# A STUDY ON THE RELIABILITY – BASED SAFETY ANALYSIS OF CONCRETE GRAVITY DAMS

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**ABSTRACT**

A STUDY ON THE RELIABILITY–BASED SAFETY ANALYSIS OF CONCRETE GRAVITY DAMS

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Dams are large hydraulic structures constructed to meet various project demands. Their roles in both environment and the economy of a country are so important that their design and construction should be carried out for negligibly small risk. Conventional design approaches are deterministic, which ignore variations of the governing variables. To offset this limitation, high safety factors are considered that increase the cost of the structure. Reliability–based design approaches are probabilistic in nature since possible sources of uncertainties associated with the variables are identified using statistical information, which are incorporated into the reliability models. Risk analysis with the integration of risk management and risk assessment is a growing trend in dam safety. A computer program, named CADAM, which is based on probabilistic treatment of random loading and resistance terms using Monte–Carlo simulation technique, can be used for the safety analysis of gravity dams. A case study is conducted to illustrate the use of this program.

Keywords: Risk Analysis, Risk Management, Risk Assessment, Dam Safety, CADAM, Monte – Carlo Simulations

# ÖZ

BETON AĞIRLIK BARAJLARINDA GÜVENİLİRLİK ESASLI EMNİYET ANALİZİ ÜZERİNE BİR ÇALIŞMA

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Barajlar, çeşitli proje gereksinimlerini karşılamak için yapılan büyük hidrolik yapılardır. Barajların rolleri hem çevre hem de ülke ekonomisi için çok önemli olduğundan, tasarım ve inşaatları çok küçük riskler kabul edilerek yapılmalıdır. Geleneksel tasarım yaklaşımları, temel değişkenlerin değişimlerini ihmal eder ve sabit oldukları varsayımına dayanır. Bu sınırlama sebebiyle, yapının maliyetini artıran büyük emniyet katsayıları kabul edilir. Güvenilirlik esaslı tasarım yaklaşımlarının doğasında değişkenlik vardır. Değişimlerden kaynaklanan belirsizlikler istatistiksel bilgilerden faydalanılarak bulunur. Baraj güvenliğinin araştırılmasında, risk yönetimi ve risk değerlendirmesiyle birleşen risk analizinin kullanımı yönünde artan bir eğilim bulunmaktadır. Bu tezde, beton ağırlık barajlarının emniyet analizinde, rasgele yükleme ve direnç terimlerinin olası değişimlerini kullanarak, Monte–Carlo benzeşim tekniğiyle güvenilirlik hesaplayan CADAM isimli bir bilgisayar programı kullanılacaktır. Bu programın kullanımı örnek bir uygulama ile gösterilecektir.

Anahtar Kelimeler: Risk Analizi, Risk Yönetimi, Risk Değerlendirmesi, Baraj Güvenliği, CADAM, Monte–Carlo Benzeşimleri

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$\phi$ : Uplift reduction coefficient

$\xi_f$ = Added damping ratio due to dam – foundation rock interaction

$\xi_r$ = Added damping ratio due to dam – water interaction and reservoir bottom absorption

$\tilde{\xi}_1$ = Damping ratio of the dam

$\overline{P}_u$ = Estimated failure probability

$\phi$ = Friction angle (peak value or residual value)

$\tilde{T}_1$ = Fundamental vibration of the dam including the influence of dam foundation rock interaction and of impounded water

$\tilde{T}_r$ = Fundamental vibration of the dam including the influence of impounded water

$\tilde{L}_1$ = Generalized earthquake force coefficient

$\tilde{M}_1$ = Generalized mass

$\overline{x}$ = Moment arm of the net vertical force with respect to toe

$S_a(\tilde{T}_1, \tilde{\xi}_1)$ = Pseudo-acceleration ordinate of the earthquake design spectrum

$\overline{\xi}_1$ = The dam foundation reservoir damping

$\phi$ = the standard normal variable probability density ordinate

$\xi_1$ = Viscous damping ratio for the dam on rigid foundation rock with empty reservoir

$\phi$ = Area of Cumulative Standard Normal Distribution for a specified variable

$\delta$ = Coefficient of variation

$\delta_f$ = Coefficient of variation of failure probability

$\delta_i$ = Initial coefficient of variation

$\rho$ = Correlation coefficient

$\sigma$ = Standard deviation

$\sigma^2$ = Variance

$F_{X_i}(x_i^*)$ = Nonnormal cumulative distribution function

$f_{X_i}(x_i^*)$ = nonnormal probability density function

$\phi(y)$ = Fundamental vibration mode shape

$T_1^r$ = 4H/C

$\Delta x$ = Width of the interval

$\alpha$ = Significance level

1 = First mode

A = Area of the base that normal pressure takes place

A(T) = Spectral acceleration coefficient

$a_1$ = Uniformly distributed random variable

$A_2$ = Area along the rock wedge failure plane

AA = Absolute Acceleration

$A_C$ = Area in compression

accv = Vertical acceleration of the rock

AFOSM: Advanced First Order Second Moment

$a_g$ = Maximum ground acceleration of the design earthquake

ALARP: As Low As Reasonably Practicable

ANCOLD: Australian Commission on Large Dams

$A_o$ = Effective horizontal ground acceleration coefficient

ASCE: American Society of Civil Engineers

ASDSO: Association of State Dam Safety Officials

B = Width of the dam

c = cohesion (apparent or real)

C = Confidence level

C = Constant

c = Crest

c = Distance from centerline to the location where stresses are computed

c and d = Limit values of z

$c_{1s}$, ..., $c_{ks}$ : The respective load effects in different failure modes

Ca = Cohesion

$C_c$ = A correction factor to account for water compressibility

CDF = Cumulative Distribution Function

CDSA: Canadian Dam Safety Association

$C_e$ = Factor depending principally on depth of water and the earthquake vibration period characterizing the frequency content of the applied ground motion

COV = Covariance

CSA: Canadian Standards Association

D = Dead load

d = Downstream

$D_f$ = Failure region

$D_s$ = Survival region

DSI: General Directorate of State Hydraulic Works

e = Eccentricity

EAP: Emergency Action Plan

EPRI: Electric Power Research Institute

$E_s$ = Young's modulus

F = Applied force

f = Flood level

$f_1(y)$ = Equivalent lateral earthquake forces associated with the fundamental vibration mode

$F_b(b_1)$ = Cumulative distribution function of $b_1$

$f_c$ = Compressive strength of concrete

FCCSET: Federal Coordinating Council for Science, Engineering, and Technology

FD = Floating debris

FEMA: Federal Emergency Management Agency

FERC: Federal Energy Regulatory Commission

FOSM: First Order Second Moment

$f_{R,S}(r,s)$ = Joint Density Function

FREQ = Frequency

FREQ NO = Frequency Number

$f_{sc}(y)$ = Lateral forces associated with the higher vibration modes

$f_t$ = Tensile strength of the material

g = acceleration of gravity

g( ) = Limit state function (performance function)

g.p (y, $\tilde{T}_r$) = Hydrodynamic pressure term

H = Depth of the impounded water

h = Horizontal

H = Horizontal hydrostatic force per unit width

h = Total depth of the reservoir

$H_1$ = Reservoir pressure head on the upstream face

$h_1$ = Upstream normal water level

$h_2$ = Downstream normal water level

$H_2$ = Tailwater pressure head on the downstream face

$H_3$ = Pressure head at the line of the drains

$H_d(y)$ = Additional total hydrodynamic horizontal force acting above the depth y for a unit width of the dam

$H_{du}$ = Horizontal hydrodynamic force per unit width induced by earthquake

HPGA = Horizontal peak ground acceleration

$H_s$ = Height of the dam from base to the crest

$h_s$ = Silt level

HSA = Horizontal spectral acceleration

I = Building importance factor

I = Ice load

I = Moment of inertia

ICODS: Interagency Committee on Dam Safety

ICOLD: International Committee on Large Dams

k = Seismic coefficient

$K_a$ = Active earth pressure coefficient according to Rankine theory

$K_\theta$ = Correction factor for the sloping dam faces with angle θ from the vertical

L = Horizontal length from upstream to downstream face

Lc = Crack length

$L_{FR}$ = Location of the force resultant along the joint

M = Masses

M = Sum of moments about the base centerline

m = Upstream slope component

MAG = Magnification Ratio (Response Spectral Shape)

MCE: Maximum Credible Earthquake

MTA: General Directorate of Mineral Research and Exploration

$N$ = Cells

n = Normal level

N = Number of total simulation cycles

n: Negative

N: Number of fatalities

NRC: National Research Council

$N_u$ = Number of simulation cycles where the failure occurs

OBE: Operating Basis Earthquake

OSF = Overturning safety factor

P = Post – tension

p(x) = Probability of Random Variable x

p: Positive

$p$ = A drain reduction factor

$p_1$ = Hydrodynamic pressure associated with fundamental vibration mode

PAA = Pseudo absolute acceleration (horizontal spectral acceleration)

PDF: Probability Density Function

$P_{dh}$ = Horizontal component of the post-tension force

PEER: Pacific Eartquake Engineering Research Center

PER = Period

$P_f$ = Pr (Failure) = Probability of Failure

PMF: Probable Maximum Flood

PMP: Probable Maximum Precipitation

PRV = Pseudo Relative Velocity

$P_s$ = Pr (Survival) = Probability of Survival

$p_{sc}$ = Hydrodynamic pressure associated with higher vibration modes

$p_{st}$ = Initial hydrostatic pressure due to various loads

$P_v$ = Anchor force

q = Dynamic

Q = Earthquake force on the dam body (dam inertia)

$Q_h$ = Horizontal dam inertia

$Q_v$ = Vertical dam inertia

R = Resistance (capacity)

R = Sliding resistance

$r_d$ = Dynamic response

RD = Relative Displacement

$r_{max}$ = total value of response quantity

$R_r$ = Period ratio

RV = Relative Velocity

$R_w$ = Period ratio

s = Higher mode

S = load (demand)

*s* = Safety factor

S = Silt

$S(T)$ = Spectrum coefficient

SDF: Spillway Design Flood

$S_h$ = Force due to sediment accumulation (lateral earth force per unit width)

SRSS = Square-root-of-the-sum-of-squares

SSF = Sliding safety factor

T = Building natural period

$T_1$ = Fundamental vibration period of the dam with an empty reservoir

$T_A$ and $T_B$ = Spectrum characteristic periods

$\tan\phi$ = Friction coefficient

$t_e$ = Period to characterize the seismic acceleration imposed to the dam (s)

U = Uplift

U = Uplift force resultant normal to the inclined joint

U = Uplift pressure force resultant

u = Upstream

$U_n$ = Uplift force per unit width

USACE: US Army Corps of Engineers

USBR: United States Bureau of Reclamation

USCOLD: The United States Committee on Large Dams

USF = Uplifting (Floating) Safety Factor

v = Vertical

V = Vertical hydrostatic force per unit width

VPGA = Vertical peak ground acceleration

W = Saturated weight of the rock wedge

$w_s(y)$ = Weight of the dam per unit height

$x_1^*, x_2^*, \ldots, x_n^*$ = Design points

$X_d$ = Distance to the drain from the upstream face

y = Distance below reservoir surface

$Y_1$ and $Y_2$ = Noncorrelated pair of random variables

$z$ = A continuous random variable

$z$ = Standard normal variate

$Z_1$ = Class of the site classification

$z_i$ = Normalized basic variables

$\alpha$ = Angle with respect to the horizontal of the sliding plane

$\alpha$ = Wave reflection coefficient

$\alpha_i$ = Direction cosine

B= Reliability index

$\gamma$ : Specific weight of water

$\gamma$ = Coefficient of skewness

$\gamma_e$ = Effective volumetric weight of water

$\gamma_s$ : Submerged specific weight of soil

$\delta_h$ = Horizontal displacement

$\delta_v$ = Vertical displacement

$\eta_f$ = Constant hysteretic damping coefficient of the foundation rock

$\theta$ = Angle of repose

$\theta$ = Angle of the face with respect to the vertical

$\lambda$ = Mean value for Log-Normal Distribution

$\lambda_1$ and $\lambda_2$ = Eigenvalues

$\mu$ = Mean value

$\xi_1$ = The dam damping on rigid foundation without reservoir interaction

$\xi^2$ = Variance of Log-Normal Distribution

$\rho_w$ = volumetric mass of water

$\Sigma \overline{V}$ = Sum of vertical static forces excluding uplift pressure

$\sigma$ = Vertical normal base pressure

$\sigma'$ = Effective normal stress

$\sigma_1$ = Maximum principal stresses associated with fundamental vibration mode

$\sigma_n$ = Normal compressive stress

$\sigma_n^*$ = Minimal compressive stress

$\sigma_{sc}$ = Maximum principal stresses associated with higher vibration modes

$\sigma_{st}$ = Initial maximum principal stresses due to various loads

$\sigma_{y,sc}$ = Normal bending stresses associated with higher vibration modes

$\sigma_{y,st}$ = Initial normal stresses due to various loads

$\sigma_{y1}$ = Normal bending stresses associated with fundamental vibration mode

$\sigma_{zu}$ = Minimum allowable compressive (normal) stress at the upstream face

# CHAPTER 1

# INTRODUCTION TO DAM SAFETY AND RISK

## 1.1 Scope of the Study

If there is even a very small chance of harm, then there is risk. People have been living in an environment where there exist various risks. Balancing a variety of risks that may have technological, personnel, and economic aspects, is the ultimate concern of an engineer in reaching the organization's goals. These risks come eventually as an organization decides to conduct a business. With lack of knowledge about risk or when these risks are in conflict with societal concerns, problems occur in risk management. When risk-based design is agreed to be applied to a project, it is known that there is even a probability of failure which is significantly small. However, with classical deterministic approach, very high factor of safety values are assigned and this causes high costs for the project. That is why the aim should be the project optimization with respect to failure probability and project cost. A logical and systematic approach to analyzing various uncertainties involved in design and analysis is provided by the concepts and methodologies of risk-based design procedures.

According to ICOLD (1998), "Risk is the measure of the probability and severity of an adverse effect to life, health, property or the environment". Both the probability of occurrence of an event and the magnitude of the resulting events are involved. The term hazard shows the existence of a threat, whereas risk implies both the existence of a threat and its occurrence potential; so a threat (hazard) may exist with no risk implied. Risk occurs if a treatment pathway exists i.e. there must be some exposure pathway to people or the environment for a threat to be meaningful. In this sense, the level of exposure can be related to the likelihood (probability) of occurrence and the magnitude of consequences of an event (Rowe, 1981).

After over a decade of research by many scientists, the philosophy behind risk-based design of hydraulic structures is beginning to be accepted and applied in real life design practices (Albertson and Kia, 1989). In this thesis, the details of a risk-based design of concrete gravity dams will be given in light of the explanations of the risk components and methods offered by researchers including the guidelines of different countries and organizations. A case study will be introduced by using a recently developed software, CADAM (Leclerc et al., 2004) to assess the safety level of an existing dam. This program can also be used in the reliability-based design of a concrete gravity dam by performing quick successive test runs to account for the effects of various geometric properties and loading possibilities.

### 1.2 Risk Management

Risk management is the systematic application of management policies, procedures and practices to the task of identifying, analyzing, assessing, treating, and monitoring risk (ICOLD, 1999).

A decision must be reached after the risk information is gained and the criteria for risk evaluation are known. Consultations with stakeholders and community, insurance issues, legal defensibility of decisions, risk information to decision-maker and to the public, clarification of the role of decision maker and documentation of the decision and its rational imposing a discipline that is helpful to sound decision making can be good actions to be taken to support the decision process.

Referring to the framework for risk management given in Figure 1.1, the risk management process is divided into two categories: assessing the risk and controlling it. The risk assessment process involves risk analysis consisting of hazard identification and risk estimation; and risk evaluation comparing it with limits of acceptability and tolerance. The risk control process consists of the determination of the risk-related decisions (decision-making) and verifying the validity of the components on which the decisions are based (monitoring). When all these are combined, risk management framework is formed and specifications involving remedial actions may be needed as a result.

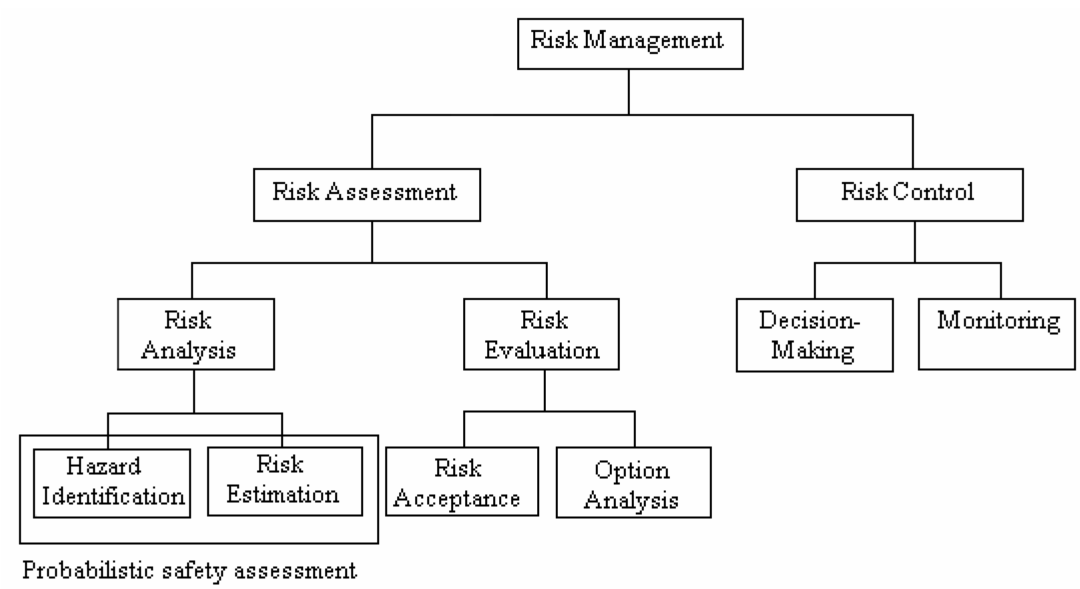


Figure 1.1 Framework for Risk Management (CSA, 1991).

### 1.2.1 Risk Assessment Process

Risk assessment can be useful to determine the types of problems and the corresponding solution approaches. The term “risk assessment” is used to describe the total process of risk analysis, which includes both the determination of levels of risk and social evaluation of risks. Risk determination, in turn, consists of both identifying and estimating the likelihood and magnitude of their occurrence (Rowe, 1981). In risk assessment, it is decided whether the existing risks are tolerable and measures of risk control are adequate and if not, whether measures of alternative risk control are needed for the time being (Figure 1.2).

Risk assessment involves making judgments about the taking of risk and all parties must recognize that the adverse consequences might materialize and owners will be required to deal effectively with the consequences of the failure event (ICOLD, 1999).

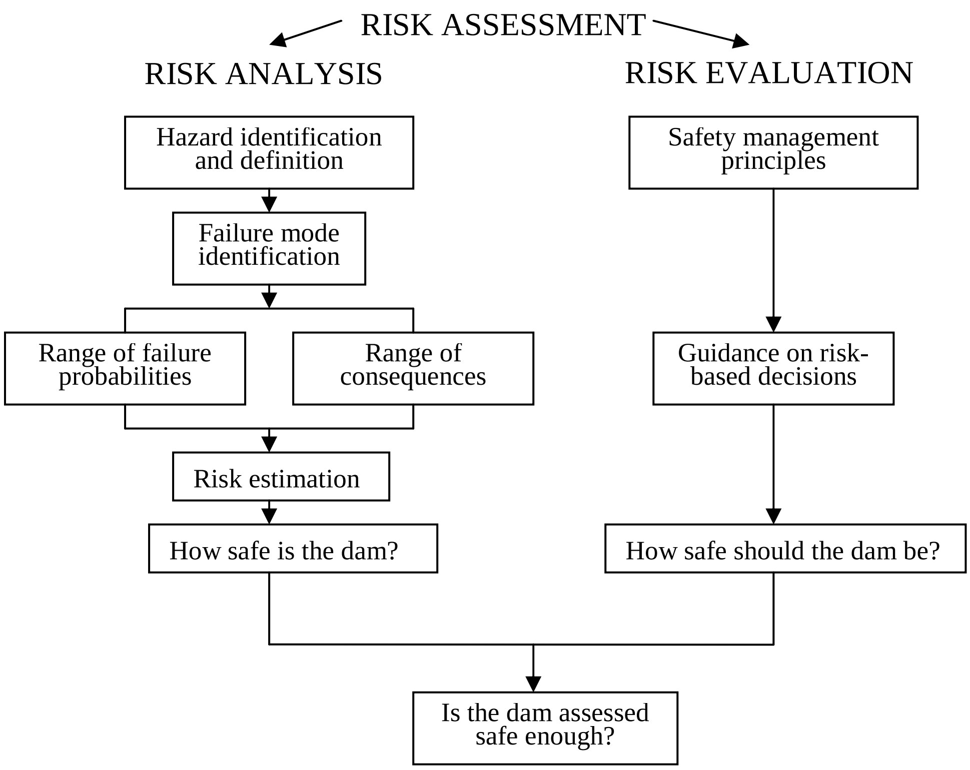


Figure 1.2 Risk Assessment (ICOLD, 1999).

Large differences of risk assessment concepts may be observed in project implementations depending on the social, cultural and institutional habits; that is why the use of risk is either promoted or limited. In the Netherlands, risk concepts are legally adapted and recognized in connection with flood control and dikes. Some other countries (Switzerland, France, Great Britain) stay with conventional procedures for dam safety concepts which are based on well proved standards. Others (Canada, Australia, US, Norway) have some examples of applications on risk basis, but they proceed in a very complex way (Rettemeier et al., 2002).

In a risk assessment process, there are four major steps (Figure 1.3) which are as follows: 1) risk identification, 2) risk estimation, 3) risk evaluation, and 4) risk treatment. In Figure 1.3, the term, risk treatment, refers to the consideration of risk treatment (reduction) alternatives using risk analysis and risk assessment. Implementation of risk treatment is part of risk management (Bowles et al., 1998).

#### 1.2.1.1 Risk Analysis

Risk analysis involves both risk identification and risk estimation (first two rows in Figure 1.3). Risk identification is the process of recognizing the reasonable failure modes if the dam were subjected to each type of initiating event. Failure modes are represented in an event tree, which becomes the risk analysis model.

##### 1.2.1.1.1 Steps in Risk Analysis

The following steps need to be executed (Megill, 1984):

1) Gathering data. One of the initial things wanted to be discovered is how much is not known.
2) Isolation of the key variables. Past experience may have to be a key. Some variables may not be known until sensitivity analyses are run.

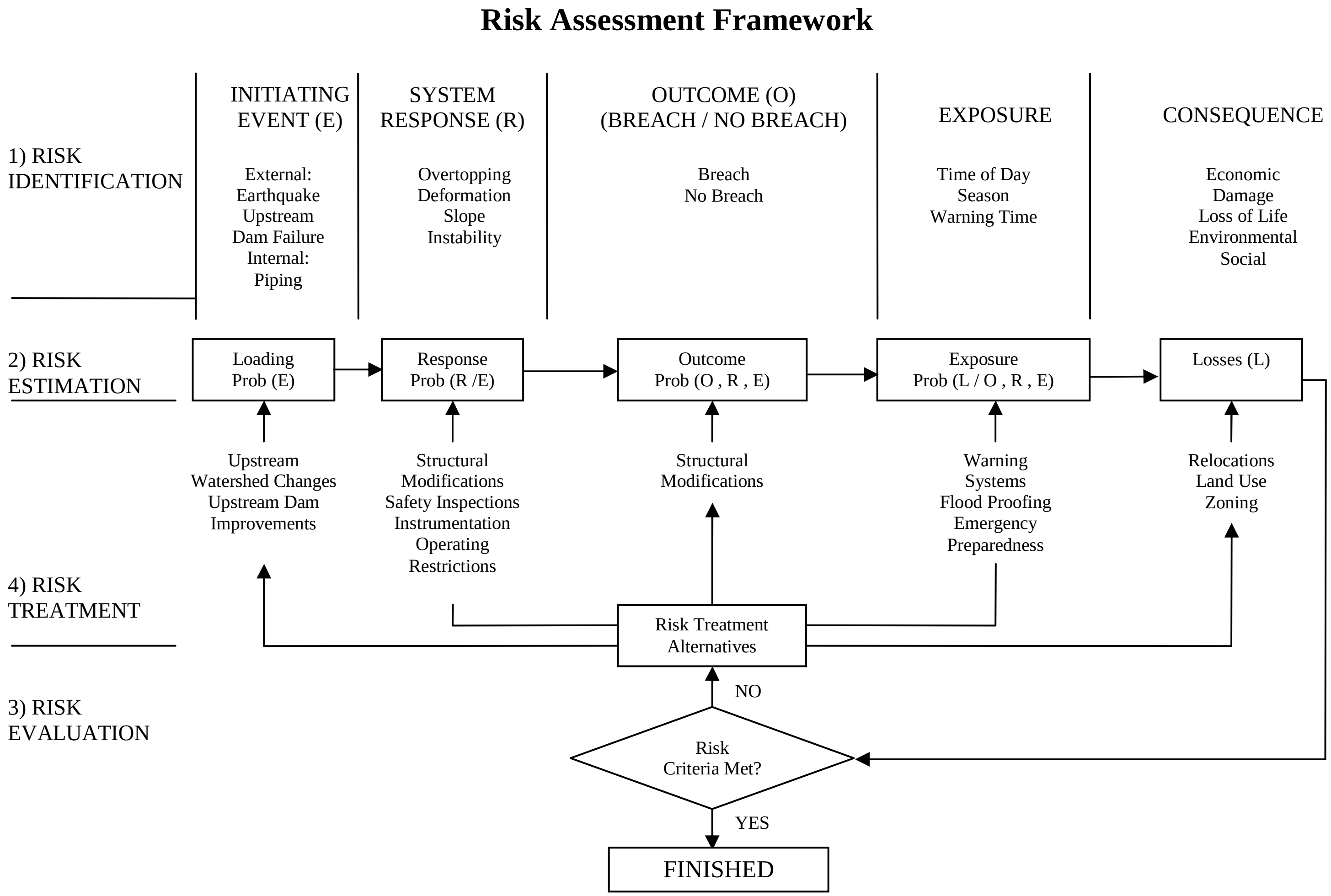


Figure 1.3 Risk Assessment Framework (Bowles et al., 1998).

3) Quantification of the key variables. Triangular distributions are recommended for uncertain variables with many possible answers. For exploratory problems, the important significance of lognormality should be considered.
4) Concepts of uncertainty should be put in at the variable level, not at the final answer level.
5) Entering input into whatever model that is used.
6) Checking the answer for reality; protecting available credibility.
7) Expressing the final answer in the form of a cumulative frequency distribution. This approach will not let one fall victim to the single-answer syndrome. In a very uncertain investment (one with many possible outcomes), the probability of a single value being the answer is near zero.

In the US, a document on the safety of existing dams has been issued by the Committee on the safety of existing dams of the National Research Council (NRC, 1983). It describes the steps involved in the most common format of a risk assessment as:

1. Identification of the events or sequences of events that can lead to dam failure and evaluation of their (relative) likelihood of occurrence.
2. Identification of the potential modes of failure that might result from the adverse initiating events.
3. Evaluation of the likelihood that a particular mode of dam failure will occur given a particular level of loading.
4. Determination of the consequence of failure for each potential failure mode.
5. Calculation of the risk costs, i.e. the summation of expected losses (economic and social) from potential dam failure.

#### 1.2.1.1.2 Uncertainties

"Uncertainty is a general concept that reflects our lack of sureness about something or someone, ranging from just short of complete sureness to an almost complete lack of conviction about an outcome" (NRC, 2000).

There are many different types of uncertainty but the majority of it can be conveniently categorized under two simple headings (Figure 1.4):

- natural variability
- knowledge uncertainty

When carrying out an uncertainty analysis, uncertainties from a variety of different sources will be needed to be combined as a requirement. Depending on the circumstances and specific uncertainties, this procedure can range from a straightforward calculation to more complex computations. Uncertainties may be estimated with reference to past data or formally quantified using experience and judgment.

Uncertainty analysis provides estimates of uncertainty distributions for selected risk analysis outputs, such as probability of failure, probability of life loss, annualized incremental life loss, and risk cost. This is useful compared with using only best estimate inputs in a deterministic approach or sensitivity analyses (Bowles and Chauhan, 2003).

In considering risk and reliability of hydraulic structure design, the first item is to delineate uncertainty and other related terms because of various opinions and connotations among authorized people. The uncertainty of a water resource system is an indeterministic characteristic and is beyond the rigid controls. In design of hydraulic structures, decisions must be made under all kinds of uncertainty.

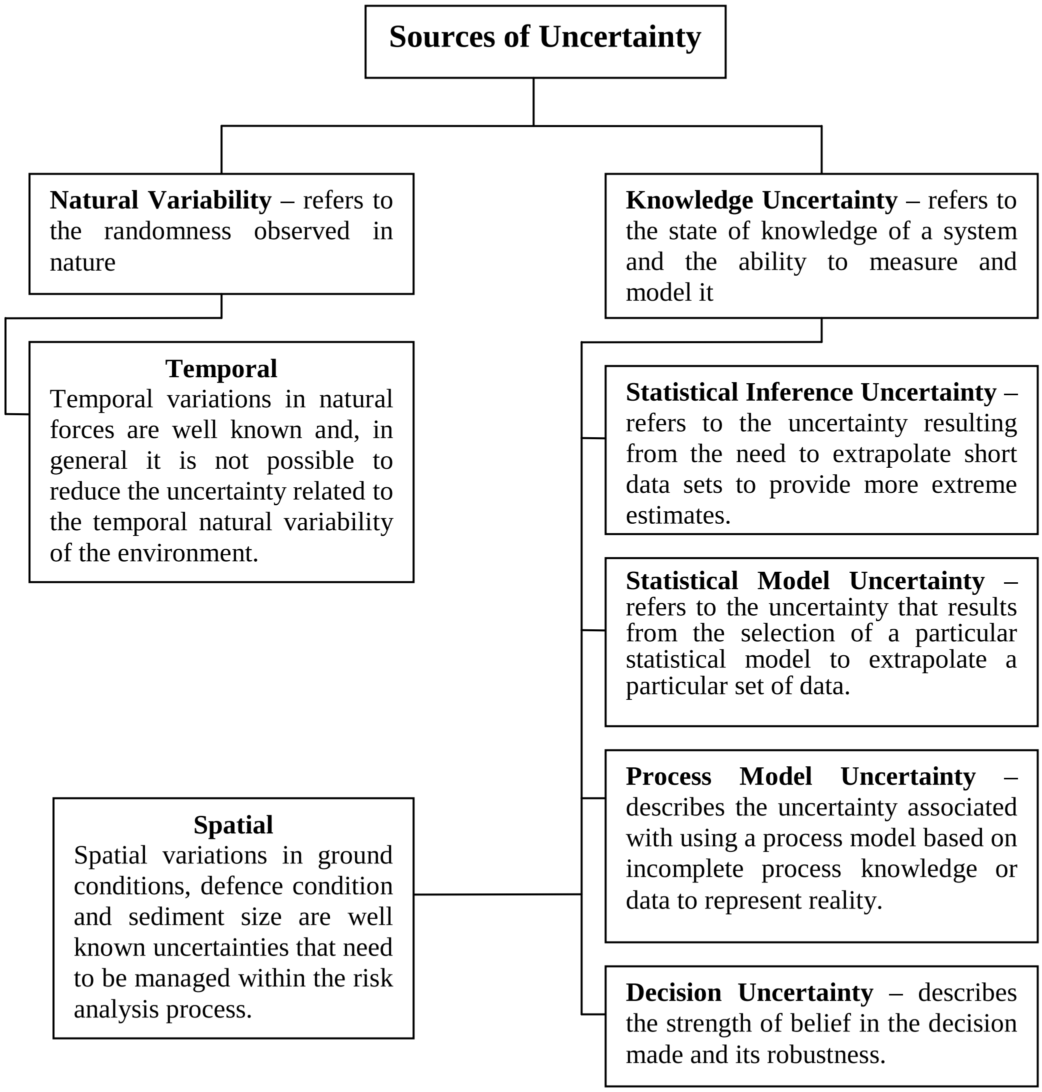


Figure 1.4 Sources of Uncertainty (Morris and Sayers, 2002).

The classification of hydraulic uncertainties for hydraulic structure design may be divided into several types: 1) model; 2) construction and material; and 3) operational conditions of the flow. The model uncertainty results from the use of a certain hydraulic model to describe flow conditions which is essentially an uncertainty in design discharges. The construction and material uncertainties result, apart from the structure size, manufacturer's tolerances or construction tolerances varying widely, from the misalignment of a hydraulic structure, material variability causing variations in the size and distribution of the surface roughness, etc. (Tung and Mays, 1980).

#### 1.2.1.2 Risk Estimation

In risk estimation, loading, system response, final probabilities, and the consequences of various dam failure and no-failure scenarios are determined, so that an estimation of various consequences can be made. These resulting estimates are then applied to the various branches of the event-tree model. Risk reduction alternatives are developed and analyzed in a similar manner for the proposed structure, by changing various inputs (e.g. system response probabilities and consequences) to represent the improved performance of each alternative. The outcome of this step is a calculation of the risk of failure. Fault-trees and event-trees are helpful in risk estimation (Slunga, 2001).

#### 1.2.1.3 Risk Evaluation

If the dam performance is continuously observed, this will help to identify any defect that may cause damage. After observing the performance of existing dams, important information can be obtained about the causes and effects of these deficiencies and what preventative measures are needed to be taken (Yenigün and Yıldız, 2001). Damages observed in large dams throughout the world, as reported in the literature, are shown in Table 1.1. Various studies on the performance of dams have identified the risk factors. These factors are found to be inadequate foundation, inadequate spillway, weak construction, irregular settlement, high vacuum pressure, effects of war, landslides, defective materials, incorrect operations, and earthquakes (Table 1.2).

Table 1.1 Damages Observed in Large Dams throughout the World (Uzel, 1991).

| Year | The number of large dam damages |
|---|---|
| Before 1900 | 38 |
| 1900 – 1909 | 15 |
| 1910 – 1919 | 25 |
| 1920 – 1929 | 33 |
| 1930 – 1939 | 15 |
| 1940 – 1949 | 11 |
| 1950 – 1959 | 30 |
| 1960 – 1965 | 25 |
| Unknown dates | 10 |
| TOTAL | 202 |

Once risks have been identified and quantified for an existing dam, they are evaluated against risk-based criteria. Risk evaluation is the process of examining and judging the significance of risk. The risk evaluation stage is the point at which values (societal, regulatory, legal and owners) and judgments join in the decision process. Consideration of the importance of the estimated risks and the social, environmental and economic consequences should be included, so that identification of alternatives for managing the risks can be done (Bowles et al., 1998).

Table 1.2 The Causes of Failure of Dams (Uzel, 1991).

| Causes of failure | Percentage (%) |
|---|---|
| Foundation problems | 40 |
| Inadequate spillway | 23 |
| Poor construction | 12 |
| Uneven settlement | 10 |
| High pore pressure | 5 |
| Acts of war | 3 |
| Embankment slips | 2 |
| Defective materials | 2 |
| Incorrect operation | 2 |
| Earthquakes | 1 |

#### 1.2.1.3.1 Risk Acceptance

Risk acceptance is an informed decision to accept the likelihood and the consequences of a particular risk (ICOLD, 1999). Risk reduction evaluation joins in the risk acceptance, so that the decision of what residual risk will be accepted for the affected community and structures can be made preliminarily. In some countries, there is a certain risk level that is defined as the limit of unacceptable risk.

Individual Risk can be defined as the total increment of risk that a dam causes. That is why the risks in all failure modes and scenarios need to be combined to obtain the overall risk. ANCOLD (1994) proposed that individual risk is the average individual risk over the population at risk, or as the individual risk for the person at the highest risk which is the value that really matters. The Individual Risk Criteria according to ANCOLD are (ANCOLD, 1998):

- Limit value of average Individual Risk: $10^{-5}$ per annum
- Limit value of Individual Risk for person at the highest risk: $10^{-4}$ per annum
- Objective value of average Individual Risk: $10^{-6}$ per annum
- Objective value of Individual Risk for person at the highest risk: $10^{-5}$ per annum

ANCOLD (1994) states that for existing dams individual risks can be up to 10 times higher than for new dams because of the higher risk reduction costs for existing dams. The ALARP (As Low As Reasonably Practicable) principle is also applied to individual risk evaluations (Slunga, 2001).

Societal Risk is the annual risk of an event that will result in a number of deaths equal to or greater than a given number and the aim is to take account for the aversion of the society to disasters that involve multiple fatalities. According to ANCOLD Guidelines (1994), two features of societal risk criteria are as follows:

- they are concerned only with the number of lives lost, and not with the identities of the people involved, and
- they are event based. Thus each individual dam failure scenario is considered separately in judging whether a dam complies.

For what societal risk is concerned, British Columbia Hydro and the US Bureau of Reclamation tolerate the loss of 0.001 lives per year per dam (Darbre, 1998). Other agencies account for aversion, i.e. few accidents with a large death toll have a greater impact on society than many accidents with a low death toll (the total number of deaths being equal). Figure 1.5 shows the frequency-fatality curves for societal risks for dams.

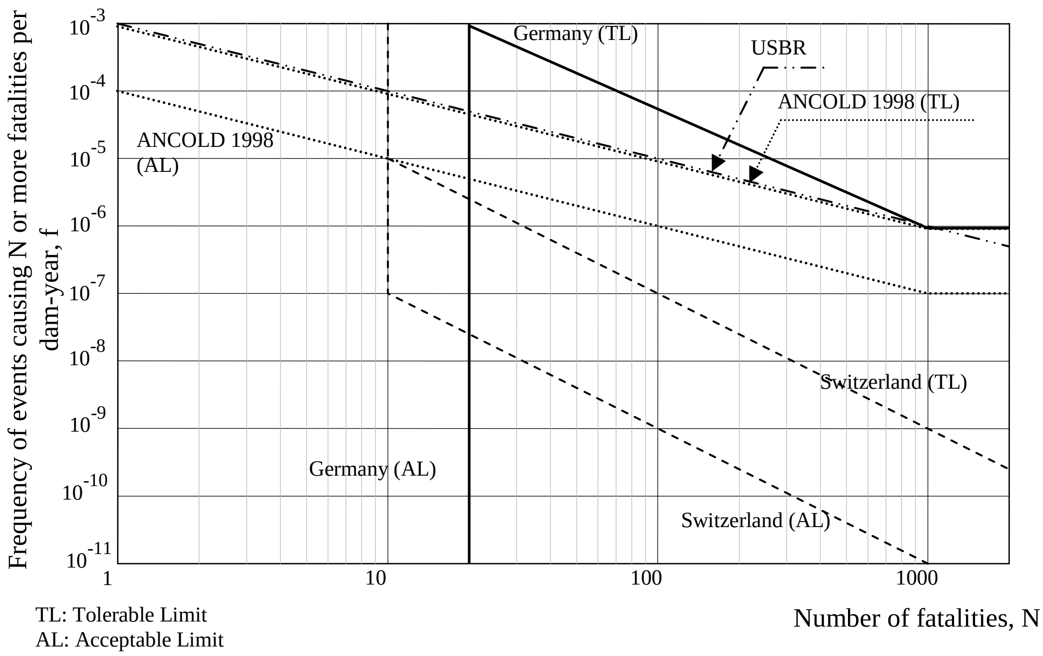


Figure 1.5 Curves of f-N in Different Countries (Darbre, 1999; ANCOLD, 1998; Rettemeier, 2001) (Source: Yanmaz and Günindi, 2004).

One of the biggest difficulties lies in risk acceptance. The economic, environmental, and cultural losses have to be taken into account. However, the loss of life is a very important issue and the acceptance depends not only on society but also on population (Figure 1.6).

Australia and Canada have poorly populated areas where dams might not always impose a risk to the population whereas Germany is densely populated. That is why the loss of life is an issue for almost every dam in Germany.

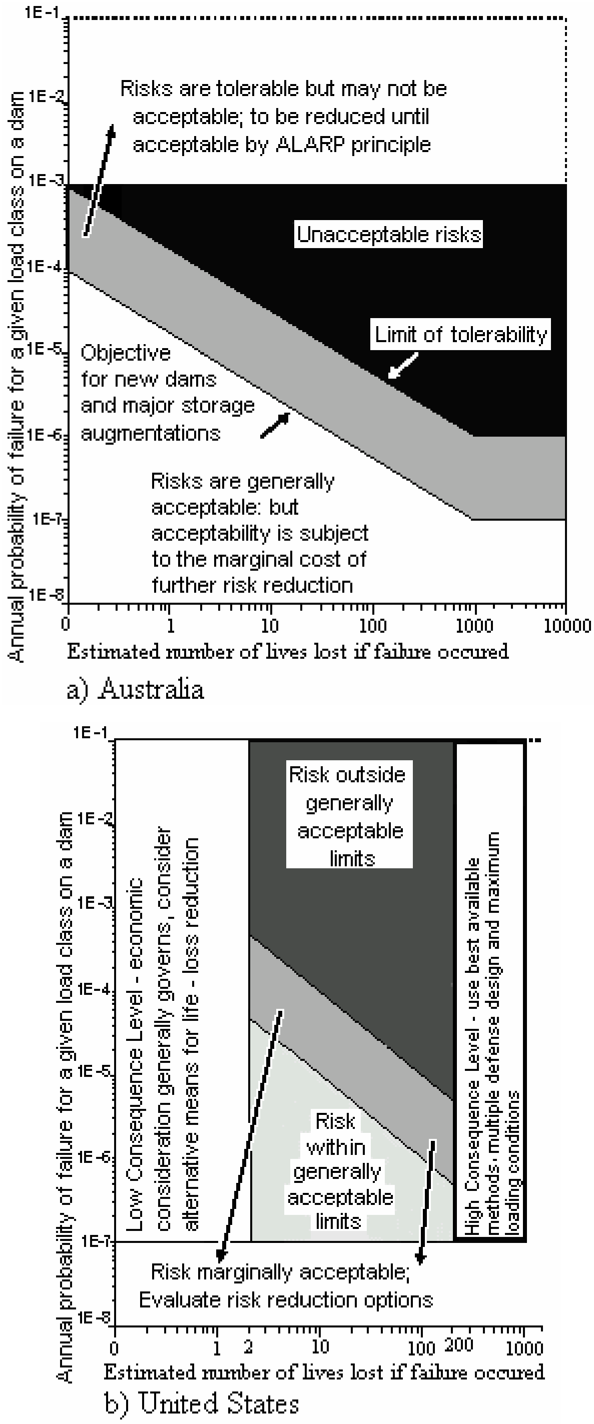


Figure 1.6 Risk Acceptance Criteria (ANCOLD, 1998; Riβler, 1998; and Rettemeier et al., 2002).

#### 1.2.1.4 Risk Treatment

From a business or management perspective, risk treatment options can be grouped into the following categories, although they are “not necessarily mutually exclusive or appropriate in all circumstances” (AS/NZS, 1995):

- Avoid the risk – this is choice which can be made before a dam is built or perhaps through decommissioning an existing dam.
- Reduce (prevent) the probability of occurrence – typically through structural measures or dam safety management activities, such as monitoring and surveillance and periodic inspections.
- Reduce (mitigate) the consequences – for example by effective early warning systems of relocating exposed populations at risk.
- Transfer the risk – for example by contractual arrangements or transfer of an asset.
- Retain (accept) the risk – after risks have been reduced or transferred, residual risks are retained and may require risk financing.

Bowles et al. (1997) state that “Practical dam safety management is intrinsically risk management”. The report, “Whither Civil Engineering?”, from the U.K. Institution of Civil Engineers (1996) states that, “Risk cannot be eliminated; therefore it must be managed”. Risk assessment and risk management can be important improvements to traditional dam engineering approaches. If the goal is to avoid the dam failures and to reduce risk as soon as possible and with optimum cost, then dam safety risk assessment and risk management have a key role in modern dam safety programs.

# CHAPTER 2

# DAM SAFETY CONCEPTS

It is impossible to quantify the overall safety of a dam. However, the approach to achieve maximum dam safety is well understood from viewpoints of design, construction, operation, and maintenance. Therefore, the most important prerequisite for dam safety is the professional competence of people associated with the dam over its life span. Because of the undetected deficiencies, a dam that is observed and stated as safe may still fail. Dam safety must take precedence over all other considerations (NRC, 1985). Therefore, the concepts have been continuously developed through time for better understanding and for more logical approaches to the design of dams.

## 2.1 History of Dam Safety

The starting point of dam safety risk assessment and management can be traced from the available technical procedures and philosophies of dam engineering and risk assessment which have been developed through time and have become a basis for new approaches.

The history of dam safety covers a much shorter time span than the construction of dams. In United States, only a limited number of states had any type of law regulating dam safety prior to 1900. California initiated a dam safety program following the failure of the St. Frances Dam in 1928. Failures of the Buffalo Creek Dam in West Virginia and the Canyon Lake Dam in South Dakota in 1972 contributed to Congress passing "The National Dam Inspection Act" in 1972. Failure of Teton Dam in Idaho in 1976 was followed by "The Reclamation Safety of Dams Act" in 1977. Failure of the Laurel Run Dam in Pennsylvania and the Kelly Barnes Dam in Georgia in 1977 set in action the development of the "Federal Guidelines for Dam Safety" issued in 1979 by the Federal Coordinating Council for Science, Engineering, and Technology (FCCSET). In 1979, President Carter created the Federal Emergency Management Agency (FEMA) and directed federal agencies to adopt and implement the Federal Guidelines for Dam Safety and report their progress to FEMA on a biennial basis. In 1980, the Interagency Committee on Dam Safety (ICODS) was formed to coordinate federal activities and work with the states to ensure implementation of dam safety practices. The Corps of Engineers is the Department of Defense representative on ICODS. In 1984, the Association of State Dam Safety Officials (ASDSO) was organized to provide a forum for the exchange of information and ideas on dam safety and to foster interstate cooperation. Nongovernmental agencies actively dealing with dam safety include the International Commission on Large Dams (ICOLD) and its United States affiliate, the United States Committee on Large Dams (USCOLD) and the Electric Power Research Institute (EPRI) (USACE, 1996).

## 2.2 Safety Concepts

### 2.2.1 Classical Safety Concepts – Development of the Swiss Concept

The safety concepts of large technical systems, such as dams have changed through time. New experiences should be gained and as the first aim is to always provide the best possible protection for the population in case of failure, authorized people should be qualified in their jobs. 1980 is selected as reference year for Switzerland, as the Federal Department of Transport, Communications and Energy is chosen to be in charge of the supervision of dam safety instead of the Federal Department of Home Affairs (Biedermann, 1997).

The dam and its environment are subjected to external and internal solicitations (actions). These solicitations are the loads commonly considered in a structural analysis. In hydraulics context, these are the water flowing into the reservoir. The dam reacts to them (reactions). This is often expressed in terms of deformation and stress (mechanics) or outflow (hydraulics). The environment largely gets affected because of the reactions of the dam (consequences). These consequences can be beneficial or adverse (Darbre, 1998).

The ultimate goal of the concept is to keep the possible adverse consequences caused by the operation of a dam to a level that is as low as reasonably feasible. However, risk management is not mentioned in the concept, although it provides the ideal framework under which to reach this goal.

#### 2.2.1.1 Basic principles

As basic principles, it is recalled that

- only those dams, which endanger lives in case of failure, are subjected to the relevant regulations, as will continue to be the case in the future, and that, consequently,
- the same safety requirements apply to all jurisdictional dams, i.e. no classes of risk are considered

and this is based on all people’s being entitled to the same level of protection against a potential hazard and to the same level of emergency preparedness, independently of the size of the reservoir (Biedermann, 1997).

#### 2.2.1.2 Swiss Safety Concept Tenets

Structural safety and monitoring were the two basic tenets of dam safety before the reference year 1980. This has become legal in 1985. In this year, a 3rd tenet is brought by the revision of the executive decree, the emergency concept (Figure 2.1). This came from the reflection that, aside from the optimal minimization of the risk (requirement 1), the possible remaining risk must also be considered (requirement 2 in Figure 2.1).

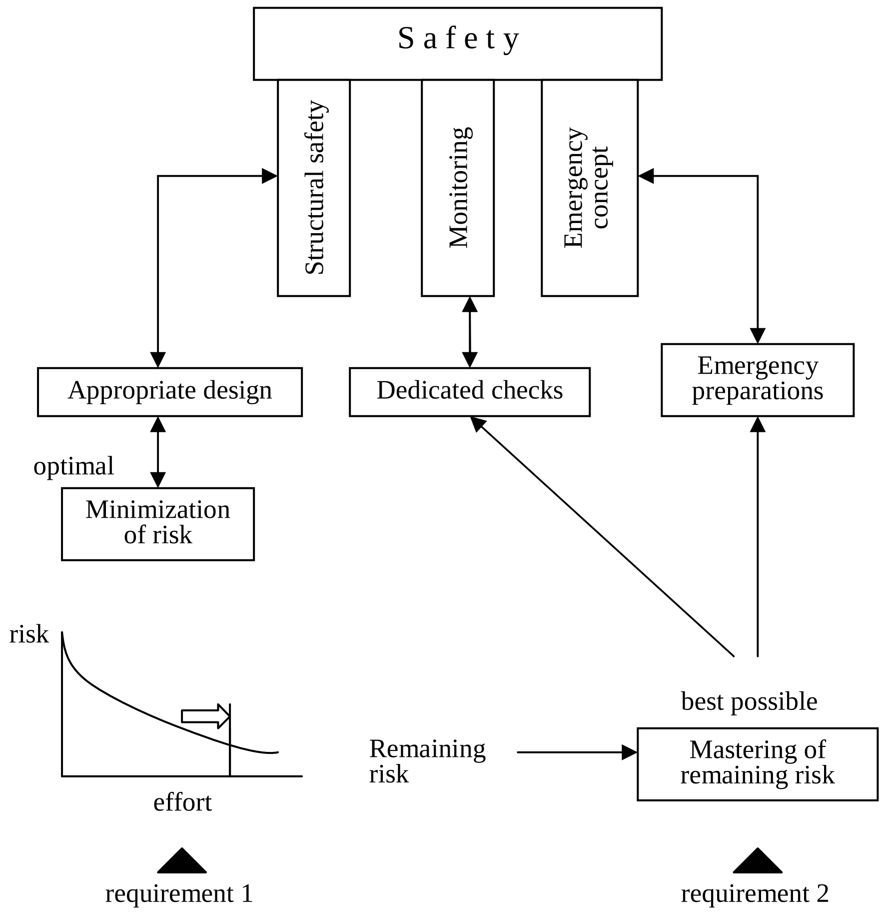


Figure 2.1 Swiss Safety Concept (Biedermann, 1997).

Not directly explained in the Swiss safety concept and added in the Figure 2.2 is the aspect of maintenance that is closely related to monitoring. Its purpose is to ensure proper functioning of the dam components and of its monitoring system.

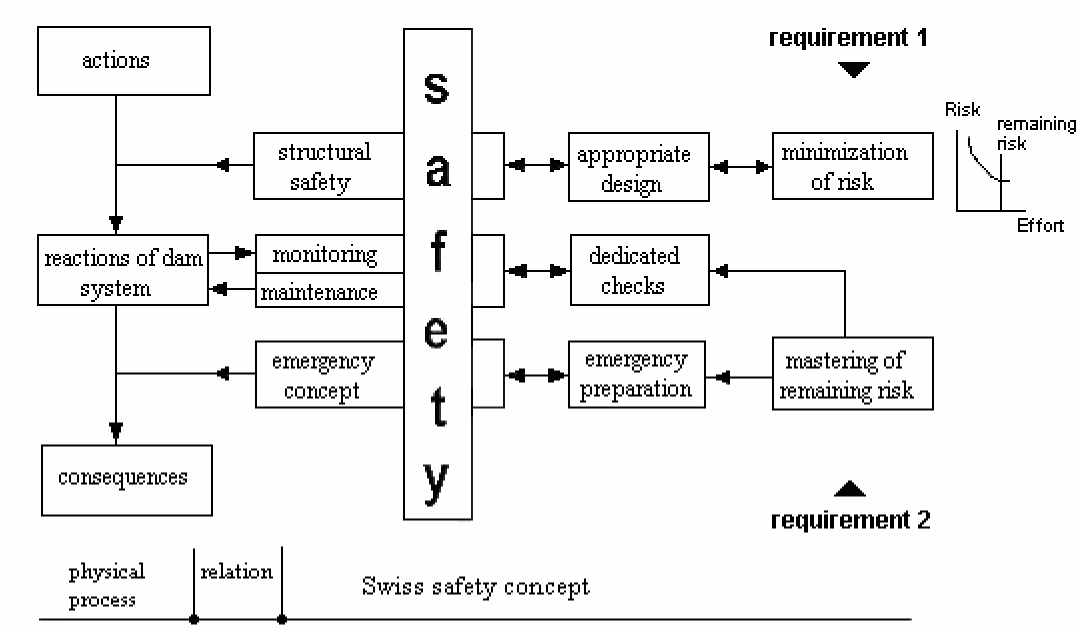


Figure 2.2 Safety Assessment for Dams: Physical Process and Swiss Concept (Darbre, 1998).

#### 2.2.1.2.1 Structural Safety

The optimal minimization of the risk (requirement 1) calls for an appropriate design of the dam, and this is for all possible loading and operational conditions, according to the most recent state of knowledge (what may require rehabilitation measures), as well as considering the protection measures that can be mobilized in the case of an emergency. For the situations of the threats of flood, earthquake, sabotage and act of war, the only preventive measure is the evacuation. Consequently, the design must be based on the largest possible events at the site when it comes to the natural hazards of flood and earthquake.

In case of floods, no critical damages should occur. That is why the requirement that the water level does not rise above the danger water level for the largest possible flood ($HQ_{max}$), i.e. that it does not rise above the level at which scour holes or erosion jeopardizing the overall stability can initiate should be satisfied. For concrete and gate-structure dams, this level is higher than the crest (or top of the parapet when one that can resist the water pressure is in place). For embankment dams, it is the crest level or some lower level if dangerous seepage flow can initiate in the crest area (Figure 2.3, bottom).

The gate mechanisms, the emergency power unit and the water level gage must remain operational and accessible because the operating rules in case of flooding anticipate a progressive opening of the outlets. It is thus also required that the crest be not overtopped up to the 1000-year flood ($HQ_{1000}$) (Figure 2.3, top).

At the design stage, it is assumed that the reservoir is filled at its normal water level. Non-critical damages are accepted so wind-induced waves can be neglected. For embankment dams, it must finally be assumed that the most powerful outlet with gates or valves cannot be opened. For concrete and gate-structure dams, this statement applies only to the 1000-year flood (Biedermann, 1997).

The dam should be designed such that it resists the largest possible earthquake, also with full reservoir, and that too much water does not escape. This has to be proved numerically, which is difficult. The necessity for a second criterion, like for floods, is still open for the moment, because it is presently not possible to reliably predict if, for example, the bottom outlet can still be operated after a 1000-year earthquake, condition that should be satisfied (Biedermann, 1997).

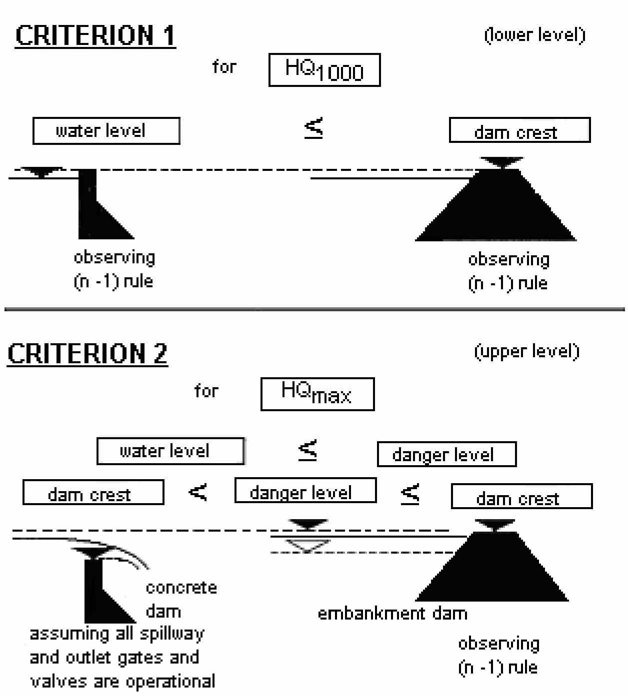


Figure 2.3 Design Criteria for Flood (Biedermann, 1997).

#### 2.2.1.2.2 Surveillance

Surveillance is made to recognize a deficiency in structural performance or an external threat as soon as possible, so that there will be sufficient time to take necessary measures to overcome the danger. At this end, regular checks of the condition and of the behavior of the dam as well as periodical safety evaluations (Figure 2.4) are needed. To obtain a complete assessment of the condition and of the behavior, visual checks, measurements, and operating tests of gates and valves (as well as of emergency power unit) are needed.

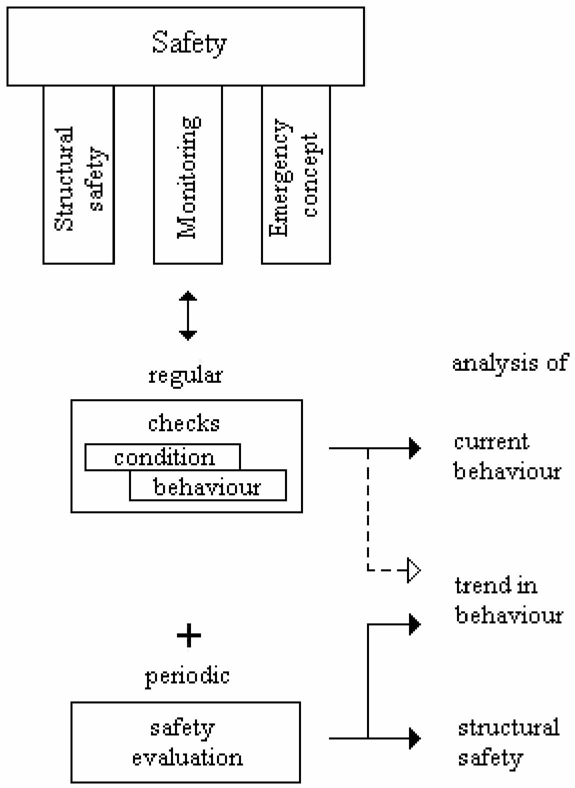


Figure 2.4 Elements of Surveillance and Objectives (Biedermann, 1997).

#### 2.2.1.2.3 Emergency Concept

As mentioned earlier, the emergency concept was introduced in 1985 in connection with the revision of the executive decree as the third tenet of the safety concept. Its purpose is to take all the preparatory measures that are needed to act as well as possible when a threat to safety is recognized (Figure 2.5).

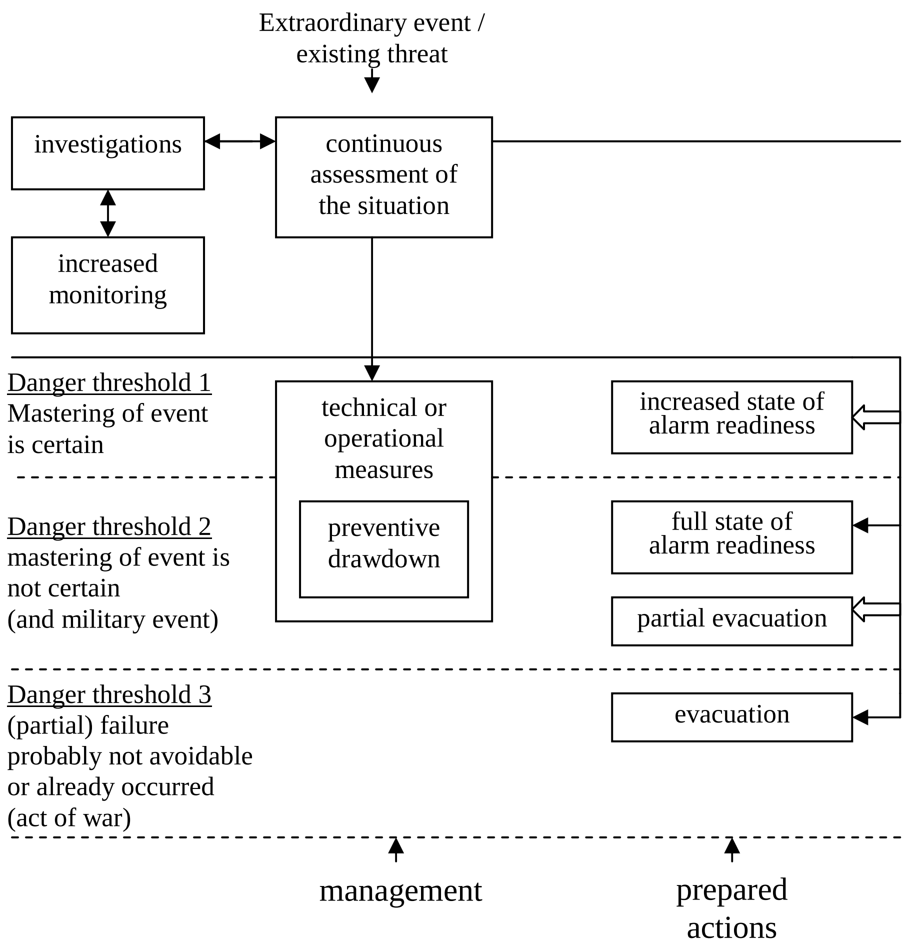


Figure 2.5 Emergency Strategy for Dams (Biedermann, 1997).

### 2.2.2 New Trends in Safety Concepts

Now, the international trend is to integrate risk management in safety concepts for dams. However, the application of this concept to dams is not yet of standard practice. It is rather a topic for research and development. Indeed, current risk management techniques have been developed mostly with reference to technical systems, such as nuclear and aerospace which differ from dams to the extent that the description of natural elements are more difficult.

#### 2.2.2.1 Risk Analysis

Risk analysis is interpreted in Figure 2.6 referring to the physical process in Figure 2.2. The actions have a certain probability of occurrence (inflow of water, water level in reservoir, temperatures, earthquake, etc) but some may also not be identified at all or not be considered (e.g. aircraft impact). The actions are introduced as inputs to uncertain models, leading to a probability distribution of reaction values (deformations, stresses, outflow of water, etc).

The impact of these reactions on the environment, i.e. the consequences, is obtained (energy production or interruption, flood control or flooding and happenings after damages and deaths, etc). The corresponding modeling is also uncertain. These consequences, expressed as a function of their probability of occurrence, are the risk components. Often, the risk analysis will be considered to be complete at this stage. However, the risk components of different types can be further evaluated in a last step, integrating sociological and political aspects. The result of this weighted integration is the risk. It is compared to acceptability and tolerance limits and as an outcome, the acceptance of the state of the dam or the specification of remedial measures is stated.

In the standard risk analysis terminology, the term hazard is used to identify a condition with the potential for causing an undesirable consequence. This applies to all actions as they contribute to this potential. In practice, hazard assessment usually refers to the assessment of a specific action in probabilistic terms, usually one that shows a strong randomness such as earthquake and flood (Darbre, 1998).

