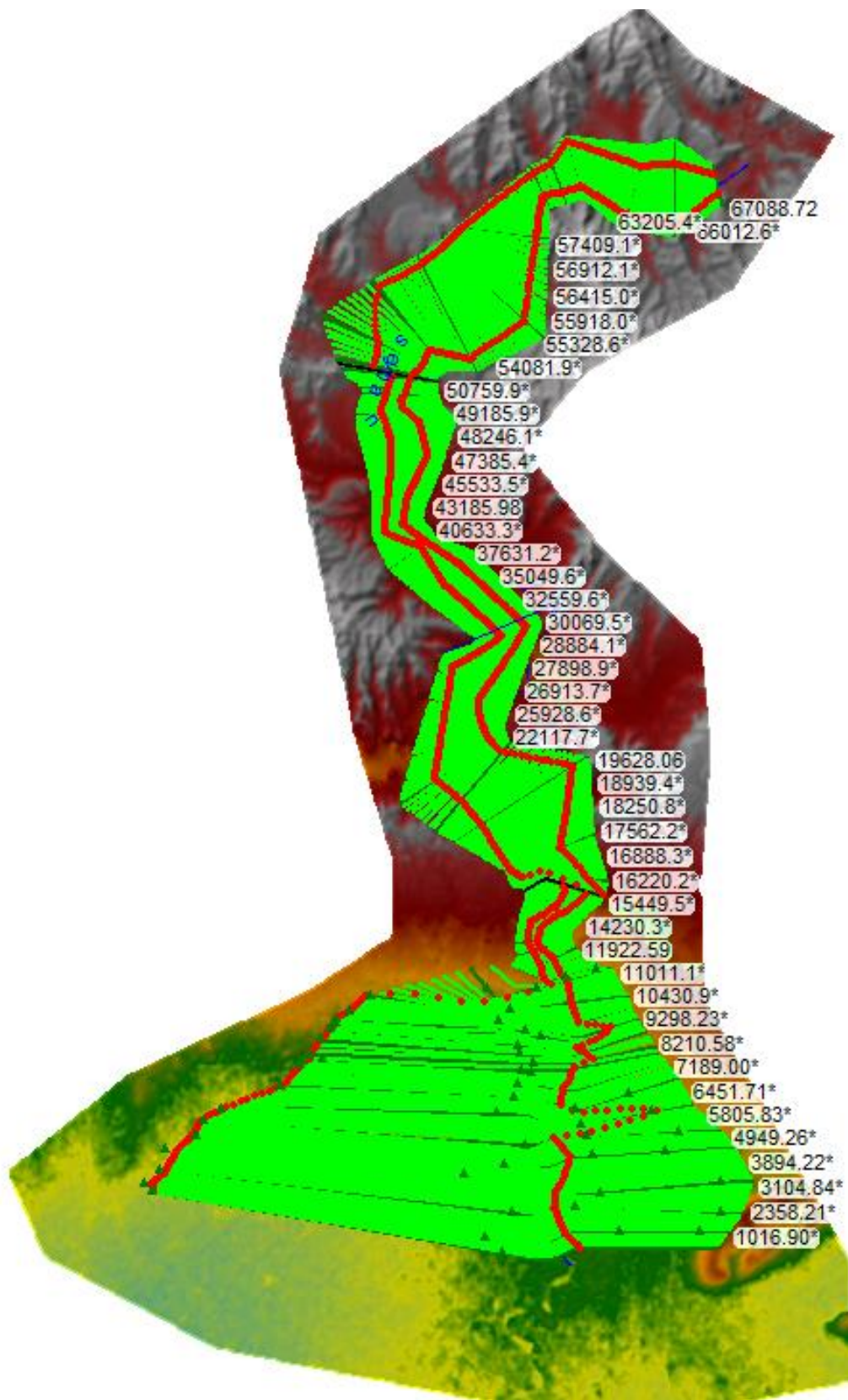


Figure 5.1. Plan View of the Model.

In order to have more accurate information about the area which is exposed to distribution of flow pattern, a map of the region which is provided by Adana Municipal is used as background for the model (Figure 5.2). By using this map, it is quite clear to mark the name of regions under threat.

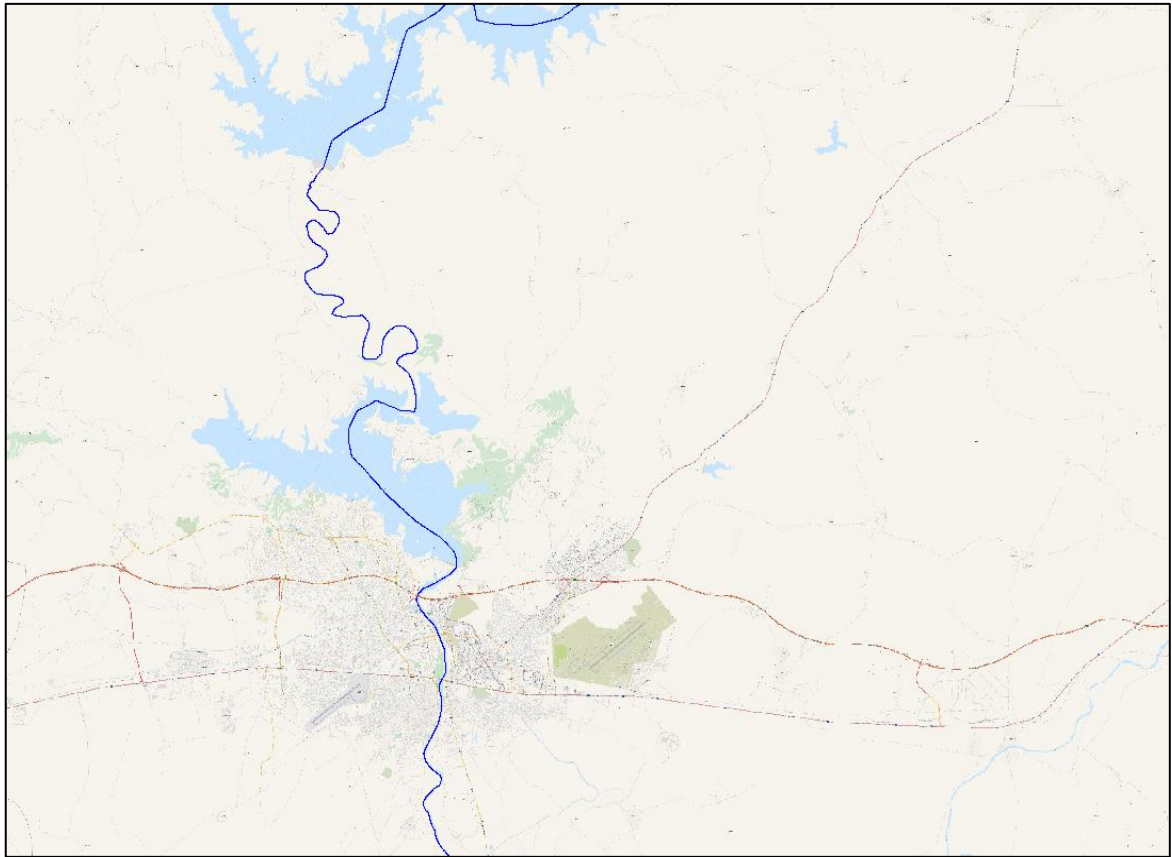


Figure 5.2. The map of the Region.

Seyhan Dam and Çatalan Dam are entered to the model as inline structures. In the inline structure editor of the model, all details and features related to the dams are elaborated. Moreover, breach parameters and spillway dimensions of the dams are also given in this section of the program. The height of Çatalan Dam extends from foundation with elevation of 65.14 m to the crest with elevation of 116 m. whereas the height of Seyhan dam extends from foundation with elevation of 33.39 m to the crest with elevation of 56 m.

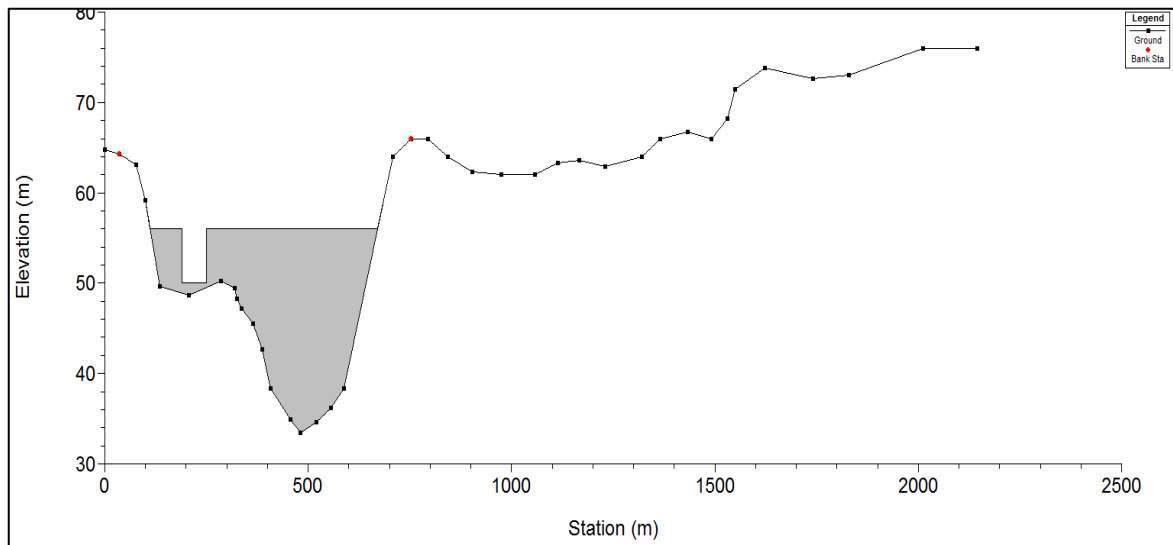


Figure 5.3. Seyhan Dam Cross Sectional View.

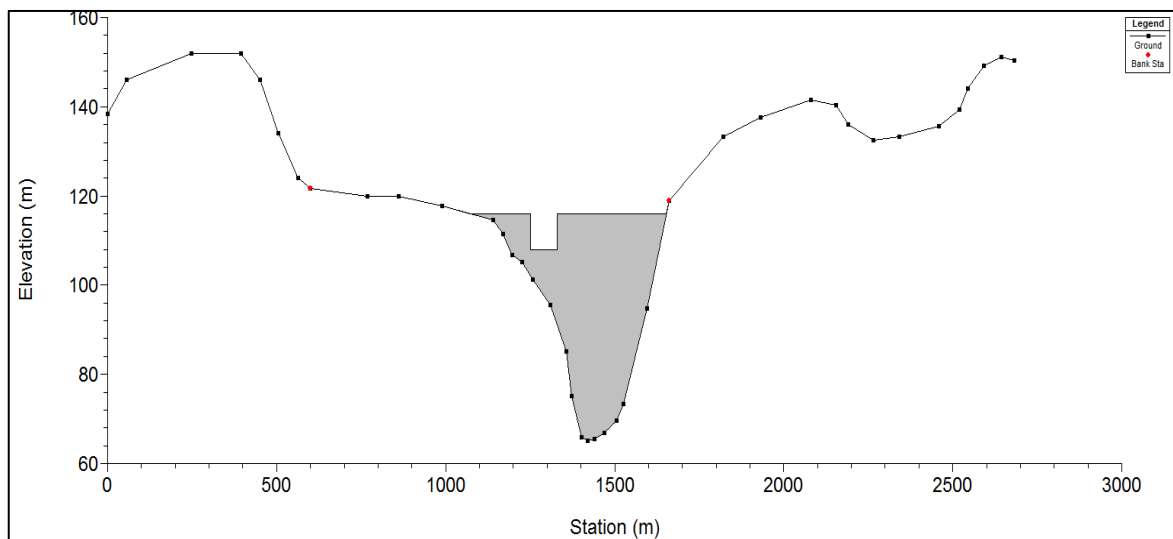


Figure 5.4. Çatalan Dam Cross Sectional View.

In some of the cross sections of the model, there are ineffective flow areas. In HEC-RAS by using ineffective flow area options, areas of the cross section that include water which is stored and is not actively conveyed are defined. Ineffective flow areas options are generally used to determine parts of the cross sections in which water ponds; however the velocity of the water, in flow direction, is nearly zero (Figure 5.5). The water ponds in the ineffective regions of the cross section, is included in the storage calculations and is not included as part of the active flow area.

