NUMERICAL SIMULATION OF THE ÇINARCIK DAM FAILURE ON THE ORHANELİ RIVER

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

FIRAT BAĞ

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

JANUARY 2005

Approval of the Graduate School of Natural and Applied Sciences

Prof.Dr. Canan ÖZGEN
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Erdal ÇOKÇA
Head of Department

This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Asst. Prof. Dr. Zafer BOZKUŞ
Supervisor

Examining Committee Members

Prof. Dr. Mustafa GÖĞÜŞ (METU, CE) ____________________

Asst. Prof. Dr. Zafer BOZKUŞ (METU, CE) ____________________

Assoc. Prof. Dr. Nuray TOKYAY (METU, CE) ____________________

Assoc. Prof. Dr. İsmail AYDIN (METU, CE) ____________________

Dr. Mehmet Ali KÖKPINAR (DSİ) ____________________

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last name : FIRAT BAĞ

Signature :

ABSTRACT

NUMERICAL SIMULATION OF THE ÇINARCIK DAM FAILURE ON THE ORHANELİ RIVER

Bağ, Fırat

M.Sc., Department of Civil Engineering
Supervisor : Asst. Prof. Dr. Zafer BOZKUŞ

January 2005, 131 pages

This thesis analyzes the probable outcome of the fictitious failure of a dam under a set of pre-defined scenarios, within the framework of a case study, the case subject being the Çınarcık Dam located within Bursa Province of Turkey. The failure of the dam is not analyzed neither structural nor hydraulic-wise but is assumed to be triggered when certain critical criteria are exceeded. Hence, the analyses focus on the aftermath of the failure and strive to anticipate the level of inundation downstream of the dam itself. For the purpose of the analyses, the FLDWAV software developed by the National Weather Service of USA is used to spatially and temporally predict the flow profiles, water surface elevations and discharges occurring downstream of the Çınarcık Dam under the defined set of scenarios. Based on these analyses, indicative inundation maps and settlements under risk will be identified, and the thesis study will further address some available pre-event measures that may be taken in advance.

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

ÖZ

ORHANELİ NEHİRİ ÜZERİNDEKİ
ÇINARCIK BARAJI YIKILMASININ NÜMERİK BENZEŞİMİ

Bağ, Fırat

Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi : Yrd. Doç. Dr. Zafer BOZKUŞ

Ocak 2005, 131 sayfa

Bu tez çalışması, önceden belirlenmiş bir dizi senaryo çerçevesinde kurmaca bir baraj yıkılmasının olası etkilerini, bir durum çalışması çerçevesinde analiz etmektedir. Durum çalışmasının konusu Türkiye'nin Bursa İli'nde yer alan Çınarcık Barajı'dır. Barajın yıkılması yapısal ya da hidrolik anlamda analiz edilmemekte; yıkılmanın belirli kritik kıstaslar aşıldığında tetiklendiği varsayılmaktadır. Dolayısıyla analizler, yıkılmanın ardından gerçekleşen olumsuz sonuçlara odaklanmakta ve barajın mansabındaki taşkın düzeyini öngörmeye çalışmaktadır. Analizler sırasında Çınarcık Barajı'nın mansabında gerçekleşen akış profillerini, su yüzü kotlarını ve deşarj değerlerini, tanımlanan senaryolar çerçevesinde uzamsal ve zamansal olarak tahmin etmek amacıyla, Amerika Birleşik Devletleri'nin resmi bir kurumu olan "National Weather Service" tarafından geliştirilmiş olan FLDWAV yazılımı kullanılmıştır. Bu analizler esas alınarak, gösterge düzeyinde taşkın haritaları ve bunlara göre risk altında olan yerleşim yerleri tespit edilecektir. Tez çalışması ayrıca olay öncesi alınabilecek mevcut bazı önlemlere de değinecektir.

Anahtar Kelimeler: FLDWAV, Baraj Yıkılması, Gedik, Sayısal Benzetim, Çınarcık Barajı

To the welfare of people

ACKNOWLEDGMENTS

The author wishes to express his deepest gratitude to his supervisor Asst. Prof. Dr. Zafer Bozkuş for his guidance, advice, criticism, encouragements and insight throughout the research.

The author would also like to thank the State Hydraulic Authority (DSİ) for the helpful assistance, and Kırsal ve Kentsel Gelişme Vakfı (Rural and Urban Development Foundation) for their understanding.

The author is further grateful to Mrs. Nursun Doğru for the motivation provided.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS AND ABBREVIATIONS

μ	The constant in Manning's equation (μ=1.49 for English system of units and μ=1.0 for SI units). (Eqns. 4.7 – 4.10)
A	The active cross-sectional area of flow, m^2. (Eqns. 4.1, 4.2 and 4.7)
A_o	The inactive (off-channel storage) cross-sectional area, m^2. (Eqn. 4.1)
A_s	Reservoir surface area, m^2.
B	The active river top width at water-surface elevation h, m. (Eqn. 4.2)
b	Terminal breach bottom width, m. (Figure 4.13)
B_i	Active storage area. (Figure 4.7)
BO_i	Inactive storage area (the portions of the cross-sections where flow in x-direction is negligible such as at confluence points etc) (Figure 4.7)
C_b	Offset factor in Von Thun and Gillette breach width equation (Eqn. 4.16). Varies from 6.1 m to 54.9 m as a function of reservoir storage (Tables 4.10 and 4.11).
CFL	Courant-Friedrich-Lewy
C_n	Courant Number
d	Spillway gate opening, m. (Table 4.6)
d/s	Downstream

DİE	“Devlet İstatistik Enstitüsü” (State Institute of Statistics of Turkey)
DSİ	“Devlet Su İşleri” (State Hydraulics Authority of Turkey)
DV	Flood severity parameter (Eqn. 6.1)
DXM_i	Term in FLDWAV Inputs for computational distance steps in the finite-difference Saint-Venant .
Eqn.	Equation
FLDWAV	Software Developed by National Weather Service of United States to perform numerical simulation of hydraulics, including dam failures.
F_r	Froude Number.
ft	Feet.
g	Gravitational acceleration, m/s^2. (Eqn. 4.2)
h	Water-surface elevation, m.
h_b	Height of breach, m. (Eqns. 4.18 and 4.19) (Table 4.10) (Figure 4.13)
h_b	Breach bottom elevation, m. (Figure 4.13)
h_{bm}	Terminal breach elevation, m. (Figure 4.13)
h_d	Height of dam, m.

HEPP	Hydroelectric Power Plant
h_i	Initial water surface elevation at a given cross-section along the reach, m. (Eqn. 4.4)
h_o	Dam crest elevation, m. (Figure 4.13)
h_w	Hydraulic depth of water at the time of failure with respect to the breach bottom, m. (Eqn. 4.16) (Table 4.10)
h_w^*	The dimensionless height of water above breach bottom (h_w/h_b) (Eqn 4.17) (Tables 4.10 and 4.12)
K	Overall conveyance factor for the cross-section (Eqn. 4.11)
K_c	The core wall correction factor (0.6 if dam contains a core wall; 1.0 otherwise). (Eqn. 4.17) (Table 4.10)
K_c	Conveyance factor for main channel (Eqns. 4.8 and 4.11)
K_l	Conveyance factor for left floodplain (Eqns. 4.9 – 4.11)
K_N	Conveyance factor at N^{th} distance step where N represents the downstream boundary (Eqn. 4.7)
K_o	Overtopping multiplier for Froehlich breach width Equation, 1.4 for overtopping, 1.0 for piping (for SI unit version of the Equation). (Eqn. 4.19) (Table 4.10)
K_r	Conveyance factor for right floodplain (Eqns. 4.10 and 4.11)
L	The momentum effect of lateral flow (Eqn. 4.2)
LPI	Local Partial Inertial Technique used in FLDWAV Model.

MW	Megawatts
n	The Manning's resistance coefficient (Eqns 4.7 – 4.10) (Table 4.9)
NWS	National Weather Service of United States
OK	Oklahoma State
Q	Discharge, m^3/s.
q	the lateral inflow or outflow per lineal distance along the channel (inflow is positive and outflow is negative in sign), m^2/s (Eqn. 4.1)
Q_1	The known steady discharge at t=0 at the upstream boundary, m^3/s. (Eqns. 4.3 and 4.4)
$Q_{2.33}$	The mean annual discharge at the same site with Q_{df} and W_{df}, m^3/s. (Eqn. 6.1)
Q_{df}	The peak discharge at a particular site caused by dam failure at the same site with $Q_{2.33}$ and W_{df}, m^3/s. (Eqn. 6.1)
q_i	Any user-specified lateral inflow at t=0 from tributaries existing between the user-specified cross sections spaced at intervals of Δx along the valley , m^2/s.
Q_p	Peak breach width outflow, m^3/s.
QTD	*Term for Turbine Discharge in FLDWAV Model.*
R	Hydraulic Radius (Eqns. 4.7 – 4.10)
S	Constant energy slope (Eqn. 4.7)

s_{co}, s_m	Sinuosity factors after DeLong (1986, 1989) which vary with h. (Eqns. 4.1, 4.2 and 4.21) (Figure 4.15)
S_e	The expansion/contraction slope term in Saint-Venant Equations. (Eqn. 4.2)
S_f	The channel/floodplain boundary friction slope. (Eqn. 4.2)
S_i	The additional friction slope associated with internal viscous dissipation of non-Newtonian fluids such as mud/debris flows. (Eqn. 4.2)
t	The time, hours.
TEFER	TURKEY EMERGENCY FLOOD & EARTHQUAKE RECOVERY
TEH	Term to denote the total duration of simulation in FLDWAV Model.
t_f	Failure time (breach formation time), hours. (Eqn. 4.18) (Table 4.10)
u/s	Upstream
US	United States
USA	United States of America
USDA	United States Department of Agriculture
V_r	Reservoir volume, m^3. (Tables 4.8 and 4.10)
V_w	Volume of water stored above breach invert at time of failure, m^3. (Eqns. 4.18 and 4.19) (Table 4.10)

W_{bottom}	Embankment widths at the bottom of the embankment, m, (Eqn. 4.17) (Table 4.12)
W_{crest}	Embankment widths at the crest of the embankment, m, (Eqn. 4.17) (Table 4.12)
W_{df}	The maximum width of flooding caused by dam failure at the same site with Q_{df} and $Q_{2.33}$, m. (Eqn. 6.1)
W_f	The effect of wind resistance on the surface of the flow (Eqn. 4.2)
WSEL	Water Surface Elevation, m.
x	The longitudinal distance along the river (channel/floodplain), m.
y	The flow depth, m. (Eqn. 4.20)
Z	Side slope of breach. (Z horizontal:1 Vertical) (Eqn. 4.17) (Tables 4.1, 4.2, 4.10 and 4.12) (Figure 4.13)
β	The momentum coefficient for velocity distribution (Eqn. 4.2)
Δt	Time Step in Iteration Scheme (Eqn. 4.12) (Figure 4.1)
Δx_{c_l}	mean flow-path distance along the floodplain (Eqn. 4.21) (Figure 4.15)
Δx_i	Computational distance steps in the finite-difference Saint-Venant equations.
Δx_l	Flow path distance along the meandering channel (below floodplain elevation) (Eqn. 4.21) (Figure 4.15)
θ	Weighting factor in forward difference quotient (Eqns. 4.13 and 4.14)

σ	Llocal parameter used by FLDWAV Model to decide between using the momentum equation with full, partial or no inertial terms included when flow is subcritical, near critical/critical, or supercritical, respectively.
ψ	Any variable (Q, h, A, A_o, s_{co}, s_m, etc.) (Eqns. 4.12 – 4.14) (Figure 4.1)
Φ	Term in FLDWAV Model to define the degree of unsteadiness of the flow. (Eqn. 4.20)
$\overline{B}$	Average breach width, m. (Eqns. 4.16 and 4.19) (Table 4.10)
τ	Failure time (Breach formation time), hours.
$\overline{A}$	Average Cross-sectional Area between adjacent distance steps, m^2. (Eqns. 4.4 and 4.5)
$\overline{S}_f$	The average channel/floodplain boundary friction slope. (Eqn. 4.4)
$\overline{S}_i$	The average additional friction slope associated with internal viscous dissipation of non-Newtonian fluids such as mud/debris flows. (Eqn. 4.4)
Δx	Distance Step in Iteration Scheme, m. (Eqn 4.3, 4.4 and 4.13) (Figure 4.1)
ρ_o	Breach formation parameter. (Tables 4.1 and 4.2)
B_{r_k}	The top-width at k^{th} elevation for right floodplain, m. (Figure 4.6)
B_{c_k}	The top-width at k^{th} elevation for main channel, m. (Figure 4.6)
B_{l_k}	The top-width at k^{th} elevation for left floodplain, m. (Figure 4.6)
$\overline{W}^*$	The dimensionless average embankment width $(W_{crest}+W_{bottom})$ / $(2h_b)$. (Eqn. 4.17) (Tables 4.10 and 4.12)

CHAPTER 1

INTRODUCTION

1 INTRODUCTION

Dams and Hydroelectric Power Plants have seen much use in Turkey, which has abundant sources of water suited to exploitation for the purpose of allocation of irrigation water, generation of electricity for public use and others. Altinbilek (2002a) comments that Turkey has a hydroelectric potential that would require a total of 526 HEPPs operational to exploit completely.

As for the purposes of dams, the Engineering Manual 1110-2-1420 by U.S. Army Corps of Engineers (1997) lists the Reservoir Purposes as follows:

- Flood Control
- Navigation
- Hydroelectric Power
- Irrigation
- Municipal and Industrial Water Supply
- Water Quality
- Fish and Wildlife
- Recreation
- Water Management Goals and Objectives

Altinbilek (2002b), in his study, further comments on the role of dams in development. Despite their great benefit to public; dams, however, might also lead to catastrophes such as dam failures because of several reasons including but not limited to failure to carry out necessary maintenance, lack of proper risk assessments, occurrence of an unforeseen runoff flood of great magnitude, or incompetent/improper operation.

In the event of a failure of a dam by the overtopping runoff flood, a breach is formed on the dam hull, which gradually expands in terms of hours or even minutes. The water stored behind the reservoir is then set free uncontrollably in great amounts and causes damage along its route in great proportions. A dam break flood is practically in much greater magnitude in comparison to a runoff flood (which occurs due to precipitation, snow melt etc). In the absence of preemptive measures geared towards such a risk, such as an Emergency Action Plan, the settlements and the residents located downstream of the dam and on the route of such a dam failure flood are put to great risk.

It is, however, quite possible to risk-assess existing dams so as to identify the risks involved and to decide on and implement pre-event measures. A complete risk analysis and the identification of pre-event measures for any given dam would surely involve several aspects of a dam system and require a versatile approach. The study delineated hereunder, however, is meant to approach the problem from dam failure perspective and is solely focused on the risk thereof, the forecast of the resulting dam break flood, the inundated areas, and the settlements at risk.

1.1 OBJECTIVE OF THE STUDY

The objectives of the study delineated hereunder are, therefore, to:

- Forecast the dam failure floods produced at Çınarcık Dam under a set of scenarios, where the dam failure is initiated by a runoff flood arriving at the dam at the upstream end of the analysis domain (i.e. the routing reach).
- Identify the inundated areas and the corresponding spatial and time-wise water depths and discharges for each scenario.
- Identify the settlements at risk judging by the time of arrival of the dam break flood to each settlement on the route.
- Present preliminary suggestive measures that can be implemented in areas under risk.

There are some previously conducted studies that focus on dam-failure analyses and emergency management measures by Bozkuş (1994, 1998a, 1998b, 2001, 2002), Merzi et al. (1997) and Sezer (1992). This thesis work is further aimed to supplement the aforementioned studies conducted in Turkey in that regard so as to draw due attention to the topic.

CHAPTER 2

DAM FAILURES

2 DAM FAILURES

The Engineering Manual 1110-2-1420 by U.S. Army Corps of Engineers (1997) lists the common 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 liquification of earthen dams from earthquakes, or landslide-generated waves within the reservoir.

On the other hand, the same manual lists the prominent causes as follows:

- (1) Earthquake.
- (2) Landslide.
- (3) Extreme storm.
- (4) Piping.
- (5) Equipment malfunction.
- (6) Structural damage.
- (7) Foundation failure.
- (8) Sabotage.

Another factor that might contribute to a dam-related flood at downstream (whether a dam-break induced failure or not) may be mis-operation. Davis (2001) 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 (Ref. 12 Nigeria, Ref. 13 Dominican Republic). 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. Mis-operation does not include structural failure of the dam.

Regardless of the reason of failure, the event of a dam failure is quite a disaster, even catastrophic in the absence of pre-event measures in place. Although there has not yet been any dam failure in Turkey so far, there are several examples throughout the world. The list of Human and Economic Consequences of Dam Failure by Graham (2001) depicts a dire image (Table 2.1):

Table 2-1 Data as to Previously Observed Dam Failures in USA (Graham, 2001)

Dam	Date and Time of Failure	Height (ft)	Volume Released (acre-ft)	Deaths	Economic Damage (in million USD)
Williamsburg Dam, MA (Mill River Dam)	May 16, 1874 at 7:20 a.m.	43	307	138	Not Available
South Fork Dam, PA, (Johnstown Dam)	May 31, 1889 at 3:10 p.m.	72	11,500	2,209	Not Available
Walnut Grove Dam, AZ,	22 February 1890 at 2 a.m.	110	60,000	about 85	Not Available
Austin Dam, PA	September 30, 1911 at2 p.m.	50	850	78	14
St. Francis Dam, CA	March 12-13, 1928 at midnight	188	38,000	420	14
Castlewood Dam, CO	August 2-3, 1933 at midnight	70	5,000	2	2

Table 2-1 Data as to Previously Observed Dam Failures in USA (Graham, 2001) (Cont'd.)

Dam	Date and Time of Failure	Height (ft)	Volume Released (acre-ft)	Deaths	Economic Damage (in million USD)
Baldwin Hills Dam, CA	December 14, 1963 at 3:38 p.m.	66	700	5	11
Buffalo Creek, WV (Coal Waste Dam)	February 26, 1972 at 8 a.m.	46	404	125	50
Black Hills Flood, SD (Canyon Lake Dam)	June 9, 1972 at about 11 p.m.	20	700	N/A	160
Teton Dam, ID	June 5, 1976 at 11:57 a.m.	305	250,000	11	400
Kelly Barnes Dam, GA	November 6, 1977 at 1:20 a.m.	40	630	39	3
Lawn Lake Dam, CO	July 15, 1982 at 5:30 a.m.	26	674	3	31
Timber Lake Dam, VA	June 22, 1995 at 11 p.m.	33	1,449	2	0

Turkey has made large-scale investments to dam projects, particularly in Southeastern Anatolia, for various purposes including water management, provision of irrigation water, generation of electricity, flood control and others. Hence, there are several dams in Turkey that may require risk assessments and pre-event failure analyses so as to mitigate potential consequences and take pre-event measures in advance.

Figure 3.1 below and Figures 3.2 and 3.3 on the next page provide a better visualization of the dam-failure concept and aftermath thereof. Figure 2.1 presents the snapshots of an embankment breach test of a homogeneous non-plastic sandy soil conducted at the ARS Hydraulic Laboratory, Stillwater, OK, whereas Figures 2.2 and 2.3 present a snapshot of the aftermath of the Coon Creek Dam that failed in 1978 in US, and the Clayton County Waste Water Pond Dam that failed in 1982 in US, respectively.

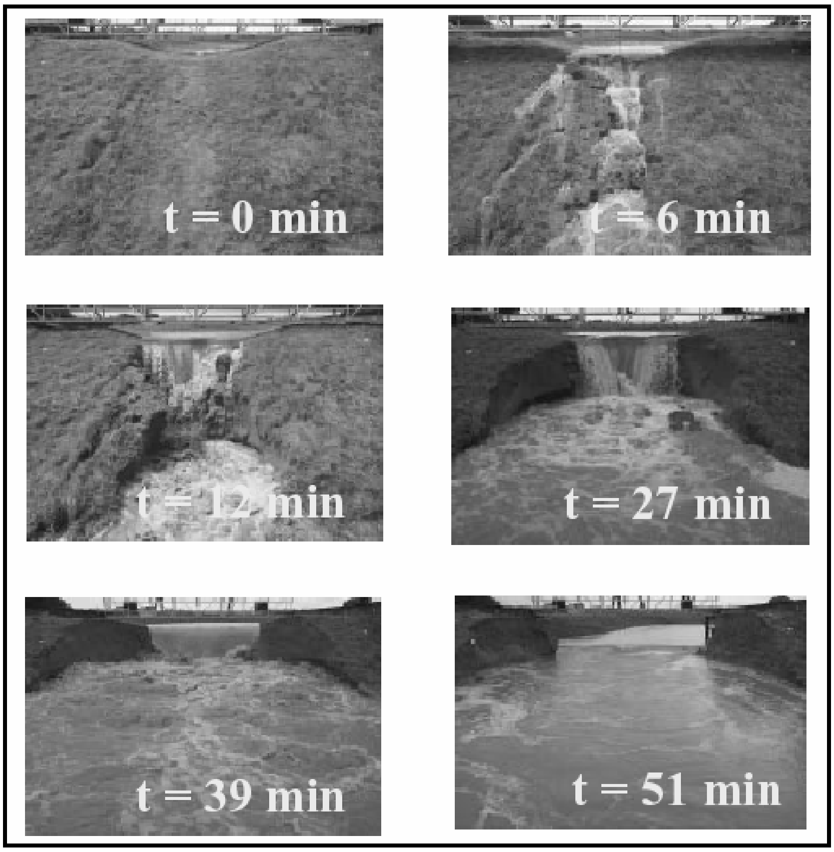

Figure 2-1 Time series of an embankment breach test of a homogeneous non-plastic sandy soil conducted at the ARS Hydraulic Laboratory, Stillwater, OK. (Temple et al., 2001)

Figure 2-2 Coon Creek Dam; a typical NRCS flood control dam from the 1960's, which was built in 1962 and failed in 1978 during the first significant reservoir filling. (Irwin, 2001)

Figure 2-3 Clayton County Waste Water Pond Dam, US, 1982. (Fiegle II, 2001)

CHAPTER 3

CASE STUDY TOPIC: ÇINARCIK DAM

3 CASE STUDY TOPIC: ÇINARCIK DAM

Emet – Orhaneli Power Project is composed of a series of multi-purpose dams and Hydroelectric Power Plants (HEPPs) located on the branches of Mustafakemalpaşa Creek, namely, Emet Creek (former name: Aliova Creek) and Orhaneli Creek (former name: Kocasu) in Marmara Geographical Region of Turkey.

Çınarcık Dam of 123 m. in height was planned to be located on Orhaneli Branch of Mustafakemalpaşa Creek at 210 m. invert elevation and approximately 30 km. east of Mustafakemalpaşa District by air distance. The Uluabat HEPP of 120 MW installed power fed by a power tunnel of 11270 m., on the other hand, was planned to be located at the south-east banks of Uluabat Lake, which is 30 km west of the city of Bursa.

An overview of the modeling domain and the case study topic, Çınarcık Dam, is given in Figure 3.1, and Figures 3.2 and 3.3, respectively.

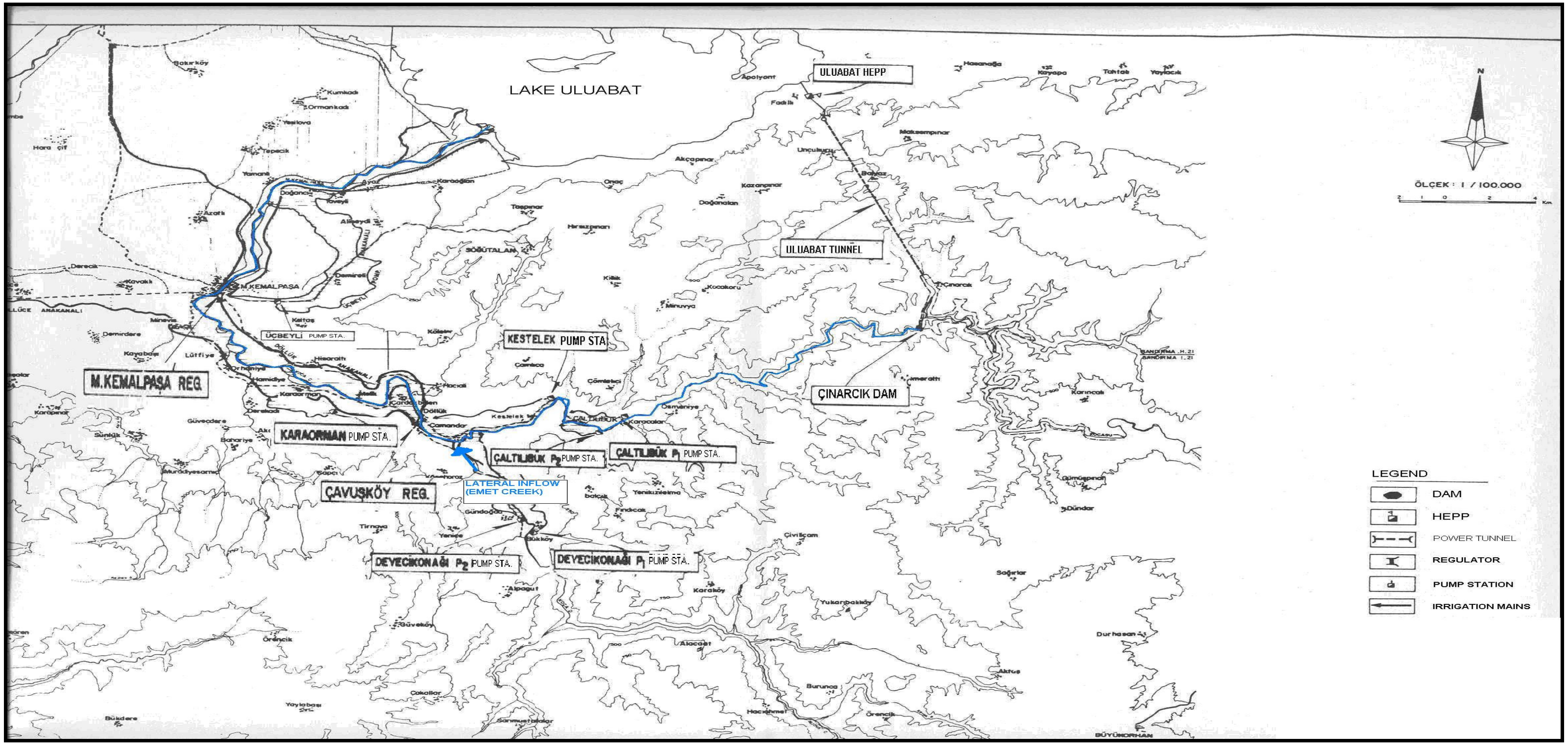

Figure 3-1 An Overview of the Modeling Domain (Not to Scale)

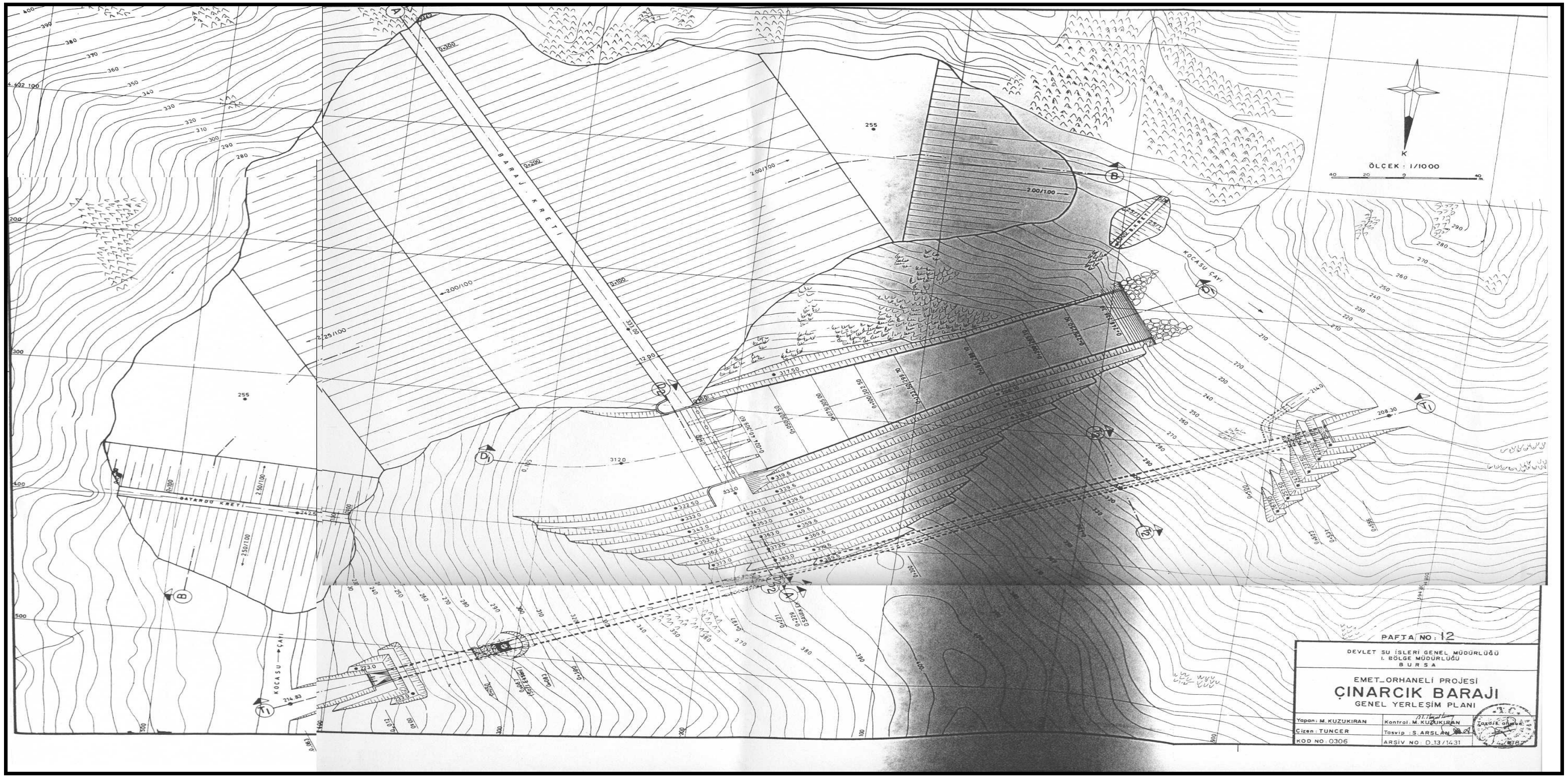

Figure 3-2 Plan View of Çınarcık Dam (Not to Scale)

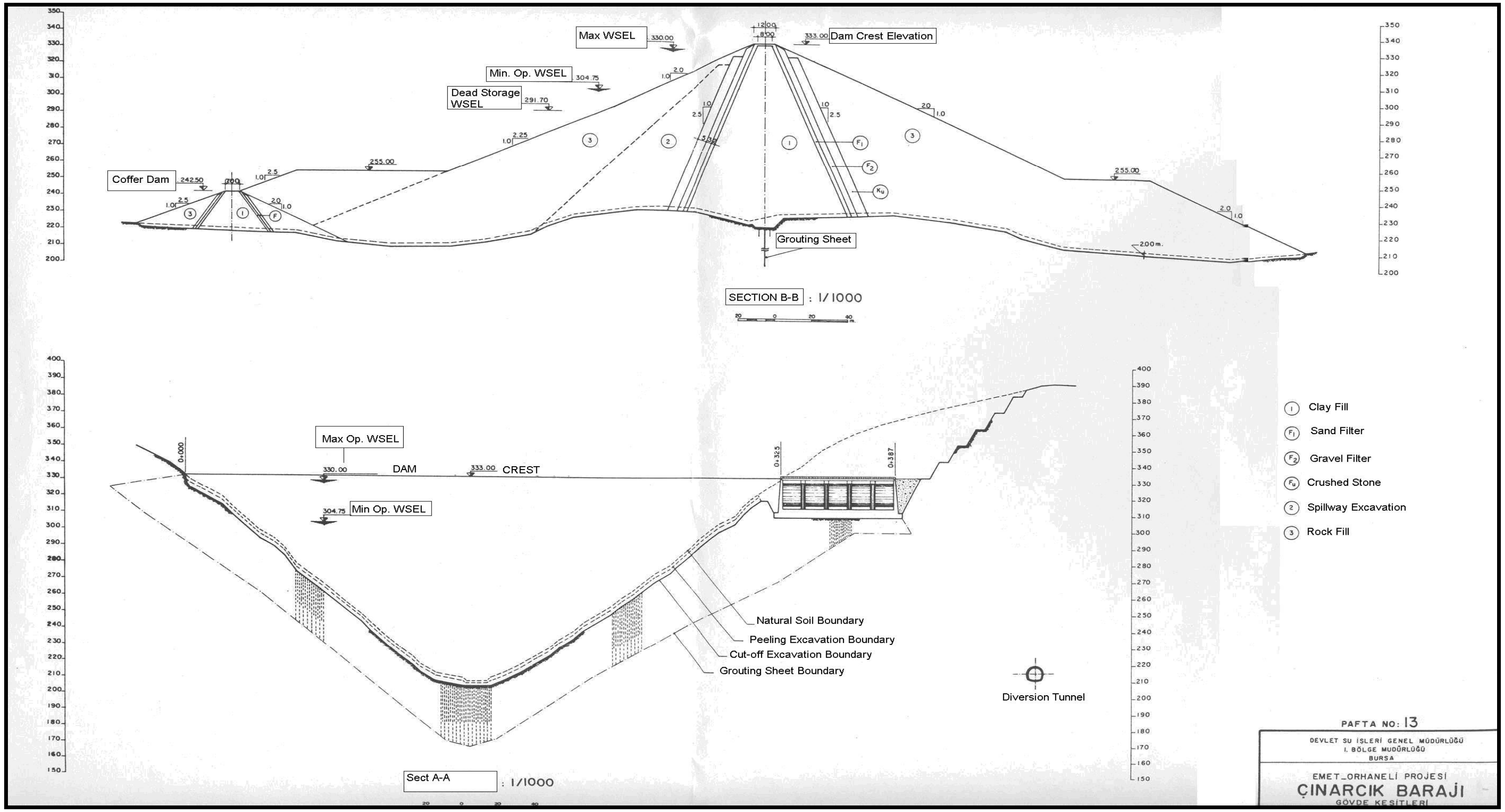

Figure 3-3 Views of Dam along B-B and A-A sections

The technical specifications of Çınarcık Dam's components related to this thesis study can be summarized as follows in Table 3.1 as per DSİ:

Table 3-1 Technical Specifications of Çınarcık Dam

Çınarcık Dam	
1. Dam Body	
Purpose	Power Generation + Irrigation
Type	Rock-Fill
Height from Talveg	123.00 m.
Height from Foundation	125.00 m.
Crest Elevation	333.00 m.
Talveg Elevation	210.00 m.
Crest Length (exclusive of spillway length)	325.00 m.
Crest Width	12.00 m.
u/s slope	1/2 - 1/2.25
d/s slope	1/2
2. Reservoir	
Max. Water Elevation	330.00 m.
Max. Operating Water Elevation	330.00 m.
Min. Operating Water Elevation	304.75 m.
3. Spillway	
Type	Front Inlet, Radial Gated
Capacity	5191.80 m^3/s
Gate Dimensions	10.00 m x 14.00 m (5 gates)
Elevation of Approach Channel	312.00 m.
Spillway Crest Elevation	316.00 m.
Total Spillway Crest Length	60.00 m.
Length of Discharge Channel	223.60 m.
Type of Energy Dissipater	Deflector Bucket
4. Bottom Outlet	
Purpose	Irrigation Water Supply, Discharge of Reservoir at emergencies
Outlet Elevation	211.05 m.
Capacity at max. water elevation	71.78 m^3/s

Table 3-1 Technical Specifications of Çınarcık Dam (Cont'd.)