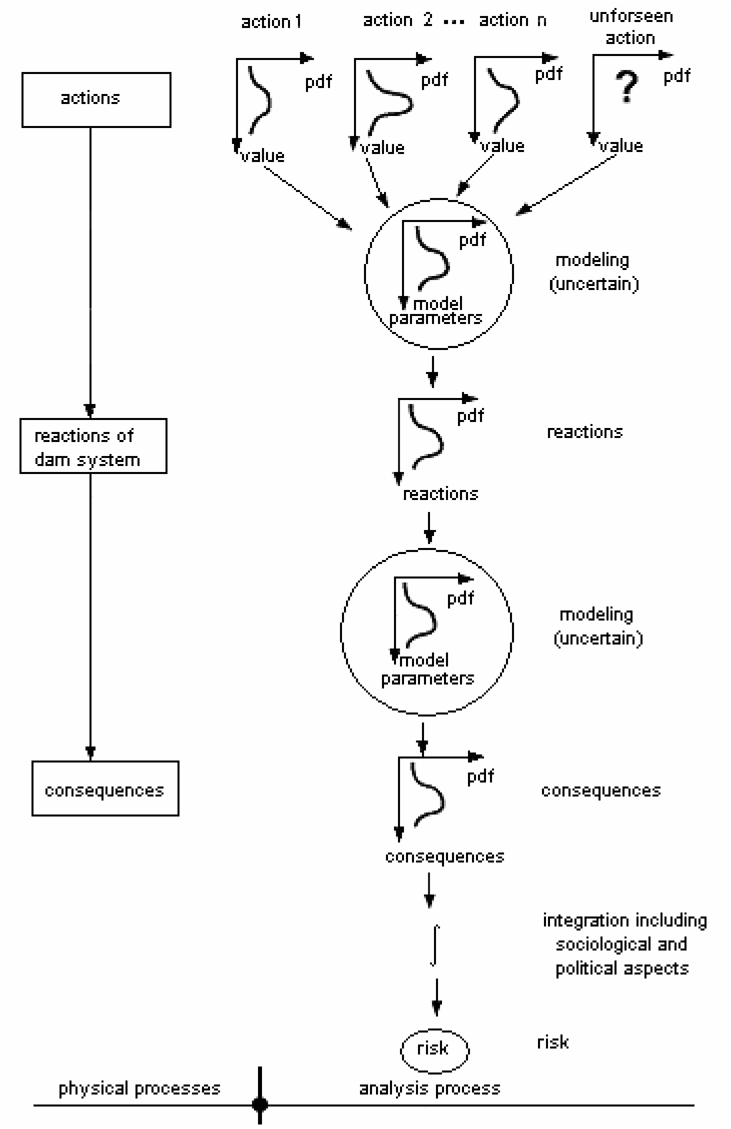


Figure 2.6 Physical and Analysis Processes for Dams (Darbre, 1998).

#### 2.2.2.1.1 Dam Safety for Spillway Design Floods

High impacts of dam failures cause restlessness on public and as a result, dam safety policies are extremely conservative. Policies for selection of the spillway design flood (i.e. policies for protection against overtopping failure) are generated accordingly. For most important dams, the spillway is designed to exceed the probable maximum flood (PMF), essentially a “no risk” policy. Efficient allocation of funds for spillway requires a cost-benefit approach that utilizes risk analysis, a much better approach than a “no risk” policy (Dubler and Grigg, 1996).

There are generally prescriptive and risk-based methods for selecting the spillway design flood (SDF). Prescriptive methods are usually stated by the authorized agency. Risk-based methods seek to minimize the total project cost, including the expected cost of damages due to failure. As risk analysis is a cost-benefit approach, application of risk analysis usually involves quantifying the benefits of saving lives which has been a major point of claim (Dubler and Grigg, 1996).

Dams are designed to resist destruction from natural forces. Two major items of design consideration are the SDF and the maximum credible earthquake (MCE). The spillway is designed to safely transmit the flood flows so that overtopping failure of the dam is prevented. The cost of the spillway may be a significant portion of the cost of the dam. As the SDF approaches the PMF, the probability of overtopping failure approaches to zero but the cost of the spillway reaches its maximum. In fact, selecting the SDF as PMF may be the appropriate choice in many instances. However, the choice of the PMF as a “no risk” criterion ignores the economic facts of life.

#### 2.2.2.2 Deterministic Analysis

The deterministic approach is used in traditional practice. Deterministic design values are selected as actions. Even when a specific return period, and hence a probability of occurrence is referenced (this is the case for flood and for earthquake), the actions are still introduced in deterministic terms (design values) and not as probability distributions. They are structured in load combinations. With this approach, very improbable occurrences are avoided and very likely ones are given more weight. The reactions corresponding to the actions specified previously are obtained (one set of reactions for each load combination) and for uncertainties, no action is taken in the models used in this step of analysis (Darbre, 1998).

The step leading to the consequences is usually made independently of the previous one. Formal considerations of the calculated reactions are not taken into account. Failure mechanisms are postulated as reactions and damages evaluated considering flood propagation and damage models. The risk is thus not calculated.

### 2.2.3 Probabilistic versus Deterministic Approaches

Dam safety community, still, could not reach to a common decision about using risk-based approaches. For many people, in the application of risk, it implies that the design would be made to accept failure and loss of life, or that risk assessment is a way of avoiding making expensive structural repairs to a dam. In addition, many think that using risk entails quantitative risk assessment, a highly complex and time-consuming analysis. Conversely, many dam safety professionals believe that using deterministic standards results in zero risk to the public. Unfortunately, this viewpoint is based on misconceptions in the engineering community about the Probable Maximum Precipitation (PMP) and the Maximum Credible Earthquake (MCE). In reality, these values are estimates of the theoretical maxima that commonly approach, rather than meet, the theoretical upper limits (Johnson, 2000).

After the appearance of risk analysis in structural safety, the community was widely curious about its relation to deterministic approaches. New risk-based approaches for dam safety make the standards and codes better by adding the lacking information for design and operation. These two should be properly addressed in the latter. Reliability indices comply with such a uniform format that uncertainty can be quantified without weakening the request for uniformity by these indices. However, an event tree of a risk analysis cannot. On the other hand, reliability indices do not provide any protection against subjectivity of decisions or against human errors. That is why whenever non-quantifiable factors enter a safety assessment, risk analysis is a better approximation for more realistic results (Kreuzer, 2000).

The factor of safety in traditional design standards provides a confidence level that is widely accepted. However, uncertainty in the factors of safety is ignored which makes the design inconsistent. A partnership between the strength of safety standards and of risk analysis is a reasonable objective, e.g. finite element analysis and risk analysis for confirming structural safety of aging effects. The application of risk analysis can then be seen as a logical extension from deterministic approaches where the stochastic input is restricted to data-randomness of geology, hydrology, and material testing. It is a matter of undertaking the next step to integrate these uncertainties into a coherent probabilistic concept.

Risk analysis fundamentally differs from traditional deterministic approaches for the following reasons when dam safety is the concern (Kreuzer, 2000):

- “It replaces a deductive with a more inductive approach to study safety. Deterministic safety analysis is a deductive process of inferring safety from an analytical framework. By contrast, risk assessment makes inductive inference from observing a part to achieve insight into the whole.

- Risk analysis addresses decision making under uncertainty, which is fully integrated in the process of a risk analysis, as compared to deterministic safety evaluations, where it is attached to the main analysis, e.g. as a number of sensitivity cases or parametric studies.

- Risk analysis replaces a limit-state analysis, leading to a deterministic safety statement, by a sequential path-to-failure process leading to a probabilistic term.

- In its result, risk analysis replaces fixed, single-value terms by accumulated probabilities."

#### 2.2.3.1 Deterministic Spillway Design Flood (SDF) Criteria

For important structures, the PMF (Probable Maximum Flood) is widely used as the SDF. This is suggested as true in determination of a PMF. However, there is a significant disagreement in combining the appropriate magnitude of events. As a result, there can be significant differences in computed flood peaks, volumes and exceedence probabilities (ASCE, 1988). Other criticisms of PMP/PMF criteria are:

1. Use of the PMP/PMF criteria "suggests that the ability to predict future extreme floods is greater than that which actually exists and leads to unrealistic expectations on the part of the public" (Dawdy and Lettenmaier, 1987).
2. Use of the PMP/PMF criteria may give the illusion of absolute safety, thus diverting the attention from greater flood risk which often may result from less extreme but more likely events (NRC, 1985).
3. The extremely small exceedence probability of the PMF as a standard for public safety is not used elsewhere in society, with the possible exception of the nuclear industry.

#### 2.2.3.2 Probabilistic Spillway Design Flood Criteria

The selection of a storm (flood) having desired exceedence probability is the basis for probabilistic SDF criteria. There are two objections to this practice (Dubler and Grigg, 1996):

1. Selection of the appropriate exceedence probability is purely judgmental. Hence no consistency is inherent in the criteria.
2. It must be recognized that (annual) exceedence probability and the length of the planning period (structure life) are directly connected. The “chance of failure” is not equal to the selected exceedence probability.

## 2.3 Dam Safety Guidelines

A guideline is a statement or other indication of policy or procedure by which to determine a course of action; a rule or principle that provides guidance to appropriate behavior. It describes critical decision points in assessment, diagnosis, treatment and evaluation of treatment. Below are some examples of guidelines from different countries.

### 2.3.1 Austrian Guidelines for Seismic Safety Evaluation

New guidelines for earthquake safety assessment were released recently in Austria in accordance with the ICOLD recommendations. Austrian recommendations are summarized as basic principles which especially focus on concrete dam structures. The concept of the Operating Basis Earthquake (OBE) and the Maximum Credible Earthquake (MCE) event is explained (Zenz et al., 1998). A working group of the Austrian Commission on Dams have established the guidelines for seismic safety evaluation for dams in 1996. These guidelines are not obligatory to be used and have no sanctions for application. However, in determination of the evaluation criteria and assisting the authorities, these should be well understood in order to benefit from these.

The guide for seismic safety evaluation applies to dams and reservoirs, river barrages as well as retention basins. Risk groups are not formed for structures; however, simplified evaluation procedures are intended for smaller dams.

#### 2.3.1.1 Seismic Parameters

ICOLD recommendations are followed while evaluating the seismic parameters (ICOLD, 1989), therefore an Operating Basis Earthquake (OBE) and a Maximum Credible Earthquake (MCE) are considered.

The seismic input is defined in terms of maximum horizontal accelerations and unified response spectra. Artificial acceleration time histories which are compatible with the response spectra are also provided and can be employed especially for non-linear analyses.

McGuire´s relationship between magnitude, distance and ground acceleration is used when evaluating the maximum horizontal acceleration which is based on the earthquake-catalogue (Lenhardt, 1995). For the OBE, a return period of 200 years is selected with a minimum value of 0.6 $m/s^2$. For the MCE, not only the results of extreme-value statistics are considered, but also the global geology and long-term tectonic processes are taken into account. The resulting ground accelerations could be considered as approximate values only and, in general, more detailed studies including the local geological situation are necessary for a specific site. The maximum acceleration of the vertical excitation is defined as 2/3 of the respective maximum horizontal acceleration.

#### 2.3.1.2 Analysis

The earthquake loadings are classified as unusual load case and extreme load case, for the OBE and MCE, respectively. OBE and MCE have to be combined with all other normal operation load cases which are, e.g. self weight, water load, temperature, and uplift pressure.

The dam, the reservoir, and a sufficient portion of the foundation should be included in the mathematical model of a dam. The degree of acceptable simplifications depend on the type and height of the dam. For most cases, it is considered acceptable to account for the reservoir using the added mass concept and to consider the foundation as a finite and massless zone.

The recommended calculation procedure for linear problems is the modal analysis combined with the response spectrum method as well as the time integration method. More simplified methods should only be used for smaller dams. If non-linearities occur or are to be expected, their influence on the overall response can be approximately estimated on the basis of the linear calculation. If the effects tend to be significant, a non-linear analysis is recommended.

Assessment of safety is based either on the comparison of maximum stresses and strains with the material strength or, if non-linear analyses are performed, on the status of deformation or damage during and after the earthquake event. The general requirement is that the dam has to pass an OBE without considerable damages and a MCE without loss of the impounding capacity.

Under earthquake conditions, assessment of safety not only concerns the dam itself, but also the reservoir slopes and the appurtenant structures like spillway and bottom outlet (Zenz et al., 1998).

### 2.3.2 US Federal Guidelines for Dam Safety

In 1977, President Carter issued a memorandum directing three actions (USACE, 1996):

1. That all Federal agencies having responsibility for dams conduct a thorough review of their practices which could affect the safety of these structures and report their findings to the FCCSET.
2. That FCCSET prepare the "Federal Guidelines for Dam Safety" for use by all Federal agencies.
3. That ICODS be established to promote and monitor Federal and state dam safety programs.

In 1979, the "Federal Guidelines for Dam Safety" was published, and ICODS was given oversight responsibility for dam safety. The key management practices outlined in these guidelines are as follows (FEMA, 1979):

1) Establish a Dam Safety Officer and appropriate staff.
2) Maintain an updated inventory of dams.
3) Document design criteria and construction activities.
4) Prepare initial reservoir filling plans and reservoir regulation criteria.
5) Prepare operation and maintenance instructions and document activities.
6) Maintain a training and awareness program.
7) Prepare and maintain EAPs for each dam.
8) Establish a program of periodic inspections and evaluation of dams.
9) Monitor and evaluate the performance of each dam and appurtenant structure and provide remedial construction as necessary.

The US National Dam Safety Program, coordinated by FEMA (2004) has been published by The National Dam Safety Program Act (1996).

### 2.3.3 The Canadian Standards Association

The Canadian Standards Association has issued a standard that sets out general requirements and gives guidelines for selecting and implementing risk analysis techniques, primarily for technological hazards (CSA, 1991). It is recognized that risk analysis is a structured process that attempts to identify both the likelihood and extent of consequences associated with such hazards. This standard also provides guidelines to ensure that the results of the risk analysis are documented so clearly that they can be reviewed and used by people other than the author(s) of the analysis.

# CHAPTER 3

# STRUCTURAL RELIABILITY APPROACH

The reliability of an engineering system can be defined as its ability to fulfill its design purpose for specified time period. This ability can be measured using the probabilistic theory. In order to determine the risk of structures, researchers have proposed methods, such as return interval, safety factor, reliability index, first order second moment method, advanced first order second moment method, and Monte Carlo simulation (Yen et al., 1986; Ang and Tang, 1990).

## 3.1 Classical Reliability Approach

The joint occurrence of various quantities and true nature of the failure domain is described fully. A structural component, a single mode of failure, and a specific direction for the forces should be considered in this type of approach. Let R and S denote random resistance (capacity) and load (demand), respectively. Probability of failure, also termed as risk, is the probability for which resistance is less than or equal to load. The complimentary of risk is defined as reliability. When the probability distributions of random resistance and load are known, the probability of failure is determined from (Ang and Tang, 1990):

$$P_f = \iint_{\{(s,r):\, r<s\}} f_{R,S}(r,s)\, dr\, ds = \int_0^{\infty}\int_0^{s} f_{R,S}(r,s)\, dr\, ds \tag{3.1}$$

where $f_{R,S}(r,s)$ is the joint density function of resistance and loading. If R and S are statistically independent, then $f_{R,S}(r,s) = f_R(r)\, f_S(s)$, which is also expressed as:

$$P_f = \int_0^{\infty} \left[ \int_0^{s} f_R(r)\, dr \right] f_S(s)\, ds \tag{3.2}$$

Failure probability depends on the assumed distributions. Therefore, the result will be correct if the distributions that are used are actually valid. The following formulation generalizes the probability of failure:

Failure: $\lfloor (R_p < S;\ S > 0) \cup (R_n > S;\ S \le 0) \rfloor$

in which p and n denote positive and negative quantities, respectively. Therefore,

$$P_f = \iiint_{\{(s, r_p, r_n): s>0 : r_p < s\}} f_{s,R_p,R_n}(s, r_p, r_n)\, ds\, dr_p\, dr_n + \iiint_{\{(s, r_p, r_n): s \le 0 : r_n > s\}} f_{s,R_p,R_n}(s, r_p, r_n)\, ds\, dr_p\, dr_n$$

$$P_f = \int_0^{\infty} \int_0^{s} f_{s,R_p}(s, r_p)\, dr_p\, ds + \int_{-\infty}^{0} \int_{s}^{0} f_{s,R_n}(s, r_n)\, dr_n\, ds \tag{3.3}$$

### 3.1.1 Probability Distributions

The mostly used probability distributions in civil engineering applications are uniform, normal, and lognormal distributions.

#### 3.1.1.1 Uniform Distribution

The random variable x is defined on the interval a to b (See Figure 3.1) with the probability distribution function:

$$p(x) = \frac{1}{b-a} \qquad \text{where } a \le x \le b \tag{3.4}$$

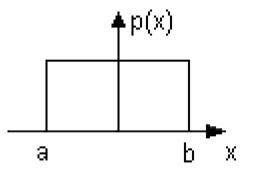


Figure 3.1 Uniform PDF.

### 3.1.1.2 Normal Distribution

Normal or Gaussian distribution is characterized by two parameters; $\mu$ (mean) and $\sigma$ (standard deviation). The random variable x is said to be normally distributed if its PDF is:

$$p(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(\frac{-(x-\mu)^2}{2\sigma^2}\right) \quad ; \quad -\infty \le x \le \infty \tag{3.5}$$

Coefficient of variation is $\delta = \sigma / \mu$. The probability that a random variable will assume a value between a and b can be determined by computing the area under its PDF between these two points (See Figure 3.2):

$$P(a \le x \le b) = \int_a^b p(x)dx \tag{3.6}$$

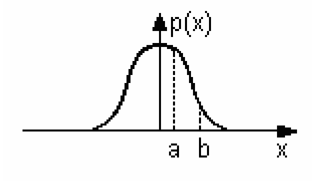


Figure 3.2 Normal PDF.

#### 3.1.1.3 Log-Normal Distribution

The log-normal distribution is advantageous over the normal distribution in such a way that numerical values of data points following a log-normal distribution are always positive. The normal distribution can be transformed to the log-normal distribution by the transformation of variables. If the random variable x is normally distributed, the random variable y is defined by the transformation: y = ln x. The two parameters are; $\lambda$ (mean) and $\xi$ (standard deviation).

The log-normal distribution of x (See Figure 3.3) is given by:

$$p(x) = \frac{1}{x\sigma_y\sqrt{2\pi}}\exp\left(\frac{-(y-\mu_y)^2}{2\sigma_y^2}\right) ; \quad x > 0 \tag{3.7}$$

where $\mu_y = \lambda = E(\ln X) = \ln \mu_x - 0.5\,\xi^2$ and $\xi \approx \delta$ for $\delta \leq 0.30$. $\mu_y$ and $\sigma_y$ are the mean and standard deviation of Y, respectively.

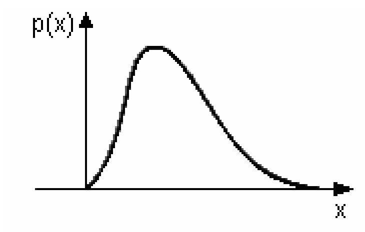


Figure 3.3 Log-Normal PDF.

### 3.1.2 Multiple Failure Modes

One advantage of the probabilistic approach is the fact that different failure modes can be considered and their influence on the failure probability can be reflected. A structural component with k different failure modes will be assumed and denoted by $M_1$, $M_2$, ... , $M_k$.

Let S be the load on the structure and $R_i$ be the capacity at the $i^{th}$ mode. $R_i$'s and S are assumed to be statistically independent:

$f_{s,R1,R2,.Ri..,Rk}$ $(s,r_1,r_2,.r_i..,r_k)$ = $f_s(s)$ $f_{R1,R2,..Ri.,Rk}(r_1,r_2,.r_i..,r_k)$, which can also be expressed as:

$$P_s = \int_0^{\infty} \left[ \int_{c_{1s}}^{\infty} \dots \int_{c_{ks}}^{\infty} f_{R1,\dots,Rk}(r_1, r_2, \dots, r_k)\, dr_1 \dots dr_k \right] f_s(s)\, ds \qquad (3.8)$$

where $c_{1s}$, ..., $c_{ks}$ represent the respective load effects in different failure modes; $f_{R1...Rk}(r_1...r_k)$ is the joint pdf of k-modal resistances.

**3.2 First Order Second Moment Method**

There is an approximate iterative calculation procedure where failure probability is obtained by using idealized failure domain and simplified version of joint pdf (generally mean, variance, and coefficient of variation (c.o.v.)).

In this method, these information should be known as the basic variables:

1- $\mu_i$: mean values (best point estimates)
2- $\sigma^2$, $\sigma$, $\delta$: variance, standard deviation, coefficient of variation (measures uncertainty and variability)
3- $\rho$, COV: correlation coefficient or covariance (measures of dependence)

The limit state function is a state beyond which a structure or part of it can no longer fulfill the functions or satisfy the conditions for which it is designed for; so that the limit state function has to be defined. Let g(x) be the limit state function (performance function).

This function should be defined as (See Figure 3.4):

$g(x) > 0$ when $x \in D_s$ ; $D_s$: survival region

$g(x) \leq 0$ when $x \in D_f$ ; $D_f$: failure region

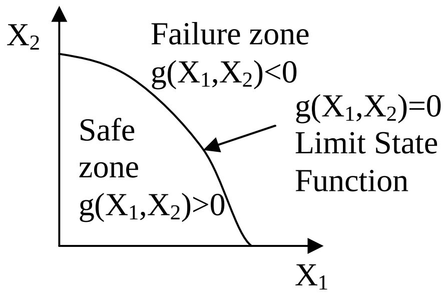


Figure 3.4 Representation of Limit State Function.

The failure probability is then expressed by:

$$P_f = \int\limits_{\substack{g(x)\leq 0 \\ \{Df\}}} \int \ldots \int f_{x_1 \ldots x_n}(x_1 \ldots x_n)\, dx_1 \cdots dx_n \qquad (3.9)$$

### 3.2.1 Reliability Index for Linear Failure Functions

The reliability index, β, that is based on the mean and standard deviation of g(x) can be used for the reliability and is formulated as:

$$\beta = \frac{\mu_M}{\sigma_M} \tag{3.10}$$

where M is the safety margin, i.e. R – S.

As an example, if the variables are normally distributed:

R : N ($\mu_R$ , $\sigma_R$)   ;   S : N ($\mu_S$ , $\sigma_S$)

Failure : $R \le S \rightarrow R - S \le 0$

Mean and standard deviation of the safety margin are:

$$\mu_M = \mu_R - \mu_S \tag{3.11}$$

where $\mu$ is the mean value.

$$\sigma_M = \sqrt{\sigma_R^2 + \sigma_S^2} \tag{3.12}$$

where $\sigma$ is the standard deviation. Probability of failure (See Figure 3.5):

$$P_f = \Pr\left(z \le \frac{-(\mu_R - \mu_S)}{\sqrt{\sigma_R^2 + \sigma_S^2}}\right) = 1 - \phi(\beta) \tag{3.13}$$

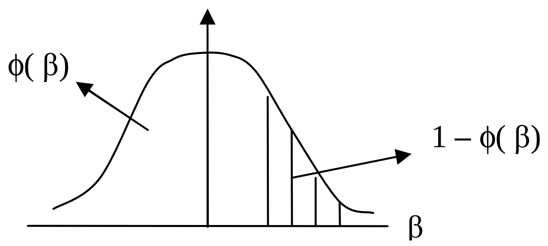


Figure 3.5  Illustrative Standard Normal Distribution.

If failure function (safety margin, M ) is non-linear, then Taylor series expansion is used at the mean point, $\mu_M = g(\mu_1, \mu_2, ..., \mu_n)$.

$$\sigma_M^2 = \sum_{i=1}^{n}\left(\frac{\partial g}{\partial x_i}\right)_{\mu}^{2}\sigma_i^2 + \sum\sum_{i\neq j}\left(\frac{\partial g}{\partial x_i}\right)_{\mu}\left(\frac{\partial g}{\partial x_i}\right)_{\mu}\rho\sigma_i\sigma_j \tag{3.14}$$

However, if g(x) is non-linear, as higher order terms are neglected, significant errors occur and mechanically equivalent formulations of the same failure criterion may give different reliability index values.

### 3.3 Advanced First Order Second Moment Method (AFOSM)

In this method, the Taylor series expansion of the limit state function is linearized at design point or checking point rather than at the mean. The reliability index in this method is defined as the shortest distance from the origin to the failure surface in the normalized z-coordinate system.

The limit state function is defined as being equal to zero; i.e. $g = g(x_1, x_2, \ldots, x_n) = 0$
All basic variables are normalized:

$$z_i = \frac{x_i - \mu_{x_i}}{\sigma_{x_i}} \tag{3.15}$$

where $z_i = \alpha_i \beta$ and $\alpha_i$'s are direction cosines. The design point is the point $\{ x_1^*, x_2^*, \ldots, x_n^* \}$ on g = 0 corresponding to the shortest distance from origin to failure surface. An iteration is used to calculate the design point and the reliability index.

Direction cosines are formulated as:

$$\alpha_i = \frac{\left(\frac{\partial g}{\partial x_i}\right)\sigma_i}{\left[\sum_{i=1}^{n}\left(\left(\frac{\partial g}{\partial x_i}\right)\sigma_i\right)^2\right]^{1/2}} \tag{3.16}$$

After $\alpha_i$ is found, then the coordinates of the design point is found as:

$$x_i^* = \mu_{x_i} - \alpha_i \sigma_i \beta \tag{3.17}$$

The sign of "$\alpha_i \sigma_i \beta$" is negative. This is because of the negativity of the nature of the direction cosines in the formulation. $\beta$ is calculated by trial and error, placing the limit in the limit state function.

### 3.3.1 Equivalent Normal Distributions

If the random variables do not fit normal distribution, the risk can be calculated using equivalent normal distributions (Ang and Tang, 1984). The random variables should be transformed to an equivalent normally distributed random variable. In order to find the equivalent normal distribution value of a variable that does not fit normal distribution, the cumulative probabilities of the equivalent normal distribution and the probability density ordinates are considered to be equal to the non-normal distribution values. The cumulative probabilities are equalized at the $x_i$ point:

$$\phi\left(\frac{x_i^* - \mu_{x_i}^N}{\sigma_{x_i}^N}\right) = F_{x_i}(x_i^*) \tag{3.18}$$

where $\mu_{x_i}^N$, $\sigma_{x_i}^N$ are the mean and standard deviation of the $x_i$ (normal distribution), and $F_{X_i}(x_i^*)$ is the nonnormal cumulative distribution function. Then,

$$\mu_{x_i}^N = X_i^* - \sigma_{xi}^N \phi^{-1} ( F_{X_i} (X_i^*)) \tag{3.19}$$

in which $f_{X_i}(x_i^*)$ is the nonnormal probability density function and $\phi$ is the standard normal variable probability density ordinate.

From the above equations, one can obtain

$$\sigma_{x_i}^N = \frac{\phi\left\{\phi^{-1}\left[F_{x_i}(X_i^*)\right]\right\}}{f_{x_i}(X_i^N)} \tag{3.20}$$

The design point coordinates are then determined from:

$$X_i^* = \mu_{x_i}^N - \alpha_i \beta \sigma_{xi}^N \tag{3.21}$$

The remaining procedures are the same as described in Section 3.3.

### 3.3.2 Correlated Random Variables

Correlated random variables are assumed to be normally distributed because additional information is required for nonnormal and correlated random variables, such as their joint probability density function or conditional distributions for their unique and full definition (Ang and Tang, 1990). This is difficult to obtain. A correlated (and normal) pair of random variables $X_1$ and $X_2$ with a correlation coefficient $\rho$ can be transformed into noncorrelated pair $Y_1$ and $Y_2$ by solving for two eigenvalues $\lambda$ and the corresponding eigenvectors as follows:

$$Y_1 = \frac{1}{2t}\left(\frac{X_1 - \mu_{X_1}}{\sigma_{X_1}} + \frac{X_2 - \mu_{X_2}}{\sigma_{X_2}}\right) \tag{3.22}$$

$$Y_2 = \frac{1}{2t}\left(\frac{X_1 - \mu_{X_1}}{\sigma_{X_1}} - \frac{X_2 - \mu_{X_2}}{\sigma_{X_2}}\right) \tag{3.23}$$

where t = $\sqrt{0.5}$. The resulting Y variables are noncorrelated with respective variances that are equal to the eigenvalues as follows:

$$\sigma_{Y_1}^2 = \lambda_1 = 1 + \rho \tag{3.24}$$

$$\sigma_{Y_2}^2 = \lambda_2 = 1 - \rho \tag{3.25}$$

Equations used in AFOSM should be revised for a correlated pair of random variables:

$$\alpha_{Y_1} = \frac{\left[\left(\frac{\partial Z}{\partial X_1}\right)t\sigma_{X_1} + \left(\frac{\partial Z}{\partial X_2}\right)t\sigma_{X_2}\right]\sqrt{1+\rho}}{\left[\left(\frac{\partial Z}{\partial X_1}\right)^2 \sigma_{X_1}^2 + \left(\frac{\partial Z}{\partial X_2}\right)^2 \sigma_{X_2}^2 + 2\rho\left(\frac{\partial Z}{\partial X_1}\right)\left(\frac{\partial Z}{\partial X_2}\right)\sigma_{X_1}\sigma_{X_2}\right]^{1/2}} \tag{3.26}$$

$$\alpha_{Y_2} = \frac{\left[\left(\frac{\partial Z}{\partial X_1}\right)t\sigma_{X_1} - \left(\frac{\partial Z}{\partial X_2}\right)t\sigma_{X_2}\right]\sqrt{1-\rho}}{\left[\left(\frac{\partial Z}{\partial X_1}\right)^2 \sigma_{X_1}^2 + \left(\frac{\partial Z}{\partial X_2}\right)^2 \sigma_{X_2}^2 + 2\rho\left(\frac{\partial Z}{\partial X_1}\right)\left(\frac{\partial Z}{\partial X_2}\right)\sigma_{X_1}\sigma_{X_2}\right]^{1/2}} \tag{3.27}$$

$$X_1^* = \mu_{X_1} - \sigma_{X_1} t\beta\left(\sigma_{Y_1}\sqrt{\lambda_1} + \alpha_{Y_2}\sqrt{\lambda_2}\right) \tag{3.28}$$

$$X_2^* = \mu_{X_2} - \sigma_{X_2} t\beta\left(\sigma_{Y_1}\sqrt{\lambda_1} - \alpha_{Y_2}\sqrt{\lambda_2}\right) \tag{3.29}$$

where partial derivatives are evaluated at the design point (Ayyub et al., 1998).

### 3.4 Monte Carlo Simulation (MCS) Methods

These methods are basically sampling processes that are used to estimate the failure probability of a structure. This is a method which solves a problem by generating suitable random numbers and observing that fraction of the numbers obeying some property or properties. The method is useful for obtaining numerical solutions to problems which are too complicated to solve analytically. It was named by S. Ulam in 1946 in honor of a relative who liked gambling. Metropolis (1987) also made important contributions to the development of such methods (Weisstein, 1999). Latin hybercube sampling can be considered as an alternative to Monte-Carlo simulation methods.

If the number of simulation cycles in which failure occurs is $N_u$ in a total of N simulation cycles, then estimated failure probability is:

$$\overline{P}_u = \frac{N_u}{N} \tag{3.30}$$

The variance of this failure probability:

$$\mathrm{Var}(\overline{P}_u) = \frac{(1-\overline{P}_u)\cdot\overline{P}_u}{N} \tag{3.31}$$

The coefficient of variation is:

$$\delta(\overline{P}_u) = \frac{1}{\overline{P}_u}\sqrt{\frac{(1-\overline{P}_u)\cdot\overline{P}_u}{N}} \tag{3.32}$$

Different formulas are presented by Melchers (1999) to estimate the required number of simulations. An accurate estimate of the probability of failure of the system analyzed is ensured with a proper convergence. The simplest formula is proposed by Broding et al. (1964):

$$N > \frac{-\ln(1-c)}{P_f} \tag{3.33}$$

where N is the number of simulations for a given confidence level C in the probability of failure $P_f$. For example, more than 3000 simulations are required for a 95% confidence level under $P_f = 10^{-3}$. This total number of simulations should be adjusted as N times the number of independent random variables considered in the analysis as shown in Figure 3.6 in which $\delta_f$ is the coefficient of variation of failure probability.

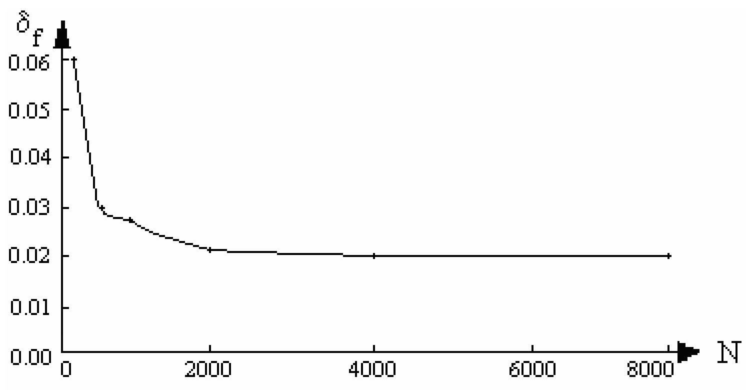


Figure 3.6 Change in Uncertainty with the Number of Cycles in Monte-Carlo Simulations (Yanmaz, 2002).

### 3.4.1 Components of a Generic Monte Carlo Simulation Algorithm

- Probability density function: The physical/mathematical model under consideration must be described statistically.
- A random number generator: Most mathematical software packages or programming languages have uniform random generators.
- Sampling rule: Generating of samples with desired PDF.
- Scoring (tallying): Counting the number of occurrences of events of interest.
- Error estimation: Estimation of error as a function of the number of trials in order to set the number of trials respectively.

Below is a simulation example of computation of π (Uysal, 2003):

Hit-and-miss experiment: Consider a circle with unit radius centered at the origin which is enclosed by a square with each side of length 2 units. Now, a random point (*x*,*y*) from inside this square is picked, then the probability that this random point lies inside the circle is given by:

$$P\left(x^2 + y^2 < 1\right) = \frac{\text{Area}_{\text{circle}}}{\text{Area}_{\text{square}}} = \frac{\pi r^2}{(2r)^2} = \frac{\pi}{4}$$

Now, suppose *N*-random points inside the square is picked and *M* of these points lie inside the unit circle, then the previous probability can be approximated by:

$$P\left(x^2 + y^2 < 1\right) \approx \frac{M}{N} \Rightarrow \pi \approx 4\frac{M}{N}$$

The following Monte Carlo simulation procedure for the computation of $\pi$ is as follows:

- Generate *x* and *y* uniform random variables within [-1,1]
- Count how many of these lie inside the circle; these are the "hits". The ratio of hits over all trials yields $\pi$ /4.

### 3.4.2 Generation of Random Variables

Most computer software libraries include a uniform random number generator. This generator can be used as a basic tool to generate random variables with other PDFs.

Let a be the uniformly distributed random variable with [0,1]. Probability density function is (See Figure 3.7):

$$f_a(a) = 1, \ \ 0 \le a \le 1 \tag{3.34}$$

Cumulative distribution function is (See Figure 3.7):

$$F_a(a) = \int_{-\infty}^{a} f_a(x)dx = \begin{cases} a, & 0 \le a \le 1 \\ 1, & a > 1 \end{cases} \tag{3.35}$$

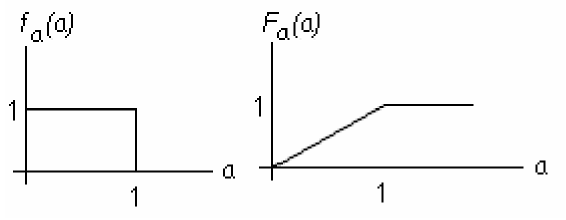


Figure 3.7 PDF and CDF of "a" (Uysal, 2003).

Now suppose that a random variable b with CDF will be generated; $F_b(b_1) = a_1$

$$b_1 = F_b^{-1}(a_1) \tag{3.36}$$

where $a_1$ is the uniformly distributed random variable. This is known as "transform method" (See Figure 3.8). If $z$ is a continuous random variable, then the distribution is first (uniformly) quantized. Denote $p_1$, $p_2$,…$p_N$, probabilities of $N$ number of cells (Figure 3.9 shaded area).

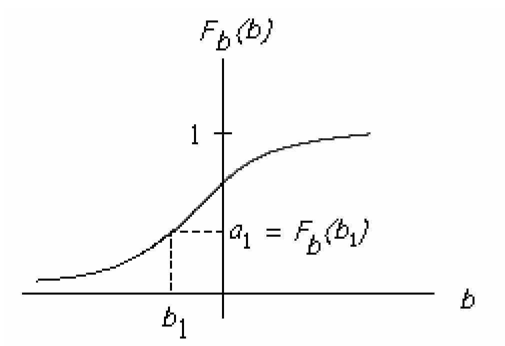


Figure 3.8  Transform Method (Uysal, 2003).

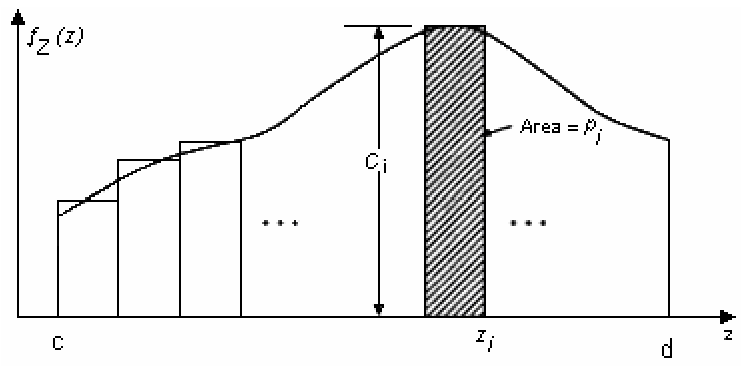


Figure 3.9  Illustrative Figure for Emprical Search Algorithm (Uysal, 2003).

When the inverse transform cannot be expressed in closed form, the following emprical search algorithm can be used:

- Generate "a" uniformly in [c, d].
- Let $F_i = \sum_{j=1}^{i} P_j$ , i = 1,2,...,N with $F_o 0$ (CDF) (3.37)
- Find the smallest value of *i* that satisfies $F_{i-1} < a \leq F_i$ i = 1,2,...,N
- Use the interpolation formula:

$$z = z_{i-1} + (a - F_{i-1}) / c_i \tag{3.38}$$

and return to the first step.

# CHAPTER 4

# SAFETY ANALYSIS OF GRAVITY DAMS

The dead weight and the base width of concrete gravity dams are the governing variables which must be large enough such that overturning and sliding tendencies are overcome. In this chapter, the forces which the dams are exposed to are introduced as shown in Figures 4.1 - 4.3.

## 4.1 Forces Acting on Concrete Gravity Dams

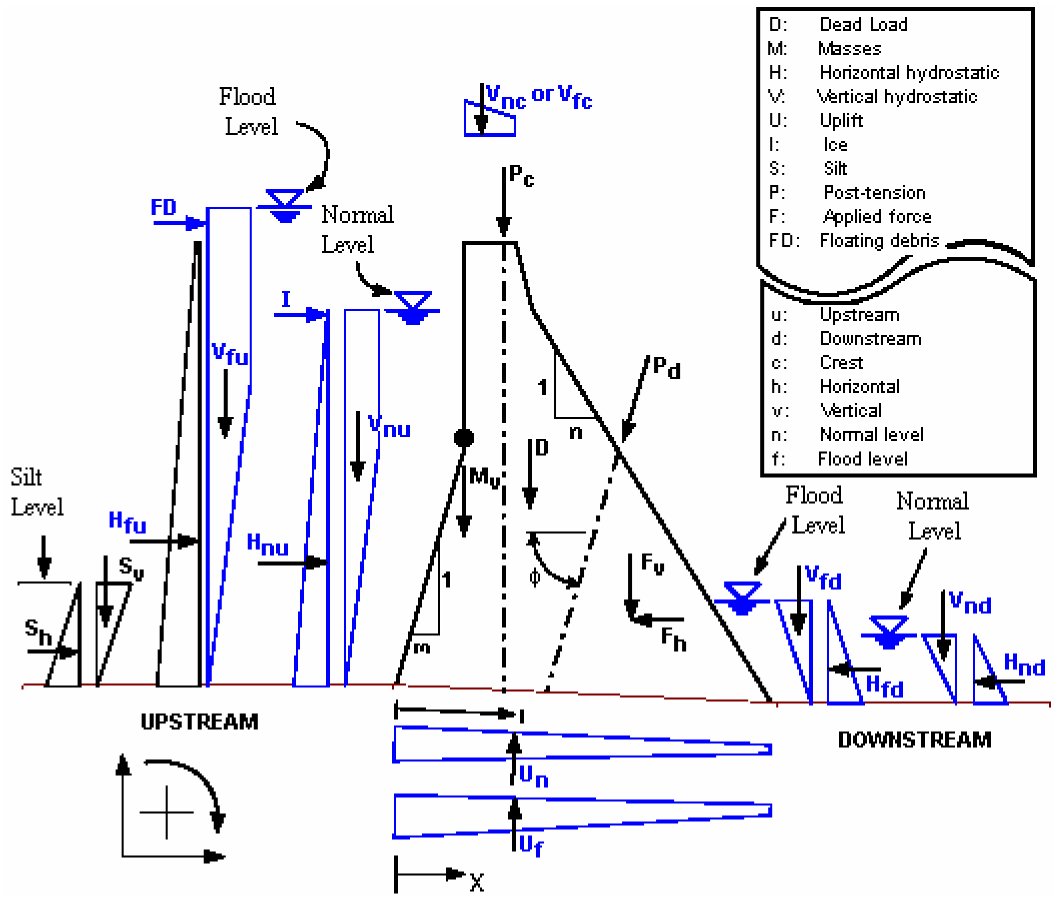


Figure 4.1 Basic Loadings – Static Analysis (Leclerc et al., 2001).

Let upstream normal water level is $h_1$, downstream normal water level is $h_2$, silt level is $h_s$, width of the dam is B, upstream slope is 1/m, and downstream slope is 1/n. All the height indicators are of the same level, i.e. if it is normal level at the upstream, then it is normal level at the downstream, too. The following equations are arranged assuming that the water level is at the normal level. Wave and wind forces may also be considered. The possible forces acting on concrete gravity dams are listed below (Yanmaz, 2001):

a) The weight (dead load), D of the dam. This force acts at the centroid of the dam.

b) Hydrostatic forces per unit width, $H_{nu}$, $V_{nu}$, $H_{nd}$, $V_{nd}$.

$$H_{nu} = \frac{1}{2}\gamma h_1^2 \quad ; \quad V_{nu} = \frac{1}{2}\gamma m h_1^2 \quad ; \quad H_{nd} = \frac{1}{2}\gamma h_2^2 \quad ; \quad V_{nd} = \frac{1}{2}\gamma n h_2^2 \qquad (4.1)$$

where $\gamma$ is the specific weight of water. These forces act at 1/3 of the heights above the base.

c) Uplift force per unit width, $U_n$.

$$U_n = \left[ h_2 + \frac{\phi}{2}(h_1 - h_2) \right] B\gamma \qquad (4.2)$$

where $\phi$ is the uplift reduction coefficient. The uplift may be reduced up to 50% by installing drains in the dam body and at the foundation level.

d) Force due to sediment accumulation (lateral earth force per unit width), $S_h$.

$$S_h = \frac{1}{2}\gamma_s h_s^2 K_a \tag{4.3}$$

where $\gamma_s$ is the submerged specific weight of soil and $K_a$ is the active earth pressure coefficient according to Rankine theory.

$$K_a = \frac{1-\sin\theta}{1+\sin\theta} \tag{4.4}$$

in which $\theta$ is the angle of repose. This force acts at 1/3 of the silt depth above the base.

e) Ice load, I. The ice thickness and rate of temperature rise should be used to compute the approximate ice loading (Yanmaz, 2001).

### 4.1.1 Pseudo – Static Seismic Analysis (Seismic Coefficient)

The inertia forces induced by the earthquake are computed from the product of the mass and the acceleration. The dynamic amplification of inertia forces along the height of the dam due to its flexibility is neglected (Figure 4.2).

f) Earthquake force on the dam body (dam inertia), Q.

$$Q = k\,D \tag{4.5}$$

where k is the effective peak ground acceleration coefficient which has horizontal and vertical values, $Q_v$ and $Q_h$, respectively which act through the center of gravity of the dam.

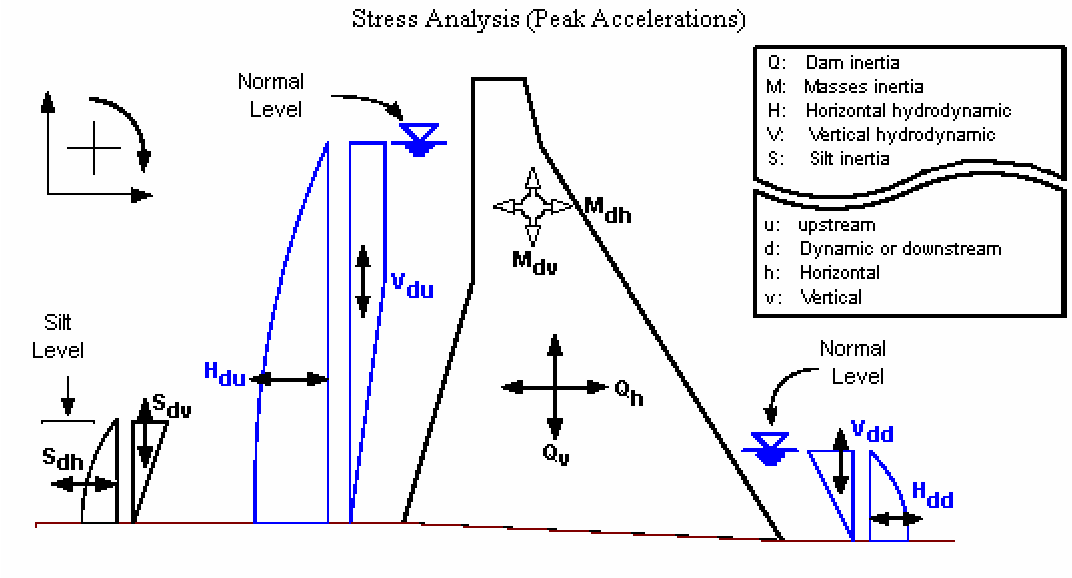


Figure 4.2 Basic Loadings Supported for Pseudo – Static Seismic Analysis (Leclerc et al., 2001).

g) Hydrodynamic force can be determined by two different ways:

1) Horizontal hydrodynamic force per unit width induced by earthquake, $H_{du}$, which acts at 0.412 $h_1$ above the bed (Yanmaz, 2001).

$$H_{du} = 0.5082\left(1 - \frac{\arctan(m)}{90}\right) k\,\gamma\,h_1^2 \tag{4.6}$$

2) Westergaard Added Masses – Vertical Upstream Face:

The added horizontal hydrodynamic force $H_d(y)$ increasingly follows a parabolic distribution for an assumed rigid gravity dam with vertical upstream face (Leclerc et al., 2001).

$$H_d(y) = \frac{2}{3} K_\theta C_e (acc) \sqrt{h}\,(y^{1.5}) \tag{4.7}$$

where $H_d(y)$ is the additional total hydrodynamic horizontal force acting above the depth y for a unit width of the dam, (acc) is the horizontal seismic acceleration coefficient applied at the base of the dam expressed in terms of peak ground acceleration or spectral acceleration (fraction of g), h is the total depth of the reservoir, y is the distance below reservoir surface, $K_\theta$ is the correction factor for the sloping dam faces with angle θ from the vertical. To compute the horizontal force, $K_{\theta H} = \cos^2\theta$ can be used as a first approximation, while the vertical force can be estimated from $K_{\theta V} = \sin\theta\cos\theta$. $C_e$ is a factor depending principally on depth of water and the earthquake vibration period characterizing the frequency of the applied ground motion.

The Westergaard approximation for the $C_e$ coefficient is:

$$C_e = 7.99\, C_c \tag{4.8}$$

where

$$C_c = \frac{1}{\sqrt{1 - 7.75\left(\dfrac{h}{1000\, t_e}\right)^2}}$$

in which $C_c$ is a correction factor in kN.s.m to account for water compressibility and $t_e$ is the period to characterize the seismic acceleration imposed to the dam in seconds.

USBR (1987) considers the following specifications for inclined faces:

For dams with a combination vertical and sloping face, the procedure to be used is governed by the relation of the height of the vertical portion to the total height of the dam as follows:

- If the height of the vertical portion of the upstream face of the dam is equal or greater than one-half of the total height of the dam, analyse as if a vertical throughout.
- If the height of the vertical portion of the upstream face of the dam is less than one-half of the total height of the dam, use the pressures on the sloping line connecting to the point of intersection of the upstream face of the dam and reservoir surface with the point of intersection of the upstream face of the dam and the foundation.

### 4.1.2 Pseudo – Dynamic Analysis (Chopra's Method)

The general analytical procedure is appropriate for analyzing the safety of existing and new dams against future earthquakes in the final stages of the evaluation and design processes, respectively, but the procedure should be simplified for convenient application in the preliminary evaluation or design stage. In response to this need, a simplified procedure was developed in 1978. In this procedure, the maximum response due to fundamental mode of vibration was represented by equivalent lateral forces and was computed directly from the earthquake design spectrum, without a response history analysis. This simplified analysis of the fundamental mode response has been extended to include the effects of dam-foundation-rock interaction and of reservoir bottom materials, in addition to the effects of dam-water interaction and water compressibility considered in the earlier procedure. Also included now in the simplified procedure are the equivalent lateral forces associated with higher vibration modes, which are computed by a static correction method based on the assumptions that: the dynamic amplification of the modes is negligible; the interactions among the dam, impounded water and foundation rock are not significant; and the effects of water compressibility can be neglected. These approximations provide a practical method for including the most important factors that affect the earthquake response of concrete gravity dams (Figure 4.3). The information for the simplified analysis procedure is taken from Chopra (1988).

#### 4.1.2.1 Design Earthquake Spectrum

A few parameters are required in the simplified analysis procedure to describe the dam-water-foundation rock system: $E_s$ (Young's modulus of elasticity of the structure), $\xi_1$ (viscous damping ratio: for the large motions and high stresses expected in a dam during intense earthquakes, the value of 5% is recommended), $H_s$ (the height of the dam from base to the crest), $\eta_f$ (constant hysteretic damping coefficient of the foundation rock. In the absence of information on damping properties of the foundation rock, a value of 0.10 is recommended). Let H be the depth of the impounded water measured from the free surface to the reservoir bottom and $\alpha$ be the wave reflection coefficient.

The horizontal earthquake ground motion is specified by a pseudo-acceleration response spectrum in the simplified analysis procedure. This should be a smooth response spectrum, without the irregularities inherent in response spectra of individual ground motions, representative of the intensity and frequency characteristics of the design earthquakes, which should be established after a seismological and geological investigation (Chopra, 1988).

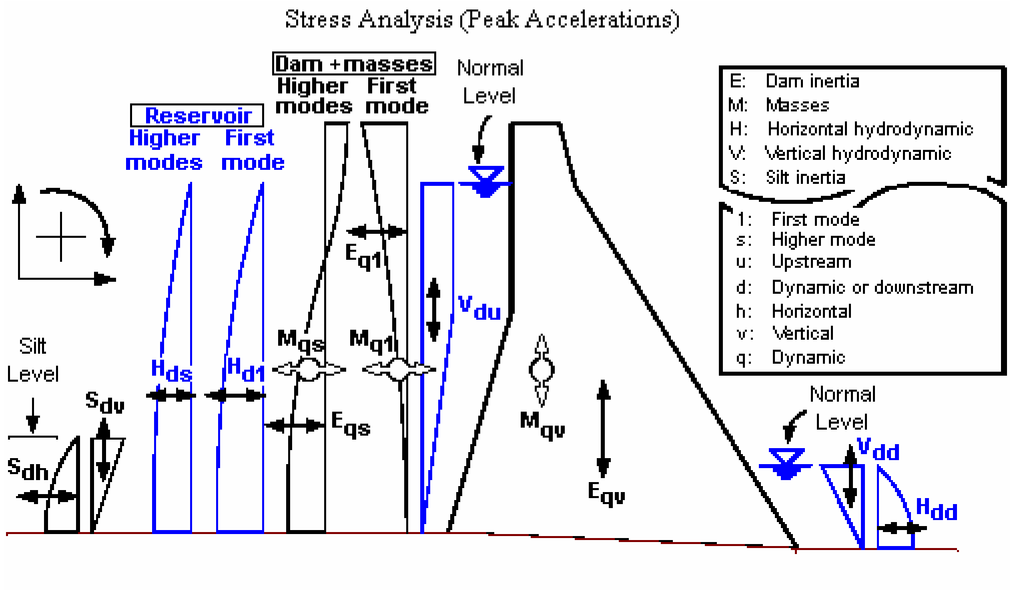


Figure 4.3 Basic Loading Condiditons Supported for Pseudo – Dynamic Seismic Analysis (Leclerc et al., 2001).

#### 4.1.2.2 Computational Steps

The computation of earthquake response of the dam is organized in four parts: Earthquake forces and stresses due to the fundamental vibration mode, earthquake forces and stresses due to the higher vibration modes, initial stresses in the dam due to various loads, and total stresses in the dam.

##### 4.1.2.2.1 Earthquake Forces and Stresses by Fundamental Vibration Mode

The earthquake forces and stresses due to the fundamental vibration mode can be determined approximately for purposes of preliminary design by the following computational steps:

1. $T_1$, the fundamental vibration period of the dam, in seconds, on rigid foundation rock with an empty reservoir is computed from:

$$T_1 = 1.4 \frac{H_s}{\sqrt{E_S}} \tag{4.9}$$

in which $H_s$ is the height of the dam in feet and $E_s$ is the design value for Young's modulus of elasticity of concrete, in pounds per square inch.

2. $\tilde{T}_r$, the fundamental vibration of the dam in seconds including the influence of impounded water, is computed from:

$$\tilde{T}_r = R_r \cdot T_1 \tag{4.10}$$

in which $R_r$ is the period ratio (Figure 4.4). As can be seen from this figure, if $H / H_s < 0.5$, $R_r \approx 1$ can be used.

3. $R_w$, the period ratio, is computed from:

$$R_w = \frac{T_1^r}{\tilde{T}_r} \tag{4.11}$$

in which $T_1^r = 4H/C$, where C = 4720 ft/sec.

4. $\tilde{T}_1$, the fundamental vibration of the dam, in seconds, including the influence of dam-foundation rock interaction and of impounded water, is computed from:

$$\tilde{T}_1 = R_r R_f T_1 \qquad (4.12)$$

in which $R_f$ is the period ratio (Figure 4.5).

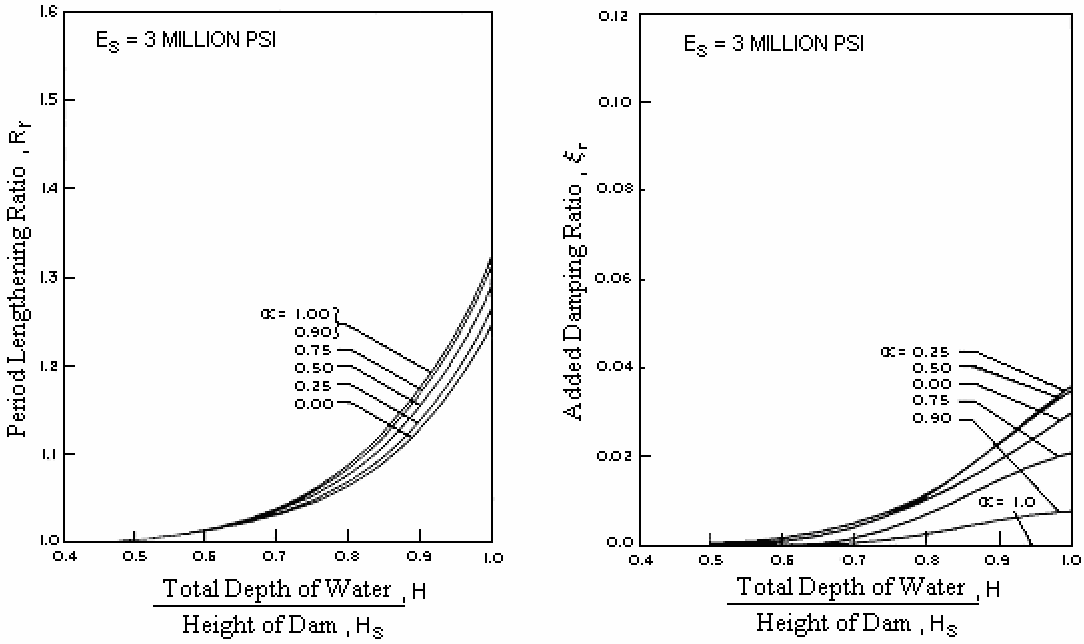


Figure 4.4 Standard Values for $R_r$, the Period Lengthening Ratio, and $\xi_r$, the Added Damping Ratio, due to Hydrodynamic Effects (Chopra, 1988).

5. $\tilde{\xi}_1$, the damping ratio of the dam is computed from:

$$\tilde{\xi}_1 = \frac{1}{R_r}\frac{1}{(R_f)^3}\xi_1 + \xi_r + \xi_f \qquad (4.13)$$

in which $\xi_1$ is the viscous damping ratio for the dam on rigid foundation rock with empty reservoir, $\xi_r$ is the added damping ratio due to dam-water interaction and reservoir bottom absorption (Figure 4.4), and $\xi_f$ is the added damping ratio due to dam-foundation rock interaction (Figure 4.5). If the computed value of $\tilde{\xi}_1 < \xi_1$, then $\tilde{\xi}_1 = \xi_1$ is used.

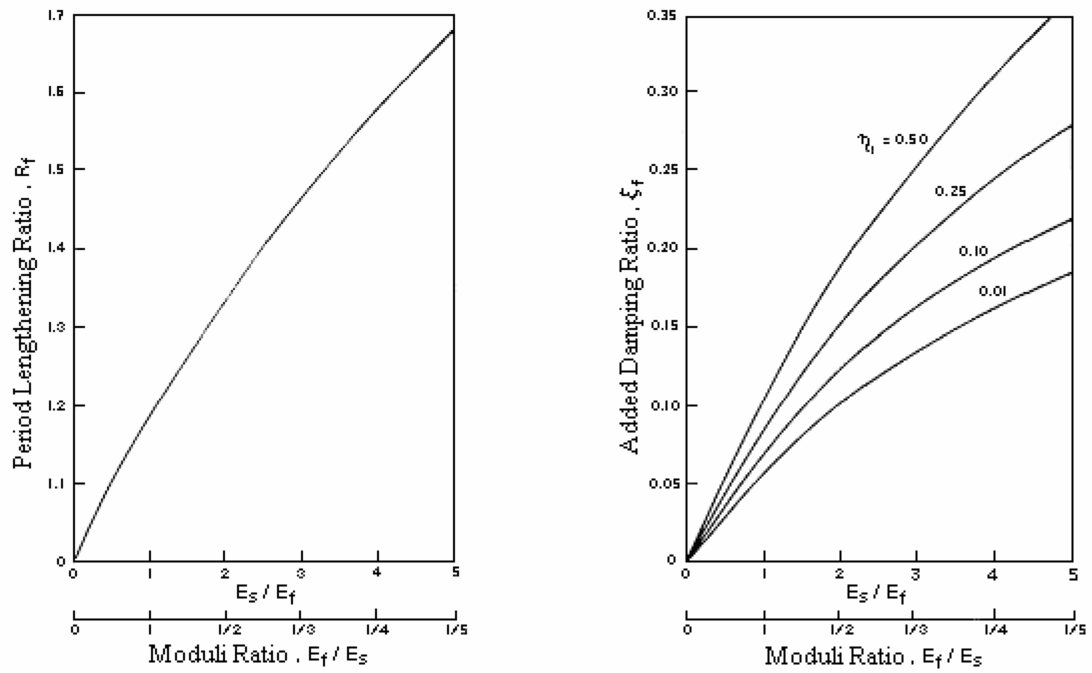


Figure 4.5 Standard Values for $R_f$, the Period Lengthening Ratio, and $\xi_f$, the Added Damping Ratio, due to Dam-Foundation Rock Interaction (Chopra, 1988).

6. g p(y, $\tilde{T}_r$), the hydrodynamic pressure term is determined (Figure 4.6). Computed $R_w$ should be rounded to one of the two nearest available values, the one giving the larger p(y). If H / $H_s$ < 0, then p (y, $\tilde{T}_r$) ≈ 0 can be used.

7. $\tilde{M}_1$, the generalized mass is computed from:

$$\tilde{M}_1 = (R_r)^2 M_1 \tag{4.14}$$

in which $M_1$ is computed from:

$$M_1 = \frac{1}{g}\int_0^{Hs} w_s(y)\phi^2(y)dy \tag{4.15}$$

in which $w_s(y)$ = the weight of the dam per unit height. The fundamental vibration mode shape $\phi(y)$ is given in Figure 4.7. This equation can be approximated as $M_1 = 0.043$ D/g, where g = 32.2 ft /s$^2$.

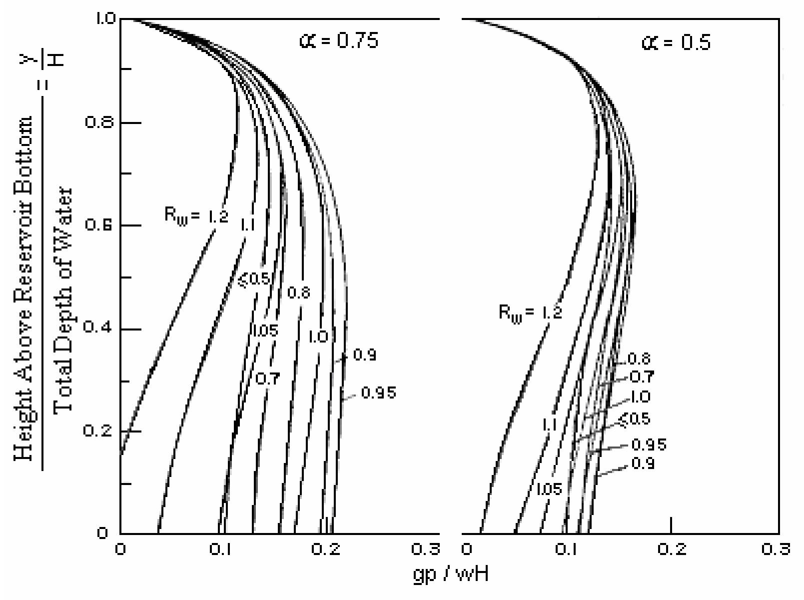


Figure 4.6 Standard Values for the Hydrodynamic Pressure Function p(ŷ) for Full Reservoir, i.e. H / $H_s$ = 1; α = 0.75 and 0.50 (Chopra, 1988).