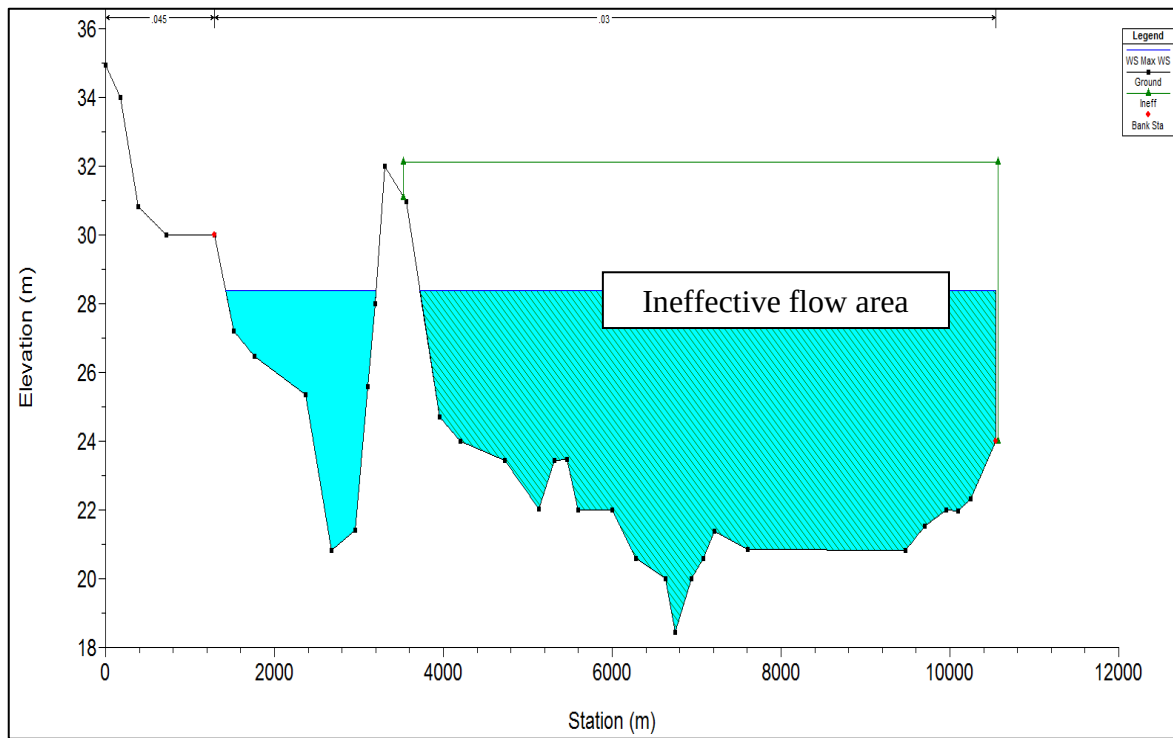


Figure 5.5. Example for an Ineffective Flow Area.

In the model, the breach parameters are determined according to Froehlich 1995 method. For Çatalan Dam piping type dam breaching is chosen because there is no harsh condition or potential threat to exceed maximum capacity of the reservoir due to abrupt inflow. However for Seyhan Dam, breaching type is chosen to be overtopping because of considerable level of threat exposed by Çatalan Dam.

According to Froehlich 1995 method, for Çatalan Dam left and right side slope are chosen to be 0.9 whereas for Seyhan Dam it is 1.4. Breach weir coefficient is taken as 2.6 for both Çatalan Dam and Seyhan Dam. In addition, for Çatalan Dam total breach formation time is determined as 7 hours whereas for Seyhan Dam it is taken as 8.96 hours. In the defined breach formation time, breach progression for Çatalan Dam is not linear whereas for Seyhan Dam it is linear. The progression rate in determined time span for Çatalan Dam is shown as in Figure 5.6.

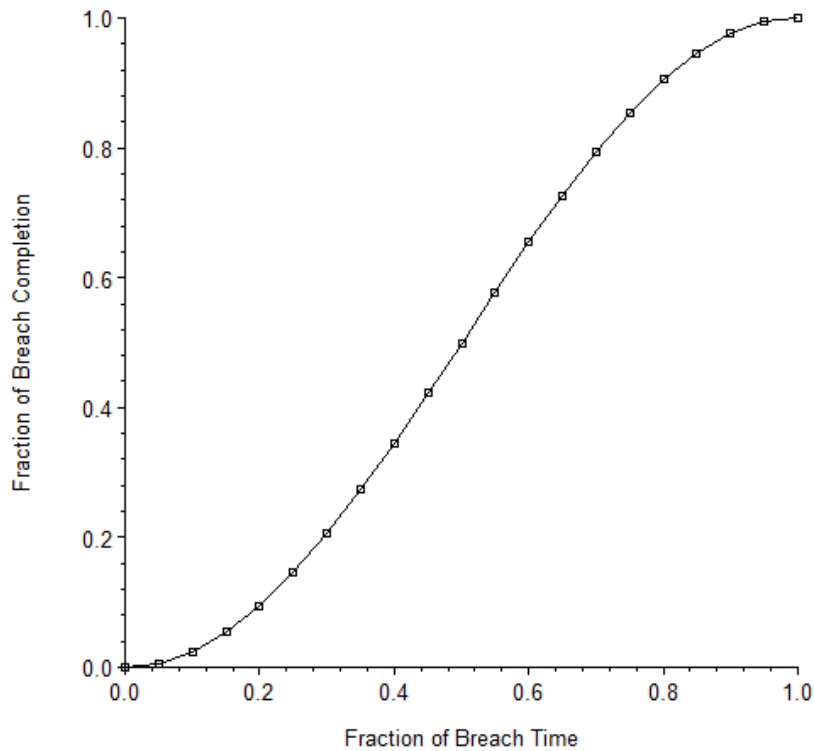


Figure 5.6. Breach Progression Plot for Çatalan Dam.

In the model, total analysis time is determined as 2 days which starts from 10 April 2016 00:00 to 12 April 2016 02:00. For the downstream boundary condition, friction slope of the model, which is determined by drawing a straight line from upstream to downstream of the model, is found as 0.0013.

In the analysis domain, there are three important bridges that affect the results. These bridges are, Adana-Şanlıurfa Highway, Mustafa Kemal Paşa Bulvarı, and Mersin-Adana Highway. These bridges affect the flood pattern significantly especially at the region velocity of the flow is considerably high. At these regions analysis are evaluated in detail.

Another important application of dam break inundation analyses is conducting Incremental Damage Analyses. In these analyses, an assessment is made of the impacts of the dam break flood relative to the damage caused by the natural flooding which precedes it. This procedure can sometimes be used for determining the magnitude of an acceptable Inflow Design Flood (IDF) and sizing the emergency spillway. In general, it has application where a dam and reservoir are "small" relative to the watershed it occupies. In

such "run of the river projects", the potential damages from a dam failure may be small relative to the magnitude of damages from natural flooding which can be produced in the tributary watershed [6]. However for the case in the scope of this thesis work, Adana Seyhan River and watershed area are comparatively large. Therefore it is quite clear that, potential damages from a dam failure (especially multi dam failure scenarios) are bigger than damages can be caused by natural flooding.

6. EVALUATION OF THE RESULTS

At the end of the analysis, the following outputs are extracted from the model. These outputs are computed for specified failure scenarios mentioned at previous sections. The results tried to find out throughout the analysis are listed as follows;

- Maximum water surface elevations and discharges at each determined cross section
- Graphical views show the maximum water conveyance in channel, area in overbanks and main channels
- Breach properties and breach analysis results
- The maximum water surface elevation for entire reach
- Variation of water surface elevations
- Inundation map of the region
- Safe zones that can be used for emergency action plan and evacuation plans
- Effects of bridges crosses the river to the flow pattern

In order to have general idea for the results, sample cross sections are used to represent whole analysis domain. All outputs of the analysis can be found in the appendix in detail. Because of its location, in order to have appropriate information cross section 111922.59 near to the downstream of Seyhan Dam, on Özdemir Sabancı Bulvarı, and by the side of Adana State Hospital is chosen. It is approximately 2500 m away from Seyhan Dam. As seen from the Figure 6.1 at the time that water surface elevation reaches its maximum height at the cross section 11922.59, the WSE is 33.72 m, maximum water height reaches up to 10 m, and width of the WSE extends from the left overbank of the river to the right overbank of the river increase up to 650 m.

At cross section 11922.59, water velocity increase up to 6.92 m/s. As evaluated in Chapter 7 Loss of Life Estimation section, flood severity class for downstream of Seyhan Dam vicinity exceeds the “high flood severity” limit significantly. Moreover, when the amount of water conveyed in the river is evaluated, it is also clear that discharge passing at cross section 11922.59 quite high. Total discharge for maximum WSE at cross section

11922.59 is calculated as 24356.7 m³/s which is consist of 24255.92 m³/s in river channel, 75.55 m³/s in left overbank of the river, and 25.26 m³/s in right overbank of the river. The other results with regard to cross section station 11922.59 can be found from the table 6.1 with respect to maximum WSE.

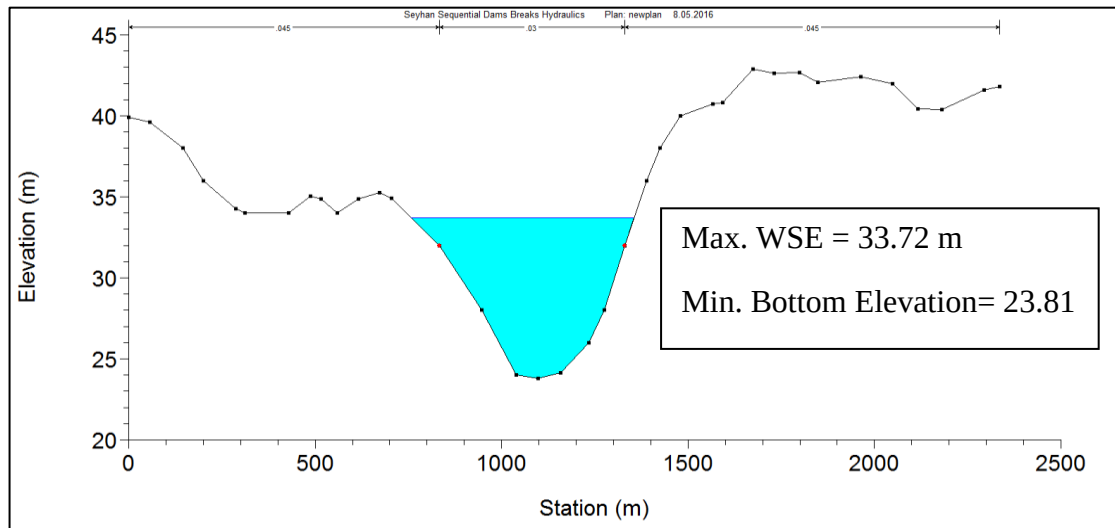


Figure 6.1. Sample Cross Sectional View of Maximum Water Surface Elevation at Cross Section Station 11922.59.

Table 6.1. Detailed Output at Cross Section Station 11922.59.

E.G. Elev (m)	36.25	Element	Left OB	Channel	Right OB
Vel Head (m)	2.53	Wt. n-Val.	0.045	0.03	0.045
W.S. Elev (m)	33.72	Reach Len. (m)	180.05	96.49	16.45
E.G. Slope (m/m)	0.00341	Flow Area (m ²)	64.52	3433.91	21.6
Q Total (m ³ /s)	24356.7	Area (m ²)	64.52	3433.91	21.6
Top Width (m)	596.73	Flow (m ³ /s)	75.55	24255.92	25.26
Vel Total (m/s)	6.92	Top Width (m)	75.24	496.3	25.19
Max Chl Dpth(m)	9.91	Avg. Vel. (m/s)	1.17	7.06	1.17
Conv. Total(m ³ /s)	417137	Hydr. Depth (m)	0.86	6.92	0.86
Length Wtd. (m)	96.72	Conv. (m ³ /s)	1293.9	415410.5	432.7
Min Ch El (m)	23.81	Wetted Per. (m)	75.26	496.68	25.25
Alpha	1.04	Shear (N/m ²)	28.66	231.16	28.61
Frctn Loss (m)	0.33	Stream Power (N/m s)	33.57	1632.82	33.45
		Cum Volume (1000 m ³)	32559.67	678680.3	20094.96
		Cum SA (1000 m ²)	20578.46	109558.8	6972.87

Although maximum WSE at cross section station 11922.59 reaches up to 33.72 m, from HEC-RAS possible results with respect to different elevations can be drawn. For instance, for different elevation values, area and flows conveyed in the channel and in the overbanks can be read from Figure 6.2. The important point should be highlighted here is the fact that after exceeding 40 m, flows conveyed in the channel increase significantly because of characteristic shape of the river cross section.

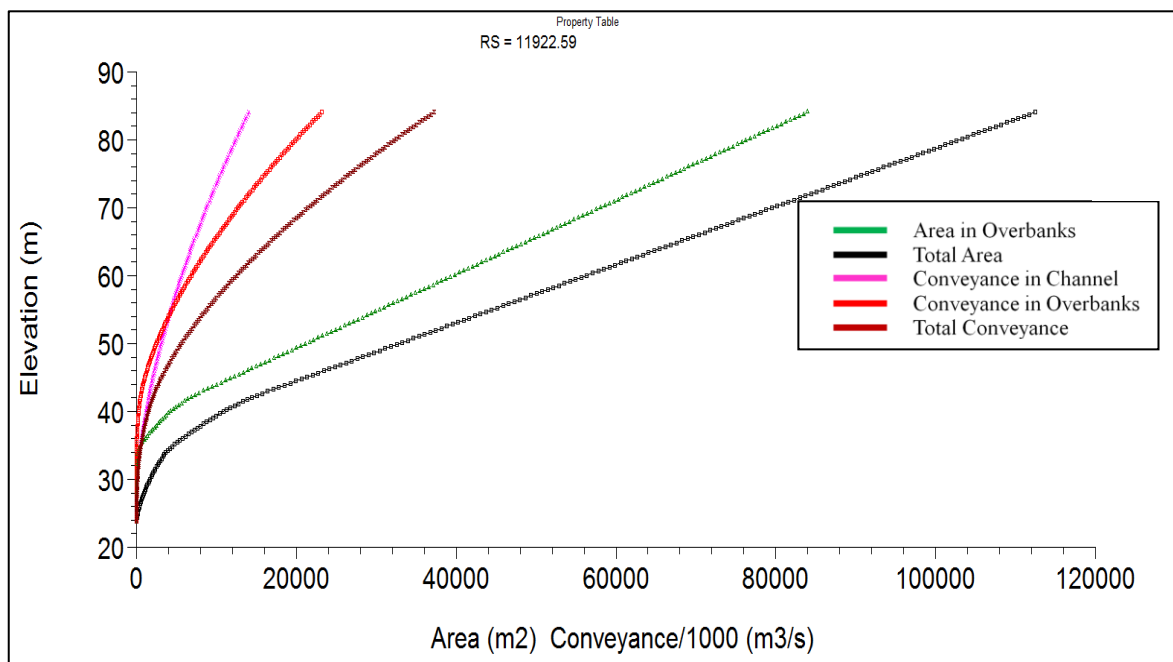


Figure 6.2. Hydraulic Property of Cross Section Station 11922.59.

