A STUDY ON THE RELIABILITY – BASED SAFETY ANALYSIS OF HARDFILL DAMS

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ABSTRACT

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Dams are important large structures providing vital benefits to human life. These strategic structures are necessary in order to supply water and energy and to control floods. Moreover, dams have important roles in regional development and national economy. Thus, the design of dams deserves rigorous studies. Deterministic approach may be acceptable for design of dams and may satisfy safety requirements if large safety factors are used. However, such an approach will not be cost-effective in economic terms. High safety factors utilized in deterministic approaches necessitates large dimensions. One remedy for this overestimation is integrating statistical information and techniques, such as Monte-Carlo simulations into the analysis and design of dams. Probabilistic approaches may result in more economical and reasonable designs. CADAM is a software program which allows the user to analyze dams using Monte-Carlo simulation technique. Uncertainties associated with tensile strength, peak cohesion, peak friction coefficient, normal upstream reservoir elevation, drain efficiency and horizontal peak ground acceleration are incorporated into stability and stress analysis using Monte-Carlo simulations. In this thesis, utilization of CADAM software is demonstrated on a case study. Cindere dam is evaluated in terms of structural safety.

Keywords: Hardfill Dam, Dam Safety, CADAM, Monte Carlo Simulations

ÖZ

KATI DOLGU BARAJLARDA GÜVENİLİRLİK ESASLI EMNİYET ANALİZİ ÜZERİNE BİR ÇALIŞMA

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Barajlar, insan hayatı üzerinde önemli yararları bulunan büyük yapılardır. Bu stratejik yapılar, su ve enerji ihtiyacını karşılamalarının yanı sıra taşkın kontrolünde de yardımcı olmaktadırlar. Bunlarla birlikte barajlar, ülkelerin bölgesel kalkınması ve ülke ekonomisin gelişmesinde de önemli rol oynamaktadırlar. Bu nedenle, baraj tasarımı özenli çalışmaları gerektirmektedir. Büyük güvenlik katsayıları göz önüne alındığında, deterministik yaklaşım barajların tasarımı için yeterli olabilir ve bu barajların güvenlik gereksinimlerini sağlayabilir. Ancak, bu yaklaşım ekonomik yönden dikkate alındığında fayda-maliyet dengesini sağlayamamaktadır. Deterministik yaklaşımda kullanılan yüksek emniyet faktörleri, büyük boyutları gerekli kılmaktadır. Bu aşırı boyutlandırmaya bir çözüm, istatiksel verileri ve

Monte-Carlo benzeşimi gibi teknikleri kullanarak barajın tasarım ve analiz çalışmalarını gerçekleştirmektir. İstatiksel yaklaşımlar daha ekonomik ve güvenli yapılar tasarlanmasını sağlayabilir. CADAM yazılımı, Monte-Carlo benzeşim tekniğini kullanarak barajların analiz etmesini sağlayan bir programıdır. Çekme dayanımı, pik kohezyon, pik sürtünme katsayısı, nornal hazne seviyesi, dren verimi ve yatay maksimum zemin ivmesi parametrelerindeki belirsizlikler Monte-Carlo benzeşim tekniği kullanılarak stabilite ve gerilme analizlerine dahil edilebilir. Bu tez çalışmasında CADAM programının kullanımı örnek bir uygulama ile gösterilmiş ve bu kapsamda Cindere Barajı yapısal güvenilirlik bakımından değerlendirilmiştir.

Anahtar Kelimeler: Katı Dolgu Baraj, Baraj Güvenliği, CADAM, Monte Carlo Benzeşimleri

To my family

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α = Significance level

α = Angle with respect to the horizontal of sliding plane

α = Wave reflection coefficient

a_1= Uniformly distributed random variable

γ = Specific weight

γ_e = Specific weight

γ_s = Submerged specific weight of soil

δ = Coefficient of variation

δ_f = Coefficient of variation of failure probability

δ_f = Coefficient of variation of failure probability

δ_h = Horizontal displacement

δ_v = Vertical displacement

η_f = Constant hysteretic damping coefficient of the foundation rock

θ = Angle of reponse

θ = Angle of the face with respect to the vertical

λ = Mean value for Log-Normal Distribution

μ = Mean value

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

ξ_f = Added damping ratio due to dam-foundation interaction

ξ_r = Added damping ratio due to dam-water interaction and reservoir bottom

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

ξ_1 = Viscous damping ratio for the dam on rigid foundation rock with empty reservoir

ξ_1 = The dam damping on rigid foundation without reservoir interaction

ξ^2 = Variance of Log-Normal Distribution

ρ = Coefficient of variation of failure probability

ρ_w = Volumetric mass of water

ρ = A drain reduction factor

σ = Standard deviation

σ^2 = Variance

σ = Vertical normal base pressure

σ' = Effective normal stress

σ_1 = Maximum principal stresses associated with fundamental vibration mode

σ_n = Normal compressive stress

σ_n^* = Minimal compressive stress

σ_{sc} = Maximum principal stresses associated with higher vibration modes

σ_{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

σ_{y1} = Normal bending stresses associated with fundamental vibration mode

σ_{y1} = Normal bending stresses associated with fundamental vibration mode

σ_{zu} = Minimum allowable compressive (normal) stress at the upstream face

ϕ = Area of cumulative Standard Normal Distribution for a specified variable

ϕ = The standard normal variable probability density ordinate

ϕ = Friction angle (peak value or residual value)

ϕ = uplift reduction factor

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

Δx = Width of the interval

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

A= Area of the base that normal pressure takes place

$A(T)$ = Spectral acceleration coefficient

A_2= Area along the rock wedge failure plane

A_c= Area in compression

accv= Vertical acceleration of the rock

AFOSM= Advanced first order second moment

ANCOLD= Australian Commision on Large Dams

A_o= Effective horizontal ground acceleration coefficient

ASCE= American Society of Civil Engineers

B= Base width of the dam

c= cohesion (apparent or real)

C= Confidence interval

C= Constant

c= Crest

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

c and d= Limit values of z

$c_{1s},\ldots,c_{ks}$= The respective load effects in different failure modes

Ca= Cohesion

C_c= A correction factor to account water compressibility

CDF= Cumulative Distribution Function

CDSA= Canadian Dam Safety Association

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

c.o.v= Covariance

CSA= Canadian Standards Association

D= Dead load

D= Downstream

D_f= Failure region

DSI= General Directorate of State Hydraulic Works

e= Eccentricity

E_s= Young's modulus

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

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

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

f_c= Compressive strength of concrete

$f_{r,s}(r,s)$= Joint density function

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

F= Applied force

F= Flood level

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

FD= Floating debris

FEMA= Federal Emergency Management Agency

FOSM= First Order Second Moment

FREQ= Frequency

f_t= Tensile strength of the material

g= Acceleration of gravity

$gp(y,\tilde{T}_r)$ = Hydrodynamic pressure term

h = Horizontal

h = Total depth of reservoir

h_1= Upstream normal water level

h_2= Downstream normal water level

H= Depth of the impounded water

H= Horizontal hydrostatic force per unit width

H_1= Reservoir pressure head on the upstream face

H_2= Reservoir 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

HAS= Horizontal spectral acceleration

I = Building importance factor

I = Ice load

I = Moment of inertia

ICOLD= International Committee on Large Dams

K= Seismic coefficient

K_a= Active earth pressure coefficient according to Rankine theory

K_θ= Correction factor for the sloping dam faces with angle θ from the vertical

L= Horizontal length from upstream to downstream face

L_c= Crack length

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

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

m= upstream slope component

$\tilde{M}_1$ = Generalized mass

M= Masses

M= Sum of moments about the base centerline

MDE= Maximum Design Earthquake

n= Normal water level

n= Negative

N= Number of total simulation cycles

N_u= Number of simulation cycles where the failure occurs

P= Post-tension

$p(x)$= Probability of failure

p= positive

p_1= Hydrodynamic pressure associated with fundamental vibration mode

PDF= Probability Density Function

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

P_f=Pr (Failure)= Probability of failure

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

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

q= Dynamic

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

Q_h= Horizontal dam inertia

Q_v= Vertical dam inertia

r_{max}= Total value of response quantity

R= Resistance (capacity)

R_d= Dynamic response

R_r= Period ratio

R_w= Period ratio

s = Higher mode

s= Safety factor

S_h= Force due to sediment accumulation

S= Silt

S= Load (demand)

S(T)= Spectrum coefficient

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

SSF= Sliding safety factor

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

$T_1^r = 4H / C$

$\tilde{T}_1$ = 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

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

U= Uplift

U= Uplift force resultant normal to the inclined joint

U= Uplift pressure force resultant

u= Upstream

Un= Uplift force per unit width

USACE= US Army Corps of Engineers

USBR= United States Bureau of Reclamation

v= Vertical

V= Vertical hydrostatic force per unit width

VPGA= Vertical peak ground acceleration

W= Saturated weight of rock wedge

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

X_d= Distance to the drain from the upstream face

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

y= Distance below reservoir surface

z= A continuous random variable

z= Standard normal variate

Z_1= Class of the site

CHAPTER 1

INTRODUCTION

Many people claim that using deterministic approaches guarantees zero risk to the public while risk-based design means accepting failure and loss of life (Johnson, 2000). In deterministic approach, even if safety factors greater than unity are used, the safety of the dam is not guaranteed. There may be high failure probability of the dam. Moreover, high safety factors used in deterministic approaches may lead to high project costs. On the other hand, risk-based approaches are believed to require highly complex and time consuming analysis. However, probabilistic approaches for dam safety allows better understanding of associated risks by quantifying the uncertainties accurately and results in more reliable designs. That is why risk-based approaches are more realistic than deterministic approaches.

In order to understand the risk-based approach, risk and risk analysis should be defined first. Risk is the measure of probability and severity of an adverse effect to life, health, property or the environment (ICOLD, 1998). In the general case, risk is estimated by the combined impact of all triplets of scenario, probability of occurence and the associated consequence (ANCOLD, 2003).

The risk may be total risk from all causes, or specific risks from individual random events, such as floods, earthquakes, or other events, e.g. piping of embankment dams or misoperation of spillway gates. Human error pervades many aspects of risk, contributing to the probability of failure in some cases and magnifying the consequences in others. Consequences may be expressed in terms of life safety, a primary consideration in dam risk assessment or in terms of socio-economic losses, bcorporate financial loss or environmental damages. It is wise to maintain life safety distinct and separate from other consequences (Stewart, 2008).

Risk analysis is the first step of dam safety risk management. It involves hazard identification and definition, identification of failure modes and risk estimation in light of the failure probabilities and consequences. This step is the basis of risk evaluation, risk treatment and risk reduction, and systematic application of these steps is named as risk management. As a complete definition, risk management is the systematic application of management policies, procedures and practices to the task of identifying, analyzing, assesing, treating and monitoring risk (ICOLD, 1999).

In this thesis, a risk-based design approach is implemented for a hardfill dam and the results are evaluated with respect to various guidelines of different organizations. The risk-based analysis is carried out by using CADAM software (Leclerc et al., 2001). Monte-Carlo simulation technique is used by CADAM to perform safety analysis. Risk analysis is performed to identify possible failure mechanisms under usual, flood, psuedo-static, psuedo-dynamic, and post-seismic loading scenarios. The probability of failure of a dam-foundation-reservoir system is computed as a function of the uncertainities in loading and strength parameters that are considered as random variables (Leclerc et al., 2001). In this thesis, tensile strength and peak cohesion of lift joints, peak friction coefficient, normal upstream reservoir elevation, drain efficiency, and horizontal peak ground acceleration are taken as random variables. In order to quantify uncertainities for these variables, probability density functions and coefficient of variation are identified using the previous studies about reliability-based safety analysis.

For better understanding, a brief description of structural reliability approach and important terms are given in the following chapter. Also, forces acting on concrete gravity dams and stability analysis are explained briefly. Necessary methods for the evaluation of safety are summarized. Seismic coefficient method and simplified response spectra analysis described by Chopra (1988) are presented for static and dynamic seismic analysis, respectively. Additionally, capabilities of CADAM are explained in detail.

CHAPTER 2

STRUCTURAL RELIABILITY APPROACH

Reliability is the ability of a system to perform its required functions under stated conditions for a specified period of time. Also, it can be defined as the probabilistic measure of assurance of performance or safety for engineered systems. Structural reliability approach reflects or represents uncertainties in the system and therefore, the assurance of performance can be represented realistically.

Classical reliability approach, first order second moment method, advanced first order second moment method, second order reliability model and Monte-Carlo simulation method are the main methods proposed by researchers. In this thesis, Monte-Carlo simulation technique is used to perform probabilistic analysis of a dam-foundation-reservoir system.

For better understanding, the basic information about classical reliability approach is given, briefly. Most common probabilistic distributions which are necessary to identify uncertainties of random variables are explained and finally, Monte-Carlo simulation technique is discussed.

2.1 Classical Reliability Approach

In classical reliability approach, a system is characterized by a single failure mode and a specific direction is considered for the forces. Failure mode can be described as the manner by which a failure is observed. It generally describes the way the failure occurs and its impact on a system or operation of an equipment (Pentti and Atte, 2002). Probability of failure or risk is described as the probability for which resistance of the system is less than or equal to the load.

Let S be the load effect on the structure and R be the capacity (resistance) of the structure. Then, the probability of failure is determined with the following equation (Ang and Tang, 1990):

$$P_f = \iint_{\{(s,r):r<s\}} f_{R,S}(r,s)drds = \int_0^{\infty}\int_0^{s} f_{R,S}(r,s)drds \tag{2.1}$$

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

$$P_f = \int_0^{\infty}\left[\int_0^{s} f_R(r)dr\right]f_S(s)ds \tag{2.2}$$

The following formulation generalizes the failure probability:

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

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

$$P_f = \int_0^{\infty}\int_0^{S} f_{s,R_p}(s,r_p)dr_p ds + \int_{-\infty}^{0}\int_{S}^{0} f_{s,R_n}(s,r_n)dr_n ds \tag{2.3}$$

where p and n denote positive and negative quantities, respectively.

2.1.1 Probability Distributions

While determining the failure probability, the distributions of random variables should be known. Most commonly used distributions in civil engineering applications are uniform, normal and log-normal distributions.

2.1.1.1 Uniform Distribution

The random variable x is defined on the interval a to b with the probability density function, PDF (See Figure 2.1):

$$p(x) = \frac{1}{b-a} \qquad \text{where } a \le x \le b \tag{2.4}$$

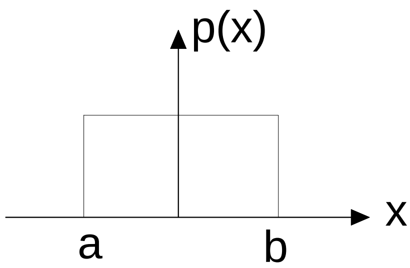


Figure 2.1 Uniform Probability Density Function

2.1.1.2 Normal Distribution

The random variable x is stated to be normally distributed if its PDF:

$$p(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(\frac{-(x-\mu)^2}{2\sigma^2}\right); \qquad -\infty \le x \le \infty \tag{2.5}$$

where statistical properties, μ and σ are mean and standard deviation, respectively. Coefficient of variation, δ is also another important property of random variables, which is equal to σ / μ. The probability that a random variable will assume a value between a and b can be computed from the area under its PDF between a and b (See Figure 2.2):

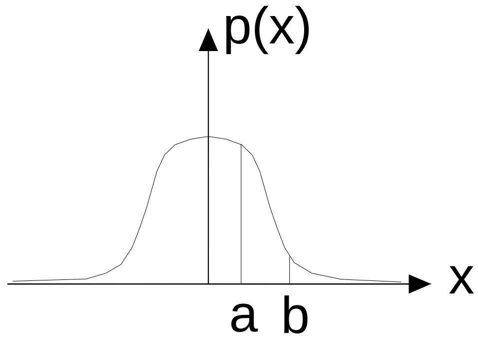


Figure 2.2 Normal Probability Density Function

2.1.1.3 Log-Normal Distribution

The log-normal distribution corresponds to a transformation of variables. If the random variable x is log-normally distributed, then random variable y, which is equal to $\ln x$ will be normally distributed. The log-normal distribution of x is given by (see Figure 2.3):

$$p(x) = \frac{1}{x\sigma_y\sqrt{2\pi}}\exp\left(\frac{-(y-\mu_y)^2}{2\sigma_y{}^2}\right); \qquad x > 0 \tag{2.6}$$

$$\mu_y = E(\ln x) = \ln \mu_x - \frac{1}{2}\xi^2 = \lambda$$

$$\sigma_y^2 = VAR(\ln x) = \ln(1+\frac{\sigma_x^2}{\mu_x^2}) = \xi^2$$

where μ_y and σ_y are the mean and standard deviation of y, respectively.

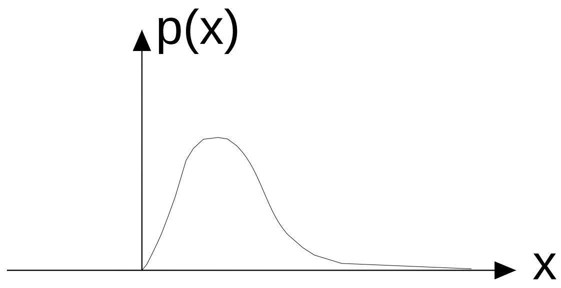


Figure 2.3 Log-Normal Probability Density Function

2.1.2 Multiple Failure Modes

The influence of different failure modes can be reflected by the probabilistic approach. If a structural component with k different failure modes is denoted by $M_1, M_2, ..., M_k$, then probability of failure is described by:

$$P_f = P_r(M_1 \cup M_2 \cup M_3 ... \cup M_k) \tag{2.7}$$

Let S be the load effect on the structure and R_i be the capacity at the i^{th} failure mode. If R_i values and S are assumed to be statistically independent, then the joint probability density function is:

$$F_{S,R1,R2,...,Rk}(s, r_1, r_2, ..., r_k) = f_s(s) f_{R1,R2,...,Rk}(r_1, r_2, ..., r_k) \tag{2.8}$$

Probability of survival can be expressed as:

$$P_s = \int_0^\infty \left[\int_{C_{1S}}^\infty ... \int_{C_{ks}}^\infty f_{R1,...,Rk}(r_1, r_2, ..., r_k) dr_1 ... dr_k \right] f_s(s) ds \tag{2.9}$$

where $c_{1s},...,c_{ks}$ represent the respective load effects in failure modes, and $f_{R1,R2,...,Rk}(r_1, r_2,..., r_k)$ is the joint pdf of k-modal resistances.

2.2 Monte Carlo Simulation (MSC) Methods

As the complexity of an engineering system increases, the required analytical model may be extremely difficult to formulate mathematically unless gross idealization and simplifications are invoked. Moreover, in some cases, even if a formulation is possible, the required solution may be analytically intractable. In these instances, a probabilistic solution may be obtained through Monte Carlo simulations. Monte Carlo simulation is simply a process of generating deterministic solutions to a given problem repeatedly. Each solution corresponds to a set of deterministic values of the underlying random variables. The main element of a Monte Carlo simulation procedure is the generation of random numbers from a specified distribution (Ang and Tang, 1984).

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

$$\bar{P_u} = \frac{N_u}{N} \tag{2.10}$$

The variance of failure probability is given by:

$$Var(\bar{P_u}) = \frac{(1-P_u).\bar{P_u}}{N} \tag{2.11}$$

The coefficient of variation is determined from:

$$\delta(\bar{P_u}) = \frac{1}{P_u}\sqrt{\frac{(1-P_u).\bar{P_u}}{N}} \tag{2.12}$$

Broding et al. (1964) suggests a formula for the number of simulations as follows:

$$N > \frac{-\ln(1-c)}{P_f} \tag{2.13}$$

where N is the number of simulations for a given confidence level C in the probability of failure, P_f .

2.2.1 Generation of Random Numbers

A key task in the application of Monte Carlo simulation is the generation of the appropriate values of the random variables in accordance with the respective prescribed probability distributions (Ang and Tang, 1984). Suppose a random variable X with a Cumulative Density Function, CDF, $F_x(x)$. Then, at a given cumulative probability $F_x(x) = u$, the value of X is

$$x = F_x^{-1}(u) \tag{2.14}$$

Suppose that u is a value of the standard uniform variate, U , with a uniform PDF between 0 and 1.0; then, as shown in Figure 2.4.

$$F_U(u) = u \tag{2.15}$$

That is, the cumulative probability of $U \leq u$.

Therefore, if u is a value of U , the corresponding value of the variate X is obtained through Equation 2.14 will have a cumulative probability,

$$P(X \leq x) = P\left[F_x^{-1}(u) \leq x\right] = P\left[U \leq F_x(x)\right] = F_U\left[F_x(x)\right] = F_x(x)$$

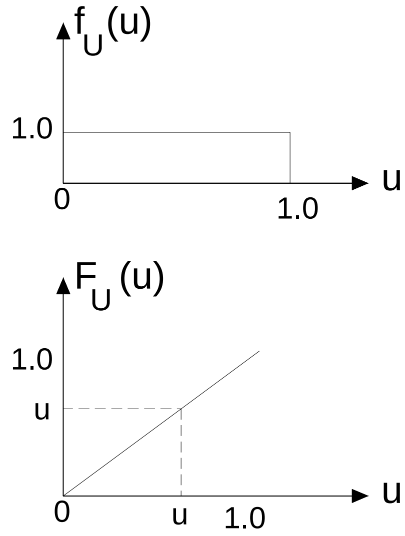


Figure 2.4 PDF and CDF of standard uniform variate U (Ang and Tang, 1984)

which means that if $(u_1, u_2, ..., u_n)$ is a set of values from U, the corresponding set of values obtained through Equation 2.14 that is,

$$x_i = F_x^{-1}(u_i) \qquad i = 1,2,...,n \tag{2.16}$$

will have the desired CDF $F_x(x)$. The relationship between u and x may be seen graphically in Figure 2.5.r

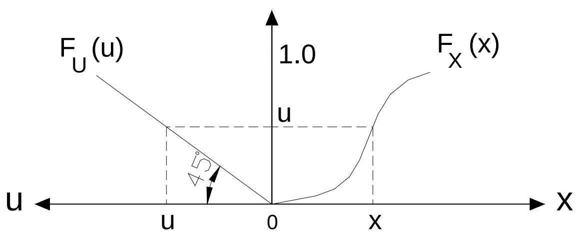


Figure 2.5 Relation between u and x (Ang and Tang, 1984)

CHAPTER 3

SAFETY ANALYSIS OF HARDFILL DAMS

Stability analyses of concrete gravity dams are performed for various loading conditions and the structure is required to prove its safety and stability under all loading possibilities that are likely to occur during its service period (Yanmaz, 2006).