Çınarcık Dam	
5. Uluabat Power Tunnel	
Tunnel Capacity	48.89 m^3/s
6. Uluabat HEPP	
Turbine Design Discharge	48.90 m^3/s

CHAPTER 4

METHODOLOGY

4 METHODOLOGY

In this case study, the overtopping and piping failure of the Çınarcık Dam caused by a large storm hydrograph will be simulated under 13 different scenarios using the FLDWAV software developed by the National Weather Service (NWS), which is one of the state institutions in the USA. The FLDWAV model, the scenarios, the modeling domain and the inputs into the model will be discussed in detail in the following sections.

4.1 AN OVERVIEW OF FLDWAV SOFTWARE

FLDWAV Model is a generalized flood routing model for unsteady flow simulation, and the main equations that the model uses are complete one-dimensional Saint-Venant Equations of unsteady flow. The Saint-Venant Equations are coupled with the internal boundary equations to represent rapidly varying flow through structures such as dams, bridges and embankments. Furthermore, external boundaries are also defined into the model at the upstream and downstream extremities of the modeling reach, which are represented by their specific equations. The system of equations thus formed is solved by an iterative, nonlinear, weighted-four-point implicit finite-difference method. (Fread et al., 1998)

The equations of Saint-Venant utilized by the FLDWAV Model are expressed in conservation form and also include terms to account for the effect of expansion/contraction, channel sinuosity, and non-Newtonian flow. The equations are basically composed of a conservation of mass equation, Eqn. (4.1), and conservation of momentum equation, Eqn. (4.2), respectively, as given below (Fread et al., 1998):

Conservation of mass equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial s_{co}(A + A_o)}{\partial t} - q = 0 \qquad (4.1)$$

Conservation of momentum equation:

$$\frac{\partial(s_m Q)}{\partial t} + \frac{\partial(\beta Q^2/A)}{\partial x} + g\,A\,(\frac{\partial h}{\partial x} + S_f + S_e + S_i) + L + W_f B = 0 \qquad (4.2)$$

where Q is the discharge or flow (- if directed upstream), h is the water-surface elevation, A is the active cross-sectional area of flow, A_o is the inactive (off-channel storage) cross-sectional area, s_{co} and s_m are sinuosity factors which vary with h, x is the longitudinal distance along the river(channel/floodplain), t is the time, q is the lateral inflow or outflow per lineal distance along the channel (inflow is positive and outflow is negative in sign), β is the momentum coefficient for velocity distribution, g is the acceleration due to gravity, S_f is the channel/floodplain boundary friction slope, S_e is the expansion-contraction slope, S_i is the additional friction slope associated with internal viscous dissipation of non-Newtonian fluids such as mud/debris flows, B is the active river topwidth at water-surface elevation h, L is the momentum effect of lateral flow and W_f is the effect of wind resistance on the surface of the flow (Fread et al., 1998).

The equations defining individual terms of the equations can be found in detail in other hydraulics related sources and specifically in “NWS FLDWAV Model” Manual (Fread et al., 1998).

4.1.1 INITIAL CONDITIONS

The initial conditions for the state of flow (h_i and Q_i) along the model reach are required at all cross-sections at the beginning of the simulation so as to solve the Saint-Venant unsteady flow equations mentioned within the previous **Section 4.1**. The initial conditions may either be steady or unsteady. In the unsteady case, the WSEL (h_i) and discharge (Q_i) at each cross-section are known and user-specified to the model for the related cross-sections. In the steady-state case, the FLDWAV Model assumes the flow to be steady, non-uniform flow, and the flow at each cross-section is initially computed using Eqn. (4.3) below:

$$Q_i = Q_{i-1} + q_{i-1}\Delta x_{i-1} \;, \; i = 2, 3, \ldots N \qquad (4.3)$$

where Q_1 is the known steady discharge at t=0 at the upstream boundary, and q_i is any user-specified lateral inflow at t=0 from tributaries existing between the user-specified cross sections spaced at intervals of Δx along the valley (Fread et al., 1998).

Unless user-specified, the FLDWAV Model, as a first step, calculates the steady-state water surface elevations for each cross-section along the reach. In case of sub-critical flow, this is accomplished by using iterative Newton-Raphson method to solve the back-water Eqn. (4.4) given below for h_i:

$$(Q^2/A)_{i+1} - (Q^2/A)_i + g\overline{A}_i (h_{i+1} - h_i + \Delta x_i \overline{S}_f + \Delta x_i \overline{S}_i) = 0 \tag{4.4}$$

in which $\overline{A}$, and $\overline{S}_f$ are defined by Eqns. (4.5) and (4.6) given below, respectively (Fread et al., 1998):

$$\overline{A} = \frac{A_i + A_{i+1}}{2} \tag{4.5}$$

$$\overline{S}_f = \frac{n_i^2 \overline{Q}\left|\overline{Q}\right|}{\left[\mu^2 \overline{A}^2 \overline{R}^{4/3}\right]} = \frac{\overline{Q}\left|\overline{Q}\right|}{\overline{K}^2} \tag{4.6}$$

The $\overline{S}_i$ term is only significant when the fluid is quite viscous and severely non-Newtonian. (Fread et al., 1998)

Eqn. (4.4) is a simplified version of the momentum equation (Eqn 4.2), and the first term in Eqn (4.4) is taken as zero in case of steady flow, and L and W_f are assumed to be zero. In case of sub-critical flow along the entire modeling reach, the computations start from downstream to upstream, and the starting WSEL may be user-specified if known, or obtained through the downstream boundary condition for either a discharge (Q_N) or elevation h_N at t=0 (Fread et al., 1998).

The Manning equation (Eqn 4.7) with a constant energy slope term (S), given below is used to compute h_N:

$$Q_N = \mu/n_N A_N R_N^{2/3} S^{1/2} = K_N S^{1/2} \tag{4.7}$$

which is then solved iteratively for h_N using the Newton-Raphson method, where μ is a units conversion factor 1.49 for English units and 1.0 for SI units, n_N is the Manning roughness coefficient, A_N is the cross-sectional area, R_N is the hydraulic radius, K is the flow conveyance factor and subscript N denotes the downstream boundary. The energy slope (S) is approximated by using the channel bottom slope (S_o) associated with the most downstream Δx_{N-1} reach. (Fread et al, 1998)

When the conveyance factor (K) is used to represent S_f, the river (channel/floodplain) cross-sectional properties are designated as left floodplain, channel, and right floodplain rather than as a single composite channel/floodplain section. Special orientation for designating left or right is not required as long as consistency is maintained. The conveyance factors for left floodplain, main channel and right floodplain are evaluated through Eqns. (4.8), (4.9) and (4.10), respectively, whereas Eqn (4.11) yields the total conveyance for a given cross-section (Fread et al., 1998):

$$K_l = \frac{\mu}{n_l} A_l R_l^{2/3} \tag{4.8}$$

$$K_c = \frac{\mu}{n_c} \frac{A_c R_c^{2/3}}{s_m^{1/2}} \tag{4.9}$$

$$K_r = \frac{\mu}{n_r} A_r R_r^{2/3} \tag{4.10}$$

$$K = K_l + K_c + K_r \tag{4.11}$$

On the other hand, if the flow is supercritical, the computations for h_i proceed from upstream to downstream. In this case, Eqn. (4.2) is again used, but to compute h_{i+1}. The starting water surface elevation (h_1) is obtained by using Eqn. (4.4) with N replaced by 1. (Fread et al., 1998)

4.1.2 FLDWAV's SOLUTION TECHNIQUE FOR SAINT-VENANT EQUATIONS

The FLDWAV Model utilizes weighted four-point scheme used by Preissman (1961, cited in Fread et al., (1998)), Chaudry and Contractor (1973, cited in Fread et al., (1998)) and Fread (1974a, 1974b, 1978b; cited in Fread et al., (1998)), which, being an implicit scheme, is advantageous as it does not require equal distance and time steps and its stability-convergence properties can conveniently be controlled (Fread et al., 1998).

In the weighted, four-point implicit finite-difference scheme, the continuous x-t (space-time) region in which solutions of h and Q are sought, is represented by a rectangular net of discrete points shown in Figure 4.1. The net points are determined by the intersection of lines drawn parallel to the "x" and "t" axes. Those parallel to the t-axis represent locations of cross sections; they have a spacing of Δx_i, which need not be constant. Those parallel to the x-axis represent time lines; they have a spacing of Δt_j, which also need not be constant. Each point in the rectangular network can be identified by a subscript (i), which designates the x-position, and a superscript (j), which designates the particular time line. The time derivatives are approximated by a forward-difference quotient centered between the i^{th} and i+1 points along the x-axis, by Eqn. (4.12) (Fread et al, 1998).

$$\frac{\partial \psi}{\partial t} = \frac{\psi_i^{j+1} + \psi_{i+1}^{j+1} - \psi_i^j - \psi_{i+1}^j}{2\,\Delta t_j} \qquad (4.12)$$

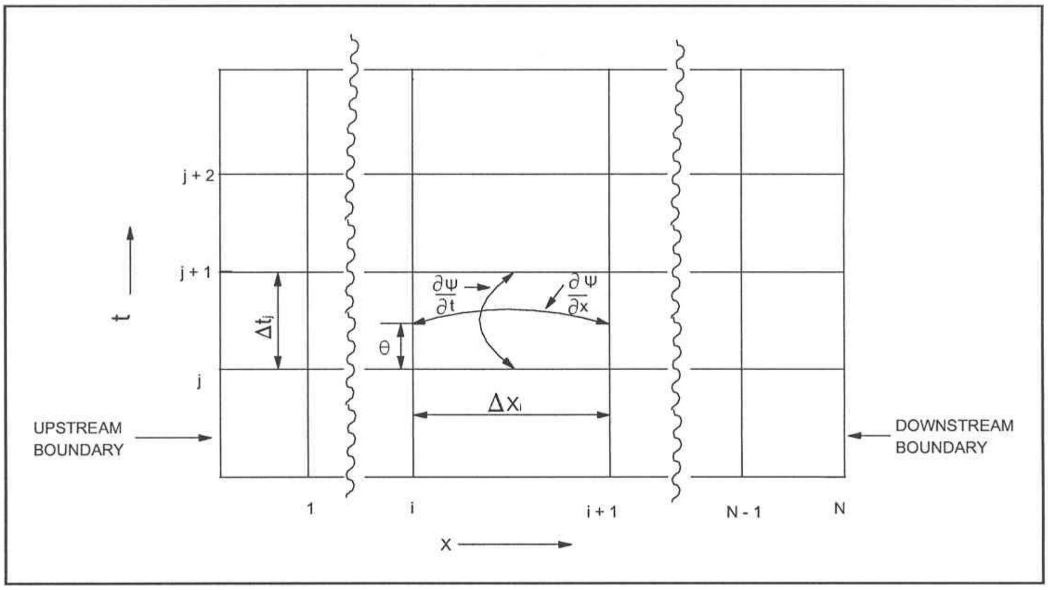

Figure 4-1 Discrete x-t Solution Domain (Fread et al., 1998)

where ψ represents any variable (Q, h, A, A_o, s_{co}, s_m, etc.). (Fread et al., 1998)

The spatial derivatives, on the other hand, are approximated by a forward-difference quotient positioned between two adjacent time lines according to weighting factors of θ and (1-θ) , by Eqn. (4.13) (Fread et al., 1998):

$$\frac{\partial \psi}{\partial x} = \theta \, \frac{\left[\psi_{i+1}^{j+1} - \psi_{i}^{j+1}\right]}{\left[\Delta x_{i}\right]} + (1 - \theta)\frac{\left[\psi_{i+1}^{j} - \psi_{i}^{j}\right]}{\left[\Delta x_{i}\right]} \tag{4.13}$$

Variables other than derivatives are approximated at the time level where the spatial derivatives are evaluated by using the same weighting factors, by Eqn. (4.14) (Fread et al., 1998):

$$\psi = \theta \frac{\left[\psi_{i}^{j+1} + \psi_{i+1}^{j+1}\right]}{[2]} + (1 - \theta)\frac{\left[\psi_{i}^{j} + \psi_{i+1}^{j}\right]}{[2]} \tag{4.14}$$

Substituting the finite-difference operators defined by Eqns. (4.12-4.14) in place of the derivatives and other variables in Eqns. (4.3) and (4.4), yields the weighted, four-point implicit, finite difference equations used by the FLDWAV Model, the details of which can be found in the Manual for the FLDWAV Model (Fread et al., 1998).

This set of equations is then solved by using the Newton-Raphson iteration scheme, coupled with the user-specified boundary conditions at upstream and downstream extremities of the modeling reach.

4.1.3 QUOTES AS TO FLDWAV FROM THE LITERATURE

The proceedings report prepared by the USDA (United States Department of Agriculture) as to the FEMA Workshop held between 26th and 28th June 2001 in Oklahoma City in the USA includes the complete set of papers presented during the workshop. A paper presented by Jonh. C. Ritchey from the Department of Environmental Protection: Dam Safety Section of the State of New Jersey is of particular interest to the FLDWAV model, and hence to the thesis study. Ritchey (2001) concludes that some states in United States reported using the Flood Wave Model (FLDWAV) and no state reported any difficulties with the FLDWAV model, however, it was the general consensus that limited information and training has been made available for the FLDWAV model.

Ritchey (2001) further recommends, in his paper, the use of a set of equations derived by Froehlich in 1987 to predict the breach parameters, which will be discussed in detail in **Section 4.5.2.2 Breach Parameters.**

4.2 MODELING DOMAIN

The modeling domain for the purpose of this thesis study is composed of Çınarcık Dam and its reservoir at the upstream boundary; the downstream-most cross-section just before the Uluabat Lake at the downstream boundary; and the reach of about 70 km lying in between these two boundaries. The reach starts with Orhaneli Creek, where the Çınarcık Dam is located, for some 30 km. at the upstream end and then confluences with the Emet Creek. Both creeks then join with the Mustafakemalpaşa Creek, which runs for some 40 km. before terminating at the Uluabat Lake to the North. Flow coming from Emet Creek was specified as lateral flow to the reach based on the compiled data as to monthly observation of flow values on Emet River by DSİ's observation station #03-04 located just upstream of the confluence point. Hence, the modeling reach is made up of a 30 km. portion of Orhaneli Creek till the confluence followed by the 40 km. long Mustafakemalpaşa Creek till the Uluabat Lake from the confluence, for a total of some 71 km., as previously-given in Figure 3.1.

There are several villages and districts located in the vicinity of Orhaneli and Mustafakemalpaşa Creeks. Mustafakemalpaşa District through which the latter creek flows is therefore one of the most critical and populated settlement in the vicinity in terms of a potential flood risk.

The details of input data will be discussed in detail in the next section.

4.3 INPUT PARAMETERS

The FLDWAV model requires several input data for an accurate modeling of the simulated Dam Break Failure and the resultant WSELs and discharge values spatial and temporal-wise. These data can be categorized in two groups as:

- External Data
- Internal Data

4.3.1 EXTERNAL DATA

The term "*external data*" implies that this data has to be acquired from external sources such as State Institutions or Universities so as to base it on official documents and measurements and that it lies outside the scope of judgment of the user of the FLDWAV Model. In case of modeling a Dam Break Failure and the resultant flood, these data mainly include but not limited to:

- Geometrical and Technical Data of the subject dam, its spillway(s), and reservoir;
- Inflow Storm Hydrograph to the subject dam (which will initiate the failure and is of much larger magnitude than the regular hydrographs observed in the modeling domain);
- Cross-sectional data of the river reach downstream of the dam to define the geometry of the reach for flood routing purposes, and the stations kms of these cross-sections;
- Data as to upstream and downstream boundaries of the modeling domain (Çınarcık Dam is the upstream boundary; downstream boundary was specified as rating curve in the case of this thesis study); and
- Inflow Hydrographs or long term (in the order of days) Discharge Values for lateral flow(s).

In this regard, the following data was acquired from DSİ (State Hydraulics Authority) of Turkey, which is the major agency responsible for managing and executing Turkey's dam and HEPP projects *inter alia*:

- The Catastrophic Inflow Hydrograph (Storm/Runoff Flood) to the dam at upstream boundary;
- Geometrical and Technical Data of Çınarcık Dam, which is located at the upstream end of the modeling reach;
- Cross-sectional data for the reach (i.e. Orhaneli and Mustafakemalpaşa Creeks in the modeling domain);
- Station Kms of Cross-sections;
- Spillway Rating Curve of Çınarcık Dam;
- Discharge through the turbines;
- Reservoir Surface Area versus Water Surface Elevation (WSEL) table for Çınarcık Dam;
- Topographic maps of the modeling domain of 1/100000 scale;

- Monthly observed flow values for the Emet Creek, which was specified as a lateral flow to the Orhaneli Creek.

Unfortunately, DSİ was unable to provide:

- Manning roughness values for each cross-section or an average Manning roughness value for Orhaneli and Mustafakemalpaşa Creeks due to unavailability of previous measurements on those rivers. Although the author examined the detailed design dossiers of the Çınarcık Dam, he was unable to come across any indications as to the Manning roughness values used during the design phase of the said dam. Neither the "*Operations and Maintenance*" nor the "*Surveys*" Department inside DSİ were able to provide any measured Manning roughness values for the said creeks. Therefore, the author had to use fictitious Manning values during modeling, based on Chow (1959).
- Maps of finer detail because of the national security regulations set by the "General Commandership of Maps". Therefore, the Author had to rely on the provided maps of 1/100000 scale.

The lack of accurate Manning roughness data and the small scale of the maps available do present, to a certain extent, some unreliability to the outputs. This issue will be discussed in detail **in Section 5 Outputs of Modeling**.

4.3.2 INTERNAL DATA

The term "*internal data*" implies that these are the data that either the operator has to base on his/her own judgment, on the Model's Manual or on pertinent equations or preceding academic studies. These data can mainly be listed as follows:

- **Initial Conditions:** The initial WSELs and discharges along the reach. These can either be user-specified or calculated by the FLDWAV Model through the Newton-Raphson iteration scheme for a couple of preliminary runs, except for the reservoir WSEL behind a dam (if any), which has to be user-specified.
- **Breach Parameters:** Parameters pertaining to the Breach such as the width, height, geometry and the formation time of the Breach.
- **Failure Initiation Criteria**: The FLDWAV Model presents two alternatives to the user as failure initiation criteria for failures. The criteria can either be a specific water surface elevation, which initiates dam failure in case the water elevation

behind the dam exceeds it or a specific time step at which the failure commences.

- **Finite Differentiation Technique:** The FLDWAV Model presents a set of options to the user to specify the finite differentiation method for each sub-reach. The stability and accuracy of these models differ from each other.
- **Type of Fluid**: The FLDWAV Model lets user specify the type of fluid, either Newtonian or Non-Newtonian (Mud/Debris Flow).
- **Levee Modeling:** The FLDWAV Model allows the user to specify levees to account for interaction between the levees and the main channel.
- **Low Flow Filter Option:** The FLDWAV Model has a built-in low flow filter to prevent occurrence of unrealistic WSEL and discharge values along the reach during iteration calculations. It thereby prevents occurrence of practically impossible values such as WSEL lower than the waterbed elevation itself.
- **Automatic Manning n Calibration Option:** The FLDWAV Model has a built-in feature called "Automatic Manning n Calibration" that calculates the Manning roughness values for a given reach based on a previously observed hydrograph along the reach.
- **Time and Distance Steps for Iteration:** The FLDWAV Model allows for either manual specification of automatic calculation of both the distance and the time steps.
- **Warm-up Procedure Option:** The FLDWAV Model has a Warm-Up Procedure option that holds the boundary conditions constant during several computational time steps, thereby refining the initial conditions for the actual calculations where the boundaries are transient.
- **Volume Losses Option:** The FLDWAV Model allows the user to specify the volume losses occurring through the reach.
- **Landslide Option:** The FLDWAV Model can account for the effect of a landslide to the dam reservoir.
- **Expansion/Contraction Coefficients:** The FLDWAV Model has a term for expansion/contraction effects integrated into the momentum equation Component (Eqn. 4.2) of the Saint-Venant Equations through the S_e, expansion/contraction slope term.
- **Sinuosity Coefficients:** Similarly, the continuity and momentum components (Eqn. 4.1 and 4.2, respectively) of the Saint-Venant equations possess the terms S_{co} and S_m, respectively, so as to account for the effect of sinuosity.

The details and justification of the selection of values for these parameters will be discussed under **Section 4.5.1 JUSTIFICATION OF EXTERNAL DATA.**

4.4 MODELING SCENARIOS

To investigate a potential risk of dam-failure induced flood to the settlements in the vicinity of Orhaneli and Mustafakemalpaşa Creeks, the modeling domain was tested for a set of scenarios under two main categories, namely, overtopping dam failure and piping dam failure.

The overtopping failure scenarios are tabulated in Table 4.1.

Table 4-1 Scenario Definitions for Overtopping Failure Modeling

SCE-NARIO ID	Season	Lateral Flow from Emet Creek (m^3/s)	Terminal Breach Bottom Width (m)	Terminal Breach Height (m)	Breach Formation Time (hours)	Side Slope of Breach (1V : zH)	Spillway Gates Opening Width (d) (m)	Type of Failure	Failure Criteria (m)	Breach Formation Parameter (ρ_o)
S1	Summer	Gradually increasing flow of Q_{max}=350 m^3/s	70	113	1.3	1.27	10 (Completely Open)	Overtop-ping	333.16	1
S2	Summer	Gradually increasing flow of Q_{max}=350 m^3/s	70	113	1.3	1.27	7.5	Overtop-ping	333.16	1
S3	Summer	Gradually increasing flow of Q_{max}=350 m^3/s	70	113	1.3	1.27	5	Overtop-ping	333.16	1
S4	Summer	Gradually increasing flow of Q_{max}=350 m^3/s	70	113	1.3	1.27	2.5	Overtop-ping	333.16	1
S5	Summer	Gradually increasing flow of Q_{max}=350 m^3/s	70	113	1.3	1.27	0 (Completely Closed)	Overtop-ping	333.16	1
W1	Winter	Gradually increasing flow of Q_{max}=680 m^3/s	70	113	1.3	1.27	10 (Completely Open)	Overtop-ping	333.16	1
W2	Winter	Gradually increasing flow of Q_{max}=680 m^3/s	70	113	1.3	1.27	7.5	Overtop-ping	333.16	1
W3	Winter	Gradually increasing flow of Q_{max}=680 m^3/s	70	113	1.3	1.27	5	Overtop-ping	333.16	1
W4	Winter	Gradually increasing flow of Q_{max}=680 m^3/s	70	113	1.3	1.27	2.5	Overtop-ping	333.16	1
W5	Winter	Gradually increasing flow of Q_{max}=680 m^3/s	70	113	1.3	1.27	0 (Completely Closed)	Overtop-ping	333.16	1
S4m	**Same as Scenario S4 with Manning Roughness Values Halved for Sensitivity Test**									

As seen in Table 4.1, the only difference between the “Summer” and “Winter” scenarios is the value of the lateral inflow from Emet Creek. The values for the lateral inflow from Emet Creek were obtained from the daily observations collected at DSİ’s observation station #03-04, where the recorded maximum flow and the annual average flow were 680 and 350 m^3/s (approximate), respectively.

In both Winter and Summer scenarios, a catastrophic inflow hydrograph of 5191.8 m^3/s peak discharge and 280 hours of duration was used as the inflow hydrograph at the upstream end (i.e. Çınarcık Dam) of the modeling reach. Given that the total duration of the hydrograph is impractically long, the modeling was performed only for a duration of 100 hours, which also covered the peak magnitude of 5191.8 m^3/s occurring at 88 hours.

As for the piping scenarios, only two scenarios, adopted from overtopping scenarios S4 and W4 in Table 4.1, were analyzed, the details of which are tabulated in Table 4.2.

Table 4-2 Scenario Definitions for Piping Modeling

SCENARIO ID	Season	Lateral Flow from Emet Creek (m^3/s)	Terminal Breach Bottom Width (m)	Terminal Breach Height (m)	Breach Formation Time (hours)	Side Slope of Breach (1V : zH)	Spillway Gates Opening Width (d) (m)	Type of Failure	Failure Criteria (m)	Breach Formation Parameter (ρ_o)
S4P	Summer	Gradually increasing flow of Q_{max}=350 m^3/s	214	63	2.05	0	2.5	Piping	333	2
W4P	Winter	Gradually increasing flow of Q_{max}=680 m^3/s	214	63	2.05	0	2.5	Piping	333	2

The reasons for selection of breach parameters such as width, height, side slope of the breach, breach formation parameter and the breach formation time will be given in detail under **Section 4.5.2.2 Breach Parameters**.

4.5 JUSTIFICATION OF PARAMETERS SET DURING MODELING

4.5.1 JUSTIFICATION OF EXTERNAL DATA

4.5.1.1 The Catastrophic Inflow Hydrograph (Storm/Runoff Flood) to the dam at upstream boundary:

The catastrophic inflow hydrograph for Çınarcık Dam provided by DSİ is given below in Figure 4.2.

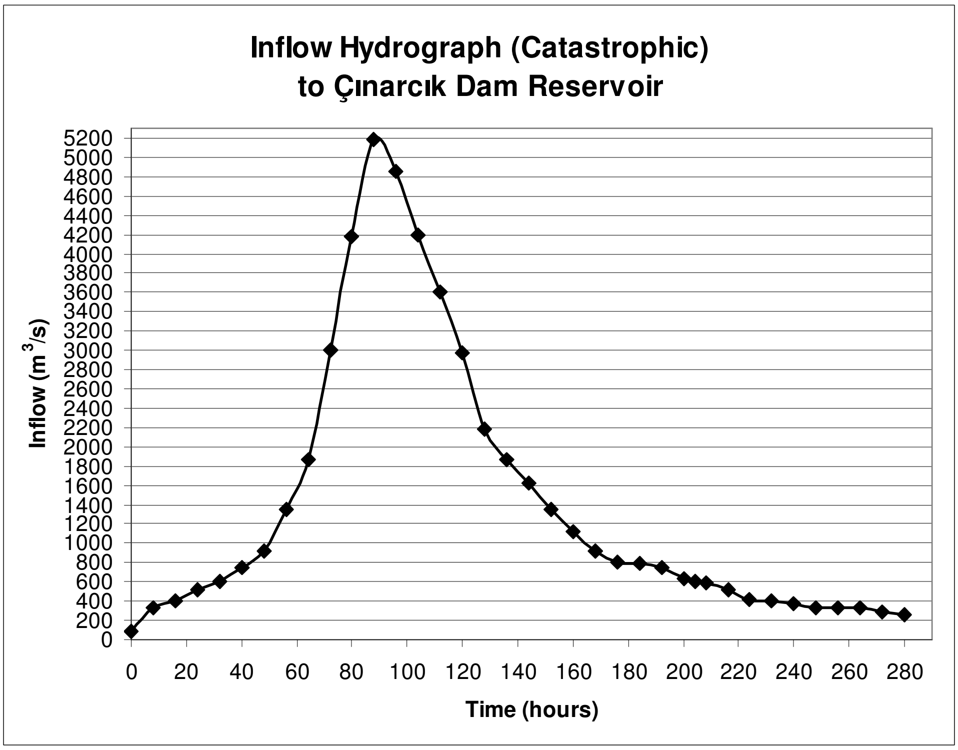

Figure 4-2 Inflow Hydrograph (Catastrophic) for Çınarcık Dam

The tabulated values of the catastrophic inflow hydrographs were measured on the hydrograph and are given below in Table 4.3:

Table 4-3 Data as to the Inflow Hydrograph (Catastrophic)

Discharge (m^3/s)	Time (hours)		Discharge (m^3/s)	Time (hours)		Discharge (m^3/s)	Time (hours)
85	0		4200	104		600	204
330	8		3600	112		590	208
400	16		2975	120		515	216
515	24		2190	128		415	224
600	32		1870	136		400	232
750	40		1625	144		375	240
920	48		1350	152		330	248
1350	56		1120	160		330	256
1870	64		920	168		330	264
3000	72		805	176		290	272
4180	80		790	184		260	280
5191,8	88		750	192			
4850	96		635	200			

The time step of the hydrograph is therefore constant as 8 hours. During modeling, the total duration of simulation ("*TEH*") was specified as 100 hours given that the total duration of hydrograph, which is 280 hours, is unrealistically long, as previously discussed under **Section 5.4 Modeling Scenarios** herein.

Among the data provided by DSİ are a set of hydrographs with varying return periods. Of these, the ones with 100-years and 50-years return periods have peak flows of 1250 m^3/s and 1100 m^3/s in magnitude, respectively. The return period of the catastrophic inflow hydrograph presented above In Figure 5.3 is not available on the official documents however, based on the foregoing, it can be deduced that the catastrophic inflow hydrograph of 5191.8 m^3/s peak flow magnitude must have a much longer return period, most probably more than 500 years.

It is noteworthy to mention here that DSİ has designed the spillway to accommodate this catastrophic inflow hydrograph, which implies that even in case of an inflow of such great magnitude to the reservoir, the spillway can release the inflow, thereby preventing the rise of water level behind the reservoir. Although this could help to prevent the water behind the reservoir from overtopping the dam crest and hence failure of the dam itself, the water released through the spillway could still present a risk to the settlements downstream of the dam given the magnitude of the discharge. Furthermore, in case of failure of any of the spillway gates, the capacity of the spillway would be decreased significantly, making the occurrence of overtopping of dam crest and hence failure of the dam itself unavoidable. The outputs of the simulations run under FLDWAV model will be given and discussed in detail under **Section 5 Outputs of Modeling**.

4.5.1.2 Geometrical and Technical Data of Çınarcık Dam, which is located at the upstream end of the modeling reach

The technical specifications provided by DSİ as to Çınarcık Dam and Uluabat HEPP are given below in Table 4.4. The plan view of the modeling domain and the plan and sectional views of the Çınarcık Dam were previously given in Figures 3.1, 3.2 and 3.3, respectively:

Table 4-4 Technical Specifications of Çınarcık Dam

Çınarcık Dam	
1. Dam Body	
Type	Rock-Fill
Height from Talveg	123.00 m.
Height from Foundation	125.00 m.
Crest Elevation	333.00 m.
Talveg Elevation	210.00 m.
Crest Length (exclusive of spillway length)	325.00 m.
Crest Width	12.00 m.
2. Reservoir	
Max. Water Elevation	330.00 m.
Max. Operating Water Elevation	330.00 m.
Min. Operating Water Elevation	304.75 m.

Table 4-4 Technical Specifications of Çınarcık Dam (Cont'd.)

Çınarcık Dam	
3. Spillway	
Type	Front Inlet, Radial Gated
Capacity	5191.80 m^3/s
Gate Dimensions	10.00 m x 14.00 m (5 gates)
Spillway Crest Elevation	316.00 m.
Total Spillway Crest Length	60.00 m.
4. Bottom Outlet	
Purpose	Irrigation Water Supply, Discharge of Reservoir at emergencies
Capacity at max. water elevation	71.78 m^3/s
5. Uluabat Power Tunnel	
Tunnel Capacity	48.89 m^3/s
6. Uluabat HEPP	
Turbine Design Discharge	48.90 m^3/s

Figure 4.3 given below illustrates discharge components of an ordinary dam:

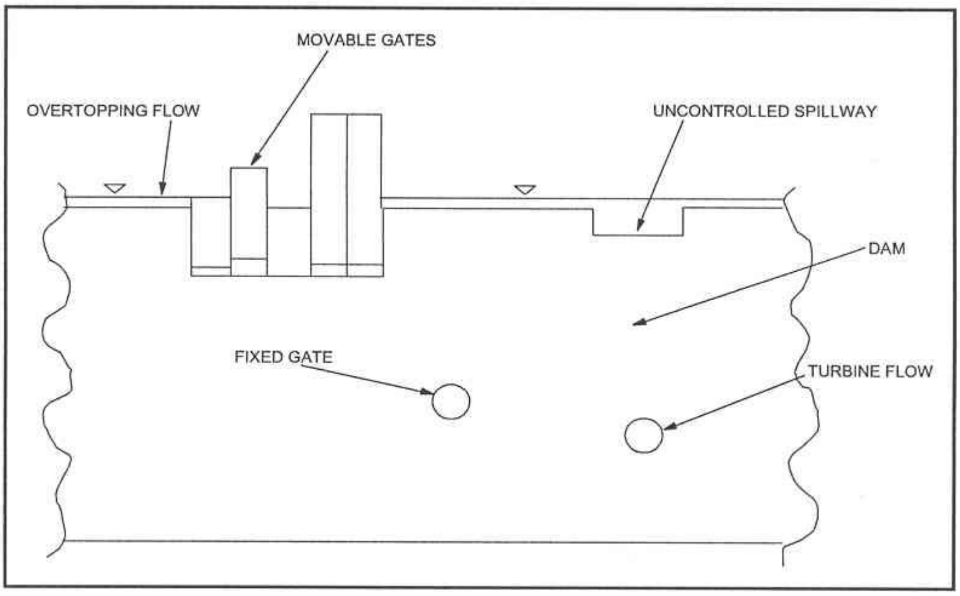

Figure 4-3 Discharge Components of an Ordinary Dam (Fread et al., 1998)

4.5.1.3 Cross-sectional data for the reach (i.e. Orhaneli and Mustafakemalpaşa Creeks in the modeling domain)

Unfortunately, there is not any systematic measurement of the cross-sectional data for the river reaches in Turkey. Interviews with DSİ indicated that the cross-sections upstream of the dams were of major concern during design stage. There are also several observation stations set up on several locations along various rivers, either by DSİ or other state institutions, some of which conduct daily measurements of flow values. As far as cross-sectional data is concerned, these stations are able to provide the cross-section measurements at their location only. Due to all these, it was not at first possible to find any cross-sectional data pertaining to the area in the vicinity of Çınarcık Dam. Fortunately, the Government of Turkey has initiated a new project by the name TEFER (TURKEY EMERGENCY FLOOD & EARTHQUAKE RECOVERY), which is now being carried out by DSİ in some geographical regions of Turkey including the Marmara Region where Çınarcık Dam is located. DSİ provided the author with cross-section measurements for Emet, Mustafakemalpaşa and Orhaneli Creeks conducted within the scope of TEFER project. A plan view of the cross-section stations and the creeks are given in Figure 4.4.