At some certain cross sections, main channel and overbanks regions are not separated so clearly. Especially in flat areas, when water level increases, flow conveyed in overbanks may be greater than flow conveyed in main channel because of its flat geometric shape. For example, it can be seen from the Figure 6.3 that for max. WSE water conveyed in left overbanks is greater than water conveyed in main channel. This situation may cause error in calculation because water discharge in overbanks has different behavior compared to discharge in main channel. This difference mainly caused by difference in roughness coefficient, difference in velocity, and the locations where water ponds. In order to eliminate this problem, in HEC-RAS portions at overbanks are defined as ineffective flow area in the program.

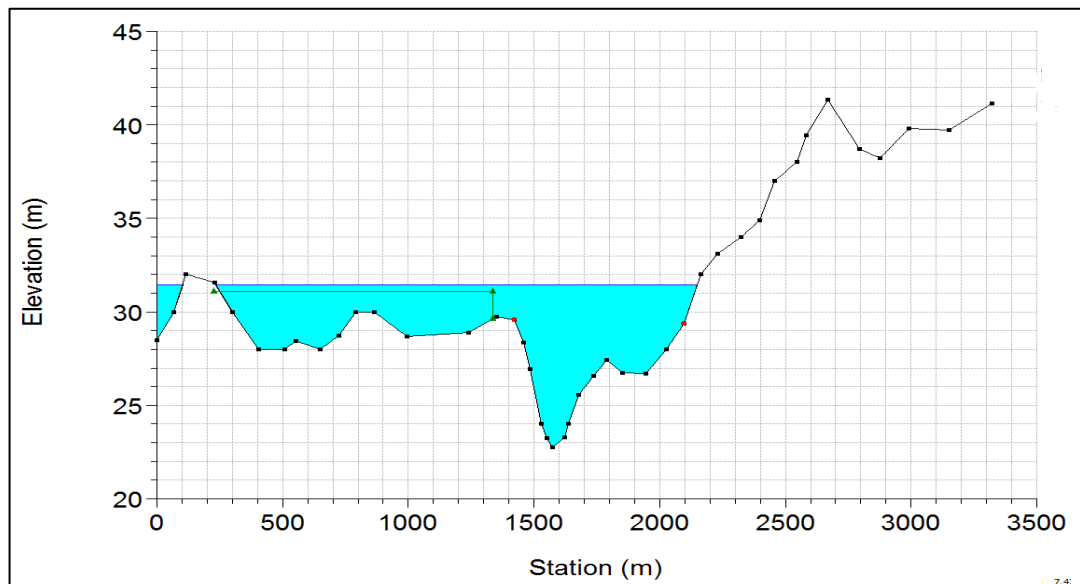


Figure 6.3. Cross Section Station 11247.15 for Max. WSE.

When dam failure mechanism starts to take place at Çatalan Dam at full reservoir height, maximum WSE is observed after some certain time that is different for each cross section. After failure of Çatalan Dam, WSE at reservoir of Seyhan Dam increases too fast. Because of flood progression, maximum WSE is observed at different time intervals for different cross sections. However in HEC-RAS, maximum WSE for each cross section from side view can be shown simultaneously. In Figure 6.6, comparison between maximum WSE and normal WSE in operation is made. As shown from the Figure 6.4, at upstream of Çatalan Dam maximum WSE and normal WSE is same because Çatalan Dam failure is determined as piping failure at maximum reservoir capacity. The reason why Çatalan Dam failure is chosen as piping failure is the fact that at upstream of the dam there is no serious threat to cause overtopping failure. For Seyhan Dam, maximum WSE is computed as 58.34 m whereas normal operation WSE is calculated as 50 m (Figure 6.5). This normal WSE also equals to the bottom elevation of spillways of Seyhan Dam.

Overtopping failure at Seyhan Dam takes place at the level of dam crest which is at 56 m. The biggest difference between maximum WSE and normal WSE is measure approximately as 20 m at cross section station 29770.75.

As mentioned in section 4.2.2.2 the most appropriate method to represent the real breach behavior is Froehlich 1995. For both piping failure of Çatalan Dam and overtopping failure of Seyhan Dam Froehlich 1995 method is used. Because of vicinity to Adana City,

breach parameters used for Seyhan Dam have key role to determine flood mechanism. As examined in the following sections, if a population concentrated near to downstream of a dam, time for breach progression is vital. Therefore, breach parameters used for Seyhan Dam examined attentively.

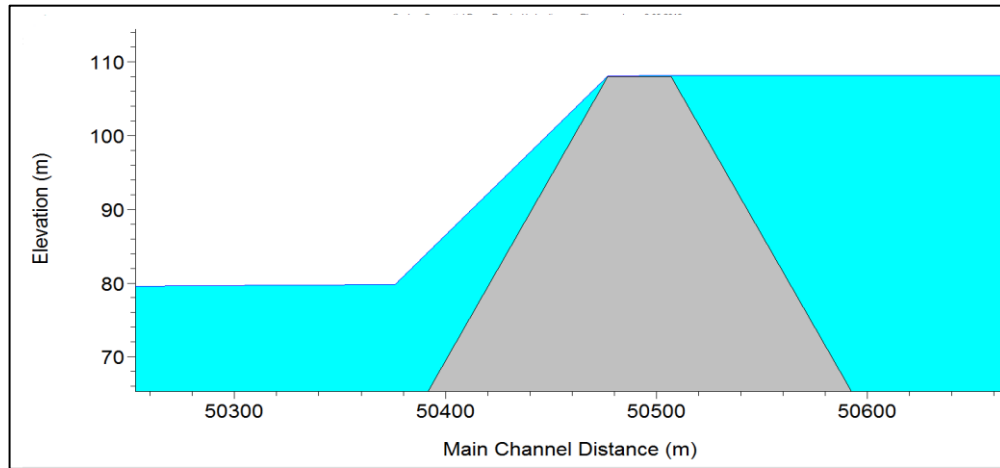


Figure 6.4. WSE at Çatalan Dam at the Instant the Dam Starts to Break.

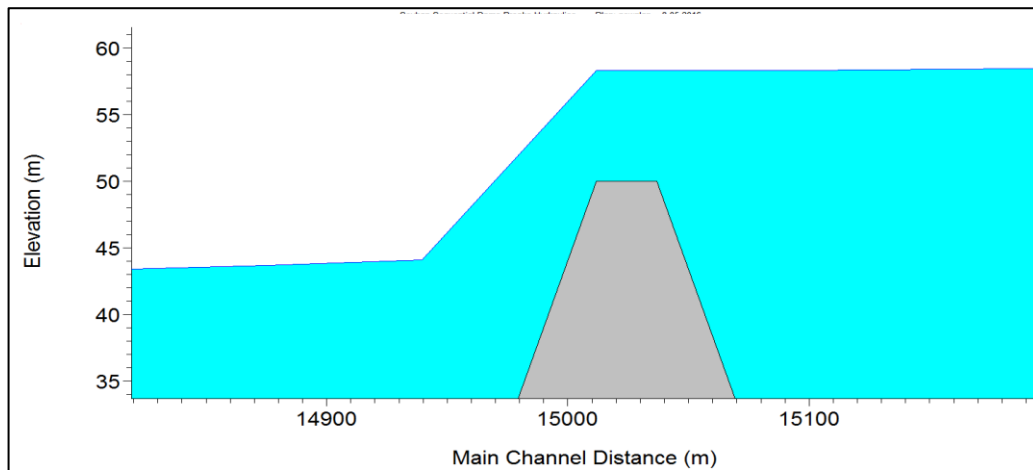


Figure 6.5. Maximum WSE at Seyhan Dam After Dam Starts to Break.

According to Froehlich 1995 method, left and right side slope of the breach is taken as 1.4. For the breach formation time 8.96 hours is taken. It is also supposed that the dam operates properly unto WSE reaches dam crest level which is at 56 m and max WSE reaches up to 58.34 m (Figure 6.7). Breach progression pattern is also accepted as linear. In other words breaching proceeds from 56 m height to final bottom elevation of 36 m without any change in breach progression speed. In addition, final width of the breach is calculated as 287 m with final bottom elevation of 36 m (Figure 6.8).

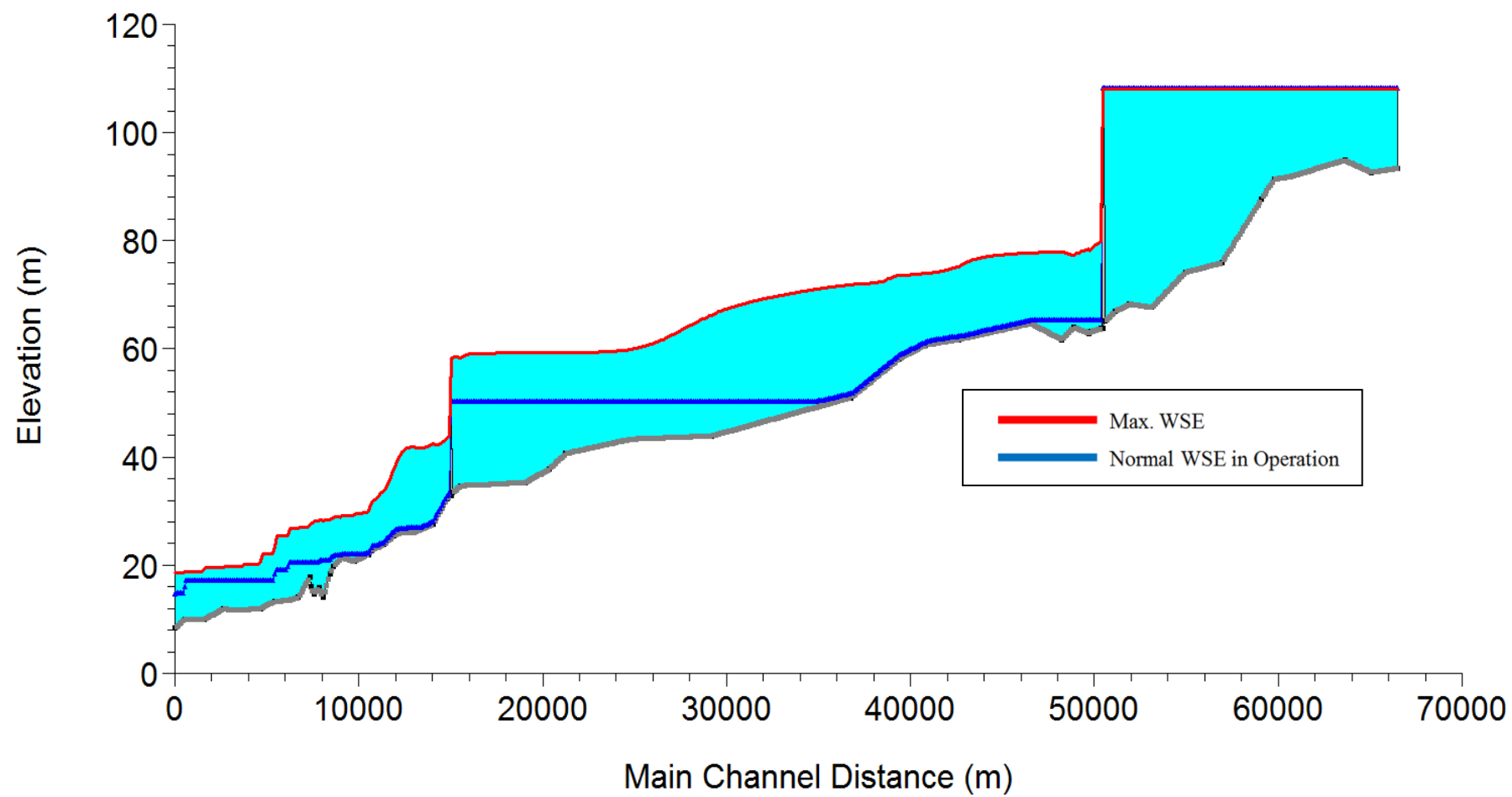


Figure 6.6. Comparison of Max. WSE and Normal WSE in Operation

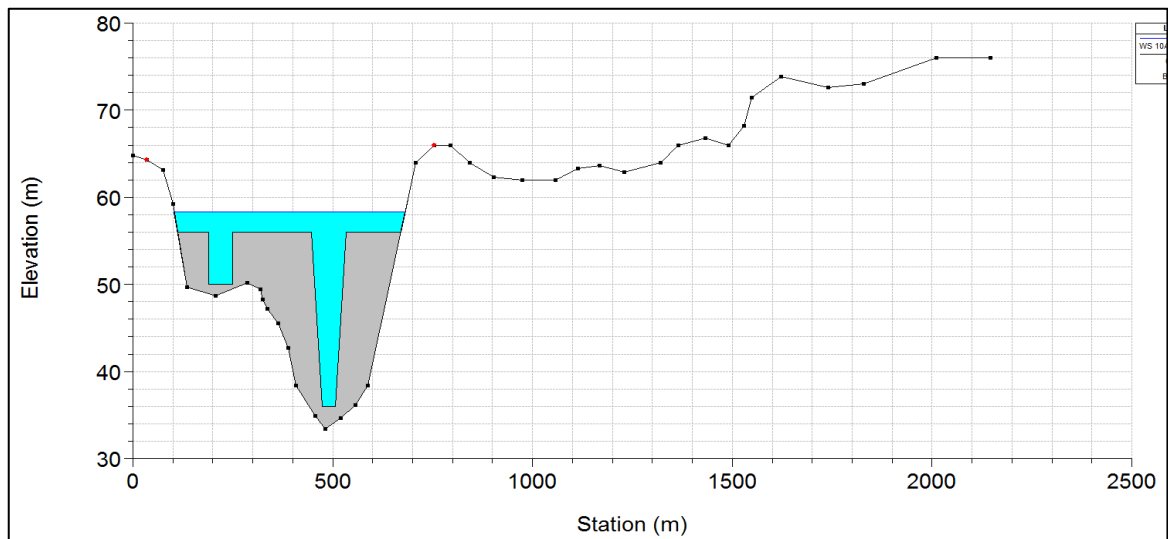


Figure 6.7. Seyhan Dam Breach Phase at Max. WSE.