Hardfill dams are gravity type structures, construction of which is similar to RCC (roller compacted concrete) dams. Their stability requirements and methods of analysis are similar to those of gravity dams. RCC and hardfill dams only differ from gravity dams principally in mix design, details of appurtenances and construction methods (Corns et al., 1988).

In this chapter, necessary information in order to perform safety analysis is given. First, forces acting on concrete gravity dams for usual and flood conditions and loads supported by both static and dynamic seismic conditions are explained. Simplified response spectra analysis described by Chopra (1988) is presented in detail. Stability analyses for overturning, sliding, uplifting, etc., are also discussed in this chapter.

3.1 Forces Acting on a Gravity Dam

Figure 3.1 shows the possible forces acting on a gravity dam. The forces include

- W_c, the weight of the dam. This force acts at the centroid of the structure.

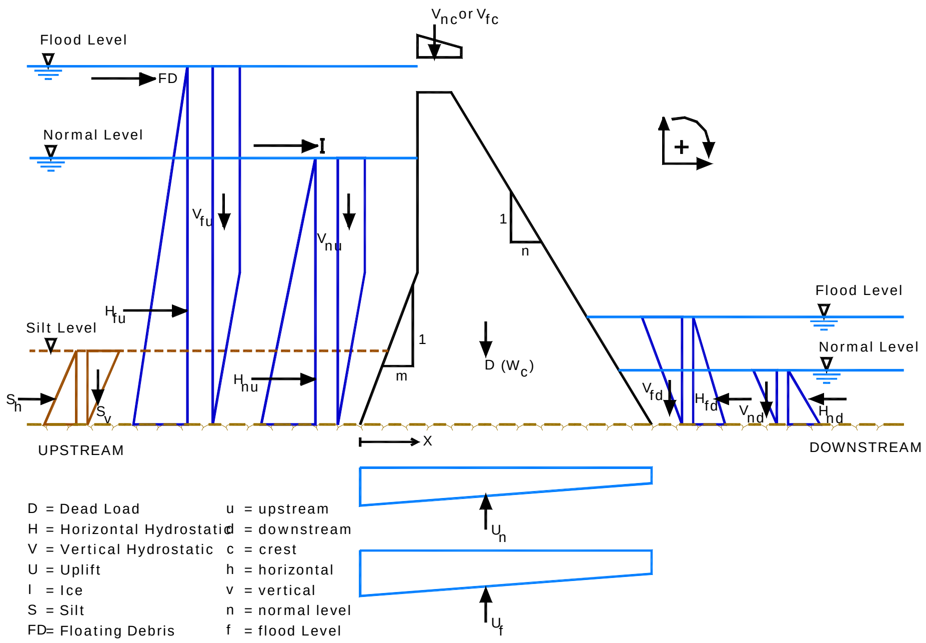


Figure 3.1 Forces Acting on a Dam (Static Analysis)

- Hydrostatic forces. H_u and V_u are the horizontal and vertical components of the reservoir water forces per unit width, respectively, H_d and V_d are the horizontal and the vertical components of the hydrostatic force produced by the tailwater, respectively, which are expressed as (Yanmaz, 2006).

$$H_u = \frac{1}{2}\gamma h_1^2 ; \quad V_u = \frac{1}{2}\gamma m h_1^2 ; \qquad H_d = \frac{1}{2}\gamma h_2^2 ; \quad V_d = \frac{1}{2}\gamma m h_2^2 \tag{3.1}$$

where γ is the specific weight of the water, h_1 and h_2 are the water depths in the reservoir and the tailwater, respectively.

- U , uplift force per unit width acting under the base of the dam.

$$U = \left[h_2 + \frac{\phi}{2}(h_1 - h_2) \right] B\gamma \tag{3.2}$$

where B is the bottom width of the dam and ϕ is the uplift reduction factor.

The uplift reduction factor is determined according to installation of drains. The porosity of the foundation material, jointing and faulting are the other main factors affecting the magnitude of the uplift force.

- S_h, force due to sediment accumulation determined from Rankine's lateral earth pressure formula

$$F_s = \frac{1}{2}\gamma_s h_s^2 K_a\,; \qquad K_a = \frac{1-\sin\theta}{1+\sin\theta} \tag{3.3}$$

where γ_s is the submerged specific weight of soil, K_a is the active earth pressure coefficient , h_s is the depth of sediment material, and θ is the angle of repose.

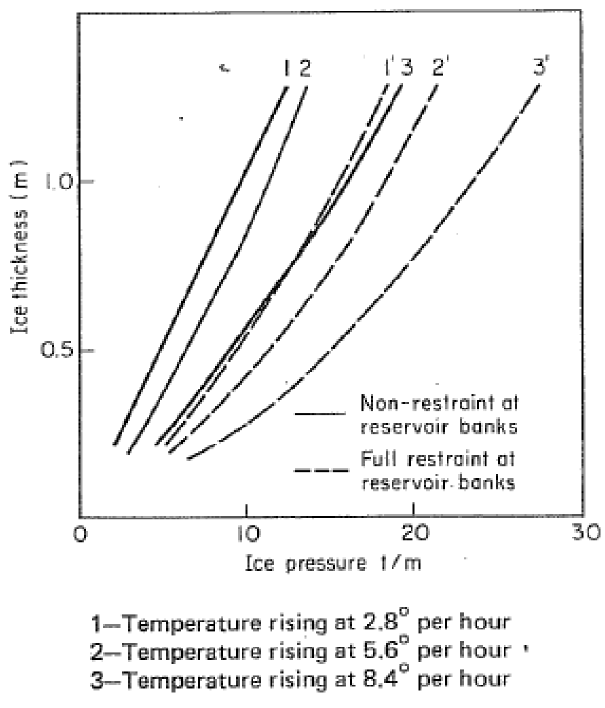


Figure 3.2 Approximate Ice Loading (Thomas, 1976)

- *I*, ice load. The melting of the ice sheet on the reservoir surface causes stresses on the dam. These stresses caused by thermal expansion of the ice depend on the thickness of the ice sheet and the temperature rise of the ice (Yanmaz, 2006). Figure 3.2 gives approximate ice loading.

3.1.1 Psuedo – Static Seismic Analysis (Seismic Coefficient)

Basic forces supported for psuedo-static seismic analysis are given as (See Figure 3.3):

- Earthquake forces (inertia forces) on the dam body are computed from:

$$Q = kW_c \tag{3.4}$$

where k is the earthquake coefficient both in horizontal and vertical directions. These forces act through center of gravity of the dam.

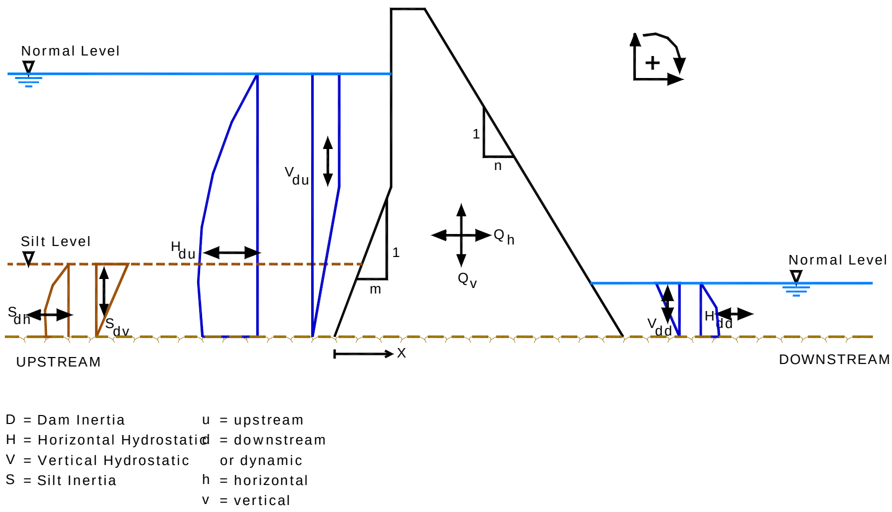


Figure 3.3 Forces Acting on a Dam (Psuedo-Static Seismic Analysis)

- Hydrodynamic force due to earthquake is determined from the following expression:

$$H_{du} = 0.726Ck\gamma h_1^2\,; \qquad C = 0.7\left(1-\frac{\theta'}{90}\right) \tag{3.5}$$

where θ' is the angle between the upstream face of the dam and the vertical line (Yanmaz, 2006). The Westergaard parabola based on added mass concept can also be used. The added horizontal hydrodynamic force acting above the depth y increases following a parabolic distribution given by:

$$H_d(y) = \frac{2}{3}K_\theta C_e(acc)\sqrt{h}(y^{1.5}) \tag{3.6}$$

where h is the total depth of the reservoir, y is the distance below reservoir surface, acc is the horizontal acceleration coefficient applied at the base of the dam expressed in term of peak ground acceleration or spectral acceleration, K_θ is the correction factor for the sloping dam faces with angle θ'. As a first approximation for the horizontal and the vertical correction factors, $K_{QH} = \cos^2\theta'$ and $K_{QV} = \sin\theta'\cos\theta'$ can be used, respectively. C_e is the factor depending on depth of water and the earthquake vibration period characterizing the frequency content of the applied ground motion (Leclerc et al., 2001).

The Westergaard approximation for the C_e is given by:

$$C_e = 0.799C_c\,; \qquad C_c = \frac{1}{\sqrt{1-7.75\left(\dfrac{h}{1000t_e}\right)^2}} \text{ (kN.sec.m)} \tag{3.7}$$

where C_c is the Westergaard correction factor for water compressibility and t_e is the period to characterize the seismic acceleration imposed to dam.

USBR (1987) considers a slope correction method for dams with a combination vertical and sloping face:

- ❖ 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, analyze as if 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.

In this thesis, the second way is used to determine the hydrodynamic force due to earthquake.

3.1.2 Psuedo – Dynamic Seismic Analysis (Chopra's Method, (1988))

Psuedo-dynamic seismic analysis is based on response spectra method. It is conceptually similar to a psuedo-static analysis except that it recognizes 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 the stress and stability analyses are performed with the inertia forces continuously applied in the same direction (Leclerc et al., 2001).

Forces acting on a dam which are used in pseudo-dynamic seismic analysis are given in Figure 3.4.

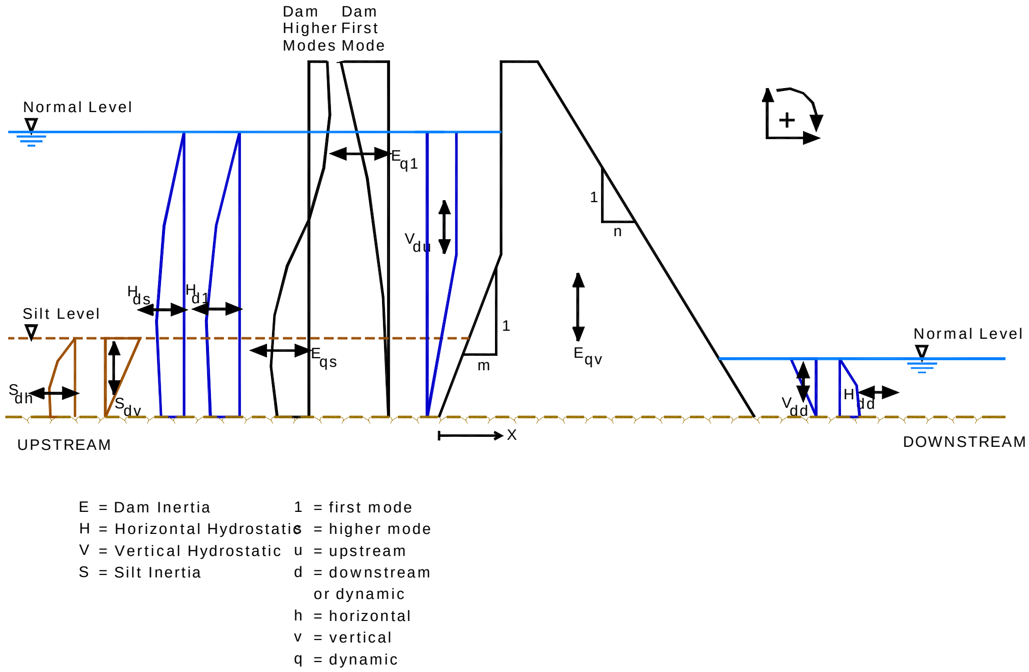


Figure 3.4 Forces Acting on a Dam (Psuedo-Dynamic Seismic Analysis)

3.1.2.1 Computation of the Equivalent Lateral Earthquake Force

The maximum effects of the horizontal earthquake ground motion can be represented by equivalent lateral forces acting on the upstream face of the dam (Chopra, 1988). These forces can be examined in two parts:

a) Equivalent Lateral Earthquake Force due to Fundamental Vibration Mode is given by the following equation

$$f_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} \frac{S_a(\tilde{T}_1, \tilde{\xi}_1)}{g} \left[W_s(y)\phi(y) + g p_1(y, \tilde{T}_r) \right] \tag{3.8}$$

The fundamental vibration period of concrete gravity dams, in sec, on rigid foundation rock with empty reservoir is given by:

$$T_1 = 1.4 \frac{H_s}{\sqrt{E_s}} \tag{3.9}$$

where H_s is the height of the dam in ft and E_s is the Young's modulus of the elasticity of concrete in psi.

The natural vibration period of the dam in seconds on rigid foundation rock with impounded water is computed from:

$$\tilde{T}_r = R_r T_1 \tag{3.10}$$

where R_r (Figure 3.5) is the period lengthening ratio due to hydrodynamic effects. If $H / H_s < 0.5$, R_r can be accepted as equal to 1.

The natural vibration period of the dam in seconds on flexible foundation rock with empty reservoir is given by:

$$\tilde{T}_f = R_f T_1 \tag{3.11}$$

where R_f (Figure 3.6) is the period lengthening ratio due to foundation-rock flexibility effects. The natural vibration period of the dam in seconds on flexible foundation rock with impounded water is:

$$\tilde{T}_1 = R_r R_f T_1 \tag{3.12}$$

Effective damping factor for dam on flexible foundation rock with impounded water is computed from:

$$\tilde{\xi}_1 = \frac{1}{R_r}\frac{1}{\left(R_f\right)^3}\xi_1 + \xi_r + \xi_f \tag{3.13}$$

where ξ_1 is the damping ratio of the dam on rigid foundation rock with empty reservoir, ξ_r (Figure 3.5) is the added damping due to dam-water interaction and reservoir bottom absorption and ξ_f (Figure 3.6) is the added radiation material and material damping due to dam-foundation rock interaction.

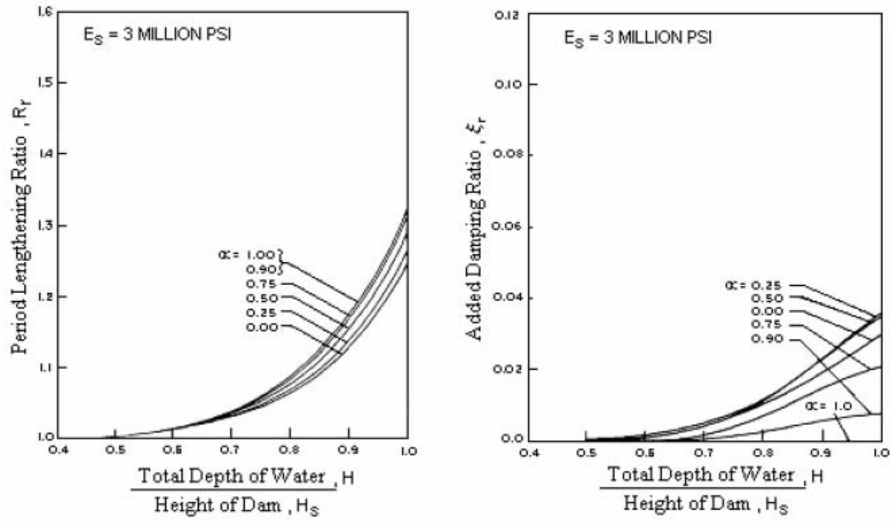


Figure 3.5 Standard Values for Period Lengthening Ratio R_r and Added Damping Ratio ξ_r due to Hydrodynamic Effects (Chopra, 1988)

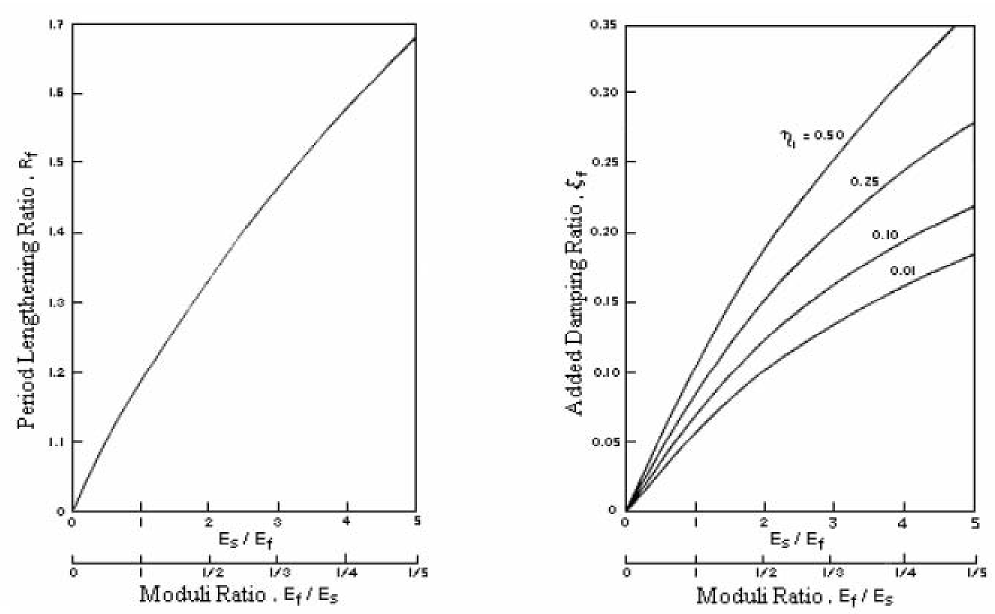


Figure 3.6 Standard Values for Period Lengthening Ratio R_f and Added Damping Ratio ξ_f due to Dam-Foundation Rock Interaction (Chopra,1988)

The period ratio necessary to compute the hydrodynamic pressure term, $gp(y, \tilde{T}_r)$:

$$R_w = \frac{T_1^r}{\tilde{T}_r} \tag{3.14}$$

where the fundamental vibration period of the impounded water $T_1^r = 4H / C$ in which H is the depth of impounded water and C is the velocity of pressure waves in water. The hydrodynamic pressure term can be determined from Figure 3.7 in which α is the wave reflection coefficient. The generalized mass is given by:

$$\tilde{M}_1 = (R_r)^2 M_1 \tag{3.15}$$

where M_1 is determined from:

$$M_1 = \frac{1}{g}\int_0^{H_S} w_s(y)\phi^2(y)dy \tag{3.16}$$

where $w_s(y)$ is the weight of the dam per unit height, $\phi(y)$ is the fundamental vibration mode shape (Figure 3.8).