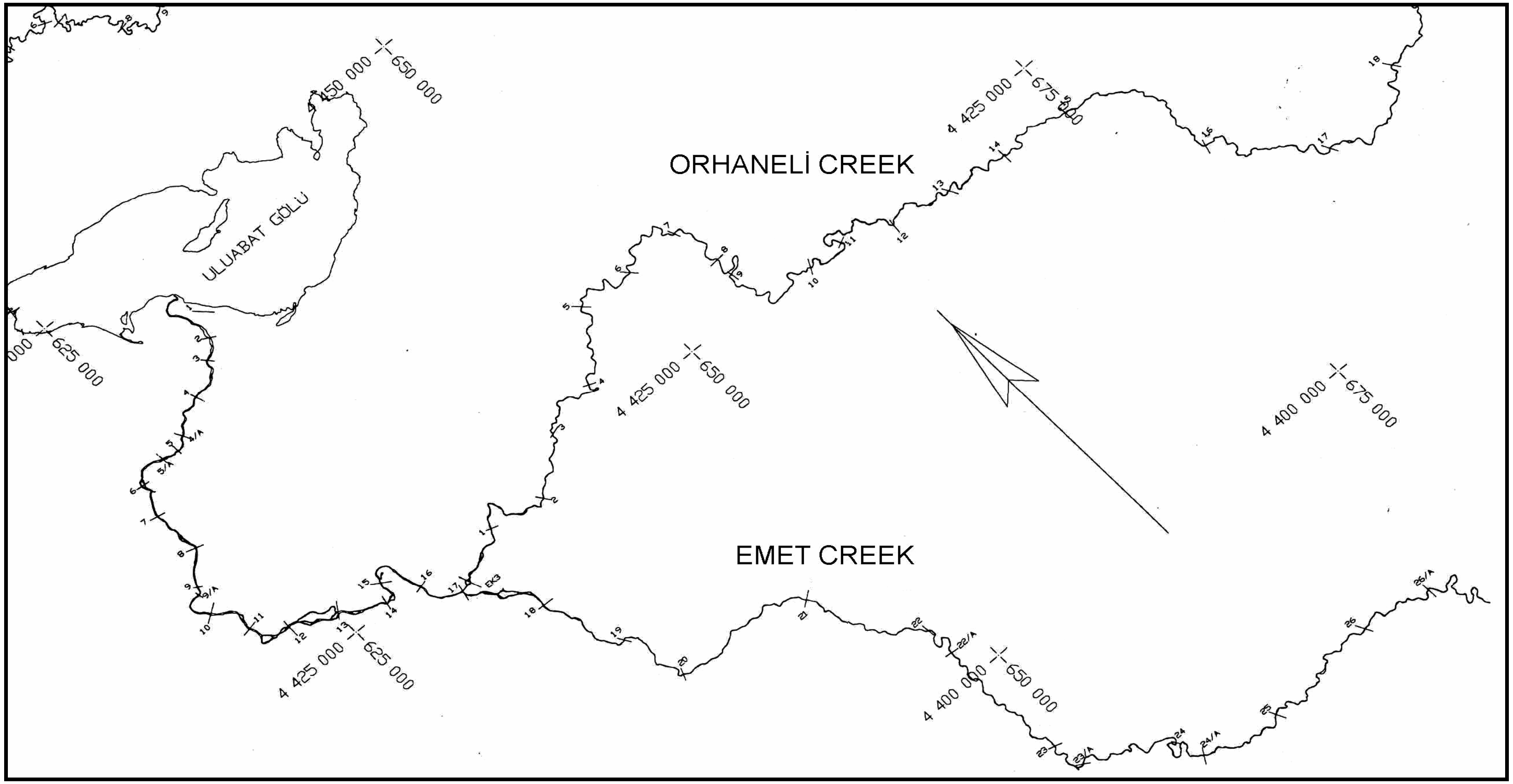

Figure 4-4 Plan View of the Modeling Reach and the Cross-section Stations, Net-CAD output provided by DSİ. (Not to Scale)

The data includes 26 cross-section stations on the subject modeling domain. An example view of one of the cross-sections provided is given in Figure 4.5.

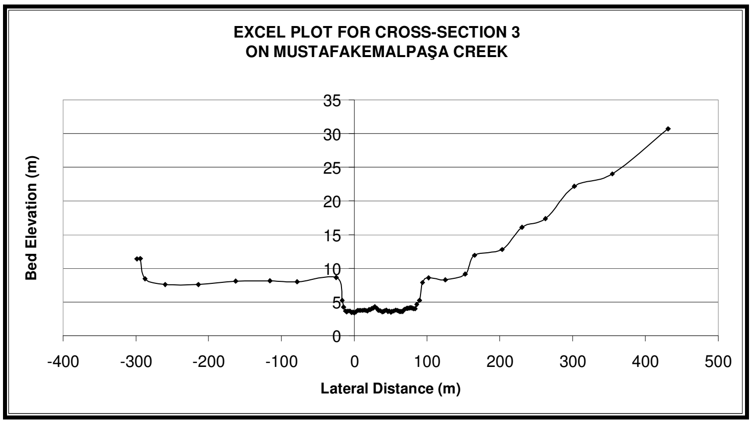

Figure 4-5 An example view of one of the cross-section data provided by DSİ

In the FLDWAV Model, the cross-sections are defined into the model as tables of top-width versus top-width elevations. That is to say, each cross-section is divided into a number of top-widths by elevation and is defined into the model. The number of top-widths, hence the number of corresponding elevations for a given cross-section in FLDWAV can go up to 8 for each cross-section (Fread et al., 1998). The "*top-width versus elevation*" concept is illustrated in Figures 4.6 and 4.7.

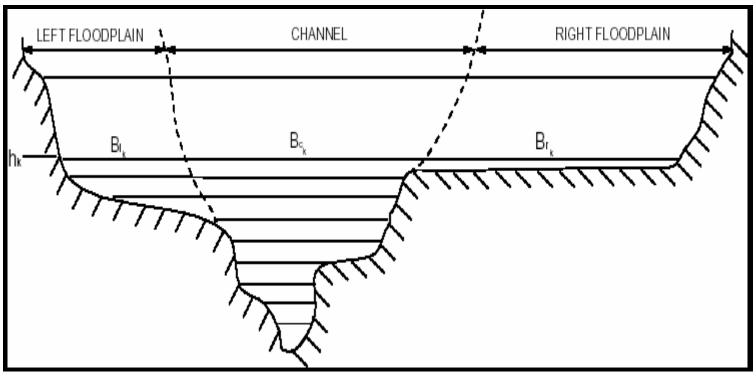

Figure 4-6 Illustration as to Main Channel and Floodplains (Fread et al., 1998)

In Figure 4.6, B_{r_k} , B_{c_k} , and B_{l_k} stand for the top-widths at k^th elevation for right floodplain, main channel and left floodplain, respectively (Fread et al., 1998).

During the modeling, 7 top-width versus top-width elevation values were used to define each cross-section. This is enough to adequately define the cross-sections, as also acknowledged by Fread et al. (1998): "*…Generally about 4 to 12 sets of topwidths and associated elevations provide a sufficiently accurate description of the cross section….*"

In Figure 4.7, B_i stands for active storage area and BO_i stands for inactive storage area and the inactive storage area represents the portions of the cross-sections where flow in x-direction is negligible such as at confluence points etc (Fread et al., 1998).

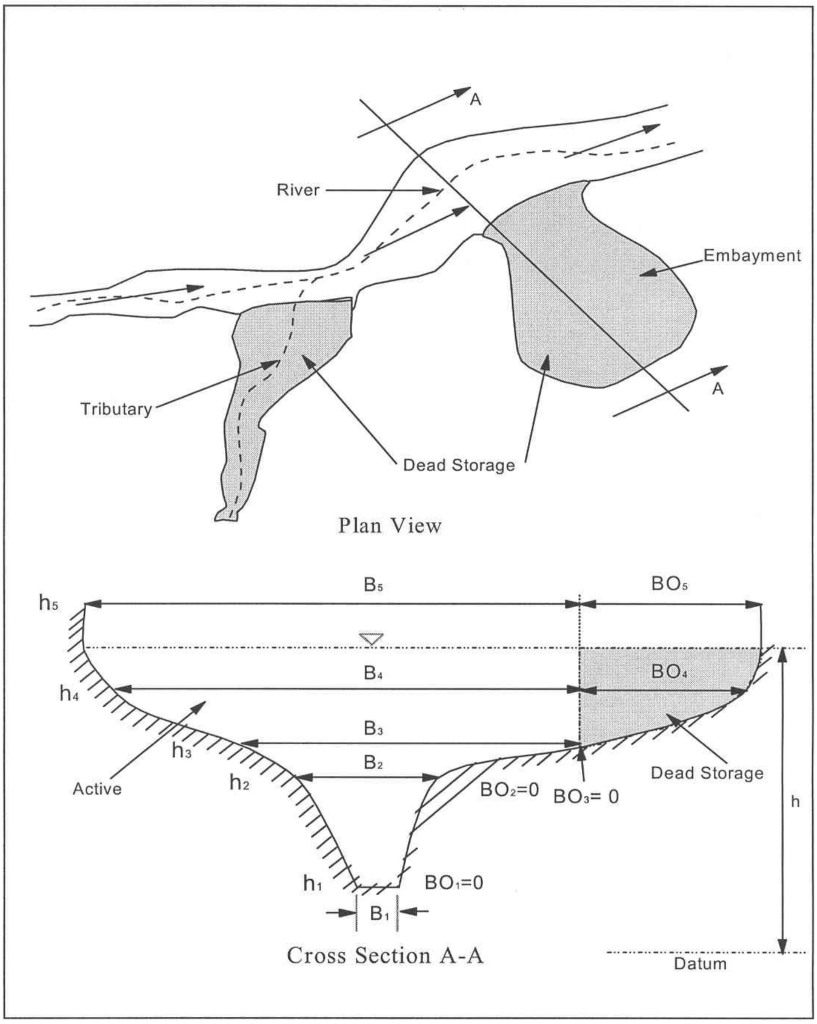

Figure 4-7 Illustration of Active and Dead Storage Concepts (Fread et al., 1998)

To simplify the case study, the cross-sections were assumed not to have any in-active storage areas (i.e. BO_i=0 for all cross-sections).

4.5.1.4 Station Kms of Cross-sections

The station kms of the cross-sections 1-6 on Orhaneli Creek and 1-17 on Emet Creek in Figure 4.8 were identified by measuring the sub-reaches on the same figure (as per the original scale of 1/250000), where the dam is located 1 km upstream of cross-section #6.

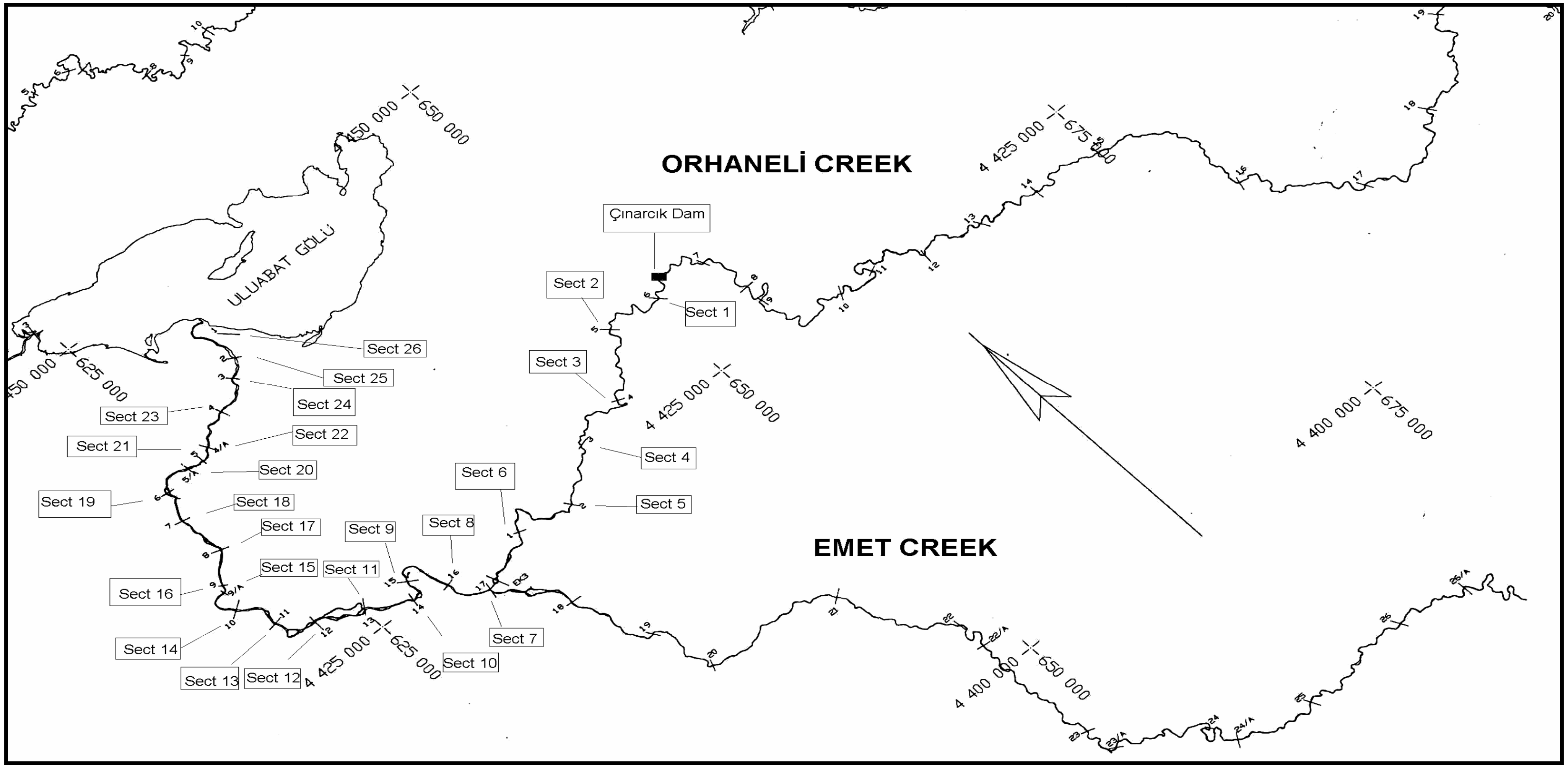

Figure 4-8 Plan View of the Modeling Reach and the Cross-section Stations, Net-CAD output provided by DSİ. (*Figures in Boxes indicate cross-section ID # as per Table 4.5) (Not to Scale)*

The calculated lengths of sub-reaches are given in Table 4.5 below:

Table 4-5 Data as to Cross-sections for the Modeling Reach

x-section ID	**Station Km.**		**x-section ID**	**Station Km.**		**x-section ID**	**Station Km.**
D1	0.00		9	34.63		19	57.00
D2	0.10		10	36.38		20	59.00
1	1.00		11	39.25		21	59.93
2	5.00		12	42.00		22	60.80
3	11.00		13	44.50		23	63.55
4	15.63		14	47.00		24	66.30
5	20.50		15	49.25		**25**	**67.55**
6	24.13		16	49.75		26	71.05
7	28.63		17	52.25			
8	31.38		**18**	**55.00**			

The cross-section ID numbers in boxes in Figure 4.8 were added later on by the Author to match those in Table 4.5.

It is necessary to note that cross-sections #D1 and #D2 were additionally specified as input to FLDWAV, upstream of cross-section #1 in Table 4.5, at km stations 0.00 and 0.10 km. These two cross-sections are Çınarcık Dam's body sections, and the Dam is located within the small sub-reach of 0.10 km formed by these two station kms.

The cross-sections in **bold** in Table 4.5 were not specified as input to FLDWAV because the invert bed elevation at those cross-sections were lower than the subsequent down-stream cross-section and therefore were leading to problems during computations.

Furthermore, the cross-section data provided by DSİ (cross-sections 1 to 26 in Table 4.5) were spanning maximum top-widths in the range of 30 m. to 600 m. By making use of the topographic map of the reach provided by DSİ, the cross-section spans were extended to cover larger spans so as to achieve more realistic results. The improved cross-section data was then specified as input to the FLDWAV Model.

4.5.1.5 Spillway Rating Curve of Çınarcık Dam

The flow through the spillways were represented by a spillway rating-curve only therefore, the additional inputs regarding the uncontrolled spillway discharge coefficient and geometry and the gate controlled spillway discharge coefficients and geometry were not needed as input.

The spillway rating curve of Çınarcık Dam was obtained from DSİ. By measuring on the rating curve, the following tabular values (Table 4.6) were obtained for d=0, 2.5, 5, 7.5, 10 m., where d represents the width of the gate opening for each of the 5 gates.

Table 4-6 Data as to Spillway Rating Curve for varying spillway gate opening, d (m), values

Elevation of Water (m)	**Spillway Discharge (m^3/s)**				
	d=10.0 m. (gates completely open)	d=7.5 m.	d=5.0 m.	d=2.5 m	d=0.0 m.
316	0	0	0	0	0
318	250.00	250.00	250.00	250.00	0
320	773.00	772.73	772.73	545.50	0
322	1386.00	1370.00	1227.30	772.73	0
324	2114.00	2000.00	1636.40	909.10	0
326	2986.00	2500.00	1909.10	1045.50	0
328	4091.00	2950.00	2136.40	1227.30	0
330	5318.00	3272.73	2454.50	1272.73	0

The excel plot of the spillway rating curve for 5 different spillway gate opening widths, (d), is given in Figure 4.9, plotted as per the data given in Table 4.6.

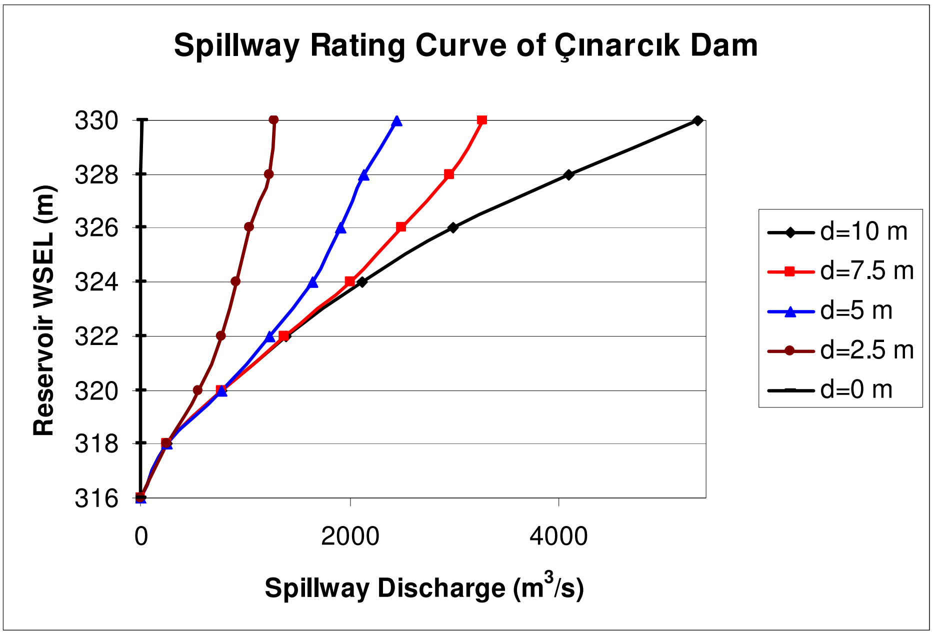

Figure 4-9 Spillway Rating Curve of Çınarcık Dam

4.5.1.6 Discharge through the turbines

The discharge capacity of the turbine is indicated as 48.90 m^3/s by DSİ. This value would be specified as the turbine flow into the FLDWAV Model by parameter "*QTD*", which represents the turbine discharge in FLDWAV Model. However, as previously mentioned under **Section 4.1.1 Initial Conditions**, the Model uses the following equation (Eqn. 4.15, Fread et al., 1998) to calculate the initial discharge values, which serve as a basis to calculate the initial water surface elevations:

$$Q_i = Q_{i-1} + q_{i-1}\Delta x_{i-1} \tag{4.15}$$

To ensure that the model uses the realistic discharge values to compute the initial water surface elevations along the reach, the flow released through the bottom outlet was superposed with the turbine discharge. It is necessary to mention that the turbine flow the case of Çınarcık Dam is not released to downstream of dam but at the Uluabat HEPP through a power tunnel of some 12000 m as indicated previously in Figure 3.1. Unfortunately, there were not any parameters in FLDWAV model to specify this distinction therefore the turbine discharge and the discharge through the bottom outlet (Figure 4.10) had to be superposed.

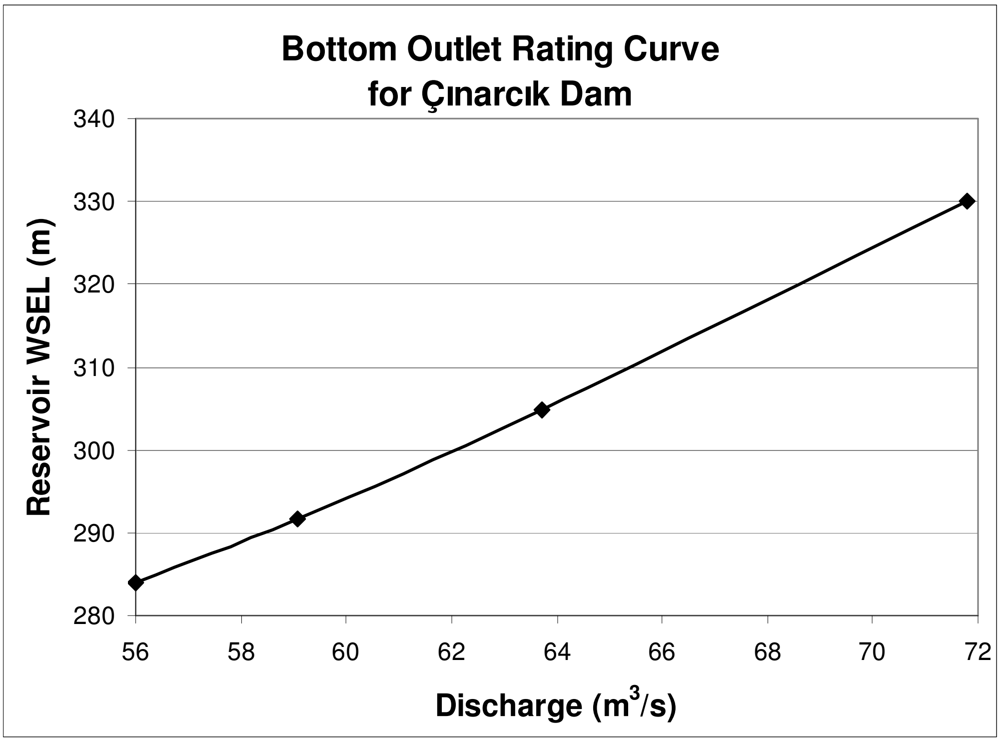

Figure 4-10 Rating Curve for the Bottom Outlet of Çınarcık Dam

The tabulated discharge values for the bottom outlet are given in Table 4.7.

Table 4-7 Discharge Values for Bottom Outlet

WSEL (m)	Discharge through Bottom Outlet (m^3/s)		WSEL (m)	Discharge through Bottom Outlet (m^3/s)
284	56.00		322	69.23
291.7	59.08		324	69.86
304.75	63.72		326	70.50
316	67.31		328	71.14
318	67.95		330	71.78
320	68.59			

The average bottom outlet discharge of 66.83 m^3/s was superposed with the turbine discharge of 48.90 m^3/s. The sum of the two was specified as turbine flow to the Model, as 116 m^3/s. Although the turbine flow is not released to downstream of the dam but to Ulubat HEPP towards the North, the turbine flow still contributes to depletion of the reservoir therefore, this approach was followed.

4.5.1.7 Reservoir Surface Area vs Water Surface Elevation (WSEL) table for Çınarcık Dam

The Surface Area versus WSEL and Volume versus WSEL chart for reservoir of Çınarcık dam was procured from DSİ. The values measured on the chart are tabulated in Table 4.8.

Table 4-8 Reservoir Surface Area and Volume Data for Çınarcık Dam

WSEL (m)	Surface Area (10^6 m^2)	VOLUME (10^6 m^3)
210	0.00	0.00
220	0.09	0.29
230	0.36	2.45
240	0.69	7.66
250	1.18	16.96
260	1.74	31.42
270	2.40	52.07
280	3.16	79.81
290	4.03	115.76
300	5.00	159.87
310	6.10	216.26
320	7.75	284.11
330	10.14	372.94

The excel plots of the Surface Area versus WSEL and Volume versus WSEL for reservoir of Çınarcık Dam are given in Figures 4.11 and 4.12, respectively, as per the data given in Table 4.8.

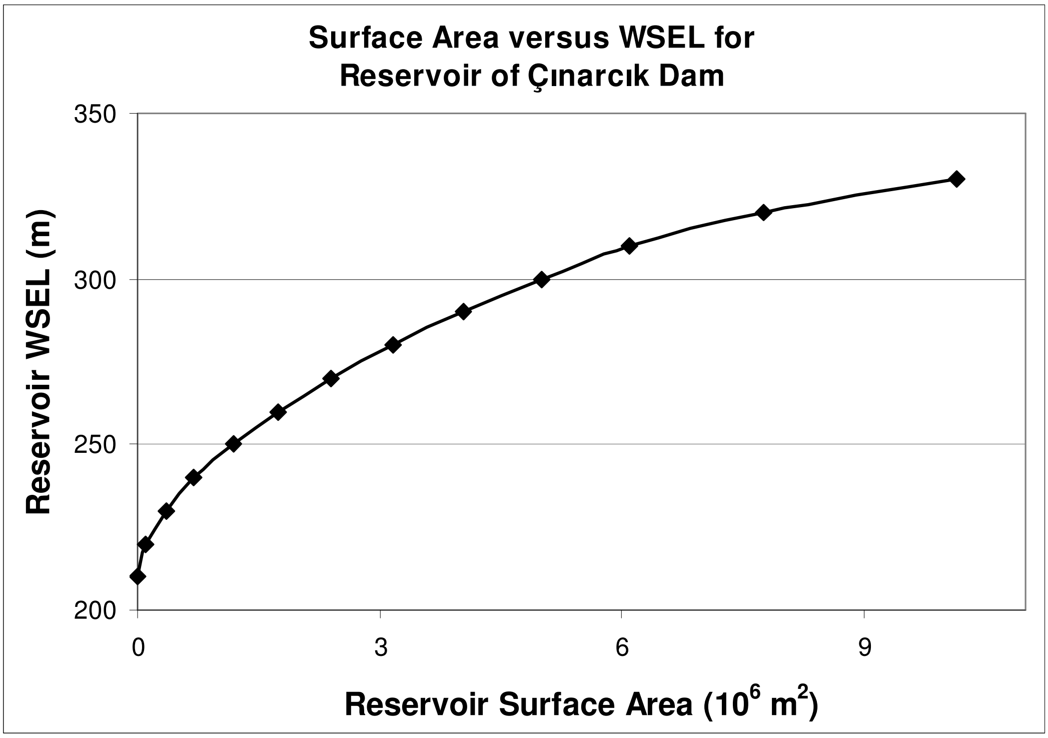

Figure 4-11 Surface Area versus WSEL curve for reservoir of Çınarcık Dam

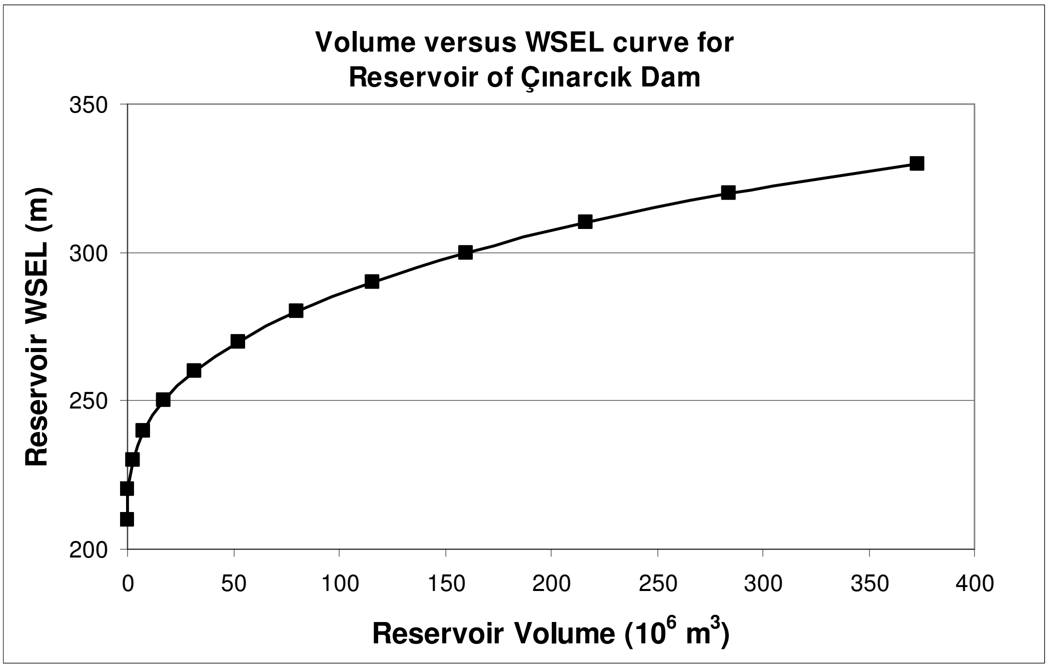

Figure 4-12 Volume versus WSEL curve for reservoir of Çınarcık Dam

4.5.1.8 Topographic maps of the modeling domain of 1/100000 scale.

Topographic maps of 1/100000 scale of the modeling domain were obtained from DSİ. As previously mentioned under **Section 4.3.1 External Data**, it was not possible to obtain maps of larger scale (and finer detail) because of the national security measures put in place by "*Harita Genel Komutanlığı (General Command of Mapping)*" of Turkey.

These maps were used to prepare inundation maps following the completion of modeling. The maps indicating the inundated areas will be given under **Section 5 Outputs of Modeling**.

4.5.1.9 Monthly observed flow values for the Emet Creek, which was specified as a lateral flow to the Orhaneli Creek.

DSİ has a station designated as Station #03-04 just upstream of the confluence point of Emet Creek with Orhaneli Creek to *inter alia* conduct periodic measurement of the flow on Emet Creek. The measurement data provided by DSİ at Station #03-04 spans a period from 1960 to 2000. The maximum observed flow on Emet Creek was 680 m^3/s in 1987, which was specified as the maximum constant value of the gradually increasing lateral inflow to sub-reach #8 bounded by cross-sections ID#6 and #7 in Table 4.5 at the upstream and the downstream, respectively, in the Winter Case Scenarios; and as 350 m^3/s in the Summer Case Scenarios; as previously mentioned under **Section 4.4 Modeling Scenarios**.

4.5.1.10 Manning Roughness

As previously mentioned under **Section 4.3.1 External Data**, the Author was unable to access any official Manning roughness values for the Orhaneli and Mustafakemalpaşa Creeks, therefore had to use arbitrarily but judicially chosen Manning roughness values based on Chow (1959).

Chow (1959) presents a table showing indicatory ranges of Manning roughness values for several conditions. Two of those basic categories, which are of particular interest as far as this thesis work is considered, are the categories titled as "Minor Streams" and "Major Streams. Chow (1959) defines "Major Streams" as streams with surface width at flood stage more than 100 ft (i.e. ~30.48 m.). In Çınarcık Dam's case study, the smallest top-width at floodstage is 22.38 m. occurring at Section ID #5 in Table 4.5. There are a few more cross-sections where top-width at floodstage is within the range 24-30 m. Nevertheless, the cross-sections along the entire reach were assumed to be larger than 30.48 m and the stream was evaluated as "Major Stream" while determining the Manning roughness values. Given the humid and fertile characteristic of Marmara Region of Turkey, where Çınarcık Dam is located, it was assumed that there would be some vegetation on the banks and the floodplains. Based on these assumptions, the Manning roughness value ranges suggested by Chow (1959) for main channels and the floodplains in case of "<u>Minor Streams</u>" were identified as follows:

<u>Minor streams (surface width at flood stage less than 100 feet):</u>

- Some weeds, light brush on banks: ....0.035 - 0.050

<u>Flood plains (adjacent to natural streams):</u>

- Light brush and trees:
  a. Winter..0.050 - 0.060
  b. Summer..0.060 - 0.080

Chow (1959), however, suggests that roughness coefficient for major streams (i.e. surface width at flood stage more than 100 feet) is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks and adds that values of n may be somewhat reduced. In light of all these, Manning roughness values falling within the ranges given above were selected for the main channel and the floodplains.

Furthermore, in FLDWAV Model, the user can divide the reach into sub-reaches in terms of Manning roughness values. In the case study of Çınarcık Dam, the total reach was divided into just one "*Manning Reach*". The Manning roughness values corresponding to each cross-section top-width elevation has to be specified to the FLDWAV Model. The assumed Manning roughness values for the "*Manning Reach*" spanning the entire modeling domain are given in Table 4.9 for the 7 top-width elevations:

Table 4-9 Manning Roughness Values Specified for Each of 7 Top-width Elevation Values for any Cross-section

Manning Roughness, n	Manning Roughness Corresponding to Top-width Elevation						
	1	2	3	4	5	6	7
Main Channel	0.04	0.04	0.04	0.04	0.04	0.04	0.04
Left Floodplain	0.06	0.06	0.06	0.06	0.06	0.06	0.06
Right Floodplain	0.06	0.06	0.06	0.06	0.06	0.06	0.06

4.5.2 JUSTIFICATION OF INTERNAL DATA

The justifications for the internal data listed earlier under **Section 4.3.2 Internal Data** are presented individually under separate headings, as given below:

4.5.2.1 Initial Conditions

As previously discussed under **Section 4.1 An Overview of FLDWAV Software**, the FLDWAV Model uses the equations associated with upstream and downstream boundary conditions and the iterative Newton-Raphson technique to calculate the initial WSELs along the reach, unless they have been user-specified to the Model.

Only the initial surface elevation of the water behind the reservoir (which is the upstream-most cross-section) was user-specified as 315 m. during this case study. This value was arbitrarily chosen between the possible reservoir WSEL range from 310 m. to 330 m., which were the minimum and maximum operating levels, respectively, as specified by DSİ. All the remaining water surface elevations were then calculated by the Model as per the process delineated under **Section 4.1 An Overview of FLDWAV Software**.

4.5.2.2 Breach Parameters

These are the parameters pertaining to the breach such as the width, height, and geometry of the breach and the time of failure. There have been many studies as to the how to define the breach parameters in dam failure analyses by Fread et al. (1998) and Wahl (1998 and 2001).

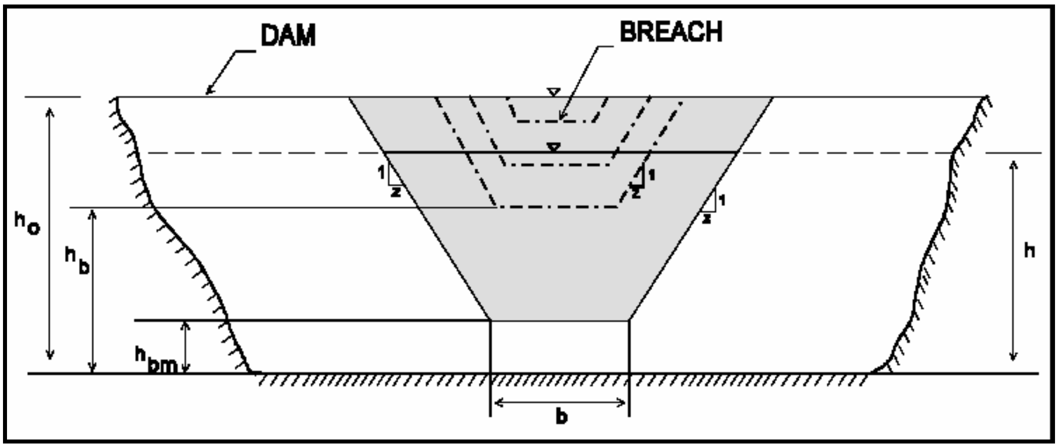

Figure 4-13 Definition of Breach as per the FLDWAV Model (Fread et al., 1998)

As seen in Figure 4.13 (Fread et al., 1998), the model assumes that the breach bottom width starts at a point at the dam and enlarges downward at a linear or nonlinear rate over the failure time (τ) until the terminal bottom width (b) is attained and the breach bottom has eroded to the terminal elevation h_{bm}. If (τ) is less than one minute, the width of the breach bottom starts at a value of b rather than zero; this represents more of a sudden collapse failure than an erosion failure. The bottom elevation of the breach (h_b) is simulated as a function of time (τ). (Fread et al, 1998).

Regarding overtopping dam failures, Fread et al (1998) comment that the time elapsed from initiation of breach formation on the upstream face of the dam till full-formation of the breach 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 used in construction, the extent of compaction of the materials, and the magnitude and duration of the overtopping flow of the escaping water, and add that the time of failure is usually considerably longer for piping failures than an overtopping failure since the upstream face is slowly being eroded in the very early phase of the piping development.