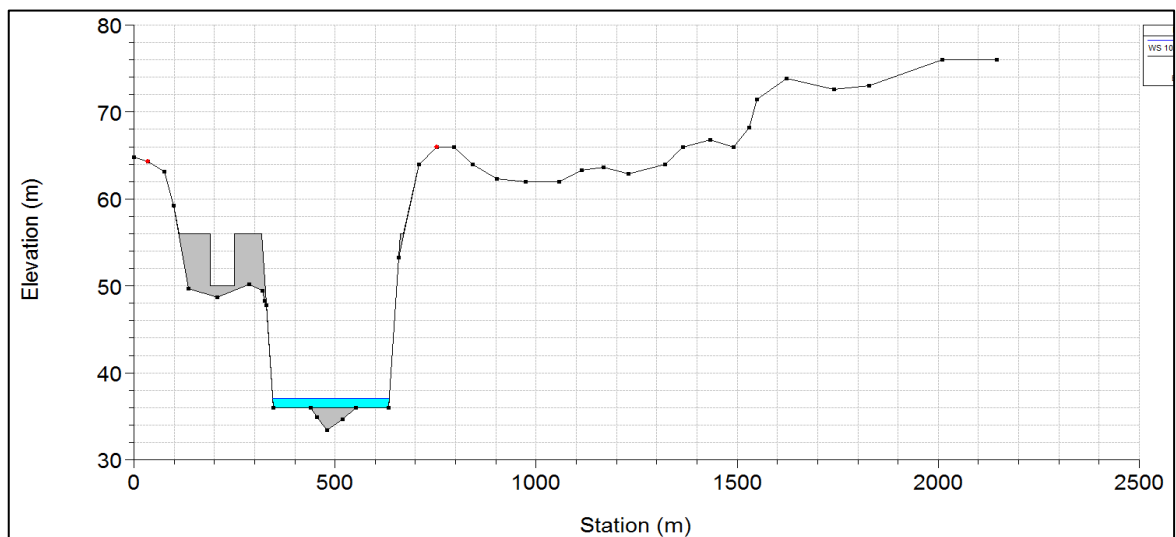


Figure 6.8. Seyhan Dam Full Breach Phase.

At maximum WSE, the discharge passing fully developed breach is calculated as $15092.96 \text{ m}^3/\text{s}$ whereas the total discharge conveyed after overtopping failure starts at maximum WSE is calculated as $20603.55 \text{ m}^3/\text{s}$. When overtopping failure starts to take place, because of abrupt change in elevation at the crest of the dam total velocity head increase suddenly and average flow velocity conveyed in breach at maximum WSE is calculated as 10.76 m/s . Total water depth measured from the breach bottom elevation to WSE increase up to 22.44 m whereas breach weir top width propagate up to 579.4 m . All the other elements and analysis results related to the breach defined for Seyhan Dam are listed in Table 6.2.

Table 6.2. Seyhan Dam Breach Properties and Analysis Results.

E.G. Elev (m)	58.68	Weir Sta Lft (m)	102.6
W.S. Elev (m)	58.34	Weir Sta Rgt (m)	682
Q Total (m ³ /s)	20603.55	Min El Weir Flow (m)	36
Q Weir (m ³ /s)	20603.55	Wr Top Wdth (m)	579.4
Q Breach (m ³ /s)	15092.96	Weir Max Depth (m)	22.44
Breach Avg Velocity (m/s)	10.76	Weir Avg Depth (m)	5.07
Breach Flow Area (m ²)	1403.04	Weir Flow Area (m ²)	2936.01
Breach WD (m)	31.5	Weir Coef (m ^{1/2})	2.386
Breach Bottom El (m)	36	Weir Submerg	0.15
Breach SSL (m)	0.43		
Breach SSR (m)	0.43		

At the end of the analysis, inundation map of Adana City which is located at the downstream of Seyhan Dam is sketched (Figure 6.9 and Figure 6.10). Because of its topographical features and bridges crossing the river perpendicularly, flow pattern is divided into two main branches. The left limb of inundation map covers the largest area of the city. This left branch of the inundation area extends up to Yenidam, Camuzcu, and Yalmanlı districts. It should be also highlighted that, in case of a dam failure airport transportation cannot be used for evacuation action and emergency action plan, because Adana Airport is also in danger of inundation. However at the east side of the city which is approximately 10 km away from the center of Seyhan district, there is İncirlik Military Base which can be used for emergency action plan. At the left branch of the inundation area, south of the city is affected significantly. As seen from Figure 6.10, the first option as an evacuation action for south of left inundation branch of area is to relocate the population living in this area to safe zone 1 shown on the map. For the evacuation action, as main highway, Güney Kuşak Bulvarı road has an important role. Therefore it is recommended to conduct capacity analysis for this highway in case of a dam break evacuation action.

At right branch of inundation area, there is less population living compared to left branch of inundation area. Therefore because of less population and easy accessibility to Safe Zone 2, to evacuate population at right branch of inundation area to safe zone 2 is safer than to evacuate people located at left branch of inundation area to safe zone 1. Because there are D815 and D400 highways, transportation to safe zone 2 is also fast and safe.

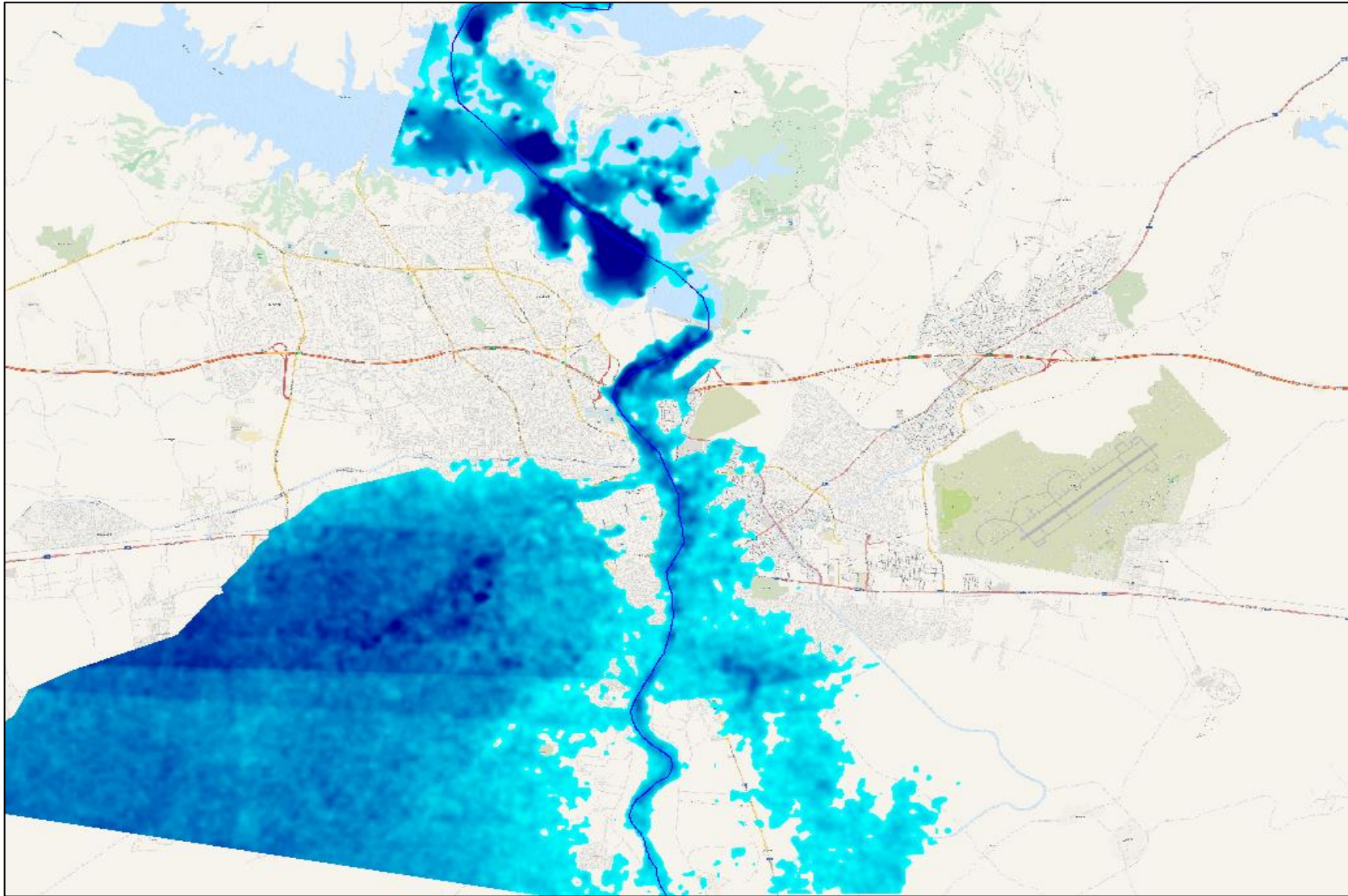


Figure 6.9. Inundation Map of Adana City.

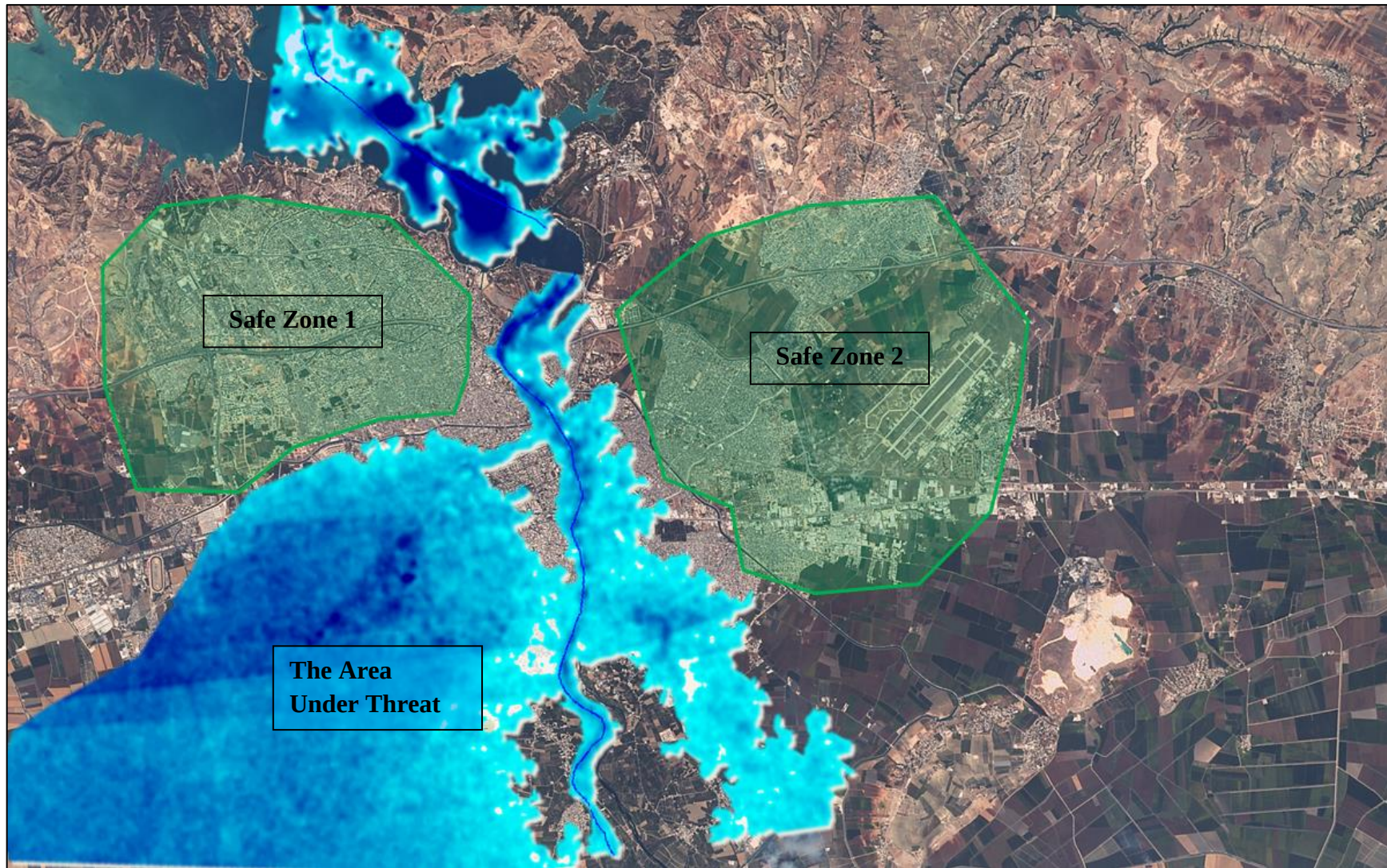


Figure 6.10. Safe Zones that can be Used by Adana City for Evacuation Actions and Emergency Action Plans.