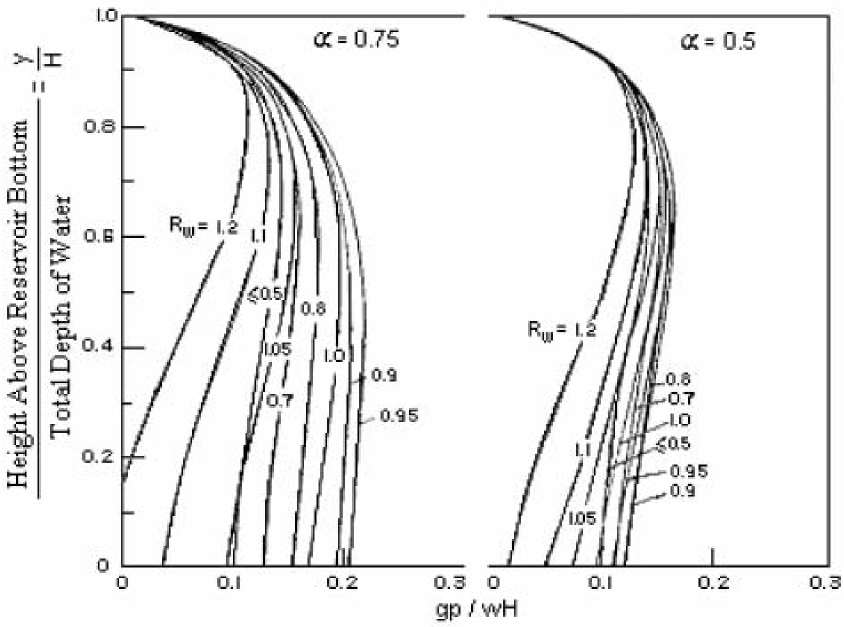


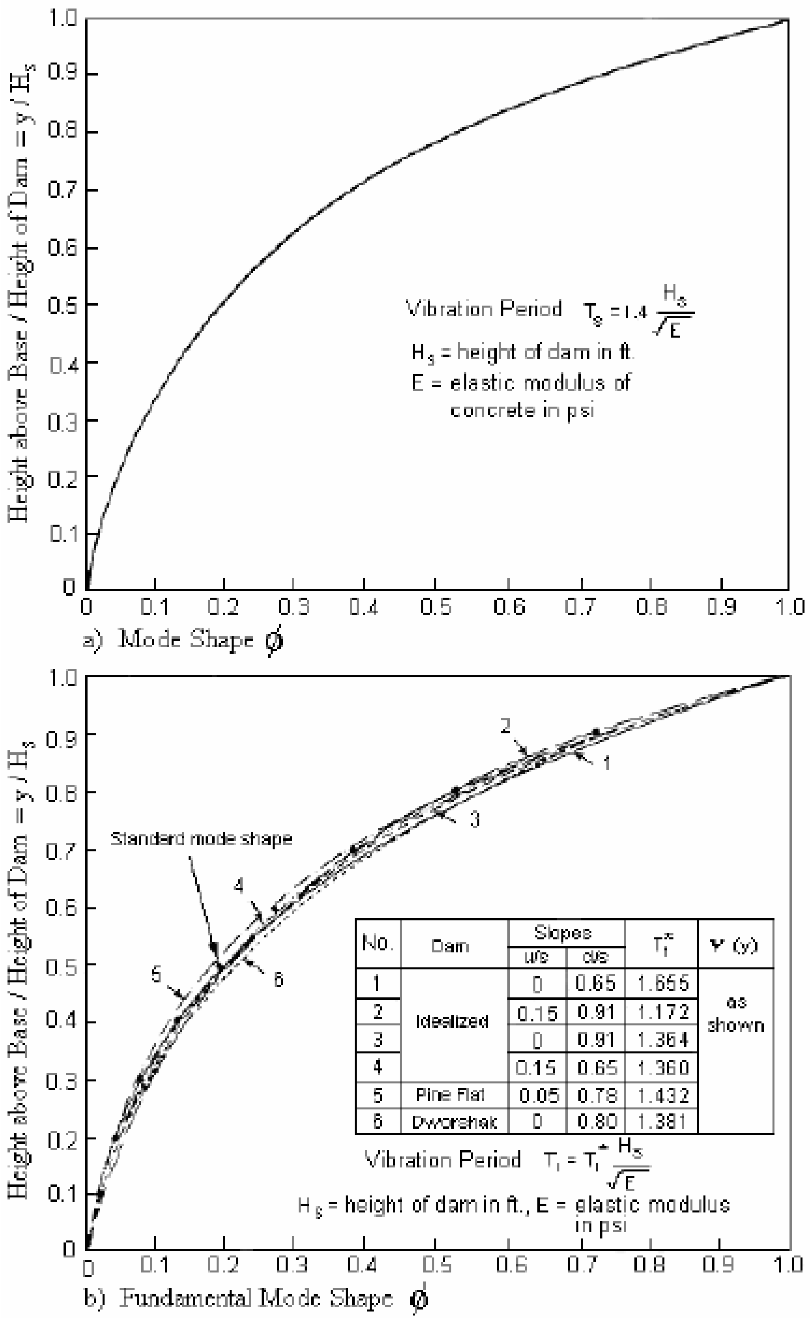
Figure 3.7 Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e. $H/H_s = 1$; $\alpha = 0.75$ *and* 0.50 (Chopra, 1988)

The generalized earthquake force coefficient is computed from:

$$\tilde{L}_1 = L_1 + \frac{1}{8} F_{st} \left(\frac{H}{H_s}\right)^2 A_p \tag{3.17}$$

where F_{st} is the total hydrostatic force on the dam$(wH^2/2)$. A_p is the hydrodynamic force coefficient tabulated in Tables 3.1 and 3.2 for a range of values for the period ratio R_w and the wave reflection coefficient α. The value of L_1 is determined from:

$$L_1 = \frac{1}{g}\int_0^{H_S} w_s(y)\phi(y)dy \tag{3.18}$$



No.	Dam	Slopes u/s	Slopes d/s	T_1^*	Ψ (y)
1	Idealized	0	0.65	1.655	as shown
2		0.15	0.91	1.172	
3		0	0.91	1.364	
4		0.15	0.65	1.360	
5	Pine Flat	0.05	0.78	1.432	
6	Dworshak	0	0.80	1.381	

Figure 3.8 Fundamental Period and Mode Shape of Vibration for Concrete Gravity Dams: (a) Standard Period and Mode Shape; (b) Comparison of Standard Values with Properties of Six Dams (Chopra, 1988)

Table 3.1 Standard Values for Hydrodynamic Force Coefficient A_p in $\tilde{L}_1$; α=1 (Chopra, 1988)

R_w (1)	Value of A_p for α=1 (2)
0.99	1.242
0.98	0.893
0.97	0.739
0.96	0.647
0.95	0.585
0.94	0.539
0.93	0.503
0.92	0.474
0.90	0.431
0.85	0.364
0.80	0.324
0.75	0.279
≤0.50	0.237

Table 3.2 Standard Values for Hydrodynamic Force Coefficient A_p in $\tilde{L}_1$; *α=0.90, 0.75, 0.50, 0.25 and 0* (Chopra, 1988)

R_w	Value of A_p				
(1)	α=0.90 (2)	α=0.75 (3)	α=0.50 (4)	α=0.25 (5)	α=0 (6)
1.20	0.071	0.111	0.159	0.178	0.181
1.10	0.110	0.177	0.204	0.197	0.186
1.05	0.194	0.249	0.229	0.205	0.189
1.00	0.515	0.340	0.252	0.213	0.191
0.95	0.518	0.378	0.267	0.219	0.193
0.90	0.417	0.361	0.274	0.224	0.195
0.80	0.322	0.309	0.269	0.229	0.198
0.70	0.278	0.274	0.256	0.228	0.201
≤0.50	0.237	0.236	0.231	0.222	0.206

The stresses throughout the dam subjected to equivalent lateral forces $f_1(y)$: The finite element method may be used for this static stress analysis. Alternatively, traditional procedures for design calculations may be used wherein the normal bending stresses σ_{y1} across a horizontal section are computed by elementary formulas for stresses in beams. The maximum principal stresses at the upstream and downstream faces can be computed from the normal bending stresses σ_{y1} by an appropriate transformation (Chopra, 1988):

$$\sigma_1 = \sigma_{y1} \sec^2 \theta' + p_1 \tan^2 \theta' \tag{3.19}$$

If no tailwater 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:

$$p_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} S_a\left(\tilde{T}_1, \tilde{\xi}_1\right) p\left(y, \tilde{T}_r\right) \tag{3.20}$$

b) Equivalent Lateral Earthquake Force due to Higher Vibration Modes can be computed by using the following formulation

$$f_{sc}(y) = \frac{1}{g}\left\{ w_s(y)\left[1 - \frac{L_1}{M_1}\phi(y)\right] + \left[g p_0(y) - \frac{B_1}{M_1} w_s(y)\phi(y)\right]\right\} a_g \tag{3.21}$$

where a_g is the maximum ground acceleration, $p_0(y)$ is the hydrodynamic pressure function associated with the higher modes for the loading condition with the reservoir at depth H, and at a *y*-distance above the foundation (Figure 3.9). B_1 is computed from:

$$B_1 = 0.052 \frac{F_{st}}{g} \left(\frac{H}{H_s} \right)^2 \tag{3.22}$$

in which F_{st} is the total hydrostatic force on dam.

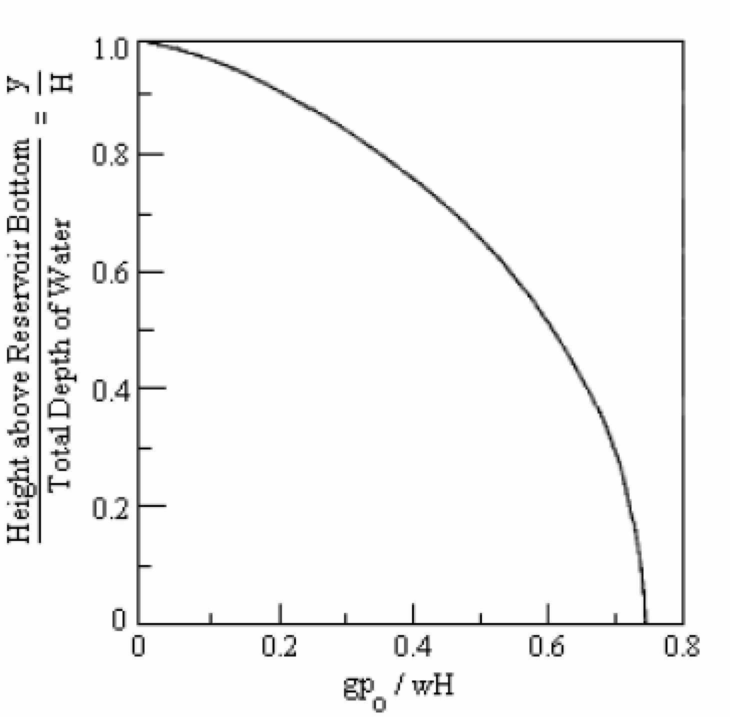


Figure 3.9 Standard Values for Hydrodynamic Pressure Function $p_o\left(\hat{y}\right)$

(Chopra, 1988)

Computation of the stresses by higher vibration modes is the same as the computation of stresses by fundamental vibration mode except that the normal bending stresses and the hydrodynamic pressures at the downstream face are defined as $\sigma_{y,sc}$ and p_{sc}, respectively.

$$\sigma_{sc} = \sigma_{y,sc} \sec^2 \theta' + p_{sc} \tan^2 \theta' \tag{3.23}$$

At the upstream face, the hydrodynamic pressure p_{sc} is given by:

$$p_{sc}(y) = \left[g p_0(y) - \frac{B_1}{M_1} w_s(y)\phi(y) \right] \frac{a_g}{g} \tag{3.24}$$

The initial stresses in the dam due to the self weight of the dam, hydrostatic pressure, creep, construction sequence, and thermal effects are computed from:

$$\sigma_{st} = \sigma_{y,st} \sec^2 \theta' + p_{st} \tan^2 \theta' \tag{3.25}$$

where $\sigma_{y,st}$ is the normal stresses across horizontal sections. The hydrostatic pressure $p_{st} = w(H-y)$ on the upstream face and $p_{st} = 0$ on the downstream face if tailwater is excluded.

Total stresses in the dam are computed from the square-root-of-the-sum-of-squares (SRSS) combination rule:

$$r_d = \sqrt{(r_1)^2 + (r_{sc})^2} \tag{3.26}$$

where r_1 and r_{sc} are the values of the response quantity associated with the fundamental and higher vibration modes, respectively.

The total value of any response quantity is computed from:

$$r_{\max} = r_{st} \pm \sqrt{(r_1)^2 + (r_{sc})^2} \tag{3.27}$$

where r_{st} is its initial value prior to the earthquake.

Implementation of the procedure in metric units is straightforward because most quantities are presented in nondimensional form.

Conversion to metric system:

The fundamental vibration period in seconds is determined from:

$$T_1 = 0.38 \frac{H_s}{\sqrt{E_s}}$$

where H_s is in meters and E_s is in MPa. In the conversion, the following values are used: 1 million psi (pounds per square inch) $= 7000$ MPa, the unit weight of the water, $w = 9.81$ kN/m^3, the gravitational acceleration, $g = 9.81$ m/s^2, and velocity of pressure waves in water, $C = 1440$ m/s.

3.1.2.2 Spectral Acceleration Coefficient

The spectral acceleration coefficient is the ordinate of pseudo-acceleration response spectrum for the ground motion evaluated at period $\tilde{T}_1$ and damping ratio $\tilde{\xi}_1$ of dam. That is, in order to determine the spectral acceleration coefficient, response spectrum should be obtained. Therefore, there should be earthquake data about the site under investigation. However, earthquake data may not be available for each site. For such cases, simplified procedures in specifications can be followed.

In Turkey, Specification for Structures to be Built in Disaster Areas (2007) which is published by Republic of Turkey Ministry of Public Works and Settlement offers the following simplified solution:

$$A(T) = A_0 I S(T) \tag{3.28}$$

where $A(T)$ is the spectral acceleration coefficient, A_0 is the horizontal ground acceleration coefficient, I is the building importance factor and $S(T)$ is the spectrum coefficient (RTMPWS, 2007).

Elastic spectral acceleration, $S_{ae}(T)$ corresponding to the ordinate of 5% damped elastic response spectrum is equal to the multiplication of the spectral acceleration coefficient and the gravitational acceleration, g (RTMPWS, 2007).

$$S_{ae}(T) = A(T)g \tag{3.29}$$

A_0 values for different seismic zones are given in Table 3.3.

$$S(T) = 1 + 1.5T / T_A \tag{3.30}$$

$$S(T) = 2.5 \tag{3.31}$$

$$S(T) = 2.5(T_B / T)^{0.8} \tag{3.32}$$

where T is the building natural period, T_A and T_B are the spectrum characteristic periods (RTMPWS, 2007). These spectrum characteristic periods for different soil groups defined in the reference are given in Table 3.4. The detailed definition of the soil classes can be obtained from the reference.

Table 3.3 The Effective Horizontal Ground Acceleration Values

Seismic Zone	A_0
1	0.40
2	0.30
3	0.20
4	0.10

Table 3.4 The Spectrum Characteristic Periods

Soil Class	T_A	T_B
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

3.2 Stability Analysis

A dam should be safe against overturning and sliding at any plane under all load combinations. These analyses should be performed after the stress analysis and the computation of the crack length. Also, additional performance indicators, such as uplifting safety factor should be computed.

3.2.1 Normal Base Pressure

The total normal stresses along the base are given by:

$$\sigma = \frac{\sum V}{A} \pm \frac{\sum Mc}{I} \qquad (3.33)$$

where

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

A = Area of uncracked ligament

$\sum M$ = Moment about the center of gravity of the uncracked ligament of all loads including uplift pressures

I = Moment of inertia of the uncracked ligament

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

3.2.2 Overturning Stability

The factor of safety against overturning is defined as:

$$OSF = \frac{\sum M_s}{\sum M_o} \tag{3.34}$$

where $\sum M_s$ is the sum of stabilizing moment about the downstream or the upstream end of the joint considered and $\sum M_o$ is the sum of overturning moments.

3.2.3 Sliding Stability

The shear friction sliding safety factor along a horizontal plane is given by:

$$SSF = \frac{\left(\sum \overline{V} + U + Q_v\right)\tan\phi + cA_c}{\sum H + \sum H_d + Q_h} \tag{3.35}$$

where

$\sum V$ = Sum of vertical forces excluding uplift pressures

U = Uplift pressure force resultant

Q_v =Vertical concrete inertia forces

ϕ = friction angle (peak or residual value)

c = cohesion (apparent or real)

A_c = Area in compression

$\sum H$ = Sum of horizontal forces

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

Q_h = Horizontal hydrodynamic forces

3.2.3.1 Shear Friction Method

The shear friction safety factor is given by:

$$SSF = \frac{R}{\sum H} \tag{3.36}$$

where

R = maximum horizontal driving force that can be resisted (sliding resistance)

$\sum H$ = summation of horizontal forces

The sliding resistance may be obtained from the principles of statics by resolving forces parallel and perpendicular to the sliding plane (Figure 3.10):

$$R = \sum V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi \tan\alpha)} \tag{3.37}$$

where

$\sum V$ = Sum of vertical forces including uplift forces

ϕ= friction angle (peak or residual value)

c = cohesion

A = area of potential failure plane developing cohesion c

α = angle between inclined sliding plane and the horizontal

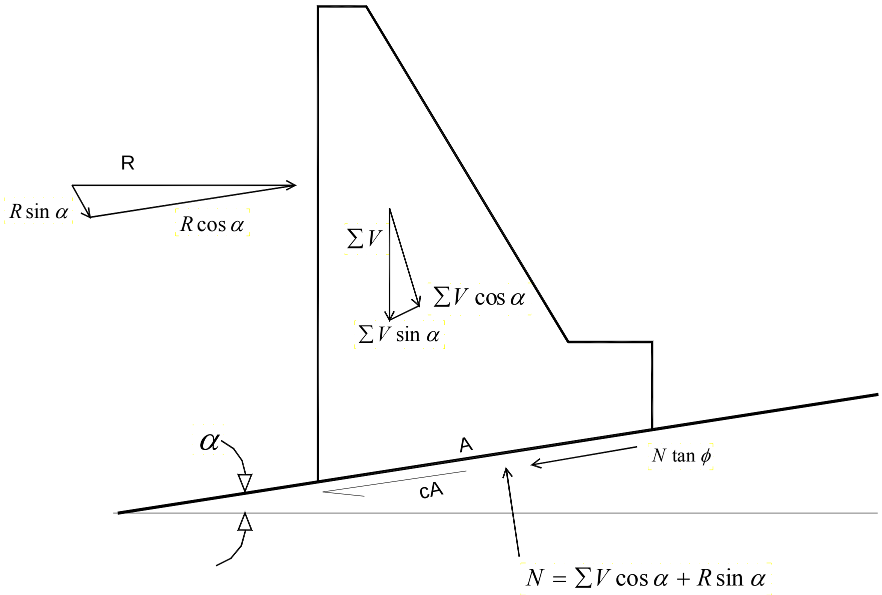


Figure 3.10 Sliding Resistance (Corns et al., 1988)

3.2.3.2 Limit Equilibrium Method

The limit equilibrium method defines the factor of safety as the ratio of the shearing strength to the applied shear stress. For inclined joints:

$$SSF = \frac{\left|\left(\sum \overline{V}\cos(\alpha) - \sum H\sin(\alpha)\right) + U\right| + \tan\phi + cA_c}{\left|\sum H\cos(\alpha) + \sum \overline{V}\sin(\alpha)\right|} \tag{3.38}$$

where

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

$\left|(\sum H\cos(\alpha) - \sum \overline{V}\sin(\alpha))\right|$ = 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

3.2.3.3 Passive Wedge Resistance

While computing the sliding safety factor, the passive resistance of a rock wedge located at the toe of the dam can be considered (See Figure 3.11). When a passive rock wedge resistance is considered, the SSF should be computed using the shear friction method.

$$SSF = \frac{\left(\sum \overline{V} + U\right)\tan\phi_1 + c_1 A_1 + \left[\dfrac{c_2 A_2}{\cos\alpha(1-\tan\phi_2 \tan\alpha)} + W\tan(\alpha+\phi_2)\right]}{\sum H} \quad (3.39)$$

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

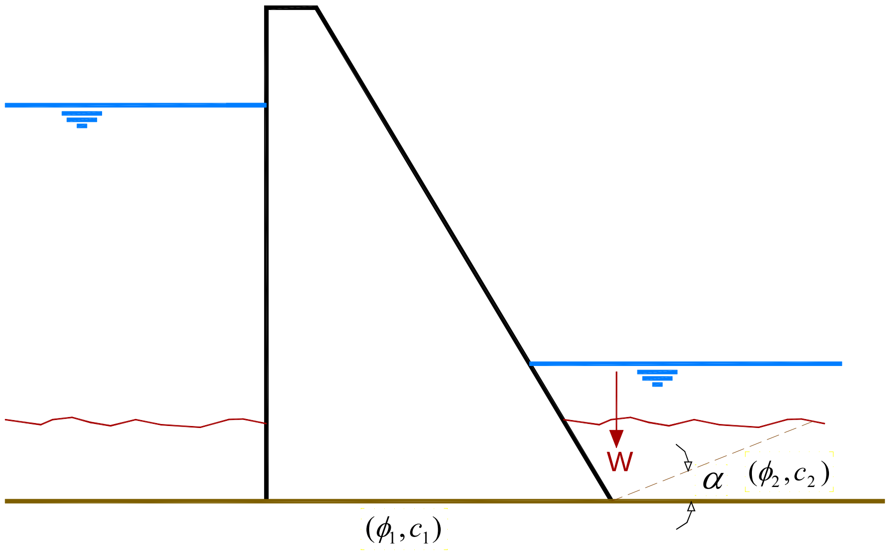


Figure 3.11 Passive Wedge Resistance (Leclerc et al., 2001)

3.2.4 Uplifting Stability Analysis

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

$$USF = \frac{\sum \overline{V}}{U} \tag{3.40}$$

where

$\sum V$ = Sum of vertical forces excluding uplift pressures

U = Uplift forces due to uplift pressures

CHAPTER 4

CAPABILITIES OF CADAM

4.1. Introduction

In this chapter, general information about CADAM software is presented.

4.1.1 Objectives

CADAM is a computer program that was primarily designed to provide support for learning the principles of structural stability evaluation of concrete gravity dams. CADAM is also used to support research and development on structural behavior and safety of concrete dams. This program was developed in the context, of the research and development activities, of the industrial chair on *Structural Safety of Existing Concrete Dams,* which was established in 1991 at École Polytechnique de Montréal.

CADAM is based on the gravity method. It performs stability analyses for hydrostatic and seismic loads. Several modeling options have been included to allow users to explore the structural behavior of gravity dams including Roller Compacted Concrete and hardfill (e.g. geometry, uplift pressures and drainage, crack initiation and propagation). CADAM allows user (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 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).
-

4.1.2 Basic Analytical Capabilities

- Static Analyses: CADAM performs stability analysis for normal operating reservoir level and flood level taking into account overtopping pressures on the crest.
- Seismic Analyses: CADAM performs seismic analysis using the pseudo-static method or the pseudo-dynamic method based on Chopra’s (1987) simplified method for gravity dams.
- Post Seismic Analyses: In post-seismic analyses the specified cohesion is not applied over the length of crack induced by the seismic event. The post-seismic uplift pressures can either build-up to its full value in seismic cracks or return to its initial value if the seismic crack is closed after the earthquake.
- Probabilistic Safety Analysis (Monte-Carlo Simulations): CADAM can 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. Monte-Carlo simulation method is used to estimate the failure probability of the system.
- Incremental Load Analysis: CADAM automatically performs sensitivity analysis by computing and plotting the evolution of typical performance indicator (ex: sliding safety factor) as a function of a progressive application in the applied loading, e.g. variable reservoir elevation.

4.1.3 Modeling Capabilities

Input parameters necessary for a typical analysis of a gravity dam-foundation-reservoir system can be listed as below (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 vertical forces in a static analysis and inertia forces in a seismic analysis. The masses can be used to represent fixed equipment located on the crest, or to introduce corrections to the basic cross-section to represent holes or a non-uniform mass distribution along the length of the dam.
- 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.
- 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 can be performed through linearization of effective stresses (FERC 1999, CDSA1995, USACE 1995, USBR 1987) or superposition of total stresses with uplift pressures (FERC 1991).
- Post-tension cables: 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 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 water compressibility effects, inclination of the upstream face, 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): peak ground and spectral acceleration data, dam and foundation stiffness and damping properties, reservoir bottom damping properties and velocity of an impulsive pressure wave in water, modal summation rules.
- Cracking options: Specification of a tensile strengths for crack initiation and propagation, dynamic amplification factor for the tensile strength, the incidence of cracking on static uplift pressure distributions (drain effectiveness), 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), 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: normal operating, flood, seismic 1, seismic 2 and post-seismic.
- Probabilistic Analysis: 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.