8. $\tilde{L}_1$, the generalized earthquake force coefficient is computed from:

$$\tilde{L}_1 = L_1 + \frac{1}{8}F_{st}\left(\frac{H}{H_s}\right)^2 A_p \tag{4.16}$$

in which $L_1$ is computed from:

$$L_1 = \frac{1}{g}\int_0^{Hs} w_s(y)\phi(y)dy \tag{4.17}$$

$F_{st} = wH^2/2$ ; $A_p$ is found by using $R_w$ and $\alpha$ . If $H / H_s < 0$, then $\tilde{L}_1 \approx L_1$ can be used. This equation can be approximated as $L_1 = 0.13$ D/g. For the seventh and eighth steps, conservative values can be used to avoid many unknowns: $\tilde{L}_1/\tilde{M}_1 = 4$ for dams with impounded water and $L_1/M_1 = 3$ for dams with empty reservoirs.

9. $f_1(y)$, the equivalent lateral earthquake forces associated with the fundamental vibration mode is computed from:

$$f_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1}\frac{S_a(\tilde{T}_1, \tilde{\xi}_1)}{g}\left[w_s(y)\phi(y) + gp(y, \tilde{T}_r)\right] \tag{4.18}$$

   in which $S_a(\tilde{T}_1, \tilde{\xi}_1)$ is the pseudo-acceleration ordinate of earthquake design spectrum in feet per squared second at $\tilde{T}_1$ and $\tilde{\xi}_1$.

10. The stresses throughout the dam by static analysis of the dam subjected to equivalent lateral forces $f_1(y)$, applied to the upstream face of the dam are determined. The finite element method may be used for this analysis. Alternatively, traditional procedures for design calculations may be used wherein the normal bending stresses $\sigma_{y1}$ across a horizontal section are computed by elementary formulas for stresses in beams. A correction factor may be needed for the sloping part of the downstream face because the beam theory overestimates the stresses near the sloped downstream face (Chopra, 1988).

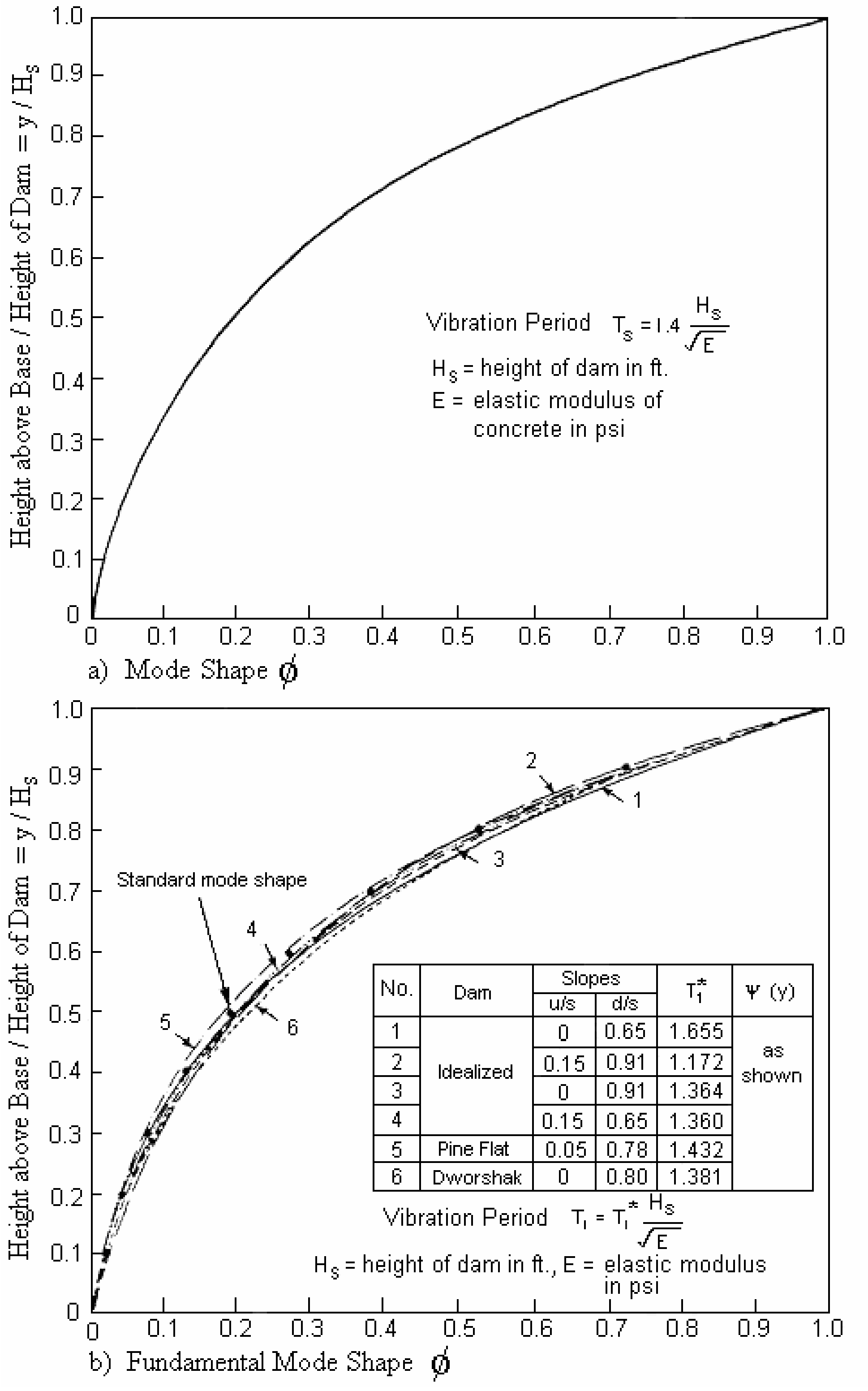


Figure 4.7 a) Standard Period and Mode Shape for Concrete Gravity Dams.
b) Comparison of Standard Values with Properties of Six Dams (Chopra, 1988).

The maximum principal stresses at the upstream and downstream faces can be computed from the normal bending stresses $\sigma_{y1}$ by an appropriate transformation:

$$\sigma_1 = \sigma_{y1} \sec^2\theta + p_1 \tan^2\theta \tag{4.19}$$

where θ is the angle of the face with respect to the vertical. If no tail water is included in the analysis, the hydrodynamic pressure $p_1 = 0$ for the downstream face. At the upstream face, the hydrodynamic pressure $p_1$ is given by (second part of step 9):

$$p_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} S_a(\tilde{T}_1, \tilde{\xi}_1) p(y, \tilde{T}_r) \tag{4.20}$$

#### 4.1.2.2.2 Earthquake Forces and Stresses by Higher Vibration Modes

The earthquake forces and stresses due to the higher vibration modes can be determined approximately for purposes of preliminary design by the following computational steps:

11. $f_{sc}(y)$, the lateral forces associated with the higher vibration modes is computed from:

$$f_{sc}(y) = \frac{1}{g}\left\{ w_s(y)\left[1 - \frac{L_1}{M_1}\phi(y)\right] + \left[g p_o(y) - \frac{B_1}{M_1} w_s(y)\phi(y)\right]\right\} a_g \tag{4.21}$$

in which $g.p_o(y)$ is determined from Figure 4.8; $a_g$ is the maximum ground acceleration of the design earthquake in feet per squared second. $B_1$ is computed from:

$$B_1 = 0.2 \frac{F_{st}}{g} \left( \frac{H}{H_s} \right)^2 \tag{4.22}$$

If $H / H_s < 0.5$, $p_o(y) \approx 0$, and hence $B_1 \approx 0$.

12. This step is the same as Step 10 except that the subscripts are "sc" because this process is associated with the higher vibration modes.

$$\sigma_{sc} = \sigma_{y,sc} \sec^2\theta + p_{sc} \tan^2\theta \tag{4.23}$$

If no tail water is included in the analysis, the hydrodynamic pressure $p_{sc}=0$ for the downstream face. At the upstream face, the hydrodynamic pressure $p_{sc}$ is given by (second part of step 11):

$$p_{sc}(y) = \left[ g p_o(y) - \frac{B_1}{M_1} w_s(y) \phi(y) \right] \frac{a_g}{g} \tag{4.24}$$

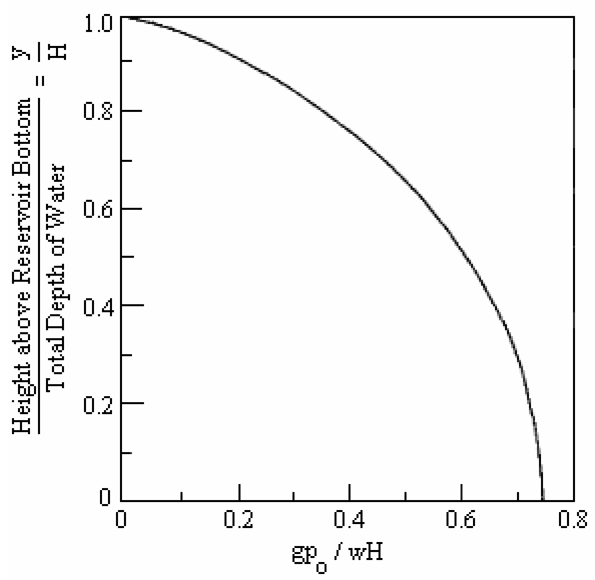


Figure 4.8 Standard Values for $p_o(\hat{y})$ (Chopra, 1988).

#### 4.1.2.2.3 Initial Stresses in the Dam due to Various Loads

The initial stresses in the dam due to various loads, prior to the earthquake, including the self-weight of the dam, hydrostatic pressure, creep, construction sequence and thermal effects are determined by the following computational step:

13. The normal stresses, $\sigma_{y,st}$, across horizontal sections firstly and then the maximum principal stresses are computed from:

$$\sigma_{st} = \sigma_{y,st} \sec^2\theta + p_{st} \tan^2\theta \tag{4.25}$$

The hydrostatic pressure $p_{st} = w\,(H - y)$ on the upstream face and $p_{st} = 0$ on the downstream face if tail water is excluded.

#### 4.1.2.2.4 Total Stresses in the Dam

The total stresses in the dam are determined by the following steps:

14. The dynamic response is computed from the square-root-of-the-sum-of-squares (SRSS) combination rule:

$$r_d = \sqrt{(r_1)^2 + (r_{sc})^2} \tag{4.26}$$

15. The total value of any response quantity is computed by:

$$r_{max} = r_{st} \pm \sqrt{(r_1)^2 + (r_{sc})^2} \tag{4.27}$$

in which $r_{st}$ is its initial value prior to the earthquake.

The SRSS combination rule is also appropriate to determine the principal stresses because the upstream face for most of the gravity dams are almost vertical.

Most of the quantities that are in the simplified analysis procedure are in nondimensional form, so implementing these to metric units is straightforward:

1. $T_1$, the fundamental vibration period:

$$T_1 = 0.38 \frac{H_s}{\sqrt{E_S}} \tag{4.28}$$

where $H_s$ is in meters and $E_s$ is in mega-Pascals (MPa).

2. 1 million psi (pounds per square inch) = 7000 MPa

3. The unit weight of water, w = 9.81 kN/m$^3$
   The gravitational acceleration = 9.81 m/s$^2$
   Velocity of pressure waves in water, C = 1440 m/s.

#### 4.1.2.3 Spectral Acceleration Coefficient

In order to complete the pseudo-dynamic analysis, the spectral acceleration ($S_a(\tilde{T}_1, \tilde{\xi}_1)$) or in other words, the pseudo-acceleration ordinate of the earthquake design spectrum at period $\tilde{T}_1$ and damping ratio $\tilde{\xi}_1$ should be known and inputted in the calculations. The determination of this coefficient is earthquake-dependent, because the earthquake data are put into the response spectrum with the vibration period for different damping ratios, by which the spectral acceleration can be found after plotting them together. If a response spectrum is not available for the site under investigation, then theoretical or design formulations are needed in order to obtain the spectral acceleration coefficient. In Turkey, Ministry of Public Works and Settlement has provided a specification called "Specification for Structures to be Built in Disaster Areas". Determination of the spectral acceleration coefficient corresponding to 5% damped elastic design acceleration spectrum is presented as follows (RTMPWS, 1997):

$$A(T) = A_o\, I\, S(T) \tag{4.29}$$

where A(T) is the spectral acceleration coefficient, $A_o$ is the effective horizontal ground acceleration coefficient ("k" was assigned to it in section 4.1.1. The values for this coefficient depends on the seismic zones. In Turkey, $A_o$ values are 0.4, 0.3, 0.2, and 0.1 for the 1$^{st}$, 2$^{nd}$, 3$^{rd}$,and 4$^{th}$ seismic zones, respectively, I is the building importance factor, which is 1.5 for power generation and distribution facilities (RTMPWS, 1997), S(T) is the spectrum coefficient (See Figure 4.9).

$$S(T) = 1 + 1.5\, T / T_A \qquad (0 \leq T \leq T_A) \tag{4.30}$$

$$S(T) = 2.5 \qquad (T_A \leq T \leq T_B) \tag{4.31}$$

$$S(T) = 2.5\, (T_B / T)^{0.8} \qquad (T > T_B) \tag{4.32}$$

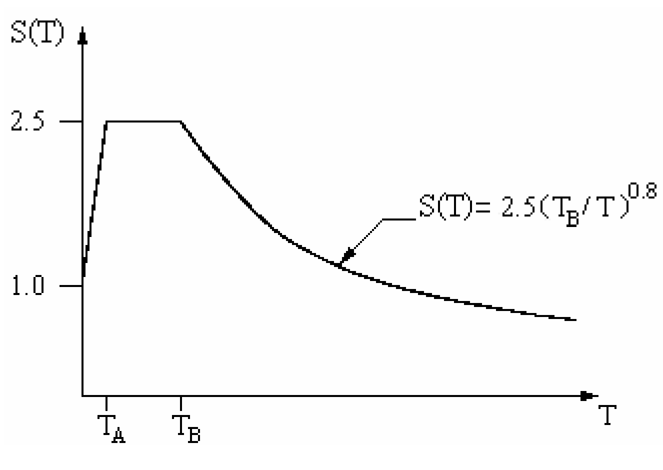


Figure 4.9 Special Design Acceleration Spectra (Figure 6.6 of RTMPWS, 1997).

in which T is the building natural period, $T_A$ and $T_B$ are the spectrum characteristic periods (RTMPWS, 1997).

In order to find the values of $T_A$ and $T_B$, the description of the soil group forming the site should be known to determine the soil group (RTMPWS, 1997). This group is entered as input into local site classes table (Table 12.2 of RTMPWS, 1997). As all the data are obtained, $T_A$ and $T_B$ values can be found within the given reference.

## 4.2 Stability Analysis

Anderson (2001) states that modern engineering is based on predicting the performance of structures before they are actually built. This requires an assessment of how well the system performance can be predicted for the intended materials, expected use, foreseeable abuse, the expected service environment, and the expected life of the system. The transition from engineering model to reality is usually facilitated by including a factor of safety in the design to accommodate uncertainty in material properties and the design process, the consequences of failure, risk to people, and degree of characterization of and control over the service environment.

### 4.2.1 Normal Base Pressure

$$\sigma = \frac{\sum V}{A} \pm \frac{Mc}{I} \tag{4.33}$$

where

$\sigma$ = Vertical normal base pressure

$\Sigma V$ = Sum of all vertical loads including uplift pressures

A = Area of the base that normal pressure takes place

M = Sum of moments about the base centerline

c = distance from centerline to the location where stresses are computed

I = Moment of inertia.

If the stress analysis is performed to compute the potential crack length and compressive stresses along each joint, the normal force resultant (the stress at the crack tip), $\sigma_n$ is computed by the same equation given for vertical normal base pressure but this time, the components relate to the following:

$\Sigma V$ = Sum of all vertical loads including uplift pressures

A = Area of uncracked ligament

M = Moments about the center of gravity of the uncracked ligament of all loads

c = distance from center of gravity of the uncracked ligament to the location where stresses are computed

I = Moment of inertia of the uncracked ligament.

### 4.2.2 Overturning Stability

If the crack lengths are limited such that the allowable compressive stress is not exceeded, the overturning stability could be obtained. The overturning safety factor (OSF) is computed by:

$$OSF = \frac{\sum M_s}{\sum M_o} \quad (4.34)$$

where $\Sigma M_s$ is the sum of stabilizing moment about the downstream or the upstream end of the joint considered and $\Sigma M_o$ is the sum of destabilizing (overturning) moments. To assess the overturning stability of the section above the crack plane considered, also the location of the force resultant along the joint, $L_{FR}$ is used.

### 4.2.3 Sliding Stability

The basic formula of the sliding safety factor (SSF) for horizontal sliding plane including seismic loads is:

$$SSF = \frac{\left(\sum \overline{V} + U + Q_v\right)\tan\phi + cA_c}{\sum H + \sum H_d + Q_h} \tag{4.35}$$

where

$\Sigma \overline{V}$ = Sum of vertical static forces excluding uplift pressure

$Q_v$ = Vertical concrete inertia forces

U = Uplift pressure force resultant

$\Sigma H_d$ =Sum of horizontal concrete inertia forces

$Q_h$ = Horizontal hydrodynamic forces

$\phi$ = Friction angle (peak value or residual value)

c = cohesion (apparent or real)

$A_C$ = Area in compression

$\Sigma H$ = Sum of horizontal static forces.

If post-tension forces are available, it should be determined first which type of load they are, i.e. active or passive. If they are active, the horizontal component of the post-tension force, $P_{dh}$, is placed in the denominator of the SSF formula. If they are passive, then $P_{dh}$ is placed in the numerator of the formula. In both cases vertical component of the anchor force, $P_v$, is placed in the numerator and should be multiplied by $\tan\phi$ (Leclerc et al., 2001).

#### 4.2.3.1 Shear Friction Method

In the shear friction method, the sliding safety factor is computed as the ratio of the maximum horizontal driving force that can be resisted (sliding resistance), R, and the summation of horizontal driving forces, ΣH (Figure 4.10).

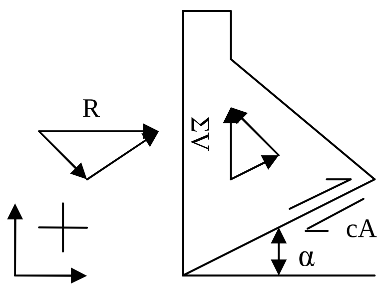


Figure 4.10 Forces Acting on Inclined Dam (Leclerc et al., 2001).

$$SSF = \frac{R}{\sum H} \tag{4.36}$$

The sum of tangential forces to the inclined plane is equal to zero:

$$R\cos\alpha + \Sigma V \sin\alpha + (\Sigma V \cos\alpha - R \sin\alpha)\tan\phi - cA = 0 \tag{4.37}$$

ΣV includes the vertical uplift pressure. When "R" is solved, the following equation is obtained:

$$R = -\sum V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi\tan\alpha)} \tag{4.38}$$

#### 4.2.3.2 Limit Equilibrium Method

When the lift joint considered is inclined, force resultants have to be computed in the normal and tangential directions to the joint to evaluate the sliding safety factor.

$$\mathrm{SSF} = \frac{\left|\left(\sum \overline{\mathrm{V}}\cos(\alpha) - \sum \mathrm{H}\sin(\alpha)\right) + \mathrm{U}\right|\tan\phi + \mathrm{cA_c}}{\left|\sum \mathrm{H}\cos(\alpha) + \sum \overline{\mathrm{V}}\sin(\alpha)\right|} \tag{4.39}$$

where

$\left|\left(\sum \overline{\mathrm{V}}\cos(\alpha) - \sum \mathrm{H}\sin(\alpha)\right)\right|$ = Sum of normal forces to the sliding plane

$\left|\sum \mathrm{H}\cos(\alpha) + \sum \overline{\mathrm{V}}\sin(\alpha)\right|$ = Sum of tangential forces to the sliding plane

U = Uplift force resultant normal to the inclined joint

α = Angle with respect to the horizontal of the sliding plane.

#### 4.2.3.3 Passive Wedge Resistance

The passive resistance of a rock wedge located at the toe of the dam can also be considered while computing the sliding safety factor (Figure 4.11). When a passive rock wedge resistance is considered, the SSF should be computed by using the shear friction method.

The peak strengths from the passive wedge and the weak joint may not be additive because the deformation rates are often unequal.

The sliding safety factor (SSF) including the effect of passive wedge can be computed by using Equation (4.40). The SSF is computed here for a horizontal joint.

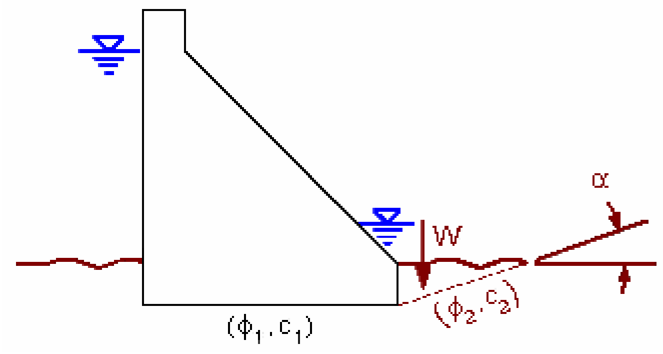


Figure 4.11 Passive Wedge Resistance (Leclerc et al., 2001).

$$SSF = \frac{\left(\sum \overline{V} + U\right)\tan\phi_1 + c_1A_1 + \left[\frac{c_2A_2}{\cos\alpha(1-\tan\phi_2\tan\alpha)} + W\tan(\alpha+\phi_2)\right]}{\sum H} \tag{4.40}$$

where W is the saturated weight of the rock wedge and $A_2$ is the area along the rock wedge failure plane.

### 4.2.4 Uplifting (Floating) Stability Analysis

The dam must resist to the vertical thrust coming from the water pressure that tend to uplift it in the case of significant immersion (Leclerc et al., 2001). The safety factor against this "floating" failure mechanism is computed as:

$$USF = \frac{\Sigma\overline{V}}{U} \tag{4.41}$$

where $\Sigma\overline{V}$ is the sum of vertical loads excluding uplift pressures (but including the weight of water above the submerged components) and U is the uplift forces due to uplift pressures.

## CHAPTER 5

## CAPABILITIES OF CADAM

### 5.1 Introduction

The computer program CADAM (Computer Analysis of Dams) was developed in the context of the research and development activities of the industrial chair on Structural Safety of Existing Concrete Dams. This chair was established in 1991 at École Polytechnique de Montréal and is funded jointly by NSERC (Natural Sciences and Engineering Research Council), Hydro-Québec and Alcan. The original work belongs to Martin Leclerc, Pierre Léger, and René Tinawi.

#### 5.1.1 Objectives

CADAM, which is used to support research and development on structural behaviour and safety of concrete dams, is a computer program that was primarily designed to provide support for learning the principles of structural stability evaluation of concrete gravity dams.

The gravity method (rigid body equilibrium and beam theory) is the basis for CADAM. Stability analyses for hydrostatic loads and seismic loads can be performed with several modelling options so that users can explore the structural behaviour of gravity dams (e.g. geometry, uplift pressures and drainage, crack initiation and propagation criteria). Within the context of training engineering students, CADAM allows (Leclerc et al., 2001):

- To confirm hand calculations with computer calculations to develop the understanding of the computational procedures.

- To conduct parametric analysis on the effects of geometry, strength of material, and load magnitude on the structural response.
- To compare uplift pressures, crack propagation and shear strength (peak, residual) assumptions from different dam safety guidelines (CDSA, 1995; USACE, 1995; FERC, 1991; FERC, 1999; and USBR, 1987).
- To study different strengthening scenarios (post-tensioning, earth backing, buttressing).

### 5.1.2 Basic Analytical Capabilities

The program supports the following analysis capabilities (Leclerc et al., 2001):

- Static Analyses: CADAM could perform static analyses for the normal operating reservoir elevation or the flood elevation including overtopping over the crest.
- Seismic Analyses: CADAM could perform seismic analysis using the pseudo-static method or the pseudo-dynamic method , which corresponds to the simplified response spectra analysis described by Chopra (1988) for gravity dams (See Section 4.1.1 and 4.1.2).
- Post-Seismic Analyses: CADAM could perform post-seismic analysis. In this case the specified cohesion is not applied over the length of crack induced by the seismic event. The post-seismic uplift pressure could either (a) build-up to its full value in seismic cracks or (b) return to its initial value if the seismic crack is closed after the earthquake.
- Probabilistic Safety Analysis (Monte-Carlo simulations): CADAM could perform a probabilistic analysis to compute the probability of failure of a dam-foundation-reservoir system as a function of the uncertainties in loading and strength parameters that are considered as random variables with specified probability density functions. A Monte-Carlo simulations

computational procedure is used (See Section 3.4). Static and seismic analysis could be considered.

- Incremental Load Analysis: CADAM could automatically perform sensitivity analysis by computing and plotting the evolution of typical performance indicator (e.g. sliding safety factor) as a function of a progressive application in the applied loading (e.g. reservoir elevation).

### 5.1.3 Modelling Capabilities

CADAM performs the analysis of a single 2D monolith of a gravity dam-foundation reservoir system subdivided into lift joints. The definition of the following input parameters is required for a typical analysis (Leclerc et al., 2001):

- Section geometry: Specification of the overall dimensions of the section geometry. Inclined upstream and downstream faces as well as embedding in the foundation (passive rock wedge) are supported.
- Masses: Concentrated masses can be arbitrarily located within or outside the cross-section to add or subtract (hole) vertical forces in a static analysis and inertia forces in a seismic analysis.
- Materials: Definition of tensile, compressive and shear strengths (peak and residual) of lift joints, base joint, and rock joint (passive rock wedge).
- Lift joints: Assign elevation, inclination and material properties to lift joints.
- Pre-cracked lift joints: Assign upstream/downstream cracks in joint(s) as initial conditions.
- Reservoir, ice load, floating debris and silt: Specification of water density, normal operating and flood headwater and tailwater elevations, ice loads, floating debris and silt pressure (equivalent fluid, frictional material at rest, active or passive).

- Drainage system: Specification of drain location and effectiveness. The stress computations could be performed through linearization of effective stresses (FERC, 1999; CDSA, 1995; USACE, 1995; USBR, 1987) or superposition of total stresses with uplift pressures (FERC, 1991).
- Post-tension cable: Specification of forces induced by straight or inclined post-tension cables installed along the crest and along the downstream face.
- Applied forces: User defined horizontal and vertical forces can be located anywhere.
- Pseudo-static analysis: Specification of the peak ground horizontal and vertical accelerations as well as the sustained accelerations. Westergaard added mass is used to represent the hydrodynamic effects of the reservoir. Options are provided to account for (a) water compressibility effects, (b) inclination of the upstream face, (c) limiting the variation of hydrodynamic pressures over a certain depth of the reservoir. Hydrodynamic pressures for the silt are approximated from Westergaard formulation for a liquid of higher mass density than water.
- Pseudo-dynamic analysis: Specification of the input data required to perform a pseudo-dynamic analysis using the simplified method proposed by Chopra (1988): (a) peak ground and spectral acceleration data, (b) dam and foundation stiffness and damping properties, (c) reservoir bottom damping properties and velocity of an impulsive pressure wave in water, (d) modal summation rules.
- Cracking options: Specification of (a) tensile strengths for crack initiation and propagation, (b) dynamic amplification factor for the tensile strength, (c) the incidence of cracking on static uplift pressure distributions (drain effectiveness), (d) the effect of cracking on the transient evolution of uplift pressures during earthquakes (full pressure, no change from static values, zero pressures in seismic cracks), (e) the evolution of uplift

pressures in the post-seismic conditions (return to initial uplift pressures or build-up full uplift pressures in seismically induced cracks).

- Load combinations: Specification of user defined multiplication factors of basic load conditions to form load combinations. Five load combinations are supported: (a) normal operating, (b) flood, (c) seismic 1, (d) seismic 2, and (e) post-seismic.
- Probabilistic Analyses: Estimation of the probability of failure of a dam-foundation-reservoir system, using the Monte-Carlo simulation, as a function of uncertainties in loading and strength parameters that are considered as random variables.
- Incremental Analysis: Automatically compute the evolution of safety factors and other performance indicators as a function of a user specified stepping increment applied to a single load condition.

### 5.1.4 Output Results

Output results are presented in three distinct formats:

1 - CADAM reports:
- Input parameters
- loads
- load combinations
- stability drawings

2 - MS Excel reports:
- Input parameters
- loads
- load combinations

3 - Graphical plots:
- Joint cracking, stresses and resultants

- Probabilistic analyses results (CDF / PDF)
- Incremental analyses results (SF versus Load)

## 5.2 Basic Modelling Information

### 5.2.1 Units

Metric units using kN for forces and metres for length or alternatively imperial units (kip, feet) could be used. The program could automatically switch from one set of unit to the other.

### 5.2.2 Two-Dimensional Modelling of Gravity Dams

CADAM performs the analysis of a 2D monolith of unit thickness (1m in metric system, or 1ft in imperial system). All input data regarding forces (masses) should, therefore, be specified as kN/m or Kips/ft, (post-tension forces, user-defined forces, concentrated masses etc.).

### 5.2.3 Basic Assumptions of the Gravity Method

The evaluation of the structural stability of the dam against sliding, overturning, and uplifting is performed considering two distinct analyses:

1. A stress analysis to determine eventual crack length and compressive stresses,
2. A stability analysis to determine the (i) safety margins against sliding along the joint considered, and (ii) the position of the resultant of all forces acting on the joint.

The gravity method is based (a) on rigid body equilibrium to determine the internal forces acting on the potential failure plane (joints and concrete-rock interface), and

(b) on beam theory to compute stresses. The use of the gravity method requires several simplifying assumptions regarding the structural behaviour of the dam and the application of the loads (Leclerc et al., 2001):

- The dam body is divided into lift joints of homogeneous properties along their length, the mass concrete and lift joints are uniformly elastic,
- All applied loads are transferred to the foundation by the cantilever action of the dam without interactions with adjacent monoliths,
- There is no interaction between the joints, that is each joint is analysed independently from the others,
- Normal stresses are linearly distributed along horizontal planes,
- Shear stresses follow a parabolic distribution along horizontal plane in the uncracked condition (Corns et al., 1988).

### 5.2.4 Sign Convention

- Positive directions of forces and stresses: The sign convention shown in Figure 5.1 is used to define positive forces and moments acting in the global coordinate system.

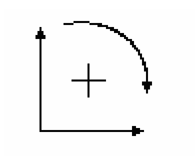


Figure 5.1 Sign Convention-1.

The sign convention shown in Figure 5.2 is used to define stresses acting on concrete (joints) elements.

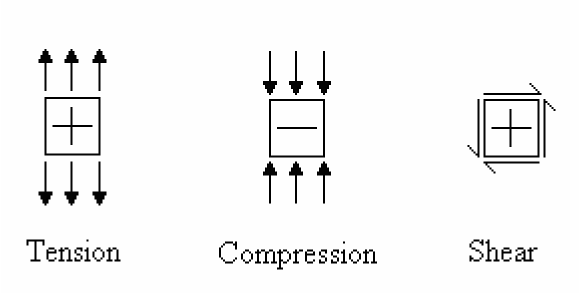


Figure 5.2  Sign Convention-2.

Positive direction of inertia forces: According to d'Alembert principle, the inertia forces induced by an earthquake are in the opposite direction of the applied base acceleration (Figure 5.3).

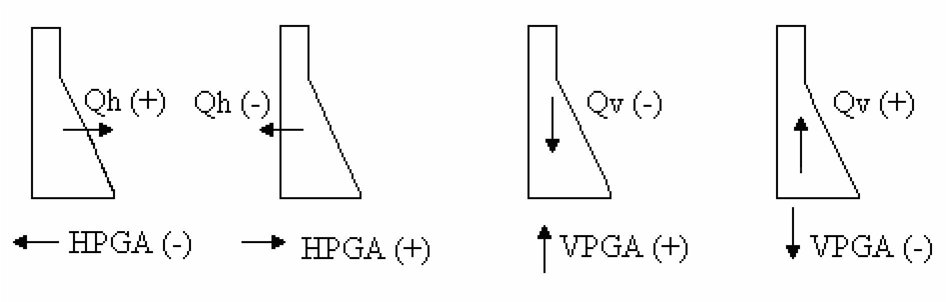


Figure 5.3  Directions of Inertia Forces (Leclerc et al., 2001).

## 5.3  Entering Data as Inputs

The meaning of various buttons in the program is shown in Figure 5.4. Also, CADAM user interface can be seen in Figure 5.5.

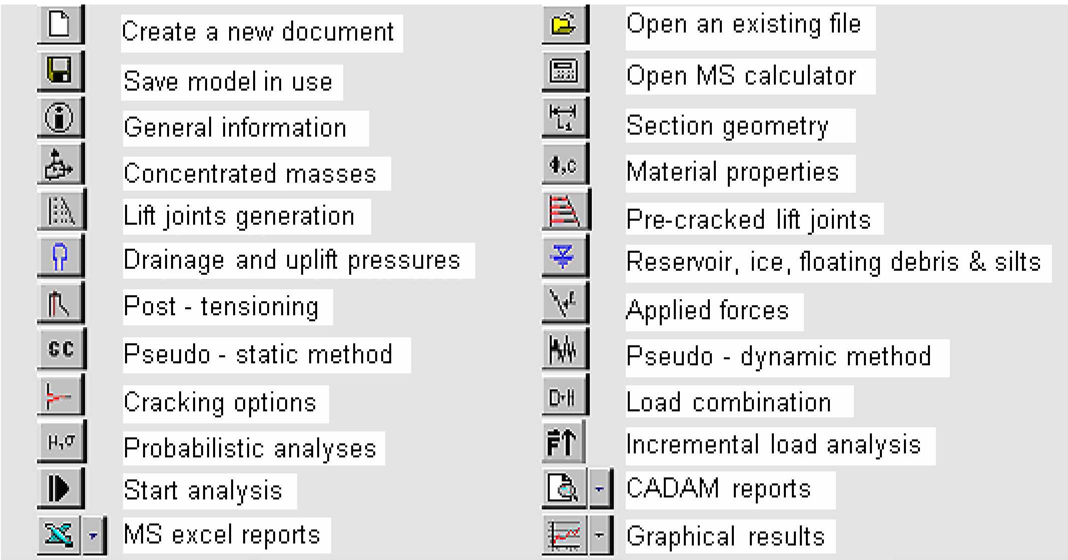


Figure 5.4 Various Buttons in CADAM (Leclerc et al., 2001).

### 5.3.1 Material Properties

#### 5.3.1.1 Lift Joints

A list of lift joint material properties can be created in CADAM. Many materials can be defined to describe variations of strength properties along the height of the dam. A lift joint is a concrete-concrete joint located above the concrete-rock interface where the base joint is located.

Minimal normal compressive stresses to mobilize cohesion: Apparent cohesion, Ca, is sometimes specified for an unbonded rough joint (with zero tensile strength) due to the presence of surface asperities. For normal compressive stresses below the minimal compressive stress ($\sigma_n^*$), two options are offered to the user (See Figure 5.6) (Leclerc et al., 2001):

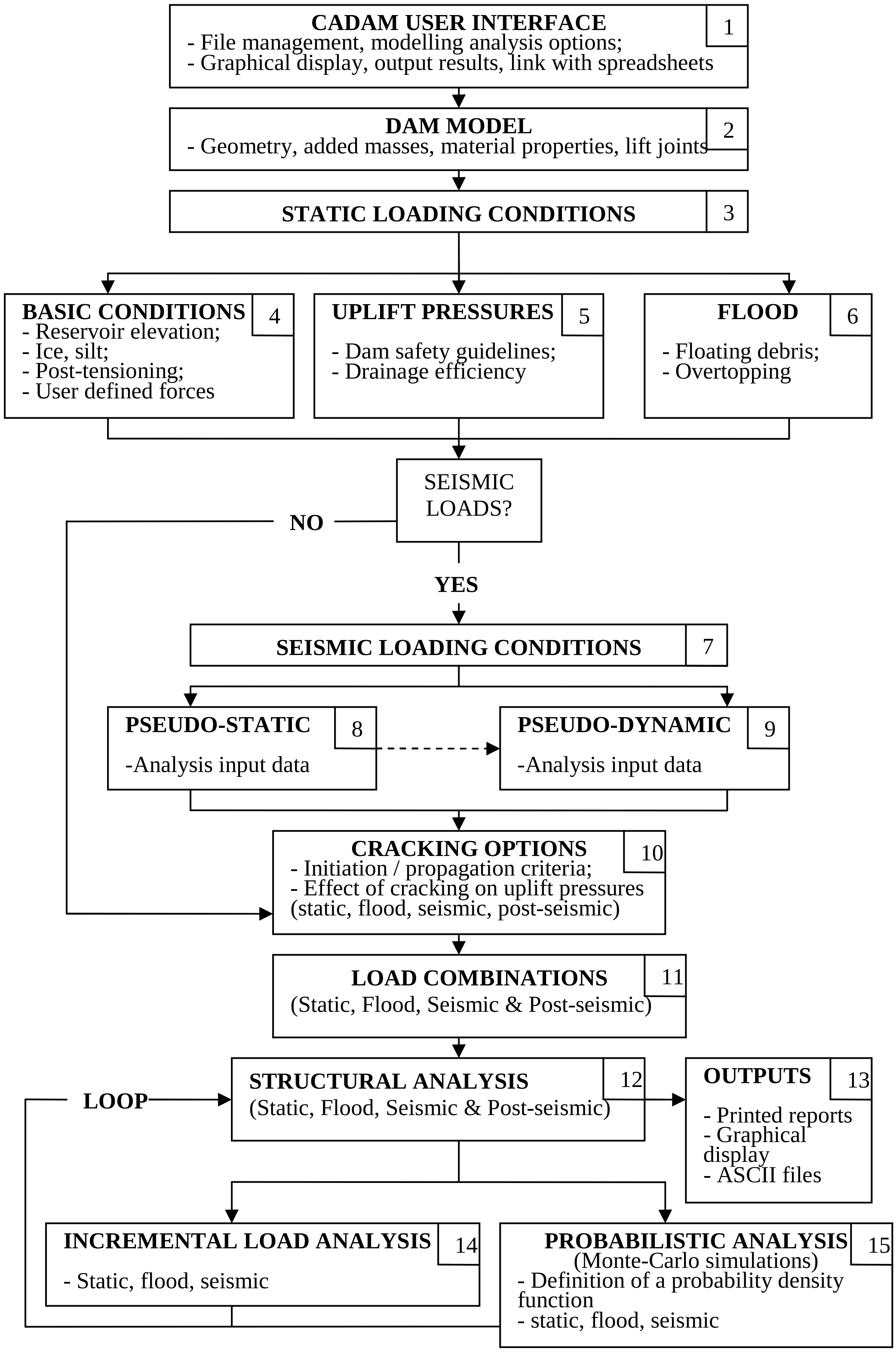


Figure 5.5 CADAM User Interface (Leclerc et al., 2001).

Option 1: The shear resistance is equal to the normal compressive stress times the friction coefficient, which is $\tan\phi$. The cohesion Ca (real or apparent) is only used if $\sigma_n \geq \sigma_n^*$.

Option 2: The shear resistance is equal to the normal compressive stress times the friction coefficient, which is $\tan(\phi+i)$. There is no cohesion for $\sigma_n < \sigma_n^*$, but a larger friction angle is used ($\phi$+i). For $\sigma_n \geq \sigma_n^*$, the friction angle $\phi$ is used with the cohesion (Ca).

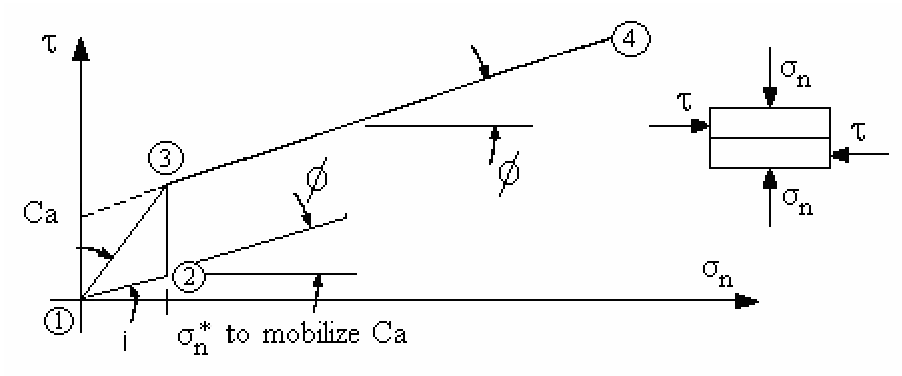


Figure 5.6 Normal Compressive Stress versus Shear Resistance (Leclerc et al., 2001).

Option 1 (Pathway 1-2): $\tau = \sigma_n \tan(\phi)$ (5.1)

Option 2 (Pathway 1-3): $\tau = \sigma_n \tan(\phi + i)$ (5.2)

Option 1 and 2 (Pathway 3-4): $\tau = \sigma_n \tan(\phi) + Ca$ (5.3) where $\tau$ is the shear resistance, $\sigma_n$ is the normal compressive stress, Ca is the apparent cohesion, $\sigma_n^*$ is the minimal compressive stress to mobilize cohesion, $\tan\phi$ is the friction coefficient and $\tan(\phi+i)$ is the transformed friction coefficient. For the pathways 1-2 and 1-3, $\sigma_n < \sigma_n^*$ whereas for the pathway 3-4, $\sigma_n \geq \sigma_n^*$.

The apparent cohesion is often derived as the shear strength for zero normal stress from the straight-line regression of a series of shear tests carried out at different normal stress intensities. However, for unbonded joint, it is obvious that the shear strength should be zero if there is no applied normal stress. A minimal value of normal compressive stresses could therefore be specified to mobilize Ca along a joint.

It should be noted that options 1 and 2 will give the same results for $\sigma_n^* = 0$ or Ca = 0, where the usual two parameters for the Mohr failure envelope is obtained.

Residual shear strength is the lowest strength which occurs after large displacements as some amount of stress stay in the material because of the deformation as shown in Figure 5.7 in which $\sigma'$ is the effective normal stress and $\delta_h$ and $\delta_v$ are the horizontal and vertical displacements, respectively.

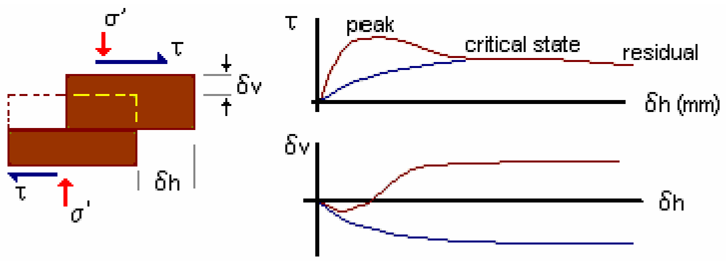


Figure 5.7  Stages of Shear Strength (Davison and Springman, 2000).

As the load is removed from a rod which is streched beyond the yield point, the rod does not regain its original length, so it had been permanently deformed. However, after the load is removed, all stresses disappear. It should not be assumed that this will always be the case. When only some of the parts of an indeterminate structure undergo plastic deformations, or when different parts of the structure will not, in general, return to zero after the load has been removed. Residual stresses will remain in the various parts of the structure. Residual stresses due to welding, casting, and hot rolling may be quite large. These stresses may be removed by reheating and then allowing it to cool slowly (Beer and Johnston, 1981).

#### 5.3.1.2 Base Joint

The material strength properties at the concrete-rock interface are specified, using the same models (options) as those for lift joints.

#### 5.3.1.3 Rock Joint

Parameters including the contribution of a passive wedge resistance to the sliding resistance of the dam can be defined in the case where the dam is embedded in the foundation. If the tailwater elevation is above the rock failure plane, CADAM computes automatically the uplift pressure acting on the failure plane (Leclerc et al., 2001).

### 5.3.2 Uplift Pressures and Drainage System

#### 5.3.2.1 Uplift Pressures – Computation of Effective Stresses

To perform the computation of effective stresses and related crack length, uplift pressures could be considered (Leclerc et al., 2001):

- As an external load acting on the surface of the joint (FERC, 1999; USACE, 1995; CDSA, 1995; USBR, 1987 (crack propagation)): In this case, normal stresses are computed using beam theory considering all loads acting on the free-body considered (including the uplift pressure resultant). The computed effective normal stresses then follow a linear distribution along the joint even in the presence of a drainage system that produces a non-linear distribution of uplift pressures along the joint. The effective tensile stress at the crack tip is compared to the allowable tensile strength to initiate or propagate tensile cracks.
- As an internal load along the joint (FERC, 1991): In this case, normal stresses are computed considering all loads acting on the free-body considered but excluding uplift pressure. The computed total stresses are then added along the joint to the uplift pressures. Effective stresses computed using this procedure follow a non-linear distribution along the joint in the presence of a drainage system. For example, in the case of a no-tension material, crack initiation or propagation takes place when the uplift pressure is greater than the total stress acting at the crack tip.

#### 5.3.2.2 Drain Effectiveness – User specified value

The position of the drains, the drain effectiveness and the elevation of the drainage gallery can be specified by activating related windows according to particular versions of Dam Safety Guidelines (USACE, 1995; USBR, 1987 for uplift pressures considered as external loads; FERC 1991 for uplift pressures considered as internal loads). When elevation of drainage gallery is above tailwater elevation, the reference elevation to determine the pressure head at drain line becomes the elevation of the gallery (FERC, 1991; FERC, 1999; USBR, 1987; USACE, 1995).

### 5.3.3 Pseudo – Static Seismic Analysis

#### 5.3.3.1 Basic Assumption – Rigid Body Behavior

The inertia forces induced by the earthquake are computed from the product of the mass and the acceleration in a pseudo-static seismic analysis. The dynamic amplification of inertia forces along the height of the dam due to its flexibility is neglected. The dam-foundation-reservoir system is thus considered as a rigid system with a period of vibration equal to zero. The analysis interface is given in Figure 5.8.

At the initial state before the earthquake, each seismic analysis begins with a static analysis to determine the initial condition before applying the seismically induced inertia forces. If cracking takes place under the static load conditions, the crack length and updated uplift pressures (if selected by the user) are considered as initial conditions for the seismic analysis (Leclerc et al., 2001).

Stress and stability analyses: The basic objective of the stress analysis is to determine the tensile crack length that will be induced by the inertia forces applied to the dam. Horizontal and vertical peak ground acceleration values perform the stress analysis. The basic objective of the stability analysis is to determine the sliding and overturning response of the dam. The pseudo-static method does not recognize the oscillatory nature of seismic loads. It is, therefore, generally accepted to perform the stability calculation using sustained acceleration values taken as 0.67 to 0.5 of the peak acceleration values. In this case, sliding safety factors are computed considering crack lengths determined from stress analysis.

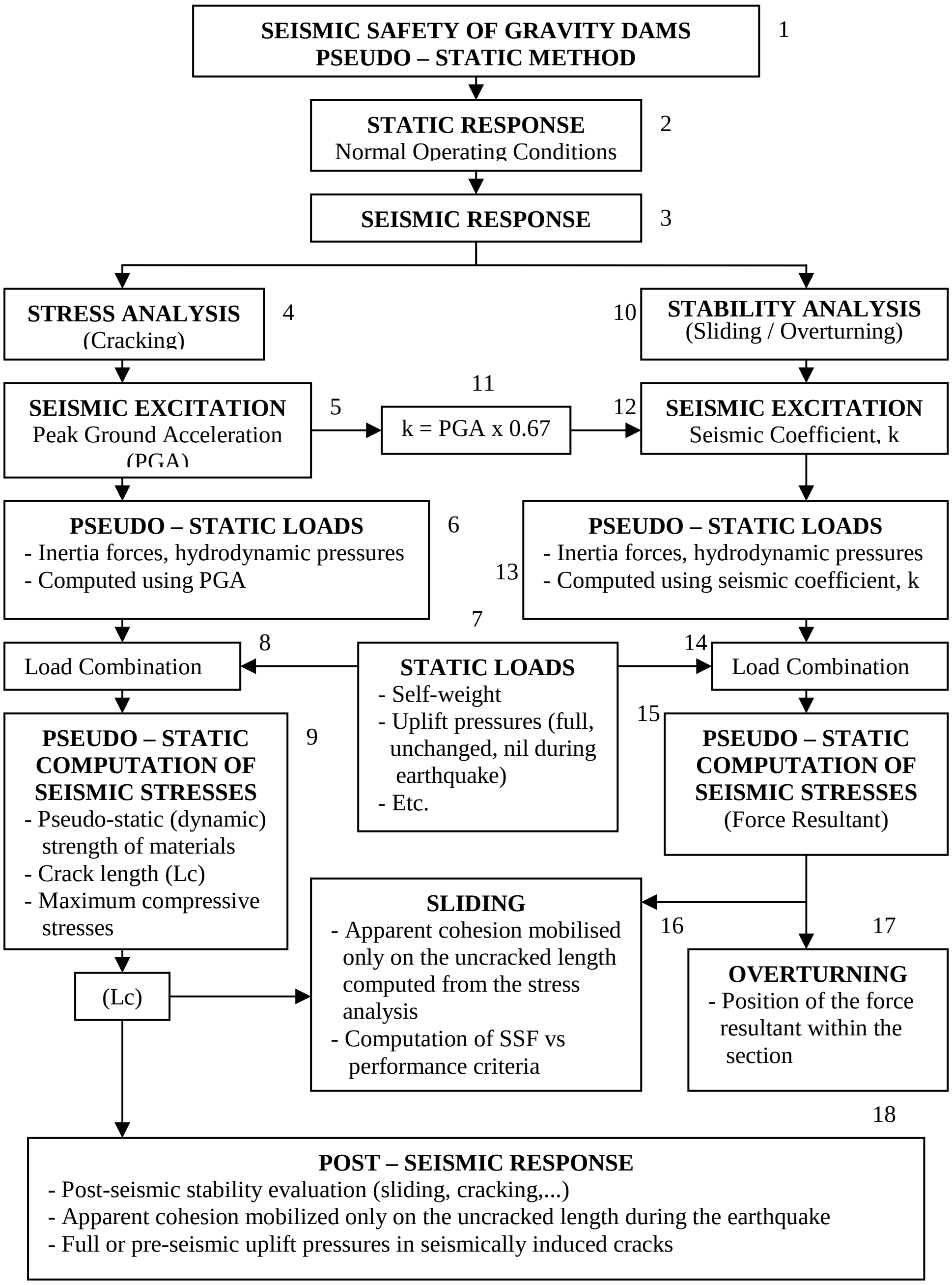


Figure 5.8 Pseudo – Static Analysis (Leclerc et al., 2001).

#### 5.3.3.2 Hydrodynamic Pressures (Westergaard Added Masses)

The hydrodynamic pressures acting on the dam are modelled as added mass (added inertia forces) according to the Westergaard formulation. Options have been provided for (Leclerc et al., 2001):

● Correction for water compressibility: According to the predominant period of the base rock acceleration, a correction factor is applied to the Westergaard formulation (USACE, 1995; Corns et al. 1988).

● Inclination of the upstream face: The hydrodynamic pressures act in a direction normal to the surface that is accelerated against the reservoir. To transform these pressures to the global coordinate system two options have been provided using either the cosine square of the angle of the upstream face about the vertical or the function derived from USBR (1987) as given by Corns et al. (1988).

● A reservoir depth beyond which Westergaard added pressure remains constant: Beyond a depth, there is no more significant variation of hydrodynamic pressure with depth. The value computed at that depth is then maintained constant from that point to the bottom of the reservoir.

### 5.3.4 Pseudo – Dynamic Seismic Analysis

#### 5.3.4.1 Basic Assumption – Dynamic Amplification

The pseudo-dynamic analysis is based on the simplified response spectra method as described by Chopra (1988). A pseudo-dynamic analysis is conceptually similar to a pseudo-static analysis except that it recognises the dynamic amplification of the inertia forces along the height of the dam. However, the oscillatory nature of the amplified inertia forces is not considered. That is why the stress and stability analyses are performed with the inertia forces continuously applied in the same direction (Leclerc et al., 2001).

#### 5.3.4.2 Dam Properties

To ensure the accuracy of the pseudo-dynamic method, the structure has to be divided in thin layers to perform numerical integrations. The user may specify a number of divisions up to 301. The dynamic flexibility of the structure is modeled with the dynamic concrete Young's modulus ($E_s$). The dam damping ($\xi_1$) on rigid foundation without reservoir interaction is necessary to compute the dam foundation reservoir damping ($\bar{\xi}_1$). Any change to these basic parameters affect the fundamental period of vibration and the damping of the dam-foundation-reservoir system computed in this dialog window (Leclerc et al., 2001).

#### 5.3.4.3 Reservoir Properties

The wave reflection coefficient ($\alpha$) is the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a vertical propagating pressure wave incident on the reservoir bottom. A value of $\alpha = 1$ indicates that pressure waves are completely reflected, and smaller values of $\alpha$ indicate increasingly absorptive materials. In CADAM, the value of 0.5 is used. The velocity of pressure waves in water is in fact the speed of sound in water (1440 m/s). Westergaard added mass procedure, with possibility of a correction for an inclined face, is used for the downstream reservoir and the silt (Leclerc et al., 2001).

#### 5.3.4.4 Modal Combination

Because the maximum response in the natural vibration mode and in higher modes doesn't occur at the same time, a modal combination has to be considered. Four options are offered to the user: (i) Only the first mode; (ii) Only the static correction computed for higher modes; (iii) SRSS (square-root-of-the-sum-of-squares of the first mode and static correction for higher modes); or (iv) Sum of absolute values which provides always conservative results. The SRSS combination is often considered to be preferable (Leclerc et al., 2001).

### 5.3.5 Cracking Options

#### 5.3.5.1 Tensile Strength – Crack Initiation and Propagation Criteria

Tensile strength to be used to determine the cracking response along the joints can be specified. The user should first indicate if cracking is allowed to take place during the analysis. In cracking options of CADAM, there are two criteria for crack initiation and crack propagation. The crack initiation and crack propagation criteria can be chosen either by setting a tensile initiation (or propagation) strength or by setting the tensile initiation (or propagation) strength equal to zero. The user defined coefficients that are used in CADAM are presented in Table 5.1.

$$ft_{ini} = ft_{joint}/\kappa_{ini} \tag{5.4}$$

$$ft_{prop} = ft_{joint}/\kappa_{prop} \tag{5.5}$$

where $ft_{ini}$, $ft_{prop}$, and $ft_{joint}$ are tensile initiation, propagation, and joint strengths; $\kappa_{ini}$ and $\kappa_{prop}$ are the user defined coefficients for cracking, respectively.

Table 5.1 User Defined Coefficients for Cracking (Leclerc et al., 2001).

| Coefficients | Usual | Flood | Seismic | Post – Seismic |
|---|---|---|---|---|
| $\kappa_{ini}$ | 3 | 2 | 1 | 3 |
| $\kappa_{prop}$ | 10 | 10 | 10 | 10 |

In CADAM, the dynamic magnification of tensile strength can be assigned for seismic analysis in "Cracking Options" window. The tensile strength of concrete under rapid loading during a seismic event is larger than that under static loading. The tensile strength could be magnified by a factor for seismic crack initiation and propagation criteria. By default, this factor is given as 1.5 (Leclerc et al., 2001).

USBR (1987) uses the following simplified equation for the minimum allowable compressive (normal) stress at the upstream face ($\sigma_{zu}$) from uplift forces to determine crack initiation (not propagation) is used in USBR (1987):

$$\sigma_{zu} = pwh - f_t / \mathrm{s} \tag{5.6}$$

where $\sigma_{zu}$ is equal to the absolute value of the stress at the upstream face induced from uplift forces minus the allowable tensile stress. $f_t$ is the tensile strength of the material and *s* is the safety factor. The term *pwh* represents the transformed uplift pressure at the heel of the dam considering the effect of a drain reduction factor (*p*). Cracking initiates at the heel of the dam when the compressive stress $\sigma_z$ does not achieve the minimum compressive stress $\sigma_{zu}$ value. CADAM computes automatically the drain reduction factor *p* when the USBR guideline is selected.

Figure 5.9 is the graph that may also be used to obtain the drain reduction factor (*p*) (Leclerc et al., 2001). The procedure is as follows:

1. Calculate ratios ($X_d$/L) and ($H_3$-$H_2$)/($H_1$-$H_2$)
2. Obtain value of *p* from graph
3. Correct *p* for tailwater using equation $[p(H_1-H_2)+H_2]/H_1$

where

*p*: drain reduction factor

$H_1$: reservoir pressure head on the upstream face

$H_2$: tailwater pressure head on the downstream face

$H_3$: pressure head at the line of the drains

$X_d$: distance to the drain from the upstream face

L: horizontal length from upstream to downstream face

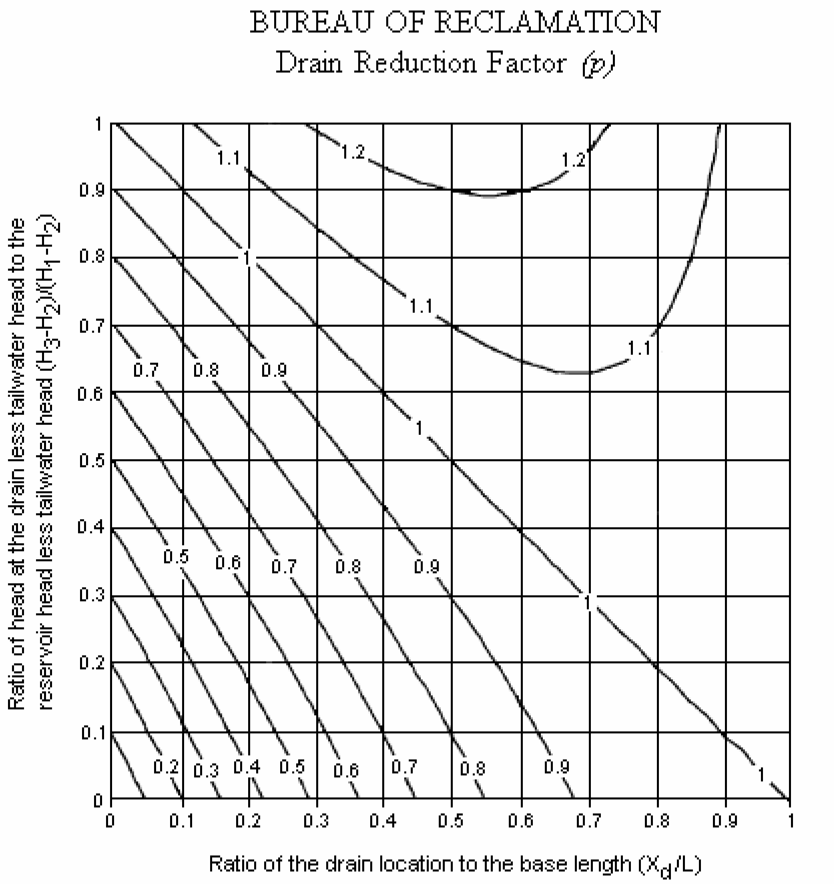


Figure 5.9 Determination of Drain Reduction Factor ($p$) (Leclerc et al., 2001; Source:USACE, 1995).

When cracking is allowed, a distinction is made between the criteria for crack initiation and crack propagation (Figure 5.10). After crack initiation, say at the upstream end of a joint where stress concentration is minimal; it is likely that stress concentration will occur near the tip of the propagating crack (ANCOLD, 1991). The allowable tensile strengths for crack initiation and propagation are specified for different load combinations: (a) usual normal operating, (b) flood, (c) seismic (1 and 2), and (d) post-seismic (Leclerc et al., 2001).

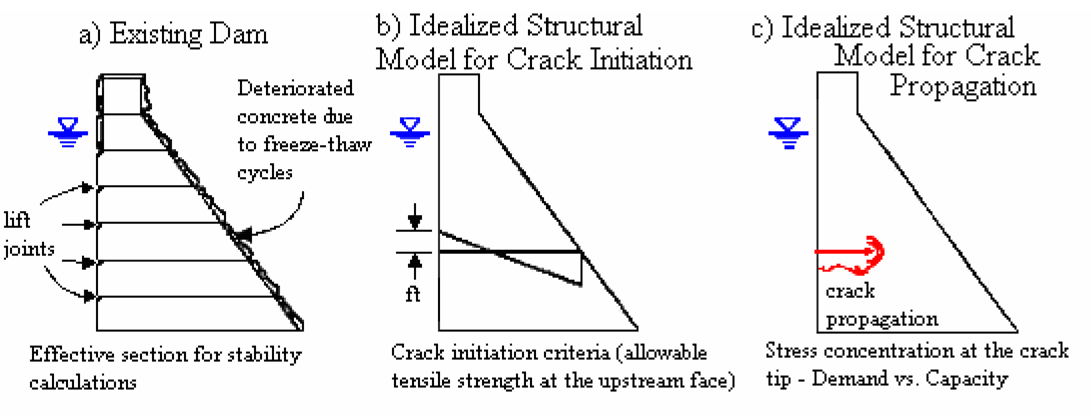


Figure 5.10 Criteria of Cracking (Leclerc et al., 2001).

### 5.3.6 Required Safety Factors

For each load combination, the required safety factors to ensure an adequate safety margin for structural stability are specified. These values are not used in the computational algorithm of the program. They are reported in the output results to facilitate the interpretation of the computed safety factors in comparison with the corresponding allowable values. In CADAM, required safety factors are already available but these can be changed manually if needed. Values of the safety factors are presented in Table 5.2. Also allowable stress factors are attached to this table.

Table 5.2 Safety Cases for Different Loadings (Leclerc et al., 2001).

| Safety Cases | Usual | Flood | Seismic | Post - seismic |
|---|---|---|---|---|
| Peak Sliding Factor (PSF) | 3.00 | 2.0 | 1.3 | 2.0 |
| Residual Sliding Factor (RSF) | 1.50 | 1.3 | 1.0 | 1.1 |
| Overturning Factor (OF) | 1.20 | 1.1 | 1.1 | 1.1 |
| Uplifting Factor (UF) | 1.20 | 1.1 | 1.1 | 1.1 |
| ASF* in tension | 0.00 | 0.50 | 0.909 | 0.667 |
| ASF* in compression | 0.333 | 0.50 | 0.909 | 0.667 |

* ASF: Allowable Stress Factor (used with allowable strength)

### 5.3.7 Probabilisitic Safety Analysis (Monte – Carlo Simulations)

#### 5.3.7.1 Overview of CADAM Probabilistic Analysis Module

- Objectives: The objectives of CADAM probabilistic analysis module is to compute the probability of failure of a dam-foundation-reservoir system as a function of the uncertainties in loading and strength parameters that are considered random variables.
- Computational procedure-Monte Carlo Simulation: Due to concrete cracking and related modifications in uplift pressures, the stress and stability analysis of a dam is in general a non-linear process. Monte Carlo simulation is used as the computational procedure to perform the probabilistic non-linear analysis in CADAM. Monte Carlo simulation

technique involve sampling at random to simulate artificially a large number of experiments and to observe the results (See Figure 5.11).

#### 5.3.7.2 CADAM Input Parameters for a Probabilistic Analysis

In this part of CADAM, input parameters for a probabilistic analysis are specified in a list. This list is composed of five strength parameters and nine loading parameters, which are:

- Strength Variable Parameters:

1. Tensile strength;
2. Peak cohesion;
3. Residual cohesion;
4. Peak friction coefficient;
5. Residual friction coefficient;

- Loading Variable Parameters:

6. Normal upstream reservoir elevation;
7. Flood upstream reservoir increase;
8. Silt elevation;
9. Silt volumetric weight;
10. Drain efficiency;
11. Floating debris;
12. Ice load;
13. Last applied force;
14. Horizontal peak ground acceleration.