After completion of analysis, it is observed that there are two main inundation area separated at the downstream flood area. The reason why flow pattern changes abruptly at the point of Mustafa Kemalpaşa Bulvarı crosses the river is the fact that Mustafa Kemalpaşa Bulvarı acts like a levee diverting the flow very sharply. After confronting with the bridge, flood pattern follows the bridge in parallel till the other bridge called Alparslan Türkeş Bulvarı. At the intersection point of Mustafa Kemalpaşa Bulvarı and Alparslan Türkeş Bulvarı velocity head of the flow decreases and water surface level starts to rise (Figure 6.12).

In the scope of this thesis work it is assumed that all of the elements in the analysis domain are structurally safe. All calculations are conducted with respect to the situation there are no significant deformations in bridges crossing the river such as Mustafa Kemalpaşa Bulvarı and Alparslan Türkeş Bulvarı. It is recommended to conduct structural analysis according to impact loads and hydraulic forces implied by the flood.

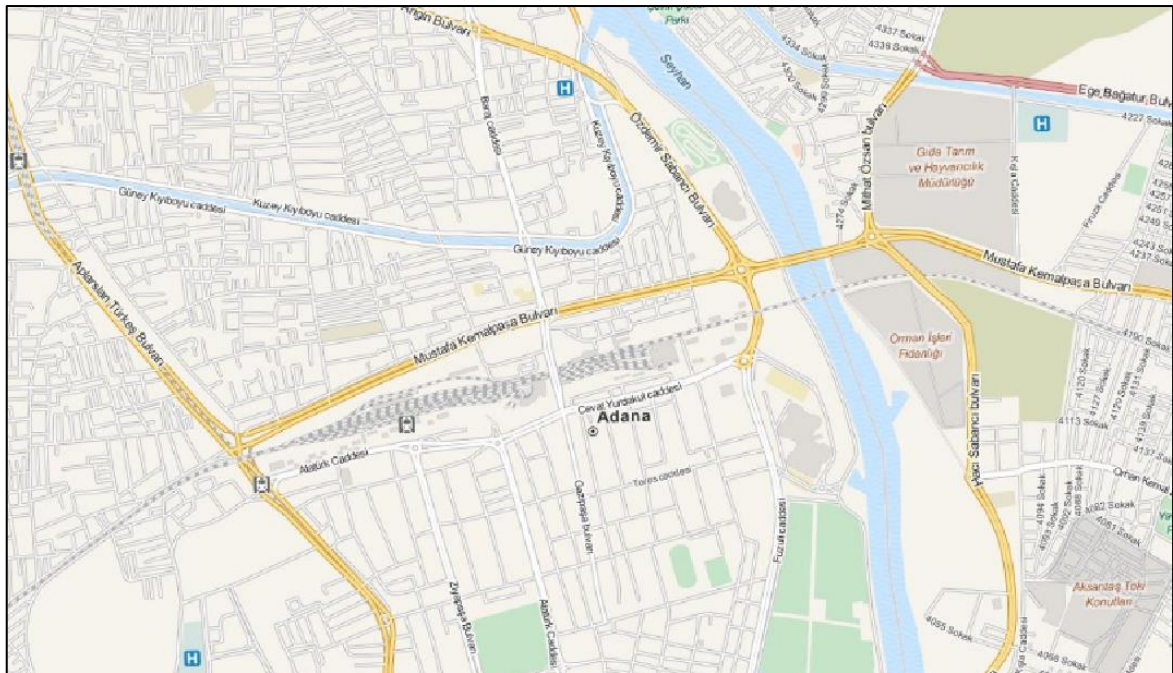


Figure 6.11. Location of Mustafa Kemalpaşa Bulvarı and Alparslan Türkeş Bulvarı.

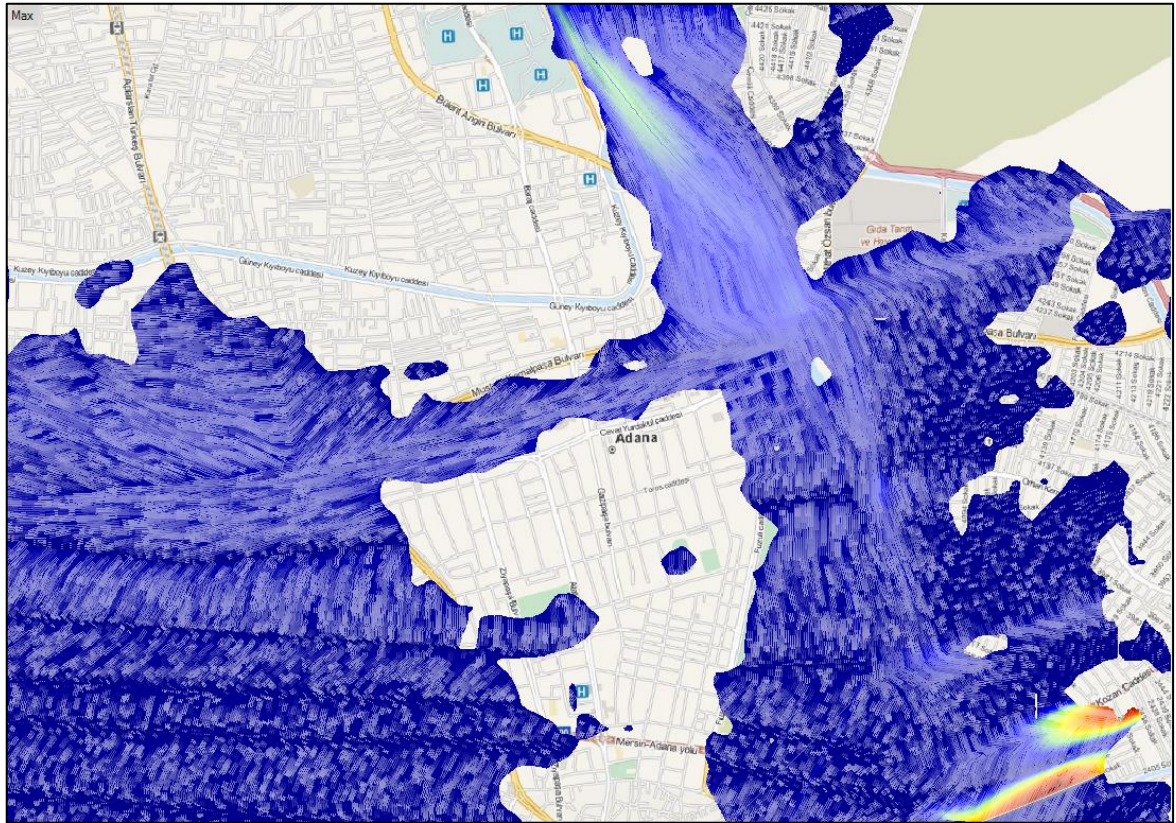


Figure 6.12. Effect of Mustafa Kemalpaşa Bulvarı Bridge on Flow Pattern.

7. LOSS OF LIFE ESTIMATION

Life loss is the most severe damage. Life loss estimation is mainly needed for the following purposes [32];

- For reducing the life safety risks related to dams
- For evaluating the existing and residual risks of dams
- For deciding structural and non-structural risk reduction measures in affected areas
- For the estimation of cost effectiveness of risk reduction measures in affected areas
- For improving the warning efficiency and evacuation planning

Several factors will determine the number of fatalities resulting from dam failure. Included among these are the [33]:

- Cause and type of dam failure
- Number of people at risk
- Timeliness of dam failure warnings
- Flood depths and velocities in the downstream floodplain prior to dam failure
- Flood depths and velocities resulting from dam failure
- Availability of sensory clues (sight of floodwater or sounds created by rushing floodwater) to the people at risk
- Time of day, day of week and time of year of failure
- Weather, including air and water temperature
- Activity in which people are engaged
- General health of people threatened by floodwater
- Type of structure in which people are located
- Ease of evacuation

The number of fatalities resulting from dam failure is most influenced by three of the factors described above. These factors are: 1) The number of people occupying the dam failure flood plain, 2) The amount of warning provided to the people exposed to dangerous flooding, and 3) The severity of the flooding. Without exception, dam failures that have

caused high fatality rates were those in which residences were destroyed and timely dam failure warnings were not issued [33].

There are many methods in literature to estimate the life loss. For instance the methods developed by Brown and Graham, 1988, Dekay and McClelland, 1993 [34], Graham, 1999, Reiter, 2001 [35]. Although it is not possible to estimate the potential life losses due to dam break flooding precisely, there is a comprehensive method developed by Ehsan, 2009 depending on the available data of the Jhelum river valley downstream of Mangla dam in Pakistan. The derived form of the method is as follows [37];

$$LOL_i = PAR_i \times FAT_{BASE} \times F_{sv} \times F_{age} \times F_{mt} \times F_{st} \times F_h \times F_{war} \times F_{ev} \quad (7.1)$$

Where LOL_i is Loss of life at a particular location “ i ” downstream of the dam, PAR_i is Population at risk at a particular location “ i ” downstream of the dam.

FAT_{BASE} is base fatality rate of 0.15 from Graham, 1999 for the worst case of medium severity for definition 2 (Table 7.1). An average value of 1.0 for all other factors with average conditions is assumed. The other factors could differ from 1.0 depending on the specific available information. These factors have been defined after a qualitative plausibility analysis according to the related information available in the literature.

F_{sv} is flood severity factor based on flood severity indication. It has been defined quantitatively in terms of the probability of life loss due to collapse of buildings.

High Severity very likely 1.0
 Medium Severity unlikely 0.3
 Low Severity very unlikely 0.1

In literature, there are many definitions describe the severity of flood due to a dam break. In his Ph.D. thesis, Ehsan, 2009 categorizes flood severity definitions into two main titles. In the following, these two flood severity definitions have been discussed [33,41,42].

Table 7.1. Flood Severity Definition [37].

Definition 1	Definition 2
<u>High severity</u> : total or major damage to buildings	<u>High severity</u> : clean sweep of area and little or no evidence of prior human and habitation remaining (rare type of flooding like e.g. Vajont dam, Italy)
<u>Medium severity</u> : partial or moderate structural damage, little damage to the major elements of buildings	<u>Medium severity</u> : destruction of houses but still trees and mangled homes remain for refuge
<u>Low severity</u> : inundation and no damage to structures	<u>Low severity</u> : no buildings are washed off their foundations

For definition 1, the flood severity indication criteria are the following (Ehsan, 2009),

If $vh^3 \geq 7 \text{ (m}^2/\text{s)}$ then high flood severity

Where v is Average flow velocity of the flood water (m/s) and h is flood depth (structures are exposed to) (m)

If $7 > vh^3 \geq 3 \text{ (m}^2/\text{s)}$ then medium flood severity, otherwise low severity.

The above mentioned vh - thresholds are applicable to materials that are mostly used in buildings such as concrete, brick and masonry.

For definition 2, the available criteria are incomplete. They are defined in the literature only for medium and low flood severity. There is no threshold available for high flood severity. As no specific building type is mentioned in the literature [33], the following criteria are assumed to be applicable to all common types of buildings.

If $DV > 4.6$ (m^2/s) then medium flood severity, otherwise low severity.

$$\text{Where } DV = \frac{Q_{df} - Q_{safe}}{W_{df}} \quad (\text{m}^2/\text{s}) \quad (7.2)$$

Where Q_{df} is discharge at a particular site caused by dam failure (m^3/s), Q_{safe} is mean annual discharge or safe channel capacity at the same site (m^3/s) and W_{df} is maximum width of flooding caused by dam failure at the same site (m).

There is also another criterion for this flood severity definition depending on flood depth (h). This criterion is also incomplete as it gives thresholds only for medium and low flood severity [33].

If $h \geq 10\text{ft}$ (3.05 m) then medium flood severity, otherwise low severity.

F_{age} is age risk factor depending on different age groups in PAR; Based on the available data, three age groups have been defined, number of people in group A (<10yrs+(>=65yrs)), B (10-15)yrs and C (15-64)yrs.

A	1+0.25 (0.25 is the extra risk)	1.25 (more than likely)
B	1+0.1	1.1 (likely)
C	1-0.2	0.8 (unlikely)

$$F_{age} = 1.25 \times A\% + 1.1 \times B\% + 0.8 \times C\% \quad (\text{general form}) \quad (7.3)$$

F_{mt} is material risk factor: For flood severity indication, materials that are frequently used for house construction such as concrete, bricks, masonry etc.

$$F_{mt} = 1 \times X\% + 1.5 \times Y\% \quad (7.4)$$

X is % of other types of houses and Y is % of very low strength houses

F_{st} is story risk factor: It is assumed that all types of houses (single and more stories) will be damaged during high flood severity, because upper stories can also collapse due to the failure of the ground story. For medium and low severity cases, more story houses could provide refuge to people in upper stories and reduce the overall risk. So the suggested relation for the story risk factor is,

$$F_{st} = 1 \text{ (for high severity and all types of houses)}$$

$$F_{st} = 1 - S\% \text{ (for medium and low severity)}$$

Where, S is % of more story houses

F_h is Health risk factor: An average value of 1.0 was assumed with FAT_{BASE} for normal healthy PAR. Extra risk for the disabled people has been considered [38].

$$F_h = 1 \times H\% + 1.25 \times D\% \text{ (general form)} \quad (7.5)$$

Where H is % of PAR with avg. health and D is % of disabled PAR. The percentage of PAR will be used as weighting factor in the equation.