The basic modeling and analysis capabilities of CADAM are summarized in Figure 4.1

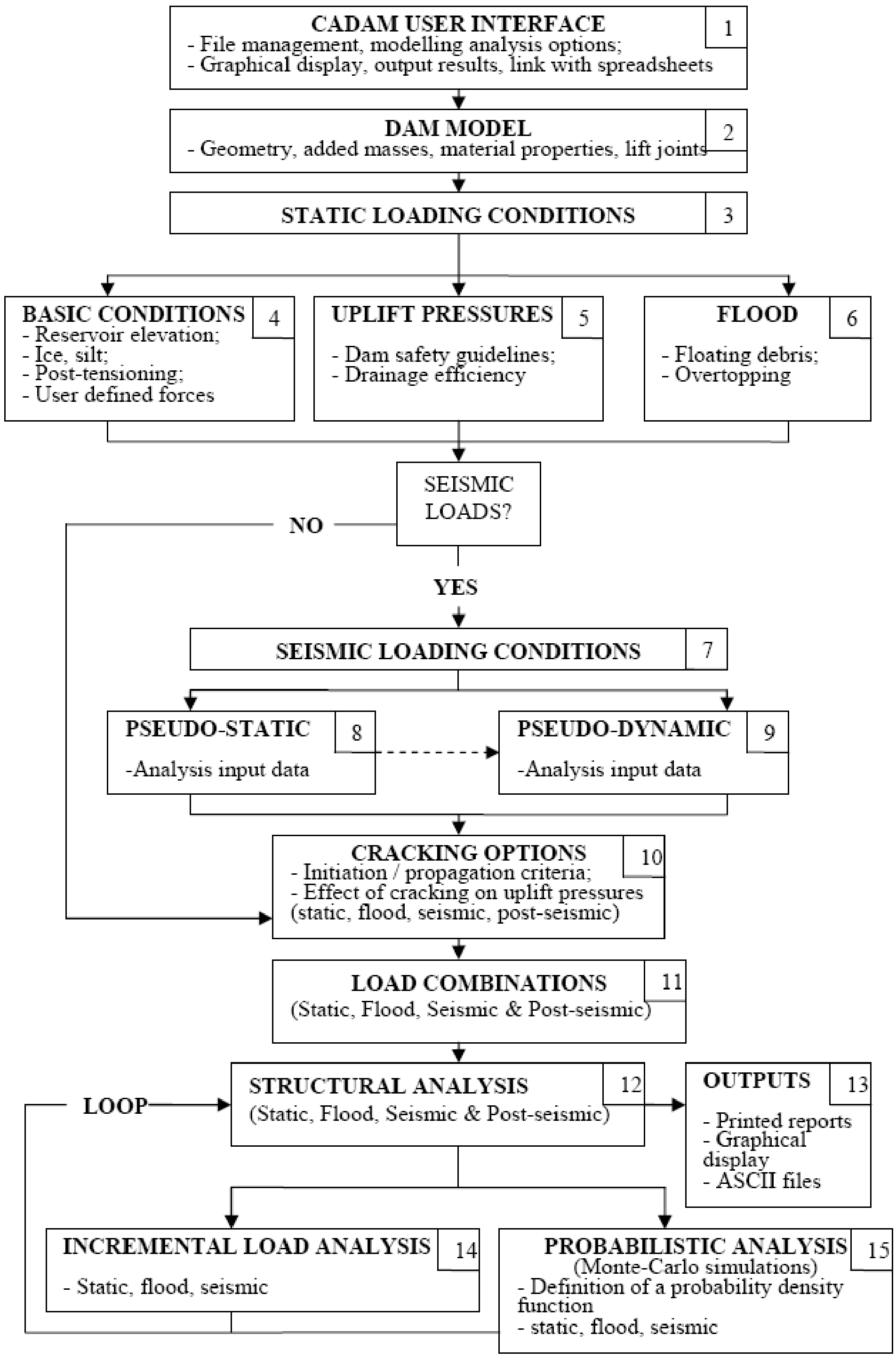


Figure 4.1 CADAM User Interface (Leclerc et al., 2001)

4.1.4 Output Results

Output results are presented in three different formats:
CADAM reports: Input parameters, loads, load combinations and stability drawings.
MS Excel reports: Input parameters, loads and load combinations.
Graphical plots: Joint cracking, stress and resultants, probabilistic analyses results (CDF or PDF of input parameters (random variables) or output parameters (safety factors), incremental analyses results (SF versus Load).

4.2 BASIC MODELING INFORMATION

4.2.1 Units

The loads, geometry and other characteristics of the dam can be defined either in metric units (kN, m) or imperial units (kip, feet). The unit system can be easily changed one from to the other automatically using the appropriate option.

4.2.2 Two-Dimensional Modeling of Gravity Dams

CADAM performs analysis for a unit thickness (i.e. 1 m or 1 ft) of the dam-foundation-reservoir system. Therefore, all input data should be specified as kN/m or Kips/ft.

4.2.3 Basic Assumptions of the Gravity Method

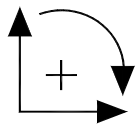
The structural stability of the dam against sliding, overturning and uplifting is evaluated through the stress and stability analyses. Stress analysis is performed to determine eventual crack length and compressive stresses. Stability analysis is performed to determine safety margins against sliding and the position of the resultant of all forces acting on a joint. A joint represents a concrete-concrete or concrete-rock interface.

The gravity method is based on rigid body equilibrium to determine the internal forces acting on the potential failure plane (joints and concrete-rock interface) and on beam theory to compute stresses. The use of the gravity method requires several simplifying assumptions regarding the structural behavior of the dam and the application of the loads (Leclerc et al., 2001):

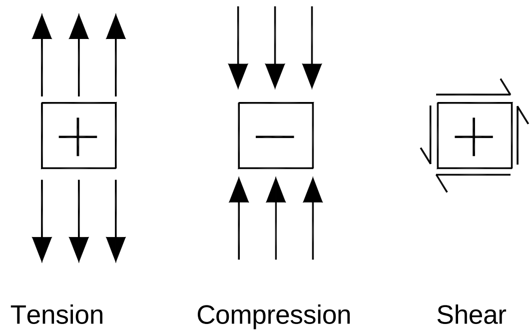
- The dam body is divided into lift joints of homogeneous properties along their length. The mass concrete and lift joints are uniformly elastic,
- All applied loads are transferred to the foundation by the cantilever action of the dam without interactions with adjacent monoliths,
- There is no interaction between the joints; that is each joint is analyzed 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, USBR 1976).

4.2.4 Sign Convention

- Global system of axis: The origin of the global axis system is located at the heel of the dam.
- Local joint axis system: The dam base joint and each lift joint are assigned a local one-dimensional coordinate system, along their lengths. The origin of this local coordinate system is at the upstream face of the dam at the upstream elevation of the joint considered.
- Positive directions of forces and stresses: The positive directions of the forces and moments acting in the global coordinate system are shown in Figure 4.2 (a). The sign convention used to define stresses acting on concrete elements is shown in Figure 4.2 (b).



(a)



(b)

Figure 4.2 Sign Convention

- 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 (See Figure 4.3).

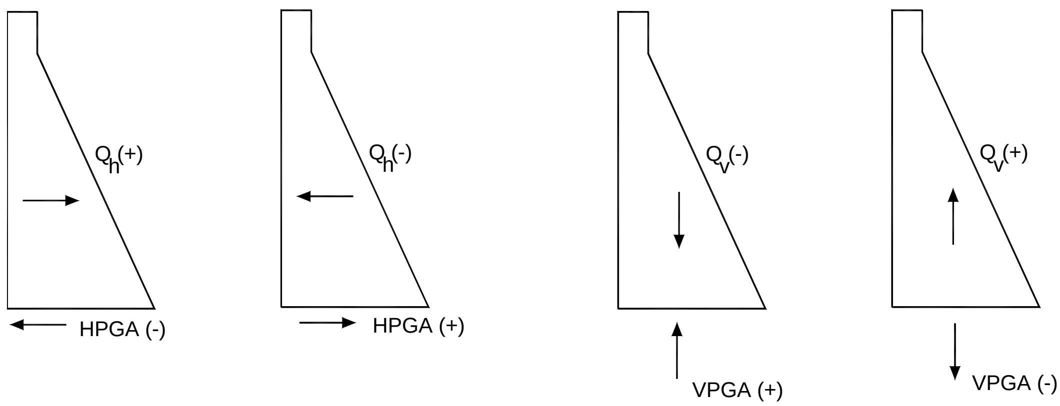


Figure 4.3 Directions of the Inertia Forces (Leclerc et al., 2001)

4.3 INPUTTING DATA

4.3.1 Section Geometry and Basic Data

Basic geometrical dimensions defining the dam cross-section, the system of units, gravitational acceleration, and volumetric mass of concrete are specified at the initial stage.

4.3.2 Concentrated Masses

Concentrated masses option can be used to represent fixed equipment located on the crest or to define holes in the cross-section, or to modify the hydrodynamic forces used in seismic analysis.

4.3.3 Material Properties

4.3.3.1 Lift Joints

A lift joint is a concrete-concrete joint. The material strength properties (compressive strength, tensile strength, and shear strength) of lift joints can be defined using the appropriate option. CADAM allows defining as many materials as needed to describe variations of strength properties along the height of the dam.

Minimal normal compressive stress to mobilize cohesion, σ_n :

Apparent cohesion, Ca, is sometimes specified for an unbounded rough joint (with zero tensile strength) due to the presence of surface asperities. For unbounded joint, it is obvious that the shear strength should be zero if there is no applied normal stress. A minimal value of normal compressive stress can therefore be specified to mobilize Ca along a joint. For normal compressive stresses below the minimal compressive stress (σ_n^*), two options are offered to the user (Leclerc et al., 2001) (See Figure 4.4):

Option 1: The shear resistance (τ) is equal to the normal compressive stress (σ_n) 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 ϕ is used with cohesion (Ca).

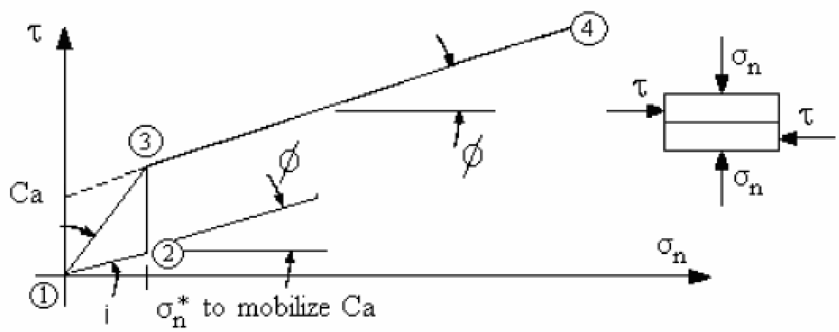


Figure 4.4 Normal Compressive Stress versus Shear Resistance (Leclerc et al., 2001)

When $\sigma_n^* = 0$ or $Ca = 0$, Options 1 and 2 will gave the same results, where the usual two parameters for the Mohr failure envelope is obtained.

4.3.3.2 Base Joint

This option is used to define the material strength properties at the concrete-rock interface.

4.3.3.3 Rock Joint

This option is used when the dam is embedded in the foundation (Figure 4.5). CADAM allows the specification of parameters including the contribution of a passive wedge resistance to the sliding resistance of the dam. The uplift pressures acting on the failure plane is computed automatically if the tailwater elevation is above the rock failure plane.

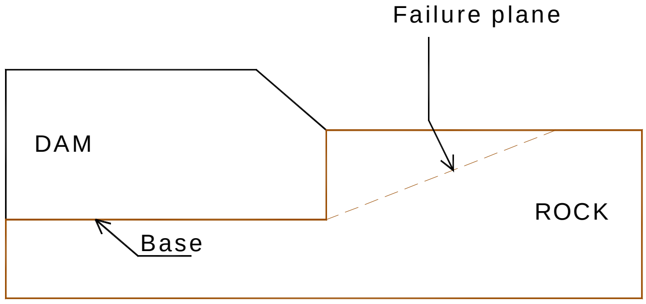


Figure 4.5 Passive Wedge Resistance

The sliding safety factor for a dam-foundation system including a passive wedge resistance should be computed by the shear-friction method (Leclerc et al., 2001).

4.3.4 Reservoir, Ice, Silt and Floating Debris

CADAM allows the user to define the volumetric weight of water, as well as the normal and flood headwater and tailwater elevations.

CADAM allows the user to specify the properties of the silt accumulated along the upstream face of the dam. The force due to sediment accumulation can be determined from the lateral earth pressure. Also, silt can be considered as a fluid. CADAM allows the user to define the linear pressure distribution acting on the crest of the dam during a severe flood. The upstream and downstream pressures are defined in terms of a percentage of the overtopping depth, h using the parameters p_u

and p_d, respectively. In other words, p_u and p_d are the upstream and downstream pressure percentages of the overtopping depth, respectively.

4.3.5 Uplift Pressures and Drainage System

4.3.5.1 Uplift Pressures and Computation of "Effective Stresses"

Uplift pressures should be computed to determine effective stresses and related crack length.

- As an external load acting on the surface of the joint (FERC 1999, USACE 1995, CDSA 1995, USBR 1987): In this case, normal stresses are computed using beam theory considering all loads acting on the free-body considered (including the resultant uplift pressure). 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 observe whether tension cracks initiate or propagate.
- 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 uplift pressure is larger than total stress acting at crack tip (Leclerc et al., 2001).

4.3.5.2 USBR Guidance on Crack Initiation

USBR (1987) uses the following formula to determine crack initiation.

$$\sigma_{zu} = pwh - \frac{f_t}{s} \tag{4.1}$$

where σ_{zu} is the minimum allowable compressive stress at the upstream face from uplift forces, in other words the absolute value of the stress at the upstream face induced from uplift forces minus the allowable tensile stress. In Equation (4.1), 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. Cracking initiates at the heel of the dam when the compressive stress σ_z does not achieve the minimum compressive stress σ_{zu} (Leclerc et al., 2001). CADAM computes the drain reduction factor, p automatically if the USBR guideline is selected. Also, the drain reduction factor p can be specified using Figure 4.6.

The procedure for determining the drain efficiency is as follows:

1. Calculate ratios (X_d / L) and $(H_3 - H_2)/(H_1 - H_2)$
2. Obtain value of *p* from Figure 4.6
3. Correct *p* for tailwater using equation $\left[p(H_1 - H_2) + H_2\right] / H_1$

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

H_d : distance to the drain from the upstream face

L : horizontal length from upstream to downstream face as shown in Figure 4.7)

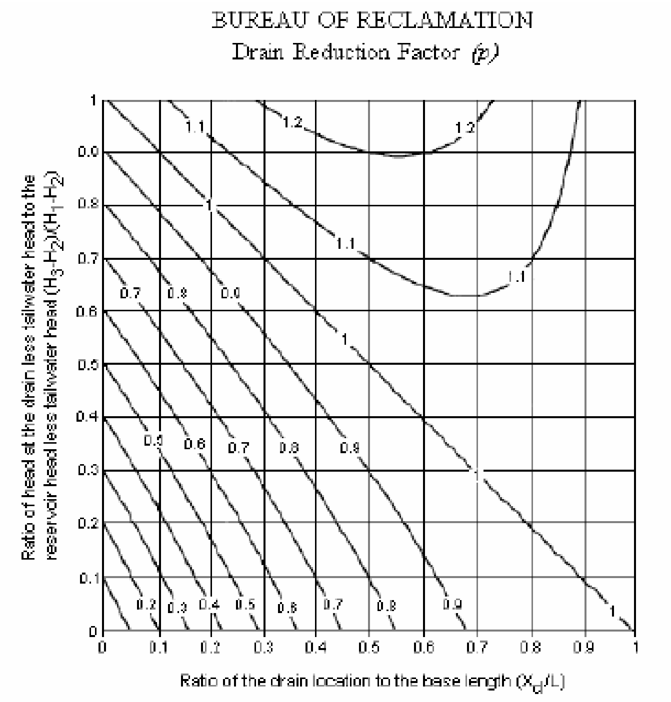


Figure 4.6 Determination of Drain Reduction Factor (p) (Leclerc et al., 2001; Source: USACE, 1995)

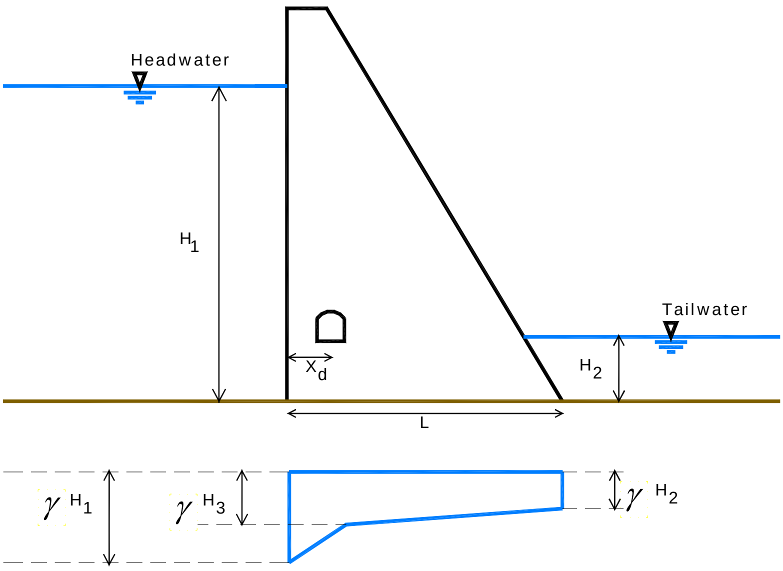


Figure 4.7 The Illustration of Uplift Pressure Distribution (USBR, 1987)

4.3.6 Applied Forces

CADAM allows the definition of active external forces acting within or outside the dam body using "applied forces" option.

4.3.7 Psuedo – Static Seismic Analysis

4.3.7.1 Basic Assumption – Rigid Body Behavior

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

Each seismic analysis begins with a static analysis in order to determine the initial condition before applying the seismically induced inertia forces. If cracking is taking place under the static-load conditions, the crack length and updated uplift pressures are considered as initial conditions for the seismic analysis (Leclerc et al., 2001).

4.3.7.2 Seismic Analysis

The seismic analysis includes two stages. Successively a stress analysis and then a stability analysis are performed. The procedure is summarized in Figure 4.8. The basic objective of the stress analysis is to determine the tensile crack length over which cohesion will be applied in the stability analysis. The main 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 0.50 of the peak acceleration values.

In this case, the sliding safety factors are computed considering crack lengths determined from the stress analysis (Leclerc et al., 2001).

4.3.7.3 Hydrodynamic Pressures (Westergaard Added Masses)

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

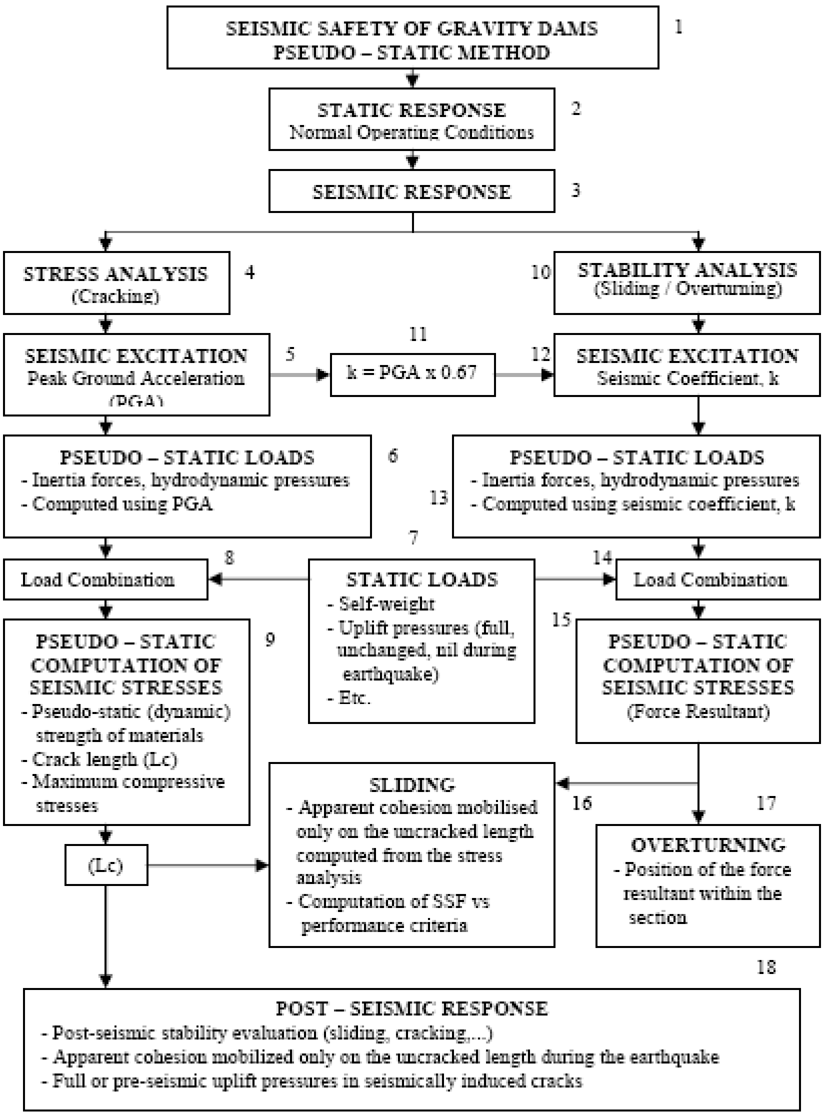


Figure 4.8 Psuedo- Static Seismic Analysis (Leclerc et al., 2001)

● 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 are acting 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 (Priscu et al. 1985) or the function derived from USBR (1987) as given by Corns et al. 1988 (Figure 4.9).