Monte-Carlo simulations require that random variable must be independent to each other. CADAM will thus consider that the cohesion (real or apparent) is independent of the tensile strength, which may not be the case. CADAM users have to be aware of the assumptions concerning random variables before proceeding with probabilistic analyses.

In the simulation used in CADAM software which this thesis deals with, samples of basic noncorrelated variables according to their corresponding probabilistic characteristics are generated and assigned as inputs into the probabilistic analysis. If the distribution is unknown, a probability distribution function should be fitted to the available data. At this point, chi-square test with frequency analysis can be applied which is the case in this thesis for the upstream water elevations.

### 5.3.8 Incremental Load Analysis

In dam safety evaluation there is most often high uncertainties with the loading intensity associated with extreme events with very long return periods: (a) the reservoir elevation corresponding to the 10,000 yrs event or Probable Maximum Flood (PMF), and (b) the peak ground acceleration (PGA) (spectral ordinates) corresponding to the 10,000 yrs event or the Maximum Credible Earthquake. It is essential to know the evolution of typical sliding safety factors (for peak and residual strengths) as well as performance indicators (e.g. crack length) as a function of a progressive increase in the applied loading (i.e. reservoir elevation or PGA). It is then possible to evaluate for which loading intensity, safety factors will fall below allowable values such that proper action could be planned. The reservoir elevation or PGA (spectral ordinate) that will induce failure can also be readily evaluated (safety factors just below one). The concept of imminent failure flood is used in dam safety guidelines. A parallel could be established with earthquakes where the concept of imminent failure earthquake (ground motion) could be developed. There are also uncertainties for other loads, such as ice forces acting under usual load combination, e.g. magnitude of ice forces (Leclerc et al., 2001).

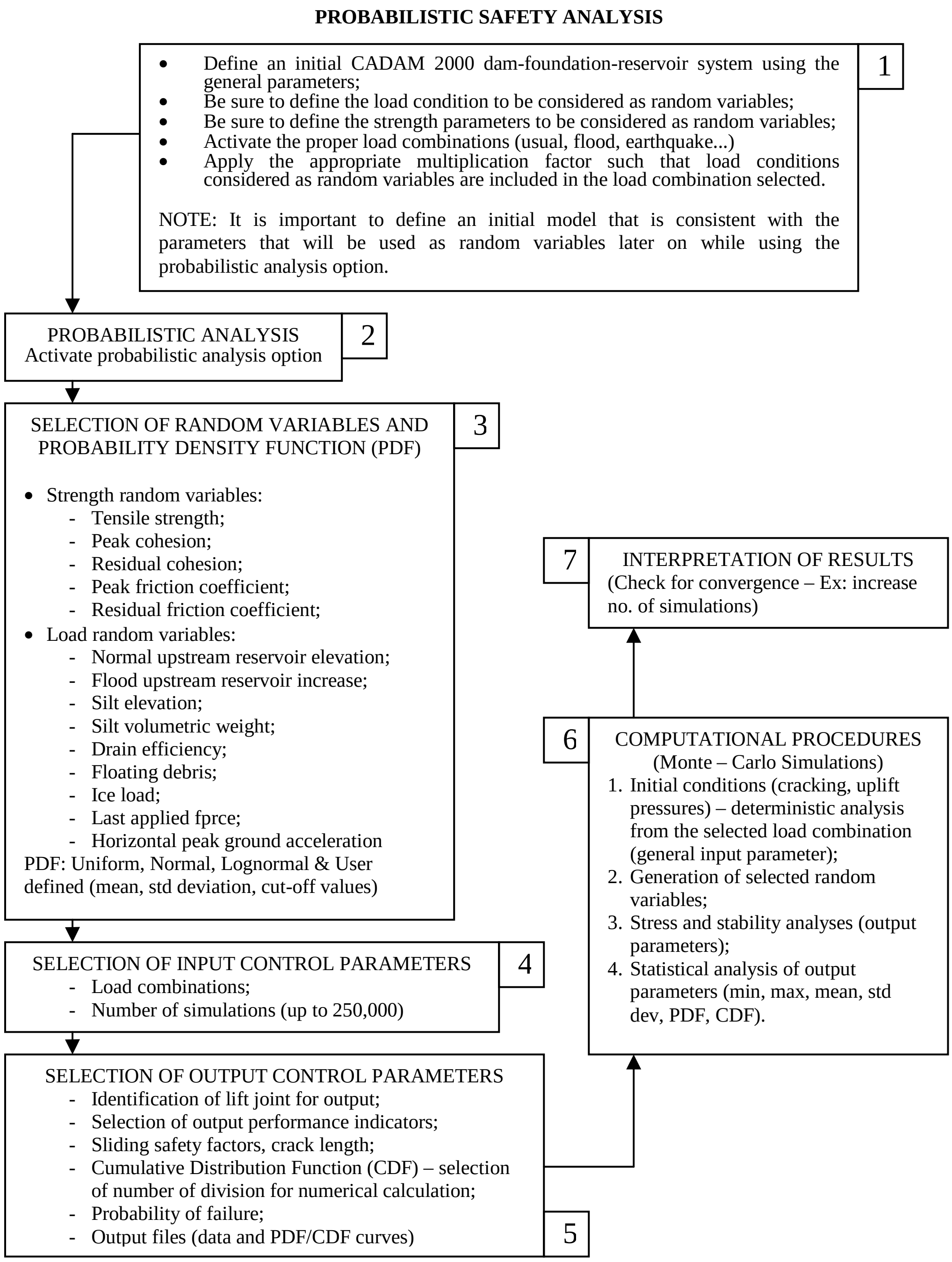


Figure 5.11 Probabilistic Safety Analysis in CADAM (Leclerc et al., 2001).

## 5.4 Stress and Stability Analyses

The objectives of structural analyses of dam-foundation reservoir systems are:

- interpreting field data, explain the observed behaviour and investigate deterioration and damage mechanisms.
- predicting the structural stability and identify possible failure mechanisms under usual, unusual (e.g. flood), and extreme (e.g. seismic) loading scenarios.
- assisting in the development of remedial work, corrective measures, and most efficient rehabilitation methods of existing facilities.

In a safety evaluation, the engineer must always relate the physical reality of the actual dam-foundation-reservoir system (Figure 5.10) to the assumptions made in developing structural models to study the potential failure mechanisms and to uncertainties related to those models as well as the required input parameters. Computer programs, such as CADAM allows to perform parametric analyses to develop confidence intervals in which appropriate decisions could be taken regarding the safety of a particular dam and the need for remedial actions to increase safety, if necessary (Leclerc et al., 2001).

### 5.4.1 Performing the Structural Analysis

After “Start Analysis” option is selected, the structural analysis begins. The first step performed by CADAM is to process the geometry data to compute joint lengths and tributary areas (volumes). Then all the loads acting on the structure are computed. For each load combination, the normal force resultant, the net driving shear (tangential) force resultant, and the overturning moments are computed about the centre line of the uncracked joint ligament.

Using these forces resultants:

(a) The stress analysis is first performed to compute the potential crack length and compressive stresses along each joint;

(b) The sliding stability is performed along each joint considering the specified shear strength joint properties;

(c) The overturning stability is performed by computing the position of the resultant of all forces along each joint. In CADAM, $L_{FR}$ is expressed in a percentage of the total length of the joint from the upstream end.

$$L_{FR} = \frac{\sum M_{Upst.}}{\sum V} \tag{5.7}$$

where $\Sigma M_{Upst.}$ is the summation of moments about the upstream end of the joint and $\Sigma V$ is the summation of vertical forces including uplift pressures.

(d) Additional performance indicators, such as the floating (uplifting) safety factor are computed.

Closed form formulas for crack length computations: Closed form formulas have been developed to compute crack length for simple undrained cases considering a no-tension material for a horizontal crack plane (Corns et al., 1988; USBR, 1987; FERC, 1991) and even for some more complicated cases considering drainage, and tensile strength within the assumption of beam theory (ANCOLD, 1991; Lo et al., 1990 with linear distribution of normal stresses). However, to consider a range of complex cases, such as inclined joints with various drainage conditions, it is more efficient to compute the crack length from an iterative procedure (USBR, 1987).

Iterative Procedure for Crack Length Calculation: CADAM uses an iterative procedure summarized in Figure 5.12 to compute the crack length. Two different crack criteria (initiation and propagation) are supported by CADAM.

### 5.4.2 Safety Evaluation for Static Loads

Load Conditions, Combinations and Safety Evaluation Format: By proper definition of basic loading condition parameters and multiplication factors to form load combinations, a variety of loading scenarios could be defined to assess the safety of the dam-foundation-reservoir system. In CADAM, if there is inclination at the base of the dam, then the sliding safety factors for inclined joints can be computed either from the limit equilibrium method or the shear friction method. A choice should be made between these two in “Load Combinations” window of this software.

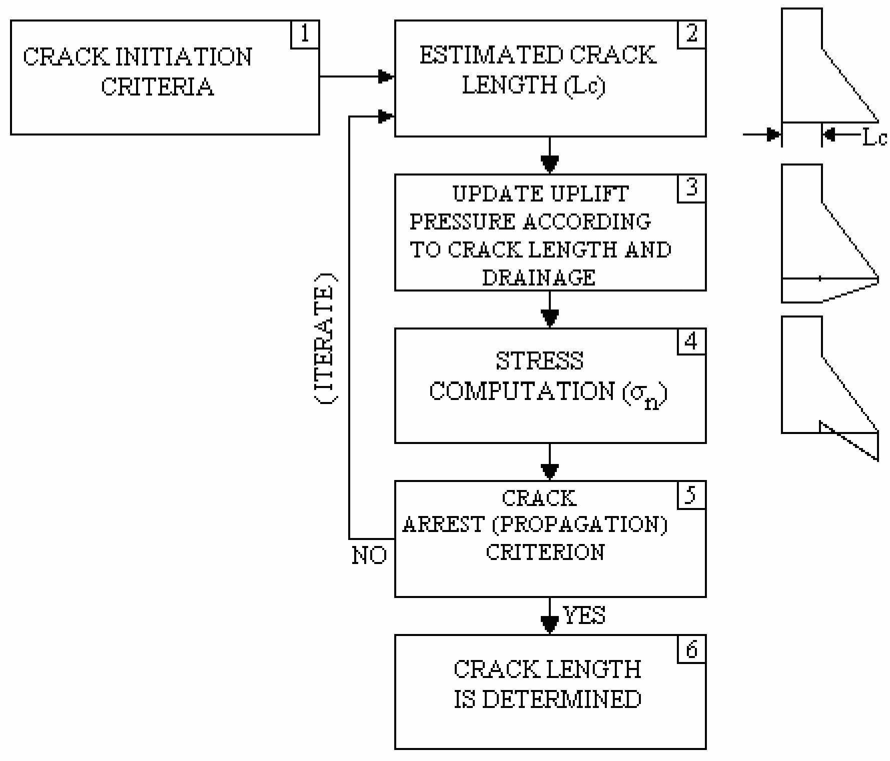


Figure 5.12 Procedure for Crack Length Computations (Leclerc et al., 2001).

Tailwater Condition: USACE (1995) mentions that the effective tailwater depth used to calculate pressures and forces acting on the downstream face of an overflow section may be reduced to 60% of the full water depth due to fluctuations in the stilling basin (hydraulic jump). However, the full tailwater depth is to be used to calculate the uplift pressure at the toe of the dam regardless of the overflow conditions.

To model an effective tailwater depth of 60% of the full depth, CADAM Load Combinations window allows to specify different multiplication factors; hydrostatic upstream, hydrostatic downstream, and uplift pressures. In this case the tailwater uplift pressure is computed using the full tailwater depth while the 0.6 factor applies to the tailwater hydrostatic pressures (and water weight on the downstream face).

Limit analysis (ANCOLD, 1991): The Australian National Committee on Large Dams (1991) presented a dam safety evaluation format based on a limit state approach. Various magnification and reduction factors are applied to basic load conditions and material strength parameters to reflect related uncertainties. By adjusting the input material parameters and applying the specified load multiplication factors, CADAM could be used to perform limit analysis of gravity dams as described by ANCOLD (1991).

Vertical Acceleration of Reservoir Bottom and Hydrostatic Pressure: In addition to the vertical motion of the upstream face of the dam, some analysts consider the effect of the vertical acceleration of the reservoir bottom on the applied hydrostatic pressures. According to d'Alembert principle, an upward vertical acceleration of the rock is going to produce an increase in the effective volumetric weight of water ($\gamma_e = \rho_w (g + acc_v)$) for an incompressible reservoir, where $\rho_w$ is the volumetric mass of water, g is the acceleration of gravity, and $acc_v$ is the vertical acceleration of the rock. The increase in the volumetric weight of water produces an increase in the initially applied hydrostatic pressures on the submerged parts of the dam. In reverse, rock acceleration directed downward produces a reduction in the effective volumetric weight of water ($\gamma_e = \rho_w (g - acc_V)$) and related initial hydrostatic pressures. These considerations are independent of the Westergaard hydrodynamic pressure computations (Leclerc et al., 2001). CADAM includes the effect of the vertical rigid body acceleration of the reservoir bottom on the initial hydrostatic pressures.

Uplift Pressures in Cracks During Earthquakes: Due to the lack of historical and experimental evidences, there is still a poor knowledge on the transient evolution of uplift pressures in cracks due to the cyclic movements of the crack surfaces during earthquakes.

- ICOLD (1986) mentions that the assumption that pore pressure equal to the reservoir head is instantly attained in cracks is probably adequate and safe.
- USACE (1995) and FERC (1991) assume that uplift pressures are unchanged by earthquake load (i.e at the pre-earthquake intensity during the earthquake).
- USBR (1987) gives that when a crack develops during an earthquake event, uplift pressure within the crack is assumed to be zero.
- CDSA (1997) states that in areas of low seismicity, the uplift pressure prior to the seismic event is normally assumed to be maintained during the earthquake even if cracking occurs. In areas of high seismicity, the assumption is frequently made that the uplift pressure on the crack surface is zero during the earthquake when the seismic forces tend to open the crack.

CADAM provides three options to consider the transient evolution of uplift pressures in cracks during earthquakes (Figure 5.13): (a) no uplift pressures in the opened crack, (b) uplift pressures remain unchanged, (c) full uplift pressures applied to the crack section irrespective of the presence of drains (Leclerc et al., 2001).

### 5.4.3 Safety Evaluation for Seismic Conditions

Concrete Inertia Forces in Pseudo-Static Analysis: The horizontal and vertical concrete inertia forces are computed as the product of the concrete mass by the applied base accelerations in the horizontal and vertical directions, respectively (peak ground acceleration or sustained acceleration).

Hydrodynamic Pressures: The formulation implemented in CADAM to model hydrodynamic pressures for seismic analysis using the pseudo-static method are available (See Section 4.1.1).

Pseudo-Dynamic Analysis: In pseudo-dynamic analyses, the hydrodynamic pressures acting on the upstream face are computed from an analytical formulation taking into account water compressibility as derived by Chopra and Fenves (Chopra, 1988; Fenves and Chopra, 1984; 1985a,b; 1986; 1987). Any slope of the upstream face is neglected in these calculations. However, the weight of water above the inclined portion is modified according to the imposed vertical accelerations at the base of the dam. The added hydrodynamic pressures acting on the downstream face are computed only in the horizontal direction using the Westergaard formulation for a sloping face. In the vertical direction, the dam is assumed rigid. The concrete inertia forces are computed as the product of the vertical base acceleration and the concrete mass. The incidence of the vertical acceleration of the reservoir bottom on the initial hydrostatic pressure could be included using a similar approach to that used in the pseudo-static method (Leclerc et al., 2001).

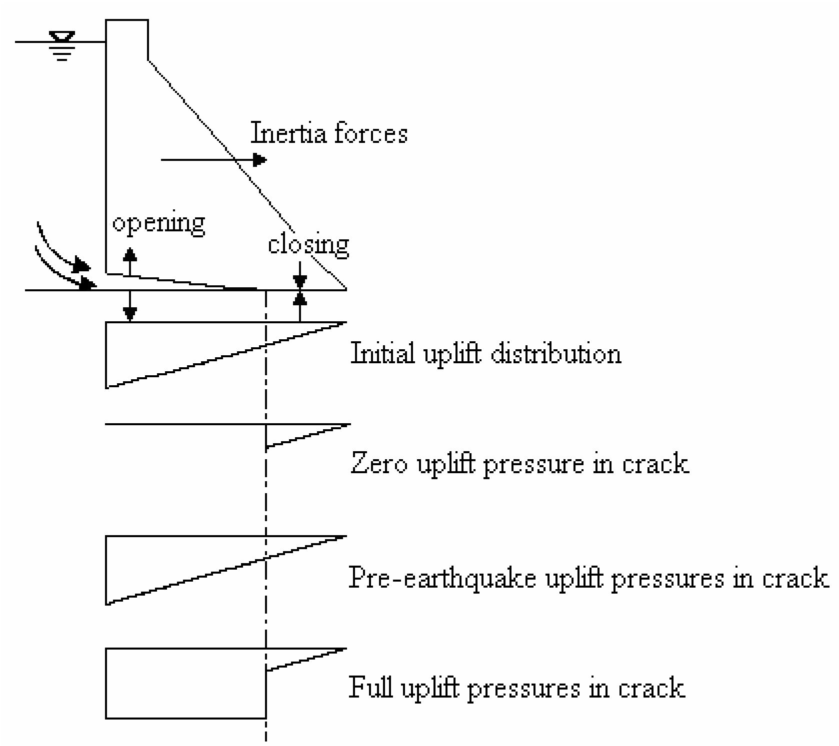


Figure 5.13 Transient Evolutions of Uplift Pressures in Seismically Induced Crack (Leclerc et al., 2001).

Crack length computation: In a pseudo-dynamic analysis, the moment and axial force acting on the lift joint considered are computed from the selected modal combination rule. The resulting moment and axial force are then used to compute the related stresses and crack length. This approach is generally conservative. In linear (uncracked) analysis, it is more appropriate to compute stresses separately for the first mode and the higher modes and then apply the modal combination rule to stresses. However, this approach, adopted in linear analysis, is not suitable to estimate crack length in a consistent manner with pseudo-static calculations, especially if uplift pressures are to be varied within the seismic crack (e.g. No uplift pressure in an opened crack). Moreover, it is assumed that the period of vibration of the dam is unaffected by cracking which is obviously an approximation that might be overcome only if transient nonlinear dynamic analysis are considered (Leclerc et al., 2001).

### 5.4.4 Safety Evaluation for Post-Seismic Conditions

Effect of Seismically Induced Cracks on Sliding Safety: The cohesion (real or apparent) is considered null along the seismically induced crack length to compute the sliding safety factors in post-seismic condition.

Uplift Pressure in Seismically Induced Cracks for Post-Seismic Analysis:

- CDSA (1997) mentions that the disruption of the dam and/or the foundation condition due to an earthquake should be recognized in assessing the internal water pressure and uplift assumptions for the post-earthquake case.
- According to CDSA (1997), a conservative assumption for post-seismic uplift pressures would be to use the full reservoir pressure in earthquake-induced cracks in the post-seismic safety assessment. However, as an alternative, the post-seismic load case could be defined from the calculation of the crack mouth opening width, crack length and drainage conditions to delineate uplift pressures.
- According to FERC (1991), the uplift pressures to be used for the post-seismic condition are the same that were acting prior to the earthquake.

That is the pre-earthquake uplift pressure intensity is used immediately after the earthquake.

Crack Length Computation in Post-Seismic Analysis: If the full reservoir pressure is assumed to be developed in seismically induced crack, a new calculation of the crack length (stress analysis) must be performed to obtain a solution that is in equilibrium. In that case the seismically induced crack may propagate more, or may close along the joint (Leclerc et al., 2001).

# CHAPTER 6

# CASE STUDY

This study deals with the probabilistic safety analysis of an existing concrete gravity dam in Turkey. Porsuk Dam is selected as a model study. It is a concrete gravity dam which is situated on the Porsuk Stream, tributary of Sakarya River, 25 km southwest from Eskişehir (See Figure 6.1). It is used for irrigation, flood control, domestic, and industrial water supply. The construction was started in 1966 and completed in 1972 (Orhon et al., 1991).

## 6.1 Input File for CADAM

Most of the inputs and properties of Porsuk Dam are listed in Table 6.1. Apart from the available data for the software to be run, some of the inputs are obtained by combining the available data with the related information present in other references.

### 6.1.1 Determination of Vertical Ground Acceleration

As the software CADAM can calculate the stability and reliability against seismic action, the input for horizontal and vertical peak ground accelerations are needed. Porsuk Dam is located in the second seismic zone (GDDAERD, 2004 and MTA, 2004). After the related horizontal peak ground acceleration is obtained (RTMPWS, 1997), the vertical peak ground acceleration is obtained using Newmark et al.'s (1973) relation who state that the vertical to horizontal ratio of the earthquake acceleration is 2/3. As the horizontal peak ground acceleration is 0.3g, the vertical peak ground acceleration becomes 0.2g.

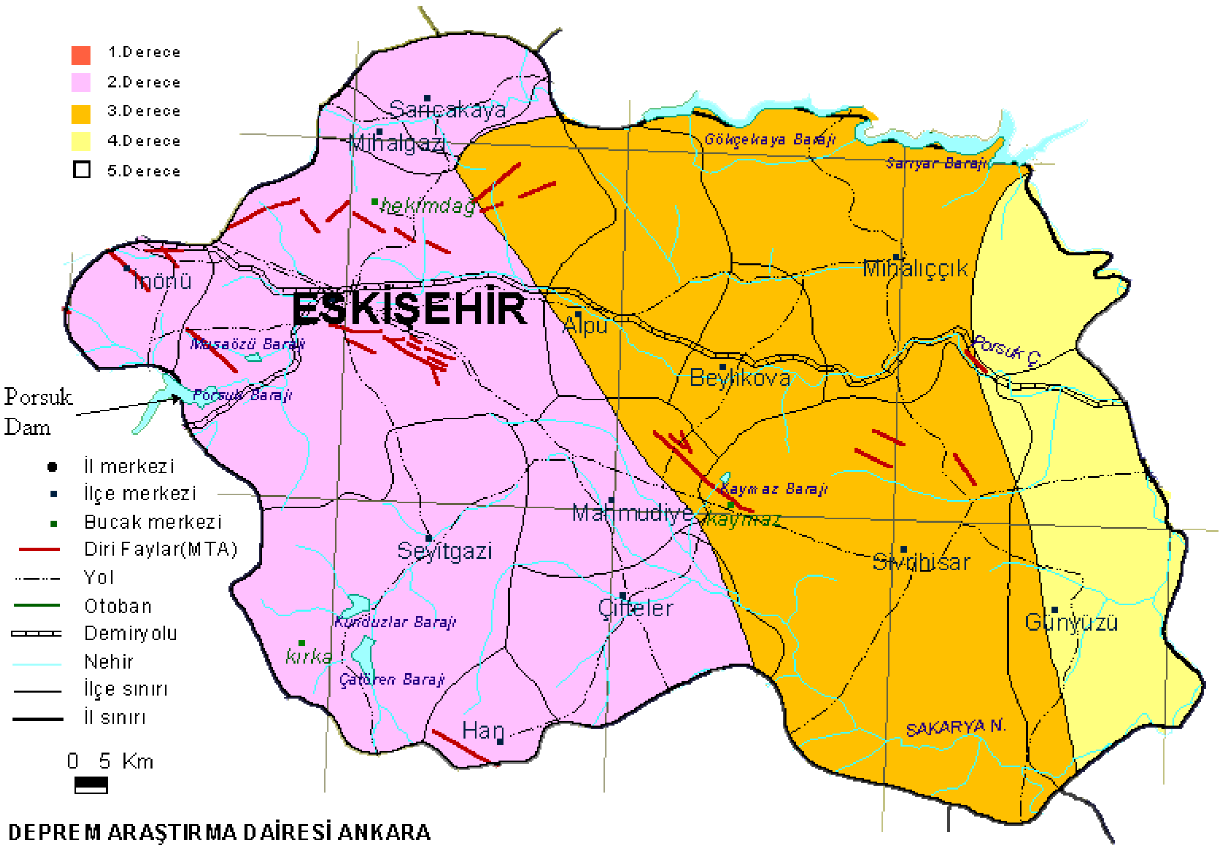


Figure 6.1 Earthquake Zones of Eskişehir (GDDAERD, 2004).

Table 6.1 Input Data for Stability Analysis of Porsuk Dam.

| Characteristics | Value |
|---|---|
| Height (from river bed) | 49.70 m (Orhon et al., 1991; DSI, 1998) |
| Elevation of river bed | 844.65 m (Orhon et al., 1991) |
| Crest Elevation | 894.35 m (DSI, 2004) |
| Upstream face slope, n | 0.00 (Seçkiner, 1999) |
| Downstream face slope, m | 0.85 (Seçkiner, 1999) |
| Depth of normal reservoir level ($H_n$) | 45.60 m (Seçkiner, 1999) (This will be used in CADAM as the mean upstream water elevation) |
| Depth of maximum reservoir level ($H_m$) | 48.20 m (Seçkiner, 1999) |
| Crest thickness ($T_c$) | 4.50 m (Seçkiner, 1999) |
| Bottom width (B) | 39.4 m (Orhon et al., 1991) |
| Tailwater depth | 6 m (Seçkiner, 1999) |
| Specific weight (concrete) | 24 $kN/m^3$ (Seçkiner, 1999) |
| Submerged specific weight of sediment | 11 $kN/m^3$ (Seçkiner, 1999) |
| Height of sediment accumulation | 3 m (Seçkiner, 1999) |
| Angle of repose of sediment | 31º (Seçkiner, 1999) |
| Horizontal peak ground acceleration | 0.30g (RTMPWS, 1997) |
| Vertical peak ground acceleration | 0.20g (Newmark, 1973) |
| Ice thickness | 0.52 m (Seçkiner, 1999) |
| Rate of temperature increase | 2.8 ºC (Seçkiner, 1999) |
| Ice Load / Unit Length | 100 kN/m (Thomas, 1976) |
| Uplift reduction coefficient | 0.6 (Seçkiner, 1999) |
| Drain position and elevation | 3.54 m from heel ; 16.85 m (Orhon et al., 1991) |
| Angle of internal friction | 55º (peak) (Leclerc et al., 2001 ; CDSA, 1995)<br>45º (residual) |
| Allowable compressive stress in concrete | 3750 $kN/m^2$ (Seçkiner, 1999) |
| Allowable compressive stress at foundation | 4000 $kN/m^2$ (Seçkiner, 1999) |
| Allowable shear stress at foundation | 1500 $kN/m^2$ (Seçkiner, 1999) |
| Compressive strength of concrete | 30 MPa (Analysis Committee, 1971) |
| Cohesion | 931 kPa (Leclerc et al., 2001) |

### 6.1.2 Determination of the Spectral Acceleration Coefficient

A suitable spectral acceleration coefficient, which is needed for the pseudo-dynamic analysis in CADAM software for seismic analysis, is also assigned. In the case study, the following two possible sets of data are entered as inputs into CADAM:

1. In order to have a spectral acceleration coefficient, an earthquake data should be obtained from the available data so that the response spectrum can be drawn and the needed spectral acceleration coefficient can then be reached. However, there are no such data for Porsuk Dam site close to Eskişehir province. That is why the data of an earthquake with similar properties that may occur in Eskişehir is found in a database containing earthquake records (PEER, 2000). This earthquake carries almost all the properties of a possible earthquake that might occur in Porsuk Dam area. These properties are determined as the distance of the area to the nearest active fault, the geological formation of the area, the horizontal peak ground acceleration according to the seismic zone that the area is in and the magnitude of such an earthquake that can occur for that seismic zone.

Distance to the nearest fault is estimated to be approximately 10 km (GDDAERD, 2004 and MTA, 2004) (See Figure 6.1).

Geological formation is peridotite (Orhon et al., 1991) which refers to “A” Rock (Geomatrix, 2000).

Horizontal peak ground acceleration is determined as 0.3g (RTMPWS, 1997) (See Figure 6.1).

Magnitude is estimated to be in between 5.9 and 6.2 (Wells and Coppersmith, 1994).

Using this information, the most likely earthquake is determined as Whittier Narrows Earthquake that occurred in the USA in October 01 in 1987 (See Tables A.2 and A.3).

Two other data are needed to find the spectral acceleration coefficient, which are $\tilde{T}_1$, the fundamental vibration period of the dam and $\tilde{\xi}_1$, the damping ratio of the dam. In CADAM, when the section geometry is put into the program, these values are calculated and printed on the screen.

$\tilde{T}_1$, the fundamental vibration of the dam : 0.163 seconds

$\tilde{\xi}_1$, the damping ratio of the dam : 0.132

In PEER's database, spectra with the damping ratio of 13% is not available but damping ratios of 10% and 15% are available (PEER, 2000) (See Figure 6.2). Thus, the weighted average of the two spectral acceleration coefficients corresponding to these damping ratios is calculated. The spectral acceleration is denoted as $S_a(\tilde{T}_1, \tilde{\xi}_1)$. However, it should be noted that spectral acceleration coefficient is denoted as "PAA", pseudo absolute acceleration, in PEER's database and as "HSA", horizontal spectral acceleration in CADAM.

$S_a$ (0.163 ; 0.10) = 0.494g (Table A.2 and Figure 6.2)

$S_a$ (0.163 ; 0.15) = 0.449g (Table A.3 and Figure 6.2)

The spectral acceleration coefficient for the first set of data is:

$S_a$ (0.163 ; 0.132) = 0.465g

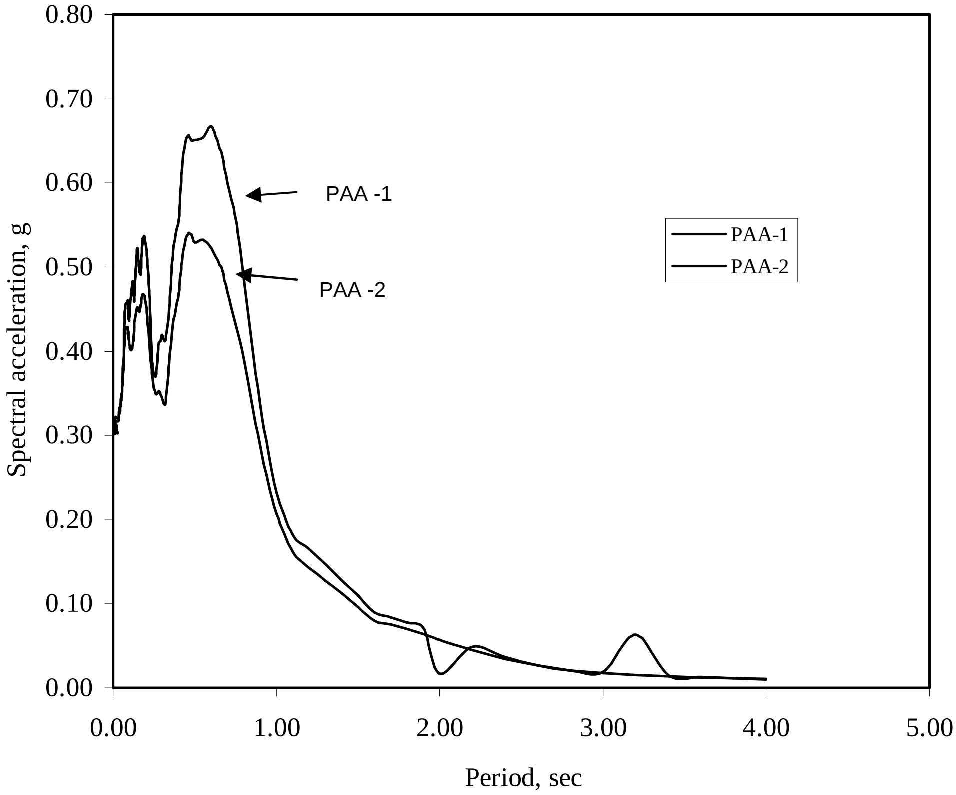


Figure 6.2 Whittier EQ - Spectral Acceleration Plot.

2. In Turkey, Ministry of Public Works and Settlement has provided a specification called "Specification for Structures to be Built in Disaster Areas" in which how to determine the spectral acceleration coefficient corresponding to 5% damped elastic design spectrum is explained (RTMPWS, 1997). This spectral acceleration coefficient is the design value. The spectral acceleration is denoted as $S_a(\tilde{T}_1, \tilde{\xi}_1)$. However, it should be noted that spectral acceleration coefficient is denoted as "A(T)" in the specification of RTMPWS (1997) and it is computed from Equation (4.29). The following values are used in the computations:

$A_o = 0.3g$

$I = 1.5$

For site classification $Z_1$, $T_A = 0.10$ s; $T_B = 0.30$ s

Fundamental vibration of the dam, $\tilde{T}_1 = 0.163$ s

$S(T) = 2.5$ for $(T_A \leq T \leq T_B)$

The spectral acceleration coefficient for the second set of data is:

$A(T) = (0.3g) . (1.5) . (2.5) = 1.125g$

### 6.1.3 Determination of Probability Distribution of Upstream Water Level

Another information to be generated is the estimation of probability distribution of reservoir water levels. For the probabilistic analysis present in CADAM, elevation data are used to find a good fitted probability distribution function. Upstream water elevations are obtained from DSI (2004) and put as inputs to frequency analysis. A Chi-square test is applied to check the goodness of fit of the probability distribution function assigned (See Section 6.2).

### 6.1.4 Determination of Cohesion

The value of cohesion is needed in CADAM. There should be two known values of cohesion which are cohesion for peak and cohesion for residual analyses. For the peak value, cohesion is calculated as follows (Leclerc et al., 2001):

$$\text{Cohesion}(c) = 0.17\sqrt{f_c} \qquad (6.1)$$

where $f_c$ is the compressive strength of concrete in MPa. For the residual value, if there are no tests to support the given decision, then cohesion should be considered zero (Leclerc et al., 2001).

## 6.2 Frequency Analysis

The available data are the Porsuk Dam upstream water elevations (DSI, 2004), (Table 6.2). Probability distribution of these elevations is investigated. To this end, normal and log-normal probability distribution functions are tested for goodness of fit. When these elevations are observed, it is clearly seen that they do not follow a uniform trend. CADAM also allows a distribution that can be defined by the user who is expected to give 500 data points. However, there are 60 available water elevation data which are obtained from the monthly operation of the reservoir. Frequency analysis is performed by ignoring some data according to the outlier test proposed by U.S. Water Resources Council (1981) (Chow et al., 1988). In the analysis, the outlier test is performed for the 10-percent significance level whereas confidence level is chosen as 95-percent for the Chi-square test.

After the outlier test is performed, three data are discarded. Thus, the normal probability distribution function is fitted to 57 data out of 60. The discarded data, which are the lower outliers corresponding to the water elevations observed in October, November, and December of 2001 (See Table 6.2). Figure 6.3 provides a visual scene of the fitted distribution with the frequency histogram. According to the calculations, the standard deviation of the fitted normal distribution function is obtained as 1.706 meters. The normal operating level is proposed to be 45.6 m (Seçkiner, 1999) which is used as the mean of the upstream water elevations.

The random variables should be defined in CADAM for the probabilistic analyses. There is limited information in the literature concerning the uncertainties of resistance and loading variables. The uncertainties required in the safety analyses, which are expressed in terms of coefficients of variations, and the corresponding PDFs are presented in Table 6.3 with reference to the previous studies reflecting reliability-based analysis of some hydraulic structures.

Table 6.2 Porsuk Dam Monthly Reservoir Routing (DSI, 2004).

| | | Beginning of month | | | | | | End of month | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Year | Month | Reservoir Elevation (m) | Reservoir Volume ($10^6$ $m^3$) | Irrigation ($10^3$ $m^3$) | Drinking Water ($10^3$ $m^3$) | Evaporation ($10^3$ $m^3$) | Spilled Water ($10^3$ $m^3$) | Reservoir Elevation (m) | Reservoir Volume ($10^6$ $m^3$) | Total Water Usage (1000 $m^3$) | (+ -) Storage (1000 $m^3$) | Water Usage Coming to the Lake (1000 $m^3$) |
| 1999 | Jun | 889.96 | 449.586 | 21572.40 | 2592.0 | 3476.0 | 0.00 | 889.46 | 436.427 | 27640.40 | -13159 | 14481.40 |
| | Jul | 889.46 | 436.427 | 21270.80 | 2678.4 | 5100.9 | 0.00 | 888.68 | 416.301 | 29050.10 | -20126 | 8924.10 |
| | Aug | 888.68 | 416.301 | 20784.40 | 2678.4 | 4265.6 | 0.00 | 887.85 | 395.419 | 27728.40 | -20882 | 6846.40 |
| | Sep | 887.85 | 395.419 | 15026.70 | 2592.0 | 2571.8 | 10825.92 | 886.89 | 371.956 | 31016.40 | -23463 | 7553.40 |
| | Oct | 886.89 | 371.956 | 0.00 | 2678.4 | 1431.0 | 28902.53 | 885.94 | 349.464 | 33011.90 | -22492 | 10519.90 |
| | Nov | 885.94 | 349.464 | 0.00 | 2592.0 | 195.5 | 6137.90 | 885.95 | 349.697 | 8925.30 | 233 | 9158.30 |
| | Dec | 885.95 | 349.697 | 0.00 | 2678.4 | 0.0 | 0.00 | 886.21 | 355.783 | 2678.40 | 6086 | 8764.40 |
| 2000 | Jan | 886.21 | 355.783 | 0.00 | 2678.4 | 0.0 | 5012.10 | 886.41 | 360.502 | 7690.50 | 4719 | 12409.50 |
| | Feb | 886.41 | 360.502 | 0.00 | 2505.6 | 0.0 | 0.00 | 886.93 | 372.919 | 2505.60 | 12417 | 14922.60 |
| | Mar | 886.93 | 372.919 | 0.00 | 2678.4 | 0.0 | 0.00 | 888.22 | 404.660 | 2678.40 | 31741 | 34419.40 |
| | Apr | 888.22 | 404.660 | 0.00 | 2592.0 | 880.6 | 7155.44 | 890.18 | 455.417 | 10546.20 | 50757 | 61303.20 |
| | May | 890.18 | 455.417 | 38586.20 | 2678.4 | 3371.3 | 0.00 | 889.79 | 445.089 | 63903.20 | -10328 | 53575.20 |
| | Jun | 889.79 | 445.089 | 32201.30 | 2592.0 | 4505.8 | 0.00 | 888.98 | 423.984 | 39299.10 | -21105 | 18194.10 |
| | Jul | 888.98 | 423.984 | 27491.60 | 2678.4 | 6270.0 | 0.00 | 887.95 | 397.906 | 36440.00 | -26078 | 10362.00 |
| | Aug | 887.95 | 397.906 | 29949.70 | 2678.4 | 4924.7 | 0.00 | 886.87 | 371.475 | 37552.80 | -26431 | 11121.80 |
| | Sep | 886.87 | 371.475 | 24198.90 | 2592.0 | 2715.3 | 3782.67 | 885.90 | 348.533 | 33245.60 | -22942 | 10303.60 |
| | Oct | 885.90 | 348.533 | 0.00 | 2678.4 | 1638.8 | 0.00 | 886.00 | 350.863 | 7220.20 | 2330 | 9550.20 |
| | Nov | 886.00 | 350.863 | 0.00 | 2592.0 | 0.0 | 0.00 | 886.22 | 356.018 | 2592.00 | 5155 | 7747.00 |
| | Dec | 886.22 | 356.018 | 0.00 | 2678.4 | 0.0 | 0.00 | 886.53 | 363.348 | 2678.40 | 7330 | 10008.40 |
| 2001 | Jan | 886.53 | 363.348 | 0.00 | 2678.4 | 0.0 | 0.00 | 886.84 | 370.754 | 2678.40 | 7406 | 10084.40 |
| | Feb | 886.84 | 370.754 | 0.00 | 2419.2 | 0.0 | 0.00 | 887.16 | 378.480 | 2419.20 | 7726 | 10145.20 |
| | Mar | 887.16 | 378.480 | 3145.00 | 2678.4 | 0.0 | 0.00 | 887.43 | 385.063 | 5823.40 | 6583 | 12406.40 |
| | Apr | 887.43 | 385.063 | 29401.90 | 2592.0 | 0.0 | 0.00 | 886.76 | 368.836 | 31993.90 | -16227 | 15766.90 |
| | May | 886.76 | 368.836 | 23258.90 | 2678.4 | 0.0 | 0.00 | 886.14 | 354.139 | 28630.40 | -14697 | 13933.40 |
| | Jun | 886.14 | 354.139 | 36232.70 | 2592.0 | 0.0 | 0.00 | 884.71 | 321.423 | 38824.70 | -32716 | 6108.70 |
| | Jul | 884.71 | 321.423 | 27881.30 | 2678.4 | 1567.5 | 0.00 | 883.38 | 292.601 | 32127.20 | -28822 | 3305.20 |
| | Aug | 883.38 | 292.601 | 23015.20 | 2678.4 | 3072.3 | 0.00 | 882.09 | 266.188 | 28765.90 | -26413 | 2352.90 |

Table 6.2 Porsuk Dam Monthly Reservoir Routing (DSI, 2004) (continued).

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2001 | Sep | 882.09 | 266.188 | 12262.80 | 2592.0 | 2254.0 | 0.00 | 881.32 | 251.145 | 17108.80 | -15043 | 2065.80 |
| | Oct | 881.32 | 251.145 | 4800.40 | 2678.4 | 1195.8 | 0.00 | 881.08 | 246.567 | 8674.60 | -4578 | 4096.60 |
| | Nov | 881.08 | 246.567 | 777.60 | 2592.0 | 0.0 | 0.00 | 881.30 | 250.761 | 3369.60 | 4194 | 7563.60 |
| | Dec | 881.30 | 250.761 | 0.00 | 2678.4 | 0.0 | 0.00 | 884.38 | 314.121 | 2678.40 | 63360 | 66038.40 |
| 2002 | Jan | 884.38 | 314.121 | 2164.30 | 2678.4 | 0.0 | 0.00 | 886.35 | 359.083 | 4842.70 | 44962 | 49804.70 |
| | Feb | 886.35 | 359.083 | 0.00 | 2419.2 | 0.0 | 0.00 | 888.25 | 400.668 | 2419.20 | 47681 | 50100.20 |
| | Mar | 888.25 | 400.668 | 0.00 | 2678.4 | 0.0 | 0.00 | 889.87 | 443.890 | 2678.40 | 43222 | 45900.40 |
| | Apr | 889.87 | 443.890 | 0.00 | 2592.0 | 0.0 | 80953.38 | 890.47 | 460.584 | 82619.10 | 16694 | 99313.10 |
| | May | 890.47 | 460.584 | 0.00 | 2678.4 | 0.0 | 55349.57 | 889.99 | 447.144 | 58028.00 | -13440 | 44588.00 |
| | Jun | 889.99 | 447.144 | 34663.70 | 2592.0 | 0.0 | 0.00 | 889.26 | 427.352 | 37255.70 | -19792 | 17463.70 |
| | Jul | 889.26 | 427.352 | 31561.90 | 2678.4 | 0.0 | 0.00 | 888.54 | 408.260 | 34240.30 | -19092 | 15148.30 |
| | Aug | 888.54 | 408.260 | 29998.10 | 2678.4 | 0.0 | 0.00 | 887.65 | 385.278 | 32676.50 | -22982 | 9694.50 |
| | Sep | 887.65 | 385.278 | 18385.90 | 2592.0 | 0.0 | 0.00 | 887.36 | 377.949 | 20977.90 | -7326 | 13651.90 |
| | Oct | 887.36 | 377.949 | 16372.80 | 2678.4 | 0.0 | 0.00 | 887.10 | 371.378 | 19051.20 | -6571 | 12480.20 |
| | Nov | 887.10 | 371.378 | 0.00 | 2592.0 | 0.0 | 41986.96 | 886.10 | 346.886 | 44098.60 | -24492 | 19606.60 |
| | Dec | 886.10 | 346.886 | 0.00 | 2678.4 | 0.0 | 39975.55 | 885.26 | 327.062 | 42654.00 | -19824 | 22830.00 |
| 2003 | Jan | 885.26 | 327.062 | 0.00 | 2678.4 | 0.0 | 27177.98 | 885.07 | 322.598 | 29858.40 | -4464 | 25394.40 |
| | Feb | 885.07 | 322.598 | 0.00 | 2419.2 | 0.0 | 0.00 | 886.16 | 348.350 | 2419.20 | 25752 | 28171.20 |
| | Mar | 886.16 | 348.350 | 0.00 | 2678.4 | 0.0 | 0.00 | 887.63 | 384.772 | 2678.40 | 36422 | 39100.40 |
| | Apr | 887.63 | 384.772 | 0.00 | 2592.0 | 0.0 | 20556.48 | 889.57 | 435.757 | 22913.30 | 50985 | 73898.30 |
| | May | 889.57 | 435.757 | 20736.00 | 2678.4 | 0.0 | 36908.35 | 889.57 | 435.757 | 60322.80 | 0.00 | 60322.80 |
| | Jun | 889.57 | 435.757 | 46424.40 | 2592.0 | 0.0 | 0.00 | 888.92 | 418.209 | 49016.40 | -17549 | 31467.40 |
| | Jul | 888.92 | 418.209 | 44262.70 | 2678.4 | 0.0 | 0.00 | 887.89 | 391.343 | 46941.10 | -26866 | 20075.10 |
| | Aug | 887.89 | 391.343 | 41809.80 | 2678.4 | 0.0 | 0.00 | 886.87 | 365.678 | 44488.20 | -25665 | 18823.20 |
| | Sep | 886.87 | 365.678 | 32127.00 | 2592.0 | 0.0 | 0.00 | 886.00 | 344.445 | 34719.00 | -21233 | 13486.00 |
| | Oct | 886.00 | 344.445 | 6780.70 | 2678.4 | 0.0 | 0.00 | 886.02 | 344.933 | 9459.10 | 488 | 9947.10 |
| | Nov | 886.02 | 344.933 | 0.00 | 2592.0 | 0.0 | 0.00 | 886.15 | 348.106 | 2592.00 | 3173 | 5765.00 |
| | Dec | 886.15 | 348.106 | 0.00 | 2678.4 | 0.0 | 4821.12 | 886.55 | 357.624 | 7499.50 | 9518 | 17017.50 |
| 2004 | Jan | 886.55 | 357.624 | 0.00 | 2678.4 | 0.0 | 4821.12 | 887.44 | 379.971 | 7499.50 | 22347 | 29846.50 |
| | Feb | 887.44 | 379.971 | 0.00 | 2505.6 | 0.0 | 20805.12 | 888.11 | 397.003 | 23310.72 | 17032 | 40342.72 |
| | Mar | 888.11 | 397.003 | 0.00 | 2678.4 | 0.0 | 52885.44 | 888.34 | 403.024 | 55563.84 | 6021 | 61584.84 |
| | Apr | 888.34 | 403.024 | 27184.03 | 2592.0 | 0.0 | 10385.28 | 888.53 | 407.998 | 40161.31 | 4974 | 45135.31 |
| | May | 888.53 | 407.998 | 85235.33 | 2678.4 | 0.0 | 0.00 | 888.47 | 406.428 | 87913.73 | -1570 | 86343.73 |

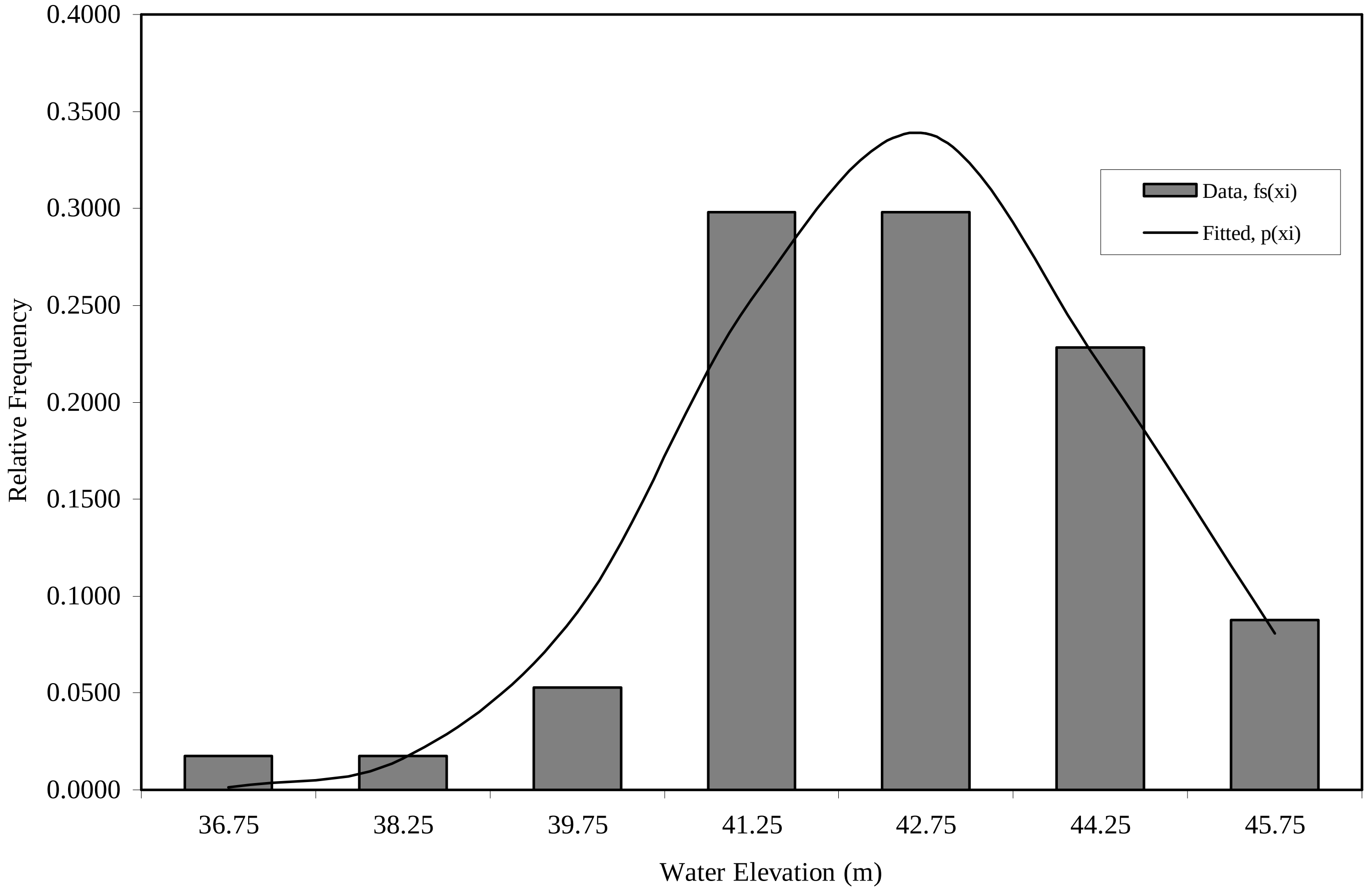


Figure 6.3 Relative Frequency Function.

Table 6.3 Random Variables Needed for Probabilistic Analysis.

| Variable | $\mu$ | $\sigma$ | $\delta$ | PDF | Reference |
|---|---|---|---|---|---|
| Tensile Strength (kPa) | 3000 | 300 | 0.10 | Normal | Ang and Tang (1990) |
| Peak Cohesion (kPa) | 931 | 46.5 | 0.05 | Normal | Equations (3.14) and (6.1) |
| Peak Friction Coefficient | 1.428 | 0.057 | 0.04 | Normal | Ang and Tang (1990) |
| Normal Upstream Reservoir Elevation (m) | 45.6 | 1.706 | 0.037 | Normal | Present Study |
| Drain Efficiency | 0.6 | 0.18 | 0.3 | Normal | Assumed |
| Ice Load (kN) | 52 | 15.6 | 0.3 | Normal | Assumed |
| Horizontal PGA(g) | 0.3 | 0.075 | 0.25 | Normal | Ang and Tang (1990) |

There are several more random variables that can be considered in the probabilistic analysis, which are residual cohesion, residual friction coefficient, upstream reservoir increase (flood), silt elevation, silt volumetric weight, floating debris, and last applied external force. However, there are no available probabilistic data for all variables. That is why residual cohesion, residual friction coefficient, silt elevation, and silt volumetric weight are accepted as deterministic variables (See Table 6.1). These values are entered as constant inputs into CADAM but they are excluded in the probabilistic analysis.

## 6.3 CADAM Output and Results

Tables B.1 to B.5 are obtained after the related data into CADAM software are entered as inputs. The load parameters, input including the geometry report, and the results are available in these tables (Table B.1 includes information for both sets. Tables B.2 and B.3 for Data Set 1, and Tables B.4 and B.5 for Data Set 2 are presented). Tables B.1 to B.5 are determined by running the Monte-Carlo simulations in usual load combination in CADAM software. Tables 6.4 to 6.8 and 6.9 to 6.13 present the summaries that show the safety factors and failure probabilities when Monte-Carlo simulations are run for all load combinations with data set 1 and data set 2, respectively. Figures B.10 to B.14 present the probability distributions of the safety factors that are determined in Table 6.4.

Table 6.4 Results of Probabilistic Analysis (Usual Combination: Data Set 1).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.850 | 0.506 | 4.327 | 7.979 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.558 | 0.136 | 1.204 | 2.049 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.443 | 0.058 | 4.291 | 4.626 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.893 | 0.137 | 1.530 | 2.371 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.446 | 0.105 | 3.078 | 3.793 | 1.00000 | 0.00000 |

Table 6.5 Results of Probabilistic Analysis (Flood Combination: Data Set 1).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.237 | 0.178 | 4.541 | 5.929 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.381 | 0.000 | 1.381 | 1.381 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.533 | 0.000 | 4.533 | 4.533 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.704 | 0.000 | 1.704 | 1.704 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.299 | 0.000 | 3.299 | 3.299 | 1.00000 | 0.00000 |

Table 6.6 Results of Probabilistic Analysis (Seismic-1 Combination: Data Set 1).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 2.839 | 0.597 | 1.526 | 5.775 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 0.781 | 0.136 | 0.432 | 1.531 | 0.06716 | 0.93284 |
| Overturning Safety Factor toward Upst. | 4.032 | 0.113 | 3.629 | 4.465 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.183 | 0.131 | 0.825 | 1.893 | 0.93850 | 0.06150 |
| Uplifting Safety Factor | 2.379 | 0.195 | 1.855 | 3.260 | 1.00000 | 0.00000 |

Table 6.7 Results of Probabilistic Analysis (Seismic-2 Combination: Data Set 1).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 2.694 | 0.566 | 1.468 | 5.989 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 0.772 | 0.137 | 0.420 | 1.603 | 0.06144 | 0.93856 |
| Overturning Safety Factor toward Upst. | 4.320 | 0.101 | 3.977 | 4.607 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.097 | 0.135 | 0.740 | 1.902 | 0.75278 | 0.24722 |
| Uplifting Safety Factor | 2.379 | 0.195 | 1.863 | 3.333 | 1.00000 | 0.00000 |

Table 6.8 Results of Probabilistic Analysis

(Post-Seismic Combination: Data Set 1).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.894 | 0.511 | 4.372 | 8.099 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.574 | 0.138 | 1.220 | 2.074 | 0.99998 | 0.00002 |
| Overturning Safety Factor toward Upst. | 4.460 | 0.058 | 4.311 | 4.640 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.892 | 0.136 | 1.531 | 2.369 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.455 | 0.105 | 3.088 | 3.803 | 1.00000 | 0.00000 |

Table 6.9  Results of Probabilistic Analysis (Usual Combination: Data Set 2).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.851 | 0.506 | 4.265 | 7.948 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.558 | 0.136 | 1.205 | 2.043 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.443 | 0.058 | 4.296 | 4.630 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.893 | 0.136 | 1.530 | 2.360 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.447 | 0.105 | 3.078 | 3.792 | 1.00000 | 0.00000 |

Table 6.10  Results of Probabilistic Analysis (Flood Combination: Data Set 2).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.237 | 0.178 | 4.574 | 5.914 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.381 | 0.000 | 1.381 | 1.381 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.533 | 0.000 | 4.533 | 4.533 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.704 | 0.000 | 1.704 | 1.704 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.299 | 0.000 | 3.299 | 3.299 | 1.00000 | 0.00000 |

Table 6.11 Results of Probabilistic Analysis (Seismic-1 Combination: Data Set 2).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 2.842 | 0.595 | 1.438 | 6.042 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 0.782 | 0.135 | 0.432 | 1.570 | 0.06736 | 0.93264 |
| Overturning Safety Factor toward Upst. | 4.032 | 0.113 | 3.612 | 4.473 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.183 | 0.130 | 0.830 | 1.925 | 0.94066 | 0.05934 |
| Uplifting Safety Factor | 2.380 | 0.194 | 1.860 | 3.304 | 1.00000 | 0.00000 |

Table 6.12 Results of Probabilistic Analysis (Seismic-2 Combination: Data Set 2).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 8.540 | 27.948 | 0.000 | 100.000 | 0.91460 | 0.08540 |
| Sliding Safety Factor (peak) | 1.688 | 0.513 | 0.379 | 4.890 | 0.91460 | 0.08540 |
| Sliding Safety Factor (residual) | 0.551 | 0.124 | 0.266 | 1.264 | 0.00358 | 0.99642 |
| Overturning Safety Factor toward Upst. | 5.802 | 0.290 | 4.758 | 6.905 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 0.801 | 0.131 | 0.489 | 1.547 | 0.07724 | 0.92276 |
| Uplifting Safety Factor | 2.379 | 0.195 | 1.858 | 3.236 | 1.00000 | 0.00000 |

Table 6.13 Results of Probabilistic Analysis

(Post-Seismic Combination: Data Set 2).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.901 | 0.515 | 4.383 | 8.252 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.576 | 0.139 | 1.221 | 2.071 | 0.99998 | 0.00002 |
| Overturning Safety Factor toward Upst. | 4.460 | 0.058 | 4.311 | 4.644 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.895 | 0.137 | 1.531 | 2.367 | 0.99998 | 0.00002 |
| Uplifting Safety Factor | 3.456 | 0.106 | 3.089 | 3.802 | 1.00000 | 0.00000 |

Effective stress analyses are presented in Figures B.1 to B.9. The first five figures, which are analyses of usual, flood, seismic-1, and post-seismic combinations, are the same for both data sets (Figures B.1 to B.5). However, for seismic-2 combinations, as the data are different for each set, the figures are shown seperately (Figures B.6 and B.7 for Data Set 1; Figures B.8 and B.9 for Data Set 2). The effective stresses are determined for each lift joint and presented in the figures.

In CADAM, Monte-Carlo simulations can be run for all loading combinations. However, user should specify which combination is chosen. Effective stress analyses, in this thesis, are performed after choosing the usual combination in Monte-Carlo simulations and Figures B.1 to B.9 are determined for each loading combination accordingly. However, Tables 6.4 to 6.13 are determined by choosing the appropriate combination under the headings that they specify.

# CHAPTER 7

# DISCUSSION OF RESULTS

In the case study, two sets of input data are entered into CADAM and the results are investigated. The only difference between these sets is the spectral acceleration coefficient, which is 0.465g for Data Set 1 and 1.125g for Data Set 2. These values are used in the pseudo-dynamic analysis (Chopra's Method), so most of the tables are the same, but the tables showing the seismic dynamic analyses are different. However, it should be noted that in Monte-Carlo simulations, random numbers are generated firstly; that is why the results of the same data input may be slightly different from each other. These differences do not affect the overall view of results.

In Monte-Carlo analysis, the number of simulation cycles, i.e. the number of trials to generate random numbers, influences the level of reliability. The number of cycles required in a Monte-Carlo simulation to determine the exact reliability must be large in order to obtain a significant sampling of simulation events. The accuracy of the mean risk under a particular simulation cycle may be estimated by the coefficient of variation of failure probability, $\delta_f$, which decreases with increasing sample size. Therefore, simulations should be carried out several times for large cycles such that the corresponding value of $\delta_f$ is relatively small. According to Johnson (1999), it is desirable to have $\delta_f < 0.1$. Variations of $\delta_f$ against number of simulation cycles are shown in Figure 7.1. It is observed that as the number of simulation cycles increases, $\delta_f$ approaches a constant value, which is approximately 0.006. Therefore, it can be considered that further increases in number of simulation cycles would not lead to significant accuracy in the computations (Yanmaz, 2003). To this end, a 50,000-cycle is brought into the analysis (See Figure 7.1).

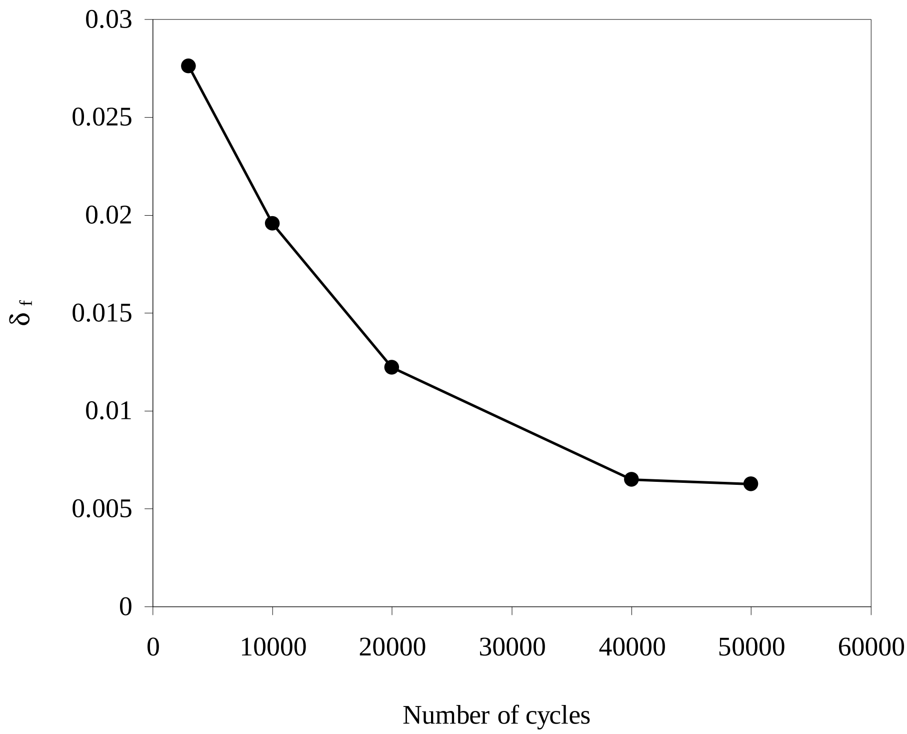


Figure 7.1 Variation of $\delta_f$ against the Number of Simulation Cycles.

The sign conventions are presented in Figures 5.1 and 5.2. Negative sign is assigned to compression in CADAM. In this thesis, the signing of the forces is used in the way that the stability and the safety of the dam get more critical, i.e. considering the horizontal and vertical seismic forces in the positive direction but changing it to the negative direction in the final calculations, so that the dam inertia will be towards the negative direction, too. This is done by multiplying the seismic forces with "– 1" in the "Load Combinations" window of CADAM. The minus sign is not directly assigned to the ground acceleration values in "Seismic

Analysis" windows because the hydrodynamic force in the reservoir could get affected by the sign change.