Fread et al (1998) suggest equations derived by Froehlich in 1987 and 1995 to estimate the breach parameters however, there are other studies conducted to date, which mainly focus on statistical analyses of the previously observed breach dimensions so as to identify a correlation between various parameters of the dam and the breach characteristics. There are already several equations derived to express the breach parameters in terms of dam's various properties. Some of these studies are given on the next pages.

4.5.2.2.1 Tony L. Wahl (2001)

Wahl (2001) investigated the accuracy of a number of existing equations by comparing the equations with a collection of observed breach values in 108 dam failures, which he compiled in 1998. The equations he investigated in his work can be classified as:

- **Breach Width Prediction**
    - Bureau of Reclamation (1982, cited in Wahl (2001))
    - MacDonald and Langridge-Monopolis (1984, cited in Wahl (2001))
    - Von Thun and Gilette (1990, cited in Wahl (2001))
    - Froehlich (1995a, cited in Wahl (2001))
- **Failure Time Equations**
    - Bureau of Reclamation (1982, cited in Wahl (2001))
    - MacDonald and Langridge-Monopolis (1984, cited in Wahl (2001))
    - Von Thun and Gilette (1990, cited in Wahl (2001))
    - Froehlich (1995a, cited in Wahl (2001))
- **Peak Flow Equations**
    - 14 equations, skipped since outside the scope of interest of this thesis work.

The following Figure 4.14 from his work (Wahl, 2001) shows the plot of predicted breach width values versus observed breach width values for 3 of these equations, namely, Von Thun & Gillette (1990, cited in Wahl (2001)); Froehlich (1995, cited in Wahl (2001)); and Reclamation (1998, cited in Wahl (2001)).

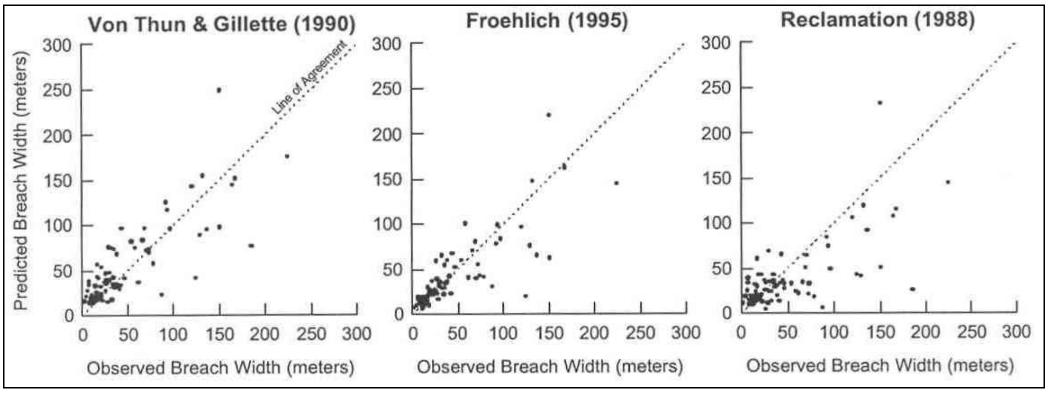

Figure 4-14 Predicted Breach Width versus Observed Breach Width (Wahl, 2001)

However, he comments that all these methods listed above were based on regression analyses of data collected from actual dam failures but the database of dam failures used to develop those relations was relatively lacking in data from failures of large dams, with about 75% of the cases having a height less than 15 m. (Wahl, 2001).

Wahl (2001) concludes that:

- The four methods for predicting <u>breach width</u> (or volume of material eroded, from which breach width can be estimated) all had absolute mean prediction errors less than one-tenth of an order of magnitude, indicating that on average their predictions are on-target.

- The five methods for predicting <u>failure time</u> all under-predict the failure time on average, by amounts ranging from about one-fifth to two-thirds of an order of magnitude. This is consistent with the previous observation that these equations are designed to conservatively predict fast breaches, which will cause large peak outflows. The uncertainty bands on all of the failure time equations are very large, ranging from about ±0.6 to ±1 order of magnitude, with the <u>Froehlich (1995b, cited in Wahl (2001)) equation having the smallest uncertainty</u>.

It appears from Table 4.10 by Wahl (2001) that the most reliable, but not necessarily accurate, of the equations to predict the Breach Width and the failure time are the equations developed by Von Thun and Gillette (1990, cited in Wahl (2001)) and Froehlich (1995b, cited in Wahl (2001)), respectively, which have the smallest prediction intervals around a hypothetical predicted value of 1.0 of 0.37 –1.8 and 0.38 – 7.3, respectively.

Table 4-10 Uncertainty estimates of breach parameter and failure time prediction equations (Wahl, 2001)

Equation	Number of Case Studies: Before outlier exclusion	Number of Case Studies: After outlier exclusion	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
BREACH WIDTH EQUATIONS					
USBR (1988) $\overline{B} = 3(h_w)$	80	70	-0.09	±0.43	0.45 — 3.3
MacDonald and Langridge-Monopolis (1984) $V_{er} = 0.0261(V_w \cdot h_w)^{0.769}$ *earthfill* $V_{er} = 0.00348(V_w \cdot h_w)^{0.852}$ *non-earthfill (e.g., rockfill)*	60	58	-0.01	±0.82	0.15 — 6.8
Von Thun and Gillette (1990) $\overline{B} = 2.5h_w + C_b$ *where C_b is a function of reservoir size*	78	70	+0.09	±0.35	0.37 — 1.8
Froehlich (1995b) $\overline{B} = 0.1803K_o V_w^{0.32} h_b^{0.19}$ *where K_o = 1.4 for overtopping, 1.0 for piping*	77	75	+0.01	±0.39	0.40 — 2.4
FAILURE TIME EQUATIONS					
MacDonald and Langridge-Monopolis (1984) $t_f = 0.0179(V_{er})^{0.364}$	37	35	-0.21	±0.83	0.24 — 11.
Von Thun and Gillette (1990) $t_f = 0.015(h_w)$ *highly erodible* $t_f = 0.020(h_w) + 0.25$ *erosion resistant*	36	34	-0.64	±0.95	0.49 — 40.
Von Thun and Gillette (1990) $t_f = \overline{B}/(4h_w)$ *highly erodible* $t_f = \overline{B}/(4h_w + 61)$ *erosion resistant*	36	35	-0.38	±0.84	0.35 — 17.
Froehlich (1995b) $t_f = 0.00254(V_w)^{0.53} h_b^{-0.9}$	34	33	-0.22	±0.64	0.38 — 7.3
USBR (1988) $t_f = 0.011(\overline{B})$	40	39	-0.40	±1.02	0.24 — 27.

Wahl (2001) compares the results of the above given equations with the results of a case study conducted by Bureau of Reclamation for North Dakota Dam in January 2001. Based on the results of the comparison and analyses, Wahl (2001) concludes that the recommended values to predict failure times would generally be a compromise between the results obtained from the MacDonald and Langridge-Monopolis and Froehlich relation and concludes that predictions of breach width generally have an uncertainty of about ±1/3 order of magnitude, predictions of failure time have uncertainties approaching ±1 order of magnitude.

4.5.2.2.2 FLDWAV Manual (Fread et al, 1998)

Fread et al (1998), on the other hand, suggests the average breach bottom width $\overline{B}$ in the range of ($0.5h_d \le \overline{B} \le 8h_d$), where h_d is the height of the dam.

The height and crest length of Çınarcık Dam are 123 m and 387 m (inclusive of the spillway length), respectively. Hence, the upper and lower limits of this range yield maximum and minimum average breach bottom widths ($\overline{B}$) of 61.5 m and 984 m, respectively. However, it should be observed that these values should not yield unrealistic results, i.e., the resultant breach width should be no more and most likely less than the maximum crest length of 387 m. provided that the local material around the dam withstands the force of escaping water and is not eroded, confining the maximum limits of the breach geometry to the dam geometry. Under this assumption, a breach bottom width of 387 m would not be possible given that the length of the dam body at lower elevations is less than 387 m, as can be deduced from previously given Figure 3.3.

It is also necessary to check whether or not the breach bottom width fits into the actual boundaries of the dam, i.e., the breach bottom width at the terminal breach elevation does not extend outside the dam body. The breach width parameter is based on the breach height therefore; the breach height (hence, the terminal breach bottom elevation) has to be arbitrarily but judicially chosen so that the maximum breach bottom width does not extend beyond the actual boundaries of the dam cross-section.

4.5.2.2.3 Selected Breach Parameters

<u>FOR OVERTOPPING FAILURE CASE</u>

It is clear that the mechanism of breach formation and the problem of prediction of breach parameters need further studies to understand completely, and the uncertainty associated with these parameters will surely be inherited to any analyses and studies based on them. Nevertheless, in order to be able to obtain relatively more accurate outputs, the following equations (Eqns. 4.16 and 4.18, respectively) were used to predict the Breach Width and Breach Failure Time, based on Wahl's (2004) work:

Breach Width (Von Thun Gillette, 1990, cited in Wahl (2001))

$$\overline{B} = 2.5h_w + C_b \tag{4.16}$$

where $\overline{B}$ is the average breach width, h_w is the hydraulic depth of water above breach invert at time of failure, and C_b is the offset factor, which is defined by Table 4.11 as:

Table 4-11 Range of Values for C_b (Wahl, 2001)

Reservoir Size, m^3	**C_b, meters**
$< 1.23 * 10^6$	6.1
$1.23 * 10^6 – 6.17 * 10^6$	18.3
$6.17 * 10^6 – 1.23 * 10^7$	42.7
$> 1.23 * 10^7$	54.9

Çınarcık Dam has a reservoir size of $372.94*10^6$ m^3, which is greater than $1.23*10^7$ m^3 therefore C_b for the Çınarcık Dam's case is 54.9 m.

For a h_w of 113.16 m (i.e. dam is assumed to fail at a water elevation of 333.16 m and the terminal breach bottom elevation is 220 m) and C_b of 54.9 m, Eqn. (4.16) yields $\overline{B}$ as 337.8 m. Given that the invert elevation of the breach bottom is 220 m and the crest elevation of the dam is 333 m, the average breach width of **337.8 m** computed by Eqn. (4.16) must be located just between the two, at 276.5 m elevation. Based on that, it is necessary to calculate the side slope of the breach so as to find out the breach widths at the breach invert elevation of 220 m (i.e. breach bottom width) and at the dam crest elevation of 333 m (i.e. breach top width). For this purpose, Eqn. (4.17) below, based on 43 observed dam failure cases developed by Froehlich in 1987 (Wahl, 1998) to predict the side slope of a given breach was used:

$$Z = 0.75K_c\,(h_w^{\,*})^{1.57}\,(\overline{W}^{\,*})^{0.73}\,, \qquad (4.17)$$

where Z is the side slope of the breach (Z horizontal:1 Vertical), K_c is the core wall correction factor (0.6 if dam contains a core wall; 1.0 otherwise), h_w^* is the dimensionless height of water above breach bottom (h_w/h_b), $\overline{W}^*$ is the dimensionless average embankment width ($[W_{crest}+W_{bottom}]/[2h_b]$); where W_{crest} and W_{bottom} are the embankment widths at the crest and bottom of the embankment, respectively and h_b is the height of breach.

Eqn. (4.17) yields a breach side slope of 1.27, which gives the breach widths at 220 m and 333 m elevations as 194.29 m and 481.31 m., as indicated in Table 4.12 below:

Table 4-12 Breach Side Slope, Z, computed by Eqn. (4.17)

K_c	h_b (m)	h_w (m)	h_w^* (h_w/h_b)	W_{crest} (m)	W_{bottom} (m)	$\overline{W}^*$ (m)	Z	b_{bottom} (m)	b_{top} (m)
1.0	113	113.16	1.001	387	80	233.5	1.27	194.29	481.31

As indicated in the last two columns in Table 4.12; based on the specified side slope of 1.27, breach height of 113 m, and the calculated average breach width of 337.8 m at 276.5 m elevation; breach widths at the breach bottom elevation of 220 m and the breach top elevation of 333 m. has to be 194.29 m and 481.31 m, respectively, from geometry. This, however, is impossible given that the width of the dam body at these elevations are about 90 m and 387 m, respectively. Hence, Eqn. (4.16) by Von Thun & Gillette may have over-predicted the average breach width or the Çınarcık Dam's geometrical characteristics may be significantly different than that of 57 dam failure cases Von Thun & Gillette used while deriving the said equation, resulting in a prediction error.

As already indicated in Table 4.10 earlier, Von Thun & Gillette's equation has a prediction interval of 0.37 – 1.8 around a hypothetical predicted value of 1.0 (Wahl, 2001). Therefore, assuming that the average breach width has been over-predicted by 1.8 over the actual value of 1.0, it can be deduced that the actual average width can at least be 215 m., which is narrower than the available dam body width of 233 m at that elevation. So as to preserve conservative nature of modeling, the average breach width, therefore, was specified as 233 m., which in turn gives breach bottom widths and breach top widths of 69.49 m and 356.51 m, respectively, when the side slope of the breach (Z) is taken as 1.27 similar to the previous calculations. Thus, the selected breach bottom and top widths are 70 m and 356 m, respectively, where the breach side slope is 1.27.

As for the failure time, the following Eqn. (4.18), derived by Froehlich in 1995 (Wahl, 1998), was used:

Failure Time (Froehlich, 1995, cited in Wahl, 1998)

$$t_f = 0.00254(V_w)^{0.53} h_b^{-0.9} \qquad (4.18)$$

where t_f is the failure time in hours, V_w is the volume of water in m^3 above breach invert elevation at the time of failure, and h_b is the breach height in m. For Çınarcık Dam, the volume of water at elevations 320 m and 330 m are 284*10^6 m^3 and 372.94*10^6 m^3, respectively. Given that the dam failure is set to occur at 333.16 m, there is the need to find the volume of water behind the dam immediately before the water surface elevation reaches 333.16 m. By extrapolating the existing volume data at 320 m and 330 m elevations to 333.16 m, the volume of water above the breach invert elevation at the time of failure was found as 401.06*10^6 m^3. Using this V_w value and the h_b of 113 m yields the failure time as 1.3 hours.

FOR PIPING FAILURE CASE

Breach Width

As for the piping failure, Eqn. (4.19) by Froehlich (Wahl, 2001) given below was used to calculate the average breach width, $\overline{B}$,:

$$\overline{B} = 0.1803 K_o V_w h_b^{0.19} \qquad (4.19)$$

where K_o is the overtopping correction factor (1.4 for overtopping failures, 1.0 otherwise), h_b is the height of breach (m), and V_w is the volume of water above breach invert elevation at the time of breach (m^3/s).

In piping failure scenarios, the dam was assumed to fail at a critical WSEL of 333 m. Assuming that the terminal breach elevation (h_{bm}) is 220 m, similar to the overtopping failure case; using K_o, h_b and V_w values of 1.0, 113 m, and 399.64*10^6 m^3, respectively, Eqn. (4.19) yields average breach width for piping case as 250 m. The FLDWAV model assumes the side slope, Z, to be 0 during piping failure simulation hence the breach has to have a rectangular shape therefore, a rectangular breach of 113 m in height and 245 m in width does not seem to be possible given the geometry of the dam. Therefore, the breach height (h_b) was assumed to be 63 m, in which case V_w equals to the volume of water stored behind the reservoir between elevations 333 m and 270 m less that of between 270 m and 210 m, which is 347.57*10^6 m^3. Taking K_o as 1.0 and using the new h_b and V_w

values of 63 m and 347.57*10^6 m^3, respectively, Eqn (4.19) yields an average breach width of 214 m. The terminal breach elevation for piping case should be 270 m when breach height is 63 m, and the width of the dam body at 270 m is about 220 m. Therefore, the latter breach parameters for the piping case appear to be practical and were specified as input to the FLDWAV Model. Hence, the terminal breach width and the breach bottom elevation values specified to FLDWAV Model for <u>piping failure case</u> were 214 m and 270 m, respectively. The last input required as to breach geometry in piping failure case was the center line of the breach, which was specified as 301.5 m to the Model.

Failure Time

Similar to the overtopping case, Eqn. (4.18) by Froehlich (Wahl, 1998) was used once more but to calculate the time of failure for the piping failure case this time.

For V_w and h_b values of 347.57*10^6 m^3 (volume of water stored above breach invert elevation of 270 m at time of failure) and 63 m, respectively; Eqn (4.18) yields the time of failure as 2.05 hours. Hence, the time of failure for the piping failure scenarios was specified as 2.05 hours to the FLDWAV Model.

Another parameter that was changed is the breach formation parameter (ρ_o). As Fread et al (1998) recommends using values of $\rho_o \geq 2$ in case of piping failures, where ρ_o is the pipe formation parameter; a ρ_o of 2 in value was used while simulating the piping failure as opposed to the ρ_o value of 1 used in overtopping failure scenarios.

4.5.2.2.4 Breach Parameter Sensitivity

After an analysis of the breach parameters ($\overline{B}$ and time of failure τ) effect on the peak breach discharge (Q_p), Fread et al. (1998) conclude that it can be generalized, that, for large reservoirs Q_p is quite sensitive to $\overline{B}$ and rather insensitive to τ, while for very small reservoirs Q_p is somewhat insensitive to $\overline{B}$ and fairly sensitive to τ, where the reservoir volume (V_r), dam height (h_d) and the reservoir surface area (A_s) of the dam they analyzed were 250000 acre-ft, 260 ft and 2000 acres, respectively, which they categorize as a dam with a moderately large reservoir.

Given that the top water surface area (A_s), height of dam (h_d), and total volume of reservoir (V_r) for Çınarcık Dam are 10.14 x 10^6 m^2, 123 m. and 372,94 x 10^6 m^3, respectively, which roughly convert to English units as 2505 acres, 403 ft, and 302392 acre-ft; Çınarcık Dam can be regarded as a dam having a large reservoir based on the example above, and it can be deduced that the Q_p, peak discharge through the dam, will mostly be influenced by the average breach width parameter, ($\overline{B}$).

4.5.2.3 Failure Initiation Criteria

The FLDWAV model presents two alternatives to the user as failure initiation criteria. To that end, the user can either specify a time at which the dam failure will commence or a certain critical height of water above the crest elevation of dam, which will indicate commencement of dam failure if the elevation of water trapped behind the reservoir, exceeds it. In this study, the option of specifying a critical height of water above the dam crest elevation, which will be referred to as "*critical WSEL*" hereinafter, was preferred.

A literature review and needs assessment study conducted by Wahl (1998) includes useful information in this regard. The study makes a quotation from Singh and Snorrason's studies in 1982 as to the critical WSEL. It is quoted therein that: "*Singh and Snorrason (1982, cited in Wahl (1998)) provided the first quantitative guidance on breach width.…They also found that for overtopping failures, the maximum overtopping depth prior to failure ranged from 0.16 to 0.61 meters (0.5 to 2.0 ft).*"

Based on this, the overtopping depth for the purpose of case study of Çınarcık Dam was taken as the lower range as 0.16 m so as to ensure conservativeness of the analyses. That is to say, the breach formation would commence when the WSEL behind the reservoir reached 333.16 m.

As for the piping failure cases, the failure initiation criteria was selected as a critical WSEL of 333 m, which is equal to Çınarcık Dam's crest elevation.

4.5.2.4 Finite Differentiation Technique

The FLDWAV model presents a set of options to the user to specify the finite differentiation method for each sub-reach, where the term "sub-reach" defines the smaller reaches formed between the cross-sections defined in the model. The stability and accuracy of these models differ from each other as described in detail by Fread et al. (1998).

Given the complexity and magnitude of the flow due to a dam failure, in case of an almost-instantaneous failure in particular, there may be sub-reaches where flow is subcritical and others where flow is supercritical. Furthermore, the flow regime in a sub-reach may change from one to another during the course of routing. The finite differentiation techniques offered by the FLDWAV model have been geared towards accommodating such changes in flow regime (in other words, the mixed flow) so as to ensure stability and accuracy of the analyses (Fread et al., 1998). These options can be summarized as follows:

- **Local Partial Inertia (LPI) Technique:** Fread et al. (1998) comment that this technique has been developed in order to overcome the possible numerical stability problems encountered in Saint-Venant Equations while solving the four-point implicit numerical scheme for certain mixed flows, particularly where the flow is in the critical flow range (i.e. F_r~1.0). Through a local parameter, σ, the model decides between using the momentum equation with full, partial or no inertial terms included when flow is subcritical, near critical/critical, or supercritical, respectively. Hence, the model takes the advantage of the stability of the diffusion flow while modeling flows near the critical flow range (Fread et al, 1998).

Fread et al. (1998) introduces a "Φ" term to define the degree of unsteadiness of the flow, and states that overall errors in using the LPI technique are very small (less than 2%) for almost all flow conditions ($\Phi > 10$) and that less than 6% for very special flow situations ($5 \leq \varphi \leq 10$) which are only applicable for near instantaneous large dam-failure induced floods in channels of very flat bed slopes, $S_o<0.0003$., where Φ is given by Eqn (4.20):

$$\Phi = \frac{n^2 g^{3/2} y^{1/6}}{\mu \ \partial y / \ \partial t} \tag{4.20}$$

where n is the Manning's resistance coefficient, y is the flow depth, and μ is the constant in Manning's equation (μ=1.49 for English system of units and μ=1.0 for SI units).

- **Mixed-Flow Algorithm:** The FLDWAV Model provides a second option to model the mixed flows. The mixed-flow algorithm (Fread et al., 1998) divides the reaches into sub-reaches in terms of the flow regime, such as sub-critical, critical and super-critical. The model decides on the flow regime by checking the Froude number of the sub-reaches, calculated by the computed initial water depths and categorizes those with $F_r \leq 0.95$ as sub-critical and with $F_r \geq 1.05$ as super-critical. Contiguous sub-reaches where sub-critical flow is dominant are grouped within sub-critical reach group; and with super-critical reach group where super-critical flow is dominant. In case of the sub-critical reach group, backwater computations proceed from downstream-most sub-reach with sub-critical flow to the upstream-most sub-reach with sub-critical flow, and in the reverse order for the super-critical reach groups. (Fread et al., 1998)

- **Characteristics-Based Upwind Explicit Routing:** Fread et al. (1998) comment that it has been observed that the four-point implicit scheme, using the mixed-flow technique previously described, has difficulties when solving the Saint-Venant equations for some near instantaneous, very large dam-break induced flood waves which often produce a moving supercritical-subcritical mixed-flow interface therefore, a technique called "*Characteristics Based Upwind Explicit Numerical Scheme*" has been developed by Jin and Fread (1997, cited in Fread et al., 1998) to simulate flows with strong shocks (near instantaneous dam-break waves) or subcritical/supercritical mixed flows. The explicit scheme is subject to the Courant-Friedrich-Lewy (CFL) condition for numerical stability (Fread et al., 1998).

At first, the LPI technique was used to model the failure of Çınarcık Dam during computations however, this was found to have created convergence problems in a couple of sub-reaches, particularly during scenarios that resulted in failure of the dam and formation of the breach. Therefore, the problematic sub-reaches were specified to the FLDWAV Model to be computed using the explicit scheme, while the remaining sub-reaches were still computed using LPI technique, as Fread et al. (1998) comment that the upwind, explicit algorithm, when combined with the four-point implicit scheme, enables only those portions of an entire river system being modeled to utilize the advantages of accuracy and stability of an explicit method for sharp waves or nearly critical flows, while minimizing the effect of its greater computational requirement by using the implicit algorithm for other reaches of the river system where nearly critical flows do not occur. Fread et al. (1998) suggest using C_n (Courant Number) values in the range of 0.5-0.8 for complicated channel geometry such as rapid expansions and contractions, rapid changes in slope, channel cross sections with wide floodplains, or a large portion of off-channel storage therefore; C_n was specified as 0.6 to the Model.

4.5.2.5 Type of Fluid

The FLDWAV model lets user specify the type of fluid, either Newtonian or Non-Newtonian. For the purpose of the case study of the Çınarcık Dam, the fluid was specified as Newtonian.

4.5.2.6 Levee Modeling

The FLDWAV model allows the user to specify levees to account for interaction between the levees and the main channel. Given the extent of data required to define the levees to the model and lack thereof, this option could not be used.

4.5.2.7 Low Flow Filter Option

The FLDWAV Model (Fread et al., 1998) has a built-in safety feature called "low flow filter" to prevent the flow values of any hydrograph from going below the initial flow values at t=0, thereby preventing the retention of critical errors in depth and flow in the vicinity of a rapidly rising wave front such as associated with dam-break waves or any sudden discharge releases from reservoirs. This further prevents occurrence of errors due to calculated water surface elevations lower than the streambed invert elevation.

During the case study, the low flow filter was turned on so as to prevent computational errors and increase stability of iterations.

4.5.2.8 Automatic Manning n Calibration Option

The FLDWAV Model has a built-in feature called "Automatic Manning n Calibration" that calculates the Manning roughness values for a given reach based on a previously observed hydrograph along the reach. This feature, therefore, requires an observed hydrograph to be available and to be specified as input to the Model.

Unfortunately, there were not any observed hydrographs for the subject reach available on hour-basis. There is only one measurement station (Station #03-04) on Emet Creek, which takes measurements on multiple-days basis. Therefore, the automatic calibration option could not be used in this case study.

4.5.2.9 Time and Distance Steps for Iteration

Fread et al. (1998) comment that it is most important that computational distance steps (Δx_i) in the finite-difference Saint-Venant equations be properly selected via the parameter (Δx_m) or (DXM_i) in order to avoid computational difficulties and to achieve an acceptable level of numerical accuracy.

The Model uses three criteria to select the computational distance steps, the details of which can be found in FLDWAV (Fread et al., 1998)

Briefly, the first criterion takes into account the effect of contraction/expansion. The second criteria accounts for the speed of the flood wave, and the third one for the abrupt changes in bottom slope.

The Model provides 3 options for specifying the distance step:

- **Option 1:** The distance between the two adjacent user-specified cross-sections is used as the distance step.
- **Option 2:** The distance step is user-specified for each i^{th} reach and automatic calculation is by-passed.
- **Option 3:** The distance step is user-specified for each i^{th} reach, and the model still automatically calculates a second set of distance steps values as per the abovementioned criteria whilst using the user-specified distance step values. The model then provides the distance step values calculated as output.

The third option was used for this case study to determine the distance steps since it allows for the most flexible approach. As for the time step, it can either be user-specified or automatically computed by the Model. The time step was user specified as <u>0.05 hour</u> to the Model.

4.5.2.10 Warm-up Procedure Option

Whether the initial conditions are user-specified or automatically generated within FLDWAV, the unsteady flow equations are solved for several time steps using the initial conditions together with boundary conditions, which are held constant during several computational time steps. This allows the errors in the initial conditions to dampen out which results in the initial conditions being more nearly error free when the actual simulation commences and transient boundary conditions are used. If the initial conditions represent an unsteady state, this "warm-up" procedure must not be used. Also, if the downstream boundary is a tide, the warm-up procedure must not be used since the effect of the tide would be dampened. To obtain a proper set of initial conditions for this situation, the user should assume constant inflow hydrographs and run FLDWAV (without the warm-up) for a few tidal cycles. The initial conditions for the actual simulation would be the water surface elevations and discharges computed at the end of simulating the few tidal cycles. (Fread et al., 1998)

Given that the initial conditions represented a steady-state and the downstream boundary is not a tide, the Warm-up procedure was used during this case study to obtain the proper set of initial conditions.

4.5.2.11 Volume Losses Option

The FLDWAV Model allows the user to specify the volume losses occurring through the reach. Fread et al. (1998) comment that often in the case of very large floods including dam-break floods, where the extremely high flows inundate considerable portions of overbank or floodplain, a measurable loss of flow volume occurs, which is due to infiltration into the relatively dry overbank material and flood detention storage losses due to topographic depressions and/or water trapped behind field irrigation levees.

Fread et al (1998) concludes that there is uncertainty associated with volume losses incurred by the dam-break flood as it propagates downstream and inundates large floodplains where infiltration and detention storage losses may occur, which are difficult to predict and are usually neglected, although they may be significant.

Although the Çınarcık Dam is also being used for irrigation purposes and there are several pumping station along the modeling reach, the possible volume losses (due to irrigation usage, evaporation etc.) were neglected so as to achieve more conservative results.

4.5.2.12 Landslide Option

The FLDWAV Model can account for the effect of a landslide to the dam reservoir. During this case study, however, it was assumed that there were no landslides occurring.

4.5.2.13 Expansion/Contraction Coefficients

The FLDWAV integrates the expansion/contraction slope, S_e, to the Saint-Venant Equations so as to take into account their effect on the flow.

To maintain the conservativeness of the analyses, the said coefficients were specified as 0 to the Model.

4.5.2.14 Sinuosity Coefficients

FLDWAV Model allows the user to specify the sinuosity coefficient at each top-width elevation of any given cross-section. This, however, requires there to be flow paths of the subject river available at each top-width elevation so that the user can calculate the sinuosity coefficient corresponding to each top-width elevation for each sub-reach, which is given by Eqn. (4.21) given below:

$$s_{co_k} = s_{m_k} = \Delta x_{c_l} / \Delta x_l \qquad (4.21)$$

where s_{co_k} , s_{m_k} are sinuosity factors for the conservation of mass equation and for the conservation of momentum equation, respectively, k is the index for the respective top-width and Δx_{c_l} and Δx_l are the flow path distance along the meandering channel (below floodplain elevation) and the mean flow-path distance along the floodplain, respectively; as shown in Figure 4.15.

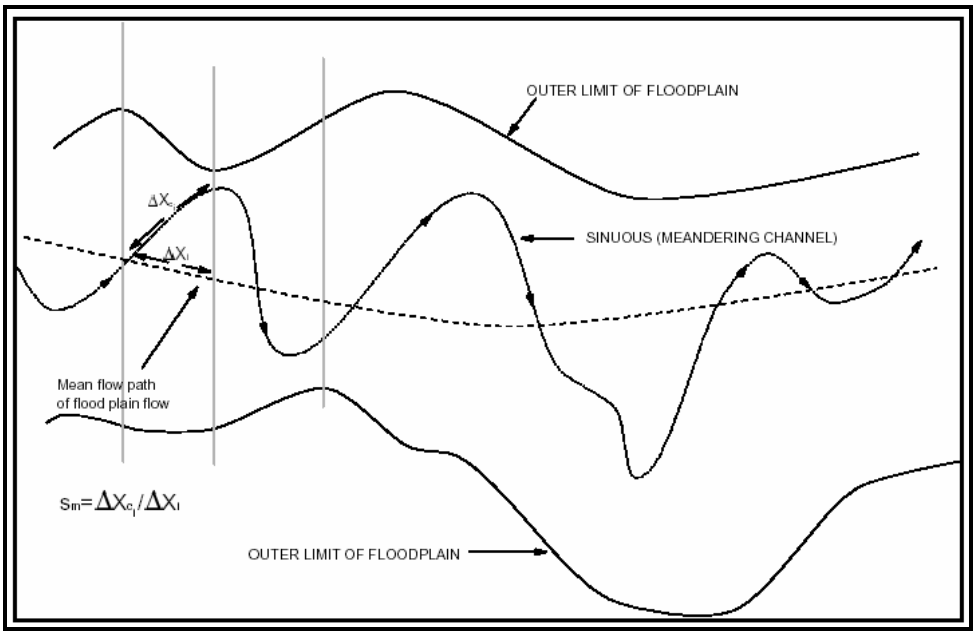

Figure 4-15 Meandering River and Floodplain Showing Sinuosity (s_m) (Fread et al., 1998)

Given the lack of any data available as to the sinuosity coefficients and the lack of plots of flow paths of Orhaneli and Mustafakemalpaşa Creeks at varying WSELs, it was not possible to calculate or use (existing) sinuosity coefficients for the modeling reach. All the sinuosity coefficients, therefore, were specified as 1.0 to the Model.

CHAPTER 5

OUTPUTS OF MODELING

5 OUTPUTS OF MODELING

Following completion of the simulation on FLDWAV for the specified scenarios, the following outputs were acquired:

- maximum WSELs and discharges at each specified cross-section
- the discharge hydrographs of each cross-section,
- graphical cross-sectional views showing the maximum WSELs, water depths and widths at each cross-section,
- times when maximum flow and maximum WSEL occur at each cross-section,
- graphical view of the maximum WSEL for the entire reach,
- graphical view of the maximum discharge for the entire reach, and
- graphical view of the temporal variation of WSELs and discharges for the entire reach.

An example of each of these outputs for cross-section ID#9 in Table 4.5 for scenario S3 are given in Figures 5.1 to 5.7:

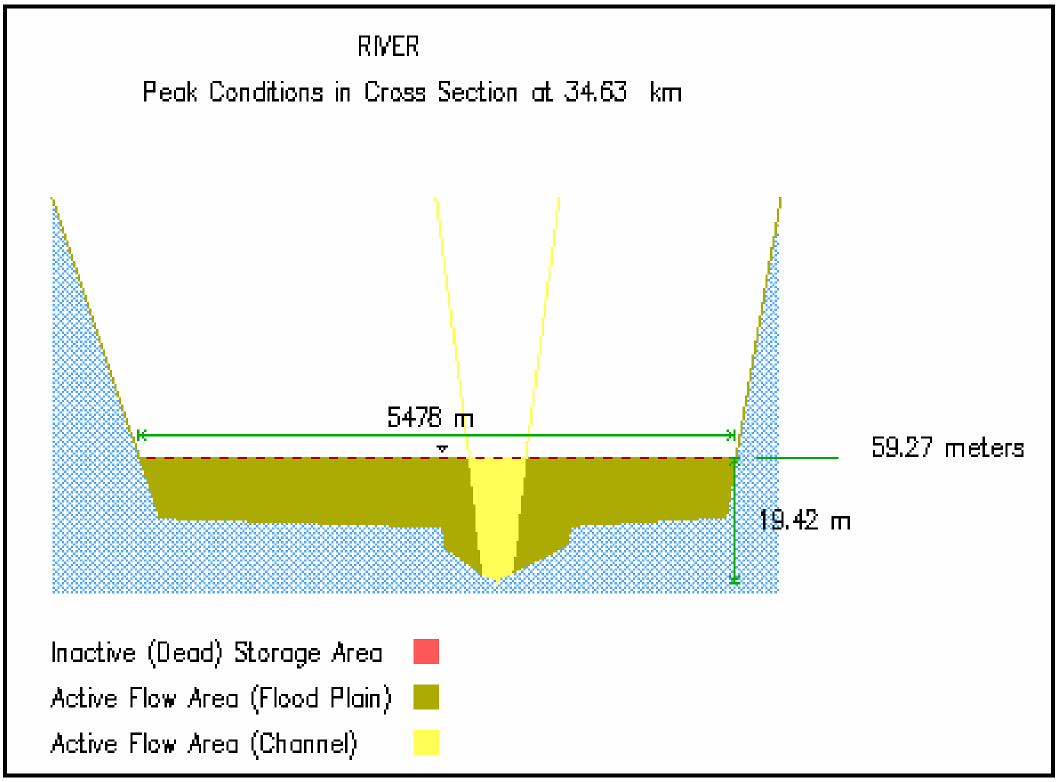

Figure 5-1 Sample Graphical View of the Maximum WSEL at Cross-section Station 34.63 km.