● A reservoir depth beyond which Westergaard added pressure remains constant: This option allows to experiment with some dam safety guideline requirements indicating that 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.

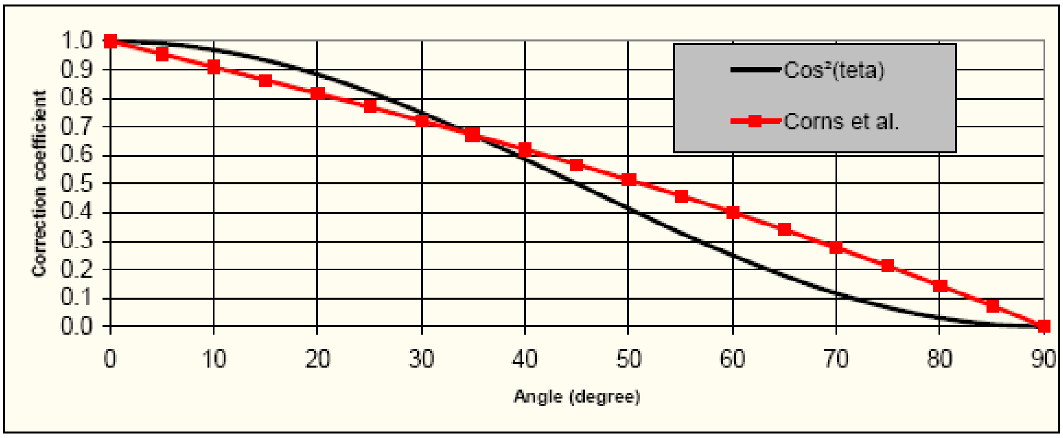


Figure 4.9 Correction factor (Ka) adopted from Corns et al. (1988)

4.3.8 Psuedo – Dynamic Seismic Analysis

4.3.8.1 Basic Assumption – Dynamic Amplification

The pseudo-dynamic analysis is based on Chopra's simplified response spectra method. A pseudo-dynamic analysis is conceptually similar to a pseudo-static analysis except that it recognizes 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 the stress and the stability analyses are performed with the inertia forces continuously applied in the same direction (Leclerc et al., 2001).

4.3.8.2 Seismic Accelerations

CADAM assumes in the dynamic analysis that the dynamic amplification applies only to the horizontal rock acceleration. The period of vibration of the dam in the vertical direction is considered sufficiently small to neglect the amplification of vertical ground motions along the height of the dam.

4.3.8.3 Dam Properties

To ensure 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 damping ratio (ξ_1) on rigid foundation without reservoir interaction is necessary to compute the dam foundation reservoir damping(ξ_1). Any change to these basic parameters affects the fundamental period of vibration and the damping of the dam-foundation-reservoir system computed in this dialog window (Leclerc et al., 2001).

4.3.8.4 Reservoir Properties

- The wave reflection coefficient (α) 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 α indicate increasingly absorptive materials.
- The velocity of pressure waves in water is in fact the speed of sound in water. Generally, it is assumed at 1440 m/sec (4720 ft/sec).
- 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).

4.3.8.5 Foundation Properties

Dam-foundation rock interaction modifies the fundamental period of vibration and added damping of the equivalent SDF system representing the fundamental vibration mode response of the dam. The foundation hysteretic damping (η_f) will affect the damping ratio of the dam foundation reservoir system (Leclerc et al., 2001).

4.3.8.6 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 the (iv) Sum of absolute values which provides always conservative results. The SRSS combination is often considered to be preferable (Leclerc et al., 2001).

4.3.9 Cracking Options

4.3.9.1 Basic Assumption – Rigid Body Behaviour

The analysis can be performed assuming linear elastic properties without any possibility for concrete cracking. When a cracking is allowed, the tensile strength to be used to determine the cracking response along the joints should be specified.

Cracking response along the joints can be defined by two criteria, crack initiation and crack propagation. The tensile crack initiation stress can be specified:

$$ft_{ini} = ft_{joint} / \kappa_{ini} \tag{4.2}$$

and the allowable tensile strength for crack propagation is given by:

$$ft_{prop} = ft_{joint} / \kappa_{prop} \tag{4.3}$$

The user defined coefficients κ_{ini} and κ_{prop} for cracking is given Table 4.1.

Table 4.1 User Defined Coefficients for Cracking (Leclerc et al., 2001)

Coefficients	Usual	Flood	Seismic	Post-Seismic
κ_{ini}	3	2	1	3
κ_{prop}	10	10	10	10

When cracking is allowed, a distinction is made between the criteria for crack initiation and crack propagation. 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).

The tensile strength of concrete under rapid loading during a seismic event is greater than that under static loading. The tensile strength can 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).

4.3.10 Load Combinations

4.3.10.1 Load Combinations and Load Conditions

CADAM allows analyzing 5 load combinations, which are usual, flood, pseudo-static seismic, pseudo-dynamic analyses, and post-seismic conditions.

4.3.10.2 Required Safety Factors

The required safety factors to ensure an adequate safety margin for structural stability are specified. Values of the safety factors are presented in Table 4.2.

Table 4.2 Safety Factors for Different Load Combinations (Leclerc et al., 2001)

Safety Cases	Usual	Flood	Seismic	Post-Seismic
Peak Sliding Factors (PSF)	3.00	2.00	1.30	2.00
Residual Sliding Factor (RSF)	1.50	1.30	1.00	1.10
Overturning Factor (OF)	1.20	1.10	1.10	1.10
Uplifting Factor (UF)	1.20	1.10	1.10	1.10

4.3.10.3 Allowable Stress Factors

Allowable stresses can be defined by applying multiplication factors to the tensile and compressive strengths. These values are not used in the computational algorithm of the program. Values of the allowable stress factors are presented in Table 4.3.

Table 4.3 Allowable Stress Factors for Different Load Combinations (Leclerc et al., 2001)

Safety Cases	Usual	Flood	Seismic	Post-Seismic
Allowable Stress Factor in tension	0.00	0.50	0.909	0.667
Allowable Stress Factor in compression	0.333	0.50	0.909	0.667

4.3.11 Probabilistic Safety Analysis (Monte-Carlo Simulations)

4.3.11.1 Overview of CADAM Probabilistic Analysis Module

Objectives: The objective 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 "involves sampling at random to simulate artificially a large number of experiments and to observe the results"

(Leclerc et al., 2001). Figure 4.10 summarizes the probabilistic safety analysis procedure in CADAM.

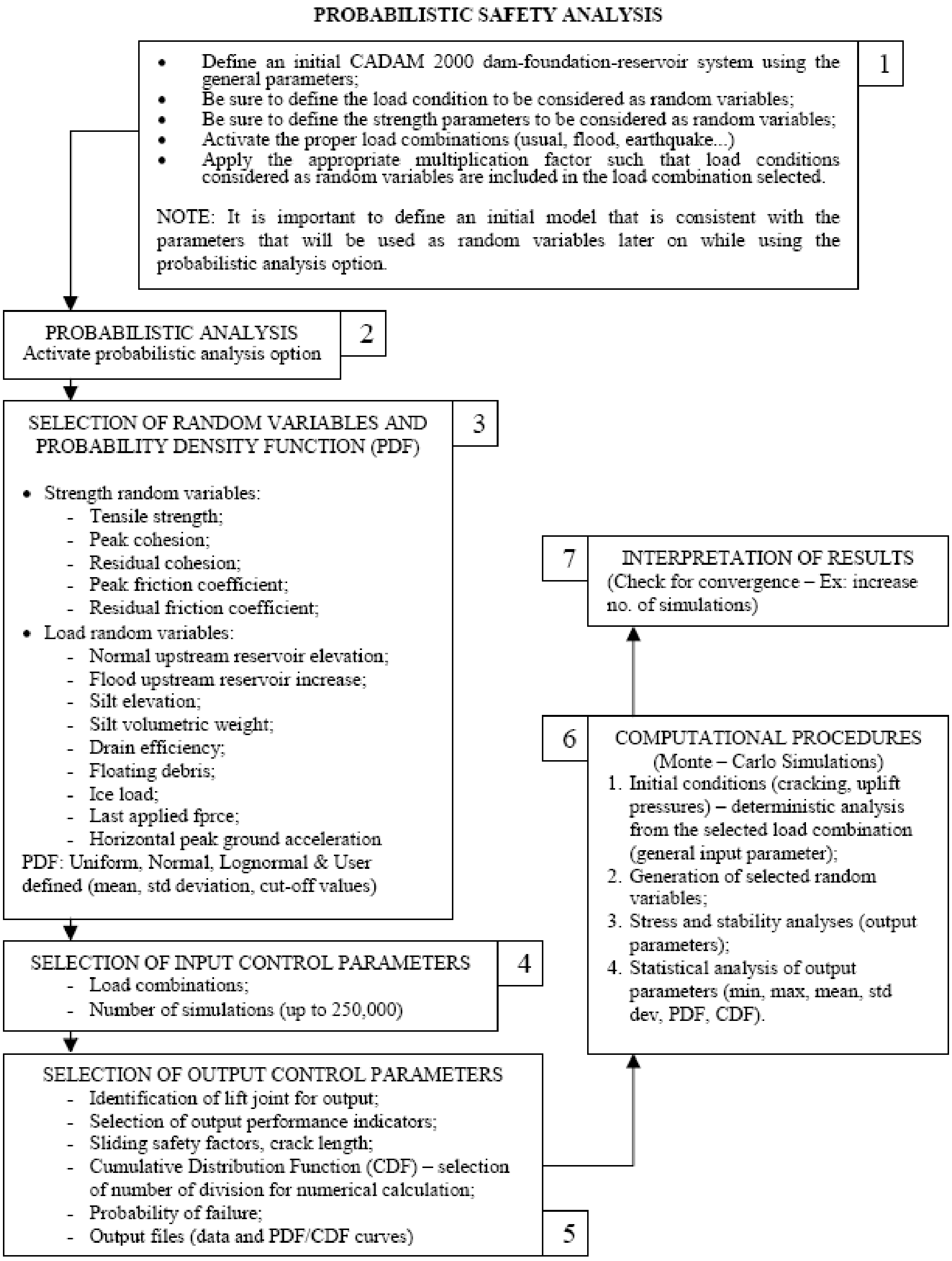


Figure 4.10 Probabilistic Safety Analysis Procedure in CADAM

4.3.11.2 Probability Density Function (PDF)

To perform a probabilistic safety analysis in CADAM, some load conditions and/or strength parameters must be specified as random variables. Strength and loads parameters that are treated as random variables must be independent. The dependent variables are considered as:

- Upstream reservoirs (normal and flood) will affect the crest vertical water pressure, normal downstream reservoir elevation, floating debris and ice load.
- The horizontal peak ground acceleration will change all dependent accelerations (VPGA, HPSA, HSGA, VSGA and HSSA). These parameters will be scaled proportionally to the ratio between the generated horizontal peak ground acceleration and the initial horizontal peak ground acceleration.

Probability distribution functions (PDF) available in CADAM are uniform distribution, normal distribution, and log-normal distribution. Also, CADAM allows the user to provide his own PDF by importing data points from a text file (ASCII).

4.3.11.3 CADAM Input Parameters for a Probabilistic Analysis

The list of random variables is composed of:

- Strength Variable Parameters: Tensile strength, peak cohesion, residual cohesion, peak friction coefficient and residual friction coefficient.
- Loading Variable Parameters: Normal upstream reservoir elevation, flood upstream reservoir increase, silt elevation, silt volumetric weight, drain efficiency, floating debris, ice load, last applied force, and 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 (Leclerc et al., 2001).

4.3.12 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 years event or Probable Maximum Flood (PMF), and (b) the peak ground acceleration (PGA) (spectral ordinates) corresponding to the 10,000 years 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 can be planned. The reservoir elevation or PGA (spectral ordinate) that will induce failure can also be readily evaluated (safety factors just below unity). The concept of imminent failure flood is used in dam safety guidelines. A parallel can be established with earthquakes where the concept of imminent failure earthquake (ground motion) can 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).

4.4 Stress and Stability Analysis

4.4.1 Performing the Structural Analysis

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 (Leclerc et al., 2001).

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

4.4.2 Stress Analysis and Crack Length Computations

Stress analysis in CADAM is performed as discussed in Section 3.2.

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 the iterative procedure summarized in Figure 4.11 to compute the crack length (Leclerc et al., 2001).

4.4.3 Stability Analysis

Stability analysis in CADAM is performed as discussed in Section 3.2.

4.4.4 Safety Evaluation for Static Loads

By proper definition of basic loading condition parameters and multiplication factors to form load combinations, a variety of loading scenarios can be defined to assess the safety of the dam-foundation-reservoir system (Leclerc et al., 2001):

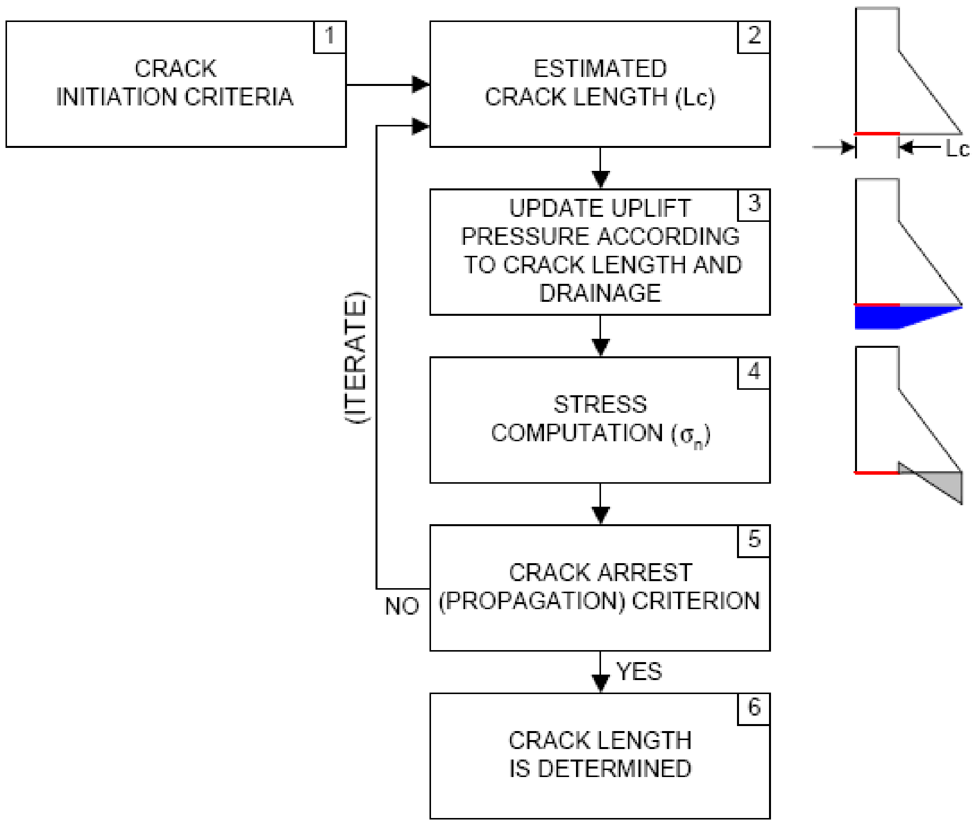


Figure 4.11 Iterative Procedure for Crack Length Computations (Leclerc et al.,2001)

Increasing applied load to induce failure: Different strategies have been adopted to study the safety margin of concrete dams as a function of the uncertainties in the applied loading and material strength parameters. In some cases, the applied loads are increased to induce failure, upstream, downstream water levels are increased, ice loads, water density etc. The safety margin is then assessed by comparing the magnitude of the load inducing failure with that of the applied load for the combination under study. CADAM can be used effectively to perform this type of study using a series of analyses while increasing the applied loads either through the basic loading input parameters or by applying appropriate load condition multiplication factors while forming the load combinations or by activating the incremental load analysis option.

Reducing material strength to induce failure: In a different approach, the specified strength of material is reduced while inputting basic data (friction coefficient (tan φ), cohesion, tensile strength, etc.). Series of analyses are then performed until a safety factor of 1 is reached for particular failure mechanisms. Comparing the material strength inducing failure to the expected material strength can then assess the safety margin.

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 can be used to perform limit analysis of gravity dams as described by ANCOLD (1991).

4.4.5 Safety Evaluation for Seismic Loads

Concrete inertia forces and hydrodynamic forces in CADAM are computed as discussed in Section 3.1.

Dynamic Silt pressures: Different approaches based on soil dynamics can be used to evaluate the hydrodynamic thrust developed by the silt. As a first approximation CADAM uses a two layer fluid model along the upstream face. It is thus assumed that there is liquefaction of the silt during the earthquake. The silt is considered as a liquid with a density larger than that of water. The Westergaard formulation is then used to compute the added mass (FERC 1991). The use of Westergaard solution for the silt is an approximation to more rigorous solutions considering the two layer fluid model, as those presented by Chen and Hung (1993). In that context, the active earth pressure for the static thrust component is questionable. If the assumption of a two layer fluid model is retained, it would be appropriate to use $K = 1$ (silt=fluid) for the static condition. The oscillatory motion of the u/s face is thus assumed to liquefy the silt layer in contact with the dam. As for the reservoirs, the dynamic silt pressure is influenced by an inclination of the upstream face of the dam. CADAM applies the same rules for slope correction to dynamic silt pressure distribution as for reservoirs (Leclerc et al., 2001).

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 $(\gamma_e = \rho_w(g + accV))$ for an incompressible reservoir, where ρ_w is the volumetric mass of water and g is the acceleration of gravity. 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 $(\gamma_e = \rho_w(g - accV))$ and related initial hydrostatic pressures. These considerations are independent of the Westergaard hydrodynamic pressure computations. CADAM includes the effect of the vertical rigid body acceleration of the reservoir bottom on the initial hydrostatic pressures (Leclerc et al., 2001).

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: 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) mentions: When a crack develops during an earthquake event, uplift pressure within the crack is assumed to be zero.
• CDSA (1997) mentions: 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 force are tending to open the crack.

CADAM provides three options to consider the transient evolution of uplift pressures in cracks during earthquakes (Figure 4.12): (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).

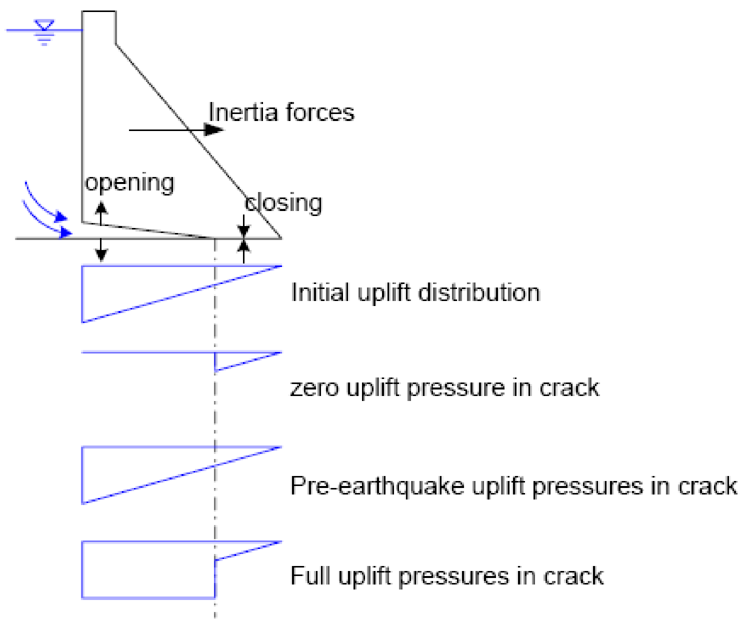


Figure 4.12 Transient Evolutions of Uplift Pressures in Seismically Induced Crack (Leclerc et al.,2001)

4.4.6 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 can 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).

To sum up, CADAM provides many modeling options to define the geometry, material properties and loading parameters necessary to explore the structural behavior of gravity dams. As a first step, the section geometry can be defined using definition points. Strength parameters of materials for peak and residual cases can be assigned to lift, base, and rock joints. Reservoir elevations during the operating or the flood can be defined. Also, ice load, floating debris, and silt properties can be entered in CADAM. Drain location and effectiveness needed to compute uplift pressures can be identified using different specifications defined in CADAM. In addition, user defined horizontal and vertical forces can be entered in the program. Also, tensile strengths for crack initiation and propagation can be specified using the cracking options.

In order to perform pseudo-seismic analysis, peak ground horizontal and vertical accelerations, as well as sustained accelerations should be specified. For pseudo-dynamic analysis, the spectral acceleration coefficient is also required. Pseudo-static and pseudo dynamic analyses are conceptually similar except that the dynamic amplification of inertia forces along the height of the dam is not neglected in pseudo-dynamic analysis. The pseudo-dynamic analysis is based on the simplified response spectra method as described by Chopra (1988).

The evaluation of the structural stability of the dam is performed considering the stress analysis to determine crack lengths and compressive stresses and the stability analysis to determine the safety margins and the resultant forces.

In order to perform, probabilistic analysis, uncertainties in parameters can be identified by assigning the probability density functions and the coefficient of variations to these variables. Then, it is possible to determine the probability of failure of a dam-reservoir-foundation system using Monte-Carlo simulations.

Besides the capabilities mentioned above, the program have some important limitations. The most important one is the inadequate number of definition points. Another important limitation is that there is no option to define fill materials for both upstream and downstream of the dam. These limitations make assumptions necessary.

CHAPTER 5

CASE STUDY: CİNDERE DAM

The reliability based safety analyses are carried out for Cindere Dam in Turkey. Cindere Dam is the first hardfill dam of Turkey. It is located 5 km northwest of Güney district of Denizli, on Büyük Menderes River (See Figure 5.1). The main purpose of the dam is to supply irrigation water and energy. It is aimed to irrigate 4600 ha area and to generate 88 GWh energy in a year. The construction had started in 1995 and completed in 2007.

5.1 Input Data

The general characteristics of Cindere Dam necessary for the stability analysis are given in Table 5.1.