F_{war} is warning factor depending on the initiation of warning and flood travel time. It has been defined quantitatively as the likelihood of no rescue for different warning time categories by interpreting Graham's warning definitions. Flood severity understanding for warning issuers defined by Graham has also been combined with the warning.

Warning time	Flood severity understanding	F_{war}
No	No	1
Some (15-60 min)	Vague/unclear	0.7
Adequate (> 60 min)	Precise/clear	0.3

Different warning time categories have been defined by Graham in the following way [33];

- No warning: This means that no warning is issued by the media or official sources in the particular area before the arrival of flood water. Only the possible sight or sound of the approaching flood will be considered as a warning.
- Some warning: This means that the officials or media start warning in the particular area for 15 to 60 minutes before the arrival of flood water. Some people will be informed about flooding through friends, neighbors or relatives.
- Adequate warning: This means that the officials or the media start warning in a particular area for more than 60 minutes before the arrival of flood water. Some people will be informed about flooding through friends, neighbors or relatives.

F_{ev} is ease of evacuation factor: It will depend on the warning efficiency and the available evacuation facilities, transportation. In principle, this factor would be different for urban and rural areas. Depending on the available data, urban and rural areas have not been considered separately. But more research is required to estimate the ease of evacuation in urban and rural areas. At the moment, no empirical value or guideline is available for ease of evacuation. So this factor has been defined quantitatively as the likelihood of no rescue for all combinations of population at risk (PAR) with respect to warning efficiency.

Warning time	Ease of evacuation	F_{ev}
No	No	1
Some (15-60 min)	Some	0.7
Adequate (> 60 min)	Good	0.3

There is a significant work done at Adana Seyhan Dam by using the method developed by Ehsan [37] in order to show the importance of flood warning system for reduction of loss of life in dam break scenarios. According to the analysis results the estimated number of people will be under risk in case of a flood event due to break of Seyhan Dam is 493790. The studies have shown that without system of warnings before

the dam break 94808 (19.20% of people at risks) people will lose their lives. With warnings between 15 to 60 minutes the loss of lives reduces to 5925 people (1.20%) and with warning before more than 1 hour, the loss reduces to 2903(0.59%) people. Because of proximity of the population to the dam, the estimation of loss of life will be minimum 2903 [39]. This work is one of the significant indicators that underline the importance of warning systems.

Table 7.2. Estimation of Loss of Life at Downstream of Seyhan Dam in Adana City for Three Warning Time Scenarios [39].

	Population of Adana	PAR	No Warning	15-60 min.	More than 60 min.
Number of People	1645965	493790	94808	5925	2903
Percentage	100.00%	30.00%	19.20%	1.20%	0.59%

Although this estimation shown in table 7.2 gives the main idea about the possible outcome in case of a failure, it is difficult to give a precise estimate of the loss of life that would occur from a dam failure for the following reasons [33]:

- The time of dam failure (day, week, season), conditions existing at the time of failure (clear, rain, snow, darkness) and the number of people at risk at the time of dam failure (seasonal recreational usage, special events) are either unknown or can only be estimated.
- It is not known exactly when a dam failure warning message would be given. Experience indicates that there is sometimes a reluctance to issue dam failure warnings. Examples include the failure to issue warnings before the Buffalo Creek Coal Waste Structure failure in West Virginia, as well as the delay in initiating the dam failure warnings at Teton Dam in Idaho. The operating procedures or emergency plans that may be available for a dam should provide some guidance regarding when a warning would be issued. There is no assurance, however, that a warning would be initiated as directed in a plan. A study investigating loss of life

from dam failure can be used to highlight weaknesses in the dam failure warning process and provide some guidance on how improvements in the process would reduce the loss of life.

- The procedure for estimating loss of life is not precise. Even if the time of the failure, conditions existing at the time of failure, number of people at risk, and the time at which warnings are initiated are known with certainty, there will be error in the loss of life estimate.

8. RECOMMENDED MEASURES

Because of their incomplete understanding or uncertainties associated with dam failure reasons (such as earthquakes, sabotages, etc.), no dam can ever be completely safe. However it is possible to reduce the possibility of a dam failure significantly by applying relevant guidelines and legislation. These guidelines and legislations are established in different countries within the body of different institutions or agencies. One of the significant examples to these guidelines and legislation is the Interagency Committee on Dam Safety (ICOLDS). In 1979, the Interagency Committee on Dam Safety (ICOLDS) issued its report containing the first dam-safety guidelines for Federal agency dam owners [30].

Increasing focus on dam safety has initiated the setting up of relevant guidelines and legislations, especially, in the European countries at the national level. Meanwhile, Turkey, as a country at the edge of the European Union membership, has been putting efforts in setting up relevant regulations on dam safety since 1998 by the activities of DSI, the primary executive state agency responsible for planning, management, development, and operation of overall water resources of the nation. As a result of ongoing efforts to set up the necessary legislations and guidelines, DSI Dam Safety Branch is formed on December 08, 2005 within the body of Dams and HEPPs Department at the General Directorate of State Hydraulic Works (DSI) [3].

There are several preventive and recovery measures that can be taken before and after a possible dam failure in order to minimize potential death toll and economic losses. These are mainly listed as preparing inundation maps, setting up emergency action plans, constructing observation stations at the upstream of the dams, determining downstream hazard classification, preparing risk determination works, constructing levees, flood detention basins and diversion channels, organizing flood warning channels, and encouraging some certain types of training. These measures are detailed under following subtitles.

8.1. Inundation Maps

Inundation maps are the reference manuals that point out the possible flood affected areas with respect to different dam failure scenarios. It enables authorities to refer to these maps in order to determine different flood stages and the influence levels for considered areas. The Engineer Manual 1110-2-1420 [5] suggests addressing the following issues during preparation of inundation maps:

- *Preparation of maps.* To evaluate the effects of dam failure, maps should be prepared delineating the area, which would be inundated in the event of failure. Land uses and significant development or improvements within the area of inundation should be indicated. The maps should be equivalent to or more detailed than the USGS quadrangle maps, 7.5-min series, or of sufficient scale and detail to identify clearly the area that should be evacuated if there is evident danger of failure of the dam. Copies of the maps should be distributed to local government officials for use in the development of an evacuation plan. The intent of the maps is to develop evacuation procedures in case of collapse of the dam, so the travel time of the flood wave should be indicated on every significant habitation area along the river channel.
- *Evaluation of hazard potential.* To assist in the evaluation of hazard potential, areas delineated on inundation maps should be classified in accordance with the degree of occupancy and hazard potential. The potential for loss of life is affected by many factors, including but not limited to the capacity and number of exit roads to higher ground and available transportation. Hazard potential is greatest in urban areas. The evaluation of hazard potential should be conservative because the extent of inundation is usually difficult to delineate precisely.
- *Hazard potential for recreation areas.* Potential for affected recreation areas varies greatly, depending on the type of recreation offered, intensity of use, communications facilities, and available transportation. The potential for loss of life may be increased where recreationists are widely scattered over the area of potential inundation because they would be difficult to locate on short notice.

- *Industries and utilities.* Many industries and utilities requiring substantial quantities of water are located on or near rivers or streams. Flooding of these areas and industries, in addition to causing the potential for loss of life, can damage machinery, manufactured products, raw materials and materials in process of manufacture, plus interrupt essential community services.
- *Least hazard potential.* Rural areas usually have the least hazard potential. However, the potential for loss of life exists, and damage to large areas of intensely cultivated agricultural land can cause high economic loss.

According to the analysis results for Adana city with respect to possible dam failure scenarios it can be seen from the figure that a great portion of the city is under risk of flooding because of its geological features (Figure 8.1). It can be also understood from this inundation map that, determined safe zones are located out of the city center. Therefore it should be underlined that emergency action plans for the city must be prepared precisely because it may be very difficult to evacuate the population under risk to the determined safety zones.

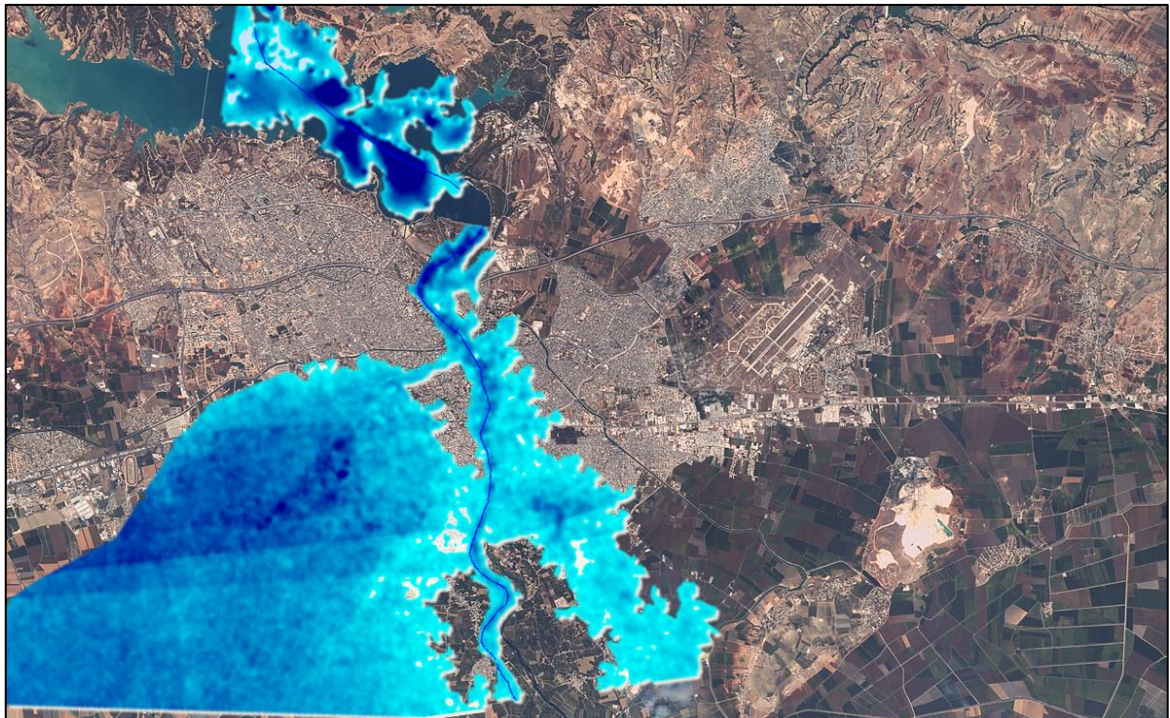


Figure 8.1. The Flood Inundation Map of Adana City.

8.2. Emergency Action Plans

The main purpose of an emergency action plan (EAP) is to evacuate the population under risk to pre-determined safe zones. From past experiences it is observed that by using an EAP many lives can be saved and many fatal accidents can be prevented. In the study of Bozkuş et al., 2000 [44] it is quoted from the study of Danielson and Mc Intyre, 1989 that the validity of this observation was positively tested at Baldwin Hill and San Fernando dam failures; in both cases it was possible to evacuate thousands of people. More recently, the collapse of Quail Creek dam with no injuries illustrates the value of evacuation plans and other safety requirements [52].

In Turkey in order to mitigate the potential risks caused by both natural and technical reasons and to minimize the damages and casualties to the downstream in case of a failure of a dam in operation, DSI Dam Safety Branch is established within body of the Dams and Hydroelectric Power Plants (HEPPs) Department at the General Directorate of State Hydraulic Works (DSI) and preparing Emergency Action Plans is among the responsibilities of the Branch [3].

There are two big responsibilities for EAP during the event and disaster of dam breaks. First, Dam owners are responsible for developing, maintaining, and implementing the EAP and second, the State governor and local emergency management officials are responsible for warning and evacuation within affected areas [43].

In an EAP some certain issues should be targeted. Engineer Manual 1110-2-1420, Hydrologic Engineering Requirements for Reservoirs [5] suggests the following regarding the evacuation plans:

- Evacuation plans should be prepared and implemented by the local jurisdiction controlling inundation areas. The assistance of local civil defense personnel should be requested in preparation of the evacuation plan. State and local law enforcement agencies usually will be responsible for the execution of much of the plan and should be represented in the planning effort. State and local laws and ordinances may require that other state, county and local government agencies have a role in the preparation, review, approval, or execution of the plan. Before finalization, a

copy of the plan should be furnished to the dam agency or owner for information and comment.

- Evacuation plans will vary in complexity in accordance with the type and degree of occupancy in the potentially affected area. The plans may include delineation of the area to be evacuated; routes to be used; traffic control measures; shelter; methods of providing emergency transportation; special procedures for the evacuation of people from institutions such as hospitals, nursing homes, and prisons; procedures for securing the perimeter and for interior security of the area procedures for the lifting of the evacuation order and reentry to the area; and details indicating which organizations are responsible for specific functions and for furnishing the materials, equipment, and personnel resources required.