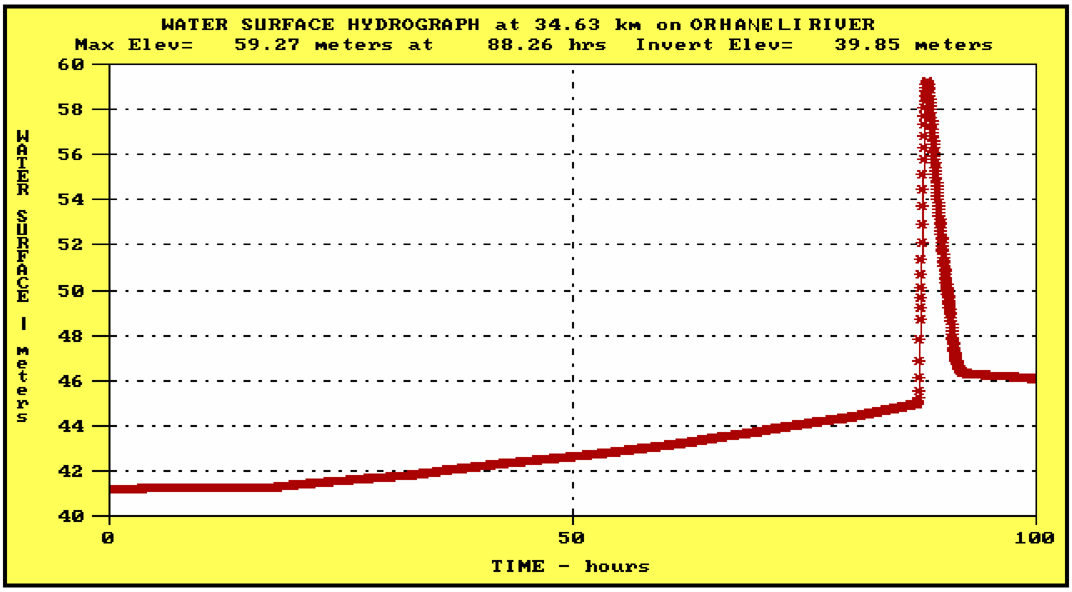

Figure 5-2 Sample Water Surface Hydrograph for Cross-section Station 34.63 km.

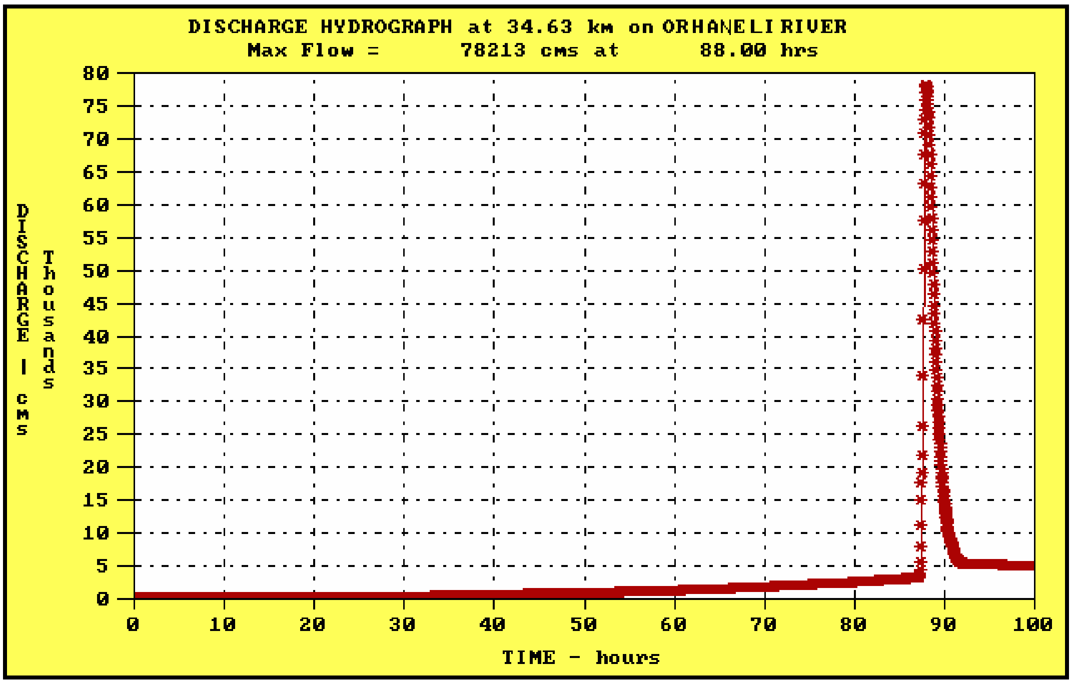

Figure 5-3 Sample Discharge Hydrograph for Cross-section Station 34.63 km.

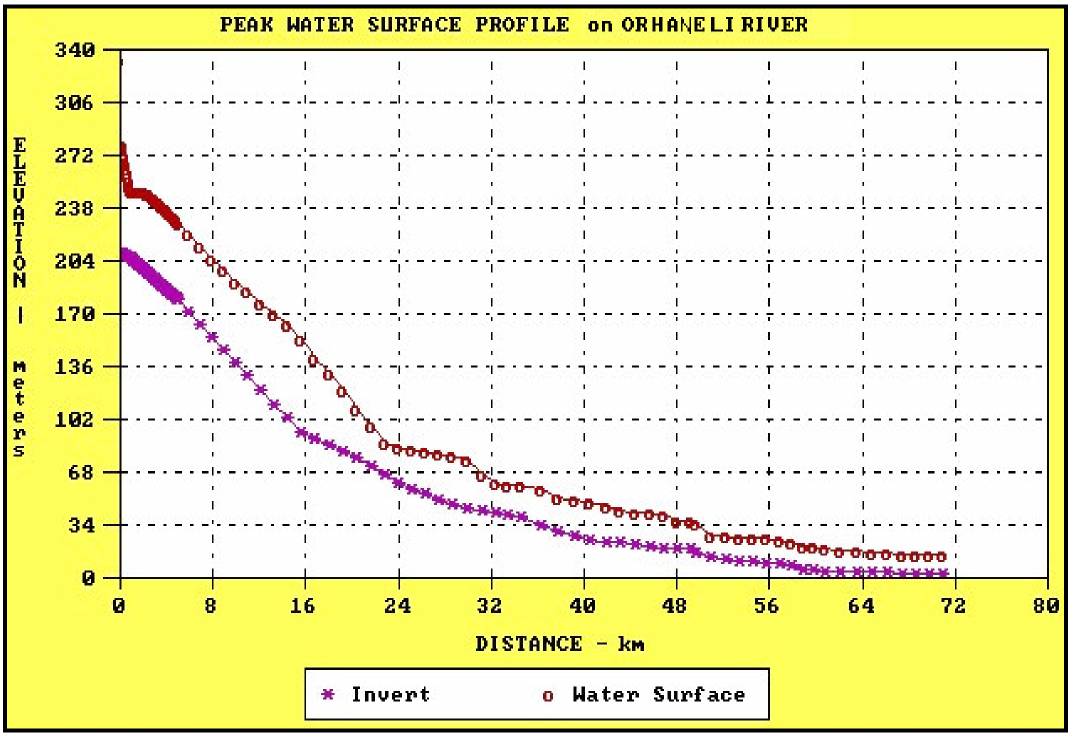

Figure 5-4 Sample Peak Water Surface Profile for the Entire Reach

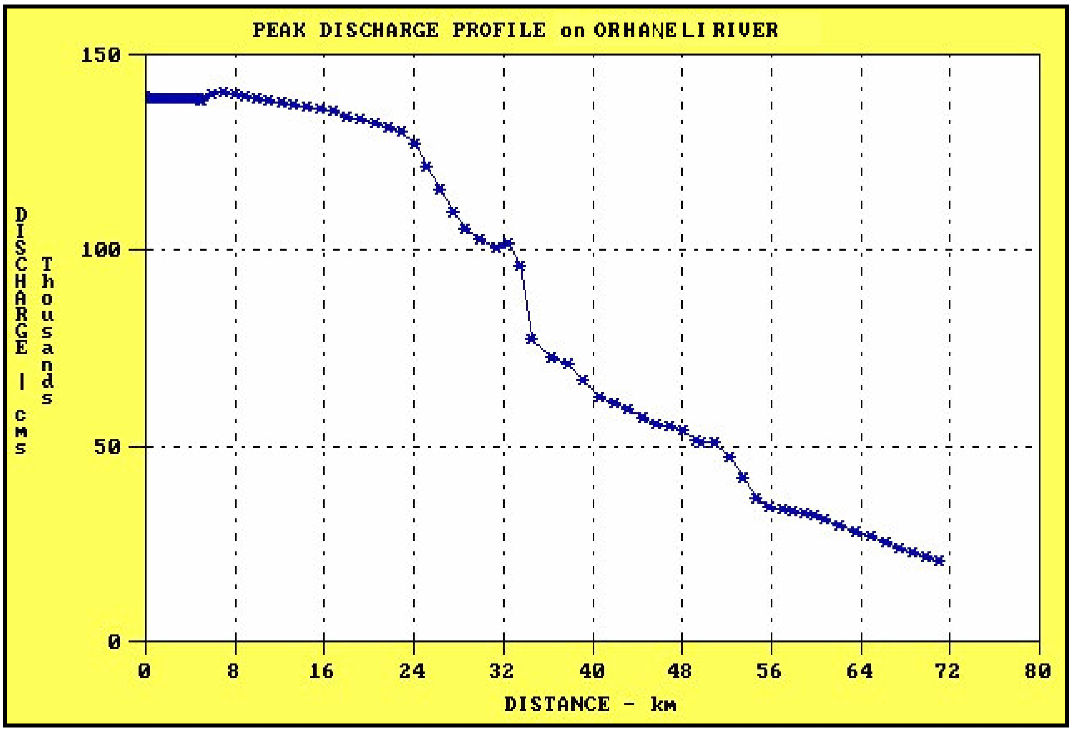

Figure 5-5 Sample Peak Discharge Profile for the Entire Reach

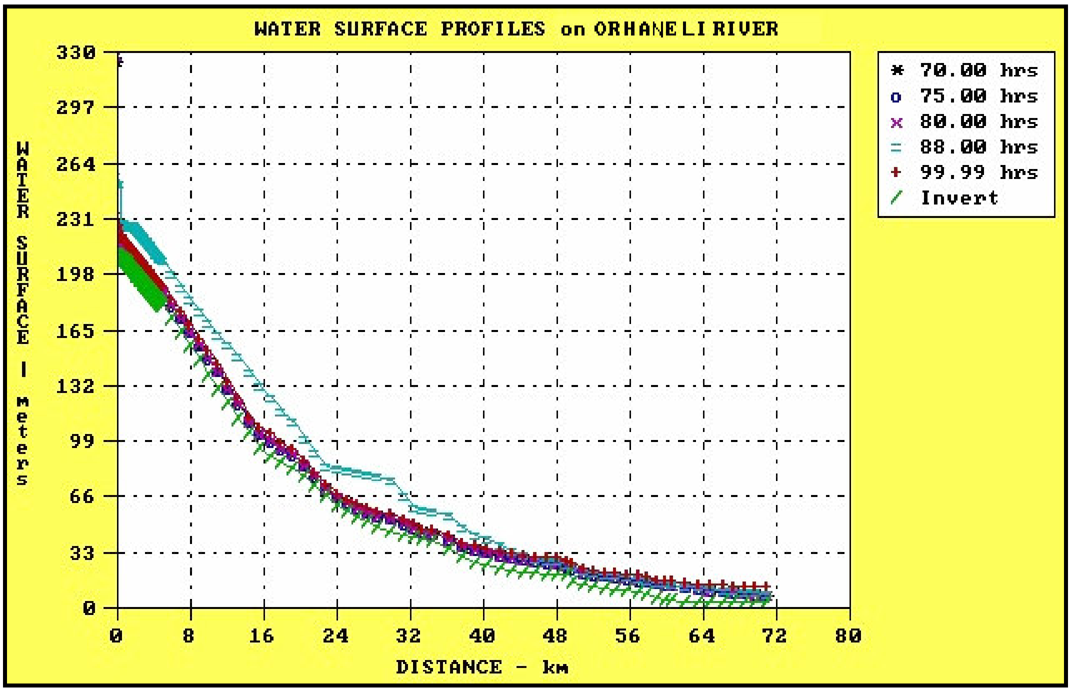

Figure 5-6 Sample Temporal Variation of Water Surface Profile for the Entire Reach

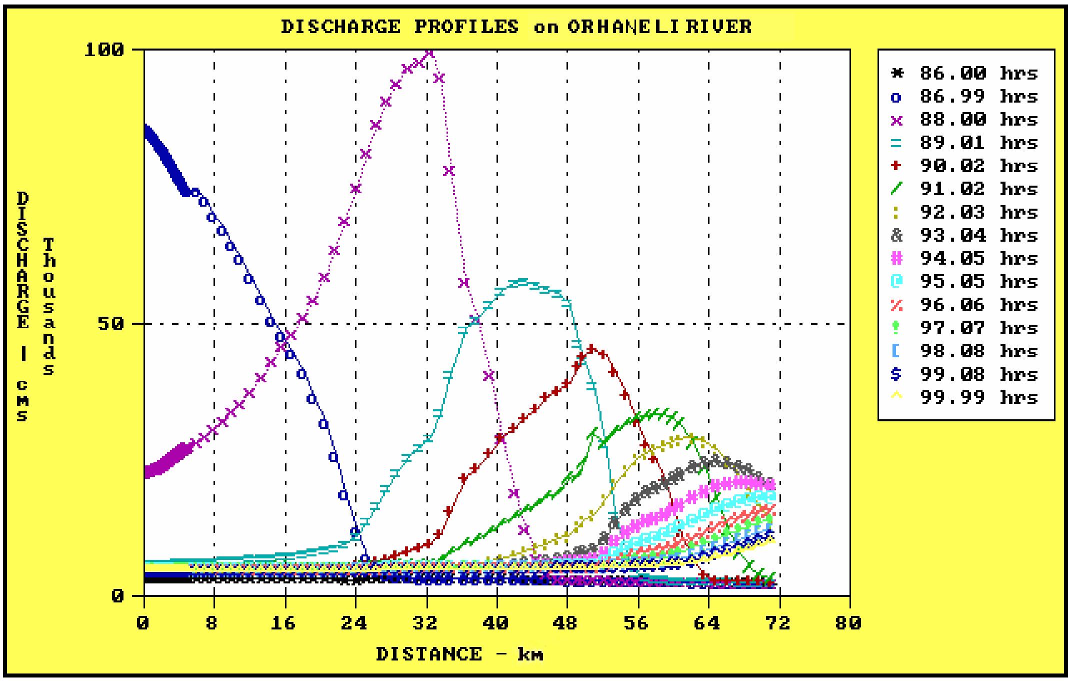

Figure 5-7 Sample Temporal Variation of Discharge Profiles for the Entire Reach

Comparison of the outputs for cross-section ID #9 and #20 in Table 4.5 for each scenario is given in Table 5.1 and Table 5.2, respectively:

Table 5-1 Outputs for Cross-section <u>ID #9</u> for all scenarios (Station Km 34.63)

Failure Type	OVERTOPPING										PIPING		Manning Halved
	SUMMER CASE					WINTER CASE					Summer	Winter	
Scenario ID	**S1**	**S2**	**S3**	**S4**	**S5**	**W1**	**W2**	**W3**	**W4**	**W5**	**S4P**	**W4P**	**S4m**
Max Dis-charge (m^3/s)	5165	74525	78213	77053	72061	5363	75463	78315	77115	72080	55604	55699	116300
Time of Max Dis-charge (hrs)	96.20	93.82	88.00	80.80	68.00	96.90	93.85	88.00	80.80	68.00	81.50	81.50	80.43
Max WSEL (m)	46.19	59.53	59.27	59.12	58.55	46.30	59.65	59.29	59.13	58.56	56.87	56.88	58.35
Time of Max WSEL (hrs)	96.35	94.14	88.26	81.06	68.29	97.05	94.14	88.26	81.09	68.29	81.80	81.80	80.70

Table 5-2 Outputs for Cross-section ID #20 for all scenarios (Station Km 59.00)

Failure Type	OVERTOPPING										PIPING		Manning Halved
	SUMMER CASE					WINTER CASE					Summer	Winter	
Scenario ID	**S1**	**S2**	**S3**	**S4**	**S5**	**W1**	**W2**	**W3**	**W4**	**W5**	**S4P**	**W4P**	**S4m**
Max Dis-charge (m^3/s)	5131	32682	33278	31279	28134	5327	33112	33482	31455	28156	25204	25330	48189
Time of Max Dis-charge (hrs)	99.95	96.78	90.86	83.76	71.12	99.95	96.78	90.86	83.76	71.12	84.70	84.70	82.13
Max WSEL (m)	16.65	20.85	20.89	20.71	20.45	16.73	20.88	20.9	20.73	20.45	20.21	20.23	19.61
Time of Max WSEL (hrs)	99.95	97.04	91.15	84.02	71.38	99.95	97.00	91.15	84.02	71.38	85.05	85	82.49

As can be seen in Tables 5.1 and 5.2, the failure naturally occurs earlier as the spillway gate openings are decreased, which is because the spillway gates are unable to discharge the incoming flow, thereby causing faster WSEL rise inside the reservoir under those circumstances. Hence, the more the spillway gate openings are decreased, the sooner the WSEL behind the reservoir reaches the “critical WSEL”, which triggers the failure of dam and formation of the breach, hence release of the water volume trapped behind the dam.

Another issue that is noteworthy to mention is that in the “spillway gates completely open” case (Scenarios S1 and W1), there is still a high rise of WSEL at each cross-section although the dam does not fail and breach is not formed. For instance, the maximum WSEL at cross-sections ID#13 (Station Km 44.5) and #17 (Station Km 55.25) for scenario S1 are 31.50 and 21.93 m, respectively, which are still enough to inundate some settlements in the vicinity and to risk lives. Therefore, it is evident that completely opening the spillway gates should not be considered as an action while devising an Emergency Action Plan.

A third issue that comes out of the outputs in Tables 5.1 and 5.2 is that the change of 50% in Manning roughness values did not have significant effect on the computed maximum WSELs but on their time of occurrence. For cross-section ID#20 for instance (Table 5.2), a decrease of 50% in Manning roughness values resulted in only some 5% of decrease in maximum WSEL but a 1.53 hours decrease in its time of occurrence, which has significance in terms of arrival time of the flood to the settlement at downstream of the dam. The 50% decrease in Manning roughness values further caused some 50% of increase in the maximum discharge in comparison to the first case, with a 1.63 hours decrease in its time of occurrence. These percentages, certainly, do not yield any strict correlations but still present rough indicators as to the effect of change in Manning roughness values to maximum WSELs and discharges computed. Given that the WSELs occurring at each cross-section are of the essence in the analysis, it can be deduced that the computed WSELs are close to the actual values and the judicial selection of the Manning roughness values do not create any problems in terms of accuracy of the outputs, as far as the WSELs are considered. However, accurate Manning roughness data should be sought to obtain more precise flood arrival times so as to devise sound emergency action plans, where flood arrival times will be of crucial importance as well.

Furthermore, it is evident that the change in value of lateral flow from Emet Creek does not have any significant effect either on the maximum WSELs or the maximum discharges, given the insignificance of the lateral flow value in comparison to the observed discharges in the order of 10000 m^3/s.

It appears that Scenario W3 yields the largest discharge for cross-section ID #9. Furthermore, the maximum computed WSEL at cross-section ID #9 for Scenarios W2 and W3 are quite close to each other, the former being greater. As for cross-section ID #20, Scenario W3 yields highest discharge and WSEL values. Therefore, the output data for Scenario W3 was used while preparing the inundation map for the reach. The map was prepared by plotting the maximum water surface width at each cross-section for Scenario W3 on their places on the topographic map whilst accounting for the respective maximum WSELs as well, as shown in Figure 5.8:

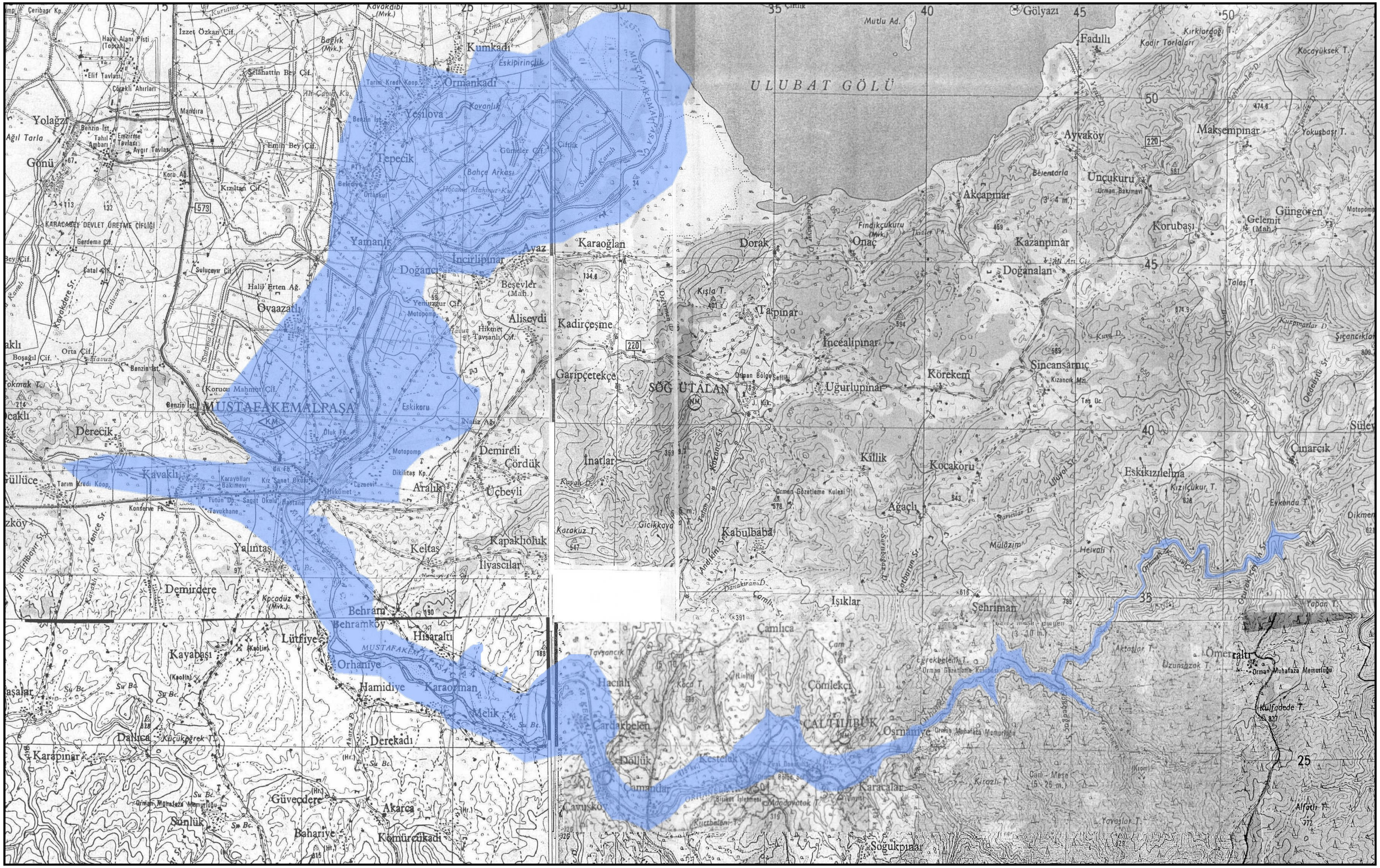

Figure 5-8 Inundation Map for the Modeling Reach for Scenario W3

The maximum WSELs and surface widths for Scenario W3 at each cross-section, used while preparing the inundation map in Figure 5.8, is given below in Table5.3:

Table 5-3 The Maximum WSELs and Surface Widths at each Cross-section for Scenario W3

Cross-Section ID	Station Km	Max WSEL (m)	Max Surface Width (m)
D1	0.00	333.16	325.00
D2	0.10	279.03	198.00
1	1.00	249.38	233.00
2	5.00	228.98	289.00
3	11.00	184.30	239.00
4	15.63	153.13	254.00
5	20.50	108.54	1001.00
6	24.13	85.05	1832.00
7	28.63	78.90	1365.00
8	31.38	67.02	759.00
9	34.63	59.29	5479.00
10	36.38	56.91	1060.00
11	39.25	50.99	2454.00
12	42.00	46.50	1350.00
13	44.50	42.75	1791.00
14	47.00	40.26	1542.00
15	49.25	36.85	11855.00
16	49.75	35.17	3795.00
17	52.25	26.81	10094.00
18	**OMITTED**		
19	57.00	24.56	3259.00
20	59.00	20.90	6556.00
21	59.93	19.91	6195.00
22	60.80	19.21	7690.00
23	63.55	17.31	6683.00
24	66.30	15.98	6356.00
25	**OMITTED**		
26	71.05	14.35	3869.00

One of the important issues to account for while devising Emergency Action Plans is the warning time. Wahl (1997) comments that 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. Wahl (1997) further comments after Brown and Graham that case history-based procedures developed by the Bureau of Reclamation indicate that 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.

For Scenario W3, the closest (in terms of time of occurrence of flooding WSEL) settlement under inundation risk in case of Çınarcık Dam is Kestelek Village, to which the flooding WSEL (not the flood-wave) reaches at t = 70.0 hours, as can be seen in Table 5.6 on the next page. This implies that there would be ample time to warn the settlement for evacuation in case of failure of the Çınarcık Dam provided that the warning is issued before a certain reservoir elevation, the maximum operation level of 330 m. for instance, is exceeded. Another issue with crucial importance in terms of inundation and life risk is that the settlements in the vicinity of the modeling reach may still be under risk of inundation even before the dam has failed, because of the rise in WSEL of the respective creeks due to the high magnitude of water released by the spillways. In the “spillway gates completely open” case for instance (Scenarios S1 and W1), the discharge released through the spillway is about 5200 m^3/s in magnitude for a certain duration, which is enough to inundate settlements close to or nearby the streambed. Furthermore, there might be people working on agricultural lands located on the floodplains at that time as well as people working at factories or other industrial facilities/plants located close to the streambed or on the floodplain, whose lives might be under risk. Therefore, an emergency evacuation warning issuance may still be necessary when the dam does not fail but the spillway releases a discharge of significant magnitude to downstream to prevent failure of the dam itself. For such a purpose, a critical WSEL for each cross-section and the critical spillway discharge that can cause such a WSEL should be identified, and a secondary warning time should be calculated for pre-dam failure conditions. That is to say, the emergency action plans should not solely be based on the failure of the dam itself.

Three settlements that are closest to the dam among the settlements located close to the streambed are Karacalar Village in the vicinity of cross-section ID #5; Kestelek Village in the vicinity of cross-section ID #6; and Çamandar Village in the vicinity of cross-section ID #7. For Scenarios W1-W5, the time when the WSEL in the vicinity reach these settlements' elevations and the settlements' details are given in Tables 5.4 - 5.8, respectively.

Table 5-4 Summary Analysis of Scenario W1 for Three Settlements Located Close to the Streambed

Settlement Name	Settlement Elevation	Distance to Dam	Time of Max WSEL	Max WSEL	Max Depth of Flow	Max Surface Width	Time of Start of Rise of WSEL	Time when WSEL reaches Settlement Elevation	Time of Initiation of Breach Formation
Karacalar Village	90 m	20.50 km	94.85 hours	87.42 m	9.25 m	221 m	14.6 hours	N/A	
Kestelek Village	65 m	24.13 km	95.30 hours	67.94 m	7.38 m	638 m	15.0 hours	59.2 hours	N/A
Çamandar Village	55 m	28.63 km	96.45 hours	57.93 m	10.44 m	440 m	15.6 hours	71.2 hours	

Table 5-5 Summary Analysis of Scenario W2 for Three Settlements Located Close to the Streambed

Settlement Name	Settlement Elevation	Distance to Dam	Time of Max WSEL	Max WSEL	Max Depth of Flow	Max Surface Width	Time of Start of Rise of WSEL	Time when WSEL reaches Settlement Elevation	Time of Initiation of Breach Formation
Karacalar Village	90 m	20.50 km	93.40 hours	108.59 m	30.42 m	1004 m	14.9 hours	92.6 hours	
Kestelek Village	65 m	24.13 km	93.53 hours	84.87 m	24.31 m	1824 m	15.3 hours	59.7 hours	**92.1 hours**
Çamandar Village	55 m	28.63 km	93.66 hours	78.60 m	31.11 m	1352 m	15.6 hours	71.9 hours	

Table 5-6 Summary Analysis of Scenario W3 for Three Settlements Located Close to the Streambed

Settlement Name	Settlement Elevation	Distance to Dam	Time of Max WSEL	Max WSEL	Max Depth of Flow	Max Surface Width	Time of Start of Rise of WSEL	Time when WSEL reaches Settlement Elevation	Time of Initiation of Breach Formation
Karacalar Village	90 m	20.50 km	87.55 hours	108.54 m	30.37 m	1001 m	14.4 hours	86.8 hours	**86.21 hours**
Kestelek Village	65 m	24.13 km	87.71 hours	84.04 m	24.49 m	1832 m	14.8 hours	59.9 hours	
Çamandar Village	55 m	28.63 km	87.84 hours	78.90 m	34.41 m	1365 m	14.7 hours	74.4 hours	

Table 5-7 Summary Analysis of Scenario W4 for Three Settlements Located Close to the Streambed

Settlement Name	Settlement Elevation	Distance to Dam	Time of Max WSEL	Max WSEL	Max Depth of Flow	Max Surface Width	Time of Start of Rise of WSEL	Time when WSEL reaches Settlement Elevation	Time of Initiation of Breach Formation
Karacalar Village	90 m	20.50 km	80.35 hours	108.53 m	30.36 m	1000 m	14.4 hours	79.6 hours	**79 hours**
Kestelek Village	65 m	24.13 km	80.51 hours	84.97 m	24.41 m	1829 m	14.8 hours	68.6 hours	
Çamandar Village	55 m	28.63 km	97.64 hours	78.82 m	34.33 m	1361 m	14.7 hours	79.9 hours	

Table 5-8 Summary Analysis of Scenario W5 for Three Settlements Located Close to the Streambed

Settlement Name	Settlement Elevation	Distance to Dam	Time of Max WSEL	Max WSEL	Max Depth of Flow	Max Surface Width	Time of Start of Rise of WSEL	Time when WSEL reaches Settlement Elevation	Time of Initiation of Breach Formation
Karacalar Village	90 m	20.50 km	67.54 hours	108.17 m	30.00 m	975 m	N/A	66.8 hours	**66.2 hours**
Kestelek Village	65 m	24.13 km	67.67 hours	84.44 m	23.88 m	1803 m	N/A	67.0 hours	
Çamandar Village	55 m	28.63 km	67.84 hours	78.10 m	30.71 m	1334 m	N/A	67.2 hours	

As can be deduced from Table 5.4, through an analysis oriented around the occurrence of dam failure, these settlements seem to have indefinite time for issuance of a warning as the dam does not fail in Scenario W1. It is, however, clear that it takes 59.2 and 71.2 hours for the WSEL to reach the elevations of these settlements therefore, an evacuation warning should be issued before that time has elapsed, and preferably well before 59.0 hours if spillway gates are to be opened completely

When the time of initiation of breach formation for scenarios W2-W5 is plotted against the time when the WSEL reaches the settlement elevation, the following graph in Figure 5.9 below is obtained:

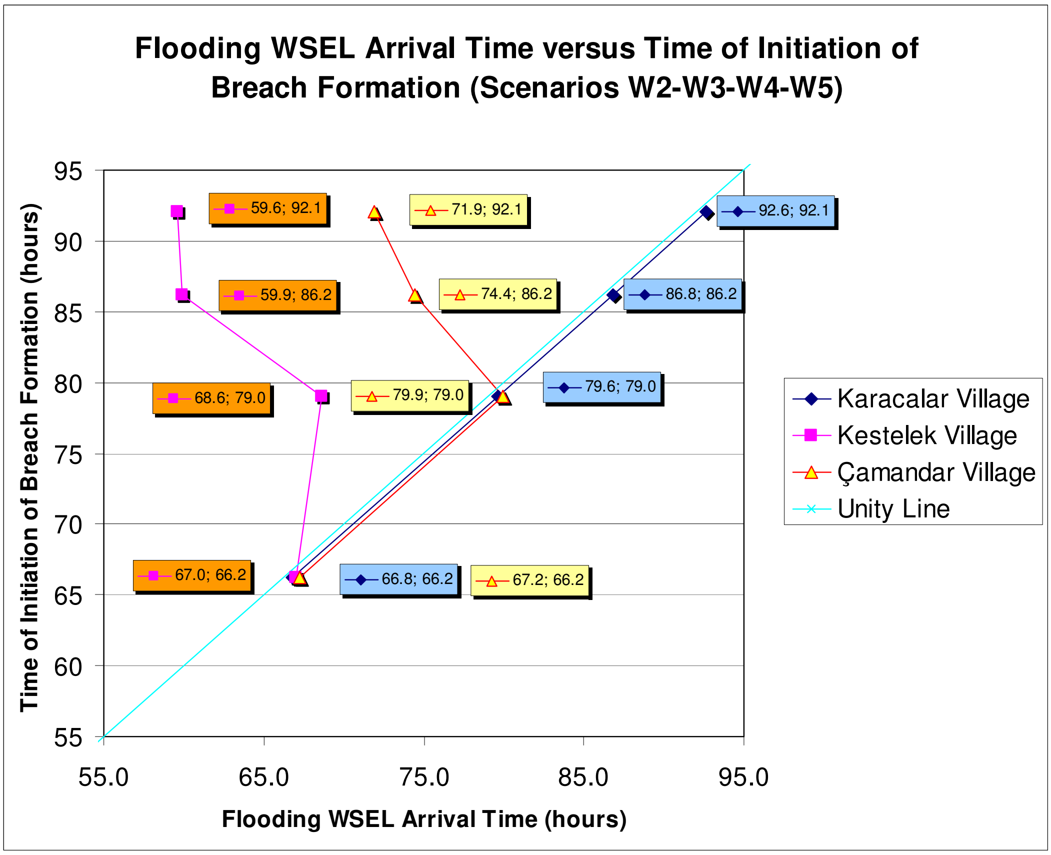

Figure 5-9 Time of Initiation of Breach Formation versus Flooding WSEL Arrival Time (Scenarios W2-W5) *(Flooding WSEL arrival times are in the order of Scenarios W2-W3-W4-W5 from top to bottom)*

The unity line in Figure 5.9 divides the plot into two regions. In the region left of the unity line, dam failure occurs after the time when flooding WSEL reaches the settlements' elevation and vice-versa in the region right of the unity line. Based on Figure 5.9, Scenario W4 can be considered to yield the most favorable dam failure times and flood reaching times as far as the said three settlements are concerned given that the Scenario W4 not only yields a high time of dam failure but also the highest flooding WSEL arrival times for Kestelek and Çamandar Villages, hence allowing for longer evacuation time. An emergency action plan devised based on the gate opening configuration in Scenario W4 (5 gates are 2.5 m open over a total of 10.0 m) would allow about 68 hours of time before the WSEL at Kestelek Village reaches the settlement's elevation. Hence, in case of such an emergency action plan, the issuance of evacuation warning may be decided based on the progress of the inflow and should take place well before 68 hours for the subject reach (in the case of Kestelek Village), giving the evacuees ample time. Once the residents in the settlements are safely evacuated, the spillway gates can be opened completely so as to secure the dam against overtopping failure. Based on the numerical outputs of Scenario W4, the reservoir WSEL at 68 hours appears to be 325.69 m, an elevation lower than the maximum operating level of 330 m as well as the overtopping failure criterion of 333.16 m. Therefore, the safety of the dam can still be ensured by opening the spillway gates completely after 68 hours. Given that the specified inflow hydrograph peaks at 88 hours with peak discharge of about 5200 m^3/s and that the spillway at its maximum capacity can release that discharge, the reservoir WSEL would certainly decrease after 70 hours, thereby securing the dam. The water thus released would however still pose some risk to the properties close to the streambed such as factories, agricultural lands, roads, administrative buildings, residential areas, environmentally sensitive areas etc. Nevertheless, the life risk would be minimized by such an approach although property damage might be inevitable. The 68 hours of time given would further allow the officials to take necessary measures in the settlements such as closing the roads, ensuring electricity and food supplies, securing official documents, reinforcing security measures in place, and take further measures such as deploying barricades or levees to partially obstruct or divert the potential flood.