The forces that are considered in usual combination are dead load, hydrostatic (upstream and downstream), uplift, silt, post-tensioning, applied external forces, and ice load. In the flood combination, the forces are as in the usual combination but instead of ice load, floating debris is considered. The forces that are considered in seismic analysis are the ones in the usual combination and seismic forces. It is the same forces considered in usual combination for post-seismic combination. In this case study, there are no post-tensioning, floating debris, or applied external forces on the dam.

According to the analyses, there is almost 100% reliability for usual, flood, and post-seismic combinations (Tables 6.4, 6.8, 6.9 and 6.13). The dam resists and manages to stay 100% safe. However, when seismic analyses are made, i.e. pseudo-static (seismic coefficient) and pseudo-dynamic (Chopra's Method) analyses, then sliding stability factors, as well as overturning towards the upstream safety factors, become critical and the dam is in danger of failure with very high risks (Tables 6.5, 6.6, 6.7 and 6.10, 6.11, 6.12). The evaluation of the results, possible causes of failure and management of different inputs are discussed below:

1. In the analyses with usual and flood combinations, the seismic forces are excluded in the calculations and dam is determined to be safe without any risk. Also, the analyses of after-seismic events prove to have almost full reliability.

2. The tensile strength value is taken as 10% of the compressive strength of the material (CDA, 1999). The compressive strength is 30 MPa, so the tensile strength becomes 3 MPa. However, thinking that the dam is over 30

years old, it would be a more critical analysis to accept no tensile strength as there may have been cracks where there is no tensile strength. This analysis is done at Section 7.1. In the analysis with seismic-1 combination (pseudo-static analysis), only one type of failure is determined which is the failure because of the residual sliding safety factor. Also, an increment in the probability of failure is observed for the overturning safety factor towards downstream.

When the material properties are entered as inputs in CADAM, the cohesion is entered as zero in the residual shear strength window. Actually, this value can be taken up to 100 kPa if supported by tests but it is advised to consider it as zero in the absence of tests (Leclerc et al., 2001). It should not be forgotten that concrete dams are elevated with various heights of blocks during the construction depending on the concrete quality, available instruments and technology. There are key trenches at each block to resist sliding. For sliding stability, it is very important. However, in CADAM, there are no definitions of key trenches but the angle of friction is advised to be considered as 55º for peak and 45º for residual angles of friction (CDSA, 1995). Tables 7.1 to 7.5 and 7.6 to 7.10 present the determined safety factors with residual cohesion.

3. In data set 2, spectral acceleration value is 1.125g which is determined from the specification of Republic of Turkey Ministry of Public Works and Settlement (1997). When the program is run with 1.125g in seismic-2 combination, very high failure probabilities are observed. The failure reasons are determined as residual sliding safety factor and even overturning safety factor towards downstream. Also, cracks are observed throughout the dam. These show that there will definitely be various damages in the proposed dam if such conditions occur. In pseudo-dynamic

analysis, the dynamic amplification of the inertia forces along the height of the dam is recognized but the oscillatory nature of the amplified inertia forces is not considered. With the given spectral acceleration, forces within the dam increase so high that there is no way for the dam to resist to this stress.

4. In risk analysis, instead of safety factors, the probability of failure is more important. In certain cases, the safety factors may be smaller than the required limiting values but keeping the reliability values relatively high. This is because of the nature and elements of the random variables, i.e. the mean, the standard deviation, and the cut-off values. In CADAM, safety factors can be assigned but they are not used in the computational algorithm.

5. The bounds (cut-off values) are determined to be three standard deviations away from the mean which provides a convergence of 99.73%. For a higher convergence, up to five standard deviations can be used in Monte-Carlo simulations.

## 7.1 Analyses with New Data

The analyses of Data Set 1 and Data Set 2 are repeated considering zero tensile strength and 100 kPa of residual shear strength cohesion. This analysis is made in order to show how important the tensile strength is especially for cracking and how the residual sliding safety factor changes. New data sets are called Data Sets 3 and 4 to replace Data Sets 1 and 2, respectively. The results are presented in Tables 7.1 through 7.10.

Table 7.1 Results of Probabilistic Analysis (Usual Combination: Data Set 3).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Fact. peak | 5.849 | 0.505 | 4.355 | 7.991 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.947 | 0.165 | 1.520 | 2.539 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.443 | 0.058 | 4.294 | 4.624 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.893 | 0.137 | 1.530 | 2.364 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.446 | 0.105 | 3.077 | 3.793 | 1.00000 | 0.00000 |

Table 7.2 Results of Probabilistic Analysis (Flood Combination: Data Set 3).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.237 | 0.179 | 4.541 | 5.891 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.731 | 0.000 | 1.381 | 1.731 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.533 | 0.000 | 4.533 | 4.533 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.704 | 0.000 | 1.704 | 1.704 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.299 | 0.000 | 3.299 | 3.299 | 1.00000 | 0.00000 |

Table 7.3 Results of Probabilistic Analysis (Seismic-1 Combination: Data Set 3).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 84.998 | 22.948 | 0.000 | 100.000 | 0.44472 | 0.55528 |
| Sliding Safety Fact. peak | 1.499 | 0.801 | 0.598 | 5.847 | 0.69578 | 0.30422 |
| Sliding Safety Factor (residual) | 0.821 | 0.197 | 0.427 | 1.865 | 0.17476 | 0.82524 |
| Overturning Safety Factor toward Upst. | 4.032 | 0.112 | 3.627 | 4.491 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.182 | 0.130 | 0.825 | 1.844 | 0.93858 | 0.06142 |
| Uplifting Safety Factor | 2.378 | 0.194 | 1.852 | 3.230 | 1.00000 | 0.00000 |

Table 7.4 Results of Probabilistic Analysis (Seismic-2 Combination: Data Set 3).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 93.505 | 16.155 | 0.000 | 100.000 | 0.21951 | 0.78049 |
| Sliding Safety Fact. peak | 1.277 | 0.619 | 0.534 | 5.731 | 0.66927 | 0.33073 |
| Sliding Safety Factor (residual) | 0.791 | 0.177 | 0.419 | 1.941 | 0.11516 | 0.88484 |
| Overturning Safety Factor toward Upst. | 4.320 | 0.101 | 3.973 | 4.602 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.097 | 0.135 | 0.744 | 1.855 | 0.75684 | 0.24316 |
| Uplifting Safety Factor | 2.380 | 0.195 | 1.852 | 3.298 | 1.00000 | 0.00000 |

Table 7.5 Results of Probabilistic Analysis
(Post-Seismic Combination: Data Set 3).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 100.000 | 0.000 | 100.000 | 100.000 | 0.00000 | 1.00000 |
| Sliding Safety Fact. peak | 0.665 | 0.145 | 0.289 | 1.264 | 0.01949 | 0.98051 |
| Sliding Safety Factor (residual) | 0.466 | 0.820 | 0.222 | 0.835 | 0.00000 | 1.00000 |
| Overturning Safety Factor toward Upst. | 1.287 | 0.004 | 1.278 | 1.308 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.168 | 0.070 | 0.984 | 1.407 | 0.99650 | 0.00350 |
| Uplifting Safety Factor | 1.265 | 0.047 | 1.140 | 1.422 | 1.00000 | 0.00000 |

Table 7.6 Results of Probabilistic Analysis (Usual Combination: Data Set 4).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Fact. peak | 5.854 | 0.510 | 4.351 | 7.991 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.949 | 0.166 | 1.522 | 2.536 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.443 | 0.058 | 4.296 | 4.629 | 1.00000 | 0.00000 |
| Overturning Safety Factor tow. Downst. | 1.894 | 0.137 | 1.531 | 2.361 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.447 | 0.106 | 3.080 | 3.792 | 1.00000 | 0.00000 |

Table 7.7 Results of Probabilistic Analysis (Flood Combination: Data Set 4).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 5.236 | 0.179 | 4.595 | 5.909 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 1.731 | 0.000 | 1.731 | 1.731 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Upst. | 4.533 | 0.000 | 4.533 | 4.533 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.704 | 0.000 | 1.704 | 1.704 | 1.00000 | 0.00000 |
| Uplifting Safety Factor | 3.299 | 0.000 | 3.299 | 3.299 | 1.00000 | 0.00000 |

Table 7.8 Results of Probabilistic Analysis (Seismic-1 Combination: Data Set 4).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 84.895 | 23.083 | 0.000 | 100.000 | 0.44582 | 0.55418 |
| Sliding Safety Fact. peak | 1.503 | 0.807 | 0.593 | 5.735 | 0.69978 | 0.30022 |
| Sliding Safety Factor (residual) | 0.822 | 0.198 | 0.427 | 1.894 | 0.17540 | 0.82460 |
| Overturning Safety Factor toward Upst. | 4.033 | 0.112 | 3.632 | 4.505 | 1.00000 | 0.00000 |
| Overturning Safety Factor tow. Downst. | 1.182 | 0.131 | 0.822 | 1.872 | 0.94038 | 0.05962 |
| Uplifting Safety Factor | 2.379 | 0.194 | 1.853 | 3.244 | 1.00000 | 0.00000 |

Table 7.9 Results of Probabilistic Analysis (Seismic-2 Combination: Data Set 4).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 99.756 | 3.018 | 13.935 | 100.000 | 0.01033 | 0.98967 |
| Sliding Safety Fact. peak | 0.795 | 0.220 | 0.379 | 4.736 | 0.11937 | 0.88063 |
| Sliding Safety Factor (residual) | 0.553 | 0.127 | 0.268 | 1.614 | 0.00642 | 0.99358 |
| Overturning Safety Factor toward Upst. | 5.801 | 0.289 | 4.720 | 6.908 | 1.00000 | 0.00000 |
| Overturning Safety Factor tow. Downst. | 0.802 | 0.132 | 0.491 | 1.605 | 0.07884 | 0.92116 |
| Uplifting Safety Factor | 2.381 | 0.195 | 1.862 | 3.287 | 1.00000 | 0.00000 |

Table 7.10 Results of Probabilistic Analysis
(Post-Seismic Combination for Data Set 4).

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 100.000 | 0,000 | 100.000 | 100.000 | 0.00000 | 1.00000 |
| Sliding Safety Fact. peak | 0.665 | 0.145 | 0.295 | 1.277 | 0.01914 | 0.98086 |
| Sliding Safety Factor (residual) | 0.466 | 0.100 | 0.222 | 0.836 | 0.00000 | 1.00000 |
| Overturning Safety Factor toward Upst. | 1.287 | 0.004 | 1.278 | 1.311 | 1.00000 | 0.00000 |
| Overturning Safety Factor tow. Downst. | 1.168 | 0.070 | 0.984 | 1.408 | 0.99646 | 0.00354 |
| Uplifting Safety Factor | 1.265 | 0.047 | 1.140 | 1.422 | 1.00000 | 0.00000 |

The safety factors in usual and flood combinations for Data Set 3 and 4 remain almost the same as the ones in Data Set 1 and 2. However, in seismic analysis, severe cracking is observed and the analyses with seismic combinations show that the probability of failure from cracking is very high. Cracking initiates at the heel of the dam when the compressive stress does not achieve the minimum compressive value. As the tensile strength input is entered as zero, no benefit comes from tensile strength in order to resist the cracking action. In cracking options, the criterion may be chosen as the tensile initiation strength being equal to zero. However, no difference should be observed.

Assigning a 100 kPa residual shear strength cohesion increases the residual sliding safety factor from 1.558 to 1.947 in usual combination for data set 1. For the analyses with other combinations, slightly increased residual sliding safety factors are observed but again, in seismic loading, the probability of failure for this factor reaches almost unity. This is because of the extreme shear forces which are generated by very high seismic accelerations.

## 7.2 Deterministic Safety Factor and Failure Probability Analyses

In conventional deterministic approaches, safety factors are calculated using forces and moments from the assigned dimensions. As long as the minimum requirements of safety factors are satisfied, the effect of further increases in safety factors on the overall stability cannot be assessed on rational basis. More realistic evaluation of safety can be achieved using the concept of probability of failure which can be obtained through a probability-based method. To this end, various base widths are assigned to Porsuk Dam and the corresponding safety levels are checked under usual loading using both deterministic and probabilistic approaches of CADAM. The outputs of this analysis provided a mean to compare deterministic safety factors and probability of failure values for the base widths tested. The results are presented in Tables 7.11 and 7.12. The following analysis is performed to show the related graphs of deterministic safety factors corresponding the failure probabilities for a given dam width.

Table 7.11 Deterministic Safety Factors for Data Set 1 in Usual Combination.

| B (m) | SSF(Peak) | SSF(Residual) | OSF(Upst.) | OSF(Downst.) | Uplifting |
|---|---|---|---|---|---|
| 39.4 | 5.797 | 1.543 | 4.461 | 1.874 | 3.443 |
| 35 | 5.198 | 1.404 | 4.934 | 1.679 | 3.504 |
| 30 | 4.182 | 1.247 | 5.734 | 1.429 | 3.593 |
| 25 | 3.197 | 1.089 | 7.045 | 1.152 | 3.719 |
| 21 | 0.542 | 0.380 | 2.549 | 0.723 | 1.413 |
| 20 | 0.537 | 0.376 | 2.727 | 0.687 | 1.429 |

Table 7.12 Failure Possibilities for Data Set 1 in Usual Combination.

| B (m) | SSF(Peak) | SSF(Residual) | OSF(Upst.) | OSF(Downst.) | Uplifting |
|---|---|---|---|---|---|
| 39.4 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 |
| 35 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 |
| 30 | 0.00000 | 0.00210 | 0.00000 | 0.00000 | 0.00000 |
| 25 | 0.01592 | 0.14222 | 0.00000 | 0.04378 | 0.00000 |
| 21 | 0.69998 | 0.73260 | 0.00000 | 0.80332 | 0.00000 |
| 20 | 0.89526 | 0.89954 | 0.00000 | 0.94058 | 0.00000 |

The graphical views of various safety factors and failure probabilities for different values of the width, B, of the dam can be seen in Figures 7.2 and 7.3. There are possibilities of failure for each kind of factors except overturning towards the upstream and uplifting safety factors. That is why these factors are not included in the graphical view of Tables 7.11 and 7.12. Upstream crack percentage is actually the same with the failure probability of it, so this is excluded in Figures 7.2 and 7.3, too. When the base width is less than or equal to 25 m, the cracking begins resulting in a high probability of failure and safety factors below unity which represents the static equilibrium case.

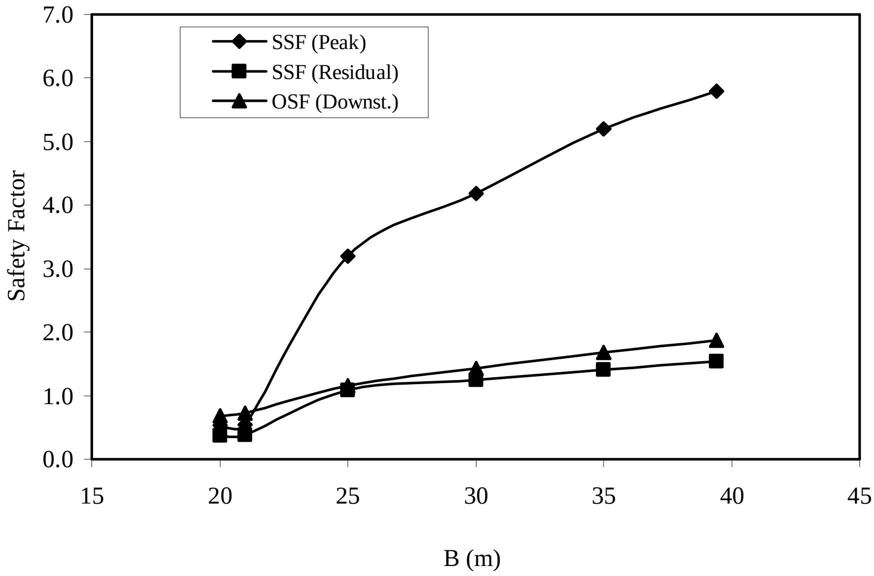


Figure 7.2 Variation of Safety Factors Against Base Width.

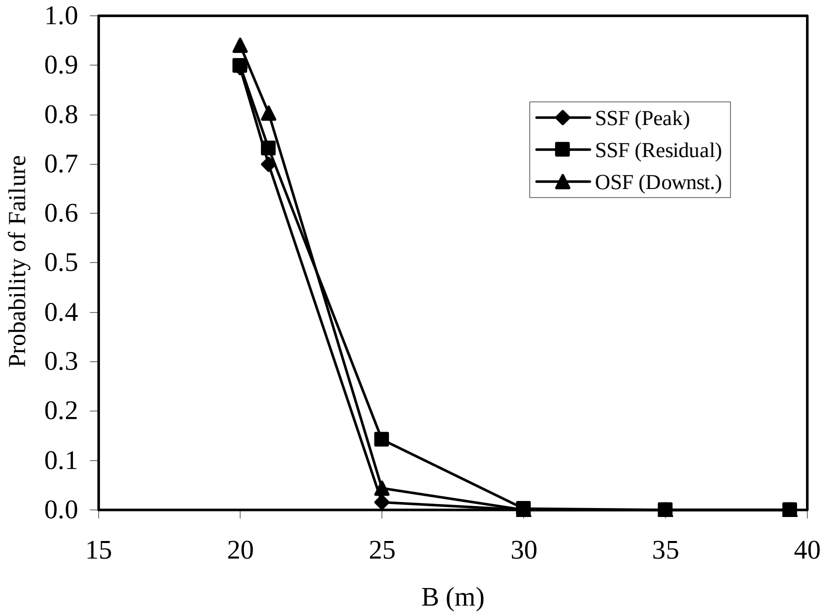


Figure 7.3 Variation of Probability of Failure Against Base Width.

Percent changes of the safety factors under each base width value relative to the original base width are determined by dividing the difference of safety factors by the safety factor obtained using the original base width. Incremental changes of the failure probabilities are determined by subtracting the initial failure probability value from the failure probability under a particular base width (See Figures 7.4 and 7.5).

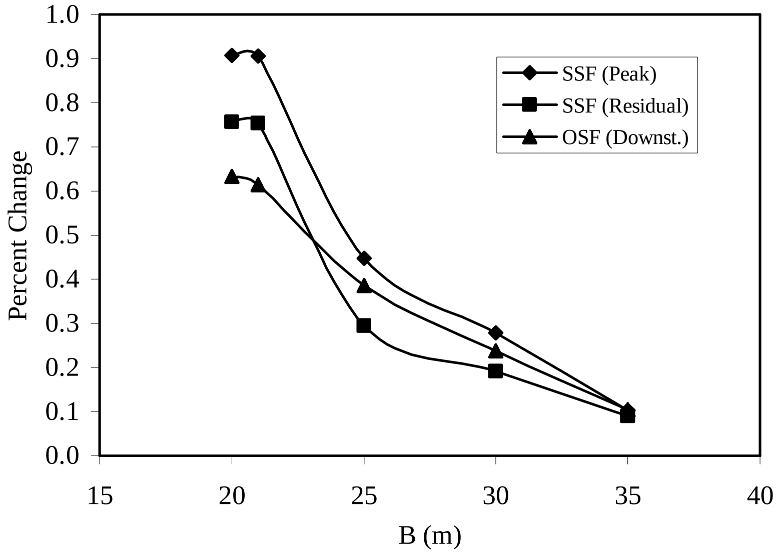


Figure 7.4 Percent Changes in Safety Factors.

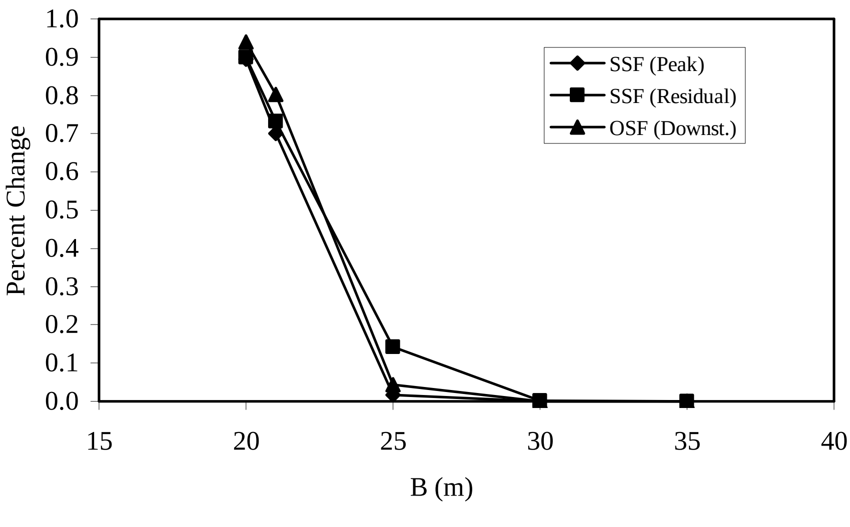


Figure 7.5 Percent Changes in Failure Probabilities.

When the analysis is executed at the actual width of the dam for usual combination, it can be seen that there is a full reliability for the dam. However, when the width begins to decrease, the safety factors decrease and corresponding failure probabilities increase. Even if some safety factors are over unity, again high failure probabilities are observed, so probabilistic approach is more realistic.

## 7.3 Analysis of the Changes in Stresses in the Vertical Direction

In this section, variations in normal and uplift stresses throughout the height of the dam are investigated. To reduce the number of possible combinations, this analysis is only carried out for the usual loading. In the analysis of Porsuk Dam, a reduction factor is used in the uplift force consideration because of an existing drain, which is located at 16.85 m above the base. In CADAM, the uplift pressure is calculated for each joint which are presented in the stability drawings (Figures B.1 to B.9). The uplift pressure distribution along the height of the dam at the level of joints is presented in Figure 7.6. As can be seen from this figure, the uplift force reduces rapidly at the drain level. Uplift distributions for the rest of the load combinations are the same as the one for usual combination.

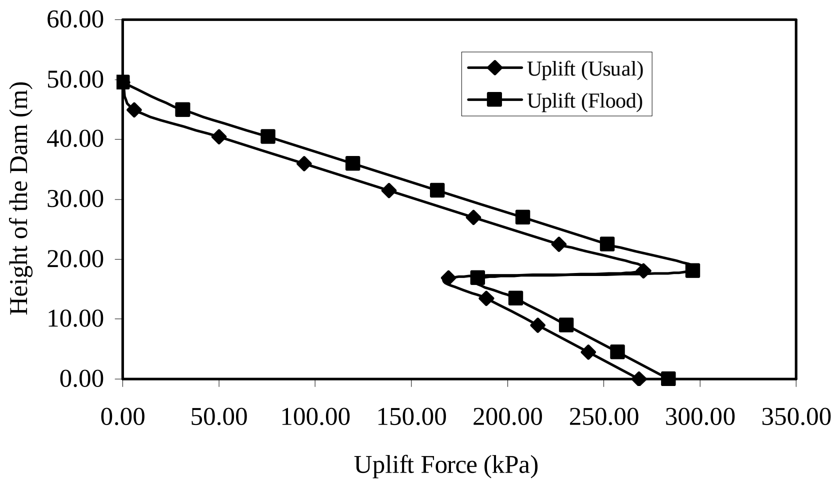


Figure 7.6 Uplift Distribution Along the Height of the Dam.

Additional analysis is carried out to observe the variation of vertical normal stresses on both sides of the dam throughout the height of the dam.

An analysis for the normal stress distribution for different loading combinations is performed in order to obtain at which joints the stresses exceed the allowable values.

The normal stresses, in kPa, along the height of the dam for each joint are presented in Tables 7.13 and 7.14 for Data Set 1; Tables 7.15 and 7.16 for Data Set 2; for upstream and downstream.

It should be noted that the normal stress values are adapted from the effective stress analysis. The values of the stability analysis are not considered here. Negative sign shows that the stress is compressive. Figures 7.7 to 7.10 are also presented in order to provide a visual scene of the normal stress distributions.

Table 7.13 Upstream Normal Stress Values of Data Set 1.

| Height (m) | Usual Loading | Flood Loading | Seismic-1 Loading | Seismic-2 Loading | Post-Seismic Loading |
|---|---|---|---|---|---|
| 49.50 | -4.709 | -4.709 | -3.579 | -2.773 | -4.709 |
| 45.00 | -94.592 | -63.391 | 31.913 | 430.627 | -94.592 |
| 40.50 | -24.982 | 11.788 | 420.074 | 1599.029 | -24.982 |
| 36.00 | -125.429 | -53.878 | 295.442 | 1082.614 | -125.429 |
| 31.50 | -125.105 | -37.196 | 326.733 | 978.956 | -125.105 |
| 27.00 | -116.103 | -18.837 | 385.510 | 959.487 | -116.103 |
| 22.50 | -107.600 | -4.295 | 451.875 | 962.648 | -107.600 |
| 18.00 | -100.657 | 6.863 | 520.669 | 971.419 | -100.657 |
| 13.50 | -221.099 | -120.674 | 464.215 | 855.410 | -221.099 |
| 9.00 | -234.376 | -131.567 | 516.124 | 849.442 | -234.376 |
| 4.50 | -248.252 | -143.361 | 568.133 | 848.286 | -248.058 |
| 0.00 | -276.151 | -151.834 | 622.916 | 857.962 | -258.063 |

Table 7.14 Downstream Normal Stress Values of Data Set 1.

| Height (m) | Usual Loading | Flood Loading | Seismic-1 Loading | Seismic-2 Loading | Post-Seismic Loading |
|---|---|---|---|---|---|
| 49.50 | -4.709 | -4.709 | -3.955 | -4.761 | -4.709 |
| 45.00 | -120.836 | -126.531 | -203.078 | -601.792 | -120.836 |
| 40.50 | -321.781 | -333.045 | -687.478 | -1866.433 | -321.781 |
| 36.00 | -169.153 | -215.198 | -512.272 | -1299.445 | -169.153 |
| 31.50 | -186.673 | -249.077 | -548.492 | -1200.714 | -186.673 |
| 27.00 | -233.687 | -305.448 | -628.850 | -1202.826 | -233.687 |
| 22.50 | -289.205 | -367.004 | -723.996 | -1234.769 | -289.205 |
| 18.00 | -347.867 | -429.880 | -825.336 | -1276.087 | -347.867 |
| 13.50 | -407.873 | -492.994 | -929.604 | -1320.799 | -407.873 |
| 9.00 | -468.506 | -556.012 | -1035.344 | -1368.663 | -468.506 |
| 4.50 | -515.339 | -605.137 | -1127.762 | -1407.915 | -515.744 |
| 0.00 | -389.516 | -635.574 | -1290.039 | -1125.766 | -526.843 |

Table 7.15 Upstream Normal Stress Values of Data Set 2.

| Height (m) | Usual Loading | Flood Loading | Seismic-1 Loading | Seismic-2 Loading | Post-Seismic Loading |
|---|---|---|---|---|---|
| 49.50 | -4.709 | -4.709 | -3.579 | -1.471 | -4.709 |
| 45.00 | -94.592 | -63.391 | 31.913 | 1093.708 | -94.592 |
| 40.50 | -24.982 | 11.788 | 420.074 | 3675.538 | -24.982 |
| 36.00 | -125.429 | -53.878 | 295.442 | 2607.611 | -125.429 |
| 31.50 | -125.105 | -37.196 | 326.733 | 2371.800 | -125.105 |
| 27.00 | -116.103 | -18.837 | 385.510 | 2313.087 | -116.103 |
| 22.50 | -107.600 | -4.295 | 451.875 | 2301.517 | -107.600 |
| 18.00 | -100.657 | 6.863 | 520.669 | 2298.580 | -100.657 |
| 13.50 | -221.099 | -120.674 | 464.215 | 2165.838 | -221.099 |
| 9.00 | -234.376 | -131.567 | 516.124 | 2134.897 | -234.376 |
| 4.50 | -248.252 | -143.361 | 568.133 | 2099.417 | -248.058 |
| 0.00 | -241.160 | -151.834 | 622.916 | 2065.161 | -258.063 |

Table 7.16 Downstream Normal Stress Values of Data Set 2.

| Height (m) | Usual Loading | Flood Loading | Seismic-1 Loading | Seismic-2 Loading | Post-Seismic Loading |
|---|---|---|---|---|---|
| 49.50 | -4.709 | -4.709 | -3.955 | -6.063 | -4.709 |
| 45.00 | -120.836 | -126.531 | -203.078 | -1264.873 | -120.836 |
| 40.50 | -321.781 | -333.045 | -687.478 | -3942.942 | -321.781 |
| 36.00 | -169.153 | -215.198 | -512.272 | -2824.441 | -169.153 |
| 31.50 | -186.673 | -249.077 | -548.492 | -2593.559 | -186.673 |
| 27.00 | -233.687 | -305.448 | -628.850 | -2556.427 | -233.687 |
| 22.50 | -289.205 | -367.004 | -723.996 | -2573.639 | -289.205 |
| 18.00 | -347.867 | -429.880 | -825.336 | -2603.248 | -347.867 |
| 13.50 | -407.873 | -492.994 | -929.604 | -2631.227 | -407.873 |
| 9.00 | -468.506 | -556.012 | -1035.344 | -2654.117 | -468.506 |
| 4.50 | -515.339 | -605.137 | -1127.762 | -2659.046 | -515.744 |
| 0.00 | -558.500 | -635.574 | -1199.010 | -2641.255 | -544.649 |

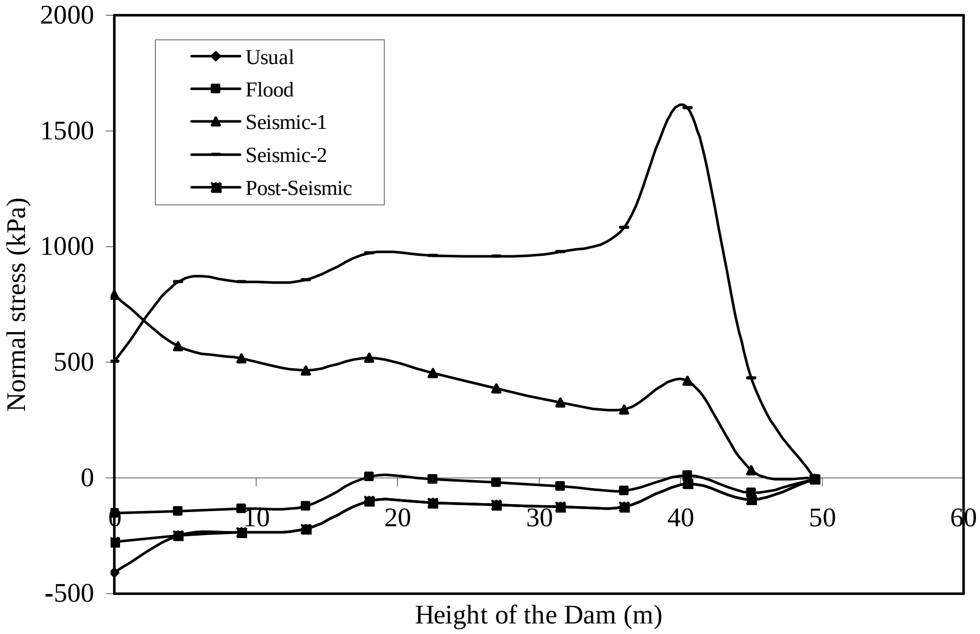


Figure 7.7 Upstream Normal Stress Values of Data Set 1.

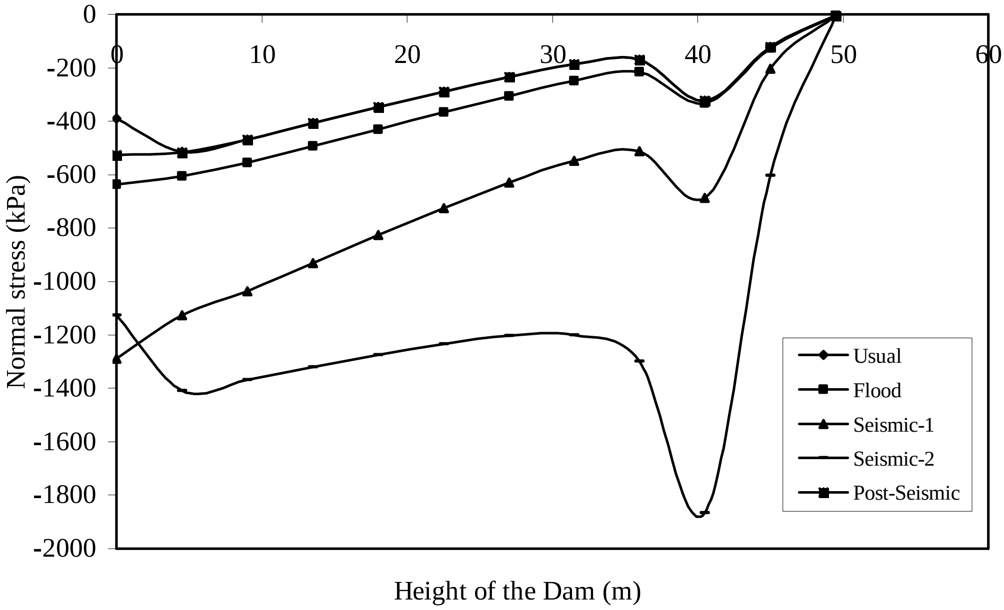


Figure 7.8 Downstream Normal Stress Values of Data Set 1.

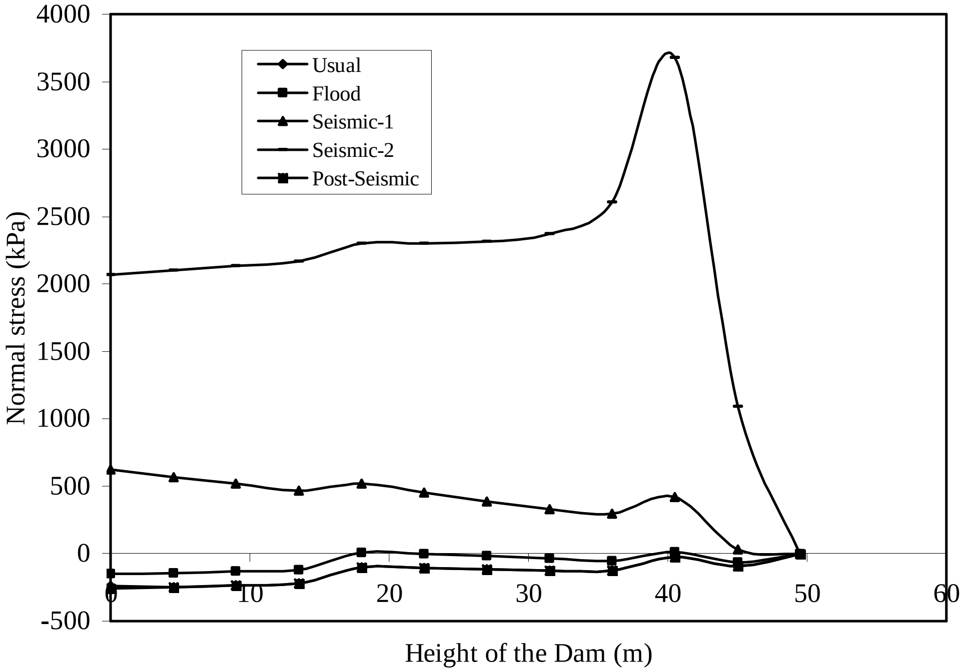


Figure 7.9 Upstream Normal Stress Values of Data Set 2.

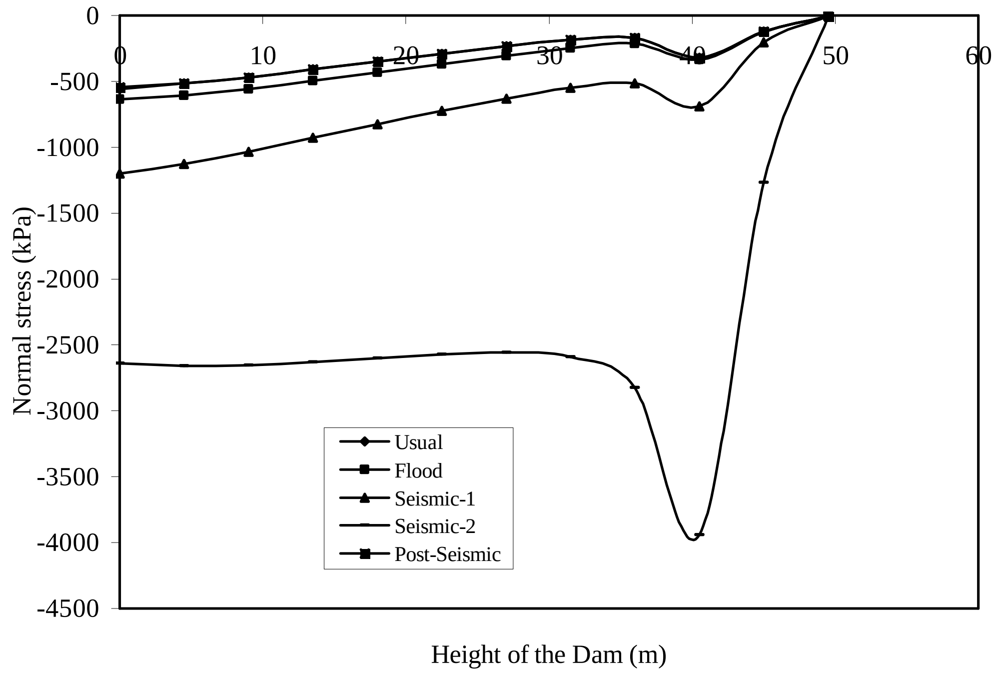


Figure 7.10 Downstream Normal Stress Values of Data Set 2.

The normal stress analyses show that the usual, flood, and post-seismic combinations have compressive values which are within limits, i.e. 1/3 of the allowable compressive stress. When seismic-1 and seismic-2 combinations are investigated, very high tensile stresses for the upstream and very high compressive stresses for the downstream are observed. In the downstream, especially, the compressive stresses for the seismic-2 combination are very high. However, in the upstream, for both of the seismic combinations, the tensile stresses are so high that they exceed the limit value for tension, i.e. 10% of the allowable compressive stress, and may cause the failure of the dam. Porsuk Dam has been under operation for over 30 years. The seismic analyses are performed for very high spectral accelerations, so when the analyses with usual combination are checked, there is no probability of failure for this dam as the seismic forces are not considered. It is not known when and how hard an earthquake occurs but if the accelerations of a possible earthquake are less than the ones that are presented in this thesis, then Porsuk Dam will continue to safely serve its intended mission.

### 7.4 Sensitivity Analysis

The sensitivity analyses are performed to observe the effect of variations in statistical information. Additional data that would be available in the future may change the coefficients of variation of the relevant variables involved in the phenomenon. In fact, various possible combinations for PDFs and coefficients of variation should be considered. To reduce the number of possible combinations, only the following analysis is carried out. To this end, coefficient of variation of each random variable is increased by 10%, 20%, and 30% while the means of these random variables and the corresponding PDFs are kept constant (See Tables 7.18, 7.20, and 7.22).

The analyses are executed using seismic-1 load combination in CADAM because the earthquake effect is included in this kind of combination and it is the same for both data sets 1 and 2 as the horizontal spectral acceleration is not needed (See Tables 7.17, 7.19, 7.21, 7.23). Summary of sensitivity analysis, i.e. the safety factors and failure probabilities, is presented in Table 7.24 (See Figures 7.11 and 7.12). Table 7.17 has been already generated with the initial coefficients of variation (Table 6.6). However, it is also presented here in order to compare the safety factors with the tables of increased coefficients of variation.

Table 7.17 Output with the Initial Coefficients of Variation.

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 2.839 | 0.597 | 1.526 | 5.775 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 0.781 | 0.136 | 0.432 | 1.531 | 0.06716 | 0.93284 |
| Overturning Safety Factor toward Upst. | 4.032 | 0.113 | 3.629 | 4.465 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.183 | 0.131 | 0.825 | 1.893 | 0.93850 | 0.06150 |
| Uplifting Safety Factor | 2.379 | 0.195 | 1.855 | 3.260 | 1.00000 | 0.00000 |

Table 7.18 Random Variables Needed for Probabilistic Analysis with %10 Increased Coefficients of Variation.

| Variable | Mean | Std. Dev. | c.o.v. | PDF |
|---|---|---|---|---|
| Tensile Strength (kPa) | 3000 | 330 | 0.11 | Normal |
| Peak Cohesion (kPa) | 931 | 51.21 | 0.055 | Normal |
| Peak Friction Coefficient | 1.428 | 0.063 | 0.044 | Normal |
| Normal Upstream Reservoir Elevation (m) | 45.6 | 1.877 | 0.041 | Normal |
| Drain Efficiency | 0.6 | 0.198 | 0.33 | Normal |
| Ice Load (kN) | 52 | 17.16 | 0.33 | Normal |
| Horizontal PGA(g) | 0.3 | 0.083 | 0.275 | Normal |

Table 7.19 Output with %10 Increased Coefficients of Variation.

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 2.861 | 0.666 | 1.424 | 6.129 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 0.785 | 0.152 | 0.402 | 1.582 | 0.08896 | 0.91104 |
| Overturning Safety Factor toward Upst. | 4.034 | 0.124 | 3.574 | 4.524 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.186 | 0.146 | 0.797 | 1.938 | 0.91914 | 0.08086 |
| Uplifting Safety Factor | 2.384 | 0.217 | 1.809 | 3.343 | 1.00000 | 0.00000 |

Table 7.20 Random Variables Needed for Probabilistic Analysis with %20 Increased Coefficients of Variation.

| Variable | Mean | Std. Dev. | c.o.v. | PDF |
|---|---|---|---|---|
| Tensile Strength (kPa) | 3000 | 360 | 0.12 | Normal |
| Peak Cohesion (kPa) | 931 | 55.86 | 0.06 | Normal |
| Peak Friction Coefficient | 1.428 | 0.069 | 0.048 | Normal |
| Normal Upstream Reservoir Elevation (m) | 45.6 | 2.05 | 0.045 | Normal |
| Drain Efficiency | 0.6 | 0.216 | 0.36 | Normal |
| Ice Load (kN) | 52 | 18.72 | 0.36 | Normal |
| Horizontal PGA(g) | 0.3 | 0.09 | 0.30 | Normal |

Table 7.21 Output with %20 Increased Coefficients of Variation.

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 2.881 | 0.726 | 1.370 | 7.803 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 0.788 | 0.167 | 0.389 | 1.946 | 0.10692 | 0.89308 |
| Overturning Safety Factor toward Upst. | 4.035 | 0.133 | 3.558 | 4.567 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.189 | 0.160 | 0.785 | 2.262 | 0.90202 | 0.09798 |
| Uplifting Safety Factor | 2.387 | 0.237 | 1.783 | 3.654 | 1.00000 | 0.00000 |

Table 7.22 Random Variables Needed for Probabilistic Analysis with %30 Increased Coefficients of Variation.

| Variable | Mean | Std. Dev. | c.o.v. | PDF |
|---|---|---|---|---|
| Tensile Strength (kPa) | 3000 | 390 | 0.13 | Normal |
| Peak Cohesion (kPa) | 931 | 60.5 | 0.065 | Normal |
| Peak Friction Coefficient | 1.428 | 0.074 | 0.052 | Normal |
| Normal Upstream Reservoir Elevation (m) | 45.6 | 2.217 | 0.049 | Normal |
| Drain Efficiency | 0.6 | 0.234 | 0.39 | Normal |
| Ice Load (kN) | 52 | 20.28 | 0.39 | Normal |
| Horizontal PGA(g) | 0.3 | 0.0975 | 0.325 | Normal |

Table 7.23 Output with %30 Increased Coefficients of Variation.

| Output Parameters | Mean | Standard Deviation | Minimum Value | Maximum Value | Performance Index | Probability of Failure |
|---|---|---|---|---|---|---|
| Upstream crack length (% of joint) | 0.000 | 0.000 | 0.000 | 0.000 | 1.00000 | 0.00000 |
| Sliding Safety Factor (peak) | 2.903 | 0.793 | 1.224 | 7.932 | 1.00000 | 0.00000 |
| Sliding Safety Factor (residual) | 0.792 | 0.183 | 0.348 | 1.921 | 0.12792 | 0.87208 |
| Overturning Safety Factor toward Upst. | 4.036 | 0.143 | 3.540 | 4.547 | 1.00000 | 0.00000 |
| Overturning Safety Factor toward Downst. | 1.192 | 0.175 | 0.743 | 2.238 | 0.88208 | 0.11792 |
| Uplifting Safety Factor | 2.391 | 0.259 | 1.703 | 3.635 | 1.00000 | 0.00000 |

Table 7.24 Summary of Sensitivity Analyses.

| | Safety Factors | | | | |
|---|---|---|---|---|---|
| c.o.v. (δ) multiplier | SSF (Peak) | SSF (Residual) | OSF (Upst.) | OSF (Downst.) | USF |
| 1 | 2.839 | 0.781 | 4.032 | 1.183 | 2.379 |
| 1.1 | 2.861 | 0.785 | 4.034 | 1.186 | 2.384 |
| 1.2 | 2.881 | 0.788 | 4.035 | 1.189 | 2.387 |
| 1.3 | 2.903 | 0.792 | 4.036 | 1.192 | 2.391 |

| | Failure Probabilities | | | | |
|---|---|---|---|---|---|
| c.o.v. (δ) multiplier | SSF (Peak) | SSF (Residual) | OSF (Upst.) | OSF (Downst.) | USF |
| 1 | 0.00000 | 0.93284 | 0.00000 | 0.06150 | 0.00000 |
| 1.1 | 0.00000 | 0.91104 | 0.00000 | 0.08086 | 0.00000 |
| 1.2 | 0.00000 | 0.89308 | 0.00000 | 0.09798 | 0.00000 |
| 1.3 | 0.00000 | 0.87208 | 0.00000 | 0.11792 | 0.00000 |

When Table 7.24 and Figure 7.11 are investigated, it is clearly seen that, all of the means of the safety factors increase as the coefficients of variation increase. This may be due to the fact that increasing coefficients of variation would cover wider ranges of relevant variables involved in safety analysis resulting in an increment in the reliability of the system.

Results of the analysis indicate that the failure probability of the overturning towards the downstream increase whereas the failure probability of residual sliding decrease as the coefficients of variation increase (See Figure 7.12). Since the failure probabilities for uplifting, peak sliding, and overturning towards the upstream safety factors are zero, these are not included in Figure 7.12. As a concluding remark, it can be stated that the effect of increase in the coefficients of variation is not significant in the overall stability. Variations of mean values for the variables can also be tested additionally in sensitivity analysis.

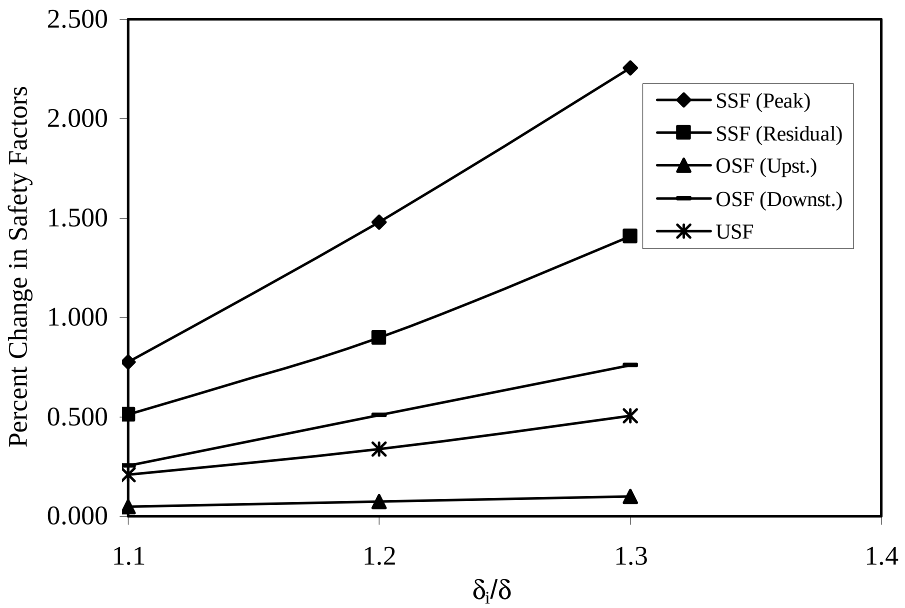


Figure 7.11 Percent Changes of Safety Factors in Sensitivity Analysis.

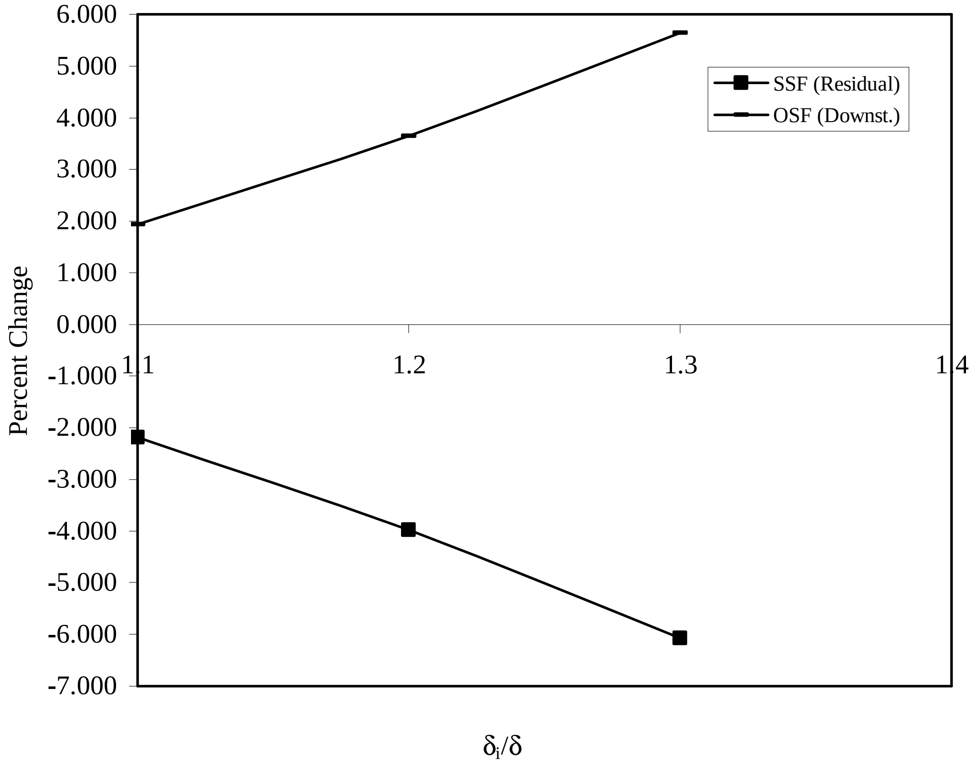


Figure 7.12 Percent Changes of Failure Probabilities in Sensitivity Analysis.

# CHAPTER 8

# CONCLUSIONS AND RECOMMENDATIONS

There is a need to ensure that the design standards and criteria of dams meet contemporary requirements for operational and public safety as dams get older. If a dam is going to be constructed, then besides the safe-design concerns, cost concern is also an important issue. Reliability-based designs decrease the cost since risk is computed using a more realistic basis which reflects the probabilistic nature of all loading and resistance parameters. However, deterministic dam design approaches yield huge project costs since very high safety factors are adopted which are unnecessary for most of the cases. Safety factors over unity, which represents the static equilibrium case, may even lead to high failure probability. Thus, the probabilistic safety analysis is much more realistic and rational than the conventional safety methods.

Risk assessment and risk management can increase the quality and value of the achievements compared to traditional dam engineering approaches. Since the goal is to avoid the dam failures by reducing risk to almost zero with optimum cost, dam safety risk assessment and risk management have a key role in modern dam safety programs. The purpose of risk assessment is to assist decision-makers to make better decisions. When decisions need to be made, based on partial and incomplete information, the decision-maker does not have adequate time to reach a complete, unquestioned solution. In that case, the available imperfect information must be synthesized at a particular stage to represent as closely as possible the state of the knowledge at that time.

This thesis deals with the probabilistic assessment of probability of failure of a concrete gravity dam under various possible failure modes. Needed information is collected from the General Directorate of State Hydraulic Works and the related literature. Risk and dam safety concepts in general are described and the standard mathematical procedures are given in order to have a better realization of the concept. As a case study, Porsuk Dam, which is a concrete gravity dam situated in close vicinity to Eskişehir, is analyzed using CADAM software that is based on the Monte-Carlo simulations to determine the reliability. CADAM is a very useful tool in order to determine the failure probability of a concrete gravity dam provided that the required input data are available. However, it is lack of the definitions of certain terms as random variables. Most important ones are the fundamental period and the modulus of elasticity which can also be defined as random variables for a more realistic analysis.

In research of various dam safety cases, a probabilistic seismic hazard assessment, which takes into account the recurrence rates of potential earthquakes on each fault and the potential ground motion that may result from each of those earthquakes, can be used. It consists of the specifications of the likelihood, magnitude, location, and nature of earthquakes that may severely occur in the region or at the site and estimating the peak ground acceleration. The basis for all seismic hazard assessment is the analysis of seismicity or the occurrence of earthquake in space and time. Seismic hazard analysis requires an assessment of the future earthquake potential of the area under consideration, so that the safety analysis, based on the determined quantities, can be conducted.

Under normal circumstances, Porsuk Dam is determined to be safe but in the condition of a severe earthquake, a high risk of failure is obtained. It should, however, be stated that the analyses are carried out for certain combinations of governing parameters. In fact, several additional scenarios can be generated to observe the variation of dam safety under almost universal conditions. As a concluding remark, it can be stated that the risk-based analysis of Porsuk Dam needs to be supplemented by integrating risk management and risk assessment steps to the methodology, which are assumed to be the scope of a future research.

Probabilistic evaluation of the safety of an existing dam can be achieved using related information obtained from continuous monitoring. To this end, periodic reviews of hazard determinations and safety decisions for all dams should be required, especially when safety evaluations are based on criteria less conservative than the probable maximum flood or the maximum credible earthquake. Research efforts, designed to provide better bases for estimating magnitudes and frequencies of extreme floods and earthquakes, for estimating reactions of dams to such natural hazards, and for establishing acceptable levels of risks, should be continued. As there is progress in seismology, hydrology, meteorology, and the relevant data bases, and as public gets aware of the risk concept, a review of dam safety practices and standards should be periodically formulated in this respect. It is now time for Turkey to develop contemporary dam safety guidelines based on risk analysis and management concepts which should be prepared by the collaborated activities of universities and public agencies.

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# APPENDIX A

## WHITTIER EARTHQUAKE SPECTRUM

The quantities of the symbols that are used in Tables A.2 and A.3 which present the strong motion database spectrum for a certain earthquake are given in Table A.1.

Table A.1 Quantity of Symbols for PEER Database Spectrum (PEER, 2000).

| Symbol | Quantity | Units |
|---|---|---|
| FREQ NO | Frequency Number | none |
| FREQ | Frequency | Hz |
| RD | Relative Displacement (SD) | cm |
| RV | Relative Velocity | cm/sec |
| PRV | Pseudo Relative Velocity (SV) | cm/sec |
| AA | Absolute Acceleration | g |
| PAA | Pseudo Absolute Acceleration (SA) | g |
| MAG | Magnification Ratio (Response Spectral Shape) | (none) |
| PER | Period | seconds |

Table A.2 PEER Strong Motion Database Spectrum, Whittier EQ-1 (PEER, 2000).

Whittier Earthquake Spectrum – 1 (Damping = 10 %)

PROCESSING BY USC. WHITTIER 10/01/87 14:42, SAN GABRIEL-E GRAND AV, 180 (USC STATION 90019)
FILTER POINTS: HP=0.35 Hz LP=25.0 Hz
NO FREQ=112,DAMP=.100:

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 1 | .50000E-01 | .33459E+01 | .22951E+02 | .10511E+01 | .14805E-02 | .33662E-03 | .48760E-02 | .20000E+02 |
| 2 | .66670E-01 | .33654E+01 | .23007E+02 | .14097E+01 | .19963E-02 | .60198E-03 | .65744E-02 | .14999E+02 |
| 3 | .71430E-01 | .33728E+01 | .23033E+02 | .15137E+01 | .21470E-02 | .69254E-03 | .70708E-02 | .13999E+02 |
| 4 | .76920E-01 | .33789E+01 | .23064E+02 | .16330E+01 | .23219E-02 | .80454E-03 | .76468E-02 | .13000E+02 |
| 5 | .83330E-01 | .33939E+01 | .23090E+02 | .17769E+01 | .25247E-02 | .94840E-03 | .83149E-02 | .12000E+02 |
| 6 | .90910E-01 | .34047E+01 | .23135E+02 | .19448E+01 | .27704E-02 | .11324E-02 | .91239E-02 | .10999E+02 |
| 7 | .10000E+00 | .34279E+01 | .23183E+02 | .21538E+01 | .31009E-02 | .13795E-02 | .10212E-01 | .10000E+02 |
| 8 | .10526E+00 | .34415E+01 | .23217E+02 | .22761E+01 | .33207E-02 | .15345E-02 | .10936E-01 | .95002E+01 |
| 9 | .11111E+00 | .34583E+01 | .23253E+02 | .24143E+01 | .35734E-02 | .17181E-02 | .11768E-01 | .90000E+01 |
| 10 | .11765E+00 | .34789E+01 | .23297E+02 | .25716E+01 | .38706E-02 | .19378E-02 | .12747E-01 | .84997E+01 |
| 11 | .12500E+00 | .35029E+01 | .23353E+02 | .27512E+01 | .42250E-02 | .22026E-02 | .13914E-01 | .80000E+01 |
| 12 | .13333E+00 | .35347E+01 | .23420E+02 | .29611E+01 | .46487E-02 | .25287E-02 | .15309E-01 | .75001E+01 |
| 13 | .14286E+00 | .35728E+01 | .23507E+02 | .32070E+01 | .51605E-02 | .29344E-02 | .16995E-01 | .69998E+01 |
| 14 | .15385E+00 | .36206E+01 | .23620E+02 | .34999E+01 | .57901E-02 | .34488E-02 | .19068E-01 | .64998E+01 |
| 15 | .16667E+00 | .36822E+01 | .23772E+02 | .38561E+01 | .65825E-02 | .41164E-02 | .21678E-01 | .59998E+01 |
| 16 | .18182E+00 | .37595E+01 | .23986E+02 | .42949E+01 | .76018E-02 | .50016E-02 | .25035E-01 | .54999E+01 |
| 17 | .20000E+00 | .38562E+01 | .24301E+02 | .48458E+01 | .89500E-02 | .62074E-02 | .29475E-01 | .50000E+01 |
| 18 | .20833E+00 | .39005E+01 | .24468E+02 | .51057E+01 | .96142E-02 | .68127E-02 | .31662E-01 | .48000E+01 |
| 19 | .21739E+00 | .39478E+01 | .24669E+02 | .53923E+01 | .10369E-01 | .75081E-02 | .34149E-01 | .46000E+01 |
| 20 | .22727E+00 | .39976E+01 | .24910E+02 | .57085E+01 | .11231E-01 | .83096E-02 | .36987E-01 | .44000E+01 |
| 21 | .23810E+00 | .40489E+01 | .25201E+02 | .60574E+01 | .12220E-01 | .92375E-02 | .40245E-01 | .41999E+01 |
| 22 | .25000E+00 | .40992E+01 | .25556E+02 | .64390E+01 | .13355E-01 | .10310E-01 | .43982E-01 | .40000E+01 |
| 23 | .26316E+00 | .41452E+01 | .25990E+02 | .68541E+01 | .14661E-01 | .11552E-01 | .48285E-01 | .37999E+01 |
| 24 | .27778E+00 | .41819E+01 | .26522E+02 | .72989E+01 | .16163E-01 | .12985E-01 | .53231E-01 | .35999E+01 |
| 25 | .29412E+00 | .42081E+01 | .27170E+02 | .77767E+01 | .17877E-01 | .14649E-01 | .58876E-01 | .33999E+01 |
| 26 | .31250E+00 | .42090E+01 | .27958E+02 | .82645E+01 | .19807E-01 | .16541E-01 | .65231E-01 | .32000E+01 |

Table A.2 PEER Strong Motion Database Spectrum, Whittier EQ-1 (PEER, 2000) (continued-1).

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 27 | .33333E+00 | .41632E+01 | .28897E+02 | .87193E+01 | .21920E-01 | .18615E-01 | .72192E-01 | .30000E+01 |
| 28 | .35714E+00 | .40464E+01 | .29983E+02 | .90800E+01 | .24115E-01 | .20770E-01 | .79419E-01 | .28000E+01 |
| 29 | .38462E+00 | .45297E+01 | .31164E+02 | .10946E+02 | .30118E-01 | .26966E-01 | .99188E-01 | .25999E+01 |
| 30 | .41667E+00 | .51751E+01 | .32294E+02 | .13548E+02 | .39435E-01 | .36157E-01 | .12987E+00 | .23999E+01 |
| 31 | .45455E+00 | .58027E+01 | .33061E+02 | .16572E+02 | .51787E-01 | .48249E-01 | .17055E+00 | .21999E+01 |
| 32 | .50000E+00 | .62432E+01 | .32911E+02 | .19613E+02 | .67004E-01 | .62812E-01 | .22066E+00 | .20000E+01 |
| 33 | .52632E+00 | .63146E+01 | .35233E+02 | .20882E+02 | .74990E-01 | .70394E-01 | .24696E+00 | .18999E+01 |
| 34 | .55555E+00 | .62428E+01 | .37350E+02 | .21791E+02 | .82727E-01 | .77538E-01 | .27244E+00 | .18000E+01 |
| 35 | .58824E+00 | .60087E+01 | .38810E+02 | .22208E+02 | .89116E-01 | .83672E-01 | .29348E+00 | .16999E+01 |
| 36 | .62500E+00 | .57268E+01 | .39136E+02 | .22489E+02 | .93789E-01 | .90025E-01 | .30887E+00 | .16000E+01 |
| 37 | .66667E+00 | .61296E+01 | .37938E+02 | .25675E+02 | .11506E+00 | .10963E+00 | .37895E+00 | .14999E+01 |
| 38 | .71429E+00 | .62002E+01 | .35197E+02 | .27826E+02 | .13374E+00 | .12730E+00 | .44046E+00 | .13999E+01 |
| 39 | .76923E+00 | .61683E+01 | .35861E+02 | .29812E+02 | .15179E+00 | .14688E+00 | .49992E+00 | .13000E+01 |
| 40 | .83333E+00 | .58916E+01 | .40058E+02 | .30848E+02 | .16950E+00 | .16465E+00 | .55822E+00 | .12000E+01 |
| 41 | .90909E+00 | .54763E+01 | .41686E+02 | .31280E+02 | .18637E+00 | .18213E+00 | .61380E+00 | .11000E+01 |
| 42 | .10000E+01 | .57665E+01 | .42928E+02 | .36232E+02 | .23785E+00 | .23206E+00 | .78332E+00 | .10000E+01 |
| 43 | .10526E+01 | .62801E+01 | .44532E+02 | .41536E+02 | .28639E+00 | .28003E+00 | .94320E+00 | .95000E+00 |
| 44 | .11111E+01 | .68029E+01 | .50355E+02 | .47493E+02 | .34775E+00 | .33799E+00 | .11452E+01 | .90000E+00 |
| 45 | .11764E+01 | .73797E+01 | .56282E+02 | .54551E+02 | .42091E+00 | .41105E+00 | .13862E+01 | .85000E+00 |
| 46 | .12500E+01 | .76965E+01 | .64487E+02 | .60448E+02 | .49512E+00 | .48395E+00 | .16305E+01 | .80000E+00 |
| 47 | .13333E+01 | .77902E+01 | .69398E+02 | .65263E+02 | .56877E+00 | .55733E+00 | .18731E+01 | .75000E+00 |
| 48 | .14285E+01 | .73119E+01 | .67344E+02 | .65631E+02 | .61057E+00 | .60051E+00 | .20107E+01 | .70000E+00 |
| 49 | .15000E+01 | .69845E+01 | .63424E+02 | .65827E+02 | .64406E+00 | .63242E+00 | .21210E+01 | .66666E+00 |
| 50 | .15384E+01 | .67572E+01 | .63297E+02 | .65318E+02 | .65557E+00 | .64362E+00 | .21590E+01 | .65000E+00 |
| 51 | .16666E+01 | .59699E+01 | .58640E+02 | .62517E+02 | .67942E+00 | .66735E+00 | .22375E+01 | .59999E+00 |
| 52 | .18181E+01 | .49191E+01 | .49503E+02 | .56196E+02 | .66537E+00 | .65442E+00 | .21912E+01 | .55000E+00 |
| 53 | .20000E+01 | .40460E+01 | .45745E+02 | .50843E+02 | .66065E+00 | .65129E+00 | .21757E+01 | .50000E+00 |
| 54 | .20833E+01 | .37212E+01 | .43647E+02 | .48710E+02 | .66301E+00 | .64997E+00 | .21835E+01 | .48000E+00 |
| 55 | .21739E+01 | .34520E+01 | .42406E+02 | .47151E+02 | .66702E+00 | .65651E+00 | .21967E+01 | .46000E+00 |
| 56 | .22727E+01 | .31094E+01 | .39658E+02 | .44403E+02 | .65591E+00 | .64635E+00 | .21601E+01 | .43999E+00 |
| 57 | .23809E+01 | .26962E+01 | .35050E+02 | .40336E+02 | .62223E+00 | .61511E+00 | .20492E+01 | .42000E+00 |
| 58 | .25000E+01 | .22097E+01 | .27901E+02 | .34709E+02 | .55935E+00 | .55578E+00 | .18421E+01 | .40000E+00 |
| 59 | .26315E+01 | .19323E+01 | .23888E+02 | .31950E+02 | .54928E+00 | .53852E+00 | .18089E+01 | .37999E+00 |

Table A.2 PEER Strong Motion Database Spectrum, Whittier EQ-1 (PEER, 2000) (continued-2).