5.1.1 Determination of Spectral Acceleration Coefficient

Spectral acceleration coefficient is required by CADAM for the pseudo-dynamic seismic analysis. In order to determine the spectral acceleration coefficient, "Specification for Structures to Build in Disaster Areas" published by Ministry of Public Works and Settlements in Turkey can be used. The spectral acceleration coefficient is computed as discussed in Chapter 3. The values necessary to determine the spectral acceleration coefficient is as follows:

Horizontal peak ground acceleration is determined as 0.40 g (maximum design earthquake) which has a return period of 2000 years and has a 5% probability of exceedance in 100 years (Temelsu Müh.Hiz. A.Ş., 2000).

Table 5.1 Input Data for Stability Analysis of Cindere Dam

Characteristics	
Crest elevation	272.00 m (Temelsu Müh.Hiz. A.Ş., 2000)
Elevation of foundation	165.00 m (Temelsu Müh.Hiz. A.Ş., 2000)
Height from foundation	107.00 m (Temelsu Müh.Hiz. A.Ş., 2000)
Crest thickness, T_c	10.00 m (Temelsu Müh.Hiz. A.Ş., 2000)
Upstream face slope,m	0.70 (Temelsu Müh.Hiz. A.Ş., 2003)
Downstream face slope,n	0.70 (Temelsu Müh.Hiz. A.Ş., 2003)
Bottom width, B	142.30 m (Temelsu Müh.Hiz. A.Ş., 2003)
Normal reservoir level, H_n	256.50 m (Temelsu Müh.Hiz. A.Ş., 2000)
Maximum reservoir level, H_m	267.70 m (Temelsu Müh.Hiz. A.Ş., 2000)
Normal water level at the downstream	216.00 m (Temelsu Müh.Hiz. A.Ş., 2000)
Maximum water level at the downstream	217.50 m (Temelsu Müh.Hiz. A.Ş., 2000)
Height of fill material	200.00 m (Temelsu Müh.Hiz. A.Ş., 2000)
Specific weight of fill material	21 kN/m^3(assumed)
Height of sediment accumulation	220.00 m (DSİ, 1986)
Submerged specific weight of sediment material	11 kN/m^3 (assumed)
Angle of repose of sediment	31^o (Yanmaz, 2006)
Specific weight of concrete	25 kN/m^3 (Temelsu Müh.Hiz. A.Ş., 2003)
Horizontal peak ground acceleration (Maximum Design Earthquake, MDE)	0.40 g (Temelsu Müh.Hiz. A.Ş., 2000)
Horizontal seismic coefficient for MDE	0.20 (Temelsu Müh.Hiz. A.Ş., 2000)
Drain position	12.50 m from the heel (Temelsu Müh.Hiz. A.Ş., 2003)
Drain elevation	5.50 m (Temelsu Müh.Hiz. A.Ş., 2003)
Drain effectiveness	0.67 (USACE, 1995)
Compressive strength of hardfill concrete	6000 kPa (Temelsu Müh.Hiz. A.Ş., 2003)
Tensile strength of hardfill concrete	600 kPa (Temelsu Müh.Hiz. A.Ş., 2003)
Internal friction angle of hardfill concrete	45^o (peak) (Temelsu Müh.Hiz. A.Ş., 2003)
Cohesion of hardfill concrete	800 kPa (Temelsu Müh.Hiz. A.Ş., 2003)
Compressive strength of foundation	8000 kPa (Temelsu Müh.Hiz. A.Ş., 2003)
Tensile strength of foundation	800 kPa (Temelsu Müh.Hiz. A.Ş., 2003)
Internal friction angle of foundation	25^o (peak) (Temelsu Müh.Hiz. A.Ş., 2003) , 20^o (residual) (Hunt, 1984)
Cohesion of foundation	200 kPa (Temelsu Müh.Hiz. A.Ş., 2003)

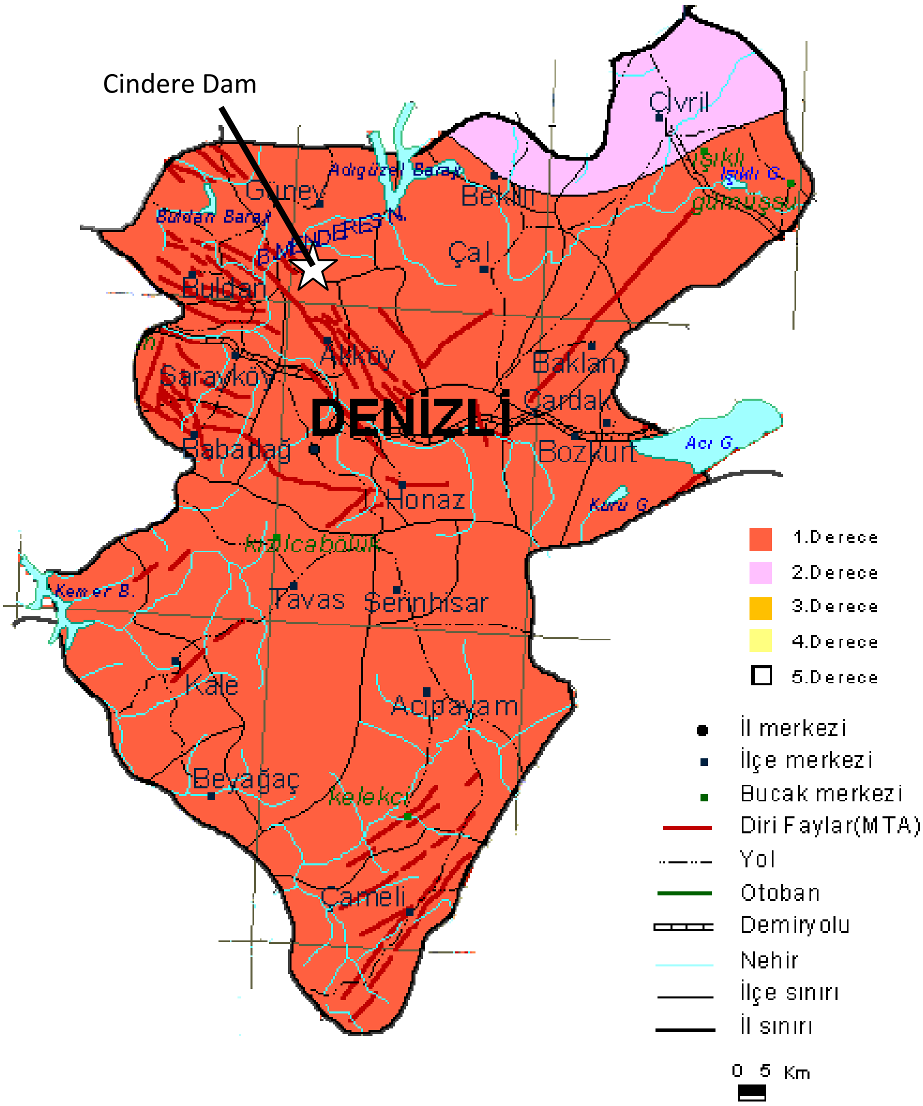


Figure 5.1 Earthquake Zones of Denizli (GDDAERD, 2010)

Site classification is Z_1 ($T_A = 0.10$ s, $T_B = 0.30$ s)

The fundamental vibration period, T= 0.601; $T > T_B$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} = 1.434$$

Therefore, the spectral acceleration coefficient is:

$A(T)$ = (0.4) x (1.434) =0.574

5.2 Input Parameters for Probabilistic Analysis

In order to perform a probabilistic analysis, uncertainties associated with loading and strength parameters need to be treated by utilization of random variables for these parameters. The uncertainties associated with these parameters are propagated into the probability of failure of the dam through probabilistic analysis. For probabilistic safety analyses, probability density functions, mean values, and standard deviation of the random variables must be specified.

Random variables are identified by considering available data. In this study, tensile strength, peak cohesion, and peak friction coefficient are modeled as random strength variables and normal upstream reservoir elevation, ice load, drain efficiency and horizontal peak ground acceleration are modeled as random load variables. The mean values, standard deviations, and probability density functions of these random variables are given in Table 5.2. In the analysis conducted, cut-off values defining the lower bound and upper bound are kept within three standard deviations of the mean. A data range within three standard deviations corresponds to a 99.73% confidence interval.

Table 5.2 Random Variables Utilize for the Probabilistic Analysis

Variable	μ	σ	δ	PDF	Reference
Tensile Strength (kPa)	600	60	0.10	Normal	Ang and Tang (1990)
Peak Cohesion (kPa)	200	10	0.05	Normal	Assumed
Peak Friction Coefficient	0.466	0.0186	0.04	Normal	Ang and Tang (1990)
Normal Upstream Reservoir Elevation (m)	91.50	3.386	0.037	Normal	Beşer (2005)
Drain Efficiency	0.67	0.067	0.10	Normal	Assumed
Horizontal Peak Ground Acceleration (g)	0.4	0.1	0.25	Normal	Ang and Tang (1990)

5.3 Loading Assumptions

- Due to limitations of the of definition points for geometrical data in CADAM, the dam body cross-section is approximated as shown in Figure 5.2. Approximation is marked with the red line. Since the downstream wedge is ignored, the computations are accepted to be on the safer side.

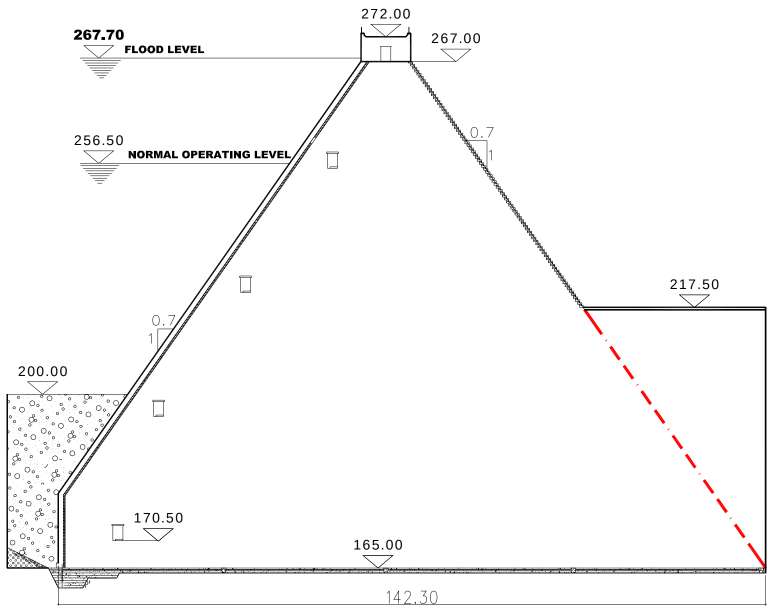


Figure 5.2 The maximum cross-section of Cindere Dam

- The vertical component of the ground acceleration is considered 2/3 of the horizontal component (RTMPWS, 1997).
- There is no additional option to define fill material properties in CADAM software. Force due to fill material at the upstream face of the dam is computed from active earth pressure formula as is the case with the computation of forces due to silt accumulation. In addition, fill material and silt accumulated at the upstream face of Cindere Dam have nearly same characteristics (unit weight, angle of repose). Therefore, the fill material was defined as silt, that is, the silt height is calculated from the base of the dam.

5.4 CADAM Output and Results

Probabilistic analysis is carried out for 6 different cases:

1. Seismic-1 Combination with no water
2. Usual Combination
3. Flood Combination
4. Seismic-1 Combination with water
5. Seismic-2 Combination
6. Post-seismic Combination

Seismic-1 combination with no water refers to pseudo-static seismic analysis with empty reservoir, and this combination includes dead load and earthquake force on the dam body.

Usual combination includes dead load, vertical and horizontal hydrostatic forces produced by a reservoir at normal operating level, uplift force, and force due to sediment accumulation.

Flood combination includes dead load, vertical and horizontal hydrostatic forces produced by a reservoir at flood level, uplift force, and force due to sediment accumulation.

Seismic-1 combination with water includes dead load, vertical and horizontal hydrostatic forces produced by a reservoir at normal operating level, uplift force, and forces due to sediment accumulation, earthquake force on the dam body, and hydrodynamic force in the reservoir induced by earthquake.

In pseudo-static seismic analysis expressed as Seismic-1 analysis, the dynamic amplification of the 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. A pseudo-dynamic analysis which is expressed as Seismic-2 combination is conceptually similar to a pseudo-static analysis except that it recognizes the dynamic amplification of the inertia forces along the height of the dam (Leclerc et al., 2001).

Post-seismic combination includes dead load, vertical and horizontal hydrostatic forces produced by a reservoir at normal operating level, uplift force, and force due to sediment accumulation. The cohesion is disregarded along the seismically induced crack length to compute the sliding safety factors in post-seismic condition.

The results of the probabilistic analysis (Monte-Carlo simulations) are given in Tables 5.3 to 5.8 for each case as listed above. In these analyses, the upstream crack length L_c as percent of the dam width at the corresponding joint elevation, the sliding safety factors for peak (PFS) and residual conditions (RFS), overturning safety factors considering toe (OF_t) and heel (OF_h), uplifting safety factors (UF), reliability indexes (β) and probability of failures (P_f) are computed for the aforementioned loading combinations. The safety factors are presented in terms of their minimum, maximum, and mean values. Reliability index is a measure of reliability, which is the ratio of the mean to the standard deviation of the safety margin. The model properties (input data), the loads and stability and stress analyses' results are given in Appendix A. In this study, L_c is chosen at the base joint.

Table 5.3 Results of Probabilistic Analysis (Seismic-1 Combination with no water)

Parameter	**Mean**	**Standard Deviation**	**Minimum Value**	**Maximum Value**	**β**	$\mathbf{P_f}$
L_c	2.135	10.384	0.000	90.210	1.00000	0.00000
PSF	2.408	0.863	0.787	10.367	0.99530	0.00470
RSF	1.400	0.512	0.629	5.289	0.82276	0.17724
OF_h	7.892	2.198	4.577	26.920	1.00000	0.00000
OF_t	3.874	1.203	2.059	14.290	1.00000	0.00000

Table 5.4 Results of Probabilistic Analysis (Usual Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	P_f
L_c	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	3.080	0.285	2.280	4.452	1.00000	0.00000
RSF	1.741	0.149	1.321	2.322	1.00000	0.00000
OF_h	2.915	0.047	2.782	3.109	1.00000	0.00000
OF_t	2.568	0.073	2.263	2.853	1.00000	0.00000
UF	2.949	0.062	2.722	3.150	1.00000	0.00000

Table 5.5 Results of Probabilistic Analysis (Flood Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	2.388	0.079	2.092	2.708	1.00000	0.00000
RSF	1.354	0.018	1.300	1.408	1.00000	0.00000
OFh	2.908	0.053	2.755	3.076	1.00000	0.00000
OFt	2.373	0.054	2.218	2.548	1.00000	0.00000
UF	2.868	0.070	2.668	3.095	1.00000	0.00000

Table 5.6 Results of Probabilistic Analysis (Seismic-1 Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	23.505	37.006	0.000	100.000	0.91826	0.08174
PSF	1.018	0.308	0.365	2.452	0.59842	0.40158
RSF	0.574	0.130	0.294	1.379	0.00788	0.99212
OFh	2.359	0.107	2.042	2.870	1.00000	0.00000
OFt	1.639	0.154	1.244	2.344	1.00000	0.00000
UF	2.107	0.155	1.681	2.798	1.00000	0.00000

Table 5.7 Results of Probabilistic Analysis (Seismic-2 Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	27.495	37.431	0.000	100.000	0.92142	0.07858
PSF	1.058	0.319	0.387	2.476	0.61594	0.38406
RSF	0.607	0.131	0.319	1.372	0.01068	0.98932
OFh	2.417	0.098	2.130	2.866	1.00000	0.00000
OFt	1.639	0.154	1.244	2.313	1.00000	0.00000
UF	2.156	0.150	1.748	2.780	1.00000	0.00000

Table 5.8 Results of Probabilistic Analysis (Post-seismic Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	3.081	0.285	2.249	4.311	1.00000	0.00000
RSF	1.741	0.150	1.341	2.326	1.00000	0.00000
OFh	2.915	0.047	2.782	3.127	1.00000	0.00000
OFt	2.568	0.074	2.292	2.843	1.00000	0.00000
UF	2.949	0.063	2.731	3.154	1.00000	0.00000

For seismic-1 with empty reservoir, the probability of failure against peak sliding condition is found to be 0.47%, which is very low. For usual, flood and post-seismic combinations, no crack is developed and the aforementioned safety factors are found to be greater than the limiting values presented in Table 4.2. The reliability indexes for these loadings are computed as unity. Therefore, the dam is infinitely safe. For seismic-1 and seismic-2 combinations as can be seen in Tables 5.6 and 5.7, the crack lengths exceeded mean value of 20% of the base width of the dam. For these conditions, the peak and residual sliding safety factors are found to be relatively low. Although the overturning safety factors are greater than the limiting values, they have no practical importance. The probability of failure against peak sliding condition is about 40%, which is quite high. Therefore, the dam is in very critical condition for these loading combinations. For post-seismic combination, all the safety factors are found greater than the limiting values. Since Cindere Dam is an existing structure, it is recommended to inspect the faces of the dam periodically to observe possible cracks that may be induced if seismic-1 and seismic-2 prevail during the lifetime of the structure. Information obtained from jointmeters and crackmeters of the dam may also give clues about crack formation and propagation. The details of the instrumentation system of Cindere Dam can be found in Yanmaz and Sezgin (2008). As a final remark, the impacts of the concrete wedge element and the fill material at the downstream face are ignored. If these elements were included in the analysis, the sliding instability might have been improved.

5.5 Sensitivity Analysis

In the future, additional data about random variables may become available and the coefficient of variations and associated probability density functions may need to be updated. The sensitivity analyses are conducted in order to observe the impacts of such variations in statistical information.

The sensitivity analyses are performed using seismic-1 load combination in CADAM. Coefficient of variation of each random variable is increased by 10%, 20% and 30%. The input data corresponding to these increases are given in Tables 5.9 through 5.11.

Table 5.9 Sensitivity Analysis Input Data for 10% Increased in δ

Variable	**μ**	**σ**	**δ**	**PDF**
Tensile Strength (kPa)	600	66	0.11	Normal
Peak Cohesion (kPa)	200	11	0.055	Normal
Peak Friction Coefficient	0.466	0.0205	0.044	Normal
Normal Upstream Reservoir Elevation (m)	91.5	3.724	0.0407	Normal
Drain Efficiency	0.67	0.0737	0.11	Normal
Horizontal Peak Ground Acceleration	0.4	0.11	0.275	Normal

Table 5.10 Sensitivity Analysis Input Data for 20% Increased in δ

Variable	**μ**	**σ**	**δ**	**PDF**
Tensile Strength (kPa)	600	72	0.12	Normal
Peak Cohesion (kPa)	200	12	0.06	Normal
Peak Friction Coefficient	0.466	0.0224	0.048	Normal
Normal Upstream Reservoir Elevation	91.5	4.063	0.0444	Normal
Drain Efficiency	0.67	0.0804	0.12	Normal
Horizontal Peak Ground Acceleration	0.4	0.12	0.3	Normal

Table 5.11 Sensitivity Analysis Input Data for 30% Increased in δ

Variable	μ	σ	δ	PDF
Tensile Strength (kPa)	600	78	0.13	Normal
Peak Cohesion (kPa)	200	13	0.065	Normal
Peak Friction Coefficient	0.466	0.0242	0.052	Normal
Normal Upstream Reservoir Elevation	91.5	4.401	0.0481	Normal
Drain Efficiency	0.67	0.0871	0.13	Normal
Horizontal Peak Ground Acceleration	0.4	0.13	0.325	Normal

Results corresponding to initially selected coefficient of variations are given in Table 5.12. Results corresponding to 10%, 20% and 30% increases in coefficient of variations are given in Tables 5.13, 5.14 and 5.15, respectively. A summary of the results are provided in Table 5.16 for comparison purposes.

Table 5.12 Output with the Initial Coefficient of Variation

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	23.505	37.006	0.000	100.000	0.91826	0.08174
PSF	1.018	0.308	0.365	2.452	0.59842	0.40158
RSF	0.574	0.130	0.294	1.379	0.00788	0.99212
OFh	2.359	0.107	2.042	2.870	1.00000	0.00000
OFt	1.639	0.154	1.244	2.344	1.00000	0.00000
UF	2.107	0.155	1.681	2.798	1.00000	0.00000

Table 5.13 Output with 10% Increased Coefficient of Variation

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	P_f
L_c	25.213	38.380	0.000	100.000	0.89960	0.10040
PSF	1.025	0.340	0.335	2.666	0.59078	0.40922
RSF	0.580	0.149	0.278	1.507	0.01628	0.98372
OF_h	2.362	0.119	2.031	2.910	1.00000	0.00000
OF_t	1.644	0.172	1.215	2.425	1.00000	0.00000
UF	2.111	0.173	1.645	2.841	1.00000	0.00000

As can be seen from Tables 5.12, 5.13, 5.14 and 5.15, there is no significant change in safety factors and probability of failures with the increase in coefficient of variations. The upstream crack lengths as percent of dam width range from approximately 23.5 to 28.6. The probability of failure against peak sliding condition changes between 40% and 43%. Therefore, it can be concluded that the effect of increase in the coefficient of variation is not significant in the overall stability.

Table 5.14 Output with 20% Increased Coefficient of Variation

Parameter	**Mean**	**Standard Deviation**	**Minimum Value**	**Maximum Value**	**β**	**Pf**
Lc	26.974	36.667	0.000	100.000	0.87836	0.12164
PSF	1.029	0.369	0.312	3.027	0.57992	0.42008
RSF	0.585	0.165	0.256	1.697	0.02474	0.97526
OFh	2.363	0.130	2.008	3.014	1.00000	0.00000
OFt	1.646	0.189	1.180	2.566	1.00000	0.00000
UF	2.113	0.189	1.616	3.012	1.00000	0.00000

Table 5.15 Output with 30% Increased Coefficient of Variation

Parameter	**Mean**	**Standard Deviation**	**Minimum Value**	**Maximum Value**	**β**	**Pf**
Lc	28.609	40.735	0.000	100.000	0.85966	0.14034
PSF	1.034	0.406	0.297	3.613	0.57074	0.42926
RSF	0.591	0.186	0.246	2.116	0.03482	0.96518
OFh	2.366	0.142	1.985	3.030	1.00000	0.00000
OFt	1.650	0.208	1.160	2.750	1.00000	0.00000
UF	2.117	0.207	1.580	3.041	1.00000	0.00000

Percent changes of safety factors and probability of failure with the increase in coefficient of variation is illustrated in Figures 5.3 and 5.4, respectively. Since the failure probabilities for uplifting and overturning towards the upstream and downstream are zero, these are not included in Figure5.6. δ represents coefficient of variation in initial analysis and δ_i/δ represents the increase ratio in coefficient of variation.