In Federal Guidelines for Dam Safety, it is also pointed out that evacuation plans are recommended to be based on a worst-case scenario and address the followings [31]:

- Initiation of emergency warning systems
- Pre-incident planning
- Identification of critical facilities and sheltering
- Evacuation procedures, including flood wave travel time considerations (e.g., evacuation of special needs populations, lifting evacuation orders)
- Distance and routes to high ground
- Traffic control measures and traffic routes
- Potential impact of weather or releases on evacuation routes such as flooded portions of the evacuation route before the dam incident occurs
- Vertical evacuation/sheltering in place
- Emergency transportation
- Safety and security measures for the perimeter and affected areas
- Re-entry into affected areas

It is vital that the development of the EAP be coordinated with all entities, jurisdictions, and agencies that would be affected by an incident at the dam or that have statutory responsibilities for warning, evacuation, and post-incident actions. The EAP should contain clearly defined roles and responsibilities for each entity. Coordination with

emergency management authorities responsible for warning and evacuating the public is essential for ensuring agreement on individual and group responsibilities. Participation in the development of the EAP will enhance confidence in the EAP and its accuracy. Coordination will provide opportunities to discuss critical emergency planning concerns such as the order of public official notification, use of backup personnel, alternate means of communication, and special procedures for nighttime, holidays, and weekends [31].

Warning time is a crucial issue should be considered for EAP. The warning time is the sum of the breach initiation time, breach formation time, and flood wave travel time from the dam to a population center [45]. Loss-of-life can vary from 0.02% of the population-at-risk with more than 90 minutes of warning time, to 50% of the population-at-risk when warning time is less than 15 minutes [46]. In addition, 99% of the casualties usually occur in the first 15 km [42].

After the EAP has been developed, approved, and distributed, continual reviews and updates must be performed. Without periodic maintenance, the EAP will become outdated and ineffective. The EAP should be updated promptly to address changes in personnel and contact information, significant changes to the facility, or emergency procedures. The EAP should be reviewed at least annually for adequacy and updated as needed. Even if no revisions are necessary, the review should be documented [31].

8.3. Observation Stations at the Upstream of the Dams

The measurement of the precipitation and the flow discharge of upstream river of the dams has a crucial role in emergency action plan and in warning the community settled close to the proximities of the dams in order for an effective and immediate evacuation action. There are many kinds of observation stations in order to acquire precise and fast data that inform the authorities and the community. Mainly three different types of observation techniques should be applied in the stations located at the upstream of the dams. These are recording (weighing type) precipitation gages, river stages observations, and flash flood warnings.

There are two widely used recording precipitation gages that can be installed at the observation stations located upstream of the dams. These are Belfort (Fisher and Porter) gage and Universal gage. In order to obtain a correct measurement, the gages should not be located very close to the obstacles such as trees, buildings, etc. The gages should be protected as much possible as from the windy weather conditions.

The stage of the river can be measured simply by using pressure recording instruments. To install the instruments, cross sectional measurements of the river should be done at every determined intervals. To prevent the false readings from the instruments, the pressure recording instruments should be protected by using strong and durable materials. The most practical way to protect a pressure recording instrument is to use a thick walled plastic pipe. (Figure 8.2).



Figure 8.2. Pressure Recording Instruments (Water Level Logger) Used by the Author in a Watershed Project.

The instruments should be installed properly and their maintenances should be provided regularly. There are some important factors when choosing the locations of the instruments. These are as follows;

- The instruments should not be installed in the backwater of the river in order to prevent false recordings.
- The instruments should be located such that possible damages can be observed at installation area of the instruments.
- The instruments should be available even in harsh weather conditions

- The installation of the gages should be on firm ground in order for stable position for the instruments.

Thanks to advancement in technology there are some new techniques in obtaining data about a natural hazard. One of the most important and most practical techniques is satellite remote sensing systems. These systems can be used to integrate natural hazard assessments into development planning studies. These systems are, SPOT satellite, satellite radar systems, advanced very high resolution radiometer (AVHRR) on NOAA-10 and 11 satellites, metric camera, large format camera (LFC), and Sojuzkarta. Remote sensing from satellite vehicles has become increasingly important following the successful launch of the Landsat 1 satellite (formerly ERTS-1) in 1972. Since then many satellites with remote sensing capabilities have been developed and used successfully [47].

8.4. Downstream Hazard Classification

The Downstream Hazard Classification is used for a variety of purposes in the Dam Safety Regulations Chapter 173-175 WAC, in the Dam Safety Guidelines, and in the internal operations of the State Dam Safety Program [6]. Uses include:

- A reasonably concise indicator of the relative magnitude of the downstream consequences from failure of a given dam.
- An index for establishing general design requirements and criteria.
- An index for identifying those dams where an emergency action plan is required.
- A management tool for allocating time and prioritizing the State Dam Safety Program activities for: construction inspection; periodic inspection; and compliance and enforcement.
- A classification system compatible with national criteria for Downstream Hazard Classification and incorporation into national databases on dam characteristics.

In determining the downstream hazard classification of a given project, hypothetical dam failures should be evaluated for two reservoir conditions - normal pool level, and maximum storage elevation during flood conditions. The more severe consequences of failure for the two conditions should be used to establish the

classification. In most cases, failure at the maximum storage elevation will produce the greater consequences. However, there are situations, such as where temporary use or recreational areas are located downstream of dams, where a sunny day failure at normal pool condition could pose the more severe consequences [6].

In Dam Safety Guidelines Technical Note 1 prepared by Washington State Department of Ecology [6], downstream hazard classifications are tabulated with respect to population at risk, economic loss, and environment damages. (Table 8.1). According to this given downstream hazard classification, Adana City located downstream of Seyhan Dam is classified as “High Downstream Hazard Potential” and “1A Downstream Hazard Classification”.

Table 8.1. Downstream Hazard Classification Program [6].

Downstream Hazard Potential	Downstream Hazard Classification	Column 1A Population at Risk	Column 1B Economic Loss Generic Descriptions	Column 1C Environmental Damages
Low	3	0	Minimal. No inhabited structures. Limited agriculture development.	No deleterious materials in water
Significant	2	1 to 6	Appreciable. 1 or 2 inhabited structures. Notable agriculture or work sites. Secondary highway and/or rail lines.	Limited water quality degradation from reservoir contents.

Table 8.1. Downstream Hazard Classification Program [6] (cont.).

Downstream Hazard Potential	Downstream Hazard Classification	Column 1A Population at Risk	Column 1B Economic Loss Generic Descriptions	Column 1C Environmental Damages
High	1C	7 to 30	Major. 3 to 10 inhabited structures. Low density suburban area with some industry and work sites. Primary highways and rail lines.	Severe water quality degradation potential from reservoir contents and long-term effects on life.
High	1B	31-300	Extreme. 11 to 100 inhabited structures. Medium density suburban or urban area with associated industry, property and transportation features.	
High	1A	More than 300	Extreme. More than 100 inhabited structures. Highly developed densely populated suburban or urban area.	

Whereas Davis (2001) [8] classifies the dams in terms of their potential hazard levels in three major groups:

- Low hazard potential: Dams assigned the Low Hazard Potential classification are those where failure or mis-operation results in no probable loss of human life and low economic losses, low environmental damage, and no significant disruption of lifeline facilities. Losses are principally limited to the owner's property.

- Significant hazard potential: Dams assigned the Significant Hazard Potential classification are those dams where failure or mis-operation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns.
- High hazard potential: Dams assigned the High Hazard Potential classification are those where failure or mis-operation will probably cause loss of one or more human lives.

8.5. Risk Evaluation Studies

The evaluation of all kinds of risks including social, environmental, and economic loss should be conducted thoroughly and properly. By considering all dam failure scenarios, priorities should be determined in case of an evacuation action. Moreover all aspects of costs and the outcomes of the evacuation actions should be also considered. For instance, if there is scarcity of enough warning time, it may not be feasible to try to save goods and object besides trying to save lives. It may also be meaningless to try to save goods or objects by paying much more evacuation price than their worth. In addition, if there are multi safe zones that are determined by inundation studies, the primary safe zone that is preferred in case of a dam failure should be determined with respect to proximity of affected areas, evacuation resources, etc. Especially in Adana City, because of its crowded population it is very important to conduct risk evaluation studies in order to minimize the possible loss by spending time for putting in order the priorities that will be saved in case of an inundation caused by Seyhan or Çatalan dam failure.

8.6. Levees, Flood Detention Basin and Diversion Channels

In the dam sites to improve the flood carrying capacity of the river, additional levees, flood detention basin and diversion channels can be constructed if the capacity of the existing structures are not enough or cannot be improved. The levee structures can provide an effective flood protection measures by increasing the creek capacity. To obtain an effective flood control levee, the structural design should be done appropriately. Although existing levees along the downstream of the dams are taken into consideration in the HEC-RAS model, if additional levees are constructed, these new added levee structures should be also taken into consideration later on.

Because Adana city is located very close to the downstream of the Seyhan Dam and the population is located very close to the downstream river, it could be hard to construct flood detention basins. Therefore, constructing a flood detention basin in the projected area is not the primary measure should be taken but one of the effective secondary preventive measures that can be constructed in the area.

Another important precaution can be taken in the area is constructing additional diversion tunnels. If the capacity of the existing diversion tunnels is not adequate, additional diversion tunnels can be applied especially at the high hazard level areas determined in consequence of flood severity definition.

8.7. Flood Warning Communication Channels

A flood warning service may be highly organized and integrated on a technical and administrative level, but perception and response are always dependent on social structures and frameworks. These can be highly variable and often unpredictable. The goal at the community level is that warnings should be received by all individuals. The way in which messages are disseminated in communities will depend on local conditions, but may include some or all of the following [48] ;

- Media warnings
- General warning indicators, for example sirens
- Warnings delivered to areas by community leaders or emergency services
- Dedicated automatic telephone warnings to properties at risk
- Dissemination of information about flooding and flood conditions affecting communities upstream. One approach to achieve this is to pass warning messages from village to village as the flood moves downstream
- Keeping watch and giving regular information about the river level and the embankment conditions in the local area. The frequency of the river and embankment watches should be increased as the flood height increases and crosses the critical danger level
- A community-based warning system to pass any information about an approaching flood to every family

There are various means of transmitting data (telemetry), including wired lines, multiplex radio links (VHF, HF and UHF), public telecommunication lines, mobile telephone network and satellite (VSAT) communication links. Each communication channel has its own specific technical features, advantages and disadvantages [48].

The vital items of information to be passed on to users are: (a) where will it flood; (b) when will it flood; (c) how large will the flood be; and (e) how long will the flood last. The type of data disseminated ranges from formal statements made to press and government departments, pro-forma warnings to professional partners formulated jointly according to internal procedures and standing orders, media briefs and warnings, etc. These require rapid conversion of forecast and warning information into a number of formats for onward transmission. Transmission can be by telephone, fax, courier, email or website. Each medium has its advantages and disadvantages in terms of reliability, manpower needs and accessibility, and should be planned for accordingly [48].

Flood warning messages should be delivered in ways that are designed to motivate or arouse. This can be done by using ‘arresting’ language, or by having messages accompanied by a siren or other alarm. The tone of messages is important. As far as possible, messages should [48];

- be positive rather than negative, saying what to do rather than what not to do (e.g. “stay at home” rather than “do not leave your home”)
- suggest action rather than inaction (e.g. “raise your belongings” encourages definite action)
- invite sociability rather than isolation (Social interaction is part of the process of message confirmation, and messages like “advise your neighbors” or “check to see whether your neighbors need help” encourage sociability and help ensure people are assisted where necessary and are in touch with others.)

In many situations and over a wide range of social conditions, flood warnings often fail to be effective. There are many reasons for this, including [48];

- inhabitants may be unwilling to leave their property, belongings and livestock out of fear of looting and vandalism

- the actions of neighbors or weather may contradict the official warning (people often seek confirmation of a flood event before they act)
- some people are averse to following the dictates of authority and may ignore official advice
- some people cannot heed warnings because they may lack the physical or mental capacity to respond, or they may be absent from their dwellings
- some of those at risk may not be worried about flooding until they suffer a loss
- populations at risk will be very diverse, which may mean that they have different priorities, languages and levels of understanding of the flood warning

8.8. Training

Staff in a flood forecasting and warning organization should have access to regular training programs, either to complete their basic education or to maintain and develop new skills. Training courses in core activities, for instance flood forecasting, meteorology, hydrometry, hydrology and hydraulics, administration, public awareness and environmental responsibilities, are important. These programs may be offered annually to allow new staff to be trained. Key technical staff such as instrument technicians, observers and hydrometric technicians should receive several weeks of on-the-job training provided either by visiting specialists or by spending a period of time with suppliers or with an organization already using the relevant equipment. On the basis of their experience, trained staff should be encouraged to provide training internally to their staff in their home organizations. Senior professionals and management can benefit from study tours where they spend time with organizations currently using advanced techniques [48].

The means by which the flood forecasting and warning units deliver their services are best implemented through the staff structure, where senior, more experienced members pass on knowledge to newly-recruited staff. This may require operational training at several levels [48];

- Technical skill training to keep up-to-date with latest developments. New equipment and technologies.

- Familiarization training for different units within the organization. Field Officers made aware of data processing staff needs, etc.
- Training for the whole unit, including internal and joint exercises.
- Training for interfacing with other organizations, including technical training in associated fields. Communications skills, e.g. for press and media briefings.

It is essential for flood warning managers to recognize and understand fully the range of user requirements so that flood warning products, data and information can be adapted to meet their specific needs. Catering to a range of users helps improve the esteem of the flood forecasting and warning service, while encouraging investment and sustainability [48].

9. CONCLUSIONS

Through this thesis work the main objective is to forecast the flood and its impacts due to multiple dam failures of Seyhan Dam and Çatalan Dam under specified circumstances and dam failure scenarios within the HEC-RAS computer software analysis program. Another objective of this study is to determine the inundated areas and the flood features affect the corresponding regions. Based on the flood routing analysis results and evaluations, the recommendations are presented in order to implement them to the areas under threat.