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 60 | .27777E+01 | .16365E+01 | .20999E+02 | .28562E+02 | .51364E+00 | .50816E+00 | .16915E+01 | .36000E+00 |
| 61 | .29411E+01 | .12945E+01 | .19063E+02 | .23923E+02 | .45165E+00 | .45066E+00 | .14874E+01 | .33999E+00 |
| 62 | .31250E+01 | .10504E+01 | .18388E+02 | .20624E+02 | .41811E+00 | .41281E+00 | .13769E+01 | .32000E+00 |
| 63 | .33333E+01 | .93812E+00 | .16668E+02 | .19648E+02 | .41881E+00 | .41947E+00 | .13792E+01 | .30000E+00 |
| 64 | .34500E+01 | .86051E+00 | .15798E+02 | .18653E+02 | .42452E+00 | .41217E+00 | .13981E+01 | .28985E+00 |
| 65 | .35714E+01 | .79865E+00 | .14504E+02 | .17921E+02 | .41651E+00 | .40995E+00 | .13717E+01 | .27999E+00 |
| 66 | .38461E+01 | .62121E+00 | .11621E+02 | .15012E+02 | .38191E+00 | .36981E+00 | .12577E+01 | .26000E+00 |
| 67 | .41666E+01 | .54602E+00 | .11851E+02 | .14294E+02 | .39570E+00 | .38149E+00 | .13031E+01 | .23999E+00 |
| 68 | .45454E+01 | .56736E+00 | .14413E+02 | .16203E+02 | .47732E+00 | .47174E+00 | .15719E+01 | .22000E+00 |
| 69 | .50000E+01 | .52122E+00 | .11953E+02 | .16374E+02 | .52136E+00 | .52438E+00 | .17170E+01 | .20000E+00 |
| 70 | .52631E+01 | .48193E+00 | .10861E+02 | .15937E+02 | .54556E+00 | .53724E+00 | .17967E+01 | .18999E+00 |
| 71 | .55555E+01 | .42813E+00 | .95688E+01 | .14944E+02 | .53605E+00 | .53178E+00 | .17653E+01 | .17999E+00 |
| 72 | .58823E+01 | .35307E+00 | .10273E+02 | .13049E+02 | .49970E+00 | .49164E+00 | .16456E+01 | .17000E+00 |
| 73 | .62500E+01 | .31498E+00 | .10936E+02 | .12369E+02 | .50258E+00 | .49516E+00 | .16551E+01 | .16000E+00 |
| 74 | .66666E+01 | .29223E+00 | .10473E+02 | .12241E+02 | .53468E+00 | .52269E+00 | .17608E+01 | .14999E+00 |
| 75 | .71428E+01 | .24496E+00 | .91498E+01 | .10994E+02 | .51454E+00 | .50296E+00 | .16945E+01 | .13999E+00 |
| 76 | .75000E+01 | .21020E+00 | .81866E+01 | .99058E+01 | .47604E+00 | .47584E+00 | .15677E+01 | .13333E+00 |
| 77 | .76923E+01 | .19285E+00 | .77397E+01 | .93213E+01 | .46884E+00 | .45924E+00 | .15440E+01 | .13000E+00 |
| 78 | .83333E+01 | .17276E+00 | .68293E+01 | .90462E+01 | .48464E+00 | .48283E+00 | .15960E+01 | .12000E+00 |
| 79 | .90909E+01 | .14063E+00 | .58625E+01 | .80329E+01 | .47581E+00 | .46772E+00 | .15670E+01 | .11000E+00 |
| 80 | .10000E+02 | .10859E+00 | .49744E+01 | .68230E+01 | .45042E+00 | .43700E+00 | .14833E+01 | .10000E+00 |
| 81 | .10526E+02 | .10062E+00 | .46849E+01 | .66550E+01 | .45243E+00 | .44867E+00 | .14899E+01 | .95000E-01 |
| 82 | .11111E+02 | .92611E-01 | .44290E+01 | .64654E+01 | .46160E+00 | .46011E+00 | .15202E+01 | .90000E-01 |
| 83 | .11764E+02 | .82253E-01 | .39316E+01 | .60801E+01 | .46599E+00 | .45815E+00 | .15346E+01 | .85000E-01 |
| 84 | .12500E+02 | .72657E-01 | .33653E+01 | .57065E+01 | .45702E+00 | .45686E+00 | .15051E+01 | .80000E-01 |
| 85 | .13333E+02 | .62929E-01 | .26687E+01 | .52719E+01 | .45661E+00 | .45021E+00 | .15037E+01 | .75000E-01 |
| 86 | .14285E+02 | .53625E-01 | .20430E+01 | .48133E+01 | .43987E+00 | .44041E+00 | .14486E+01 | .70000E-01 |
| 87 | .15000E+02 | .45490E-01 | .17080E+01 | .42873E+01 | .41595E+00 | .41189E+00 | .13698E+01 | .66666E-01 |
| 88 | .15384E+02 | .42040E-01 | .15397E+01 | .40638E+01 | .40378E+00 | .40043E+00 | .13298E+01 | .65000E-01 |
| 89 | .16666E+02 | .33342E-01 | .12659E+01 | .34916E+01 | .37394E+00 | .37272E+00 | .12315E+01 | .59999E-01 |
| 90 | .18181E+02 | .26615E-01 | .95285E+00 | .30405E+01 | .35426E+00 | .35407E+00 | .11666E+01 | .55000E-01 |
| 91 | .20000E+02 | .21484E-01 | .67594E+00 | .26997E+01 | .34578E+00 | .34583E+00 | .11387E+01 | .50000E-01 |
| 92 | .20833E+02 | .19554E-01 | .61040E+00 | .25596E+01 | .34332E+00 | .34154E+00 | .11306E+01 | .48000E-01 |

Table A.2 PEER Strong Motion Database Spectrum, Whittier EQ-1 (PEER, 2000) (continued-3).

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 93 | .21739E+02 | .17737E-01 | .54380E+00 | .24228E+01 | .33968E+00 | .33734E+00 | .11186E+01 | .46000E-01 |
| 94 | .22727E+02 | .16150E-01 | .48490E+00 | .23062E+01 | .33796E+00 | .33571E+00 | .11130E+01 | .43999E-01 |
| 95 | .23809E+02 | .14690E-01 | .44234E+00 | .21976E+01 | .33652E+00 | .33513E+00 | .11082E+01 | .42000E-01 |
| 96 | .25000E+02 | .13235E-01 | .40609E+00 | .20790E+01 | .33323E+00 | .33290E+00 | .10974E+01 | .40000E-01 |
| 97 | .28000E+02 | .10266E-01 | .31219E+00 | .18060E+01 | .32521E+00 | .32389E+00 | .10710E+01 | .35714E-01 |
| 98 | .31000E+02 | .82718E-02 | .26174E+00 | .16111E+01 | .32122E+00 | .31990E+00 | .10578E+01 | .32258E-01 |
| 99 | .34000E+02 | .69125E-02 | .26690E+00 | .14767E+01 | .32248E+00 | .32157E+00 | .10620E+01 | .29411E-01 |
| 100 | .40000E+02 | .48010E-02 | .25546E+00 | .12066E+01 | .31084E+00 | .30913E+00 | .10237E+01 | .25000E-01 |
| 101 | .45000E+02 | .38220E-02 | .19816E+00 | .10806E+01 | .31096E+00 | .31146E+00 | .10240E+01 | .22222E-01 |
| 102 | .50000E+02 | .30710E-02 | .15042E+00 | .96479E+00 | .30965E+00 | .30897E+00 | .10197E+01 | .20000E-01 |
| 103 | .55000E+02 | .25541E-02 | .12981E+00 | .88265E+00 | .31189E+00 | .31093E+00 | .10271E+01 | .18181E-01 |
| 104 | .60000E+02 | .21712E-02 | .10803E+00 | .81855E+00 | .31498E+00 | .31456E+00 | .10373E+01 | .16666E-01 |
| 105 | .65000E+02 | .18396E-02 | .80037E-01 | .75131E+00 | .31343E+00 | .31278E+00 | .10322E+01 | .15384E-01 |
| 106 | .70000E+02 | .15676E-02 | .56872E-01 | .68950E+00 | .30958E+00 | .30913E+00 | .10195E+01 | .14285E-01 |
| 107 | .75000E+02 | .13534E-02 | .46844E-01 | .63778E+00 | .30675E+00 | .30636E+00 | .10102E+01 | .13333E-01 |
| 108 | .80000E+02 | .11807E-02 | .41931E-01 | .59351E+00 | .30451E+00 | .30411E+00 | .10028E+01 | .12500E-01 |
| 109 | .85000E+02 | .10393E-02 | .42708E-01 | .55510E+00 | .30249E+00 | .30220E+00 | .99621E+00 | .11764E-01 |
| 110 | .90000E+02 | .92914E-03 | .42162E-01 | .52542E+00 | .30273E+00 | .30287E+00 | .99699E+00 | .11111E-01 |
| 111 | .95000E+02 | .83943E-03 | .39744E-01 | .50106E+00 | .30486E+00 | .30487E+00 | .10040E+01 | .10526E-01 |
| 112 | .10000E+03 | .75917E-03 | .36160E-01 | .47700E+00 | .30601E+00 | .30551E+00 | .10077E+01 | .10000E-01 |

Table A.3 PEER Strong Motion Database Spectrum, Whittier EQ-2 (PEER, 2000).

Whittier Earthquake Spectrum – 2 (Damping = 15 %)

PROCESSING BY USC.WHITTIER 10/01/87 14:42, SAN GABRIEL-E GRAND AV, 180 (USC STATION 90019)
FILTER POINTS: HP=0.35 Hz LP=25.0 Hz
NO FREQ=112,DAMP=.150:

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 1 | .50000E-01 | .33496E+01 | .22997E+02 | .10523E+01 | .21933E-02 | .33700E-03 | .72232E-02 | .20000E+02 |
| 2 | .66670E-01 | .33661E+01 | .23090E+02 | .14101E+01 | .29519E-02 | .60213E-03 | .97215E-02 | .14999E+02 |
| 3 | .71430E-01 | .33733E+01 | .23114E+02 | .15139E+01 | .31698E-02 | .69264E-03 | .10439E-01 | .13999E+02 |
| 4 | .76920E-01 | .33786E+01 | .23145E+02 | .16329E+01 | .34242E-02 | .80448E-03 | .11277E-01 | .13000E+02 |
| 5 | .83330E-01 | .33885E+01 | .23185E+02 | .17741E+01 | .37233E-02 | .94689E-03 | .12262E-01 | .12000E+02 |
| 6 | .90910E-01 | .34042E+01 | .23231E+02 | .19445E+01 | .40771E-02 | .11322E-02 | .13427E-01 | .10999E+02 |
| 7 | .10000E+00 | .34255E+01 | .23297E+02 | .21523E+01 | .45074E-02 | .13785E-02 | .14844E-01 | .10000E+02 |
| 8 | .10526E+00 | .34373E+01 | .23338E+02 | .22733E+01 | .47590E-02 | .15326E-02 | .15673E-01 | .95002E+01 |
| 9 | .11111E+00 | .34526E+01 | .23381E+02 | .24103E+01 | .50382E-02 | .17153E-02 | .16592E-01 | .90000E+01 |
| 10 | .11765E+00 | .34710E+01 | .23436E+02 | .25658E+01 | .53539E-02 | .19334E-02 | .17632E-01 | .84997E+01 |
| 11 | .12500E+00 | .34930E+01 | .23502E+02 | .27434E+01 | .57119E-02 | .21964E-02 | .18811E-01 | .80000E+01 |
| 12 | .13333E+00 | .35213E+01 | .23582E+02 | .29499E+01 | .61195E-02 | .25191E-02 | .20153E-01 | .75001E+01 |
| 13 | .14286E+00 | .35550E+01 | .23683E+02 | .31910E+01 | .66293E-02 | .29198E-02 | .21832E-01 | .69998E+01 |
| 14 | .15385E+00 | .35966E+01 | .23813E+02 | .34767E+01 | .73414E-02 | .34259E-02 | .24177E-01 | .64998E+01 |
| 15 | .16667E+00 | .36480E+01 | .23986E+02 | .38202E+01 | .82203E-02 | .40781E-02 | .27072E-01 | .59998E+01 |
| 16 | .18182E+00 | .37102E+01 | .24222E+02 | .42385E+01 | .93266E-02 | .49359E-02 | .30715E-01 | .54999E+01 |
| 17 | .20000E+00 | .37851E+01 | .24558E+02 | .47566E+01 | .10754E-01 | .60931E-02 | .35417E-01 | .50000E+01 |
| 18 | .20833E+00 | .38180E+01 | .24732E+02 | .49976E+01 | .11458E-01 | .66685E-02 | .37735E-01 | .48000E+01 |
| 19 | .21739E+00 | .38514E+01 | .24938E+02 | .52607E+01 | .12247E-01 | .73248E-02 | .40334E-01 | .46000E+01 |
| 20 | .22727E+00 | .38854E+01 | .25180E+02 | .55482E+01 | .13136E-01 | .80762E-02 | .43262E-01 | .44000E+01 |
| 21 | .23810E+00 | .39176E+01 | .25468E+02 | .58608E+01 | .14139E-01 | .89378E-02 | .46565E-01 | .41999E+01 |
| 22 | .25000E+00 | .39461E+01 | .25810E+02 | .61986E+01 | .15272E-01 | .99253E-02 | .50295E-01 | .40000E+01 |
| 23 | .26316E+00 | .39693E+01 | .26218E+02 | .65631E+01 | .16551E-01 | .11062E-01 | .54508E-01 | .37999E+01 |
| 24 | .27778E+00 | .39863E+01 | .26706E+02 | .69574E+01 | .17990E-01 | .12378E-01 | .59248E-01 | .35999E+01 |
| 25 | .29412E+00 | .39844E+01 | .27285E+02 | .73633E+01 | .19596E-01 | .13871E-01 | .64537E-01 | .33999E+01 |

Table A.3 PEER Strong Motion Database Spectrum, Whittier EQ-2 (PEER, 2000) (continued-1).

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 26 | .31250E+00 | .39527E+01 | .27965E+02 | .77611E+01 | .21359E-01 | .15534E-01 | .70342E-01 | .32000E+01 |
| 27 | .33333E+00 | .38753E+01 | .28746E+02 | .81163E+01 | .24174E-01 | .17327E-01 | .79615E-01 | .30000E+01 |
| 28 | .35714E+00 | .39752E+01 | .29610E+02 | .89204E+01 | .28782E-01 | .20405E-01 | .94791E-01 | .28000E+01 |
| 29 | .38462E+00 | .44357E+01 | .30496E+02 | .10719E+02 | .35032E-01 | .26407E-01 | .11537E+00 | .25999E+01 |
| 30 | .41667E+00 | .49272E+01 | .31267E+02 | .12899E+02 | .43427E-01 | .34425E-01 | .14302E+00 | .23999E+01 |
| 31 | .45455E+00 | .53814E+01 | .31671E+02 | .15369E+02 | .54348E-01 | .44745E-01 | .17898E+00 | .21999E+01 |
| 32 | .50000E+00 | .56690E+01 | .31614E+02 | .17809E+02 | .67541E-01 | .57034E-01 | .22243E+00 | .20000E+01 |
| 33 | .52632E+00 | .57039E+01 | .33276E+02 | .18862E+02 | .74513E-01 | .63587E-01 | .24539E+00 | .18999E+01 |
| 34 | .55555E+00 | .56308E+01 | .34669E+02 | .19655E+02 | .81180E-01 | .69937E-01 | .26735E+00 | .18000E+01 |
| 35 | .58824E+00 | .54174E+01 | .35520E+02 | .20022E+02 | .86925E-01 | .75438E-01 | .28627E+00 | .16999E+01 |
| 36 | .62500E+00 | .50769E+01 | .35533E+02 | .19937E+02 | .91058E-01 | .79809E-01 | .29988E+00 | .16000E+01 |
| 37 | .66667E+00 | .53589E+01 | .34498E+02 | .22447E+02 | .10873E+00 | .95849E-01 | .35810E+00 | .14999E+01 |
| 38 | .71429E+00 | .54697E+01 | .32556E+02 | .24548E+02 | .12555E+00 | .11230E+00 | .41348E+00 | .13999E+01 |
| 39 | .76923E+00 | .53248E+01 | .31708E+02 | .25736E+02 | .13985E+00 | .12679E+00 | .46057E+00 | .13000E+01 |
| 40 | .83333E+00 | .50807E+01 | .35074E+02 | .26602E+02 | .15372E+00 | .14198E+00 | .50624E+00 | .12000E+01 |
| 41 | .90909E+00 | .48615E+01 | .36360E+02 | .27768E+02 | .17535E+00 | .16168E+00 | .57750E+00 | .11000E+01 |
| 42 | .10000E+01 | .51306E+01 | .36607E+02 | .32236E+02 | .22241E+00 | .20647E+00 | .73246E+00 | .10000E+01 |
| 43 | .10526E+01 | .54606E+01 | .39215E+02 | .36115E+02 | .26055E+00 | .24349E+00 | .85808E+00 | .95000E+00 |
| 44 | .11111E+01 | .58085E+01 | .42901E+02 | .40551E+02 | .30719E+00 | .28858E+00 | .10116E+01 | .90000E+00 |
| 45 | .11764E+01 | .60985E+01 | .46785E+02 | .45080E+02 | .35812E+00 | .33968E+00 | .11794E+01 | .85000E+00 |
| 46 | .12500E+01 | .62015E+01 | .51805E+02 | .48707E+02 | .40457E+00 | .38995E+00 | .13323E+01 | .80000E+00 |
| 47 | .13333E+01 | .60273E+01 | .54211E+02 | .50494E+02 | .44480E+00 | .43121E+00 | .14648E+01 | .75000E+00 |
| 48 | .14285E+01 | .57277E+01 | .51711E+02 | .51412E+02 | .49000E+00 | .47041E+00 | .16137E+01 | .70000E+00 |
| 49 | .15000E+01 | .54767E+01 | .51467E+02 | .51617E+02 | .51409E+00 | .49590E+00 | .16930E+01 | .66666E+00 |
| 50 | .15384E+01 | .52962E+01 | .51016E+02 | .51195E+02 | .52588E+00 | .50446E+00 | .17318E+01 | .65000E+00 |
| 51 | .16666E+01 | .46785E+01 | .46719E+02 | .48993E+02 | .54553E+00 | .52299E+00 | .17966E+01 | .59999E+00 |
| 52 | .18181E+01 | .40027E+01 | .40350E+02 | .45726E+02 | .54726E+00 | .53249E+00 | .18023E+01 | .55000E+00 |
| 53 | .20000E+01 | .32913E+01 | .35934E+02 | .41360E+02 | .55060E+00 | .52981E+00 | .18133E+01 | .50000E+00 |
| 54 | .20833E+01 | .30821E+01 | .34461E+02 | .40344E+02 | .55601E+00 | .53833E+00 | .18311E+01 | .48000E+00 |
| 55 | .21739E+01 | .28422E+01 | .32988E+02 | .38821E+02 | .55753E+00 | .54054E+00 | .18361E+01 | .46000E+00 |
| 56 | .22727E+01 | .25511E+01 | .30869E+02 | .36430E+02 | .54777E+00 | .53030E+00 | .18039E+01 | .43999E+00 |
| 57 | .23809E+01 | .22163E+01 | .28079E+02 | .33155E+02 | .52136E+00 | .50561E+00 | .17170E+01 | .42000E+00 |
| 58 | .25000E+01 | .18644E+01 | .23529E+02 | .29286E+02 | .48294E+00 | .46893E+00 | .15904E+01 | .40000E+00 |

Table A.3 PEER Strong Motion Database Spectrum, Whittier EQ-2 (PEER, 2000) (continued-2).

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 59 | .26315E+01 | .16075E+01 | .19373E+02 | .26580E+02 | .46345E+00 | .44801E+00 | .15263E+01 | .37999E+00 |
| 60 | .27777E+01 | .13716E+01 | .16903E+02 | .23939E+02 | .43644E+00 | .42591E+00 | .14373E+01 | .36000E+00 |
| 61 | .29411E+01 | .11093E+01 | .15933E+02 | .20500E+02 | .39795E+00 | .38617E+00 | .13105E+01 | .33999E+00 |
| 62 | .31250E+01 | .85845E+00 | .15250E+02 | .16855E+02 | .34986E+00 | .33737E+00 | .11522E+01 | .32000E+00 |
| 63 | .33333E+01 | .77000E+00 | .13840E+02 | .16126E+02 | .35489E+00 | .34430E+00 | .11687E+01 | .30000E+00 |
| 64 | .34500E+01 | .72667E+00 | .13127E+02 | .15752E+02 | .35973E+00 | .34807E+00 | .11847E+01 | .28985E+00 |
| 65 | .35714E+01 | .68700E+00 | .12127E+02 | .15416E+02 | .36013E+00 | .35264E+00 | .11860E+01 | .27999E+00 |
| 66 | .38461E+01 | .58658E+00 | .10569E+02 | .14175E+02 | .35174E+00 | .34920E+00 | .11583E+01 | .26000E+00 |
| 67 | .41666E+01 | .52862E+00 | .10093E+02 | .13839E+02 | .37276E+00 | .36932E+00 | .12276E+01 | .23999E+00 |
| 68 | .45454E+01 | .49387E+00 | .10790E+02 | .14105E+02 | .42856E+00 | .41064E+00 | .14114E+01 | .22000E+00 |
| 69 | .50000E+01 | .45156E+00 | .10052E+02 | .14186E+02 | .46671E+00 | .45431E+00 | .15370E+01 | .20000E+00 |
| 70 | .52631E+01 | .41854E+00 | .92427E+01 | .13840E+02 | .47982E+00 | .46657E+00 | .15802E+01 | .18999E+00 |
| 71 | .55555E+01 | .37603E+00 | .88271E+01 | .13126E+02 | .47903E+00 | .46705E+00 | .15776E+01 | .17999E+00 |
| 72 | .58823E+01 | .32611E+00 | .88370E+01 | .12053E+02 | .46825E+00 | .45410E+00 | .15421E+01 | .17000E+00 |
| 73 | .62500E+01 | .28418E+00 | .89372E+01 | .11159E+02 | .45484E+00 | .44673E+00 | .14979E+01 | .16000E+00 |
| 74 | .66666E+01 | .25272E+00 | .83422E+01 | .10585E+02 | .46349E+00 | .45201E+00 | .15264E+01 | .14999E+00 |
| 75 | .71428E+01 | .21774E+00 | .74711E+01 | .97722E+01 | .46132E+00 | .44707E+00 | .15192E+01 | .13999E+00 |
| 76 | .75000E+01 | .19340E+00 | .66876E+01 | .91139E+01 | .44191E+00 | .43780E+00 | .14553E+01 | .13333E+00 |
| 77 | .76923E+01 | .18003E+00 | .62844E+01 | .87013E+01 | .43344E+00 | .42870E+00 | .14274E+01 | .13000E+00 |
| 78 | .83333E+01 | .14605E+00 | .56794E+01 | .76475E+01 | .41152E+00 | .40817E+00 | .13552E+01 | .12000E+00 |
| 79 | .90909E+01 | .12052E+00 | .50084E+01 | .68845E+01 | .41538E+00 | .40085E+00 | .13680E+01 | .11000E+00 |
| 80 | .10000E+02 | .10149E+00 | .43416E+01 | .63769E+01 | .42221E+00 | .40843E+00 | .13904E+01 | .10000E+00 |
| 81 | .10526E+02 | .94780E-01 | .40267E+01 | .62686E+01 | .42686E+00 | .42262E+00 | .14057E+01 | .95000E-01 |
| 82 | .11111E+02 | .86179E-01 | .37049E+01 | .60164E+01 | .43636E+00 | .42816E+00 | .14370E+01 | .90000E-01 |
| 83 | .11764E+02 | .76565E-01 | .32987E+01 | .56597E+01 | .43797E+00 | .42646E+00 | .14424E+01 | .85000E-01 |
| 84 | .12500E+02 | .68100E-01 | .28615E+01 | .53486E+01 | .43033E+00 | .42821E+00 | .14172E+01 | .80000E-01 |
| 85 | .13333E+02 | .58790E-01 | .22988E+01 | .49252E+01 | .42820E+00 | .42060E+00 | .14101E+01 | .75000E-01 |
| 86 | .14285E+02 | .49747E-01 | .18416E+01 | .44653E+01 | .41338E+00 | .40857E+00 | .13613E+01 | .70000E-01 |
| 87 | .15000E+02 | .43676E-01 | .15805E+01 | .41163E+01 | .39583E+00 | .39547E+00 | .13035E+01 | .66666E-01 |
| 88 | .15384E+02 | .40663E-01 | .14708E+01 | .39307E+01 | .39174E+00 | .38731E+00 | .12901E+01 | .65000E-01 |
| 89 | .16666E+02 | .32639E-01 | .11729E+01 | .34179E+01 | .37024E+00 | .36486E+00 | .12193E+01 | .59999E-01 |
| 90 | .18181E+02 | .26376E-01 | .89598E+00 | .30132E+01 | .35366E+00 | .35089E+00 | .11647E+01 | .55000E-01 |

Table A.3  PEER Strong Motion Database Spectrum, Whittier EQ-2 (PEER, 2000) (continued-3).

| NO | FREQ | RD | RV | PRV | AA | PAA | MAG RAT | PER |
|---|---|---|---|---|---|---|---|---|
| 91 | .20000E+02 | .21225E-01 | .65021E+00 | .26672E+01 | .34274E+00 | .34166E+00 | .11287E+01 | .50000E-01 |
| 92 | .20833E+02 | .19375E-01 | .58477E+00 | .25362E+01 | .34062E+00 | .33842E+00 | .11217E+01 | .48000E-01 |
| 93 | .21739E+02 | .17618E-01 | .52410E+00 | .24064E+01 | .33771E+00 | .33506E+00 | .11121E+01 | .46000E-01 |
| 94 | .22727E+02 | .15964E-01 | .47967E+00 | .22797E+01 | .33510E+00 | .33185E+00 | .11036E+01 | .43999E-01 |
| 95 | .23809E+02 | .14452E-01 | .44063E+00 | .21621E+01 | .33248E+00 | .32971E+00 | .10949E+01 | .42000E-01 |
| 96 | .25000E+02 | .13030E-01 | .40183E+00 | .20467E+01 | .32924E+00 | .32773E+00 | .10842E+01 | .40000E-01 |
| 97 | .28000E+02 | .10153E-01 | .31788E+00 | .17863E+01 | .32309E+00 | .32034E+00 | .10640E+01 | .35714E-01 |
| 98 | .31000E+02 | .82009E-02 | .26538E+00 | .15973E+01 | .31909E+00 | .31715E+00 | .10508E+01 | .32258E-01 |
| 99 | .34000E+02 | .67805E-02 | .24149E+00 | .14485E+01 | .31685E+00 | .31543E+00 | .10435E+01 | .29411E-01 |
| 100 | .40000E+02 | .47532E-02 | .21319E+00 | .11946E+01 | .30714E+00 | .30605E+00 | .10115E+01 | .25000E-01 |
| 101 | .45000E+02 | .37649E-02 | .17216E+00 | .10645E+01 | .30701E+00 | .30681E+00 | .10111E+01 | .22222E-01 |
| 102 | .50000E+02 | .30549E-02 | .13781E+00 | .95974E+00 | .30862E+00 | .30735E+00 | .10163E+01 | .20000E-01 |
| 103 | .55000E+02 | .25338E-02 | .11468E+00 | .87565E+00 | .31010E+00 | .30846E+00 | .10212E+01 | .18181E-01 |
| 104 | .60000E+02 | .21427E-02 | .92963E-01 | .80779E+00 | .31098E+00 | .31042E+00 | .10241E+01 | .16666E-01 |
| 105 | .65000E+02 | .18196E-02 | .72865E-01 | .74314E+00 | .30974E+00 | .30938E+00 | .10201E+01 | .15384E-01 |
| 106 | .70000E+02 | .15568E-02 | .56374E-01 | .68472E+00 | .30718E+00 | .30699E+00 | .10116E+01 | .14285E-01 |
| 107 | .75000E+02 | .13447E-02 | .47777E-01 | .63370E+00 | .30457E+00 | .30441E+00 | .10030E+01 | .13333E-01 |
| 108 | .80000E+02 | .11733E-02 | .42192E-01 | .58977E+00 | .30233E+00 | .30219E+00 | .99566E+00 | .12500E-01 |
| 109 | .85000E+02 | .10363E-02 | .39482E-01 | .55349E+00 | .30166E+00 | .30133E+00 | .99346E+00 | .11764E-01 |
| 110 | .90000E+02 | .92750E-03 | .37447E-01 | .52449E+00 | .30291E+00 | .30233E+00 | .99758E+00 | .11111E-01 |
| 111 | .95000E+02 | .83577E-03 | .35088E-01 | .49887E+00 | .30421E+00 | .30354E+00 | .10018E+01 | .10526E-01 |
| 112 | .10000E+03 | .75642E-03 | .32172E-01 | .47527E+00 | .30507E+00 | .30441E+00 | .10047E+01 | .10000E-01 |

# APPENDIX B

# CADAM OUTPUT TABLES AND STABILITY DRAWINGS

Table B.1 CADAM Input and Geometry Report.

## CADAM Input and Geometry report

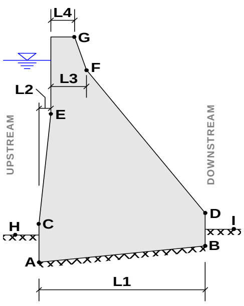
| | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26.08.2004 09:49 |
| Dam location: | Eskişehir | Units: | Metric |

**Geometry**

| | |
|---|---|
| L1= | 39.400 m |
| L2= | 0.000 m |
| L3= | 4.500 m |
| L4= | 4.500 m |
| Elev. A= | 0.000 m |
| Elev. B= | 0.000 m |
| Elev. C= | 0.000 m |
| Elev. D= | 0.000 m |
| Elev. E= | 0.000 m |
| Elev. F= | 41.000 m |
| Elev. G= | 49.700 m |
| Elev. H= | 0.000 m |
| Elev. I = | 0.000 m |

**Concrete Volumetric Mass**

| | |
|---|---|
| ρ= | 2400 kg/m³ |




**Lift Joint Material Properties**

| Material name | Concrete strength f'c (kPa) | Concrete strength ft (kPa) | Peak friction Cohesion (kPa) | Peak friction Angle (deg) | Residual friction Cohesion (kPa) | Residual friction Angle (deg) | Minimal compressive stress for cohesion (kPa) |
|---|---|---|---|---|---|---|---|
| Base joint | 30000 | 3000 | 931 | 55 | 0 | 45 | 0 |
| Concrete | 30000 | 3000 | 931 | 55 | 0 | 45 | 0 |

**Lift Joint(s)**

| Joint id | material name | Upstream end Elevation (m) | Upstream end Position x (m) | Downstream end Elevation (m) | Downstream end Position x (m) | Length (m) | Inertia (m^4) |
|---|---|---|---|---|---|---|---|
| 1 | Concrete | 49.500 | 0.000 | 49.500 | 4.500 | 4.500 | 7.59375 |
| 2 | Concrete | 45.000 | 0.000 | 45.000 | 4.500 | 4.500 | 7.59375 |
| 3 | Concrete | 40.500 | 0.000 | 40.500 | 4.926 | 4.926 | 9.958610722 |
| 4 | Concrete | 36.000 | 0.000 | 36.000 | 8.756 | 8.756 | 55.9436154 |
| 5 | Concrete | 31.500 | 0.000 | 31.500 | 12.587 | 12.587 | 166.1661398 |
| 6 | Concrete | 27.000 | 0.000 | 27.000 | 16.417 | 16.417 | 368.7278622 |
| 7 | Concrete | 22.500 | 0.000 | 22.500 | 20.248 | 20.248 | 691.7304608 |
| 8 | Concrete | 18.000 | 0.000 | 18.000 | 24.078 | 24.078 | 1163.275614 |
| 9 | Concrete | 13.500 | 0.000 | 13.500 | 27.909 | 27.909 | 1811.464999 |
| 10 | Concrete | 9.000 | 0.000 | 9.000 | 31.739 | 31.739 | 2664.400296 |
| 11 | Concrete | 4.500 | 0.000 | 4.500 | 35.570 | 35.570 | 3750.183181 |
| Base | Base joint | 0.000 | 0.000 | 0.000 | 39.400 | 39.400 | 5096.915333 |

**Pre-Cracked Lift Joint(s)**

| Joint id | material name | Usptream end Crack length (m) | Usptream end Crack length (%) | Downstream end Crack length (m) | Downstream end Crack length (%) |
|---|---|---|---|---|---|
| 1 | Concrete | - | - | - | - |
| 2 | Concrete | - | - | - | - |
| 3 | Concrete | - | - | - | - |
| 4 | Concrete | - | - | - | - |
| 5 | Concrete | - | - | - | - |
| 6 | Concrete | - | - | - | - |
| 7 | Concrete | - | - | - | - |
| 8 | Concrete | - | - | - | - |
| 9 | Concrete | - | - | - | - |
| 10 | Concrete | - | - | - | - |
| 11 | Concrete | - | - | - | - |
| Base | Base joint | - | - | - | - |

Table B.1 CADAM Input and Geometry Report (continued – 1).

## *CADAM Input and Geometry report*

| | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26.08.2004 09:49 |
| Dam location: | Eskişehir | Units: | Metric |

**Water Volumetric Mass**

| | |
|---|---|
| ρ= | 9.810 kg/m³ |

**Ice cover**

| | |
|---|---|
| Load= | 100 kN |
| Thickness= | 0.520 m |
| Elevation= | 45.340 m |

**Silts**

| | |
|---|---|
| Elevation= | 3.000 m |
| γ= | 11 kN/m³ |
| φ= | 31 deg |
| Assumption= | active |

**Reservoirs**

| | Upstream side | Downstream side |
|---|---|---|
| **Normal operating level:** | 45.600 m | 6.000 m |
| **Flood level:** | 48.200 m | 6.000 m |
| Crest overtopping pressure | 100.00 % | 50.00 % |

**Drainage system**

| | |
|---|---|
| Gallery position from heel of dam= | 3.540 m |
| Gallery elevation= | 16.850 m |
| Drain Efficiency= | N/A |
| Highest drained elevation= | 36.500 m |
| Modelisation: | CDSA 1995 - Alternative 1<br>K1 = 0.600 |

**Uplift pressures:** Uplift pressures are considered as an external load (linearisation of effective stresses)

**Pseudo-static (seismic coefficient)**

| | | | |
|---|---|---|---|
| Horizontal Peak Ground Acceleration (HPGA)= | 0.3000 g | Earthquake return period= | 1000 years |
| Vertical Peak Ground Acceleration (VPGA)= | 0.2000 g | Earthquake accelerogram period (te)= | 1 sec |
| Horizontal Sustained Acceleration (HSA)= | 0.2000 g | Depth where pressures remain constant= | Generalized |
| Vertical Sustained Acceleration (VSA)= | 0.1330 g | Westergaard correction for Inclined surface= | Corns et al. |

**Pseudo-dynamic (Chopra's method)**

| | |
|---|---|
| Earthquake return period= | 1000 years |
| Horizontal Peak Ground Acceleration (HPGA)= | 0.3000 g |
| Vertical Peak Ground Acceleration (VPGA)= | 0.2000 g |
| Horizontal Peak Spectral Acceleration (HPSA)= | 0.4650 g (Set 1)<br>1.1250 g (Set 2) |
| Horizontal Sustained Acceleration (HSA)= | 0.2000 g |
| Vertical Sustained Acceleration (VSA)= | 0.1330 g |
| Vertical Sustained Spectral Acceleration (VSSA)= | 0.3100 g (Set 1)<br>0.7500 g (Set 2) |
| Modal combination: | SRSS combination |

| **Dam only** | |
|---|---|
| Dam divisions for analysis= | 201 divisions |
| Dam damping on rigid foundation without reservoir= | 0.05 of critical |
| Concrete Young's modulus (dynamic)= | 27400 MPa |
| **Reservoir only** | |
| Wave reflection coefficient= | 0.5 |
| Velocity of pressure waves in water= | 1440 m/sec |
| **Foundation only** | |
| Foundation constant hysteretic damping= | 0.10 TL |
| Foundation Young's modulus (dynamic)= | 27400 MPa |
| **Dam-reservoir-foundation system** | |
| Period of vibration= | 0.16296503 sec |
| Damping= | 0.13213394 of critical |

**Cracking options**

| | |
|---|---|
| cracking considered for all combinations: | **Yes** |
| Numerical options | |
| Convergence method: | Bi-section |
| Accuracy= | Medium (1E-6) |

**Tensile strength**

| | Usual | Flood | Seismic | Post-seismic |
|---|---|---|---|---|
| Crack initiation= | ft / 3.000 | ft / 3.000 | ft / 3.000 | ft / 3.000 |
| Crack propagation= | ft /10.000 | ft /10.000 | ft /10.000 | ft /10.000 |
| Seismic magnification= | | | 1.500 | |

**Uplift pressures**

| | |
|---|---|
| Static analysis: | Full uplift pressures applied to the crack section |
| Dynamic analysis: | Uplift pressures remain unchanged |
| Post-seismic analysis: | Full uplift pressures applied to the crack section |
| D/S closed crack: | Restore uncracked uplift condition |
| Drain effectiveness: | No drain effectiveness under any cracking condition |

Table B.2 CADAM Loads for Data Set 1.

## CADAM Loads for Data Set 1

| | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26 Ağustos 04 |
| Dam location: | Eskişehir | Units: | Metric |

### STATIC LOADS (1/3)

| Joint | | Self-Weight | | | | Normal Operating level | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dam | | Concentrated masses | | Upstream reservoir | | | | Downstream reservoir | | | | Crest Overtopping | | Uplift | | Ice | |
| | | Vertical load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Vertical load | | Normal load | | Horizontal load | |
| ID | Upstream elevation (m) | D (kN) | position x (m) | Mv (kN) | position x (m) | Hnu (kN) | elevation (m) | Vnu (kN) | position x (m) | Hnd (kN) | elevation (m) | Vnd (kN) | position x (m) | Vnc (kN) | position x (m) | Un (kN) | position l (m) | Un (kN) | position l (m) |
| 1 | 49.500 | -21.2 | 2.250 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | -498.0 | 2.250 | | | 1.8 | 45.200 | | | | | | | | | 13.2 | 1.500 | 100.0 | 45.340 |
| 3 | 40.500 | -977.2 | 2.256 | | | 127.6 | 42.200 | | | | | | | | | 123.2 | 1.642 | 100.0 | 45.340 |
| 4 | 36.000 | -1702.0 | 2.790 | | | 452.0 | 39.200 | | | | | | | | | 412.3 | 2.919 | 100.0 | 45.340 |
| 5 | 31.500 | -2832.6 | 3.829 | | | 975.2 | 36.200 | | | | | | | | | 870.5 | 4.196 | 100.0 | 45.340 |
| 6 | 27.000 | -4369.0 | 5.047 | | | 1696.9 | 33.200 | | | | | | | | | 1497.8 | 5.472 | 100.0 | 45.340 |
| 7 | 22.500 | -6311.3 | 6.325 | | | 2617.4 | 30.200 | | | | | | | | | 2294.2 | 6.749 | 100.0 | 45.340 |
| 8 | 18.000 | -8659.4 | 7.622 | | | 3736.4 | 27.200 | | | | | | | | | 3259.6 | 8.026 | 100.0 | 45.340 |
| 9 | 13.500 | -11413.4 | 8.925 | | | 5054.2 | 24.200 | | | | | | | | | 2636.5 | 9.303 | 100.0 | 45.340 |
| 10 | 9.000 | -14573.1 | 10.227 | | | 6570.5 | 21.200 | | | | | | | | | 3418.7 | 10.580 | 100.0 | 45.340 |
| 11 | 4.500 | -18138.7 | 11.528 | | | 8285.6 | 18.200 | | | -11.0 | 5.000 | -9.4 | 35.144 | | | 4564.1 | 12.536 | 100.0 | 45.340 |
| 12 | Base | -22110.2 | 12.827 | | | 8520.0 | 13.892 | | | -176.6 | 2.000 | -150.3 | 37.698 | | | 6447.1 | 15.495 | 57.9 | 41.417 |

### STATIC LOADS (2/3)

| Joint | | Silt | | | | Flood level | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Upstream reservoir | | | | Downstream reservoir | | | | Crest Overtopping | | Uplift | |
| | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Vertical load | | Normal load | |
| ID | Upstream elevation (m) | Sh (kN) | position x (m) | Sv (kN) | position x (m) | Hfu (kN) | elevation (m) | Vfu (kN) | position x (m) | Hfd (kN) | elevation (m) | Vfd (kN) | position x (m) | Vfc (kN) | position x (m) | Uf (kN) | position l (m) |
| 1 | 49.500 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | | | | | 50.2 | 46.067 | | | | | | | | | 70.6 | 1.500 |
| 3 | 40.500 | | | | | 290.8 | 43.067 | | | | | | | | | 186.0 | 1.642 |
| 4 | 36.000 | | | | | 730.1 | 40.067 | | | | | | | | | 524.0 | 2.919 |
| 5 | 31.500 | | | | | 1368.0 | 37.067 | | | | | | | | | 1031.0 | 4.196 |
| 6 | 27.000 | | | | | 2204.5 | 34.067 | | | | | | | | | 1707.1 | 5.472 |
| 7 | 22.500 | | | | | 3239.7 | 31.067 | | | | | | | | | 2552.4 | 6.749 |
| 8 | 18.000 | | | | | 4473.6 | 28.067 | | | | | | | | | 3566.7 | 8.026 |
| 9 | 13.500 | | | | | 5906.1 | 25.067 | | | | | | | | | 2850.1 | 9.303 |
| 10 | 9.000 | | | | | 7537.2 | 22.067 | | | | | | | | | 3661.6 | 10.580 |
| 11 | 4.500 | | | | | 9367.0 | 19.067 | | | -11.0 | 5.000 | -9.4 | 35.144 | | | 4836.3 | 12.498 |
| 12 | Base | 15.8 | 1.000 | | | 8520.0 | 13.892 | | | -58643.0 | 23.139 | -42669.2 | 22.907 | | | 6748.5 | 15.390 |

Table B.2 CADAM Loads for Data Set 1 (continued – 1).

| ***CADAM Loads for Data Set 1*** | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

**STATIC LOADS (3/3)**

| Joint | | Post-tensioning | | | | | | Applied forces | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Crest | | Downstream face | | | | | | | |
| | | Vertical load | | Vertical load | | Horizontal load | | Horizontal load | | Vertical load | |
| ID | Upstream elevation (m) | Pc (kN) | position x (m) | Pdv (kN) | position x (m) | Pdh (kN) | elevation (m) | Fh (kN) | elevation (m) | Fv (kN) | position x (m) |
| 1 | 49.500 | | | | | | | | | | |
| 2 | 45.000 | | | | | | | | | | |
| 3 | 40.500 | | | | | | | | | | |
| 4 | 36.000 | | | | | | | | | | |
| 5 | 31.500 | | | | | | | | | | |
| 6 | 27.000 | | | | | | | | | | |
| 7 | 22.500 | | | | | | | | | | |
| 8 | 18.000 | | | | | | | | | | |
| 9 | 13.500 | | | | | | | | | | |
| 10 | 9.000 | | | | | | | | | | |
| 11 | 4.500 | | | | | | | | | | |
| 12 | Base | | | | | | | | | | |

**PSEUDO-STATIC LOADS (SEISMIC COEFFICIENT)-STRESS ANALYSIS**

| Joint | | Inertia loads | | | | | | | | Reservoirs (operating level) | | | | | | | | Silt | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dam | | | | Concentrated masses | | | | Upstream | | | | Downstream | | | | | | | |
| | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | |
| ID | Upstream elevation (m) | Qh (kN) | elevation (m) | Qv (kN) | position x (m) | Mdh (kN) | elevation (m) | Mdv (kN) | position x (m) | Hdu (kN) | elevation (m) | Vdu (kN) | position x (m) | Hdd (kN) | elevation (m) | Vdd (kN) | position x (m) | Sdh (kN) | elevation (m) | Sdv (kN) | position x (m) |
| 1 | 49.500 | -6.4 | 49.600 | -4.2 | 2.250 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | -149.4 | 47.350 | -99.6 | 2.250 | | | | | -5.0 | 45.240 | | | | | | | | | | |
| 3 | 40.500 | -293.2 | 45.089 | -195.4 | 2.256 | | | | | -124.3 | 42.540 | | | | | | | | | | |
| 4 | 36.000 | -510.6 | 42.087 | -340.4 | 2.790 | | | | | -321.0 | 39.840 | | | | | | | | | | |
| 5 | 31.500 | -849.8 | 38.706 | -566.5 | 3.829 | | | | | -571.3 | 37.140 | | | | | | | | | | |
| 6 | 27.000 | -1310.7 | 35.346 | -873.8 | 5.047 | | | | | -865.6 | 34.440 | | | | | | | | | | |
| 7 | 22.500 | -1893.4 | 32.061 | -1262.3 | 6.325 | | | | | -1198.1 | 31.740 | | | | | | | | | | |
| 8 | 18.000 | -2597.8 | 28.841 | -1731.9 | 7.622 | | | | | -1564.7 | 29.040 | | | | | | | | | | |
| 9 | 13.500 | -3424.0 | 25.669 | -2282.7 | 8.925 | | | | | -1962.6 | 26.340 | | | | | | | | | | |
| 10 | 9.000 | -4371.9 | 22.532 | -2914.6 | 10.227 | | | | | -2389.4 | 23.640 | | | | | | | | | | |
| 11 | 4.500 | -5441.6 | 19.421 | -3627.7 | 11.528 | | | | | -2843.4 | 20.940 | | | -1.8 | 5.100 | -3.4 | 35.106 | | | | |
| 12 | Base | -6633.1 | 16.330 | -4422.0 | 12.827 | | | | | -3322.9 | 18.240 | | | -14.4 | 2.400 | -42.3 | 37.599 | -16.1 | 1.200 | | |

Table B.2  CADAM Loads for Data Set 1 (continued – 2).

| CADAM Loads for Data Set 1 | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26 Ağustos 04 |
| Dam location: | Eskişehir | Units: | Metric |

| PSEUDO-STATIC LOADS (SEISMIC COEFFICIENT)-STABILITY ANALYSIS | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | Inertia loads | | | | | | | | Reservoirs (operating level) | | | | | | | | Silt | |
| | | Dam | | | | Concentrated masses | | | | Upstream | | | | Downstream | | | | | |
| ID | Upstream | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | Vertical load |
| | elevation | Qh' | elevation | Qv' | position x | Mdh' | elevation | Mdv' | position x | Hdu' | elevation | Vdu' | position x | Hdd' | elevation | Vdd' | position x | Sdh' | elevation | Sdv' | position x |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | -4.2 | 49.600 | -2.8 | 2.250 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | -99.6 | 47.350 | -66.2 | 2.250 | | | | | -3.3 | 45.240 | | | | | | | | | | |
| 3 | 40.500 | -195.4 | 45.089 | -130.0 | 2.256 | | | | | -82.9 | 42.540 | | | | | | | | | | |
| 4 | 36.000 | -340.4 | 42.087 | -226.4 | 2.790 | | | | | -214.0 | 39.840 | | | | | | | | | | |
| 5 | 31.500 | -566.5 | 38.706 | -376.7 | 3.829 | | | | | -380.9 | 37.140 | | | | | | | | | | |
| 6 | 27.000 | -873.8 | 35.346 | -581.1 | 5.047 | | | | | -577.1 | 34.440 | | | | | | | | | | |
| 7 | 22.500 | -1262.3 | 32.061 | -839.4 | 6.325 | | | | | -798.7 | 31.740 | | | | | | | | | | |
| 8 | 18.000 | -1731.9 | 28.841 | -1151.7 | 7.622 | | | | | -1043.1 | 29.040 | | | | | | | | | | |
| 9 | 13.500 | -2282.7 | 25.669 | -1518.0 | 8.925 | | | | | -1308.4 | 26.340 | | | | | | | | | | |
| 10 | 9.000 | -2914.6 | 22.532 | -1938.2 | 10.227 | | | | | -1593.0 | 23.640 | | | | | | | | | | |
| 11 | 4.500 | -3627.7 | 19.421 | -2412.5 | 11.528 | | | | | -1895.6 | 20.940 | | | -1.2 | 5.100 | -2.3 | 35.106 | | | | |
| 12 | Base | -4422.0 | 16.330 | -2940.7 | 12.827 | | | | | -2215.3 | 18.240 | | | -9.7 | 2.400 | -28.2 | 37.598 | -10.8 | 1.200 | | |

| PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STRESS ANALYSIS (1/2) | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | First mode | | | | | | | | Higher modes | | | | | | | | Modal combination | | | |
| | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | SRSS | | Summation | |
| ID | Upstream | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | |
| | elevation | Eq1 | elevation | Hd1 | elevation | Md1 | elevation | Em1 | elevation | Eqs | elevation | Hds | elevation | Mds | elevation | Ems | elevation | Emc | elevation | Emc | elevation |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | -31.7 | 49.600 | | | | | -31.7 | 49.600 | 10.9 | 49.600 | | | | | 10.9 | 49.600 | -33.5 | 49.600 | | |
| 2 | 45.000 | -653.5 | 47.465 | -4.2 | 45.202 | | | -657.7 | 47.451 | 206.3 | 47.548 | 23.9 | 45.377 | | | 230.2 | 47.323 | -696.8 | 47.437 | | |
| 3 | 40.500 | -1117.5 | 45.550 | -215.1 | 42.375 | | | -1332.6 | 45.037 | 315.0 | 45.979 | 70.4 | 44.489 | | | 385.4 | 45.707 | -1387.2 | 45.089 | | |
| 4 | 36.000 | -1637.0 | 43.199 | -568.4 | 39.765 | | | -2205.4 | 42.314 | 380.2 | 44.692 | 32.9 | 52.035 | | | 413.1 | 45.276 | -2243.8 | 42.417 | | |
| 5 | 31.500 | -2254.2 | 40.603 | -965.1 | 37.283 | | | -3219.3 | 39.608 | 377.0 | 44.881 | -44.7 | 19.924 | | | 332.3 | 48.234 | -3236.4 | 39.709 | | |
| 6 | 27.000 | -2882.2 | 38.133 | -1375.9 | 34.884 | | | -4258.1 | 37.083 | 257.8 | 52.300 | -170.2 | 26.687 | | | 87.6 | 102.040 | -4259.0 | 37.134 | | |
| 7 | 22.500 | -3459.6 | 35.907 | -1782.8 | 32.573 | | | -5242.4 | 34.773 | -10.6 | -651.854 | -351.7 | 25.625 | | | -362.3 | 5.825 | -5254.9 | 34.693 | | |
| 8 | 18.000 | -3949.0 | 33.977 | -2170.9 | 30.374 | | | -6119.9 | 32.699 | -448.7 | 4.242 | -592.9 | 23.400 | | | -1041.6 | 15.147 | -6207.9 | 32.335 | | |
| 9 | 13.500 | -4329.2 | 32.386 | -2534.2 | 28.281 | | | -6863.3 | 30.870 | -1068.0 | 10.850 | -892.6 | 20.808 | | | -1960.6 | 15.384 | -7137.9 | 29.982 | | |
| 10 | 9.000 | -4595.1 | 31.172 | -2874.8 | 26.266 | | | -7469.9 | 29.284 | -1871.2 | 10.985 | -1244.1 | 18.094 | | | -3115.3 | 13.824 | -8093.5 | 27.546 | | |
| 11 | 4.500 | -4755.3 | 30.357 | -3194.6 | 24.315 | | | -7949.9 | 27.929 | -2853.7 | 9.504 | -1636.8 | 15.364 | | | -4490.5 | 11.640 | -9130.4 | 24.983 | | |
| 12 | Base | -4807.8 | 30.059 | -3491.5 | 22.441 | | | -8299.3 | 26.854 | -4016.5 | 7.387 | -2066.6 | 12.630 | | | -6083.0 | 9.168 | -10289.9 | 22.327 | | |

Table B.2 CADAM Loads for Data Set 1 (continued – 3).

| CADAM Loads for Data Set 1 | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

| PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STRESS ANALYSIS (2/2) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | Vertical loads | | | | | | | | | | horizontal loads | | | |
| | | Dam | | Reservoir (upstream) | | Reservoir (downstream) | | Concentrated masses | | Silt | | Reservoir (downstream) | | Silt | |
| ID | Upstream | Vertical load | | Vertical load | | Vertical load | | Vertical load | | Vertical load | | Horizontal load | | Horizontal load | |
| | elevation | Eqv | position x | Vdu | position x | Vdd | position x | Mdv | position x | Sdv | position x | Hdd | elevation | Sdh | elevation |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | -4.2 | 2.250 | | | | | | | | | | | | |
| 2 | 45.000 | -99.6 | 2.250 | | | | | | | | | | | | |
| 3 | 40.500 | -195.4 | 2.256 | | | | | | | | | | | | |
| 4 | 36.000 | -340.4 | 2.790 | | | | | | | | | | | | |
| 5 | 31.500 | -566.5 | 3.829 | | | | | | | | | | | | |
| 6 | 27.000 | -873.8 | 5.047 | | | | | | | | | | | | |
| 7 | 22.500 | -1262.3 | 6.325 | | | | | | | | | | | | |
| 8 | 18.000 | -1731.9 | 7.622 | | | | | | | | | | | | |
| 9 | 13.500 | -2282.7 | 8.925 | | | | | | | | | | | | |
| 10 | 9.000 | -2914.6 | 10.227 | | | | | | | | | | | | |
| 11 | 4.500 | -3627.7 | 11.528 | | | -3.4 | 35.106 | | | | | -1.8 | 5.100 | | |
| 12 | Base | -4422.0 | 12.827 | | | -42.3 | 37.599 | | | | | -14.4 | 2.400 | -16.1 | 1.200 |

| PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STABILITY ANALYSIS (1/2) | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | First mode | | | | | | | | Higher modes | | | | | | | | Modal combination | | | |
| | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | SRSS | | Summation | |
| ID | Upstream | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | |
| | elevation | Eq1' | elevation | Hd1' | elevation | Md1' | elevation | Em1' | elevation | Eqs' | elevation | Hds' | elevation | Mds' | elevation | Ems' | elevation | Emc' | elevation | Emc' | elevation |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | -21.1 | 49.600 | | | | | -21.1 | 49.600 | 7.2 | 49.600 | | | | | 7.2 | 49.600 | -22.3 | 49.600 | | |
| 2 | 45.000 | -435.7 | 47.465 | -2.8 | 45.202 | | | -438.5 | 47.451 | 137.5 | 47.548 | 15.9 | 45.377 | | | 153.5 | 47.323 | -464.6 | 47.437 | | |
| 3 | 40.500 | -745.0 | 45.550 | -143.4 | 42.375 | | | -888.4 | 45.037 | 210.0 | 45.979 | 46.9 | 44.489 | | | 256.9 | 45.707 | -924.8 | 45.089 | | |
| 4 | 36.000 | -1091.3 | 43.199 | -379.0 | 39.765 | | | -1470.3 | 42.314 | 253.5 | 44.692 | 21.9 | 52.035 | | | 275.4 | 45.276 | -1495.8 | 42.417 | | |
| 5 | 31.500 | -1502.8 | 40.603 | -643.4 | 37.283 | | | -2146.2 | 39.608 | 251.3 | 44.881 | -29.8 | 19.924 | | | 221.6 | 48.234 | -2157.6 | 39.709 | | |
| 6 | 27.000 | -1921.5 | 38.133 | -917.2 | 34.884 | | | -2838.7 | 37.083 | 171.9 | 52.300 | -113.5 | 26.687 | | | 58.4 | 102.040 | -2839.3 | 37.134 | | |
| 7 | 22.500 | -2306.4 | 35.907 | -1188.5 | 32.573 | | | -3494.9 | 34.773 | -7.1 | -651.854 | -234.5 | 25.625 | | | -241.5 | 5.825 | -3503.3 | 34.693 | | |
| 8 | 18.000 | -2632.6 | 33.977 | -1447.3 | 30.374 | | | -4079.9 | 32.699 | -299.1 | 4.242 | -395.3 | 23.400 | | | -694.4 | 15.147 | -4138.6 | 32.335 | | |
| 9 | 13.500 | -2886.1 | 32.386 | -1689.4 | 28.281 | | | -4575.5 | 30.870 | -712.0 | 10.850 | -595.1 | 20.808 | | | -1307.1 | 15.384 | -4758.6 | 29.982 | | |
| 10 | 9.000 | -3063.4 | 31.172 | -1916.5 | 26.266 | | | -4980.0 | 29.284 | -1247.4 | 10.985 | -829.4 | 18.094 | | | -2076.8 | 13.824 | -5395.7 | 27.546 | | |
| 11 | 4.500 | -3170.2 | 30.357 | -2129.7 | 24.315 | | | -5299.9 | 27.929 | -1902.4 | 9.504 | -1091.2 | 15.364 | | | -2993.7 | 11.640 | -6087.0 | 24.983 | | |
| 12 | Base | -3205.2 | 30.059 | -2327.7 | 22.441 | | | -5532.9 | 26.854 | -2677.6 | 7.387 | -1377.7 | 12.630 | | | -4055.4 | 9.168 | -6859.9 | 22.327 | | |

Table B.2  CADAM Loads for Data Set 1 (continued – 4).

## CADAM Loads for Data Set 1

| | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26 Ağustos 04 |
| Dam location: | Eskişehir | Units: | Metric |

**PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STABILITY ANALYSIS (2/2)**

| Joint | | Vertical loads | | | | | | | | | | horizontal loads | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dam | | Reservoir (upstream) | | Reservoir (downstream) | | Concentrated masses | | Silt | | Reservoir (downstream) | | Silt | |
| | | Vertical load | | Vertical load | | Vertical load | | Vertical load | | Vertical load | | Horizontal load | | Horizontal load | |
| ID | Upstream elevation (m) | Eqv' (kN) | position x (m) | Vdu' (kN) | position x (m) | Vdd' (kN) | position x (m) | Mdv' (kN) | position x (m) | Sdv' (kN) | position x (m) | Hdd' (kN) | elevation (m) | Sdh' (kN) | elevation (m) |
| 1 | 49.500 | -2.8 | 2.250 | | | | | | | | | | | | |
| 2 | 45.000 | -66.2 | 2.250 | | | | | | | | | | | | |
| 3 | 40.500 | -130.0 | 2.256 | | | | | | | | | | | | |
| 4 | 36.000 | -226.4 | 2.790 | | | | | | | | | | | | |
| 5 | 31.500 | -376.7 | 3.829 | | | | | | | | | | | | |
| 6 | 27.000 | -581.1 | 5.047 | | | | | | | | | | | | |
| 7 | 22.500 | -839.4 | 6.325 | | | | | | | | | | | | |
| 8 | 18.000 | -1151.7 | 7.622 | | | | | | | | | | | | |
| 9 | 13.500 | -1518.0 | 8.925 | | | | | | | | | | | | |
| 10 | 9.000 | -1938.2 | 10.227 | | | | | | | | | | | | |
| 11 | 4.500 | -2412.5 | 11.528 | | | -2.3 | 35.106 | | | | | -1.2 | 5.100 | | |
| 12 | Base | -2940.7 | 12.827 | | | -28.2 | 37.598 | | | | | -9.7 | 2.400 | -10.8 | 1.200 |

Table B.3 CADAM Results for Data Set 1.