The excel plots of the WSEL hydrographs and Discharge hydrographs for the said three villages for Scenario W4 are given in Figures 5.10; and 5.11, 5.12, and 5.13; respectively.

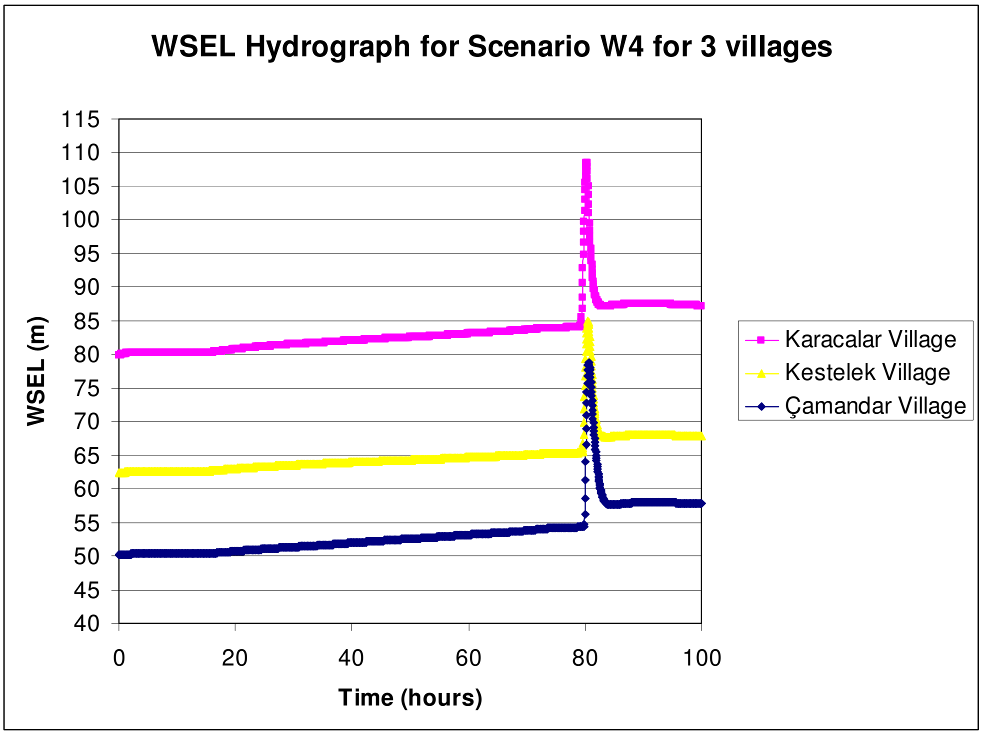

Figure 5-10 WSEL Hydrograph for Scenario W4 for 3 villages

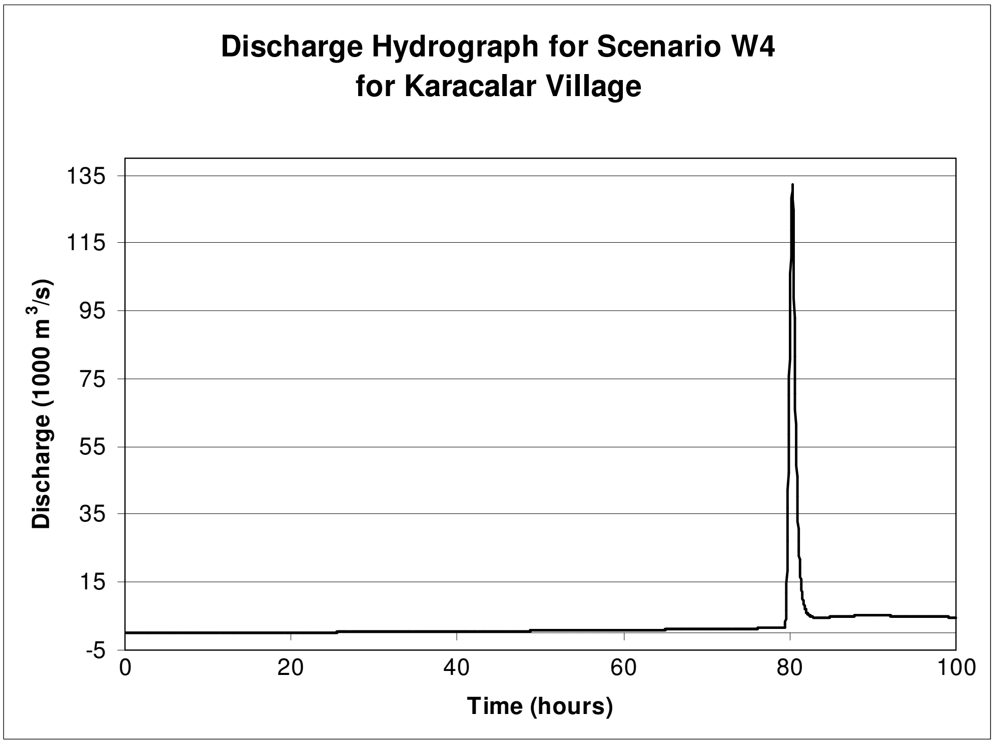

Figure 5-11 Discharge Hydrograph for Scenario W4 for Karacalar Village

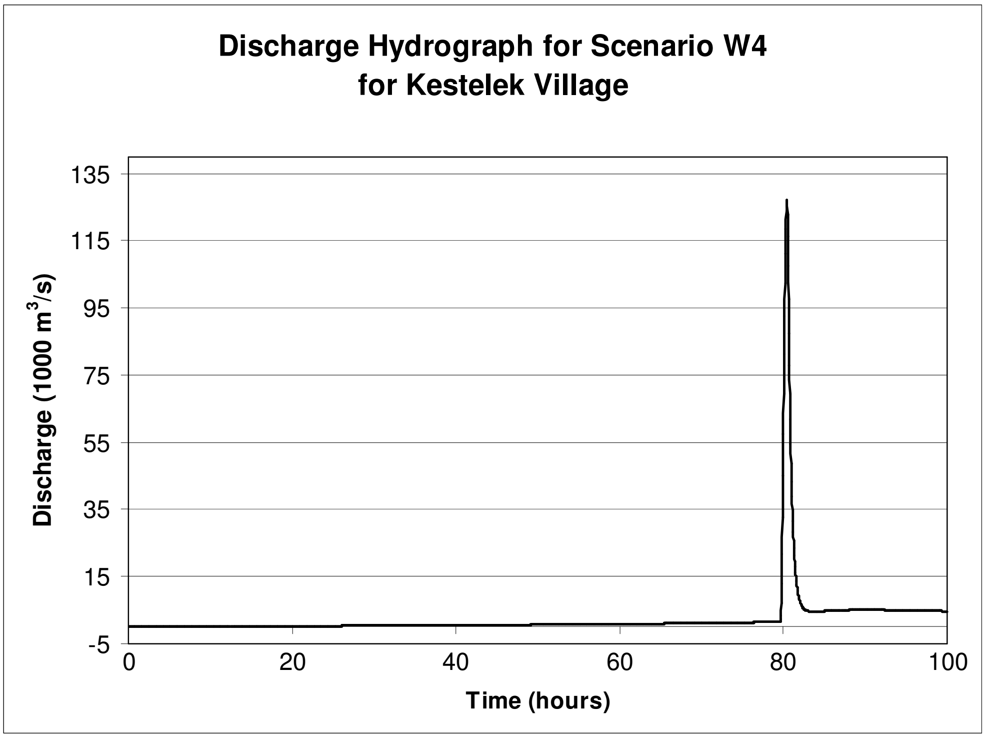

Figure 5-12 Discharge Hydrograph for Scenario W4 for Kestelek Village

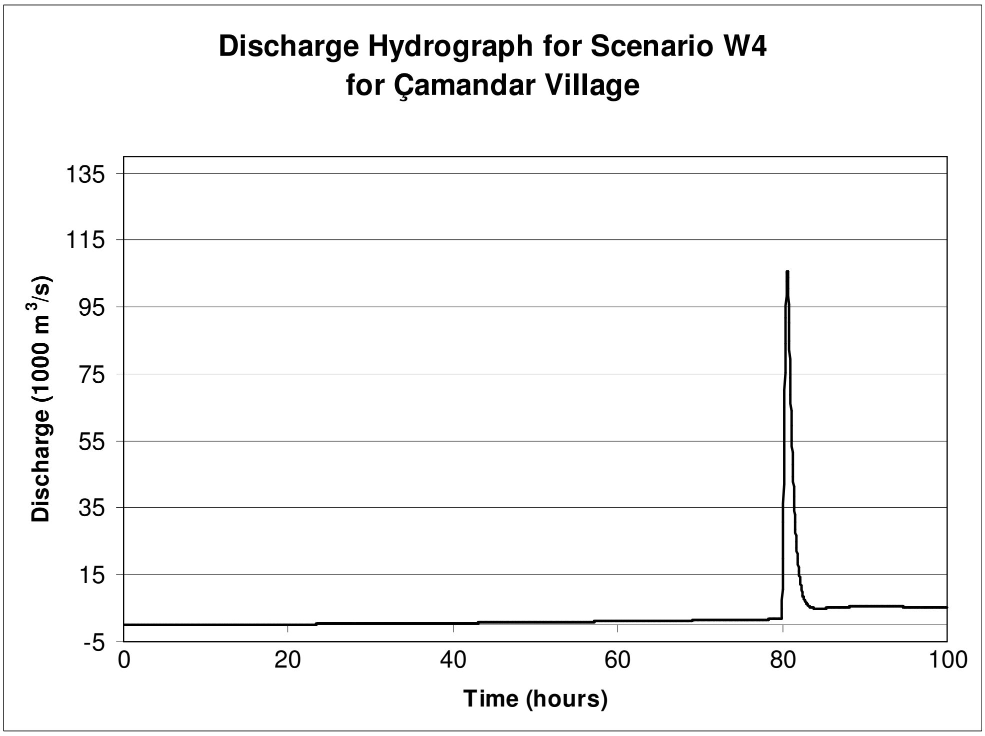

Figure 5-13 Discharge Hydrograph for Scenario W4 for Çamandar Village

The potential consequences of a dam-failure induced flood and the recommended measures will further be delineated under Chapter 6: Potential Consequences and Chapter 7: Recommended Measures, respectively, in the following pages.

CHAPTER 6

POTENTIAL CONSEQUENCES

6 POTENTIAL CONSEQUENCES

Davis (2001) classifies the dams in terms of potential hazards in three groups based on their severity as follows:

- **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.

Davis (2001) further comments that ASCE defines lifelines as transportation systems [highways, airports, rail lines, waterways, ports and harbor facilities] and utility systems [electric power plants, gas and liquid fuel pipelines, telecommunication systems, water supply and waste water treatment facilities].

Davis (2001) categorizes the lifeline facilities in two groups as "Easy to Restore" and "Difficult to Restore". Based on Davis' (2001) definition, "Easy to restore lifeline facilities" are those that generally can be returned to service in seven days or less or for which there are alternative resources or routes available; and "Difficult to restore lifeline facilities" are those that will take more then seven days to recover operation or for which there are no alternative resources available.

Easy to Restore in Seven Days or Less (Davis, 2001)

- Transportation Infrastructure
- Emergency Shelters
- Fuel Supplies
- Radio and Telephone Centers
- Municipal Services Facilities
- Fiber Optic/Phone Trunk Lines
- Water and Gas Pipelines
- Emergency Response Services
- Evacuation Routes

Difficult to Restore in Seven Days or Less (Davis, 2001)

- Potable Water Treatment Facilities
- Wastewater Treatment Facilities
- Power Generation Facilities
- Navigation Facilities
- Communication Facilities
- Fire and Police
- Medical Facilities
- Railroads
- Levies/Flood Control Dams
- Power Transmission Lines

The losses and damages that may be incurred by a potential dam-related flood may be listed as (Davis, 2001):

- **Economic Losses (Davis, 2001)**
    - Direct Physical Property Damage: Residential structures, Industrial buildings, Commercial and Public buildings, Railroads, Main highways, Bridges on main highways and on Township and County roads, Agricultural land and buildings
    - Disruption of utilities (electric, sewer, municipal and agricultural water supply)
    - Replacement Water Supply
    - Costs of alternative transportation or routings
    - Cleanup Costs
    - Repair Costs
    - Replacement Costs
    - Exclude Owner Economic Losses
    - Include Loss of Business Income

    - Commercial: Loss of commercial navigation (Not Applicable to the case of Çınarcık Dam)
    - Recreation: Economic loss due to lost recreation or damage to recreational facilities upstream and downstream of the dam

- **Environmental Damage (Davis, 2001)**
    - Habitat and Wetlands
    - Toxic and Radiological Waste
    - Mine Waste
    - Animal Waste

- **Other Concerns (Davis, 2001)**
    - National security issues (dams upstream of military facilities)
    - Archeological and historic sites
    - Facilities not easily evacuated (Assisted living establishments, prisons, hospitals)

Graham (2001) suggests a simple procedure for estimating loss of life from a dam failure, based on 40 dam failure cases in US with observed fatalities of 50 or more. As given in Table 6.1, the user first decides on the severity of the flood and then finds the appropriate warning time in the 2nd column and the understanding of the flood severity by those under risk in the 3rd column. The 4th column indicates the suggested multiplier and the multiplier range for the fatality rate, which yields the number of fatalities when multiplied by the population of the subject settlement. This is of course a very rough estimate but certainly presents some indicators that can be used while preparing the emergency action plans.

Table 6-1 Recommended Fatality Rates for Estimating the Loss of Life due to Dam Failure (Graham, 2001)

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)	
			Suggested	Suggested Range
HIGH	no warning	not applicable	0.75	0.30 to 1.00
	15 to 60	vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
		precise		
	more than 60	vague		
		precise		
MEDIUM	no warning	not applicable	0.15	0.03 to 0.35
	15 to 60	vague	0.04	0.01 to 0.08
		precise	0.02	0.005 to 0.04
	more than 60	vague	0.03	0.005 to 0.06
		precise	0.01	0.002 to 0.02
LOW	no warning	not applicable	0.01	0.0 to 0.02
	15 to 60	vague	0.007	0.0 to 0.015
		precise	0.002	0.0 to 0.004
	more than 60	vague	0.0003	0.0 to 0.0006
		precise	0.0002	0.0 to 0.0004

Graham (2001) defines the “Flood Severity” in 3 categories as Low, Medium and High, as follows:

- **Low severity:** occurs when **no** buildings are washed off their foundations. Use the low severity category if most structures would be exposed to depths of less than 10 ft (3.3 m) or if DV, defined in Eqn. (6.1), is less than 50 ft^2/s (4.6 m^2/s).

- **Medium severity:** occurs when homes are destroyed but trees or mangled homes remain for people to seek refuge in or on. Use medium flood severity if most structures would be exposed to depths of more than 10 ft (3.3 m) or if DV is more than 50 ft^2/s (4.6 m^2/s).

- **High severity:** occurs when the flood sweeps the area clean and nothing remains. High flood severity should be used only for locations flooded by the near instantaneous failure of a **concrete** dam, or an earthfill dam that turns into "jello" and washes out in seconds rather than minutes or hours. In addition, the flooding caused by the dam failure should sweep the area clean and little or no evidence of the prior human habitation remains after the floodwater recedes. Although rare, this type of flooding occurred below St. Francis Dam in California and Vajont Dam in Italy. The flood severity will usually change to medium and then low as the floodwater travels farther downstream.

Graham (2001) suggests using the parameter **DV** to separate areas anticipated to receive low severity flooding from areas anticipated to receive medium severity flooding, where DV is computed by Eqn. (6.1) as follows:

$$D\,V = \frac{Q_{df} - Q_{2.33}}{W_{df}} \tag{6.1}$$

where:

Q_{df} is the peak discharge at a particular site caused by dam failure.

$Q_{2.33}$ is the mean annual discharge at the same site. This discharge can be easily estimated and it is an indicator of the safe channel capacity.

W_{df} is the maximum width of flooding caused by dam failure at the same site.

Using the inundated area plot and the data as to maximum WSEL occurring at each cross-section, settlements under risk of inundation and loss of life were identified. A list of these settlements, showing their elevations and populations (DİE, 2000) as well as the flood severity ratings and available warning times with respect to time of dam failure for Scenario W3 are given in Table 6.2.

Please note that the following assumptions were made while preparing Table 6.2:

- The mean annual discharge ($Q_{2.33}$) for settlements located between cross-sections #1 and #6 was assumed as 116 m^3/s because during numerical analysis Çınarcık dam was assumed to release a discharge of 116 m^3/s to downstream through its spillways.
- The mean annual discharge ($Q_{2.33}$) for settlements located between cross-sections #6 - #26, on the other hand, was assumed as 116 m^3/s + 350 m^3/s where the second term was added to account for the tributary flow from Emet Creek after cross-section #6, which was assumed to have a mean annual average flow value of 350 m^3/s based on monthly peak flow observations by DSİ's observation station #03-04.
- The peak discharge caused by dam failure (Q_{df}) for each settlement was taken as the average of the peak discharges at upstream and downstream cross-sections bounding the respective reach, based on FLDWAV outputs.
- The maximum flooding width (W_{df}) for each settlement was taken as the average of the maximum water surface widths at upstream and downstream cross-sections bounding the respective reach, based on FLDWAV outputs.
- Similarly, the available warning time was taken as the average of time of Peak Discharge for the upstream and downstream cross-sections bounding the respective reach (given in the 2nd column for each settlement) less the time of initiation of breach formation, assuming that flood warning is issued immediately before the failure of the dam.

Table 6-2 Settlements under Flood Risk and Their Flood Severity Type

Settlement	Reach (u/s – d/s cross-sections)	Population	Elevation (m)	Q_{peak} upstream (m^3/s)	Q_{peak} downstream (m^3/s)	Q_{df} (m^3/s)	$Q_{2.33}$ (m^3/s)	$W_{df\ u/s}$ (m)	$W_{df\ d/s}$ (m)	W_{df} (m)	DV	Flood Severity	Time of $Q_{peak\ u/s}$ (hrs)	Time of $Q_{peak\ d/s}$ (hrs)	Time of Q_{peak} (hrs)	Time of Dam Failure (hrs)	Warning Time Available (hrs)
NONE	D1-D2																
NONE	D2-1																
NONE	1-2																
NONE	2-3																
NONE	3-4																
NONE	4-5																
Karacalar Village	5-6	111	90	132751	127567	130159	116	1001	1832	1416.5	91.81	Medium	87.545	87.610	87.578	86.210	1.368
Gendarmerie Post	5-6	20	80	132751	127567	130159	116	1001	1832	1416.5	91.81	Medium	87.545	87.610	87.578	86.210	1.368
Kestelek Village	6-7	520	65	127567	106542	117054.5	466	1832	1365	1598.5	72.94	Medium	87.610	87.773	87.692	86.210	1.482
Çamandar Village	6-7	193	55	127567	106542	117054.5	466	1832	1365	1598.5	72.94	Medium	87.610	87.773	87.692	86.210	1.482
Çavuşköy Village	7-8	169	45	106542	101491	104016.5	466	1365	759	1062	97.51	Medium	87.773	87.838	87.806	86.210	1.596
Döllük Village	7-8	119	65	106542	101491	104016.5	466	1365	759	1062	97.51	Medium	87.773	87.838	87.806	86.210	1.596
Çardakbelen Village	8-9	320	45	101491	78315	89903	466	759	5479	3119	28.67	Medium	87.838	88.000	87.919	86.210	1.709
NONE	9-10																

Table 6-2 Settlements under Flood Risk and Their Flood Severity Type (Cont'd.)

Settlement	Reach (u/s – d/s cross-sections)	Population	Elevation (m)	Q_{peak} upstream (m^3/s)	Q_{peak} downstream (m^3/s)	Q_{df} (m^3/s)	$Q_{2.33}$ (m^3/s)	$W_{df\ u/s}$ (m)	$W_{df\ d/s}$ (m)	W_{df} (m)	DV	Flood Severity	Time of $Q_{peak\ u/s}$ (hrs)	Time of $Q_{peak\ d/s}$ (hrs)	Time of Q_{peak} (hrs)	Time of Dam Failure (hrs)	Warning Time Available (hrs)
Melik Village	10-11	421	45	73945	67704	70824.5	466	1060	2454	1757	40.04	Medium	88.325	88.488	88.407	86.210	2.197
Karaorman Village	11-12	442	43	67704	61756	64730	466	2454	1350	1902	33.79	Medium	88.488	88.683	88.586	86.210	2.376
Orhaniye Village	12-13	177	43	61756	57775	59765.5	466	1350	1791	1570.5	37.76	Medium	88.683	88.845	88.764	86.210	2.554
NONE	13-14																
MUSTAFAKEMAL PAŞA DISTRICT																	
Hospital	14-15	100	30	55845	52165	54005	466	1542	11855	6698.5	7.993	Medium	89.041	89.301	89.171	86.210	2.961
Art School	14-15	100	30	55845	52165	54005	466	1542	11855	6698.5	7.993	Medium	89.041	89.301	89.171	86.210	2.961
Highways Maintenance Post	14-15	15	30	55845	52165	54005	466	1542	11855	6698.5	7.993	Medium	89.041	89.301	89.171	86.210	2.961
Tobacco Warehouse	14-15	5	30	55845	52165	54005	466	1542	11855	6698.5	7.993	Medium	89.041	89.301	89.171	86.210	2.961
Chicken Farm	14-15	50	30	55845	52165	54005	466	1542	11855	6698.5	7.993	Medium	89.041	89.301	89.171	86.210	2.961
Agricultural Cooperative	14-15	50	30	55845	52165	54005	466	1542	11855	6698.5	7.993	Medium	89.041	89.301	89.171	86.210	2.961

Table 6-2 Settlements under Flood Risk and Their Flood Severity Type (Cont'd.)

Settlement	Reach (u/s – d/s cross-sections)	Population	Elevation (m)	Q_{peak} upstream (m^3/s)	Q_{peak} downstream (m^3/s)	Q_{df} (m^3/s)	$Q_{2.33}$ (m^3/s)	$W_{df\ u/s}$ (m)	$W_{df\ d/s}$ (m)	W_{df} (m)	DV	Flood Severity	Time of Q_{peak} u/s (hrs)	Time of Q_{peak} d/s (hrs)	Time of Q_{peak} (hrs)	Time of Dam Failure (hrs)	Warning Time Available (hrs)
Girls Art School	15-16	60	30	52165	51472	51818.5	466	11855	3795	7825	6.563	Medium	89.301	89.398	89.350	86.210	3.140
Kavaklı Neighbourhood	15-16	100	30	52165	51472	51818.5	466	11855	3795	7825	6.563	Medium	89.301	89.398	89.350	86.210	3.140
Government House	15-16	50	30	52165	51472	51818.5	466	11855	3795	7825	6.563	Medium	89.301	89.398	89.350	86.210	3.140
Penitentiary	15-16	500	25	52165	51472	51818.5	466	11855	3795	7825	6.563	Medium	89.301	89.398	89.350	86.210	3.140
Gutter Factory	16-17	50	20	51472	48000	49736	466	3795	10094	6944.5	7.095	Medium	89.398	89.626	89.512	86.210	3.302
Ovaazatlı	17-19	2034	20	48000	34116	41058	466	10094	3259	6676.5	12.46	Medium	89.626	90.633	90.130	86.210	3.920
Flour Factory	17-19	50	20	48000	34116	41058	466	10094	3259	6676.5	12.46	Medium	89.626	90.633	90.130	86.210	3.920
Yamanlı	19-20	811	15	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537
Tepecik Neighbourhood	19-20	2140	14	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537
Doğancı Neighbourhood	19-20	357	20	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537

Table 6-2 Settlements under Flood Risk and Their Flood Severity Type (Cont'd.)

Settlement	Reach (u/s – d/s cross-sections)	Population	Elevation (m)	Q_{peak} upstream (m^3/s)	Q_{peak} downstream (m^3/s)	Q_{df} (m^3/s)	$Q_{2.33}$ (m^3/s)	$W_{df\ u/s}$ (m)	$W_{df\ d/s}$ (m)	W_{df} (m)	DV	Flood Severity	Time of Q_{peak} u/s (hrs)	Time of Q_{peak} d/s (hrs)	Time of Q_{peak} (hrs)	Time of Dam Failure (hrs)	Warning Time Available (hrs)
Post Office	19-20	50	15	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537
Gas Station	19-20	50	12	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537
Elementary School	19-20	100	15	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537
Municipality	19-20	30	15	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537
Yeşilova Neighbourhood	19-20	2223	12	34116	33482	33799	466	3259	6556	4907.5	5.084	Medium	90.633	90.860	90.747	86.210	4.537
Ormankadı Neighbourhood	20-21	1403	12	33482	32741	33111.5	466	6556	6195	6375.5	5.27	Medium	90.860	91.023	90.942	86.210	4.732
İncilipinar Neighbourhood	21-22	475	15	32741	31839	32290	466	6195	7690	6942.5	4.138	Low	91.023	91.185	91.104	86.210	4.894
Kovanlık	22-23	50	12	31839	28651	30245	466	7690	6683	7186.5	4.456	Low	91.185	91.868	91.527	86.210	5.317
Bahçe Arkası	22-23	50	12	31839	28651	30245	466	7690	6683	7186.5	4.456	Low	91.185	91.868	91.527	86.210	5.317
Ayaz Neighbourhood	22-23	755	15	31839	28651	30245	466	7690	6683	7186.5	4.456	Low	91.185	91.868	91.527	86.210	5.317

Table 6-2 Settlements under Flood Risk and Their Flood Severity Type (Cont'd.)

Settle-ment	Reach (u/s – d/s cross-sections)	Popula-tion	Eleva-tion (m)	Q_{peak} upstream (m^3/s)	Q_{peak} down-stream (m^3/s)	Q_{df} (m^3/s)	$Q_{2.33}$ (m^3/s)	$W_{df\ u/s}$ (m)	$W_{df\ d/s}$ (m)	W_{df} (m)	DV	Flood Se-verity	Time of Q_{peak} u/s (hrs)	Time of Q_{peak} d/s (hrs)	Time of Q_{peak} (hrs)	Time of Dam Fail-ure (hrs)	Warn-ing Time Avail-able (hrs)
Gümeler Farm	23-24	50	12	28651	25.588	14338.29	466	6683	6356	6519.5	2.183	Low	91.868	92.421	92.145	86.210	5.935
Eskipirin-çlik	23-24	50	12	28651	25.588	14338.29	466	6683	6356	6519.5	2.183	Low	91.868	92.421	92.145	86.210	5.935
NONE	24-26																

Graham (2001) defines the “**Warning Time**” in 3 categories as given below:

- **No warning:** means that no warning is issued by the media or official sources in the particular area prior to the flood water arrival; only the possible sight or sound of the approaching flooding serves as a warning.

- **Some warning**: means officials or the media begin warning in the particular area 15 to 60 minutes before floodwater arrival. Some people will learn of the flooding indirectly when contacted by friends, neighbors or relatives.

- **Adequate warning**: means officials or the media begin warning in the particular area more than 60 minutes before the floodwater arrives. Some people will learn of the flooding indirectly when contacted by friends, neighbors or relatives. The warning time for a particular area downstream from a dam should be based on when a dam failure warning is initiated and the flood travel time. For instance, assume a dam with a campground immediately downstream and a town where flooding begins 4 hours after the initiation of dam failure. If a dam failure warning is initiated 1 hour after dam failure, the warning time at the campground is zero and the warning time at the town is 3 hours. The fatality rate in areas with medium severity flooding should drop below that recommended in Table 2 as the warning time increases well beyond one hour. Repeated dam failure warnings, confirmed by visual images on television showing massive destruction in upstream areas, should provide convincing evidence to people that a truly dangerous situation exists and of their need to evacuate. This should result in higher evacuation rates in downstream areas and in a lowering of the fatality rate.

Regarding the “**Flood Severity Understanding**”, Graham (2001) comments that the warning is comprised of two elements as:

- Alerting people to danger,
- Requesting that people at risk take some action.

The “**Flood Severity Understanding**” as per Graham (2001) is defined as:

- **Vague Understanding of Flood Severity**: means that the warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding.

- **Precise Understanding of Flood Severity means:** that the warning issuers have an excellent understanding of the flooding due to observations of the flooding made by themselves or others.

Based on the severity types identified for each settlement under flooding risk in Table 6.2, the expected fatality rates were calculated using the suggested coefficients in Table 6.1, for each of the following four cases:

1. The people under risk have precise understanding of the flood warning.
2. The people under risk have vague understanding of the flood warning.
3. The warning is issued not immediately before the failure of the dam but 1 hour later.
4. A combination of cases 2 and 3.

The fatality rates thus calculated are presented in Table 6.3:

Table 6-3 # of Possible Fatalities in the Settlements under Risk for four Cases (* represents Estimated Population)

			Case 1: Precise Understanding of Warning			Case 2: Vague Understanding of Warning			Case 3: Warning issued 1 hour after failure of Dam & precise understanding of warning			Case 4: Warning issued 1 hour after failure of Dam & vague understanding of warning	
Settlement	Reach (u/s – d/s cross-sections)	Population	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)
Karacalar Village	5-6	111	1.368	0.01	2	1.368	0.03	4	0.368	0.02	3	0.04	5
Gendarmerie Post	5-6	20	1.368	0.01	1	1.368	0.03	1	0.368	0.02	1	0.04	1
Kestelek Village	6-7	520	1.482	0.01	6	1.482	0.03	16	0.481	0.02	11	0.04	21
Çamandar Village	6-7	193	1.482	0.01	2	1.482	0.03	6	0.481	0.02	4	0.04	8
Çavuşköy Village	7-8	169	1.596	0.01	2	1.596	0.03	6	0.596	0.02	4	0.04	7

Table 6-3 # of Possible Fatalities in the Settlements under Risk for four Cases (* represents Estimated Population) (Cont'd.)

			Case 1: Precise Understanding of Warning			Case 2: Vague Understanding of Warning			Case 3: Warning issued 1 hour after failure of Dam & precise understanding of warning			Case 4: Warning issued 1 hour after failure of Dam & vague understanding of warning	
Settlement	Reach (u/s – d/s cross-sections)	Population	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)
Döllük Village	7-8	119	1.596	0.01	2	1.596	0.03	4	0.596	0.02	3	0.04	5
Çardakbelen Village	8-9	320	1.709	0.01	4	1.709	0.03	10	0.709	0.02	7	0.04	13
Melik Village	10-11	421	2.197	0.01	5	2.197	0.03	13	1.197	0.01	5	0.03	13
Karaorman Village	11-12	442	2.376	0.01	5	2.376	0.03	14	1.376	0.01	5	0.03	14
Orhaniye Village	12-13	177	2.554	0.01	2	2.554	0.03	6	1.554	0.01	2	0.03	6
Hospital *	14-15	100	2.961	0.01	1	2.961	0.03	3	1.961	0.01	1	0.03	3

Table 6-3 # of Possible Fatalities in the Settlements under Risk for four Cases (* represents Estimated Population) (Cont'd.)

			Case 1: Precise Understanding of Warning			Case 2: Vague Understanding of Warning			Case 3: Warning issued 1 hour after failure of Dam & precise understanding of warning			Case 4: Warning issued 1 hour after failure of Dam & vague understanding of warning	
Settlement	Reach (u/s – d/s cross-sections)	Population	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)
Art School *	14-15	100	2.961	0.01	1	2.961	0.03	3	1.961	0.01	1	0.03	3
Highways Maintenance Post *	14-15	15	2.961	0.01	1	2.961	0.03	1	1.961	0.01	1	0.03	1
Tobacco Warehouse *	14-15	5	2.961	0.01	1	2.961	0.03	1	1.961	0.01	1	0.03	1
Chicken Farm *	14-15	50	2.961	0.01	1	2.961	0.03	2	1.961	0.01	1	0.03	2
Agricultural Cooperative *	14-15	50	2.961	0.01	1	2.961	0.03	2	1.961	0.01	1	0.03	2
Girls Art School *	15-16	60	3.140	0.01	1	3.140	0.03	2	2.140	0.01	1	0.03	2

Table 6-3 # of Possible Fatalities in the Settlements under Risk for four Cases (* represents Estimated Population) (Cont'd.)

			Case 1: Precise Understanding of Warning			Case 2: Vague Understanding of Warning			Case 3: Warning issued 1 hour after failure of Dam & precise understanding of warning			Case 4: Warning issued 1 hour after failure of Dam & vague understanding of warning	
Settlement	Reach (u/s – d/s cross-sections)	Population	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)
Kavaklı Neighbourhood *	15-16	100	3.140	0.01	1	3.140	0.03	3	2.140	0.01	1	0.03	3
Government House *	15-16	50	3.140	0.01	1	3.140	0.03	2	2.140	0.01	1	0.03	2
Penitentiary *	15-16	500	3.140	0.01	5	3.140	0.03	15	2.140	0.01	5	0.03	15
Gutter Factory *	16-17	50	3.302	0.01	1	3.302	0.03	2	2.302	0.01	1	0.03	2
Ovaazatlı	17-19	2034	3.920	0.01	21	3.920	0.03	62	2.920	0.01	21	0.03	62
Flour Factory *	17-19	50	3.920	0.01	1	3.920	0.03	2	2.920	0.01	1	0.03	2

Table 6-3 # of Possible Fatalities in the Settlements under Risk for four Cases (* represents Estimated Population) (Cont'd.)

			Case 1: Precise Understanding of Warning			Case 2: Vague Understanding of Warning			Case 3: Warning issued 1 hour after failure of Dam & precise understanding of warning			Case 4: Warning issued 1 hour after failure of Dam & vague understanding of warning	
Settlement	Reach (u/s – d/s cross-sections)	Population	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)
Yamanlı Neighbourhood	19-20	811	4.537	0.01	9	4.537	0.03	25	3.537	0.01	9	0.03	25
Tepecik Neighbourhood	19-20	2140	4.537	0.01	22	4.537	0.03	65	3.537	0.01	22	0.03	65
Doğancı Neighbourhood	19-20	357	4.537	0.01	4	4.537	0.03	11	3.537	0.01	4	0.03	11
Post Office *	19-20	50	4.537	0.01	1	4.537	0.03	2	3.537	0.01	1	0.03	2
Gas Station *	19-20	50	4.537	0.01	1	4.537	0.03	2	3.537	0.01	1	0.03	2
Elementary School *	19-20	100	4.537	0.01	1	4.537	0.03	3	3.537	0.01	1	0.03	3

Table 6-3 # of Possible Fatalities in the Settlements under Risk for four Cases (* represents Estimated Population) (Cont'd.)

			Case 1: Precise Understanding of Warning			Case 2: Vague Understanding of Warning			Case 3: Warning issued 1 hour after failure of Dam & precise understanding of warning			Case 4: Warning issued 1 hour after failure of Dam & vague understanding of warning	
Settlement	Reach (u/s – d/s cross-sections)	Population	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)
Municipality *	19-20	30	4.537	0.01	1	4.537	0.03	1	3.537	0.01	1	0.03	1
Yeşilova Neighbourhood	19-20	2223	4.537	0.01	23	4.537	0.03	67	3.537	0.01	23	0.03	67
Ormankadı Neighbourhood	20-21	1403	4.732	0.01	15	4.732	0.03	43	3.732	0.01	15	0.03	43
İncilipinar Neighbourhood	21-22	475	4.894	0.0002	1	4.894	0.0003	1	3.894	0.0002	1	0.0003	1
Kovanlık *	22-23	50	5.317	0.0002	1	5.317	0.0003	1	4.317	0.0002	1	0.0003	1
Bahçe Arkası *	22-23	50	5.317	0.0002	1	5.317	0.0003	1	4.317	0.0002	1	0.0003	1

Table 6-3 # of Possible Fatalities in the Settlements under Risk for four Cases (* represents Estimated Population) (Cont'd.)