To represent the whole analysis results, a sample cross section station is chosen. Due to its position, in order to have accurate information cross section 111922.59 near to the downstream of Seyhan Dam, on Özdemir Sabancı Bulvarı, and by the side of Adana State Hospital is chosen. It is approximately 2500 m away from Seyhan Dam. At this cross section it is observed that at the time that water surface elevation reaches its maximum height at the cross section 11922.59, the WSE is 33.72 m, maximum water height reaches up to 10 m, and width of the WSE extends from the left overbank of the river to the right overbank of the river increases up to 650 m. At this cross section station it is also observed that, flow velocity increase up to 6.92 m/s. Therefore according to the flood severity criteria, flood severity class for downstream of Seyhan Dam vicinity exceeds the “high flood severity” limit significantly. Total discharge for maximum WSE at cross section 11922.59 is calculated as 24356.7 m³/s which is consist of 24255.92 m³/s in river channel, 75.55 m³/s in left overbank of the river, and 25.26 m³/s in right overbank of the river.

For the breach mechanism type, Çatalan Dam failure is chosen as piping failure at maximum reservoir capacity because at the upstream of Çatalan Dam there is no serious threat to cause overtopping failure. However for Seyhan Dam breach failure type is chosen as overtopping failure because of its proximity to Çatalan Dam and potential threat at the upstream of Seyhan Dam. For both piping failure of Çatalan Dam and overtopping failure of Seyhan Dam Froehlich 1995 method is used for breach analysis in HEC-RAS program.

According to Froehlich 1995 method, for Seyhan Dam left and right side slope of the breach is taken as 1.4. For the breach formation time 8.96 hours is taken. It is also supposed that the dam operates properly until WSE reaches dam crest level which is at 56 m and max WSE reaches up to 58.34 m. Breaching proceeds from 56 m height to final bottom elevation of 36 m without any change in breach progression speed. Final width of the breach is also calculated as 287 m with final bottom elevation of 36 m. At maximum WSE, the discharge passing fully developed breach is calculated as $15092.96 \text{ m}^3/\text{s}$ whereas the total discharge conveyed after overtopping failure starts at maximum WSE is calculated as $20603.55 \text{ m}^3/\text{s}$. Average flow velocity conveyed in breach at maximum WSE is calculated as 10.76 m/s. Total water depth measured from the breach bottom elevation to WSE increase up to 22.44 m whereas breach weir top width propagate up to 579.4 m.

At the end of the analysis, inundation map of Adana City is sketched. It is observed that flow pattern at the downstream of Seyhan Dam is divided into two main branches. The left limb of inundation area extends up to Yenidam, Camuzcu, and Yalmanlı districts. It should be highlighted that, in case of a dam failure Adana Airport is also in danger of inundation. As an evacuation action for south of left inundation branch of area is to relocate the population living in this area to safe zone 1 shown on the inundation map prepared according to analysis results. At right branch of inundation area, there is less population living compared to left branch of inundation area. Therefore because of less population and easy accessibility to Safe Zone 2, to evacuate population at right branch of inundation area to safe zone 2 is safer than to evacuate people located at left branch of inundation area to safe zone 1. Because there are D815 and D400 highways, transportation to safe zone 2 is also fast and safe.

According to analysis results for the location under inundation threat loss of life estimation analysis is conducted. For the loss of life estimation analysis as a comprehensive method developed by Ehsan, 2009 depending on the available data of the Jhelum river valley downstream of Mangla Dam in Pakistan is used. By using this method it is found that the estimated number of people will be under risk in case of a flood event due to break of Seyhan Dam is 493790. The studies have shown that without system of warnings before the dam break 94808 (19.20% of people at risks) people will lose their lives. With warnings between 15 to 60 minutes the loss of lives reduces to 5925 people

(1.20%) and with warning before more than 1 hour, the loss reduces to 2903(0.59%) people [39].

According to the analysis results, there are several preventive and recovery measures that can be taken before and after a possible dam failure in order to minimize potential death toll and economic losses. These recommendations are examined at previous sections. These are mainly listed as preparing inundation maps, setting up emergency action plans, constructing observation stations at the upstream of the dams, determining downstream hazard classification, preparing risk determination works, constructing levees, flood detention basins and diversion channels, organizing flood warning channels, and encouraging some certain types of training.

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APPENDIX A: PROFILE OUTPUT TABLES FOR MAXIMUM WSE

Table A.1. Information of River Stations for Maximum WSE.

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m ³ /s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m ²)	(m)	
catalan ve seyhan	67088.72	Max WS	5	93.43	108.11		108.11	0	0	3800.81	393.28	0
catalan ve seyhan	65621.33	Max WS	5	92.64	108.11		108.11	0	0	13283.91	1626.54	0
catalan ve seyhan	64193.97	Max WS	5	94.88	108.11		108.11	0	0	7718.75	936.2	0
catalan ve seyhan	61129.66	Max WS	5	91.78	108.11		108.11	0	0	5559.14	581.93	0
catalan ve seyhan	60344.29	Max WS	5	91.4	108.11		108.11	0	0	4372.92	491.55	0
catalan ve seyhan	59667.04	Max WS	5	87.66	108.11		108.11	0	0	5087.07	387.91	0
catalan ve seyhan	57508.54	Max WS	5	75.91	108.11		108.11	0	0	12517.28	877.34	0
catalan ve seyhan	55520.43	Max WS	5	74.15	108.11		108.11	0	0	28624.12	2518.63	0
catalan ve seyhan	53698.36	Max WS	5	67.66	108.11		108.11	0	0	84973.07	3086.3	0
catalan ve seyhan	52491.51	Max WS	5	68.3	108.11		108.11	0	0	70285.02	2370.21	0
catalan ve seyhan	51699.32	Max WS	5	66.91	108.11		108.11	0	0	33203.19	1535.61	0
catalan ve seyhan	51176.24	Max WS	5	65.14	108.11	65.45	108.11	0	0	10066.29	442.68	0
catalan ve seyhan	51060.08		Inl Struct									
catalan ve seyhan	50950.38	Max WS	32400.22	63.72	79.81		81.98	0.001715	6.53	4963.85	481.14	0.65
catalan ve seyhan	50379.2	Max WS	32042.49	63.24	78.37		81.41	0.003669	7.72	4149.04	552.71	0.9
catalan ve seyhan	50265.47	Max WS	31549.16	62.89	78.45		79.79	0.000986	5.12	6203.34	598.23	0.5
catalan ve seyhan	49460.4	Max WS	29837.43	64	77.3		78.75	0.001533	5.33	5598.04	676.9	0.59
catalan ve seyhan	48819.92	Max WS	30301.06	61.7	77.92		78.2	0.000179	2.32	13094.05	1164.82	0.21
catalan ve seyhan	47098.59	Max WS	30155.71	64.69	77.78		77.96	0.000134	1.89	17014.4	2096.77	0.18
catalan ve seyhan	43185.98	Max WS	29377.99	61.74	75.25		76.54	0.001268	5.03	5841.14	672.22	0.54
catalan ve seyhan	41406.7	Max WS	28119.91	60.74	74.03		74.7	0.000567	3.74	8575.1	1183.55	0.37

Table A.1. Information of River Stations for Maximum WSE (cont.).

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m ³ /s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m ²)	(m)	
catalan ve seyhan	39956.75	Max WS	27881.57	58.18	73.63		74.04	0.000397	3.79	12252.82	1429.69	0.33
catalan ve seyhan	37340.52	Max WS	27027.85	50.99	71.92		72.48	0.000413	3.31	8222.35	803.21	0.32
catalan ve seyhan	29770.75	Max WS	26414.61	43.9	66.26		67.62	0.001332	5.16	5117.77	582.73	0.56
catalan ve seyhan	25337.54	Max WS	23534.07	43.31	59.87		60.84	0.000958	4.35	5407.15	623.23	0.47
catalan ve seyhan	21825.09	Max WS	21889.29	40.66	59.36		59.4	0.000048	0.99	23991.44	3015.2	0.11
catalan ve seyhan	20941.81	Max WS	21868.02	37.77	59.25		59.34	0.000065	1.36	16128.09	1429.94	0.13
catalan ve seyhan	19628.06	Max WS	21842.89	35.33	59.25		59.29	0.000028	0.85	25657.41	2419.92	0.08
catalan ve seyhan	17365.47	Max WS	21732.94	34.88	59.19		59.23	0.000024	0.95	22866.51	1631.62	0.08
catalan ve seyhan	16124.84	Max WS	20621.12	34.71	58.37		58.9	0.00094	3.22	6407.88	1143.64	0.43
catalan ve seyhan	15671.17	Max WS	20603.55	33.39	58.34	46.44	58.68	0.000181	2.57	8012.3	578.59	0.22
catalan ve seyhan	15577.57		Inl Struct									
catalan ve seyhan	15513.98	Max WS	24382.2	32.86	44.12	45.12	48.73	0.007309	9.51	2564.48	420.14	1.23
catalan ve seyhan	15385.09	Max WS	24381.69	32.23	43.37		45.47	0.003121	6.43	3794.08	591.06	0.81
catalan ve seyhan	14607.64	Max WS	24378.85	27.56	42.44		43.3	0.000766	4.52	7005.88	984.23	0.44
catalan ve seyhan	13475.84	Max WS	24372.19	26	41.89		42.32	0.000384	3.41	10780.48	1509.98	0.31
catalan ve seyhan	12850.94	Max WS	24365.53	26	40.45	40.79	42.55	0.006842	7.95	4734.19	1472.66	1.14
catalan ve seyhan	12489.45	Max WS	24362.76	25.53	37.78	39.51	42.94	0.008724	11.34	3147.89	853.14	1.37
catalan ve seyhan	11922.59	Max WS	24356.73	23.81	33.72		36.25	0.003409	7.06	3520.03	596.73	0.86
catalan ve seyhan	11247.15	Max WS	24195.29	22.74	31.45		32.52	0.002833	5.21	6505.05	2018.6	0.74
catalan ve seyhan	11107.83	Max WS	22788.04	22	29.96		30.2	0.000997	3.07	13398.61	4793.28	0.44
catalan ve seyhan	10334.21	Max WS	21830.65	20.75	29.37	29.07	31.28	0.004218	6.14	3666.16	5836.21	0.89
catalan ve seyhan	9592.97	Max WS	21387.01	21.45	28.87		29.34	0.001825	3.21	7695.2	7701.78	0.56
catalan ve seyhan	9199.981	Max WS	21427.04	19.69	28.72		29.54	0.003303	4	5353.13	8333.39	0.74
catalan ve seyhan	9004.135	Max WS	21386.07	18.44	28.38		28.94	0.001766	3.31	6469.26	8609.67	0.55
catalan ve seyhan	8636.387	Max WS	21343.96	14	28.22		28.85	0.001577	3.73	6863.43	8425.67	0.55
catalan ve seyhan	8394.065	Max WS	21507.91	15.93	28.21		28.99	0.002278	3.91	5707.4	9024.71	0.64

Table A.1. Information of River Stations for Maximum WSE (cont.).

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m ³ /s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m ²)	(m)	
catalan ve seyhan	8118.834	Max WS	21361.71	14.7	27.95		29	0.003633	4.54	4706.99	9097.09	0.78
catalan ve seyhan	7971.313	Max WS	21049.39	16	27.66		28.12	0.00112	3.07	7496.46	9162.39	0.46
catalan ve seyhan	7911.615	Max WS	20872.21	17.78	27.41	27.6	28.68	0.006869	5.25	4635.95	10201.93	1.04
catalan ve seyhan	7281.952	Max WS	20183.53	14	26.9		27.18	0.002676	2.63	8813.61	11445.35	0.61
catalan ve seyhan	6724.259	Max WS	32606.9	13.46	25.51		25.94	0.003814	3.11	11314.22	12392.11	0.73
catalan ve seyhan	5997.462	Max WS	28316.96	13.35	23.2	24.04	27.22	0.033146	8.89	3199.2	13647.05	2.13
catalan ve seyhan	5230.952	Max WS	17902.96	12	20.75	21.88	24.67	0.01785	8.77	2040.53	12643.35	1.68
catalan ve seyhan	4291.967	Max WS	13938.16	11.76	19.92		20.94	0.002868	4.49	3104.82	13116.73	0.72
catalan ve seyhan	3198.173	Max WS	12932.61	11.87	19.65		20.18	0.002716	3.22	4018.7	15507.38	0.65
catalan ve seyhan	2171.547	Max WS	11755.78	10	19.21	19.34	20.66	0.007585	5.33	2203.9	15486.67	1.08
catalan ve seyhan	1105.42	Max WS	11200.45	10	18.71		19.01	0.002219	2.43	4611.49	16495.22	0.56
catalan ve seyhan	574.2968	Max WS	11183.3	8.33	18.56	17.43	18.77	0.001302	2.08	6153.87	15819.56	0.44

APPENDIX B: ROUGHNESS COEFFICIENTS ACCORDING TO THE TYPE OF CHANNEL AND DESCRIPTION

Table B.1. Roughness Coefficients Values for Natural Streams.

Type of channel and description	Minimum	Normal	Maximum
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage < 100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

Table B.1. Roughness Coefficients Values for Natural Streams (cont.).

b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom : gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom : cobbles with large boulders	0.040	0.050	0.070
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with ,tree stumps, no sprouts	0.030	0.040	0.050

Table B.1. Roughness Coefficients Values for Natural Streams (cont.).

3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
D-3. Major streams (top width at flood stage > 100 ft), The n value is less than that for minor streams of similar description, because banks offer less effective resistance			
a. Regular section with no boulders or brush	0.025		0.060
b. Irregular and rough section	0.035		0.100