Table 5.16 Summary of Sensitivity Analyses

Safety Factors					
δ multiplier	**SSF (peak)**	**SSF (residual)**	**OSF (upstream)**	**OSF (downstream)**	**USF**
1	1.018	0.574	2.359	1.639	2.107
1.1	1.025	0.580	2.362	1.644	2.111
1.2	1.029	0.585	2.363	1.646	2.113
1.3	1.034	0.591	2.366	1.650	2.117
Failure Probabilities					
δ multiplier	**SSF (peak)**	**SSF (residual)**	**OSF (upstream)**	**OSF (downstream)**	**USF**
1	0.40158	0.99212	0.00000	0.00000	0.00000
1.1	0.40922	0.98372	0.00000	0.00000	0.00000
1.2	0.42008	0.97526	0.00000	0.00000	0.00000
1.3	0.42926	0.96518	0.00000	0.00000	0.00000

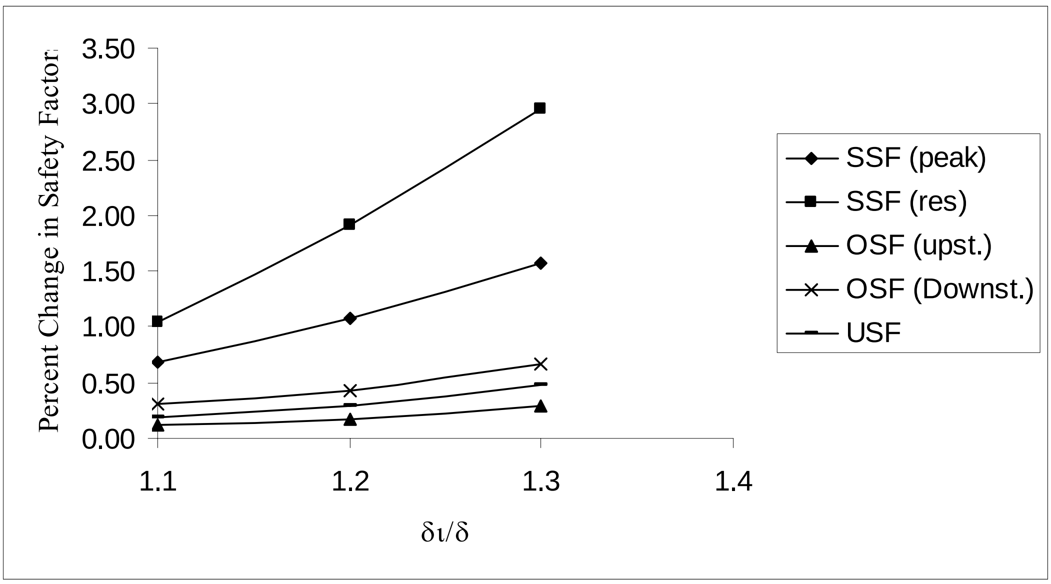


Figure 5.3 Percent Changes of Safety Factors in Sensitivity Analysis

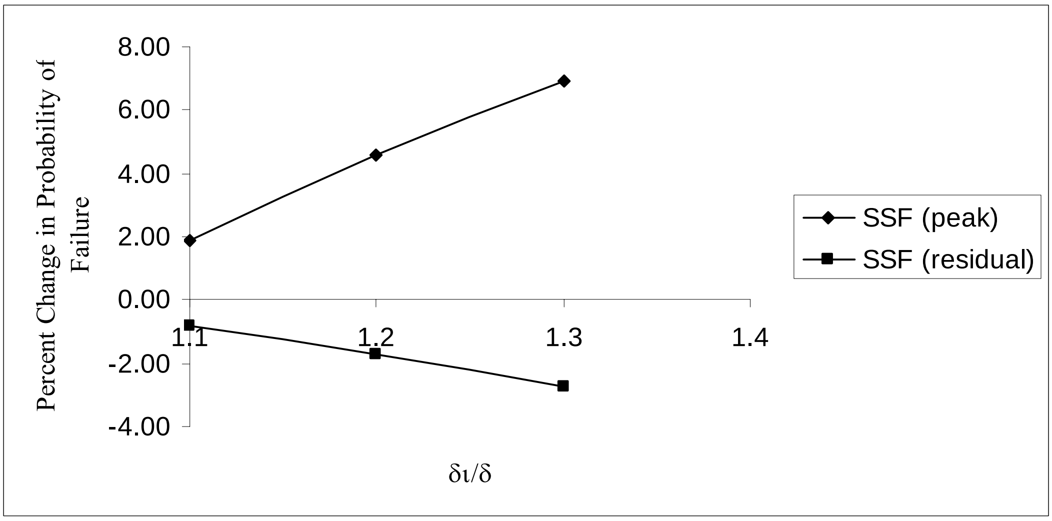


Figure 5.4 Percent Changes of Failure Probabilities in Sensitivity Analysis

5.6 Cohesion Effect on Residual Sliding Safety

Originally cohesion is ignored in the previous analysis. The foundation of the dam is mainly composed of schist in an inclined formation. The impact of this feature may be modeled for the residual state as if foundation material has some cohesion.

When a 50 kPa residual shear strength cohesion is assumed, the residual shear sliding safety factor increases from 1.741 to 1.953 in usual combination. There is also increase in residual sliding safety factors for other combinations, but again, the failure probabilities in seismic combinations are high due to extreme shear forces generated by very high seismic accelerations (Table 5.17 through 5.21).

Table 5.17 Results of Probabilistic Analysis (Usual Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	P_f
L_c	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	3.080	0.285	2.236	4.269	1.00000	0.00000
RSF	1.953	0.168	1.513	2.605	1.00000	0.00000
OF_h	2.916	0.047	2.782	3.121	1.00000	0.00000
OF_t	2.569	0.073	2.293	2.837	1.00000	0.00000
UF	2.950	0.063	2.737	3.154	1.00000	0.00000

Table 5.18 Results of Probabilistic Analysis (Flood Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	2.387	0.079	2.097	2.666	1.00000	0.00000
RSF	1.518	0.018	1.464	1.572	1.00000	0.00000
OFh	2.908	0.053	2.755	3.076	1.00000	0.00000
OFt	2.373	0.054	2.219	2.548	1.00000	0.00000
UF	2.867	0.071	2.668	3.095	1.00000	0.00000

Table 5.19 Results of Probabilistic Analysis (Seismic-1 Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	23.201	38.856	0.000	100.000	0.92092	0.07908
PSF	1.021	0.307	0.349	2.461	0.60268	0.39732
RSF	0.646	0.164	0.295	1.506	0.02736	0.97264
OFh	2.360	0.107	2.062	2.829	1.00000	0.00000
OFt	1.640	0.154	1.248	2.338	1.00000	0.00000
UF	2.109	0.155	1.696	2.778	1.00000	0.00000

Table 5.20 Results of Probabilistic Analysis (Seismic-2 Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	27.419	37.395	0.000	100.000	0.91898	0.08102
PSF	1.058	0.320	0.380	2.536	0.61685	0.38315
RSF	0.678	0.168	0.309	1.595	0.03670	0.96330
OFh	2.417	0.098	2.125	2.828	1.00000	0.00000
OFt	1.639	0.154	1.224	2.364	1.00000	0.00000
UF	2.156	0.150	1.726	2.807	1.00000	0.00000

Table 5.21 Results of Probabilistic Analysis (Post-Seismic Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	3.081	0.284	2.255	4.442	1.00000	0.00000
RSF	1.955	0.167	1.496	2.609	1.00000	0.00000
OFh	2.915	0.047	2.782	3.127	1.00000	0.00000
OFt	2.569	0.073	2.272	2.830	1.00000	0.00000
UF	2.949	0.062	2.729	3.154	1.00000	0.00000

5.7 Additional Stability Analysis

Additional analyses are carried out considering the exact geometry of the dam cross-section. That is the concrete wedge part is taken into consideration and a deterministic analysis is carried out. The results of this analysis are given in Appendix B. Then an equivalent hypothetical cross-section having symmetrical side slopes with almost the same safety factors is searched. At the end of this analysis, a symmetrical hardfill cross-section with side slope of 1V: 0.75 H is found to represent the actual geometry. Therefore, the CADAM has been executed for the aforementioned loading cases using the new geometry. The results of the final analysis are presented in Tables 5.22 to 5.26.

Table 5.22 Results of Probabilistic Analysis (Usual Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	3.277	0.303	2.363	4.562	1.00000	0.00000
RSF	1.850	0.158	1.430	2.451	1.00000	0.00000
OFh	2.892	0.046	2.760	3.101	1.00000	0.00000
OFt	2.621	0.073	2.341	2.899	1.00000	0.00000
UF	2.945	0.063	2.730	3.148	1.00000	0.00000

Table 5.23 Results of Probabilistic Analysis (Flood Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	2.540	0.084	2.220	2.839	1.00000	0.00000
RSF	1.440	0.019	1.382	1.498	1.00000	0.00000
OFh	2.877	0.053	2.724	3.044	1.00000	0.00000
OFt	2.435	0.058	2.270	2.622	1.00000	0.00000
UF	2.865	0.071	2.664	3.094	1.00000	0.00000

Table 5.24 Results of Probabilistic Analysis (Seismic-1 Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	14.896	31.556	0.000	100.000	0.95250	0.04750
PSF	1.089	0.302	0.370	2.500	0.68032	0.31968
RSF	0.598	0.138	0.292	1.372	0.01300	0.98700
OFh	2.328	0.109	2.016	2.816	1.00000	0.00000
OFt	1.683	0.158	1.254	2.386	1.00000	0.00000
UF	2.111	0.156	1.668	2.762	1.00000	0.00000

Table 5.25 Results of Probabilistic Analysis (Seismic-2 Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	17.707	32.540	0.000	100.000	0.95582	0.04418
PSF	1.136	0.314	0.414	2.605	0.73557	0.26443
RSF	0.633	0.139	0.332	1.399	0.01774	0.98226
OFh	2.380	0.101	2.089	2.832	1.00000	0.00000
OFt	1.686	0.157	1.287	2.363	1.00000	0.00000
UF	2.157	0.150	1.737	2.814	1.00000	0.00000

Table 5.26 Results of Probabilistic Analysis (Post-seismic Combination)

Parameter	Mean	Standard Deviation	Minimum Value	Maximum Value	β	Pf
Lc	0.000	0.000	0.000	0.000	1.00000	0.00000
PSF	3.276	0.304	2.374	4.574	1.00000	0.00000
RSF	1.850	0.159	1.425	2.460	1.00000	0.00000
OFh	2.892	0.049	2.761	3.084	1.00000	0.00000
OFt	2.620	0.074	2.342	2.896	1.00000	0.00000
UF	2.944	0.063	2.723	3.510	1.00000	0.00000

For usual, flood and post-seismic combinations, no crack is developed again and the aforementioned safety factors are found to be greater than those of the first probabilistic analysis. For seismic-1 and seismic-2 combinations, the peak and residual sliding safety factors of the new analysis are greater when the results of two analyses are compared. As can be seen from the results, there is significant decrease in the probability of failure for seismic conditions when the cross-section is changed. The probability of failure against peak sliding condition for seismic-1 and seismic-2 are about 32% and 26%, respectively.

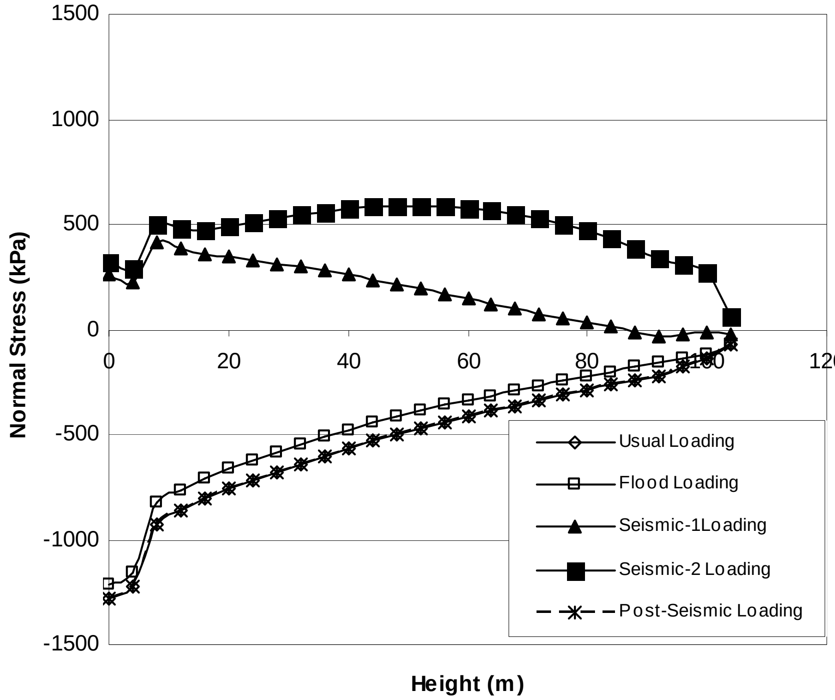


Figure 5.5 Upstream Normal Stress Values

In a different analysis the changes in normal stresses on the upstream and downstream faces are investigated along vertical direction. Upstream and downstream normal stress values from the effective stress analysis for each joint are presented in Figures 5.5 and 5.6, respectively. Negative sign shows that the stress is compressive.

The usual, flood, and post-seismic combinations only compressive stresses developed which are less than the compressive strength for upstream and downstream. In seismic-1 although tensile stresses have developed they are within allowable limits which can be obtained using the information presented in Table 4.3 (See Table 5.27). However for seismic-2 combination in an interval of 30 to 70 meters the tensile stresses exceed the allowable value of 545 kPa. For the downstream face, as can be seen from Figure 5.6, only compressive stresses developed for all loading combinations.

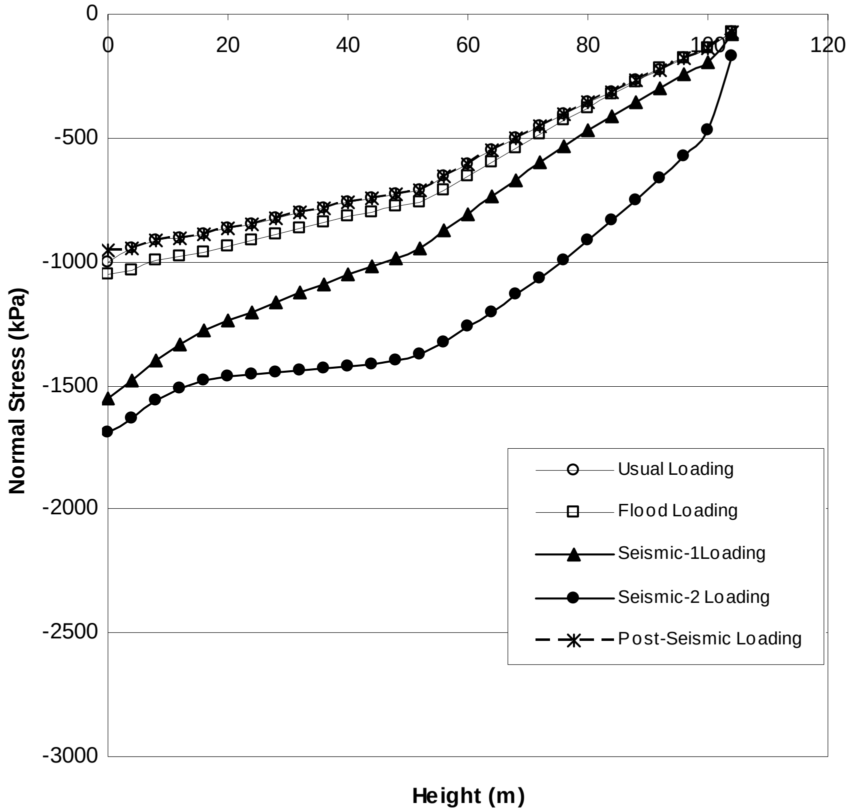


Figure 5.6 Downstream Normal Stress Values

Table 5.27 Allowable Stress Values

	Usual	Flood	Seismic-1	Seismic-2	Post-Seismic
Allowable tensile stresses (kPa)	0.00	300.00	545.40	545.40	400.20
Allowable compressive stresses (kPa)	1998.00	3000.00	5454.00	5454.00	4002.00

5.8 Deterministic Safety Factors and Failure Probability Analyses

The CADAM software is run again in order to perform deterministic analysis and illustrate the difference between the results from the probabilistic and deterministic analysis. The results are given in Table 5.28 and 5.29.

As can be seen from the results presented in Tables 5.28 and 5.29, the safety factors from deterministic and probabilistic analyses are slightly different from each other. Although almost the same safety factors are found in both approaches, the probabilistic analysis is superior to deterministic approach as it accounts for the failure probabilities. Depending on the magnitude of the failure probability, rehabilitative actions may be taken accordingly to experience and judgment.

Table 5.28 Results from Deterministic Analysis

Safety Factors					
Parameter	**Usual**	**Flood**	**Seismic-1**	**Seismic-2**	**Post-Seismic**
PSF	3.062	2.388	1.056	1.110	3.062
RSF	1.729	1.354	0.556	0.588	1.729
OF_h	2.911	2.904	2.351	2.409	2.911
OF_t	2.564	2.369	1.625	1.623	2.564
UF	2.944	2.862	2.095	2.144	2.943

Table 5.29 Results from Probabilistic Analysis

Safety Factors					
Parameter	**Usual**	**Flood**	**Seismic-1**	**Seismic-2**	**Post-Seismic**
PSF	3.080	2.388	1.018	1.058	3.081
RSF	1.741	1.354	0.574	0.607	1.741
OFh	2.915	2.908	2.359	2.417	2.915
OFt	2.568	2.373	1.639	1.639	2.568
UF	2.949	2.868	2.107	2.156	2.949
Probability of Failure					
Parameter	**Usual**	**Flood**	**Seismic-1**	**Seismic-2**	**Post-Seismic**
PSF	0.00000	0.00000	0.40158	0.38406	0.00000
RSF	0.00000	0.00000	0.99212	0.98932	0.00000
OFh	0.00000	0.00000	0.00000	0.00000	0.00000
OFt	0.00000	0.00000	0.00000	0.00000	0.00000
UF	0.00000	0.00000	0.00000	0.00000	0.00000

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Traditionally safety analyses have been conducted by deterministic approaches. Nowadays there is a tendency towards utilization of probabilistic (risk-based) methods for evaluating safety of dams. Risk-based analysis provides more complete information compared to deterministic approach since it allows inclusion of associated uncertainties in the design variables into the analysis.

In this thesis, a risk-based analysis of an existing hardfill dam is carried out. The safety of Cindere Dam is evaluated by using CADAM software which performs safety analysis using Monte-Carlo simulation technique. Relevant data to be used in modeling have been obtained and stability analyses were performed for various loading combinations. Cindere Dam is determined to be safe under usual, flood and post-seismic conditions. Although, some failure probabilities are observed under severe earthquake conditions, the dam may still be considered safe since the concrete wedge element and passive resistance of the alluvial foundation at the downstream side are ignored in modeling due to inability of the software to properly model them. Additional analyses are carried out considering the exact geometry of the dam cross-section. That is the concrete wedge part is taken into consideration and a deterministic analyses is carried out. Then an equivalent hypothetical cross-section having symmetrical side slopes with almost the same safety factors is searched. At the end of this analysis, a symmetrical hardfill cross-section with side slope of 1V: 0.75 H is found to represent the actual geometry. Therefore, the CADAM has been executed for the aforementioned loading cases using the new geometry. The analyses are also repeated with the inclusion of the cohesion effect to observe the variations in residual sliding safety. Moreover, a sensitivity analysis is also carried out to observe the effects of coefficient of variation of variables and probability density functions, which alter from those used in the present study as a result of inclusion of additional relevant data in the future.

With the application of CADAM to an existing structure, it may be possible to obtain some clues about weak behavior of the dam under various combinations of loading that may be likely to occur during the physical life of the structure. Based on such information, some guidelines regarding monitoring and surveillance practices may be developed. Development of algorithmic guidelines including effective operation of the dam instrumentation system may be a topic for a future study. Another potential future topic may be the conduction of a seismic-hazard study for the dam area.