			Case 1: Precise Understanding of Warning			Case 2: Vague Understanding of Warning			Case 3: Warning issued 1 hour after failure of Dam & precise understanding of warning			Case 4: Warning issued 1 hour after failure of Dam & vague understanding of warning	
Settlement	Reach (u/s – d/s cross-sections)	Population	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Warning Time Available (hrs)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)	Suggested Fatality Rate as per Table 7.1	# of Fatalities (Fractions Rounded-up)
Ayaz Neighbourhood	22-23	755	5.317	0.0002	1	5.317	0.0003	1	4.317	0.0002	1	0.0003	1
Gümeler Farm *	23-24	50	5.935	0.0002	1	5.935	0.0003	1	4.935	0.0002	1	0.0003	1
Eskipirinçlik *	23-24	50	5.935	0.0002	1	5.935	0.0003	1	4.935	0.0002	1	0.0003	1
TOTAL		**14250**	**TOTAL**		**151**	**TOTAL**		**405**	**TOTAL**		**165**	**TOTAL**	**418**

As can be seen in Table 6.3, understanding of the flood warning plays a major role in terms of minimizing the potential fatalities given that the number of fatalities increase by 254 (168%) in case of Çınarcık Dam Failure when the understanding of the flood warning is assumed to be vague. Furthermore, the time of issuance of warning does not seem to have significant effect on the number of fatalities, which is understandable as the settlements at 1 hour distance or less to the flood wave are mostly villages with low population. Hence, the increase in fatality coefficient does not produce a significant effect. This is certainly the case with the subject reach of this thesis work, that is to say, in different reaches and regions where there are settlements with high population close to a dam, the time of issuance of warning would certainly have considerable impact on the fatality figures.

The measures that may be taken to ensure sound communication of the flood threat to the settlements under risk will be discussed under **Chapter 7: Recommended Measures.**

CHAPTER 7

RECOMMENDED MEASURES

7 RECOMMENDED MEASURES

There are several pre-emptive measures that can be taken to minimize the risks that may be presented by a potential dam-break induced flood.

Inundation Maps

One of the preemptive measures is to prepare potential inundation maps and to prepare a detailed emergency action plan before the occurrence of an actual dam failure. The inundation map will help identification of areas under risk, the location where population is concentrated, locations that are hard to evacuate or where there are not any high grounds to seek shelter etc.

The Engineer Manual 1110-2-1420 (U.S. Army Corps of Engineers, 1997) 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 if 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 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.

Emergency Action Plans

As a pre-event measure, there is further need for preparation of comprehensive evacuation (emergency action) plans. Engineer Manual 1110-2-1420 (U.S. Army Corps of Engineers, 1997) 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 addition to these, a comprehensive emergency action plan should also indicate evacuation means, evacuation routes, safe and easily accessible gathering locations, people responsible of managing the evacuation in each village, the duties and responsibilities of each party including the dam personnel, state authorities such as the governorship, sub-governorship, municipalities, village headmen, gendarmerie, military forces, and other state institutions responsible of the subject dam. To prevent off-scene losses, it should be beneficial to construct shelters for the evacuees to take refuge at the gathering points with separate phone lines and reserve supplies such as medicines, food, water, clothing and heating. In addition to risk to life and property, emergency action plans should further take into account potential damages to any environmentally sensitive areas such as the Uluabat Lake located at the downstream-most end of the modeling reach in case of the Çınarcık Dam.

Uluabat is one of the important fishing areas in Turkey. The lake is surrounded by agricultural lands and industrial facilities processing agricultural raw products and is one of the richest wet lands in Turkey in terms aquatic vegetation biodiversity. The lake further hosts the largest water lily beds in Turkey and was declared as RAMSAR Protection Site on April 15th, 1998. Furthermore, as it is located on the bird migration route entering Turkey from North-West and close to the Bird Heaven Lake, Uluabat Lake is one of the most important wet lands in Turkey as well as in Europe and Middle East, and also one of the 97 Important Bird Sites In Turkey. (Özesmi, 2000)

Hence, another potential consequence of the failure of the Çınarcık Dam would be severe damage to the bio-diversity and habitat in Uluabat Lake, which could not be mended by monetary allocations but nature itself. Özesmi states that in 1998 “Doğal Hayatı Koruma Derneği” initiated a joint project with DSİ and the Ministry of Environment of Turkey, geared towards ensuring sustainability of the lake eco-system and controlled and planned use of the resources in a mutually sustainable manner. This implies that there are already measures being taken against such a risk.

HEC Research Documents 19 and 20 can be consulted for example emergency plans and evacuation plans, respectively (HEC1983a and 1983b).

Observation Station Upstream of Çınarcık Dam

Certainly, there is further need for an observation station farther upstream of the dam to take real-time measurements of the incoming flow, which should simultaneously forward the measurements to an emergency response station at the dam for real-time forecasting of the probable flow conditions downstream of the dam and response of the WSEL in the reservoir, thereby providing insight for the issuance of evacuation warnings for specific settlements at the downstream and deciding on the width of the spillway gate openings, hence the magnitude of the discharge released through the gates. In such a manner, emergency response team might act based on pre-specified criteria in order to minimize potential losses and prevent the failure of the dam at the same time.

Risk Assessment Study

The assessment of the risks imposed by a potential dam failure should be conducted through a risk assessment study geared towards identifying all social, economic and environmental risks and should further address the proposed measures, the agencies responsible for implementing the measures, stipulate for timeframes for implementation thereof, and identify the duties and responsibilities of each agency or party involved.

Levees, Flood Detention Basin and Diversion Channels

There are already some levees built along either side of Mustafakemalpaşa Creek in the vicinity of Mustafakemalpaşa District. The elevations of these levees were taken into consideration while improving the cross-section data and inputting them to the Model. Despite the existence of the levees, the flood wave after the dam failure still manages to penetrate further inland to the District. This is understandable given that the levees have most probably been designed and placed to mitigate or prevent the threat of run-off flood. Therefore, the existing levees should be improved to cope up with the threat of a dam-failure flood and/or further bank protection measures should be devised to be implemented in the event of dam failure. The event-time measures could be reinforcing the banks and the existing levees by placing sand-bags or dumping sand, implementing the emergency action plan for evacuation. Pre-event measures could be improving the existing levees and construction new levees at critical locations along the river, constructing additional flood detention basins and regulators at suitable locations along the modeling reach, dredging emergency release channels to divert the flow to less-populated areas in case the WSEL exceeds a certain critical value etc. In this sense, a diversion channel could be built upstream of Mustafakemalpaşa District, which is the most populated settlement in the modeling reach, so as to move the flood wave away from the District through flat plains towards Uluabat Lake whilst accounting for the additional risks it can pose to the population nearby and the habitat in the vicinity of Uluabat Lake. This approach would further require acquisition of some additional lands that the proposed diversion channel would overlap; hence would have some economic and social impacts on the affected landowners and their dependents even in the absence of dam failure and flooding and hence would require additional mitigative measures against socio-economic impacts such an option would impose. Therefore, that approach would necessitate a comprehensive cost-benefit analysis.

Furthermore, there are already two regulators commissioned along the modeling reach, which are the Mustafakemalpaşa Regulator just before Mustafakemalpaşa District centre and the Çavuşköy Regulator in the vicinity of Sect 8 in Figure 5.12, as previously presented in Figure 5.4. Given the absence of data regarding these two regulators, the regulators were not taken into account during modeling. Hence, the regulators might serve as flood detention basin and help to mitigate the flood, thereby decreasing the observed maximum WSELs and the peak discharges, provided that they withstand the impact of a dam-failure induced flood. Therefore, there is also further need to assess the capacity and structural integrity of these regulators and to reinforce them or build additional regulators along the reach, when and as necessary.

Placement of rip-raps and spurs and dredging the streambed at critical locations along the modeling reach could further be considered as additional measures to regulate the flow after a detailed engineering survey as well as an assessment of the potential socio-economic and environmental impacts of such measures.

Flood Warning Communication Channels

The understanding of the flood warning by the residents of the settlements under risk of the essence in terms of reducing the number of possible fatalities due to a dam-failure induced flood.

This particularly has crucial importance when the rural settlements, where the villagers are usually in fields (sometimes distant to the village centers) till late hours and the means of communication might somewhat be deficient, are considered. Hence, in addition to the broadcast warning messages through media, there would be further need for audio-visual warning means to ensure sound communication of the warning. In this regard, as a first action, the emergency action team might try to contact the village headmen to announce the warning in the village centers. As a second venue, the gendarmerie, security and military forces should immediately be contacted given that they can be deployed and mobilized on short notice. There would be the need for some sort of audio warning, such as a siren placed in the vicinity, in the settlements so as to ensure that the warning is received by all, in case the village headmen cannot be contacted.

Training

The residents of the settlements that have been identified to be under flood risk by using the inundation maps prepared in case the dam fails should be given prior training as to the said audio-visual warnings and their role in implementation of the emergency action plan so as to increase people's awareness in this regard.

CHAPTER 8

CONCLUSION

8 CONCLUSION

Through this thesis work, the potential flow conditions downstream in the event of fictitious failure of the Çınarcık Dam, located in Bursa Province of Turkey, were analyzed under 13 different scenarios using the FLDWAV Software developed by the National Weather Service of the USA.

The variables in the scenarios defined for analyses were mainly related with the parameters for the potential breach formed on the said dam when it fails, and the spillway gate openings. Ten of the scenarios were geared towards an analysis of the overtopping failure of the dam. Two of the three remaining scenarios were to analyze the piping failure of the dam, where the breach parameters were different than those of overtopping scenarios. The last scenario had different Manning roughness values than the foregoing twelve so as to investigate the sensitivity of the analyses to Manning roughness value, as the author, due to lack of measured data, had to use fictitious Manning roughness values for the cross-sections in the reach based on Chow (1959). Outputs sought were the maximum WSELs and the peak discharges at each cross-section station used to define the modeling reach, and the time thereof.

Overtopping failure scenarios yielded more severe flow conditions downstream of Çınarcık Dam as the cross-sectional area of the breach in overtopping failure scenarios was greater than that of in piping failure scenarios. The worst case scenario among the overtopping scenarios was found to be Scenario W3, where all 5 spillway gates were 5 m. open over a total of 10 m spillway gate opening. An inundation was therefore prepared for the subject reach based on the maximum WSEL and water surface width outputs of Scenario W3.

The inundation map was later used to identify the settlements that are under inundation risk as per Scenario W3. The settlements thus identified were than analyzed for potential loss of life in the event of failure of the dam, as per the method suggested by Graham

(2001). It was found that several neighborhoods in Mustafakemalpaşa District, being located on a flat plain, would face risk of inundation if Çınarcık Dam were to fail despite the existence on levees along either side of Mustafakemalpaşa Creek. The existing levees would not be enough to hold off the flood wave produced by the dam failure as they had been designed as run-off flood mitigation constructs. It was further found that again Mustafakemalpaşa District, being a densely populated settlement, would experience the highest number of fatality even though the flood wave would loose its strength when it spread out to flat plain Mustafakemalpaşa District is located on. Fortunately, it would take about 3 hours for the flood wave from Çınarcık Dam to arrive in Mustafakemalpaşa District, which would be ample time to take additional flood mitigation measures in the vicinity of the district and/or evacuate the areas under risk. There were already two regulators located upstream of Mustafakemalpaşa District, which were not accounted for during analyses due to lack of data, and these regulators might further mitigate the incoming flood wave in the actual case, thereby reducing the number of possible fatalities.

There were some villages located close to Çınarcık Dam, where the resulting dam failure-induced flood wave would reach in about 1.3 to 2.6 hours; the closest one being Karacalar Village, and the most distant one Orhaniye Village. In the event of failure of Çınarcık Dam, these villages would require a swiftly conducted evacuation, which would require prior studies and training as to management and implementation thereof, respectively.

The best scenario in terms of managing the high magnitude of inflow (with a peak discharge of about 5200 m^3/s and total duration of 288 hours) whilst preventing the failure of the dam and critical inundation of the settlements downstream, on the other hand, was found to be Scenario W4, where all 5 spillway gates were open 2.5 m over a total spillway gate opening of 10 m. Hence, Scenario W4 would offer the most favorable conditions in the occurrence of such an inflow to the dam, and should be used while preparing a management plan for the dam and the emergency action plans.

As for Scenario S4m, which had the same configuration with Scenario S4 except for the halved Manning roughness values and which was used to identify the sensitivity of the outputs to Manning roughness value; it was found that a 50% reduction in Manning roughness values did not have significant effect on the computed maximum WSELs but on their time of occurrence. For cross-section ID#20 for instance (Table 6.2), a decrease of 50% in Manning roughness values resulted in only some 5% of decrease in maximum WSEL but a 1.53 hours decrease in its time of occurrence, which has significance in terms of arrival time of the flood to the settlement at downstream of the dam. The 50% decrease in Manning roughness values further caused some 50% of increase in the maximum discharge in comparison to the first case, with a 1.63 hours decrease in its time of occurrence. These variations do not result in significant changes in terms of maximum WSELs but time of occurrence thereof and the peak discharges at each cross-section. Therefore, it was deduced that judicial selection of Manning roughness values would not have any significant effect while preparing inundation maps but emergency action plans and evacuation plans. Therefore, in cases where measured Manning roughness data is not available, a certain safety time margin in terms of hours might be provided for the computed flood wave arrival times by repeating the same sensitivity test for a range of possible Manning roughness values.

The analyses indicated that there would be the need to:

- Prepare emergency action plans for the dam.
- Conduct a socio-economic and environmental impact and risk assessment of the dam.
- Conduct training geared towards implementation of the said emergency action plan.
- Reinforce the existing flood protection structures and/or construct additional ones.
- Commission an observation station farther upstream of the dam for real-time feedback to the dam staff.

Recommendations for further research:

- Cross-section geometry measurements for the reaches are not easy to access. Measurements taken during TEFER Project co-implemented by DSİ would be of use in that regard. There would, however, be further need to improved the provided cross-sections by taking measurements on topographic maps of the region, for the cross-sectional data provided might not be of sufficient span-width or maximum elevation required for the analyses. The topographic maps should be procured in color for easier reading, if possible.
- DSİ is conducting run-off flood routing analyses on various rivers in Turkey, using MIKE11 software, which might be used for comparison of outputs and alignment.
- NWS is developing a windows-based version of FLDWAV software with a user-friendly interface, which might offer easier use and be used during similar prospective studies. MIKE11 software might be preferred if FLDWAV's new version is not available.

- Reaches with measured Manning roughness values and/or observed hydrographs at several gaging stations along the subject reach should be preferred for analyses. The observed hydrographs can be used to calibrate the subject reach in terms of Manning roughness values.

BIBLIOGRAPHY

2000 Genel Nüfus Sayımı, Nüfusun Sosyal ve Ekonomik Nitelikleri, İl 16 – Bursa, T.C. Başbakanlık Devlet İstatistik Enstitüsü, Necatibey Cad. No: 114, 06100 Ankara/TÜRKİYE, 2002.

Altinbilek, D., *Hydropower Development in Turkey,* Hydropower&Dams, Vol.9, No.5, pp. 61-65, 2002a.

Altinbilek, D., *The role of Dams in Development, The role of Dams in Development,* Water Resources Development, Vol. 18, No.1, pp. 9-24, 2002b.

Bozkuş, Z., *Baraj Yıkılması Analizlerinde Kullanılan Matematiksel Modellerin Kıyaslanması ve Uygulamalı Örnekler*, DSİ 40.Kuruluş Yılı Su ve Toprak Kaynaklarının Geliştirilmesi Konferansı Bildirileri, Cilt-2, Ankara, 1994.

Bozkuş, Z., *Kestel Barajı Yıkılma Analizleri Nihai Raporu*, Orta Doğu Teknik Üniversitesi, Afet Yönetimi Uygulama ve Araştırma Merkezi, Ankara, Rapor No: METU/DCM 98-02, 1998a.

Bozkuş, Z., *Comparison of Physical and Numerical Dam-Break Simulations*, Tr. J. of Engineering and Environmental Science, Vol.22, pp. 429-443,1998b.

Bozkuş, Z. and Güner, A., *Pre-event Dam Failure Analyses for Emergency Management,*, Tr. J. of Engineering and Environmental Science, Vol.25, pp. 627-641, 2001

Bozkuş, Z., *Pre-event Dam Failure Analyses of Kestel Dam for Disaster Management*, 5th International Congress on Advances in Civil Engineering, Vol. 2, pp. 1141-1150, 2002.

Chow, V. T., *Open Channel Hydraulics*, McGraw-Hill, Inc., 1959.

Davis, A.P., *Guidelines For Assigning Hazard Potential Classifications To Dams,* Proceedings FEMA Workshop, June 26-28, 2001, Oklahoma City, OK, US Department Of Agriculture, 2001.

Engineer Manual 1110-2-1420: Engineering and Design, Hydrologic Engineering Requirements For Reservoirs, Department Of The Army U.S. Army Corps of Engineers Washington, DC 20314-1000, 1997.

Example emergency plan for Blue Marsh Dam and Lake, Research Document 19, U.S. Army Corps of Engineers, Davis, CA, 1983a.

Example plan for evacuation of Reading, Pennsylvania, in the event of emergencies at Blue Marsh Dam and Lake, Research Document 20, U.S. Army Corps of Engineers, Davis, CA, 1983b.

Fiegle II, F.E., *Embankment Dam Failure Analysis,* Proceedings FEMA Workshop, June 26-28, 2001, Oklahoma City, OK, US Department Of Agriculture, 2001.

Fread, D.L. and Lewis, J.M, *NWS FLDWAV MODEL,* Hydrologic Research Laboratory, Office of Hydrology, National Weather Service (NWS), NOAA, Silver Spring, Maryland 20910, 1998.

Graham, W.J., *A Simple Procedure For Estimating Loss Of Life From Dam Failure,* Proceedings FEMA Workshop, June 26-28, 2001, Oklahoma City, OK, US Department Of Agriculture, 2001.

Irwin, B., *Current Practice Natural Resources Conservation Service,* Proceedings FEMA Workshop, June 26-28, 2001, Oklahoma City, OK, US Department Of Agriculture, 2001.

Merzi, N., Usul, N. and Molu, M., *Dam-Break Flood in a Natural Channel: A Case Study,* Tr. J. of Engineering and Environmental Science, Vol.21, pp. 19-27, 1997.

Özesmi, U.*, Conservation of Uluabat Lake Ecosystem,* http://env.erciyes.edu.tr/Uluabat/Uluabat.html#Uluabat%20Gölü%20Hakkinda, 2000. (Last Visited on December 15, 2004)

Ritchey, J.C., *Embankment Dam Failure Analysis, State Assessment Criteria, Issues and Experience Northeastern United States,* Proceedings FEMA Workshop, June 26-28, 2001, Oklahoma City, OK, US Department Of Agriculture, 2001.

Sezer, C., *Barajların Yıkılması ve Tehlike Analizlerinin Önemi,* DSİ Teknik Bülteni, Sayı: 77, 1992.

Temple, D.M. and Hanson, G.J., *Limited Overtopping, Embankment Breach and Discharge,* Proceedings FEMA Workshop, June 26-28, 2001, Oklahoma City, OK, US Department Of Agriculture, 2001.

Wahl, T.L., *Prediction of Embankment Dam Breach Parameters, A Literature Review and Needs Assessment, Dam Safety Research Report, DSO-98-004,* U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office, Water Resources Research Laboratory, 1998.

Wahl, T.L., *The Uncertainty of Embankment Dam Breach Parameter Predictions Based on Dam Failure Case Studies,* Proceedings FEMA Workshop, June 26-28, 2001, Oklahoma City, OK, US Department Of Agriculture, 2001.

OUTPUT DATA FOR SCENARIO W4

The output data given below only includes the essential parts of the original output data given that the original one was extremely long.

OUTPUT FILE

PROGRAM FLDWAV 1.0.0 (DATE: NOVEMBER 28, 1998)

HYDROLOGIC RESEACH LABORATORY
W/OH1 OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

```
*********************************************
*********************************************
***                                       ***
***  SUMMARY OF INPUT DATA                ***
***                                       ***
*********************************************
*********************************************
```

FIRAT BAG - THESIS WORK
e111914
2004
CINARCIK BARAJI

EPSY	THETA	F1	XFACT	DTHYD	DTOUT	METRIC
0.010	1.000	1.000	1000.000	0.000	0.000	1

JN	NU	ITMAX	KWARM	KFLP	NET	ICOND	FUTURE DATA
1	37	40	5	1	0	0	0 0 0

NYQD	KCG	NCG	KPRES
0	0	0	0

NCS	KPL	JNK	KREVRS	NFGRF
7	3	5	0	0

IOBS	KTERM	NP	NPST	NPEND
0	0	0	0	0

TEH	DTHII	DTHPLT	FRDFR	DTEXP	MDT
100.000	0.05000	0.05000	0.05	-0.60000	40

NLEV	DHLV	DTHLV
0	0.00000	0.00000

RIVER NO.	NBT	NPT1	NPT2	EPQJ	COFW	VWIND	WINAGL
1	26	1	26	20.00	0.00	0.00	0.00

RIVER NO.	KU	KD	NQL	NGAGE	NRCM1	NQCM	NSTR	FUTURE DATA
1	2	4	1	5	1	0	0	0 0 0

RIVER NO.	MIXF	MUD	KFTR	KLOS	FUTURE DATA
1	5	0	0	0	0 0 0 0 0 0

LPI COEFFICIENTS WHEN MIXF(J)=5
6

XT(I, 1) I=1,NB(1)

0.000	0.010	1.000	5.000	11.000	15.625	20.500	24.125
28.625	31.375	34.625	36.375	39.250	42.000	44.500	47.000
49.250	49.750	52.250	57.000	59.000	59.925	60.800	63.550
66.300	71.050						

DXM(I, 1) I=1,NB(1)

0.100	0.080	0.080	1.000	1.000	1.000	1.000	1.000
1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
1.000							

KRCH (I, 1) I=1, NRCH

11	5	5	0	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0
0							

RIVER NO. 1, DAM NO. 1

SAR(L, 1, 1) L=1,8

10.14	7.75	5.00	4.03	2.40	1.74	1.18	0.00

HSAR(L, 1, 1) L=1,8

330.00	320.00	300.00	290.00	270.00	260.00	250.00	210.00

LAD	HDD	CLL	CDOD	QTD	ICHAN
1	333.00	325.00	50.00	100.00	0

ICG	HSPD	SPL	CSD	HGTD	CGD
0	0.00	0.00	0.00	0.00	0.00

RHI (L, 1, 1), L=1, 8

316.00	318.00	320.00	322.00	324.00	326.00	328.00	330.00

RQI (L, 1, 1), L=1, 8

0.00	250.00	772.73	1227.30	1636.40	1909.10	2136.40	2454.50

TFH DTHDB HFDD BBD ZBCH YBMIN BREXP CPIP

1.300	0.00000	333.16	70.00	1.25	220.00	1.00	0.00

LQ1 (1, 1)   8

QL (K, 1, 1), K = 1, NU

50.	50.	50.	50.	50.	100.	150.	200.
250.	300.	350.	400.	450.	500.	550.	600.
650.	680.	680.	680.	680.	680.	680.	680.
680.	680.	680.	680.	680.	680.	680.	680.
680.	680.	680.	680.	680.			

PLOTTING/OBSERVED TIME SERIES FOR RIVER J= 1

I	NGS	ID
1	7	KM 20.5
2	8	KM 24.13
3	9	KM 28.63
4	10	KM 31.38
5	11	KM 36.38

ST1 (K, 1), K = 1, NU

85.00	330.00	400.00	515.00	600.00	750.00	920.00	1350.00
1870.00	3000.00	4180.00	5191.80	4850.00	4200.00	3600.00	2975.00
2190.00	1870.00	1625.00	1350.00	1120.00	920.00	805.00	790.00
750.00	635.00	600.00	590.00	515.00	415.00	400.00	375.00
330.00	330.00	330.00	290.00	260.00			

T1 (K, 1), K = 1, NU

0.00	8.00	16.00	24.00	32.00	40.00	48.00	56.00
64.00	72.00	80.00	88.00	96.00	104.00	112.00	120.00
128.00	136.00	144.00	152.00	160.00	168.00	176.00	184.00
192.00	200.00	204.00	208.00	216.00	224.00	232.00	240.00
248.00	256.00	264.00	272.00	280.00			

RIVER NO. 1

I= 1 FLDSTG= 0.00 YDI= 315.00 QDI= 0. AS1= 0.

HS=	210.00	220.00	240.00	260.00	280.00	300.00	333.00
BS=	0.0	58.0	110.0	150.0	200.0	240.0	325.0
BSL=	0.0	0.0	0.0	0.0	0.0	0.0	0.0
BSR=	0.0	0.0	0.0	0.0	0.0	0.0	0.0
BSS=	0.0	0.0	0.0	0.0	0.0	0.0	0.0

I= 2 FLDSTG= 0.00 YDI= 0.00 QDI= 0. AS1= 0.

HS=	210.00	220.00	240.00	260.00	280.00	300.00	333.00
BS=	0.0	58.0	110.0	150.0	200.0	240.0	325.0
BSL=	0.0	0.0	0.0	0.0	0.0	0.0	0.0
BSR=	0.0	0.0	0.0	0.0	0.0	0.0	0.0
BSS=	0.0	0.0	0.0	0.0	0.0	0.0	0.0

I= 3 FLDSTG= 0.00 YDI= 0.00 QDI= 0. AS1= 0.

HS=	206.05	206.50	207.08	213.59	220.00	230.00	250.00
BS=	0.0	11.0	27.0	37.2	47.2	62.9	94.2
BSL=	0.0	0.0	0.0	9.6	13.5	22.7	82.0
BSR=	0.0	0.0	0.0	5.2	8.2	44.4	59.8
BSS=	0.0	0.0	0.0	0.0	0.0	0.0	0.0

```
I=  4   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=       179.42   180.50   184.58   186.80   188.94   200.00   250.00
     BS=          0.0     18.5     44.8     54.0     66.3    118.0    413.0
    BSL=          0.0      0.0      0.0      0.0      0.0      0.0      0.0
    BSR=          0.0      0.0      0.0      0.0      0.0      0.0      0.0
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  5   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=       129.91   132.84   134.14   138.00   141.52   150.00   200.00
     BS=          0.0     11.6     24.8     42.0     56.1    118.0    295.0
    BSL=          0.0      0.0      0.0      0.0      0.0      0.0      0.0
    BSR=          0.0      0.0      0.0      0.0      0.0      0.0      0.0
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  6   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=        94.36    95.50    96.37   100.17   130.82   150.00   200.00
     BS=          0.0     13.8     27.3     66.3     88.2    235.0    531.0
    BSL=          0.0      0.0      0.0      0.0      0.0      0.0      0.0
    BSR=          0.0      0.0      0.0      0.0      0.0      0.0      0.0
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  7   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=        78.17    78.19    78.57    79.85    93.50   100.00   150.00
     BS=          0.0      3.9     18.4     22.4     67.9     89.5    256.1
    BSL=          0.0      0.0      0.0      0.0      0.0      0.0   1784.0
    BSR=          0.0      0.0      0.0      0.0    313.3    314.6   1859.9
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  8   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=        60.56    60.61    61.50    62.15    75.00   100.00   120.00
     BS=          0.0     29.5     36.5     53.3    287.5    743.1   1107.6
    BSL=          0.0      0.0      0.0      0.0     67.3    570.4    642.8
    BSR=          0.0      0.0      0.0      0.0    995.2   1236.5   1549.6
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  9   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=        47.49    47.68    47.78    48.54    50.68   100.00   120.00
     BS=          0.0     14.1     20.2     39.1    120.5    745.6    967.5
    BSL=          0.0      0.0      0.0      0.0      0.0    496.1    524.7
    BSR=          0.0      0.0      0.0      0.0      0.0   1053.3   1243.8
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I= 10   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=        43.08    44.98    47.36    50.53    55.60    60.10   100.00
     BS=          0.0     54.3     63.7     79.9    105.7    128.6    330.7
    BSL=          0.0      0.0      0.0    236.1    302.0    308.3   1399.4
    BSR=          0.0      0.0      0.0      1.6     38.5     38.5    381.9
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I= 11   FLDSTG=   0.00    YDI=   0.00    QDI=       0.   AS1=       0.
     HS=        39.85    40.28    41.00    45.40    48.40    50.00   100.00
     BS=          0.0     28.0    252.0    317.4    362.0    385.7   1128.7
    BSL=          0.0      0.0      0.0    321.3    332.0   2912.1   3540.6
    BSR=          0.0      0.0      0.0    462.7    494.0   1902.1   2030.6
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0
```

```
I=  12    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        34.40    34.61    35.12    39.20    47.82    50.00   100.00
     BS=          0.0     20.1     27.6     49.7     96.3    108.9    378.4
    BSL=          0.0      0.0      0.0     83.3    154.7    366.5   1184.2
    BSR=          0.0      0.0      0.0     44.9    145.4    225.4   1737.4
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  13    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        26.53    28.08    30.70    36.98    44.30    50.00   100.00
     BS=          0.0     24.3     63.8     95.9    133.5    162.5    418.1
    BSL=          0.0      0.0      0.0    355.3    341.1    979.7   2698.9
    BSR=          0.0      0.0      0.0    545.3    966.3   1257.8   1983.0
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  14    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        23.76    24.99    28.60    31.73    45.25    50.00   100.00
     BS=          0.0     50.6     72.3     90.3    168.5    196.0    485.2
    BSL=          0.0      0.0      0.0    570.2    591.6   1199.6   1637.4
    BSR=          0.0      0.0      0.0     20.6    110.6   1304.3   2677.4
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  15    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        21.41    22.86    25.21    26.67    28.00    50.00   100.00
     BS=          0.0     38.1     87.7     94.3    100.4    200.6    428.5
    BSL=          0.0      0.0      0.0     31.7    163.6   1578.8   1771.8
    BSR=          0.0      0.0      0.0    114.8    492.5    520.6   2499.7
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  16    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        19.68    20.12    22.70    26.59    28.08    38.90    50.00
     BS=          0.0     25.8     66.3    112.3    129.9    257.5    389.2
    BSL=          0.0      0.0      0.0    143.5    427.1    425.1   2006.0
    BSR=          0.0      0.0      0.0    382.5    529.8    585.4   1104.8
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  17    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        19.34    19.49    19.54    20.50    25.00    28.53    40.00
     BS=          0.0     47.4     69.9    102.3    107.1    111.1    112.8
    BSL=          0.0      0.0      0.0      0.0      0.0      5.8  10945.2
    BSR=          0.0      0.0      0.0      0.0      0.0      0.0   5242.1
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  18    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        15.72    16.38    19.47    21.52    25.00    30.00    50.00
     BS=          0.0     32.5     72.1     75.3     80.9     88.9    120.9
    BSL=          0.0      0.0      0.0      5.8     24.3     22.2   8956.3
    BSR=          0.0      0.0      0.0     15.3     15.3     49.8   5154.8
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  19    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
     HS=        12.27    12.95    13.50    19.00    20.00    25.00    50.00
     BS=          0.0     32.0     75.5    207.0    228.9    338.3    885.3
    BSL=          0.0      0.0      0.0      0.0    285.1   5728.0   5520.0
    BSR=          0.0      0.0      0.0      0.0    423.5   3933.8   4895.2
    BSS=          0.0      0.0      0.0      0.0      0.0      0.0      0.0

I=  20    FLDSTG=   0.00    YDI=   0.00    QDI=       0.    AS1=       0.
```

```
      HS=      9.26   10.20   13.25   15.00   15.50   25.00   50.00
      BS=       0.0    53.3    90.3   100.0   104.5   189.0   411.7
     BSL=       0.0     0.0     0.0     0.0   204.8  3125.2  3413.1
     BSR=       0.0     0.0     0.0     0.0    47.8    85.8  2375.3
     BSS=       0.0     0.0     0.0     0.0     0.0     0.0     0.0

I=  21   FLDSTG=  0.00   YDI=  0.00   QDI=     0.   AS1=     0.
      HS=      5.93    6.56    7.88   10.23   12.00   15.73   25.00
      BS=       0.0    14.1    42.8    56.6    65.0    77.2   107.5
     BSL=       0.0     0.0     0.0     0.0     0.0   401.7 10926.2
     BSR=       0.0     0.0     0.0     0.0     0.0    82.7   266.3
     BSS=       0.0     0.0     0.0     0.0     0.0     0.0     0.0

I=  22   FLDSTG=  0.00   YDI=  0.00   QDI=     0.   AS1=     0.
      HS=      5.15    5.72    6.50    7.76    8.68   13.29   25.00
      BS=       0.0    18.4    21.5    23.5    52.0   129.7   327.1
     BSL=       0.0     0.0     0.0     0.0     0.0   370.7 10037.6
     BSR=       0.0     0.0     0.0     0.0     0.0    96.3   135.3
     BSS=       0.0     0.0     0.0     0.0     0.0     0.0     0.0

I=  23   FLDSTG=  0.00   YDI=  0.00   QDI=     0.   AS1=     0.
      HS=      4.12    5.24    6.75    8.01   11.00   13.00   25.00
      BS=       0.0    48.8    54.0    56.9    65.0    70.2   101.4
     BSL=       0.0     0.0     0.0     0.0     0.0   410.6 13473.3
     BSR=       0.0     0.0     0.0     0.0     0.0     0.0   825.3
     BSS=       0.0     0.0     0.0     0.0     0.0     0.0     0.0

I=  24   FLDSTG=  0.00   YDI=  0.00   QDI=     0.   AS1=     0.
      HS=      3.88    4.03    5.00    6.14    8.68   10.50   25.00
      BS=       0.0    33.5    36.0    39.3    61.4    73.2   168.3
     BSL=       0.0     0.0     0.0     0.0     0.0    82.9 10906.0
     BSR=       0.0     0.0     0.0     0.0     0.0   496.1  2425.7
     BSS=       0.0     0.0     0.0     0.0     0.0     0.0     0.0

I=  25   FLDSTG=  0.00   YDI=  0.00   QDI=     0.   AS1=     0.
      HS=      3.42    3.60    4.01    5.24    7.87    8.61   25.00
      BS=       0.0    91.4    95.3   106.1   110.9   113.8   177.5
     BSL=       0.0     0.0     0.0     0.0     0.0   268.6 11103.4
     BSR=       0.0     0.0     0.0     0.0     0.0     7.1  2379.1
     BSS=       0.0     0.0     0.0     0.0     0.0     0.0     0.0

I=  26   FLDSTG=  0.00   YDI=  0.00   QDI=     0.   AS1=     0.
      HS=      3.02    3.45    3.85    4.31    5.37    6.05   25.00
      BS=       0.0    99.2   112.8   117.3   122.1   125.9   234.1
     BSL=       0.0     0.0     0.0     0.0     0.0   542.1  1037.3
     BSR=       0.0     0.0     0.0     0.0     0.0    38.4  6658.7
     BSS=       0.0     0.0     0.0     0.0     0.0     0.0     0.0


REACH INFO RIVER NO.  1

     SNM=      1.000   1.000   1.000   1.000   1.000   1.000   1.000
     SNC=      1.000   1.000   1.000   1.000   1.000   1.000   1.000

     SNM=      1.000   1.000   1.000   1.000   1.000   1.000   1.000
     SNC=      1.000   1.000   1.000   1.000   1.000   1.000   1.000
```

SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNM=	1.000	1.000	1.000	1.000	1.000	1.000	1.000
SNC=	1.000	1.000	1.000	1.000	1.000	1.000	1.000

```
   SNM=       1.000    1.000    1.000    1.000    1.000    1.000    1.000
   SNC=       1.000    1.000    1.000    1.000    1.000    1.000    1.000

   SNM=       1.000    1.000    1.000    1.000    1.000    1.000    1.000
   SNC=       1.000    1.000    1.000    1.000    1.000    1.000    1.000

   SNM=       1.000    1.000    1.000    1.000    1.000    1.000    1.000
   SNC=       1.000    1.000    1.000    1.000    1.000    1.000    1.000

   SNM=       1.000    1.000    1.000    1.000    1.000    1.000    1.000
   SNC=       1.000    1.000    1.000    1.000    1.000    1.000    1.000

FKEC(I,1), I = 1, NM(1)
     0.00    0.00    0.00    0.00    0.00    0.00    0.00    0.00
     0.00    0.00    0.00    0.00    0.00    0.00    0.00    0.00
     0.00    0.00    0.00    0.00    0.00    0.00    0.00    0.00
     0.00

NCM(K, 1), K=1,NRCM1( 1)
   1

  CM(K, 1, 1)=     0.0400   0.0400   0.0400   0.0400   0.0400   0.0400   0.0400
  CML(K, 1, 1)=     0.0600   0.0600   0.0600   0.0600   0.0600   0.0600   0.0600
  CMR(K, 1, 1)=     0.0600   0.0600   0.0600   0.0600   0.0600   0.0600   0.0600

    ORHANELI RIVER

METHOD OF ROUTING FOR THIS RIVER SYSTEM:

  RIVER NO.  1
 L= 1    KRTYP= 4    KRT1=  1   KRTN=  2   LEVEL POOL ROUTING
 L= 2    KRTYP= 5    KRT1=  2   KRTN=  4   EXPLICIT DYNAMIC ROUTING
 L= 3    KRTYP= 0    KRT1=  4   KRTN= 26   IMPLICIT DYNAMIC ROUTING


              SUMMARY OF ARRAY SIZES

    NO. OF RIVERS IN THE SYSTEM ...........................                  1
    MAXIMUM NO. OF CROSS SECTIONS ON ANY RIVER ............                121
    NO. OF COMPUTATIONAL TIME STEPS .......................               3077
    MAXIMUM NO. OF GAGING STATIONS ON ANY RIVER ...........                  5
    MAXIMUM NO. OF ROUTING TECHNIQUES IN THE SYSTEM .......                  3
    NO. OF SETS OF POINTS IN THE D/S RATING CURVE TABLE ...                  1
    MAXIMUM NO. OF MANNING N REACHES ON ANY RIVER .........                121
    NO. OF SETS OF POINTS IN THE MANNING N TABLE ..........                  7
    NO. OF SETS OF POINTS IN THE BS VS HSS TABLE ..........                  7
    MAXIMUM NO. OF LATERAL FLOW HYDROGRAPHS ON ANY RIVER ..                  2
    MAXIMUM NO. OF REACHES ON ANY RIVER ...................                 26
    MAXIMUM NO. OF EQUATIONS TO BE SOLVED (K2*2) ..........                242
    MAXIMUM NO. OF INTERNAL BOUNDARIES ON ANY RIVER .......                  1
    TOTAL NO. OF LEVEE REACHES IN THE SYSTEM ..............                  1
    MAXIMUM NO. OF MULTIPLE GATES ON ANY RIVER ............                  1
    NO. OF DAMS WHICH HAVE MULTIPLE GATES .................                  0
    NO. OF POINTS IN THE MOVABLE GATE TIME SERIES .........                  1
    NO. OF INTERPOLATED LEVEE REACHES IN THE SYSTEM .......                  1
    MAXIMUM NO. OF ACTUAL CROSS SECTIONS ON ANY RIVER .....                 26
    TOTAL NO. OF HYDROGRAPH POINTS USED IN FLDGRF PROGRAM .               6154
```

```
*********************************************
*********************************************
***                                       ***
***  SUMMARY OF OUTPUT DATA  ***
***                                       ***
*********************************************
*********************************************
```

RIVER NO	SECT NO	X KM	BED ELEV. M	REACH NO	LENGTH KM	SLOPE %	ROUTING	STRUCT.
1	1	0.000	210.000	1	0.01	0.00	POOL	DAM
1	2	0.010	210.000	2	0.99	0.40	EXP	
1	3	1.000	206.050	3	4.00	0.67	EXP	
1	4	5.000	179.420	4	6.00	0.83	IMP(LPI)	
1	5	11.000	129.910	5	4.63	0.77	IMP(LPI)	
1	6	15.625	94.360	6	4.87	0.33	IMP(LPI)	
1	7	20.500	78.170	7	3.62	0.49	IMP(LPI)	
1	8	24.125	60.560	8	4.50	0.29	IMP(LPI)	
1	9	28.625	47.490	9	2.75	0.16	IMP(LPI)	
1	10	31.375	43.080	10	3.25	0.10	IMP(LPI)	
1	11	34.625	39.850	11	1.75	0.31	IMP(LPI)	
1	12	36.375	34.400	12	2.88	0.27	IMP(LPI)	
1	13	39.250	26.530	13	2.75	0.10	IMP(LPI)	
1	14	42.000	23.760	14	2.50	0.09	IMP(LPI)	
1	15	44.500	21.410	15	2.50	0.07	IMP(LPI)	
1	16	47.000	19.680	16	2.25	0.02	IMP(LPI)	
1	17	49.250	19.340	17	0.50	0.72	IMP(LPI)	
1	18	49.750	15.720	18	2.50	0.14	IMP(LPI)	
1	19	52.250	12.270	19	4.75	0.06	IMP(LPI)	
1	20	57.000	9.260	20	2.00	0.17	IMP(LPI)	
1	21	59.000	5.930	21	0.92	0.08	IMP(LPI)	
1	22	59.925	5.150	22	0.87	0.12	IMP(LPI)	
1	23	60.800	4.120	23	2.75	0.01	IMP(LPI)	
1	24	63.550	3.880	24	2.75	0.02	IMP(LPI)	
1	25	66.300	3.420	25	4.75	0.01	IMP(LPI)	
1	26	71.050	3.020					

INITIAL CONDITIONS FOR RIVER NO. 1

I	DISTANCE KM	FLOW CMS	WSEL M	DEPTH M	MIN WSEL M	BOTTOM M
1	0.000	100.	315.000	105.000	213.716	210.000
2	0.010	100.	213.716	3.716	213.716	210.000
3	0.093	100.	213.359	3.688	213.359	209.671
4	0.175	100.	213.001	3.659	213.001	209.342
5	0.258	100.	212.637	3.625	212.637	209.012
6	0.340	100.	212.263	3.579	212.263	208.683
7	0.423	100.	211.880	3.526	211.880	208.354
8	0.505	100.	211.480	3.455	211.480	208.025
9	0.588	100.	211.060	3.364	211.060	207.696
10	0.670	100.	210.604	3.237	210.604	207.367

11	0.753	100.	210.116	3.078	210.116	207.038
12	0.835	100.	209.634	2.925	209.634	206.708
13	0.918	100.	208.642	2.263	208.642	206.379
14	1.000	100.	208.020	1.970	208.020	206.050
15	1.080	100.	207.509	1.991	207.509	205.517
16	1.160	100.	206.998	2.013	206.998	204.985
17	1.240	100.	206.486	2.034	206.486	204.452
18	1.320	100.	205.975	2.055	205.975	203.920
19	1.400	100.	205.463	2.076	205.463	203.387
20	1.480	100.	204.951	2.097	204.951	202.854
21	1.560	100.	204.439	2.117	204.439	202.322
22	1.640	100.	203.927	2.138	203.927	201.789
23	1.720	100.	203.415	2.159	203.415	201.257
24	1.800	100.	202.903	2.179	202.903	200.724
25	1.880	100.	202.391	2.199	202.391	200.191
26	1.960	100.	201.878	2.219	201.878	199.659
27	2.040	100.	201.363	2.237	201.363	199.126
28	2.120	100.	200.846	2.252	200.846	198.594
29	2.200	100.	200.327	2.266	200.327	198.061
30	2.280	100.	199.808	2.279	199.808	197.528
31	2.360	100.	199.285	2.289	199.285	196.996
32	2.440	100.	198.759	2.296	198.759	196.463
33	2.520	100.	198.232	2.301	198.232	195.931
34	2.600	100.	197.704	2.306	197.704	195.398
35	2.680	100.	197.177	2.311	197.177	194.865
36	2.760	100.	196.649	2.316	196.649	194.333
37	2.840	100.	196.121	2.320	196.121	193.800
38	2.920	100.	195.590	2.323	195.590	193.268
39	3.000	100.	195.059	2.324	195.059	192.735
40	3.080	100.	194.527	2.325	194.527	192.202
41	3.160	100.	193.996	2.326	193.996	191.670
42	3.240	100.	193.464	2.327	193.464	191.137
43	3.320	100.	192.932	2.327	192.932	190.605
44	3.400	100.	192.400	2.328	192.400	190.072
45	3.480	100.	191.869	2.329	191.869	189.539
46	3.560	100.	191.337	2.330	191.337	189.007
47	3.640	100.	190.805	2.331	190.805	188.474
48	3.720	100.	190.274	2.332	190.274	187.942
49	3.800	100.	189.742	2.333	189.742	187.409
50	3.880	100.	189.211	2.334	189.211	186.876
51	3.960	100.	188.679	2.336	188.679	186.344
52	4.040	100.	188.148	2.337	188.148	185.811
53	4.120	100.	187.617	2.338	187.617	185.279
54	4.200	100.	187.084	2.338	187.084	184.746
55	4.280	100.	186.550	2.336	186.550	184.213
56	4.360	100.	186.016	2.335	186.016	183.681
57	4.440	100.	185.482	2.333	185.482	183.148
58	4.520	100.	184.948	2.332	184.948	182.616
59	4.600	100.	184.414	2.331	184.414	182.083
60	4.680	100.	183.881	2.330	183.881	181.550
61	4.760	100.	183.347	2.329	183.347	181.018
62	4.840	100.	182.814	2.329	182.814	180.485
63	4.920	100.	182.281	2.328	182.281	179.953
64	5.000	100.	181.754	2.334	181.754	179.420
65	6.000	100.	173.534	2.366	173.534	171.168
66	7.000	100.	165.786	2.869	165.786	162.917
67	8.000	100.	157.508	2.843	157.508	154.665

68	9.000	100.	149.867	3.453	149.867	146.413
69	10.000	100.	141.512	3.350	141.512	138.162
70	11.000	100.	134.038	4.128	134.038	129.910
71	12.156	100.	124.309	3.286	124.309	121.023
72	13.313	100.	115.541	3.406	115.541	112.135
73	14.469	100.	105.815	2.568	105.815	103.247
74	15.625	100.	97.106	2.746	97.106	94.360
75	16.844	100.	93.188	2.875	93.188	90.313
76	18.063	100.	88.592	2.327	88.592	86.265
77	19.282	100.	84.958	2.740	84.958	82.217
78	20.500	100.	79.859	1.689	79.859	78.170
79	21.709	100.	74.401	2.101	74.401	72.300
80	22.917	100.	67.623	1.193	67.623	66.430
81	24.125	100.	62.413	1.853	62.413	60.560
82	25.250	112.	58.993	1.701	58.993	57.293
83	26.376	125.	56.210	2.185	56.210	54.025
84	27.501	137.	52.527	1.769	52.527	50.758
85	28.626	150.	50.185	2.695	50.185	47.490
86	30.001	150.	48.035	2.750	48.035	45.285
87	31.376	150.	46.109	3.029	46.109	43.080
88	32.459	150.	44.696	2.693	44.696	42.003
89	33.542	150.	43.199	2.273	43.199	40.927
90	34.626	150.	41.157	1.307	41.157	39.850
91	36.376	150.	36.856	2.456	36.856	34.400
92	37.813	150.	33.136	2.671	33.136	30.465
93	39.251	150.	30.182	3.652	30.182	26.530
94	40.626	150.	28.143	2.998	28.143	25.145
95	42.001	150.	26.746	2.986	26.746	23.760
96	43.251	150.	25.788	3.203	25.788	22.585
97	44.501	150.	24.820	3.410	24.820	21.410
98	45.751	150.	23.834	3.289	23.834	20.545
99	47.001	150.	22.855	3.175	22.855	19.680
100	48.126	150.	22.160	2.650	22.160	19.510
101	49.251	150.	20.197	0.857	20.197	19.340
102	49.751	150.	18.554	2.834	18.554	15.720
103	51.001	150.	16.229	2.234	16.229	13.995
104	52.251	150.	15.074	2.804	15.074	12.270
105	53.439	150.	14.430	2.913	14.430	11.518
106	54.626	150.	13.690	2.925	13.690	10.765
107	55.814	150.	12.854	2.842	12.854	10.012
108	57.001	150.	11.701	2.441	11.701	9.260
109	58.001	150.	10.533	2.938	10.533	7.595
110	59.001	150.	9.736	3.806	9.736	5.930
111	59.926	150.	8.476	3.326	8.476	5.150
112	60.801	150.	8.147	4.027	8.147	4.120
113	62.176	150.	7.772	3.772	7.772	4.000
114	63.551	150.	7.110	3.230	7.110	3.880
115	64.926	150.	6.631	2.981	6.631	3.650
116	66.301	150.	6.439	3.019	6.439	3.420
117	67.489	150.	6.320	3.000	6.320	3.320
118	68.676	150.	6.201	2.981	6.201	3.220
119	69.864	150.	6.081	2.961	6.081	3.120
120	71.051	150.	5.976	2.956	5.976	3.020

```
INITIAL CONDITIONS IMPROVED BY SOLVING UNSTEADY FLOW EQUATIONS WITH BOUNDARIES HELD CONSTANT

TT =   0.00000 HRS     DTH =   0.05000 HRS     ITMX=  1

 RIVER=  1    QU(1)=    100.000     YU(1)=     315.00     QU(N)=    155.647     YU(N)=       5.98
      FRMX=   1.138  IFRMX=   13          FRMN=   0.000  IFRMN=    1

  RESERVOIR OUTFLOW INFORMATION
   J   I       TT   QU(I)  USH(MSL)   YB(MSL)  DSH(MSL)      SUB       BB    QU(1)    QBRECH    QOVTOP     QOTHR
   1   1    0.000   100.000    315.00    333.00    213.72      1.00      0.00   100.000     0.000     0.000   100.000

TT =   0.00000 HRS     DTH =   0.05000 HRS     ITMX=  1

 RIVER=  1    QU(1)=    100.000     YU(1)=     315.00     QU(N)=    158.388     YU(N)=       5.98
      FRMX=   1.140  IFRMX=   13          FRMN=   0.000  IFRMN=    1

  RESERVOIR OUTFLOW INFORMATION
   J   I       TT   QU(I)  USH(MSL)   YB(MSL)  DSH(MSL)      SUB       BB    QU(1)    QBRECH    QOVTOP     QOTHR
   1   1    0.000   100.000    315.00    333.00    213.72      1.00      0.00   100.000     0.000     0.000   100.000

TT =   0.00000 HRS     DTH =   0.05000 HRS     ITMX=  1

 RIVER=  1    QU(1)=    100.000     YU(1)=     315.00     QU(N)=    159.722     YU(N)=       5.98
      FRMX=   1.140  IFRMX=   13          FRMN=   0.000  IFRMN=    1

  RESERVOIR OUTFLOW INFORMATION
   J   I       TT   QU(I)  USH(MSL)   YB(MSL)  DSH(MSL)      SUB       BB    QU(1)    QBRECH    QOVTOP     QOTHR
   1   1    0.000   100.000    315.00    333.00    213.72      1.00      0.00   100.000     0.000     0.000   100.000

TT =   0.00000 HRS     DTH =   0.05000 HRS     ITMX=  1

 RIVER=  1    QU(1)=    100.000     YU(1)=     315.00     QU(N)=    160.373     YU(N)=       5.98
      FRMX=   1.140  IFRMX=   13          FRMN=   0.000  IFRMN=    1

  RESERVOIR OUTFLOW INFORMATION
   J   I       TT   QU(I)  USH(MSL)   YB(MSL)  DSH(MSL)      SUB       BB    QU(1)    QBRECH    QOVTOP     QOTHR
   1   1    0.000   100.000    315.00    333.00    213.72      1.00      0.00   100.000     0.000     0.000   100.000

TT =   0.00000 HRS     DTH =   0.05000 HRS     ITMX=  1

 RIVER=  1    QU(1)=    100.000     YU(1)=     315.00     QU(N)=    160.691     YU(N)=       5.98
      FRMX=   1.140  IFRMX=   13          FRMN=   0.000  IFRMN=    1

  RESERVOIR OUTFLOW INFORMATION
   J   I       TT   QU(I)  USH(MSL)   YB(MSL)  DSH(MSL)      SUB       BB    QU(1)    QBRECH    QOVTOP     QOTHR
   1   1    0.000   100.000    315.00    333.00    213.72      1.00      0.00   100.000     0.000     0.000   100.000

TT =   0.05000 HRS     DTH =   0.05000 HRS     ITMX=  1

 RIVER=  1    QU(1)=    100.000     YU(1)=     315.00     QU(N)=    160.846     YU(N)=       5.98
      FRMX=   1.140  IFRMX=   13          FRMN=   0.000  IFRMN=    1

  RESERVOIR OUTFLOW INFORMATION
   J   I       TT   QU(I)  USH(MSL)   YB(MSL)  DSH(MSL)      SUB       BB    QU(1)    QBRECH    QOVTOP     QOTHR
   1   1    0.050   100.000    315.00    333.00    213.72      1.00      0.00   100.000     0.000     0.000   100.000
.
.
.
.
.
.
.
.
TT =  99.74850 HRS     DTH =   0.03250 HRS     ITMX=  1

 RIVER=  1    QU(1)=   4574.949     YU(1)=     233.90     QU(N)=   5784.867     YU(N)=      10.10
      FRMX=   1.790  IFRMX=    7          FRMN=   0.137  IFRMN=   99

  RESERVOIR OUTFLOW INFORMATION
   J   I       TT   QU(I)  USH(MSL)   YB(MSL)  DSH(MSL)      SUB       BB    QU(1)    QBRECH    QOVTOP     QOTHR
   1   1   99.748  4574.949    233.90    220.00    227.69      0.58     70.00  4574.949  4574.949     0.000     0.000
```

```
TT =  99.78100 HRS     DTH =   0.03250 HRS     ITMX=  2

 RIVER=  1    QU(1)=   4572.311    YU(1)=     233.89    QU(N)=   5779.483    YU(N)=      10.09
      FRMX=   1.790  IFRMX=    7        FRMN=   0.137  IFRMN=   99

   RESERVOIR OUTFLOW INFORMATION
    J   I      TT   QU(I)  USH(MSL)  YB(MSL)  DSH(MSL)     SUB      BB    QU(1)    QBRECH    QOVTOP    QOTHR
    1   1    99.781  4572.311    233.89    220.00    227.69     0.58    70.00  4572.311  4572.311     0.000     0.000

TT =  99.81350 HRS     DTH =   0.03250 HRS     ITMX=  1

 RIVER=  1    QU(1)=   4569.673    YU(1)=     233.89    QU(N)=   5774.159    YU(N)=      10.09
      FRMX=   1.791  IFRMX=    7        FRMN=   0.137  IFRMN=   99

   RESERVOIR OUTFLOW INFORMATION
    J   I      TT   QU(I)  USH(MSL)  YB(MSL)  DSH(MSL)     SUB      BB    QU(1)    QBRECH    QOVTOP    QOTHR
    1   1    99.813  4569.673    233.89    220.00    227.68     0.58    70.00  4569.673  4569.673     0.000     0.000

TT =  99.84600 HRS     DTH =   0.03250 HRS     ITMX=  2

 RIVER=  1    QU(1)=   4567.036    YU(1)=     233.89    QU(N)=   5768.896    YU(N)=      10.09
      FRMX=   1.791  IFRMX=    7        FRMN=   0.137  IFRMN=   99

   RESERVOIR OUTFLOW INFORMATION
    J   I      TT   QU(I)  USH(MSL)  YB(MSL)  DSH(MSL)     SUB      BB    QU(1)    QBRECH    QOVTOP    QOTHR
    1   1    99.846  4567.036    233.89    220.00    227.68     0.58    70.00  4567.036  4567.036     0.000     0.000

TT =  99.87850 HRS     DTH =   0.03250 HRS     ITMX=  1

 RIVER=  1    QU(1)=   4564.399    YU(1)=     233.88    QU(N)=   5763.693    YU(N)=      10.09
      FRMX=   1.791  IFRMX=    7        FRMN=   0.137  IFRMN=   99

   RESERVOIR OUTFLOW INFORMATION
    J   I      TT   QU(I)  USH(MSL)  YB(MSL)  DSH(MSL)     SUB      BB    QU(1)    QBRECH    QOVTOP    QOTHR
    1   1    99.879  4564.399    233.88    220.00    227.68     0.58    70.00  4564.399  4564.400     0.000     0.000

TT =  99.91100 HRS     DTH =   0.03250 HRS     ITMX=  2

 RIVER=  1    QU(1)=   4561.764    YU(1)=     233.88    QU(N)=   5758.550    YU(N)=      10.08
      FRMX=   1.791  IFRMX=    7        FRMN=   0.137  IFRMN=   99

   RESERVOIR OUTFLOW INFORMATION
    J   I      TT   QU(I)  USH(MSL)  YB(MSL)  DSH(MSL)     SUB      BB    QU(1)    QBRECH    QOVTOP    QOTHR
    1   1    99.911  4561.764    233.88    220.00    227.67     0.58    70.00  4561.764  4561.765     0.000     0.000

TT =  99.94350 HRS     DTH =   0.03250 HRS     ITMX=  1

 RIVER=  1    QU(1)=   4559.130    YU(1)=     233.87    QU(N)=   5753.465    YU(N)=      10.08
      FRMX=   1.791  IFRMX=    7        FRMN=   0.137  IFRMN=   99

   RESERVOIR OUTFLOW INFORMATION
    J   I      TT   QU(I)  USH(MSL)  YB(MSL)  DSH(MSL)     SUB      BB    QU(1)    QBRECH    QOVTOP    QOTHR
    1   1    99.944  4559.130    233.87    220.00    227.67     0.58    70.00  4559.130  4559.130     0.000     0.000

TT =  99.97601 HRS     DTH =   0.03250 HRS     ITMX=  2

 RIVER=  1    QU(1)=   4556.495    YU(1)=     233.87    QU(N)=   5748.438    YU(N)=      10.08
      FRMX=   1.791  IFRMX=    7        FRMN=   0.137  IFRMN=   99

   RESERVOIR OUTFLOW INFORMATION
    J   I      TT   QU(I)  USH(MSL)  YB(MSL)  DSH(MSL)     SUB      BB    QU(1)    QBRECH    QOVTOP    QOTHR
    1   1    99.976  4556.495    233.87    220.00    227.66     0.58    70.00  4556.495  4556.495     0.000     0.000


     TOTAL INFLOW (1000 CM)       TOTAL OUTFLOW (1000 CM)       TOTAL VOLUME    CONTINUITY ERROR
       RIVER    TRIBUTARIES         RIVER    TRIBUTARIES       CHANGE(1000 CU-M)      (PERCENT)
    992036.38          0.00     799660.25          0.00           191029.45          0.14

TOTAL VOLUME/ACTIVE VOLUME CHANGE (%) OF RIVER 1 =    -19.39     68.83

  TOTAL ITERATIONS FOR EACH OF  1 RIVERS.
     2678

  TOTAL TIME=    100.00  TOTAL NO. OF TIME STEPS: KTIME= 2000  NUMTIM= 2228
```

PROFILE OF CRESTS AND TIMES
* ORIGINAL CRESS-SECTION
\# PEAK STAGE EXCEEDED MAX HS

RVR NO.	SEC NO.	LOCATION KM	BOTTOM METERS	TIME MAX WSEL(HR)	MAX WSEL METERS	TIME MAX FLOW(CMS)	MAX FLOW CMS	MAX VL (M/S)	MAX VC (M/S)	MAX VR (M/S)
1	1*#	0.000	210.00	79.11022	333.20	80.15026	138877.	0.00	6.35	0.00
1	2*	0.010	210.00	80.15026	278.99	80.15026	138877.	0.00	17.65	0.00
1	3	0.093	209.67	80.15026	278.10	80.15026	138848.	8.38	18.04	6.20
1	4	0.175	209.34	80.15026	277.03	80.15026	138891.	8.58	18.55	5.60
1	5	0.258	209.01	80.15026	276.50	80.15026	138993.	8.85	18.83	5.65
1	6	0.340	208.68	80.15026	277.45	80.15026	139130.	8.44	18.39	5.90
1	7	0.423	208.35	80.15026	270.80	80.15026	139097.	10.50	21.87	6.95
1	8	0.505	208.03	80.15026	268.52	80.15026	139070.	10.08	23.23	8.44
1	9	0.588	207.70	80.15026	265.88	80.15026	139046.	10.32	24.85	10.02
1	10	0.670	207.37	80.15026	262.97	80.15026	139022.	10.90	26.74	11.65
1	11	0.753	207.04	80.15026	259.82	80.15026	138999.	11.91	28.85	13.58
1	12	0.835	206.71	80.15026	256.43	80.15026	138977.	13.18	31.19	15.69
1	13	0.918	206.38	80.15026	252.78	80.15026	138955.	14.85	33.69	18.19
1	14*	1.000	206.05	80.15026	249.35	80.15026	138933.	16.48	35.45	20.60
1	15	1.080	205.52	80.15026	249.33	80.15026	138912.	16.22	34.01	20.28
1	16	1.160	204.98	80.18276	249.31	80.15026	138889.	15.95	32.69	19.94
1	17	1.240	204.45	80.18276	249.28	80.15026	138867.	15.68	31.46	19.61
1	18	1.320	203.92	80.18276	249.26	80.15026	138844.	15.41	30.32	19.27
1	19	1.400	203.39	80.18276	249.22	80.18276	138845.	15.14	29.27	18.93
1	20	1.480	202.85	80.18276	249.18	80.18276	138853.	14.87	28.30	18.59
1	21	1.560	202.32	80.18276	249.13	80.18276	138861.	14.59	27.41	18.24
1	22	1.640	201.79	80.18276	249.07	80.18276	138868.	14.33	26.57	17.91
1	23	1.720	201.26	80.18276	249.00	80.18276	138874.	14.07	25.79	17.59
1	24	1.800	200.72	80.18276	248.91	80.18276	138880.	13.83	25.06	17.28
1	25	1.880	200.19	80.18276	248.82	80.18276	138884.	13.61	24.37	17.01
1	26	1.960	199.66	80.18276	248.70	80.18276	138889.	13.38	23.75	16.72
1	27	2.040	199.13	80.18276	248.57	80.18276	138892.	13.15	23.18	16.43
1	28	2.120	198.59	80.18276	248.41	80.18276	138896.	12.94	22.65	16.16
1	29	2.200	198.06	80.18276	248.23	80.18276	138899.	12.75	22.16	15.92
1	30	2.280	197.53	80.18276	248.03	80.18276	138901.	12.59	21.70	15.71
1	31	2.360	197.00	80.18276	247.80	80.18276	138903.	12.44	21.28	15.52
1	32	2.440	196.46	80.18276	247.53	80.18276	138905.	12.25	20.93	15.28
1	33	2.520	195.93	80.18276	247.23	80.18276	138906.	12.13	20.60	15.12
1	34	2.600	195.40	80.18276	246.90	80.18276	138907.	12.04	20.30	14.99
1	35	2.680	194.87	80.18276	246.54	80.18276	138907.	11.95	20.04	14.87
1	36	2.760	194.33	80.18276	246.14	80.18276	138906.	11.89	19.81	14.78
1	37	2.840	193.80	80.18276	245.71	80.18276	138904.	11.83	19.61	14.70
1	38	2.920	193.27	80.18276	245.25	80.18276	138902.	11.75	19.45	14.59
1	39	3.000	192.74	80.18276	244.77	80.18276	138899.	11.80	19.27	14.64
1	40	3.080	192.20	80.18276	244.27	80.18276	138895.	11.76	19.14	14.58
1	41	3.160	191.67	80.18276	243.74	80.18276	138889.	11.71	19.05	14.51
1	42	3.240	191.14	80.18276	243.18	80.18276	138882.	11.69	18.98	14.47
1	43	3.320	190.60	80.18276	242.59	80.18276	138874.	11.65	18.93	14.40
1	44	3.400	190.07	80.18276	241.99	80.18276	138864.	11.66	18.88	14.40
1	45	3.480	189.54	80.18276	241.39	80.18276	138853.	11.71	18.83	14.45
1	46	3.560	189.01	80.18276	240.78	80.18276	138840.	11.66	18.81	14.37
1	47	3.640	188.47	80.18276	240.16	80.18276	138827.	11.67	18.78	14.37
1	48	3.720	187.94	80.18276	239.53	80.18276	138813.	11.69	18.76	14.38
1	49	3.800	187.41	80.18276	238.92	80.18276	138798.	11.71	18.73	14.39
1	50	3.880	186.88	80.18276	238.33	80.18276	138781.	11.84	18.67	14.53
1	51	3.960	186.34	80.21526	237.74	80.18276	138762.	11.81	18.63	14.48
1	52	4.040	185.81	80.21526	237.15	80.18276	138741.	11.85	18.59	14.51
1	53	4.120	185.28	80.21526	236.56	80.18276	138718.	11.84	18.55	14.49
1	54	4.200	184.75	80.21526	235.95	80.18276	138692.	11.84	18.52	14.47
1	55	4.280	184.21	80.21526	235.35	80.18276	138663.	11.84	18.50	14.46
1	56	4.360	183.68	80.21526	234.73	80.18276	138630.	11.82	18.48	14.43
1	57	4.440	183.15	80.21526	234.10	80.18276	138594.	11.81	18.47	14.40
1	58	4.520	182.62	80.21526	233.47	80.18276	138556.	11.79	18.47	14.36
1	59	4.600	182.08	80.21526	232.83	80.18276	138517.	11.76	18.47	14.31
1	60	4.680	181.55	80.21526	232.20	80.21526	138492.	11.82	18.47	14.37
1	61	4.760	181.02	80.21526	231.58	80.21526	138494.	11.85	18.46	14.39
1	62	4.840	180.49	80.21526	230.96	80.21526	138493.	11.89	18.45	14.42
1	63	4.920	179.95	80.21526	230.30	80.21526	138492.	11.93	18.48	14.45
1	64*	5.000	179.42	80.21526	228.95	80.21526	138484.	0.00	19.01	0.00
1	65	6.000	171.17	80.21526	221.14	80.21526	139937.	0.00	19.40	0.00
1	66	7.000	162.92	80.21526	213.25	80.21526	140337.	0.00	19.74	0.00
1	67	8.000	154.67	80.21526	205.48	80.21526	139907.	0.00	19.91	0.00
1	68	9.000	146.41	80.24776	197.99	80.21526	139359.	0.00	19.90	0.00
1	69	10.000	138.16	80.24776	190.84	80.24776	138747.	0.00	19.69	0.00
1	70*	11.000	129.91	80.24776	184.27	80.24776	138482.	0.00	19.22	0.00
1	71	12.156	121.02	80.24776	177.25	80.24776	137989.	0.00	18.44	0.00
1	72	13.313	112.14	80.28027	170.21	80.24776	137288.	0.00	18.26	0.00
1	73	14.469	103.25	80.28027	162.54	80.28027	136666.	0.00	18.90	0.00
1	74*	15.625	94.36	80.28027	153.11	80.28027	136256.	0.00	21.19	0.00

1	75	16.844	90.31	80.31277	142.04	80.28027	135489.	2.71	19.22	13.05
1	76	18.063	86.26	80.31277	131.50	80.31277	134653.	3.39	18.27	11.30
1	77	19.282	82.22	80.34527	121.11	80.31277	133405.	4.09	17.49	10.08
1	78*	20.500	78.17	80.34527	108.53	80.34527	132564.	4.97	19.18	10.89
1	79	21.709	72.30	80.37777	97.69	80.37777	131684.	5.19	11.75	9.04
1	80	22.917	66.43	80.41027	86.93	80.37777	130636.	5.54	12.41	8.91
1	81*	24.125	60.56	80.50777	84.97	80.41027	127408.	3.07	6.69	4.80
1	82	25.250	57.29	80.57278	83.30	80.44277	121766.	3.05	6.20	4.49
1	83	26.376	54.03	80.60528	81.95	80.47527	115550.	3.11	5.88	4.21
1	84	27.501	50.76	80.63778	80.59	80.54028	109930.	3.25	6.12	3.92
1	85*	28.626	47.49	80.63778	78.82	80.57278	105606.	3.79	6.79	3.79
1	86	30.001	45.28	80.67028	76.24	80.63778	103236.	5.19	7.35	3.99
1	87*	31.376	43.08	80.76778	66.90	80.63778	101018.	9.52	16.69	7.10
1	88	32.459	42.00	80.60528	61.23	80.67028	102167.	4.49	9.93	4.62
1	89	33.542	40.93	81.09280	59.26	80.67028	96742.	2.45	5.24	2.63
1	90*	34.626	39.85	81.09280	59.13	80.80029	77115.	1.13	2.17	1.20
1	91*	36.376	34.40	81.06030	56.79	81.12530	72240.	6.33	12.08	5.67
1	92	37.813	30.47	81.41781	52.05	81.15780	70556.	3.55	7.64	4.33
1	93*	39.251	26.53	81.51531	50.74	81.32030	65653.	2.04	4.83	2.56
1	94	40.626	25.15	81.54781	49.13	81.41781	61936.	2.81	6.02	2.44
1	95*	42.001	23.76	81.64532	46.24	81.51531	60278.	4.28	7.47	1.68
1	96	43.251	22.59	81.84032	43.75	81.54781	58637.	3.40	7.67	3.24
1	97*	44.501	21.41	81.87283	42.51	81.64532	56258.	2.15	4.88	3.18
1	98	45.751	20.55	81.90533	41.38	81.80782	54678.	2.28	4.41	3.05
1	99*	47.001	19.68	81.97033	40.07	81.87283	54079.	2.59	4.54	3.05
1	100	48.126	19.51	82.23034	37.23	81.97033	52814.	2.27	8.93	3.12
1	101*	49.251	19.34	82.26284	36.72	82.16534	50258.	0.91	3.77	0.91
1	102*	49.751	15.72	82.36034	35.00	82.23034	49588.	2.87	14.60	3.00
1	103	51.001	14.00	82.16534	27.52	82.32784	49466.	2.50	8.25	2.63
1	104*	52.251	12.27	83.23788	26.60	82.42535	46226.	0.94	2.26	0.95
1	105	53.439	11.52	83.40038	26.28	82.62035	40500.	0.91	2.40	0.92
1	106	54.626	10.76	83.49789	25.93	82.84786	35144.	0.93	2.39	0.94
1	107	55.814	10.01	83.56289	25.41	83.33538	32560.	1.11	2.71	1.14
1	108*	57.001	9.26	83.62789	24.35	83.53039	32061.	1.68	4.31	2.19
1	109	58.001	7.60	83.69289	22.58	83.62789	31930.	2.12	5.48	1.64
1	110*	59.001	5.93	84.01791	20.73	83.75790	31455.	1.42	6.03	1.95
1	111*	59.926	5.15	84.31042	19.72	83.92040	30733.	1.16	3.20	1.94
1	112*	60.801	4.12	84.57043	19.02	84.08291	29873.	1.08	4.45	1.02
1	113	62.176	4.00	84.99294	17.97	84.37542	28358.	1.05	3.65	1.25
1	114*	63.551	3.88	85.41546	17.08	84.66793	26820.	0.97	2.64	1.23
1	115	64.926	3.65	85.90298	16.33	85.02544	25276.	0.94	2.90	1.06
1	116*	66.301	3.42	86.35799	15.71	85.41546	23736.	0.89	2.67	0.86
1	117	67.489	3.32	86.71551	15.25	85.64297	22568.	0.90	2.71	0.86
1	118	68.676	3.22	87.13802	14.81	85.90298	21403.	0.94	2.50	0.86
1	119	69.864	3.12	87.56054	14.39	86.19549	20398.	1.01	2.37	0.86
1	120*	71.051	3.02	87.85305	14.00	86.45550	19558.	1.24	2.18	0.85