## CADAM Results for Data Set 1

| | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

### LOAD COMBINATION FACTORS

| | Usual | Flood | Seismic #1 | Seismic #2 | Post-seismic |
|---|---|---|---|---|---|
| **Self-weight** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Hydrostatic (upstream)** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Hydrostatic (downstream)** | 0.600 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Uplift pressures** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Silts** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Ice** | 1.000 | | 1.000 | 1.000 | 1.000 |
| **Post-tensioning** | | | | | |
| **Applied forces** | | | | | |
| **Seismic (horizontal)** | | | -1.000 | -1.000 | |
| **Seismic (vertical)** | | | -1.000 | -1.000 | |

### USUAL COMBINATION (STRESS ANALYSIS)

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -4.709 | -4.709 | 0.000 | -1260.000 | | | | |
| 2 | 45.000 | | | | | -94.592 | -120.836 | 0.000 | -1260.000 | 0.000 | 33.922 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | -24.982 | -321.781 | 0.000 | -1260.000 | 0.000 | -33.618 | 24.848 | 273.907 |
| 4 | 36.000 | | | | | -125.429 | -169.153 | 0.000 | -1260.000 | 0.000 | 143.987 | 100.000 | 143.987 |
| 5 | 31.500 | | | | | -125.105 | -186.673 | 0.000 | -1260.000 | 0.000 | 158.900 | 100.000 | 158.900 |
| 6 | 27.000 | | | | | -116.103 | -233.687 | 0.000 | -1260.000 | 0.000 | 198.919 | 100.000 | 198.919 |
| 7 | 22.500 | | | | | -107.600 | -289.205 | 0.000 | -1260.000 | 0.000 | 246.177 | 100.000 | 246.177 |
| 8 | 18.000 | | | | | -100.657 | -347.867 | 0.000 | -1260.000 | 0.000 | 296.111 | 100.000 | 296.111 |
| 9 | 13.500 | | | | | -221.099 | -407.873 | 0.000 | -1260.000 | 0.000 | 347.189 | 100.000 | 347.189 |
| 10 | 9.000 | | | | | -234.376 | -468.506 | 0.000 | -1260.000 | 0.000 | 398.802 | 100.000 | 398.802 |
| 11 | 4.500 | | | | | -248.252 | -515.339 | 0.000 | -1260.000 | 0.000 | 438.667 | 100.000 | 438.667 |
| 12 | Base | | | | | -410.144 | -389.516 | 0.000 | -1260.000 | 0.000 | 331.564 | 100.000 | 331.564 |

### USUAL COMBINATION (STABILITY ANALYSIS)

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final | Passive |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | Force (kN) | wedge resistance (kN) |
| 1 | 49.500 | > 100 | > 100 | > 100 | > 100 | > 100 | -21.2 | 0.0 | 0.0 | 50.000 | | |
| 2 | 45.000 | 47.970 | 4.763 | 58.129 | 15.123 | 37.600 | -484.7 | 101.8 | 44.3 | 52.030 | 13.2 | 0.000 |
| 3 | 40.500 | 25.509 | 3.753 | 14.363 | 2.360 | 7.931 | -854.0 | 227.6 | 600.1 | 64.265 | 123.2 | 0.000 |
| 4 | 36.000 | 18.103 | 2.336 | 5.924 | 2.121 | 4.128 | -1289.7 | 552.0 | 279.4 | 52.474 | 412.3 | 0.000 |
| 5 | 31.500 | 13.505 | 1.825 | 4.604 | 1.869 | 3.254 | -1962.1 | 1075.2 | 812.8 | 53.291 | 870.5 | 0.000 |
| 6 | 27.000 | 10.788 | 1.598 | 4.198 | 1.728 | 2.917 | -2871.3 | 1796.9 | 2640.9 | 55.603 | 1497.8 | 0.000 |
| 7 | 22.500 | 9.048 | 1.478 | 4.027 | 1.645 | 2.751 | -4017.2 | 2717.4 | 6204.3 | 57.628 | 2294.2 | 0.000 |
| 8 | 18.000 | 7.853 | 1.408 | 3.941 | 1.593 | 2.657 | -5399.8 | 3836.4 | 11943.4 | 59.186 | 3259.6 | 0.000 |
| 9 | 13.500 | 7.473 | 1.703 | 6.488 | 2.038 | 4.329 | -8776.8 | 5154.2 | 12123.0 | 54.949 | 2636.5 | 0.000 |
| 10 | 9.000 | 6.818 | 1.672 | 6.438 | 2.008 | 4.263 | -11154.4 | 6670.5 | 19654.5 | 55.552 | 3418.7 | 0.000 |
| 11 | 4.500 | 6.267 | 1.621 | 5.713 | 1.958 | 3.975 | -13580.3 | 8379.0 | 28159.7 | 55.830 | 4564.1 | 0.000 |
| 12 | Base | 6.972 | 1.856 | 4.073 | 2.139 | 3.443 | -15753.3 | 8487.8 | -2668.5 | 49.570 | 6447.1 | 0.000 |
| | **Required:** | 3.000 | 1.500 | 1.200 | 1.200 | 1.200 | | | | | | |

Table B.3 CADAM Results for Data Set 1 (continued – 1).

| CADAM Results for Data Set 1 | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

**FLOOD COMBINATION (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -4.709 | -4.709 | 189.000 | -1890.000 | | | | |
| 2 | 45.000 | | | | | -63.391 | -126.531 | 189.000 | -1890.000 | 0.000 | 16.742 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 11.788 | -333.045 | 189.000 | -1890.000 | 0.000 | -22.801 | 21.435 | 283.495 |
| 4 | 36.000 | | | | | -53.878 | -215.198 | 189.000 | -1890.000 | 0.000 | 183.181 | 100.000 | 183.181 |
| 5 | 31.500 | | | | | -37.196 | -249.077 | 189.000 | -1890.000 | 0.000 | 212.019 | 100.000 | 212.019 |
| 6 | 27.000 | | | | | -18.837 | -305.448 | 189.000 | -1890.000 | 0.000 | 260.003 | 100.000 | 260.003 |
| 7 | 22.500 | | | | | -4.295 | -367.004 | 189.000 | -1890.000 | 0.000 | 312.401 | 100.000 | 312.401 |
| 8 | 18.000 | | | | | 6.863 | -429.880 | 189.000 | -1890.000 | 0.000 | 365.923 | 100.000 | 365.923 |
| 9 | 13.500 | | | | | -120.674 | -492.994 | 189.000 | -1890.000 | 0.000 | 419.646 | 100.000 | 419.646 |
| 10 | 9.000 | | | | | -131.567 | -556.012 | 189.000 | -1890.000 | 0.000 | 473.288 | 100.000 | 473.288 |
| 11 | 4.500 | | | | | -143.361 | -605.137 | 189.000 | -1890.000 | 0.000 | 515.105 | 100.000 | 515.105 |
| 12 | Base | | | | | -151.834 | -635.574 | 189.000 | -1890.000 | 0.000 | 541.013 | 100.000 | 541.013 |

**FLOOD COMBINATION (STABILITY ANALYSIS)**

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | > 100 | > 100 | > 100 | > 100 | > 100 | -21.2 | 0.0 | 0.0 | 50.000 | | |
| 2 | 45.000 | 95.561 | 8.508 | 11.081 | 4.220 | 7.050 | -427.3 | 50.2 | 106.5 | 55.541 | 70.6 | 0.000 |
| 3 | 40.500 | 19.115 | 2.721 | 9.662 | 1.922 | 5.253 | -791.2 | 290.8 | 697.2 | 67.890 | 186.0 | 0.000 |
| 4 | 36.000 | 13.471 | 1.614 | 5.046 | 1.685 | 3.248 | -1178.0 | 730.1 | 1030.7 | 59.992 | 524.0 | 0.000 |
| 5 | 31.500 | 10.447 | 1.317 | 4.268 | 1.525 | 2.747 | -1801.6 | 1368.0 | 2797.2 | 62.336 | 1031.0 | 0.000 |
| 6 | 27.000 | 8.658 | 1.207 | 4.028 | 1.450 | 2.559 | -2661.9 | 2204.5 | 6437.3 | 64.730 | 1707.1 | 0.000 |
| 7 | 22.500 | 7.476 | 1.160 | 3.928 | 1.413 | 2.473 | -3758.9 | 3239.7 | 12391.5 | 66.281 | 2552.4 | 0.000 |
| 8 | 18.000 | 6.558 | 1.138 | 3.879 | 1.393 | 2.428 | -5092.7 | 4473.6 | 21100.2 | 67.207 | 3566.7 | 0.000 |
| 9 | 13.500 | 6.470 | 1.450 | 6.418 | 1.786 | 4.005 | -8563.3 | 5906.1 | 24166.2 | 60.112 | 2850.1 | 0.000 |
| 10 | 9.000 | 5.988 | 1.448 | 6.390 | 1.782 | 3.980 | -10911.5 | 7537.2 | 35630.9 | 60.288 | 3661.6 | 0.000 |
| 11 | 4.500 | 5.571 | 1.423 | 5.722 | 1.758 | 3.753 | -13311.9 | 9356.0 | 48686.2 | 60.282 | 4836.3 | 0.000 |
| 12 | Base | 5.237 | 1.381 | 4.533 | 1.704 | 3.299 | -15511.9 | 11234.8 | 62578.3 | 60.239 | 6748.5 | 0.000 |
| | **Required:** | 2.000 | 1.300 | 1.100 | 1.100 | 1.100 | | | | | | |

**SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -3.579 | -3.955 | 342.000 | -3420.000 | 0.000 | 2.119 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | 31.913 | -203.078 | 342.000 | -3420.000 | 0.000 | 85.389 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 420.074 | -687.478 | 342.000 | -3420.000 | 0.000 | -38.140 | 19.831 | 585.195 |
| 4 | 36.000 | | | | | 295.442 | -512.272 | 342.000 | -3420.000 | 0.000 | 436.056 | 100.000 | 436.056 |
| 5 | 31.500 | | | | | 326.733 | -548.492 | 342.000 | -3420.000 | 0.000 | 466.887 | 100.000 | 466.887 |
| 6 | 27.000 | | | | | 385.510 | -628.850 | 342.000 | -3420.000 | 0.000 | 535.289 | 100.000 | 535.289 |
| 7 | 22.500 | | | | | 451.875 | -723.996 | 342.000 | -3420.000 | 0.000 | 616.280 | 100.000 | 616.280 |
| 8 | 18.000 | | | | | 520.669 | -825.336 | 342.000 | -3420.000 | 0.000 | 702.542 | 100.000 | 702.542 |
| 9 | 13.500 | | | | | 464.215 | -929.604 | 342.000 | -3420.000 | 0.000 | 791.297 | 100.000 | 791.297 |
| 10 | 9.000 | | | | | 516.124 | -1035.344 | 342.000 | -3420.000 | 0.000 | 881.305 | 100.000 | 881.305 |
| 11 | 4.500 | | | | | 568.133 | -1127.762 | 342.000 | -3420.000 | 0.000 | 959.973 | 100.000 | 959.973 |
| 12 | Base | | | | | 622.916 | -1199.010 | 342.000 | -3420.000 | 0.000 | 1020.621 | 100.000 | 1020.621 |

Table B.3 CADAM Results for Data Set 1 (continued – 2).

| CADAM Results for Data Set 1 | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

**SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STABILITY ANALYSIS)**

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final | Passive |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | Force (kN) | wedge resistance (kN) |
| 1 | 49.500 | > 100 | 2.667 | 5.067 | 4.687 | 5.000 | -17.0 | 6.4 | 0.6 | 50.833 | | |
| 2 | 45.000 | 16.281 | 1.503 | 6.178 | 1.723 | 4.413 | -385.1 | 256.2 | 396.5 | 72.882 | 13.2 | 0.000 |
| 3 | 40.500 | 5.871 | 1.021 | 7.003 | 0.809 | 3.067 | -658.6 | 645.0 | 2239.3 | 119.031 | 123.2 | 0.000 |
| 4 | 36.000 | 4.717 | 0.686 | 5.327 | 0.910 | 2.261 | -949.3 | 1383.6 | 5160.6 | 112.085 | 412.3 | 0.000 |
| 5 | 31.500 | 3.740 | 0.559 | 4.494 | 0.899 | 1.971 | -1395.6 | 2496.3 | 11554.6 | 115.779 | 870.5 | 0.000 |
| 6 | 27.000 | 3.103 | 0.503 | 4.108 | 0.886 | 1.842 | -1997.5 | 3973.3 | 22782.5 | 119.475 | 1497.8 | 0.000 |
| 7 | 22.500 | 2.675 | 0.474 | 3.900 | 0.877 | 1.775 | -2754.9 | 5808.8 | 40172.0 | 122.019 | 2294.2 | 0.000 |
| 8 | 18.000 | 2.373 | 0.459 | 3.774 | 0.872 | 1.735 | -3667.9 | 7999.0 | 65029.1 | 123.632 | 3259.6 | 0.000 |
| 9 | 13.500 | 2.524 | 0.616 | 5.033 | 1.001 | 2.320 | -6494.2 | 10540.8 | 90468.9 | 99.916 | 2636.5 | 0.000 |
| 10 | 9.000 | 2.344 | 0.613 | 4.956 | 1.002 | 2.301 | -8239.8 | 13431.9 | 130241.3 | 99.801 | 3418.7 | 0.000 |
| 11 | 4.500 | 2.175 | 0.597 | 4.588 | 0.996 | 2.214 | -9952.9 | 16661.3 | 178802.4 | 100.506 | 4564.1 | 0.000 |
| 12 | Base | 2.005 | 0.564 | 3.896 | 0.980 | 2.040 | -11349.0 | 20125.1 | 235690.5 | 102.709 | 6447.1 | 0.000 |
| | Required: | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

**SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -3.957 | -4.208 | 342.000 | -3420.000 | 0.000 | 1.413 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | -10.292 | -175.701 | 342.000 | -3420.000 | 0.000 | 68.233 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 271.639 | -565.629 | 342.000 | -3420.000 | 0.000 | -36.288 | 20.932 | 481.474 |
| 4 | 36.000 | | | | | 155.016 | -397.894 | 342.000 | -3420.000 | 0.000 | 338.695 | 100.000 | 338.695 |
| 5 | 31.500 | | | | | 175.957 | -427.873 | 342.000 | -3420.000 | 0.000 | 364.214 | 100.000 | 364.214 |
| 6 | 27.000 | | | | | 218.114 | -497.115 | 342.000 | -3420.000 | 0.000 | 423.154 | 100.000 | 423.154 |
| 7 | 22.500 | | | | | 265.162 | -579.052 | 342.000 | -3420.000 | 0.000 | 492.901 | 100.000 | 492.901 |
| 8 | 18.000 | | | | | 313.308 | -666.168 | 342.000 | -3420.000 | 0.000 | 567.055 | 100.000 | 567.055 |
| 9 | 13.500 | | | | | 235.493 | -755.683 | 342.000 | -3420.000 | 0.000 | 643.252 | 100.000 | 643.252 |
| 10 | 9.000 | | | | | 265.641 | -846.388 | 342.000 | -3420.000 | 0.000 | 720.462 | 100.000 | 720.462 |
| 11 | 4.500 | | | | | 295.720 | -923.747 | 342.000 | -3420.000 | 0.000 | 786.311 | 100.000 | 786.311 |
| 12 | Base | | | | | 328.875 | -980.884 | 342.000 | -3420.000 | 0.000 | 834.947 | 100.000 | 834.947 |

**SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STABILITY ANALYSIS)**

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final | Passive |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | Force (kN) | wedge resistance (kN) |
| 1 | 49.500 | > 100 | 4.335 | 7.586 | 7.048 | 7.519 | -18.4 | 4.2 | 0.4 | 50.513 | | |
| 2 | 45.000 | 23.386 | 2.044 | 8.228 | 2.447 | 6.266 | -418.5 | 204.7 | 279.1 | 64.822 | 13.2 | 0.000 |
| 3 | 40.500 | 8.168 | 1.431 | 8.015 | 1.036 | 3.860 | -724.0 | 505.9 | 1692.8 | 97.466 | 123.2 | 0.000 |
| 4 | 36.000 | 6.675 | 0.961 | 5.462 | 1.124 | 2.665 | -1063.3 | 1106.4 | 3532.6 | 87.942 | 412.3 | 0.000 |
| 5 | 31.500 | 5.225 | 0.784 | 4.523 | 1.088 | 2.271 | -1585.4 | 2022.6 | 7971.7 | 89.949 | 870.5 | 0.000 |
| 6 | 27.000 | 4.278 | 0.705 | 4.133 | 1.058 | 2.102 | -2290.2 | 3247.8 | 16064.1 | 92.726 | 1497.8 | 0.000 |
| 7 | 22.500 | 3.656 | 0.665 | 3.934 | 1.039 | 2.014 | -3177.8 | 4778.3 | 28841.5 | 94.825 | 2294.2 | 0.000 |
| 8 | 18.000 | 3.224 | 0.643 | 3.818 | 1.028 | 1.963 | -4248.1 | 6611.5 | 47321.1 | 96.264 | 3259.6 | 0.000 |
| 9 | 13.500 | 3.451 | 0.830 | 5.350 | 1.206 | 2.747 | -7258.9 | 8745.2 | 64334.5 | 81.757 | 2636.5 | 0.000 |
| 10 | 9.000 | 3.189 | 0.824 | 5.279 | 1.203 | 2.720 | -9216.2 | 11178.1 | 93351.7 | 81.914 | 3418.7 | 0.000 |
| 11 | 4.500 | 2.952 | 0.804 | 4.844 | 1.191 | 2.600 | -11169.3 | 13899.1 | 128571.4 | 82.362 | 4564.1 | 0.000 |
| 12 | Base | 2.728 | 0.765 | 4.039 | 1.166 | 2.364 | -12844.6 | 16796.2 | 169434.7 | 83.480 | 6447.1 | 0.000 |
| | Required: | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

Table B.3 CADAM Results for Data Set 1 (continued – 3).

| CADAM Results for Data Set 1 | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26 Ağustos 04 |
| Dam location: | Eskişehir | Units: | Metric |

**SEISMIC #2 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -2.773 | -4.761 | 342.000 | -3420.000 | 0.000 | 11.159 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | 430.627 | -601.792 | 342.000 | -3420.000 | 0.000 | 266.199 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 1599.029 | -1866.433 | 342.000 | -3420.000 | 0.000 | -130.860 | 21.621 | 1588.745 |
| 4 | 36.000 | | | | | 1082.614 | -1299.445 | 342.000 | -3420.000 | 0.000 | -15.663 | 10.568 | 1106.113 |
| 5 | 31.500 | | | | | 978.956 | -1200.714 | 342.000 | -3420.000 | 0.000 | 1022.071 | 100.000 | 1022.071 |
| 6 | 27.000 | | | | | 959.487 | -1202.826 | 342.000 | -3420.000 | 0.000 | 1023.869 | 100.000 | 1023.869 |
| 7 | 22.500 | | | | | 962.648 | -1234.769 | 342.000 | -3420.000 | 0.000 | 1051.060 | 100.000 | 1051.060 |
| 8 | 18.000 | | | | | 971.419 | -1276.087 | 342.000 | -3420.000 | 0.000 | 1086.230 | 100.000 | 1086.230 |
| 9 | 13.500 | | | | | 855.410 | -1320.799 | 342.000 | -3420.000 | 0.000 | 1124.290 | 100.000 | 1124.290 |
| 10 | 9.000 | | | | | 849.442 | -1368.663 | 342.000 | -3420.000 | 0.000 | 1165.032 | 100.000 | 1165.032 |
| 11 | 4.500 | | | | | 848.286 | -1407.915 | 342.000 | -3420.000 | 0.000 | 1198.445 | 100.000 | 1198.445 |
| 12 | Base | | | | | 857.962 | -1434.055 | 342.000 | -3420.000 | 0.000 | 1220.696 | 100.000 | 1220.696 |

**SEISMIC #2 COMBINATION - PEAK ACCELERATIONS (STABILITY ANALYSIS)**

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | *> 100* | *0.506* | *5.352* | *3.699* | *5.000* | -17.0 | 33.5 | 3.4 | 54.396 | | |
| 2 | 45.000 | *3.747* | *0.482* | *11.694* | *0.561* | *4.413* | -385.1 | 798.6 | 1742.2 | 150.529 | 13.2 | 0.000 |
| 3 | 40.500 | *2.112* | *0.408* | *14.414* | *0.326* | *3.067* | -658.6 | 1614.8 | 7006.5 | 265.994 | 123.2 | 0.000 |
| 4 | 36.000 | *2.076* | *0.340* | *9.999* | *0.479* | *2.261* | -949.3 | 2795.8 | 15219.2 | 233.097 | 412.3 | 0.000 |
| 5 | 31.500 | *1.959* | *0.324* | *7.452* | *0.554* | *1.971* | -1395.6 | 4311.6 | 28775.7 | 213.817 | 870.5 | 0.000 |
| 6 | 27.000 | *1.875* | *0.330* | *6.153* | *0.607* | *1.842* | -1997.5 | 6055.9 | 48565.6 | 198.100 | 1497.8 | 0.000 |
| 7 | 22.500 | *1.822* | *0.346* | *5.387* | *0.651* | *1.775* | -2754.9 | 7972.3 | 75071.8 | 184.586 | 2294.2 | 0.000 |
| 8 | 18.000 | *1.789* | *0.365* | *4.880* | *0.689* | *1.735* | -3667.9 | 10044.3 | 108583.1 | 172.948 | 3259.6 | 0.000 |
| 9 | 13.500 | *2.037* | *0.528* | *6.164* | *0.811* | *2.320* | -6494.2 | 12292.0 | 141251.6 | 127.935 | 2636.5 | 0.000 |
| 10 | 9.000 | *2.032* | *0.558* | *5.804* | *0.850* | *2.301* | -8239.8 | 14764.0 | 186203.6 | 121.200 | 3418.7 | 0.000 |
| 11 | 4.500 | *1.992* | *0.569* | *5.184* | *0.878* | *2.214* | -9952.9 | 17506.8 | 237877.0 | 117.193 | 4564.1 | 0.000 |
| 12 | Base | *1.914* | *0.555* | *4.280* | *0.890* | *2.040* | -11349.0 | 20459.0 | 296503.0 | 116.309 | 6447.1 | 0.000 |
| | **Required:** | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

**SEISMIC #2 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -3.420 | -4.745 | 342.000 | -3420.000 | 0.000 | 7.439 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | 255.517 | -441.510 | 342.000 | -3420.000 | 0.000 | 188.773 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 1057.609 | -1351.599 | 342.000 | -3420.000 | 0.000 | -98.286 | 21.908 | 1150.507 |
| 4 | 36.000 | | | | | 679.798 | -922.675 | 342.000 | -3420.000 | 0.000 | -7.362 | 8.789 | 785.399 |
| 5 | 31.500 | | | | | 610.772 | -862.688 | 342.000 | -3420.000 | 0.000 | 734.336 | 100.000 | 734.336 |
| 6 | 27.000 | | | | | 600.766 | -879.766 | 342.000 | -3420.000 | 0.000 | 748.874 | 100.000 | 748.874 |
| 7 | 22.500 | | | | | 605.678 | -919.568 | 342.000 | -3420.000 | 0.000 | 782.754 | 100.000 | 782.754 |
| 8 | 18.000 | | | | | 613.809 | -966.668 | 342.000 | -3420.000 | 0.000 | 822.847 | 100.000 | 822.847 |
| 9 | 13.500 | | | | | 496.290 | -1016.479 | 342.000 | -3420.000 | 0.000 | 865.247 | 100.000 | 865.247 |
| 10 | 9.000 | | | | | 487.853 | -1068.600 | 342.000 | -3420.000 | 0.000 | 909.614 | 100.000 | 909.614 |
| 11 | 4.500 | | | | | 482.488 | -1110.515 | 342.000 | -3420.000 | 0.000 | 945.292 | 100.000 | 945.292 |
| 12 | Base | | | | | 485.572 | -1137.580 | 342.000 | -3420.000 | 0.000 | 968.331 | 100.000 | 968.331 |

Table B.3 CADAM Results for Data Set 1 (continued – 4).

## CADAM Results for Data Set 1

| | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26 Ağustos 04 |
| Dam location: | Eskişehir | Units: | Metric |

### SEISMIC #2 COMBINATION - SUSTAINED ACCELERATIONS (STABILITY ANALYSIS)

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | > 100 | 0.823 | 7.871 | 5.559 | 7.519 | -18.4 | 22.3 | 2.2 | 52.704 | | |
| 2 | 45.000 | 5.741 | 0.739 | 13.541 | 0.827 | 6.266 | -418.5 | 566.3 | 1176.2 | 112.460 | 13.2 | 0.000 |
| 3 | 40.500 | 3.130 | 0.628 | 14.428 | 0.458 | 3.860 | -724.0 | 1152.4 | 4870.9 | 186.581 | 123.2 | 0.000 |
| 4 | 36.000 | 3.034 | 0.519 | 9.117 | 0.645 | 2.665 | -1063.3 | 2047.9 | 10238.4 | 159.965 | 412.3 | 0.000 |
| 5 | 31.500 | 2.823 | 0.490 | 6.777 | 0.724 | 2.271 | -1585.4 | 3232.8 | 19452.4 | 147.484 | 870.5 | 0.000 |
| 6 | 27.000 | 2.664 | 0.494 | 5.677 | 0.775 | 2.102 | -2290.2 | 4636.2 | 33252.8 | 138.443 | 1497.8 | 0.000 |
| 7 | 22.500 | 2.557 | 0.511 | 5.053 | 0.815 | 2.014 | -3177.8 | 6220.6 | 52108.0 | 130.986 | 2294.2 | 0.000 |
| 8 | 18.000 | 2.480 | 0.533 | 4.649 | 0.850 | 1.963 | -4248.1 | 7975.0 | 76357.1 | 124.651 | 3259.6 | 0.000 |
| 9 | 13.500 | 2.807 | 0.732 | 6.239 | 1.015 | 2.747 | -7258.9 | 9912.7 | 98189.6 | 98.469 | 2636.5 | 0.000 |
| 10 | 9.000 | 2.772 | 0.764 | 5.946 | 1.052 | 2.720 | -9216.2 | 12066.2 | 130659.8 | 94.668 | 3418.7 | 0.000 |
| 11 | 4.500 | 2.699 | 0.772 | 5.307 | 1.076 | 2.600 | -11169.3 | 14462.7 | 167954.4 | 92.275 | 4564.1 | 0.000 |
| 12 | Base | 2.588 | 0.755 | 4.330 | 1.079 | 2.364 | -12844.6 | 17018.8 | 209976.4 | 91.491 | 6447.1 | 0.000 |
| | **Required:** | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

### POST-SEISMIC COMBINATION (STRESS ANALYSIS)

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -4.709 | -4.709 | 189.000 | -2490.000 | | | | |
| 2 | 45.000 | | | | | -94.592 | -120.836 | 189.000 | -2490.000 | 0.000 | 33.922 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | -24.982 | -321.781 | 189.000 | -2490.000 | 0.000 | -33.618 | 24.848 | 273.907 |
| 4 | 36.000 | | | | | -125.429 | -169.153 | 189.000 | -2490.000 | 0.000 | 143.987 | 100.000 | 143.987 |
| 5 | 31.500 | | | | | -125.105 | -186.673 | 189.000 | -2490.000 | 0.000 | 158.900 | 100.000 | 158.900 |
| 6 | 27.000 | | | | | -116.103 | -233.687 | 189.000 | -2490.000 | 0.000 | 198.919 | 100.000 | 198.919 |
| 7 | 22.500 | | | | | -107.600 | -289.205 | 189.000 | -2490.000 | 0.000 | 246.177 | 100.000 | 246.177 |
| 8 | 18.000 | | | | | -100.657 | -347.867 | 189.000 | -2490.000 | 0.000 | 296.111 | 100.000 | 296.111 |
| 9 | 13.500 | | | | | -221.099 | -407.873 | 189.000 | -2490.000 | 0.000 | 347.189 | 100.000 | 347.189 |
| 10 | 9.000 | | | | | -234.376 | -468.506 | 189.000 | -2490.000 | 0.000 | 398.802 | 100.000 | 398.802 |
| 11 | 4.500 | | | | | -248.058 | -515.744 | 189.000 | -2490.000 | 0.000 | 439.011 | 100.000 | 439.011 |
| 12 | Base | | | | | -258.063 | -544.649 | 189.000 | -2490.000 | 0.000 | 463.616 | 100.000 | 463.616 |

### POST-SEISMIC COMBINATION (STABILITY ANALYSIS)

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | > 100 | > 100 | > 100 | > 100 | > 100 | -21.2 | 0.0 | 0.0 | 50.000 | | |
| 2 | 45.000 | 47.970 | 4.763 | 58.129 | 15.123 | 37.600 | -484.7 | 101.8 | 44.3 | 52.030 | 13.2 | 0.000 |
| 3 | 40.500 | 25.509 | 3.753 | 14.363 | 2.360 | 7.931 | -854.0 | 227.6 | 600.1 | 64.265 | 123.2 | 0.000 |
| 4 | 36.000 | 18.103 | 2.336 | 5.924 | 2.121 | 4.128 | -1289.7 | 552.0 | 279.4 | 52.474 | 412.3 | 0.000 |
| 5 | 31.500 | 13.505 | 1.825 | 4.604 | 1.869 | 3.254 | -1962.1 | 1075.2 | 812.8 | 53.291 | 870.5 | 0.000 |
| 6 | 27.000 | 10.788 | 1.598 | 4.198 | 1.728 | 2.917 | -2871.3 | 1796.9 | 2640.9 | 55.603 | 1497.8 | 0.000 |
| 7 | 22.500 | 9.048 | 1.478 | 4.027 | 1.645 | 2.751 | -4017.2 | 2717.4 | 6204.3 | 57.628 | 2294.2 | 0.000 |
| 8 | 18.000 | 7.853 | 1.408 | 3.941 | 1.593 | 2.657 | -5399.8 | 3836.4 | 11943.4 | 59.186 | 3259.6 | 0.000 |
| 9 | 13.500 | 7.473 | 1.703 | 6.488 | 2.038 | 4.329 | -8776.8 | 5154.2 | 12123.0 | 54.949 | 2636.5 | 0.000 |
| 10 | 9.000 | 6.818 | 1.672 | 6.438 | 2.008 | 4.263 | -11154.4 | 6670.5 | 19654.5 | 55.552 | 3418.7 | 0.000 |
| 11 | 4.500 | 6.271 | 1.622 | 5.715 | 1.958 | 3.976 | -13584.0 | 8374.5 | 28222.7 | 55.841 | 4564.1 | 0.000 |
| 12 | Base | 5.846 | 1.560 | 4.477 | 1.875 | 3.453 | -15813.4 | 10138.5 | 37073.7 | 55.950 | 6447.1 | 0.000 |
| | **Required:** | 2.000 | 1.100 | 1.100 | 1.100 | 1.100 | | | | | | |

Table B.4 CADAM Loads for Data Set 2.

## CADAM Loads for Data Set 2

| | | | |
|---|---|---|---|
| Project: | Risk Analysis | Project engineer: | M. Resat Beser |
| Dam: | Porsuk | Analysis performed by: | M. Resat Beser |
| Owner: | METU Civil Engineering Department | Date: | 26 Ağustos 04 |
| Dam location: | Eskişehir | Units: | Metric |

### STATIC LOADS (1/3)

| Joint | | Self-Weight | | | | Normal Operating level | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dam | | Concentrated masses | | Upstream reservoir | | | | Downstream reservoir | | | | Crest Overtopping | | Uplift | | Ice | |
| ID | Upstream | Vertical load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Vertical load | | Normal load | | Horizontal load | |
| | elevation | D | position x | Mv | position x | Hnu | elevation | Vnu | position x | Hnd | elevation | Vnd | position x | Vnc | position x | Un | position l | Un | position l |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | -21.2 | 2.250 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | -498.0 | 2.250 | | | 1.8 | 45.200 | | | | | | | | | 13.2 | 1.500 | 100.0 | 45.340 |
| 3 | 40.500 | -977.2 | 2.256 | | | 127.6 | 42.200 | | | | | | | | | 123.2 | 1.642 | 100.0 | 45.340 |
| 4 | 36.000 | -1702.0 | 2.790 | | | 452.0 | 39.200 | | | | | | | | | 412.3 | 2.919 | 100.0 | 45.340 |
| 5 | 31.500 | -2832.6 | 3.829 | | | 975.2 | 36.200 | | | | | | | | | 870.5 | 4.196 | 100.0 | 45.340 |
| 6 | 27.000 | -4369.0 | 5.047 | | | 1696.9 | 33.200 | | | | | | | | | 1497.8 | 5.472 | 100.0 | 45.340 |
| 7 | 22.500 | -6311.3 | 6.325 | | | 2617.4 | 30.200 | | | | | | | | | 2294.2 | 6.749 | 100.0 | 45.340 |
| 8 | 18.000 | -8659.4 | 7.622 | | | 3736.4 | 27.200 | | | | | | | | | 3259.6 | 8.026 | 100.0 | 45.340 |
| 9 | 13.500 | -11413.4 | 8.925 | | | 5054.2 | 24.200 | | | | | | | | | 2636.5 | 9.303 | 100.0 | 45.340 |
| 10 | 9.000 | -14573.1 | 10.227 | | | 6570.5 | 21.200 | | | | | | | | | 3418.7 | 10.580 | 100.0 | 45.340 |
| 11 | 4.500 | -18138.7 | 11.528 | | | 8285.6 | 18.200 | | | -11.0 | 5.000 | -9.4 | 35.144 | | | 4564.1 | 12.536 | 100.0 | 45.340 |
| 12 | Base | -22110.2 | 12.827 | | | 10509.4 | 15.429 | | | -176.6 | 2.000 | -150.3 | 37.698 | | | 6447.1 | 15.495 | 50.6 | 46.028 |

### STATIC LOADS (2/3)

| Joint | | Silt | | | | Flood level | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Upstream reservoir | | | | Downstream reservoir | | | | Crest Overtopping | | Uplift | |
| ID | Upstream | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Vertical load | | Normal load | |
| | elevation | Sh | position x | Sv | position x | Hfu | elevation | Vfu | position x | Hfd | elevation | Vfd | position x | Vfc | position x | Uf | position l |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | | | | | 50.2 | 46.067 | | | | | | | | | 70.6 | 1.500 |
| 3 | 40.500 | | | | | 290.8 | 43.067 | | | | | | | | | 186.0 | 1.642 |
| 4 | 36.000 | | | | | 730.1 | 40.067 | | | | | | | | | 524.0 | 2.919 |
| 5 | 31.500 | | | | | 1368.0 | 37.067 | | | | | | | | | 1031.0 | 4.196 |
| 6 | 27.000 | | | | | 2204.5 | 34.067 | | | | | | | | | 1707.1 | 5.472 |
| 7 | 22.500 | | | | | 3239.7 | 31.067 | | | | | | | | | 2552.4 | 6.749 |
| 8 | 18.000 | | | | | 4473.6 | 28.067 | | | | | | | | | 3566.7 | 8.026 |
| 9 | 13.500 | | | | | 5906.1 | 25.067 | | | | | | | | | 2850.1 | 9.303 |
| 10 | 9.000 | | | | | 7537.2 | 22.067 | | | | | | | | | 3661.6 | 10.580 |
| 11 | 4.500 | | | | | 9367.0 | 19.067 | | | -11.0 | 5.000 | -9.4 | 35.144 | | | 4836.3 | 12.498 |
| 12 | Base | 15.8 | 1.000 | | | 10509.4 | 15.429 | | | -381615.2 | 24.587 | -269464.6 | 22.102 | | | 6748.5 | 15.390 |

Table B.4 CADAM Loads for Data Set 2 (continued – 1).

| CADAM Loads for Data Set 2 | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

| STATIC LOADS (3/3) | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | Post-tensioning | | | | | | | | Applied forces | | | |
| | | Crest | | Downstream face | | | | | | | | | |
| | | Vertical load | | Vertical load | | | | Horizontal load | | Horizontal load | | Vertical load | |
| ID | Upstream elevation (m) | Pc (kN) | position x (m) | Pdv (kN) | position x (m) | | | Pdh (kN) | elevation (m) | Fh (kN) | elevation (m) | Fv (kN) | position x (m) |
| 1 | 49.500 | | | | | | | | | | | | |
| 2 | 45.000 | | | | | | | | | | | | |
| 3 | 40.500 | | | | | | | | | | | | |
| 4 | 36.000 | | | | | | | | | | | | |
| 5 | 31.500 | | | | | | | | | | | | |
| 6 | 27.000 | | | | | | | | | | | | |
| 7 | 22.500 | | | | | | | | | | | | |
| 8 | 18.000 | | | | | | | | | | | | |
| 9 | 13.500 | | | | | | | | | | | | |
| 10 | 9.000 | | | | | | | | | | | | |
| 11 | 4.500 | | | | | | | | | | | | |
| 12 | Base | | | | | | | | | | | | |

| PSEUDO-STATIC LOADS (SEISMIC COEFFICIENT)-STRESS ANALYSIS | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | Inertia loads | | | | | | | | Reservoirs (operating level) | | | | | | | | Silt | | | |
| | | Dam | | | | Concentrated masses | | | | Upstream | | | | Downstream | | | | | | | |
| | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | |
| ID | Upstream elevation (m) | Qh (kN) | elevation (m) | Qv (kN) | position x (m) | Mdh (kN) | elevation (m) | Mdv (kN) | position x (m) | Hdu (kN) | elevation (m) | Vdu (kN) | position x (m) | Hdd (kN) | elevation (m) | Vdd (kN) | position x (m) | Sdh (kN) | elevation (m) | Sdv (kN) | position x (m) |
| 1 | 49.500 | -6.4 | 49.600 | -4.2 | 2.250 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | -149.4 | 47.350 | -99.6 | 2.250 | | | | | -5.0 | 45.240 | | | | | | | | | | |
| 3 | 40.500 | -293.2 | 45.089 | -195.4 | 2.256 | | | | | -124.3 | 42.540 | | | | | | | | | | |
| 4 | 36.000 | -510.6 | 42.087 | -340.4 | 2.790 | | | | | -321.0 | 39.840 | | | | | | | | | | |
| 5 | 31.500 | -849.8 | 38.706 | -566.5 | 3.829 | | | | | -571.3 | 37.140 | | | | | | | | | | |
| 6 | 27.000 | -1310.7 | 35.346 | -873.8 | 5.047 | | | | | -865.6 | 34.440 | | | | | | | | | | |
| 7 | 22.500 | -1893.4 | 32.061 | -1262.3 | 6.325 | | | | | -1198.1 | 31.740 | | | | | | | | | | |
| 8 | 18.000 | -2597.8 | 28.841 | -1731.9 | 7.622 | | | | | -1564.7 | 29.040 | | | | | | | | | | |
| 9 | 13.500 | -3424.0 | 25.669 | -2282.7 | 8.925 | | | | | -1962.6 | 26.340 | | | | | | | | | | |
| 10 | 9.000 | -4371.9 | 22.532 | -2914.6 | 10.227 | | | | | -2389.4 | 23.640 | | | | | | | | | | |
| 11 | 4.500 | -5441.6 | 19.421 | -3627.7 | 11.528 | | | | | -2843.4 | 20.940 | | | -1.8 | 5.100 | -3.4 | 35.106 | | | | |
| 12 | Base | -6633.1 | 16.330 | -4422.0 | 12.827 | | | | | -3322.9 | 18.240 | | | -14.4 | 2.400 | -42.3 | 37.599 | -16.1 | 1.200 | | |

Table B.4 CADAM Loads for Data Set 2 (continued – 2).

| CADAM Loads for Data Set 2 | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

| PSEUDO-STATIC LOADS (SEISMIC COEFFICIENT)-STABILITY ANALYSIS | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | Inertia loads | | | | | | | | Reservoirs (operating level) | | | | | | | | Silt | | | |
| | | Dam | | | | Concentrated masses | | | | Upstream | | | | Downstream | | | | | | | |
| ID | Upstream | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | | Horizontal load | | Vertical load | |
| | elevation | Qh' | elevation | Qv' | position x | Mdh' | elevation | Mdv' | position x | Hdu' | elevation | Vdu' | position x | Hdd' | elevation | Vdd' | position x | Sdh' | elevation | Sdv' | position x |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | -4.2 | 49.600 | -2.8 | 2.250 | | | | | | | | | | | | | | | | |
| 2 | 45.000 | -99.6 | 47.350 | -66.2 | 2.250 | | | | | -3.3 | 45.240 | | | | | | | | | | |
| 3 | 40.500 | -195.4 | 45.089 | -130.0 | 2.256 | | | | | -82.9 | 42.540 | | | | | | | | | | |
| 4 | 36.000 | -340.4 | 42.087 | -226.4 | 2.790 | | | | | -214.0 | 39.840 | | | | | | | | | | |
| 5 | 31.500 | -566.5 | 38.706 | -376.7 | 3.829 | | | | | -380.9 | 37.140 | | | | | | | | | | |
| 6 | 27.000 | -873.8 | 35.346 | -581.1 | 5.047 | | | | | -577.1 | 34.440 | | | | | | | | | | |
| 7 | 22.500 | -1262.3 | 32.061 | -839.4 | 6.325 | | | | | -798.7 | 31.740 | | | | | | | | | | |
| 8 | 18.000 | -1731.9 | 28.841 | -1151.7 | 7.622 | | | | | -1043.1 | 29.040 | | | | | | | | | | |
| 9 | 13.500 | -2282.7 | 25.669 | -1518.0 | 8.925 | | | | | -1308.4 | 26.340 | | | | | | | | | | |
| 10 | 9.000 | -2914.6 | 22.532 | -1938.2 | 10.227 | | | | | -1593.0 | 23.640 | | | | | | | | | | |
| 11 | 4.500 | -3627.7 | 19.421 | -2412.5 | 11.528 | | | | | -1895.6 | 20.940 | | | -1.2 | 5.100 | -2.3 | 35.106 | | | | |
| 12 | Base | -4422.0 | 16.330 | -2940.7 | 12.827 | | | | | -2215.3 | 18.240 | | | -9.7 | 2.400 | -28.2 | 37.598 | -10.8 | 1.200 | | |

| PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STRESS ANALYSIS (1/2) | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Joint | | First mode | | | | | | | | Higher modes | | | | | | | | Modal combination | | | |
| | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | SRSS | | Summation | |
| ID | Upstream | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | | Horizontal load | |
| | elevation | Eq1 | elevation | Hd1 | elevation | Md1 | elevation | Em1 | elevation | Eqs | elevation | Hds | elevation | Mds | elevation | Ems | elevation | Emc | elevation | Emc | elevation |
| | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) | (kN) | (m) |
| 1 | 49.500 | -76.6 | 49.600 | | | | | -76.6 | 49.600 | 10.9 | 49.600 | | | | | 10.9 | 49.600 | -77.4 | 49.600 | | |
| 2 | 45.000 | -1581.1 | 47.465 | -10.2 | 45.202 | | | -1591.2 | 47.451 | 206.3 | 47.548 | 23.9 | 45.377 | | | 230.2 | 47.323 | -1607.8 | 47.448 | | |
| 3 | 40.500 | -2703.6 | 45.550 | -520.5 | 42.375 | | | -3224.1 | 45.037 | 315.0 | 45.979 | 70.4 | 44.489 | | | 385.4 | 45.707 | -3247.0 | 45.046 | | |
| 4 | 36.000 | -3960.4 | 43.199 | -1375.2 | 39.765 | | | -5335.6 | 42.314 | 380.2 | 44.692 | 32.9 | 52.035 | | | 413.1 | 45.276 | -5351.6 | 42.332 | | |
| 5 | 31.500 | -5453.7 | 40.603 | -2334.9 | 37.283 | | | -7788.6 | 39.608 | 377.0 | 44.881 | -44.7 | 19.924 | | | 332.3 | 48.234 | -7795.7 | 39.625 | | |
| 6 | 27.000 | -6973.1 | 38.133 | -3328.7 | 34.884 | | | -10301.8 | 37.083 | 257.8 | 52.300 | -170.2 | 26.687 | | | 87.6 | 102.040 | -10302.1 | 37.092 | | |
| 7 | 22.500 | -8370.1 | 35.907 | -4313.1 | 32.573 | | | -12683.2 | 34.773 | -10.6 | -651.854 | -351.7 | 25.625 | | | -362.3 | 5.825 | -12688.4 | 34.760 | | |
| 8 | 18.000 | -9553.9 | 33.977 | -5252.2 | 30.374 | | | -14806.1 | 32.699 | -448.7 | 4.242 | -592.9 | 23.400 | | | -1041.6 | 15.147 | -14842.7 | 32.635 | | |
| 9 | 13.500 | -10473.8 | 32.386 | -6131.0 | 28.281 | | | -16604.8 | 30.870 | -1068.0 | 10.850 | -892.6 | 20.808 | | | -1960.6 | 15.384 | -16720.1 | 30.710 | | |
| 10 | 9.000 | -11117.2 | 31.172 | -6955.2 | 26.266 | | | -18072.4 | 29.284 | -1871.2 | 10.985 | -1244.1 | 18.094 | | | -3115.3 | 13.824 | -18339.0 | 28.953 | | |
| 11 | 4.500 | -11504.7 | 30.357 | -7728.8 | 24.315 | | | -19233.6 | 27.929 | -2853.7 | 9.504 | -1636.8 | 15.364 | | | -4490.5 | 11.640 | -19750.8 | 27.326 | | |
| 12 | Base | -11631.8 | 30.059 | -8447.1 | 22.441 | | | -20078.9 | 26.854 | -4016.5 | 7.387 | -2066.6 | 12.630 | | | -6083.0 | 9.168 | -20980.2 | 25.838 | | |

Table B.4  CADAM Loads for Data Set 2 (continued – 3).

## CADAM Loads for Data Set 2

| | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

**PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STRESS ANALYSIS (2/2)**

| Joint | | Vertical loads | | | | | | | | | | horizontal loads | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dam | | Reservoir (upstream) | | Reservoir (downstream) | | Concentrated masses | | Silt | | Reservoir (downstream) | | Silt | |
| ID | Upstream elevation (m) | Vertical load Eqv (kN) | position x (m) | Vertical load Vdu (kN) | position x (m) | Vertical load Vdd (kN) | position x (m) | Vertical load Mdv (kN) | position x (m) | Vertical load Sdv (kN) | position x (m) | Horizontal load Hdd (kN) | elevation (m) | Horizontal load Sdh (kN) | elevation (m) |
| 1 | 49.500 | -4.2 | 2.250 | | | | | | | | | | | | |
| 2 | 45.000 | -99.6 | 2.250 | | | | | | | | | | | | |
| 3 | 40.500 | -195.4 | 2.256 | | | | | | | | | | | | |
| 4 | 36.000 | -340.4 | 2.790 | | | | | | | | | | | | |
| 5 | 31.500 | -566.5 | 3.829 | | | | | | | | | | | | |
| 6 | 27.000 | -873.8 | 5.047 | | | | | | | | | | | | |
| 7 | 22.500 | -1262.3 | 6.325 | | | | | | | | | | | | |
| 8 | 18.000 | -1731.9 | 7.622 | | | | | | | | | | | | |
| 9 | 13.500 | -2282.7 | 8.925 | | | | | | | | | | | | |
| 10 | 9.000 | -2914.6 | 10.227 | | | | | | | | | | | | |
| 11 | 4.500 | -3627.7 | 11.528 | | | -3.4 | 35.106 | | | | | -1.8 | 5.100 | | |
| 12 | Base | -4422.0 | 12.827 | | | -42.3 | 37.599 | | | | | -14.4 | 2.400 | -16.1 | 1.200 |

**PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STABILITY ANALYSIS (1/2)**

| Joint | | First mode | | | | | | | | Higher modes | | | | | | | | Modal combination | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | Dam | | Reservoir (upstream) | | Concentrated masses | | Total | | SRSS | | Summation | |
| ID | Upstream elevation (m) | Horizontal load Eq1' (kN) | elevation (m) | Horizontal load Hd1' (kN) | elevation (m) | Horizontal load Md1' (kN) | elevation (m) | Horizontal load Em1' (kN) | elevation (m) | Horizontal load Eqs' (kN) | elevation (m) | Horizontal load Hds' (kN) | elevation (m) | Horizontal load Mds' (kN) | elevation (m) | Horizontal load Ems' (kN) | elevation (m) | Horizontal load Emc' (kN) | elevation (m) | Horizontal load Emc' (kN) | elevation (m) |
| 1 | 49.500 | -51.1 | 49.600 | | | | | -51.1 | 49.600 | 7.2 | 49.600 | | | | | 7.2 | 49.600 | -51.6 | 49.600 | | |
| 2 | 45.000 | -1054.1 | 47.465 | -6.8 | 45.202 | | | -1060.8 | 47.451 | 137.5 | 47.548 | 15.9 | 45.377 | | | 153.5 | 47.323 | -1071.9 | 47.448 | | |
| 3 | 40.500 | -1802.4 | 45.550 | -347.0 | 42.375 | | | -2149.4 | 45.037 | 210.0 | 45.979 | 46.9 | 44.489 | | | 256.9 | 45.707 | -2164.7 | 45.046 | | |
| 4 | 36.000 | -2640.3 | 43.199 | -916.8 | 39.765 | | | -3557.1 | 42.314 | 253.5 | 44.692 | 21.9 | 52.035 | | | 275.4 | 45.276 | -3567.7 | 42.332 | | |
| 5 | 31.500 | -3635.8 | 40.603 | -1556.6 | 37.283 | | | -5192.4 | 39.608 | 251.3 | 44.881 | -29.8 | 19.924 | | | 221.6 | 48.234 | -5197.1 | 39.625 | | |
| 6 | 27.000 | -4648.7 | 38.133 | -2219.1 | 34.884 | | | -6867.9 | 37.083 | 171.9 | 52.300 | -113.5 | 26.687 | | | 58.4 | 102.040 | -6868.1 | 37.092 | | |
| 7 | 22.500 | -5580.0 | 35.907 | -2875.4 | 32.573 | | | -8455.5 | 34.773 | -7.1 | -651.854 | -234.5 | 25.625 | | | -241.5 | 5.825 | -8458.9 | 34.760 | | |
| 8 | 18.000 | -6369.3 | 33.977 | -3501.5 | 30.374 | | | -9870.7 | 32.699 | -299.1 | 4.242 | -395.3 | 23.400 | | | -694.4 | 15.147 | -9895.1 | 32.635 | | |
| 9 | 13.500 | -6982.5 | 32.386 | -4087.4 | 28.281 | | | -11069.9 | 30.870 | -712.0 | 10.850 | -595.1 | 20.808 | | | -1307.1 | 15.384 | -11146.8 | 30.710 | | |
| 10 | 9.000 | -7411.5 | 31.172 | -4636.8 | 26.266 | | | -12048.3 | 29.284 | -1247.4 | 10.985 | -829.4 | 18.094 | | | -2076.8 | 13.824 | -12226.0 | 28.953 | | |
| 11 | 4.500 | -7669.8 | 30.357 | -5152.6 | 24.315 | | | -12822.4 | 27.929 | -1902.4 | 9.504 | -1091.2 | 15.364 | | | -2993.7 | 11.640 | -13167.2 | 27.326 | | |
| 12 | Base | -7754.5 | 30.059 | -5631.4 | 22.441 | | | -13386.0 | 26.854 | -2677.6 | 7.387 | -1377.7 | 12.630 | | | -4055.4 | 9.168 | -13986.8 | 25.838 | | |

Table B.4 CADAM Loads for Data Set 2 (continued – 4).

## CADAM Loads for Data Set 2

| | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

**PSEUDO-DYNAMIC LOADS (CHOPRA'S METHOD)-STABILITY ANALYSIS (2/2)**

| Joint | | Vertical loads | | | | | | | | | | horizontal loads | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dam | | Reservoir (upstream) | | Reservoir (downstream) | | Concentrated masses | | Silt | | Reservoir (downstream) | | Silt | |
| | | Vertical load | | Vertical load | | Vertical load | | Vertical load | | Vertical load | | Horizontal load | | Horizontal load | |
| ID | Upstream elevation (m) | Eqv' (kN) | position x (m) | Vdu' (kN) | position x (m) | Vdd' (kN) | position x (m) | Mdv' (kN) | position x (m) | Sdv' (kN) | position x (m) | Hdd' (kN) | elevation (m) | Sdh' (kN) | elevation (m) |
| 1 | 49.500 | -2.8 | 2.250 | | | | | | | | | | | | |
| 2 | 45.000 | -66.2 | 2.250 | | | | | | | | | | | | |
| 3 | 40.500 | -130.0 | 2.256 | | | | | | | | | | | | |
| 4 | 36.000 | -226.4 | 2.790 | | | | | | | | | | | | |
| 5 | 31.500 | -376.7 | 3.829 | | | | | | | | | | | | |
| 6 | 27.000 | -581.1 | 5.047 | | | | | | | | | | | | |
| 7 | 22.500 | -839.4 | 6.325 | | | | | | | | | | | | |
| 8 | 18.000 | -1151.7 | 7.622 | | | | | | | | | | | | |
| 9 | 13.500 | -1518.0 | 8.925 | | | | | | | | | | | | |
| 10 | 9.000 | -1938.2 | 10.227 | | | | | | | | | | | | |
| 11 | 4.500 | -2412.5 | 11.528 | | | -2.3 | 35.106 | | | | | -1.2 | 5.100 | | |
| 12 | Base | -2940.7 | 12.827 | | | -28.2 | 37.598 | | | | | -9.7 | 2.400 | -10.8 | 1.200 |

Table B.5 CADAM Results for Data Set 2.

# CADAM Results for Data Set 2

| | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

## LOAD COMBINATION FACTORS

| | Usual | Flood | Seismic #1 | Seismic #2 | Post-seismic |
|---|---|---|---|---|---|
| **Self-weight** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Hydrostatic (upstream)** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Hydrostatic (downstream)** | 0.600 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Uplift pressures** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Silts** | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| **Ice** | 1.000 | | 1.000 | 1.000 | 1.000 |
| **Post-tensioning** | | | | | |
| **Applied forces** | | | | | |
| **Seismic (horizontal)** | | | -1.000 | -1.000 | |
| **Seismic (vertical)** | | | -1.000 | -1.000 | |

## USUAL COMBINATION (STRESS ANALYSIS)

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -4.709 | -4.709 | 0.000 | -1260.000 | | | | |
| 2 | 45.000 | | | | | -94.592 | -120.836 | 0.000 | -1260.000 | 0.000 | 33.922 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | -24.982 | -321.781 | 0.000 | -1260.000 | 0.000 | -33.618 | 24.848 | 273.907 |
| 4 | 36.000 | | | | | -125.429 | -169.153 | 0.000 | -1260.000 | 0.000 | 143.987 | 100.000 | 143.987 |
| 5 | 31.500 | | | | | -125.105 | -186.673 | 0.000 | -1260.000 | 0.000 | 158.900 | 100.000 | 158.900 |
| 6 | 27.000 | | | | | -116.103 | -233.687 | 0.000 | -1260.000 | 0.000 | 198.919 | 100.000 | 198.919 |
| 7 | 22.500 | | | | | -107.600 | -289.205 | 0.000 | -1260.000 | 0.000 | 246.177 | 100.000 | 246.177 |
| 8 | 18.000 | | | | | -100.657 | -347.867 | 0.000 | -1260.000 | 0.000 | 296.111 | 100.000 | 296.111 |
| 9 | 13.500 | | | | | -221.099 | -407.873 | 0.000 | -1260.000 | 0.000 | 347.189 | 100.000 | 347.189 |
| 10 | 9.000 | | | | | -234.376 | -468.506 | 0.000 | -1260.000 | 0.000 | 398.802 | 100.000 | 398.802 |
| 11 | 4.500 | | | | | -248.252 | -515.339 | 0.000 | -1260.000 | 0.000 | 438.667 | 100.000 | 438.667 |
| 12 | Base | | | | | -241.160 | -558.500 | 0.000 | -1260.000 | 0.000 | 475.406 | 100.000 | 475.406 |

## USUAL COMBINATION (STABILITY ANALYSIS)

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final | Passive |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | Force (kN) | wedge resistance (kN) |
| 1 | 49.500 | > 100 | > 100 | > 100 | > 100 | > 100 | -21.2 | 0.0 | 0.0 | 50.000 | | |
| 2 | 45.000 | 47.970 | 4.763 | 58.129 | 15.123 | 37.600 | -484.7 | 101.8 | 44.3 | 52.030 | 13.2 | 0.000 |
| 3 | 40.500 | 25.509 | 3.753 | 14.363 | 2.360 | 7.931 | -854.0 | 227.6 | 600.1 | 64.265 | 123.2 | 0.000 |
| 4 | 36.000 | 18.103 | 2.336 | 5.924 | 2.121 | 4.128 | -1289.7 | 552.0 | 279.4 | 52.474 | 412.3 | 0.000 |
| 5 | 31.500 | 13.505 | 1.825 | 4.604 | 1.869 | 3.254 | -1962.1 | 1075.2 | 812.8 | 53.291 | 870.5 | 0.000 |
| 6 | 27.000 | 10.788 | 1.598 | 4.198 | 1.728 | 2.917 | -2871.3 | 1796.9 | 2640.9 | 55.603 | 1497.8 | 0.000 |
| 7 | 22.500 | 9.048 | 1.478 | 4.027 | 1.645 | 2.751 | -4017.2 | 2717.4 | 6204.3 | 57.628 | 2294.2 | 0.000 |
| 8 | 18.000 | 7.853 | 1.408 | 3.941 | 1.593 | 2.657 | -5399.8 | 3836.4 | 11943.4 | 59.186 | 3259.6 | 0.000 |
| 9 | 13.500 | 7.473 | 1.703 | 6.488 | 2.038 | 4.329 | -8776.8 | 5154.2 | 12123.0 | 54.949 | 2636.5 | 0.000 |
| 10 | 9.000 | 6.818 | 1.672 | 6.438 | 2.008 | 4.263 | -11154.4 | 6670.5 | 19654.5 | 55.552 | 3418.7 | 0.000 |
| 11 | 4.500 | 6.267 | 1.621 | 5.713 | 1.958 | 3.975 | -13580.3 | 8379.0 | 28159.7 | 55.830 | 4564.1 | 0.000 |
| 12 | Base | 5.652 | 1.505 | 4.510 | 1.845 | 3.443 | -15753.3 | 10469.9 | 41052.1 | 56.614 | 6447.1 | 0.000 |
| | **Required:** | 3.000 | 1.500 | 1.200 | 1.200 | 1.200 | | | | | | |

Table B.5 CADAM Results for Data Set 2 (continued – 1).

| CADAM Results for Data Set 2 | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

**FLOOD COMBINATION (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -4.709 | -4.709 | 189.000 | -1890.000 | | | | |
| 2 | 45.000 | | | | | -63.391 | -126.531 | 189.000 | -1890.000 | 0.000 | 16.742 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 11.788 | -333.045 | 189.000 | -1890.000 | 0.000 | -22.801 | 21.435 | 283.495 |
| 4 | 36.000 | | | | | -53.878 | -215.198 | 189.000 | -1890.000 | 0.000 | 183.181 | 100.000 | 183.181 |
| 5 | 31.500 | | | | | -37.196 | -249.077 | 189.000 | -1890.000 | 0.000 | 212.019 | 100.000 | 212.019 |
| 6 | 27.000 | | | | | -18.837 | -305.448 | 189.000 | -1890.000 | 0.000 | 260.003 | 100.000 | 260.003 |
| 7 | 22.500 | | | | | -4.295 | -367.004 | 189.000 | -1890.000 | 0.000 | 312.401 | 100.000 | 312.401 |
| 8 | 18.000 | | | | | 6.863 | -429.880 | 189.000 | -1890.000 | 0.000 | 365.923 | 100.000 | 365.923 |
| 9 | 13.500 | | | | | -120.674 | -492.994 | 189.000 | -1890.000 | 0.000 | 419.646 | 100.000 | 419.646 |
| 10 | 9.000 | | | | | -131.567 | -556.012 | 189.000 | -1890.000 | 0.000 | 473.288 | 100.000 | 473.288 |
| 11 | 4.500 | | | | | -143.361 | -605.137 | 189.000 | -1890.000 | 0.000 | 515.105 | 100.000 | 515.105 |
| 12 | Base | | | | | -151.834 | -635.574 | 189.000 | -1890.000 | 0.000 | 541.013 | 100.000 | 541.013 |

**FLOOD COMBINATION (STABILITY ANALYSIS)**

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | > 100 | > 100 | > 100 | > 100 | > 100 | -21.2 | 0.0 | 0.0 | 50.000 | | |
| 2 | 45.000 | 95.561 | 8.508 | 11.081 | 4.220 | 7.050 | -427.3 | 50.2 | 106.5 | 55.541 | 70.6 | 0.000 |
| 3 | 40.500 | 19.115 | 2.721 | 9.662 | 1.922 | 5.253 | -791.2 | 290.8 | 697.2 | 67.890 | 186.0 | 0.000 |
| 4 | 36.000 | 13.471 | 1.614 | 5.046 | 1.685 | 3.248 | -1178.0 | 730.1 | 1030.7 | 59.992 | 524.0 | 0.000 |
| 5 | 31.500 | 10.447 | 1.317 | 4.268 | 1.525 | 2.747 | -1801.6 | 1368.0 | 2797.2 | 62.336 | 1031.0 | 0.000 |
| 6 | 27.000 | 8.658 | 1.207 | 4.028 | 1.450 | 2.559 | -2661.9 | 2204.5 | 6437.3 | 64.730 | 1707.1 | 0.000 |
| 7 | 22.500 | 7.476 | 1.160 | 3.928 | 1.413 | 2.473 | -3758.9 | 3239.7 | 12391.5 | 66.281 | 2552.4 | 0.000 |
| 8 | 18.000 | 6.558 | 1.138 | 3.879 | 1.393 | 2.428 | -5092.7 | 4473.6 | 21100.2 | 67.207 | 3566.7 | 0.000 |
| 9 | 13.500 | 6.470 | 1.450 | 6.418 | 1.786 | 4.005 | -8563.3 | 5906.1 | 24166.2 | 60.112 | 2850.1 | 0.000 |
| 10 | 9.000 | 5.988 | 1.448 | 6.390 | 1.782 | 3.980 | -10911.5 | 7537.2 | 35630.9 | 60.288 | 3661.6 | 0.000 |
| 11 | 4.500 | 5.571 | 1.423 | 5.722 | 1.758 | 3.753 | -13311.9 | 9356.0 | 48686.2 | 60.282 | 4836.3 | 0.000 |
| 12 | Base | 5.237 | 1.381 | 4.533 | 1.704 | 3.299 | -15511.9 | 11234.8 | 62578.3 | 60.239 | 6748.5 | 0.000 |
| | **Required:** | 2.000 | 1.300 | 1.100 | 1.100 | 1.100 | | | | | | |

**SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -3.579 | -3.955 | 342.000 | -3420.000 | 0.000 | 2.119 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | 31.913 | -203.078 | 342.000 | -3420.000 | 0.000 | 85.389 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 420.074 | -687.478 | 342.000 | -3420.000 | 0.000 | -38.140 | 19.831 | 585.195 |
| 4 | 36.000 | | | | | 295.442 | -512.272 | 342.000 | -3420.000 | 0.000 | 436.056 | 100.000 | 436.056 |
| 5 | 31.500 | | | | | 326.733 | -548.492 | 342.000 | -3420.000 | 0.000 | 466.887 | 100.000 | 466.887 |
| 6 | 27.000 | | | | | 385.510 | -628.850 | 342.000 | -3420.000 | 0.000 | 535.289 | 100.000 | 535.289 |
| 7 | 22.500 | | | | | 451.875 | -723.996 | 342.000 | -3420.000 | 0.000 | 616.280 | 100.000 | 616.280 |
| 8 | 18.000 | | | | | 520.669 | -825.336 | 342.000 | -3420.000 | 0.000 | 702.542 | 100.000 | 702.542 |
| 9 | 13.500 | | | | | 464.215 | -929.604 | 342.000 | -3420.000 | 0.000 | 791.297 | 100.000 | 791.297 |
| 10 | 9.000 | | | | | 516.124 | -1035.344 | 342.000 | -3420.000 | 0.000 | 881.305 | 100.000 | 881.305 |
| 11 | 4.500 | | | | | 568.133 | -1127.762 | 342.000 | -3420.000 | 0.000 | 959.973 | 100.000 | 959.973 |
| 12 | Base | | | | | 622.916 | -1199.010 | 342.000 | -3420.000 | 0.000 | 1020.621 | 100.000 | 1020.621 |

Table B.5 CADAM Results for Data Set 2 (continued – 2).

## CADAM Results for Data Set 2

| | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

### SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STABILITY ANALYSIS)

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | > 100 | 2.667 | 5.067 | 4.687 | 5.000 | -17.0 | 6.4 | 0.6 | 50.833 | | |
| 2 | 45.000 | 16.281 | 1.503 | 6.178 | 1.723 | 4.413 | -385.1 | 256.2 | 396.5 | 72.882 | 13.2 | 0.000 |
| 3 | 40.500 | 5.871 | 1.021 | 7.003 | 0.809 | 3.067 | -658.6 | 645.0 | 2239.3 | 119.031 | 123.2 | 0.000 |
| 4 | 36.000 | 4.717 | 0.686 | 5.327 | 0.910 | 2.261 | -949.3 | 1383.6 | 5160.6 | 112.085 | 412.3 | 0.000 |
| 5 | 31.500 | 3.740 | 0.559 | 4.494 | 0.899 | 1.971 | -1395.6 | 2496.3 | 11554.6 | 115.779 | 870.5 | 0.000 |
| 6 | 27.000 | 3.103 | 0.503 | 4.108 | 0.886 | 1.842 | -1997.5 | 3973.3 | 22782.5 | 119.475 | 1497.8 | 0.000 |
| 7 | 22.500 | 2.675 | 0.474 | 3.900 | 0.877 | 1.775 | -2754.9 | 5808.8 | 40172.0 | 122.019 | 2294.2 | 0.000 |
| 8 | 18.000 | 2.373 | 0.459 | 3.774 | 0.872 | 1.735 | -3667.9 | 7999.0 | 65029.1 | 123.632 | 3259.6 | 0.000 |
| 9 | 13.500 | 2.524 | 0.616 | 5.033 | 1.001 | 2.320 | -6494.2 | 10540.8 | 90468.9 | 99.916 | 2636.5 | 0.000 |
| 10 | 9.000 | 2.344 | 0.613 | 4.956 | 1.002 | 2.301 | -8239.8 | 13431.9 | 130241.3 | 99.801 | 3418.7 | 0.000 |
| 11 | 4.500 | 2.175 | 0.597 | 4.588 | 0.996 | 2.214 | -9952.9 | 16661.3 | 178802.4 | 100.506 | 4564.1 | 0.000 |
| 12 | Base | 2.005 | 0.564 | 3.896 | 0.980 | 2.040 | -11349.0 | 20125.1 | 235690.5 | 102.709 | 6447.1 | 0.000 |
| | **Required:** | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

### SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation | Crack length | | Crack length | | Upstream | Downstream | tension | Compression | Upstream | Maximum | Maximum at l-axis | Downstream |
| | (m) | (%) | (m) | (%) | (m) | (kPa) | (kPa) | (kPa) | (kPa) | (kPa) | (kPa) | (% of joint) | (kPa) |
| 1 | 49.500 | | | | | -3.957 | -4.208 | 342.000 | -3420.000 | 0.000 | 1.413 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | -10.292 | -175.701 | 342.000 | -3420.000 | 0.000 | 68.233 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | 271.639 | -565.629 | 342.000 | -3420.000 | 0.000 | -36.288 | 20.932 | 481.474 |
| 4 | 36.000 | | | | | 155.016 | -397.894 | 342.000 | -3420.000 | 0.000 | 338.695 | 100.000 | 338.695 |
| 5 | 31.500 | | | | | 175.957 | -427.873 | 342.000 | -3420.000 | 0.000 | 364.214 | 100.000 | 364.214 |
| 6 | 27.000 | | | | | 218.114 | -497.115 | 342.000 | -3420.000 | 0.000 | 423.154 | 100.000 | 423.154 |
| 7 | 22.500 | | | | | 265.162 | -579.052 | 342.000 | -3420.000 | 0.000 | 492.901 | 100.000 | 492.901 |
| 8 | 18.000 | | | | | 313.308 | -666.168 | 342.000 | -3420.000 | 0.000 | 567.055 | 100.000 | 567.055 |
| 9 | 13.500 | | | | | 235.493 | -755.683 | 342.000 | -3420.000 | 0.000 | 643.252 | 100.000 | 643.252 |
| 10 | 9.000 | | | | | 265.641 | -846.388 | 342.000 | -3420.000 | 0.000 | 720.462 | 100.000 | 720.462 |
| 11 | 4.500 | | | | | 295.720 | -923.747 | 342.000 | -3420.000 | 0.000 | 786.311 | 100.000 | 786.311 |
| 12 | Base | | | | | 328.875 | -980.884 | 342.000 | -3420.000 | 0.000 | 834.947 | 100.000 | 834.947 |

### SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STABILITY ANALYSIS)

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | > 100 | 4.335 | 7.586 | 7.048 | 7.519 | -18.4 | 4.2 | 0.4 | 50.513 | | |
| 2 | 45.000 | 23.386 | 2.044 | 8.228 | 2.447 | 6.266 | -418.5 | 204.7 | 279.1 | 64.822 | 13.2 | 0.000 |
| 3 | 40.500 | 8.168 | 1.431 | 8.015 | 1.036 | 3.860 | -724.0 | 505.9 | 1692.8 | 97.466 | 123.2 | 0.000 |
| 4 | 36.000 | 6.675 | 0.961 | 5.462 | 1.124 | 2.665 | -1063.3 | 1106.4 | 3532.6 | 87.942 | 412.3 | 0.000 |
| 5 | 31.500 | 5.225 | 0.784 | 4.523 | 1.088 | 2.271 | -1585.4 | 2022.6 | 7971.7 | 89.949 | 870.5 | 0.000 |
| 6 | 27.000 | 4.278 | 0.705 | 4.133 | 1.058 | 2.102 | -2290.2 | 3247.8 | 16064.1 | 92.726 | 1497.8 | 0.000 |
| 7 | 22.500 | 3.656 | 0.665 | 3.934 | 1.039 | 2.014 | -3177.8 | 4778.3 | 28841.5 | 94.825 | 2294.2 | 0.000 |
| 8 | 18.000 | 3.224 | 0.643 | 3.818 | 1.028 | 1.963 | -4248.1 | 6611.5 | 47321.1 | 96.264 | 3259.6 | 0.000 |
| 9 | 13.500 | 3.451 | 0.830 | 5.350 | 1.206 | 2.747 | -7258.9 | 8745.2 | 64334.5 | 81.757 | 2636.5 | 0.000 |
| 10 | 9.000 | 3.189 | 0.824 | 5.279 | 1.203 | 2.720 | -9216.2 | 11178.1 | 93351.7 | 81.914 | 3418.7 | 0.000 |
| 11 | 4.500 | 2.952 | 0.804 | 4.844 | 1.191 | 2.600 | -11169.3 | 13899.1 | 128571.4 | 82.362 | 4564.1 | 0.000 |
| 12 | Base | 2.728 | 0.765 | 4.039 | 1.166 | 2.364 | -12844.6 | 16796.2 | 169434.7 | 83.480 | 6447.1 | 0.000 |
| | **Required:** | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

Table B.5 CADAM Results for Data Set 2 (continued – 3).

## CADAM Results for Data Set 2

| | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

### SEISMIC #2 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation | Crack length | | Crack length | | Upstream | Downstream | tension | Compression | Upstream | Maximum | Maximum at l-axis | Downstream |
| | (m) | (%) | (m) | (%) | (m) | (kPa) | (kPa) | (kPa) | (kPa) | (kPa) | (kPa) | (% of joint) | (kPa) |
| 1 | 49.500 | | | | | -1.471 | -6.063 | 342.000 | -3420.000 | 0.000 | 25.789 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | 1093.708 | -1264.873 | 342.000 | -3420.000 | 0.000 | 569.858 | 50.000 | 0.000 |
| 3 | 40.500 | 100.000 | 4.926 | | | 3675.538 | -3942.942 | 342.000 | -3420.000 | 0.000 | -263.472 | 121.247 | 3356.309 |
| 4 | 36.000 | | | | | 2607.611 | -2824.441 | 342.000 | -3420.000 | 0.000 | -45.956 | 12.046 | 2404.219 |
| 5 | 31.500 | | | | | 2371.800 | -2593.559 | 342.000 | -3420.000 | 0.000 | -3.638 | 3.898 | 2207.688 |
| 6 | 27.000 | | | | | 2313.087 | -2556.427 | 342.000 | -3420.000 | 0.000 | 2176.080 | 100.000 | 2176.080 |
| 7 | 22.500 | | | | | 2301.517 | -2573.639 | 342.000 | -3420.000 | 0.000 | 2190.732 | 100.000 | 2190.732 |
| 8 | 18.000 | | | | | 2298.580 | -2603.248 | 342.000 | -3420.000 | 0.000 | 2215.935 | 100.000 | 2215.935 |
| 9 | 13.500 | | | | | 2165.838 | -2631.227 | 342.000 | -3420.000 | 0.000 | 2239.752 | 100.000 | 2239.752 |
| 10 | 9.000 | | | | | 2134.897 | -2654.117 | 342.000 | -3420.000 | 0.000 | 2259.236 | 100.000 | 2259.236 |
| 11 | 4.500 | | | | | 2099.417 | -2659.046 | 342.000 | -3420.000 | 0.000 | 2263.432 | 100.000 | 2263.432 |
| 12 | Base | | | | | 2065.161 | -2641.255 | 342.000 | -3420.000 | 0.000 | 2248.287 | 100.000 | 2248.287 |

### SEISMIC #2 COMBINATION - PEAK ACCELERATIONS (STABILITY ANALYSIS)

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation | Peak | Residual | Toward U/S | Toward D/S | | | | | | | |
| | (m) | | | | | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | 54.464 | 0.219 | 5.813 | 2.758 | 5.000 | -17.0 | 77.4 | 7.7 | 60.159 | | |
| 2 | 45.000 | 1.636 | 0.225 | 20.868 | 0.265 | 4.413 | -385.1 | 1709.6 | 3980.1 | 279.660 | 13.2 | 0.000 |
| 3 | 40.500 | 0.271 | 0.190 | 27.467 | 0.159 | 3.067 | -658.6 | 3474.6 | 15403.1 | 524.841 | 123.2 | 0.000 |
| 4 | 36.000 | 0.948 | 0.161 | 19.049 | 0.249 | 2.261 | -949.3 | 5903.7 | 34705.9 | 467.535 | 412.3 | 0.000 |
| 5 | 31.500 | 0.915 | 0.157 | 13.769 | 0.304 | 1.971 | -1395.6 | 8870.9 | 65551.9 | 423.180 | 870.5 | 0.000 |
| 6 | 27.000 | 0.899 | 0.165 | 10.976 | 0.348 | 1.842 | -1997.5 | 12099.1 | 109369.4 | 383.520 | 1497.8 | 0.000 |
| 7 | 22.500 | 0.901 | 0.179 | 9.286 | 0.388 | 1.775 | -2754.9 | 15405.7 | 166553.1 | 348.590 | 2294.2 | 0.000 |
| 8 | 18.000 | 0.918 | 0.196 | 8.138 | 0.425 | 1.735 | -3667.9 | 18679.1 | 236820.5 | 318.152 | 3259.6 | 0.000 |
| 9 | 13.500 | 1.076 | 0.297 | 9.953 | 0.495 | 2.320 | -6494.2 | 21874.3 | 311364.0 | 221.794 | 2636.5 | 0.000 |
| 10 | 9.000 | 1.125 | 0.329 | 9.075 | 0.536 | 2.301 | -8239.8 | 25009.5 | 402024.0 | 203.724 | 3418.7 | 0.000 |
| 11 | 4.500 | 1.163 | 0.354 | 7.844 | 0.573 | 2.214 | -9952.9 | 28127.2 | 501696.7 | 191.715 | 4564.1 | 0.000 |
| 12 | Base | 1.181 | 0.364 | 6.250 | 0.604 | 2.040 | -11349.0 | 31149.2 | 608837.6 | 186.159 | 6447.1 | 0.000 |
| | **Required:** | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

### SEISMIC #2 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation | Crack length | | Crack length | | Upstream | Downstream | tension | Compression | Upstream | Maximum | Maximum at l-axis | Downstream |
| | (m) | (%) | (m) | (%) | (m) | (kPa) | (kPa) | (kPa) | (kPa) | (kPa) | (kPa) | (% of joint) | (kPa) |
| 1 | 49.500 | | | | | -2.552 | -5.613 | 342.000 | -3420.000 | 0.000 | 17.193 | 50.000 | 0.000 |
| 2 | 45.000 | | | | | 697.571 | -883.564 | 342.000 | -3420.000 | 0.000 | 391.213 | 50.000 | 0.000 |
| 3 | 40.500 | 100.000 | 4.926 | | | 2441.948 | -2735.938 | 342.000 | -3420.000 | 0.000 | -186.641 | 121.408 | 2328.884 |
| 4 | 36.000 | | | | | 1696.462 | -1939.339 | 342.000 | -3420.000 | 0.000 | -26.890 | 11.237 | 1650.804 |
| 5 | 31.500 | | | | | 1539.335 | -1791.250 | 342.000 | -3420.000 | 0.000 | -0.559 | 1.878 | 1524.747 |
| 6 | 27.000 | | | | | 1503.166 | -1782.166 | 342.000 | -3420.000 | 0.000 | 1517.015 | 100.000 | 1517.015 |
| 7 | 22.500 | | | | | 1498.258 | -1812.148 | 342.000 | -3420.000 | 0.000 | 1542.535 | 100.000 | 1542.535 |
| 8 | 18.000 | | | | | 1498.582 | -1851.442 | 342.000 | -3420.000 | 0.000 | 1575.983 | 100.000 | 1575.983 |
| 9 | 13.500 | | | | | 1369.909 | -1890.098 | 342.000 | -3420.000 | 0.000 | 1608.888 | 100.000 | 1608.888 |
| 10 | 9.000 | | | | | 1344.823 | -1925.570 | 342.000 | -3420.000 | 0.000 | 1639.083 | 100.000 | 1639.083 |
| 11 | 4.500 | | | | | 1316.576 | -1944.603 | 342.000 | -3420.000 | 0.000 | 1655.284 | 100.000 | 1655.284 |
| 12 | Base | | | | | 1290.371 | -1942.380 | 342.000 | -3420.000 | 0.000 | 1653.392 | 100.000 | 1653.392 |

Table B.5 CADAM Results for Data Set 2 (continued – 4).

| CADAM Results for Data Set 2 | |
|---|---|
| Project: Risk Analysis | Project engineer: M. Resat Beser |
| Dam: Porsuk | Analysis performed by: M. Resat Beser |
| Owner: METU Civil Engineering Department | Date: 26 Ağustos 04 |
| Dam location: Eskişehir | Units: Metric |

**SEISMIC #2 COMBINATION - SUSTAINED ACCELERATIONS (STABILITY ANALYSIS)**

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | 81.736 | 0.356 | 8.334 | 4.143 | 7.519 | -18.4 | 51.6 | 5.2 | 56.249 | | |
| 2 | 45.000 | 2.504 | 0.357 | 22.375 | 0.394 | 6.266 | -418.5 | 1173.6 | 2668.2 | 191.684 | 13.2 | 0.000 |
| 3 | 40.500 | 0.432 | 0.303 | 25.724 | 0.231 | 3.860 | -724.0 | 2392.3 | 10468.7 | 343.541 | 123.2 | 0.000 |
| 4 | 36.000 | 1.424 | 0.258 | 16.196 | 0.353 | 2.665 | -1063.3 | 4119.8 | 23229.5 | 299.495 | 412.3 | 0.000 |
| 5 | 31.500 | 1.366 | 0.253 | 11.589 | 0.422 | 2.271 | -1585.4 | 6272.3 | 43969.9 | 270.351 | 870.5 | 0.000 |
| 6 | 27.000 | 1.334 | 0.264 | 9.319 | 0.475 | 2.102 | -2290.2 | 8665.0 | 73788.6 | 246.256 | 1497.8 | 0.000 |
| 7 | 22.500 | 1.329 | 0.284 | 7.986 | 0.521 | 2.014 | -3177.8 | 11176.3 | 113095.5 | 225.773 | 2294.2 | 0.000 |
| 8 | 18.000 | 1.344 | 0.309 | 7.096 | 0.563 | 1.963 | -4248.1 | 13731.6 | 161848.7 | 208.232 | 3259.6 | 0.000 |
| 9 | 13.500 | 1.560 | 0.445 | 9.218 | 0.663 | 2.747 | -7258.9 | 16300.9 | 211597.9 | 154.449 | 2636.5 | 0.000 |
| 10 | 9.000 | 1.617 | 0.488 | 8.515 | 0.710 | 2.720 | -9216.2 | 18896.5 | 274540.1 | 143.856 | 3418.7 | 0.000 |
| 11 | 4.500 | 1.657 | 0.518 | 7.374 | 0.750 | 2.600 | -11169.3 | 21543.0 | 343834.2 | 136.546 | 4564.1 | 0.000 |
| 12 | Base | 1.673 | 0.532 | 5.828 | 0.781 | 2.364 | -12844.6 | 24145.7 | 418199.5 | 132.636 | 6447.1 | 0.000 |
| | **Required:** | 1.300 | 1.000 | 1.100 | 1.100 | 1.100 | | | | | | |

**POST-SEISMIC COMBINATION (STRESS ANALYSIS)**

| Joint | | Cracking | | | | Stresses | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Upstream | | Downstream | | Normal stresses | | allowable stresses | | Shear | | | |
| ID | Upstream elevation (m) | Crack length (%) | (m) | Crack length (%) | (m) | Upstream (kPa) | Downstream (kPa) | tension (kPa) | Compression (kPa) | Upstream (kPa) | Maximum (kPa) | Maximum at l-axis (% of joint) | Downstream (kPa) |
| 1 | 49.500 | | | | | -4.709 | -4.709 | 189.000 | -2490.000 | | | | |
| 2 | 45.000 | | | | | -94.592 | -120.836 | 189.000 | -2490.000 | 0.000 | 33.922 | 50.000 | 0.000 |
| 3 | 40.500 | | | | | -24.982 | -321.781 | 189.000 | -2490.000 | 0.000 | -33.618 | 24.848 | 273.907 |
| 4 | 36.000 | | | | | -125.429 | -169.153 | 189.000 | -2490.000 | 0.000 | 143.987 | 100.000 | 143.987 |
| 5 | 31.500 | | | | | -125.105 | -186.673 | 189.000 | -2490.000 | 0.000 | 158.900 | 100.000 | 158.900 |
| 6 | 27.000 | | | | | -116.103 | -233.687 | 189.000 | -2490.000 | 0.000 | 198.919 | 100.000 | 198.919 |
| 7 | 22.500 | | | | | -107.600 | -289.205 | 189.000 | -2490.000 | 0.000 | 246.177 | 100.000 | 246.177 |
| 8 | 18.000 | | | | | -100.657 | -347.867 | 189.000 | -2490.000 | 0.000 | 296.111 | 100.000 | 296.111 |
| 9 | 13.500 | | | | | -221.099 | -407.873 | 189.000 | -2490.000 | 0.000 | 347.189 | 100.000 | 347.189 |
| 10 | 9.000 | | | | | -234.376 | -468.506 | 189.000 | -2490.000 | 0.000 | 398.802 | 100.000 | 398.802 |
| 11 | 4.500 | | | | | -248.058 | -515.744 | 189.000 | -2490.000 | 0.000 | 439.011 | 100.000 | 439.011 |
| 12 | Base | | | | | -258.063 | -544.649 | 189.000 | -2490.000 | 0.000 | 463.616 | 100.000 | 463.616 |

**POST-SEISMIC COMBINATION (STABILITY ANALYSIS)**

| Joint | | Safety factors | | | | | Resultants | | | | Uplift | Rock |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sliding | | Overturning | | Uplifting | Normal | Shear | Moment | Position | Final Force | Passive wedge resistance |
| ID | Upstream elevation (m) | Peak | Residual | Toward U/S | Toward D/S | | (kN) | (kN) | (kN·m) | (% of joint) | (kN) | (kN) |
| 1 | 49.500 | > 100 | > 100 | > 100 | > 100 | > 100 | -21.2 | 0.0 | 0.0 | 50.000 | | |
| 2 | 45.000 | 47.970 | 4.763 | 58.129 | 15.123 | 37.600 | -484.7 | 101.8 | 44.3 | 52.030 | 13.2 | 0.000 |
| 3 | 40.500 | 25.509 | 3.753 | 14.363 | 2.360 | 7.931 | -854.0 | 227.6 | 600.1 | 64.265 | 123.2 | 0.000 |
| 4 | 36.000 | 18.103 | 2.336 | 5.924 | 2.121 | 4.128 | -1289.7 | 552.0 | 279.4 | 52.474 | 412.3 | 0.000 |
| 5 | 31.500 | 13.505 | 1.825 | 4.604 | 1.869 | 3.254 | -1962.1 | 1075.2 | 812.8 | 53.291 | 870.5 | 0.000 |
| 6 | 27.000 | 10.788 | 1.598 | 4.198 | 1.728 | 2.917 | -2871.3 | 1796.9 | 2640.9 | 55.603 | 1497.8 | 0.000 |
| 7 | 22.500 | 9.048 | 1.478 | 4.027 | 1.645 | 2.751 | -4017.2 | 2717.4 | 6204.3 | 57.628 | 2294.2 | 0.000 |
| 8 | 18.000 | 7.853 | 1.408 | 3.941 | 1.593 | 2.657 | -5399.8 | 3836.4 | 11943.4 | 59.186 | 3259.6 | 0.000 |
| 9 | 13.500 | 7.473 | 1.703 | 6.488 | 2.038 | 4.329 | -8776.8 | 5154.2 | 12123.0 | 54.949 | 2636.5 | 0.000 |
| 10 | 9.000 | 6.818 | 1.672 | 6.438 | 2.008 | 4.263 | -11154.4 | 6670.5 | 19654.5 | 55.552 | 3418.7 | 0.000 |
| 11 | 4.500 | 6.271 | 1.622 | 5.715 | 1.958 | 3.976 | -13584.0 | 8374.5 | 28222.7 | 55.841 | 4564.1 | 0.000 |
| 12 | Base | 5.846 | 1.560 | 4.477 | 1.875 | 3.453 | -15813.4 | 10138.5 | 37073.7 | 55.950 | 6447.1 | 0.000 |
| | **Required:** | 2.000 | 1.100 | 1.100 | 1.100 | 1.100 | | | | | | |

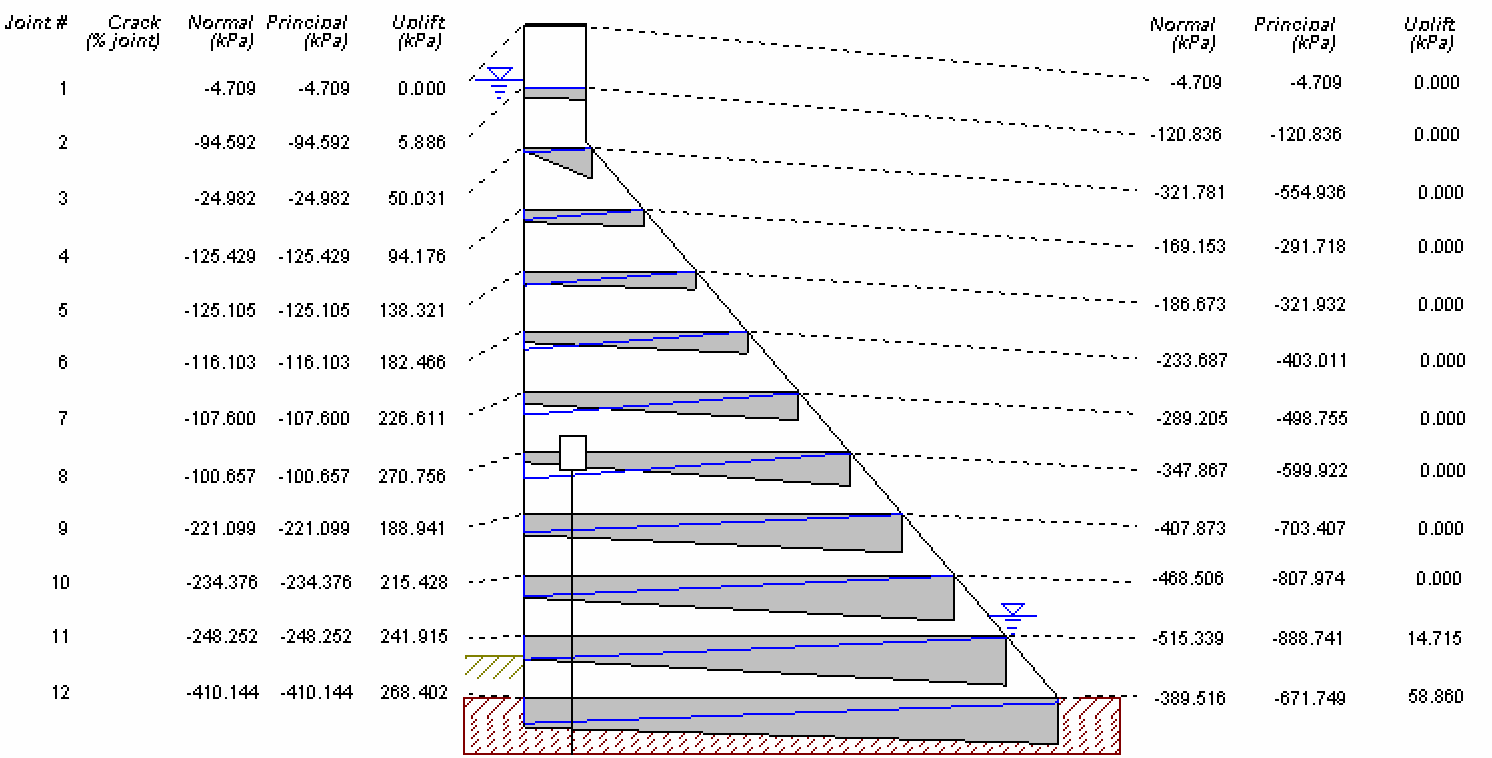



Figure B.1 CADAM Stability Drawing for Usual Combination (Effective Stress Analysis) (Leclerc et al., 2004).

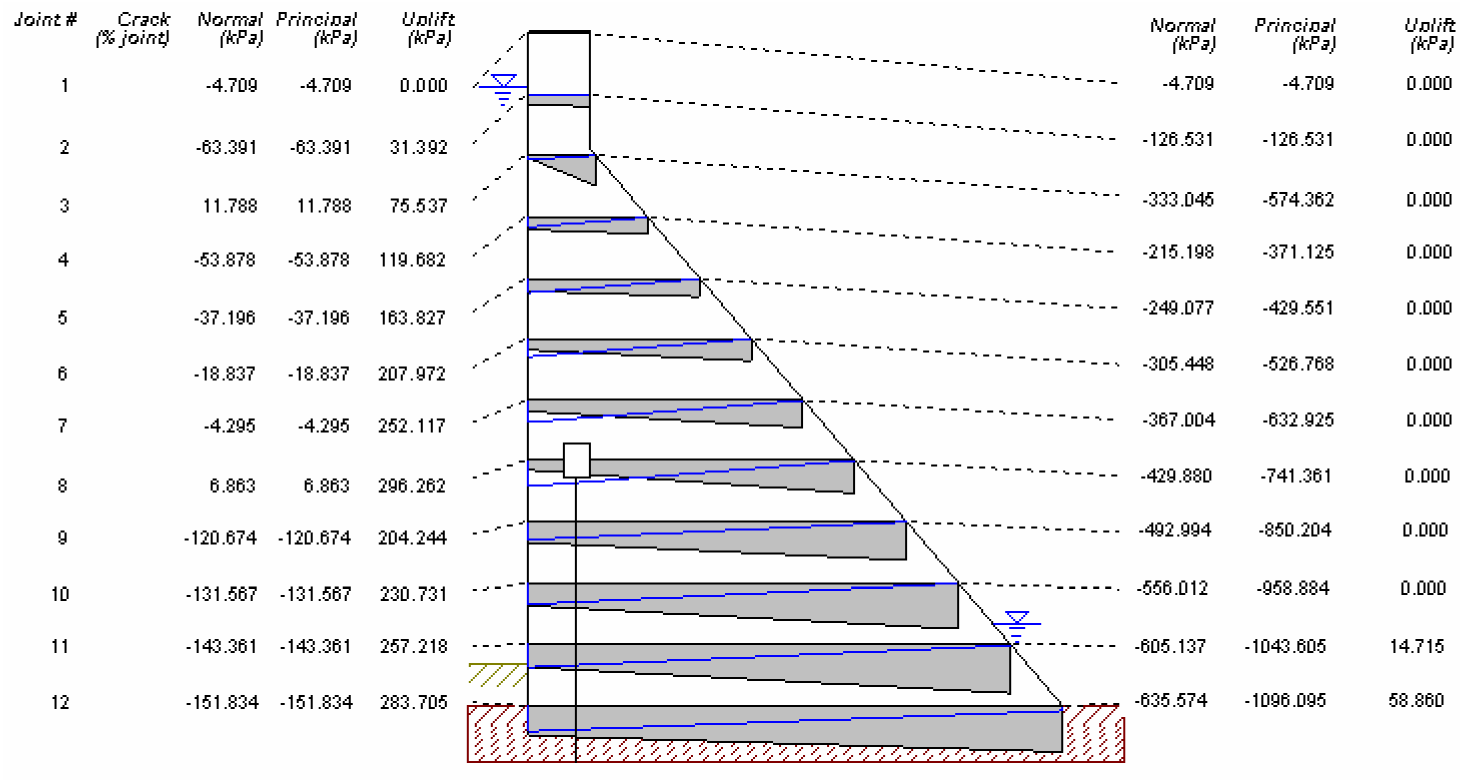



Figure B.2  CADAM Stability Drawing for Flood Combination (Effective Stress Analysis) (Leclerc et al., 2004).

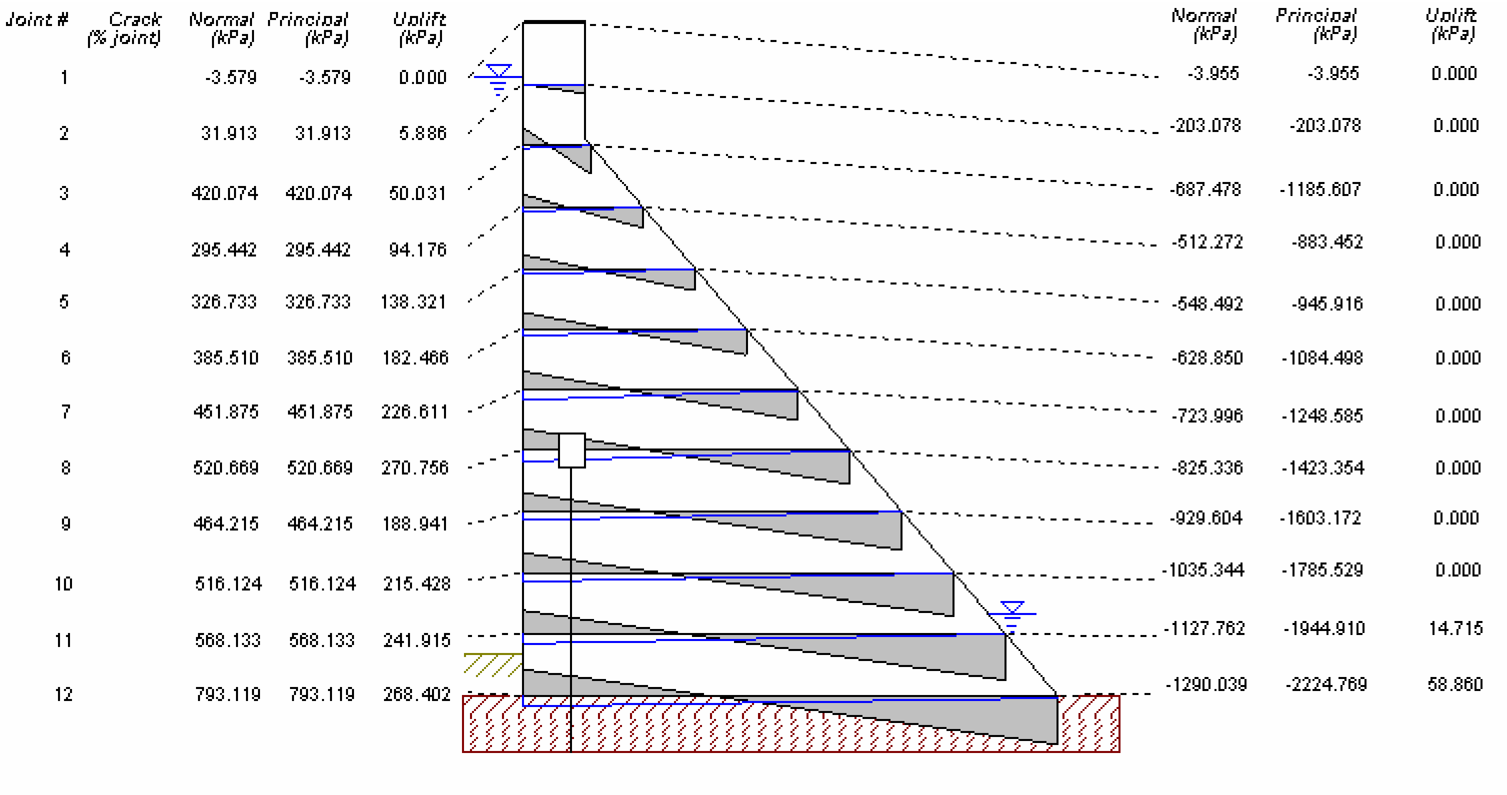



Figure B.3 CADAM Stability Drawing for Seismic-1 Combination – Peak Accelerations (Stress Analysis) (Effective Stress Analysis) (Leclerc et al., 2004).

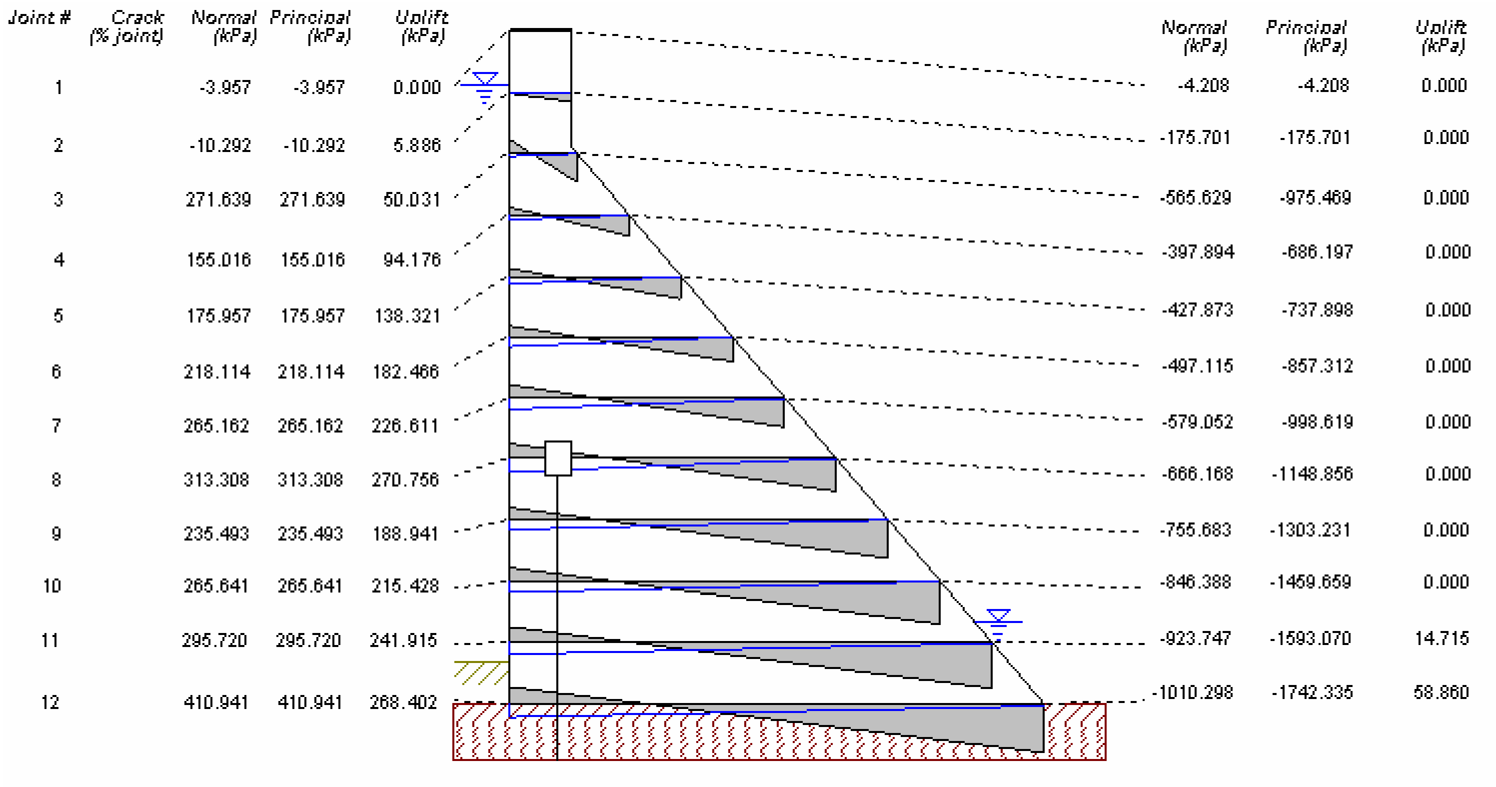



Figure B.4 CADAM Stability Drawing for Seismic-1 Combination – Sustained Accelerations (Stability Analysis) (Effective Stress Analysis) (Leclerc et al., 2004).

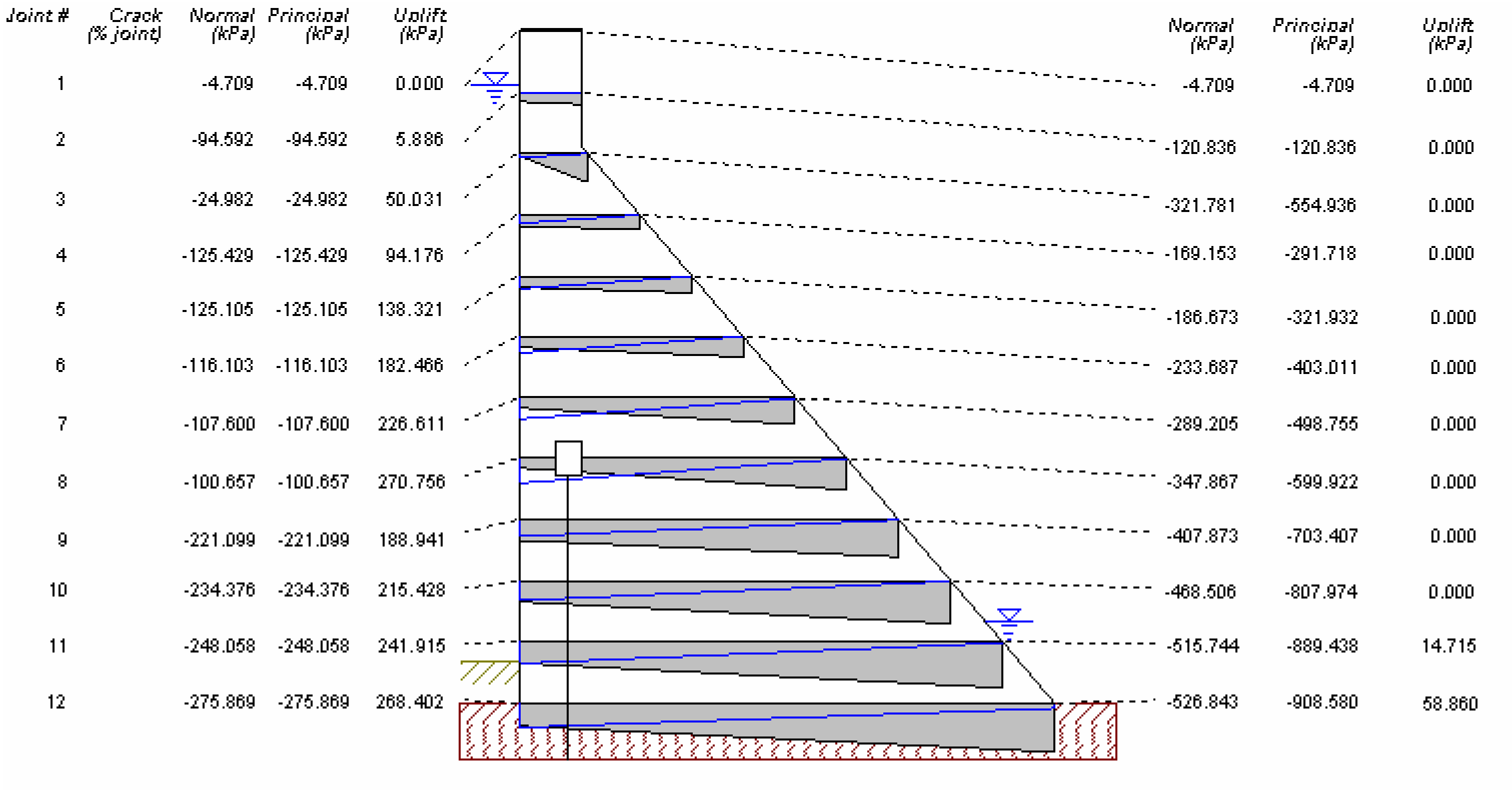



Figure B.5  CADAM Stability Drawing for Post – Seismic Combination (Effective Stress Analysis) (Leclerc et al., 2004).

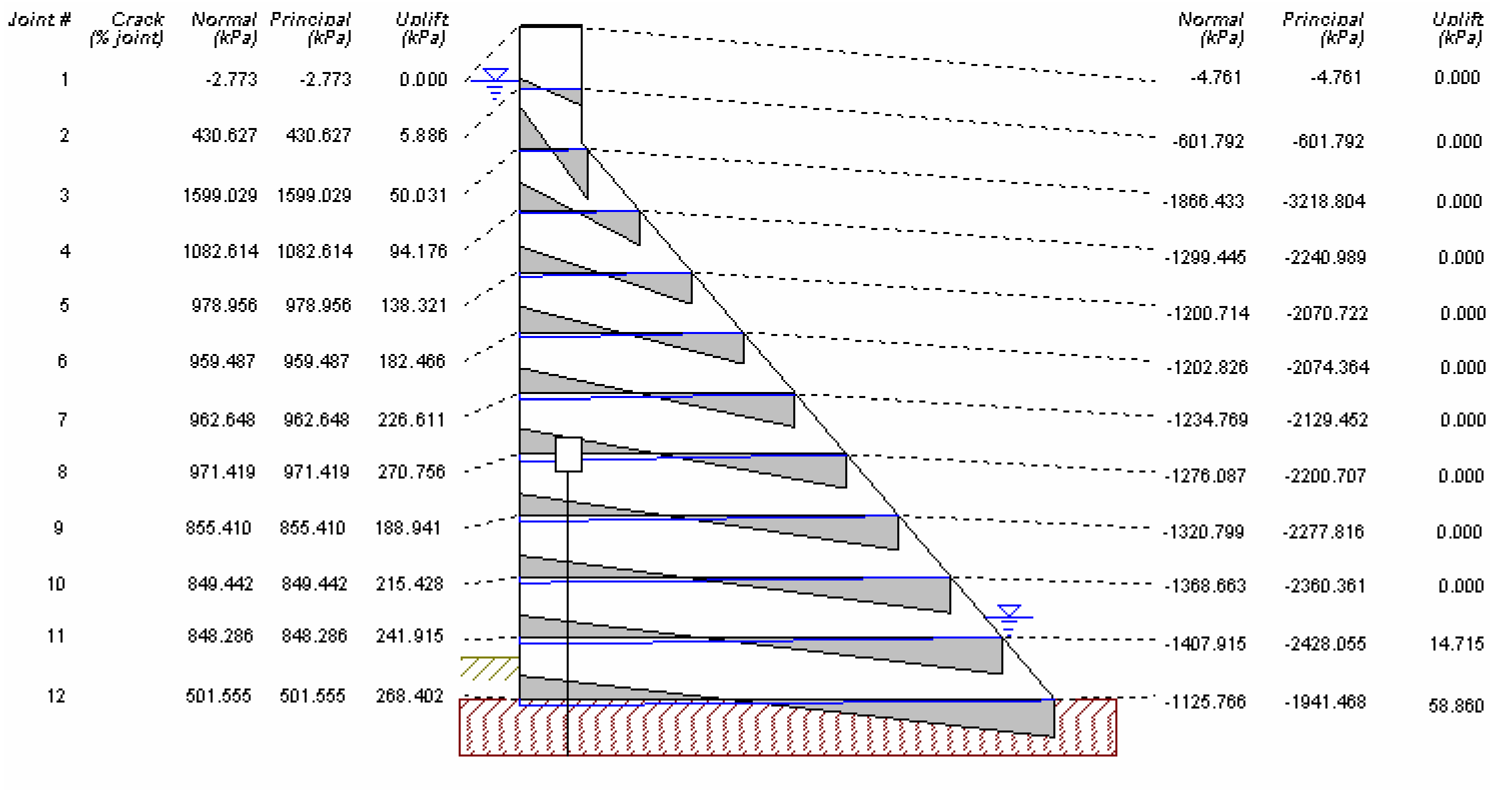



Figure B.6 CADAM Stability Drawing for Seismic-2 Combination – Peak Accelerations (Stress Analysis) for Data Set 1 (Effective Stress Analysis) (Leclerc et al., 2004).

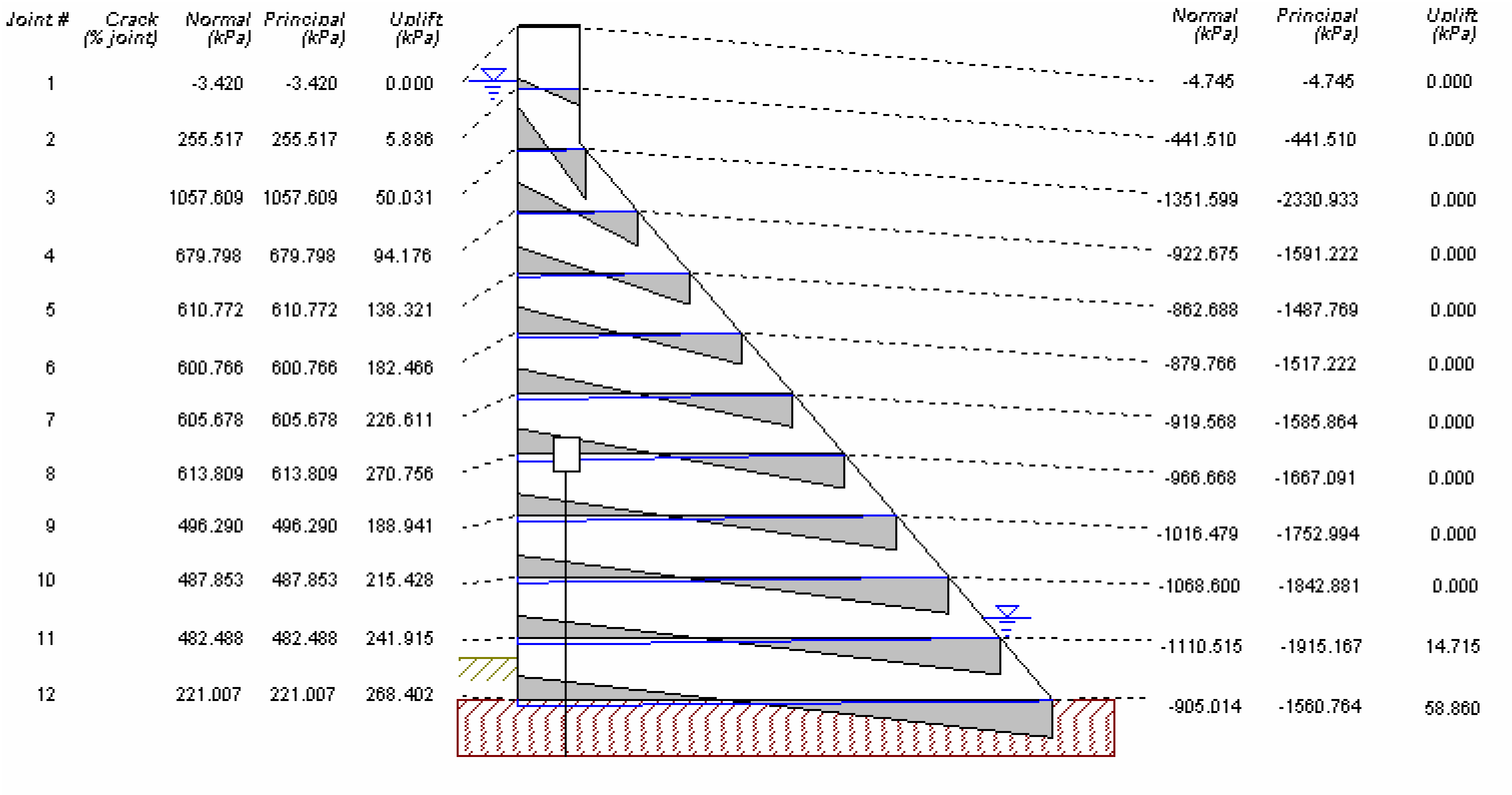



Figure B.7 CADAM Stability Drawing for Seismic-2 Combination – Sustained Accelerations (Stability Analysis) for Data Set 1 (Effective Stress Analysis) (Leclerc et al., 2004).

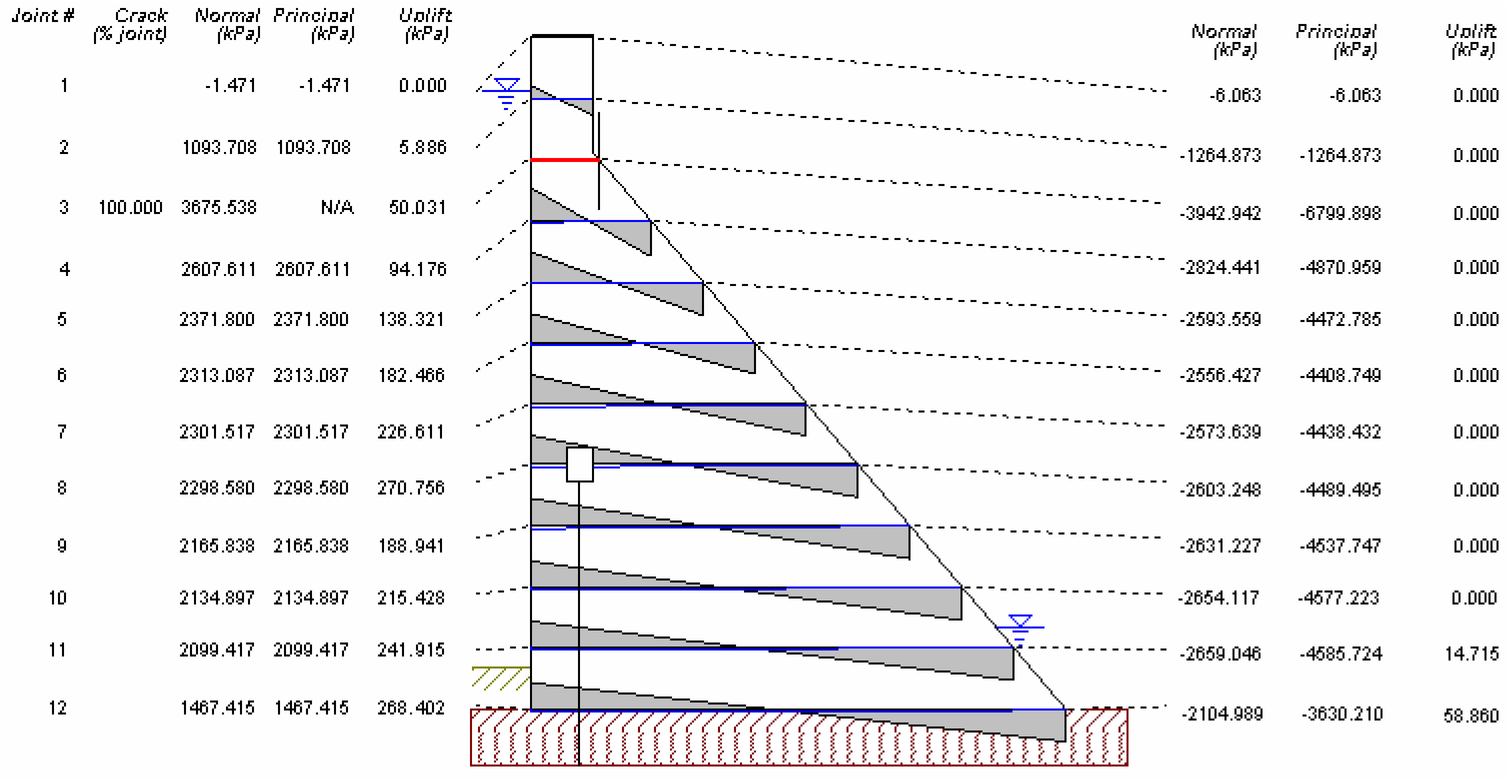



Figure B.8 CADAM Stability Drawing for Seismic-2 Combination – Peak Accelerations (Stress Analysis) for Data Set 2 (Effective Stress Analysis) (Leclerc et al., 2004).

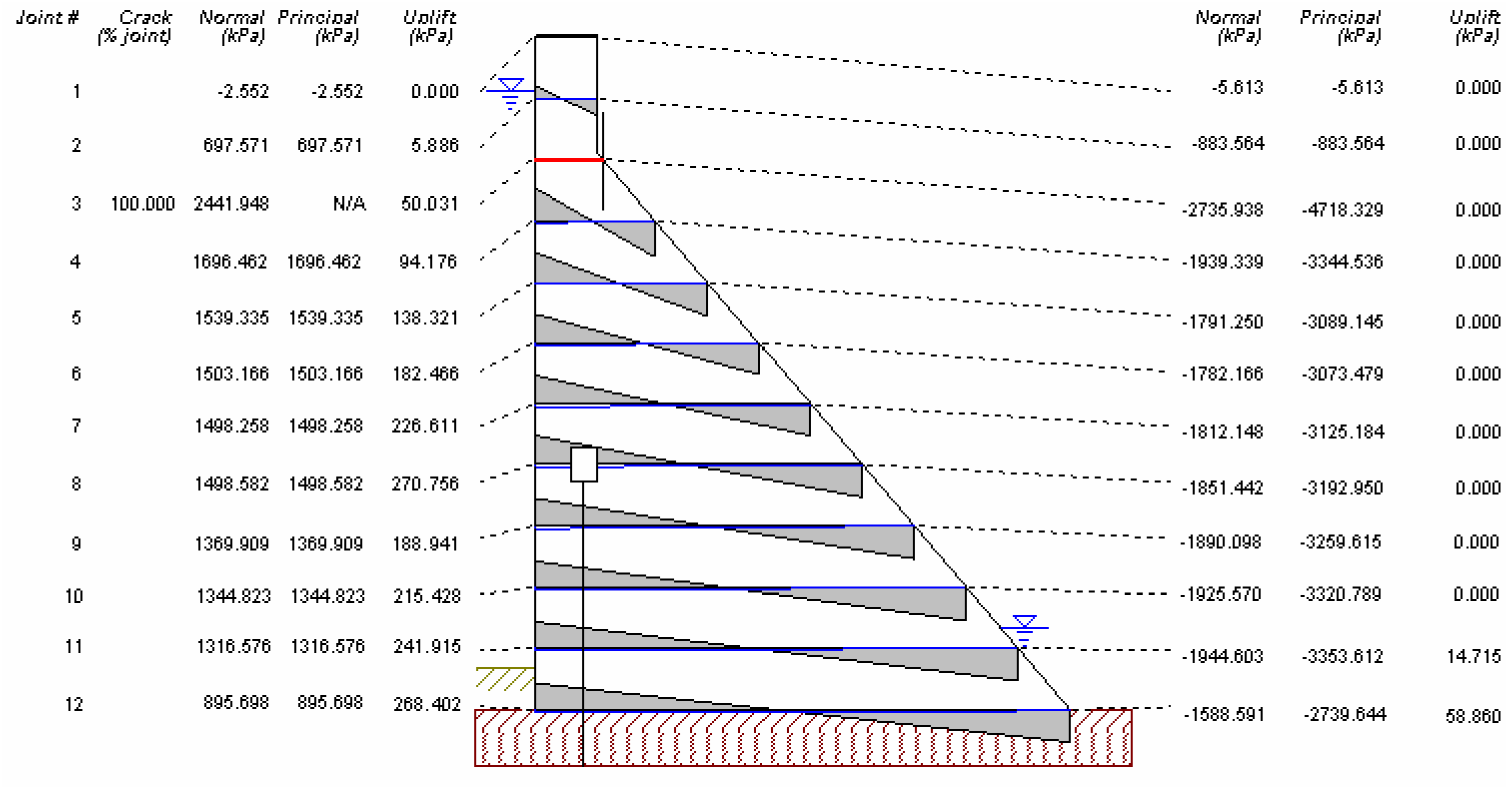



Figure B.9 CADAM Stability Drawing for Seismic-2 Combination – Sustained Accelerations (Stability Analysis) for Data Set 2 (Effective Stress Analysis) (Leclerc et al., 2004).

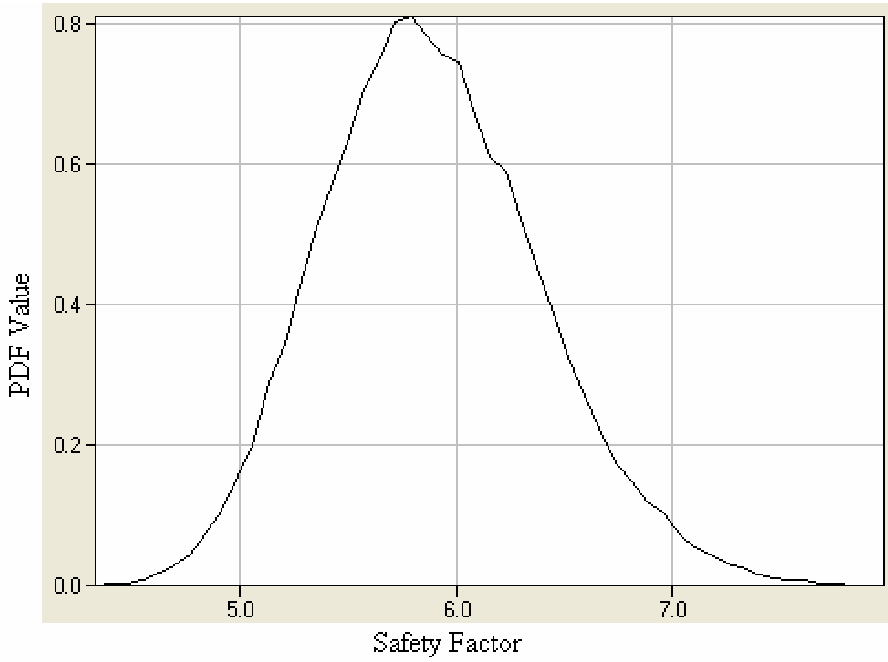



Figure B.10  PDF of Peak SSF in Usual Loading of Data Set 1.

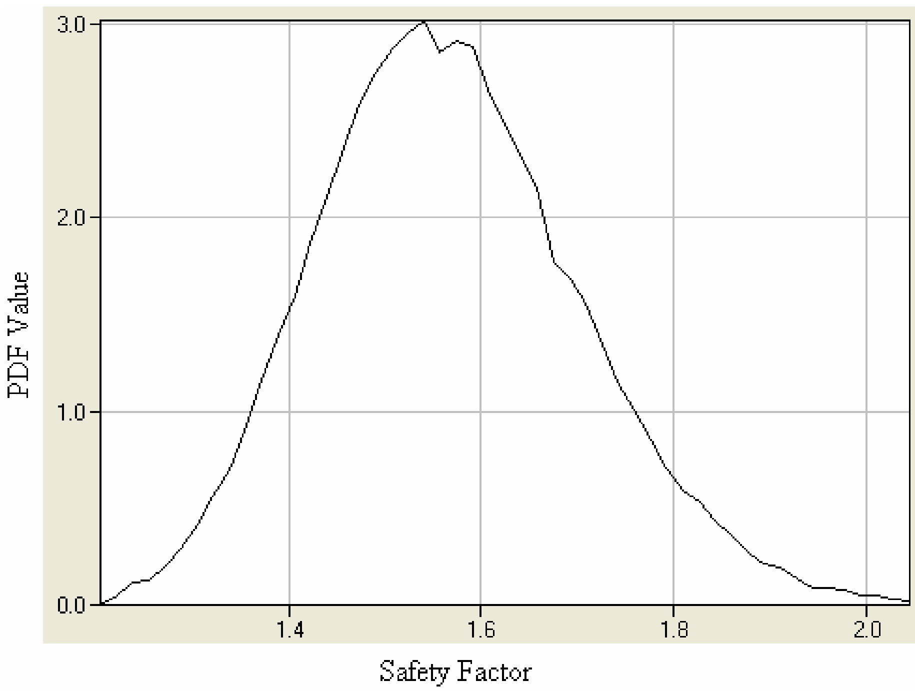



Figure B.11  PDF of Residual SSF in Usual Loading of Data Set 1.

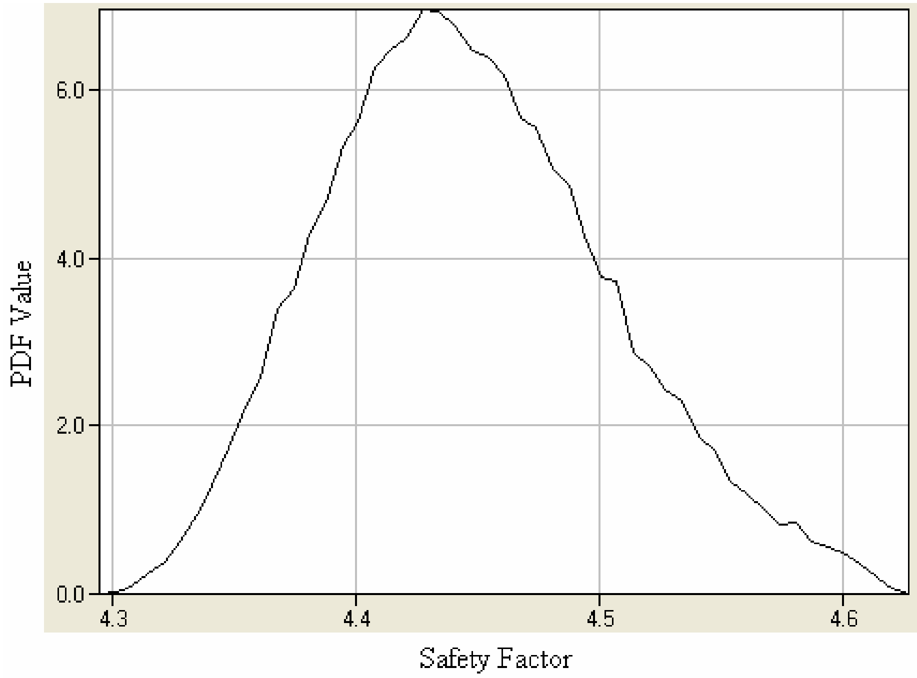



Figure B.12 PDF of OSF towards Upstream in Usual Loading of Data Set 1.


Figure B.13 PDF of OSF towards Downstream in Usual Loading of Data Set 1.


Figure B.14 PDF of USF in Usual Loading of Data Set 1.