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APPENDIX A

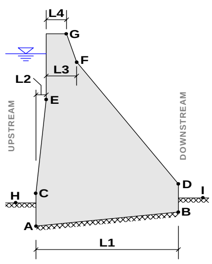
CADAM OUTPUT TABLES

Table A.1 CADAM Input and Geometry Report

CADAM 2000 - GEOMETRY REPORT	
Project:	Project engineer:
Dam:	Analysis performed by:
Owner:	Date:
Dam location:	Units: Metric

Geometry	
L1=	142.300 m
L2=	60.900 m
L3=	10.000 m
L4=	10.000 m
Elev. A=	0.000 m
Elev. B=	0.000 m
Elev. C=	15.000 m
Elev. D=	0.000 m
Elev. E=	102.000 m
Elev. F=	102.000 m
Elev. G=	107.000 m
Elev. H=	0.000 m
Elev. I =	0.000 m

Concrete Volumetric Mass	
ρ=	2548 kg/m³



Lift Joint Material Properties

Material	Concrete strength		Peak friction		Residual friction		Minimal compressive
Material name	f'c (kPa)	ft (kPa)	Cohesion (kPa)	Angle (deg)	Cohesion (kPa)	Angle (deg)	stress for cohesion (kPa)
Base joint	6000	600	200	25	0	20	0
conc	6000	600	800	45	0	30	0

Lift Joint(s)

Joint	material	Usptream end		Downstream end			
Joint id	material name	Elevation (m)	Position x (m)	Elevation (m)	Position x (m)	Length (m)	Inertia (m^4)
1	conc	104.000	60.900	104.000	70.900	10.000	83.33333333
2	conc	100.000	59.500	100.000	72.300	12.800	174.7626667
3	conc	96.000	56.700	96.000	75.100	18.400	519.1253333
4	conc	92.000	53.900	92.000	77.900	24.000	1152
5	conc	88.000	51.100	88.000	80.700	29.600	2161.194667
6	conc	84.000	48.300	84.000	83.500	35.200	3634.517333
7	conc	80.000	45.500	80.000	86.300	40.800	5659.776
8	conc	76.000	42.700	76.000	89.100	46.400	8324.778667
9	conc	72.000	39.900	72.000	91.900	52.000	11717.33333
10	conc	68.000	37.100	68.000	94.700	57.600	15925.248
11	conc	64.000	34.300	64.000	97.500	63.200	21036.33067
12	conc	60.000	31.500	60.000	100.300	68.800	27138.38933
13	conc	56.000	28.700	56.000	103.100	74.400	34319.232
14	conc	52.000	25.900	52.000	105.900	80.000	42666.66667
15	conc	48.000	23.100	48.000	108.700	85.600	52268.50133
16	conc	44.000	20.300	44.000	111.500	91.200	63212.544
17	conc	40.000	17.500	40.000	114.300	96.800	75586.60267
18	conc	36.000	14.700	36.000	117.100	102.400	89478.48533
19	conc	32.000	11.900	32.000	119.900	108.000	104976
20	conc	28.000	9.100	28.000	122.700	113.600	122166.9547
21	conc	24.000	6.300	24.000	125.500	119.200	141139.1573
22	conc	20.000	3.500	20.000	128.300	124.800	161980.416
23	conc	16.000	0.700	16.000	131.100	130.400	184778.5387
24	conc	12.000	0.000	12.000	133.900	133.900	200060.1016
25	conc	8.000	0.000	8.000	136.700	136.700	212874.8219
26	conc	4.000	0.000	4.000	139.500	139.500	226225.4063
Base	Base joint	0.000	0.000	0.000	142.300	142.300	240122.8306

Pre-Cracked Lift Joint(s)

Joint	material	Usptream end Crack length		Downstream end Crack length	
Joint id	material name	(m)	(%)	(m)	(%)
1	conc	-	-	-	-
2	conc	-	-	-	-
3	conc	-	-	-	-
4	conc	-	-	-	-
5	conc	-	-	-	-
6	conc	-	-	-	-
7	conc	-	-	-	-
8	conc	-	-	-	-
9	conc	-	-	-	-
10	conc	-	-	-	-
11	conc	-	-	-	-
12	conc	-	-	-	-
13	conc	-	-	-	-
14	conc	-	-	-	-
15	conc	-	-	-	-
16	conc	-	-	-	-
17	conc	-	-	-	-
18	conc	-	-	-	-
19	conc	-	-	-	-
20	conc	-	-	-	-
21	conc	-	-	-	-
22	conc	-	-	-	-
23	conc	-	-	-	-
24	conc	-	-	-	-
25	conc	-	-	-	-
26	conc	-	-	-	-
Base	Base joint	-	-	-	-

Table A.1 CADAM Input and Geometry Report (continued-1)

Water Volumetric Mass	
ρ=	9.810 kg/m³

Ice cover	
Load=	0 kN
Thickness=	0.000 m
Elevation=	91.500 m

Silts	
Elevation=	55.000 m
γ'=	11 kN/m³
ϕ=	31 deg
Assumption=	active

Reservoirs		
	Upstream side	Downstream side
Normal operating level:	91.500 m	51.000 m
Flood level:	102.700 m	52.500 m
Crest overtopping pressure	100.00 %	50.00 %

Drainage system	
Gallery position from heel of dam=	12.000 m
Gallery elevation=	5.500 m
Drain Efficiency=	0.6667
Highest drained elevation=	5.500 m
Modelisation:	USACE 1995

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

Pseudo-static (seismic coefficient)			
Horizontal Peak Ground Acceleration (HPGA)=	0.4000 g	Earthquake return period=	2000 years
Vertical Peak Ground Acceleration (VPGA)=	0.2700 g	Earthquake accelerogram period (te)=	1 sec
Horizontal Sustained Acceleration (HSA)=	0.2000 g	Depth where pressures remain constant=	Generalized
Vertical Sustained Acceleration (VSA)=	0.1300 g	Westergaard correction for Inclined surface=	Corns et al.

Pseudo-dynamic (Chopra's method)

Earthquake return period=	2000 years
Horizontal Peak Ground Acceleration (HPGA)=	0.4000 g
Vertical Peak Ground Acceleration (VPGA)=	0.2700 g
Horizontal Peak Spectral Acceleration (HPSA)=	0.5740 g
Horizontal Sustained Acceleration (HSA)=	0.2000 g
Vertical Sustained Acceleration (VSA)=	0.1300 g
Vertical Sustained Spectral Acceleration (VSSA)=	0.2870 g
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)=	10000 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
Foundation Young's modulus (dynamic)=	5000 MPa
Dam-reservoir-foundation system	
Period of vibration=	0.6008997 sec
Damping=	0.14650752 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 when crack is beyond drain line

Table A.2 CADAM Loads

CADAM 2000 - Loads report	
Project:	Project engineer:
Dam:	Analysis performed by:
Owner:	Date:
Dam location:	Units: Metric

STATIC LOADS (1/3)

Joint		Self-Weight				Normal Operating level			
		Dam		Concentrated masses		Upstream reservoir			
ID	Upstream	Vertical load		Vertical load		Horizontal load		Vertical load	
	elevation	D	position x	Mv	position x	Hnu	elevation	Vnu	position x
	(m)	(kN)	(m)	(kN)	(m)	(kN)	(m)	(kN)	(m)
1	104.000	-749.9	65.900						
2	100.000	-1819.7	65.900						
3	96.000	-3379.4	65.900						
4	92.000	-5499.1	65.900						
5	88.000	-8178.7	65.900			60.1	89.167	-42.1	51.917
6	84.000	-11418.1	65.900			275.9	86.500	-193.1	50.050
7	80.000	-15217.5	65.900			648.7	83.833	-454.1	48.183
8	76.000	-19576.8	65.900			1178.4	81.167	-824.9	46.317
9	72.000	-24496.0	65.900			1865.1	78.500	-1305.6	44.450
10	68.000	-29975.1	65.900			2708.8	75.833	-1896.2	42.583
11	64.000	-36014.1	65.900			3709.4	73.167	-2596.6	40.717
12	60.000	-42613.0	65.900			4867.0	70.500	-3406.9	38.850
13	56.000	-49771.8	65.900			6181.5	67.833	-4327.1	36.983
14	52.000	-57490.5	65.900			7653.0	65.167	-5357.1	35.117
15	48.000	-65769.2	65.900			9281.5	62.500	-6497.0	33.250
16	44.000	-74607.7	65.900			11066.9	59.833	-7746.8	31.383
17	40.000	-84006.2	65.900			13009.3	57.167	-9106.5	29.517
18	36.000	-93964.5	65.900			15108.6	54.500	-10576.0	27.650
19	32.000	-104482.8	65.900			17364.9	51.833	-12155.4	25.783
20	28.000	-115561.0	65.900			19778.2	49.167	-13844.7	23.917
21	24.000	-127199.0	65.900			22348.4	46.500	-15643.9	22.050
22	20.000	-139397.0	65.900			25075.6	43.833	-17552.9	20.183
23	16.000	-152154.9	65.900			27959.7	41.167	-19571.8	18.317
24	12.000	-165394.0	65.932			31000.8	38.500	-20093.7	17.850
25	8.000	-178921.8	66.062			34198.9	35.833	-20093.7	17.850
26	4.000	-192729.5	66.276			37553.9	33.167	-20093.7	17.850
27	Base	-206817.2	66.561			43434.6	31.367	-21483.8	18.457

Joint		Normal Operating level									
		Downstream reservoir				Crest Overtopping		Uplift		Ice	
ID	Upstream	Horizontal load		Vertical load		Vertical load		Normal load		Horizontal load	
	elevation	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)
1	104.000										
2	100.000										
3	96.000										
4	92.000										
5	88.000							508.2	9.867		
6	84.000							1294.9	11.733		
7	80.000							2301.4	13.600		
8	76.000							3527.7	15.467		
9	72.000							4973.7	17.333		
10	68.000							6639.4	19.200		
11	64.000							8524.9	21.067		
12	60.000							10630.1	22.933		
13	56.000							12955.1	24.800		
14	52.000							15499.8	26.667		
15	48.000	-44.1	49.000	-30.9	108.000			19523.9	30.374		
16	44.000	-240.3	46.333	-168.2	109.867			24379.8	34.305		
17	40.000	-593.5	43.667	-415.5	111.733			29675.3	37.946		
18	36.000	-1103.6	41.000	-772.5	113.600			35410.2	41.396		
19	32.000	-1770.7	38.333	-1239.5	115.467			41584.6	44.713		
20	28.000	-2594.7	35.667	-1816.3	117.333			48198.5	47.935		
21	24.000	-3575.7	33.000	-2503.0	119.200			55251.9	51.086		
22	20.000	-4713.7	30.333	-3299.6	121.067			62744.8	54.181		
23	16.000	-6008.6	27.667	-4206.0	122.933			70677.1	57.234		
24	12.000	-7460.5	25.000	-5222.4	124.800			77828.4	59.323		
25	8.000	-9069.3	22.333	-6348.5	126.667			84820.0	61.056		
26	4.000	-10835.1	19.667	-7584.6	128.533			75145.8	65.501		
27	Base	-12757.9	17.000	-8930.5	130.400			82206.1	67.133		

Table A.2 CADAM Loads (continued-1)

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	104.000																
2	100.000					35.8	100.900	-23.3	60.063							169.5	4.267
3	96.000					220.2	98.233	-152.4	58.232							604.7	6.133
4	92.000					561.6	95.567	-391.4	56.377							1259.6	8.000
5	88.000					1059.9	92.900	-740.3	54.515							2134.3	9.867
6	84.000					1715.2	90.233	-1199.0	52.652							3228.7	11.733
7	80.000					2527.5	87.567	-1767.6	50.787							4542.8	13.600
8	76.000					3496.7	84.900	-2446.0	48.922							6076.7	15.467
9	72.000					4622.9	82.233	-3234.4	47.056							7830.3	17.333
10	68.000					5906.1	79.567	-4132.6	45.190							9803.7	19.200
11	64.000					7346.2	76.900	-5140.6	43.324							11996.8	21.067
12	60.000					8943.2	74.233	-6258.6	41.458							14409.7	22.933
13	56.000					10697.3	71.567	-7486.4	39.592							17042.3	24.800
14	52.000	15.8	53.000	-34.7	26.600	12608.3	68.900	-8824.1	37.726	-1.2	52.167	-0.9	105.783			20090.9	26.927
15	48.000	86.3	50.333	-188.7	24.733	14676.2	66.233	-10271.7	35.859	-99.3	49.500	-69.5	107.650			24856.2	30.702
16	44.000	213.0	47.667	-465.9	22.867	16901.1	63.567	-11829.1	33.993	-354.4	46.833	-248.1	109.517			30061.0	34.245
17	40.000	396.1	45.000	-866.3	21.000	19283.0	60.900	-13496.4	32.126	-766.4	44.167	-536.5	111.383			35705.3	37.630
18	36.000	635.6	42.333	-1389.9	19.133	21821.8	58.233	-15273.6	30.260	-1335.4	41.500	-934.8	113.250			41789.0	40.903
19	32.000	931.3	39.667	-2036.7	17.267	24517.6	55.567	-17160.6	28.393	-2061.3	38.833	-1442.9	115.117			48312.3	44.092
20	28.000	1283.4	37.000	-2806.7	15.400	27370.3	52.900	-19157.6	26.527	-2944.2	36.167	-2061.0	116.983			55275.0	47.219
21	24.000	1691.9	34.333	-3699.9	13.533	30380.0	50.233	-21264.4	24.660	-3984.1	33.500	-2788.9	118.850			62677.3	50.297
22	20.000	2156.7	31.667	-4716.3	11.667	33546.7	47.567	-23481.0	22.794	-5180.9	30.833	-3626.6	120.717			70519.0	53.336
23	16.000	2677.8	29.000	-5855.9	9.800	36870.3	44.900	-25807.6	20.927	-6534.7	28.167	-4574.3	122.583			78800.2	56.344
24	12.000	3255.2	26.333	-6160.0	9.333	40350.9	42.233	-26406.4	20.461	-8045.4	25.500	-5631.8	124.450			86169.5	58.411
25	8.000	3889.0	23.667	-6160.0	9.333	43988.5	39.567	-26406.4	20.461	-9713.1	22.833	-6799.2	126.317			93335.5	60.134
26	4.000	4579.2	21.000	-6160.0	9.333	47783.0	36.900	-26406.4	20.461	-11537.8	20.167	-8076.5	128.183			79791.6	64.791
27	Base	5325.6	18.333	-6160.0	9.333	43434.6	31.367	-21483.8	18.457	-13519.4	17.500	-9463.6	130.050			86937.5	66.442

Table A.2 CADAM Loads (continued-2)

STATIC LOADS (3/3)											
Joint		Post-tensioning						Applied forces			
		Crest		Downstream face							
ID	Upstream elevation (m)	Vertical load Pc (kN)	position x (m)	Vertical load Pdv (kN)	position x (m)	Horizontal load Pdh (kN)	elevation (m)	Horizontal load Fh (kN)	elevation (m)	Vertical load Fv (kN)	position x (m)
1	104.000										
2	100.000										
3	96.000										
4	92.000										
5	88.000										
6	84.000										
7	80.000										
8	76.000										
9	72.000										
10	68.000										
11	64.000										
12	60.000										
13	56.000										
14	52.000										
15	48.000										
16	44.000										
17	40.000										
18	36.000										
19	32.000										
20	28.000										
21	24.000										
22	20.000										
23	16.000										
24	12.000										
25	8.000										
26	4.000										
27	Base										

Table A.2 CADAM Loads (continued-3)

PSEUDO-STATIC LOADS (SEISMIC COEFFICIENT)-STRESS 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	104.000	-300.0	105.500	-202.5	65.900																
2	100.000	-727.9	103.391	-491.3	65.900																
3	96.000	-1351.8	100.848	-912.4	65.900																
4	92.000	-2199.6	98.174	-1484.8	65.900																
5	88.000	-3271.5	95.473	-2208.2	65.900					-47.2	89.400	-11.4	51.917								
6	84.000	-4567.2	92.769	-3082.9	65.900					-148.2	87.000	-52.1	50.050								
7	80.000	-6087.0	90.068	-4108.7	65.900					-281.4	84.600	-122.6	48.183								
8	76.000	-7830.7	87.371	-5285.7	65.900					-440.3	82.200	-222.7	46.317								
9	72.000	-9798.4	84.679	-6613.9	65.900					-621.3	79.800	-352.5	44.450								
10	68.000	-11990.0	81.989	-8093.3	65.900					-822.0	77.400	-512.0	42.583								
11	64.000	-14405.6	79.303	-9723.8	65.900					-1040.6	75.000	-701.1	40.717								
12	60.000	-17045.2	76.619	-11505.5	65.900					-1275.7	72.600	-919.9	38.850								
13	56.000	-19908.7	73.937	-13438.4	65.900					-1526.2	70.200	-1168.3	36.983								
14	52.000	-22996.2	71.257	-15522.4	65.900					-1791.3	67.800	-1446.4	35.117					-32.6	53.200	-32.2	26.699
15	48.000	-26307.7	68.579	-17757.7	65.900					-2070.2	65.400	-1754.2	33.250	-28.0	49.200	-27.9	107.902	-116.2	50.800	-132.3	24.934
16	44.000	-29843.1	65.901	-20144.1	65.900					-2362.2	63.000	-2091.6	31.383	-99.8	46.800	-115.3	109.669	-228.9	48.400	-286.0	23.154
17	40.000	-33602.5	63.225	-22681.7	65.900					-2666.8	60.600	-2458.8	29.517	-196.5	44.400	-249.7	111.451	-364.4	46.000	-489.0	21.365
18	36.000	-37585.8	60.550	-25370.4	65.900					-2983.4	58.200	-2855.5	27.650	-313.0	42.000	-427.7	113.241	-519.5	43.600	-738.9	19.570
19	32.000	-41793.1	57.875	-28210.4	65.900					-3311.7	55.800	-3282.0	25.783	-446.1	39.600	-647.0	115.039	-691.9	41.200	-1034.2	17.769
20	28.000	-46224.4	55.201	-31201.5	65.900					-3651.2	53.400	-3738.1	23.917	-594.2	37.200	-906.4	116.841	-880.1	38.800	-1373.8	15.965
21	24.000	-50879.6	52.528	-34343.7	65.900					-4001.6	51.000	-4223.8	22.050	-755.8	34.800	-1204.9	118.647	-1082.7	36.400	-1756.9	14.157
22	20.000	-55758.8	49.855	-37637.2	65.900					-4362.5	48.600	-4739.3	20.183	-929.8	32.400	-1541.8	120.456	-1298.9	34.000	-2182.6	12.347
23	16.000	-60862.0	47.183	-41081.8	65.900					-4733.6	46.200	-5284.4	18.317	-1115.5	30.000	-1916.4	122.268	-1527.8	31.600	-2650.5	10.534
24	12.000	-66157.6	44.526	-44656.4	65.932					-5638.0	40.988	-5425.3	17.850	-1312.0	27.600	-2328.5	124.082	-2100.6	26.719	-2774.1	10.081
25	8.000	-71568.7	41.915	-48308.9	66.062					-6742.0	35.912	-5425.3	17.850	-1519.0	25.200	-2777.4	125.898	-2813.7	22.478	-2774.1	10.081
26	4.000	-77091.8	39.342	-52037.0	66.276					-7872.7	31.615	-5425.3	17.850	-1735.8	22.800	-3262.9	127.717	-3557.8	19.028	-2774.1	10.081
27	Base	-82726.9	36.798	-55840.6	66.561					-9029.6	27.820	-5425.3	17.850	-1962.0	20.400	-3784.7	129.536	-4331.7	15.984	-2774.1	10.081

Table A.2 CADAM Loads (continued-4)

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	104.000	-150.0	105.500	-97.5	65.900																
2	100.000	-363.9	103.391	-236.6	65.900																
3	96.000	-675.9	100.848	-439.3	65.900																
4	92.000	-1099.8	98.174	-714.9	65.900																
5	88.000	-1635.7	95.473	-1063.2	65.900					-24.4	89.400	-5.5	51.917								
6	84.000	-2283.6	92.769	-1484.4	65.900					-76.6	87.000	-25.1	50.050								
7	80.000	-3043.5	90.068	-1978.3	65.900					-145.4	84.600	-59.0	48.183								
8	76.000	-3915.4	87.371	-2545.0	65.900					-227.5	82.200	-107.2	46.317								
9	72.000	-4899.2	84.679	-3184.5	65.900					-321.0	79.800	-169.7	44.450								
10	68.000	-5995.0	81.989	-3896.8	65.900					-424.6	77.400	-246.5	42.583								
11	64.000	-7202.8	79.303	-4681.8	65.900					-537.5	75.000	-337.6	40.717								
12	60.000	-8522.6	76.619	-5539.7	65.900					-659.0	72.600	-442.9	38.850								
13	56.000	-9954.4	73.937	-6470.3	65.900					-788.4	70.200	-562.5	36.983								
14	52.000	-11498.1	71.257	-7473.8	65.900					-925.4	67.800	-696.4	35.117					-16.8	53.200	-27.3	15.922
15	48.000	-13153.8	68.579	-8550.0	65.900					-1069.4	65.400	-844.6	33.250	-14.5	49.200	-14.1	107.900	-60.0	50.800	-105.8	15.677
16	44.000	-14921.5	65.901	-9699.0	65.900					-1220.3	63.000	-1007.1	31.383	-51.5	46.800	-57.9	109.663	-118.2	48.400	-220.8	15.038
17	40.000	-16801.2	63.225	-10920.8	65.900					-1377.6	60.600	-1183.8	29.517	-101.5	44.400	-125.1	111.442	-188.3	46.000	-367.7	14.208
18	36.000	-18792.9	60.550	-12215.4	65.900					-1541.2	58.200	-1374.9	27.650	-161.7	42.000	-213.6	113.229	-268.4	43.600	-544.3	13.260
19	32.000	-20896.6	57.875	-13582.8	65.900					-1710.8	55.800	-1580.2	25.783	-230.5	39.600	-322.5	115.023	-357.4	41.200	-749.1	12.228
20	28.000	-23112.2	55.201	-15022.9	65.900					-1886.2	53.400	-1799.8	23.917	-307.0	37.200	-451.0	116.822	-454.6	38.800	-980.9	11.133
21	24.000	-25439.8	52.528	-16535.9	65.900					-2067.2	51.000	-2033.7	22.050	-390.4	34.800	-598.7	118.625	-559.3	36.400	-1238.9	9.988
22	20.000	-27879.4	49.855	-18121.6	65.900					-2253.6	48.600	-2281.9	20.183	-480.3	32.400	-765.2	120.431	-671.0	34.000	-1522.3	8.802
23	16.000	-30431.0	47.183	-19780.1	65.900					-2445.3	46.200	-2544.3	18.317	-576.2	30.000	-950.1	122.240	-789.2	31.600	-1830.7	7.582
24	12.000	-33078.8	44.526	-21501.2	65.932					-2899.1	40.988	-2612.2	17.850	-677.8	27.600	-1153.4	124.051	-1076.6	26.719	-1911.7	7.272
25	8.000	-35784.4	41.915	-23259.8	66.062					-3451.1	35.912	-2612.2	17.850	-784.7	25.200	-1374.6	125.865	-1433.2	22.478	-1911.7	7.272
26	4.000	-38545.9	39.342	-25054.8	66.276					-4016.5	31.615	-2612.2	17.850	-896.7	22.800	-1613.7	127.680	-1805.2	19.028	-1911.7	7.272
27	Base	-41363.4	36.798	-26886.2	66.561					-4594.9	27.820	-2612.2	17.850	-1013.6	20.400	-1870.5	129.497	-2192.2	15.984	-1911.7	7.272

Table A.2 CADAM Loads (continued-5)

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	
		Horizontal load		Horizontal load		Horizontal load		Horizontal load		Horizontal load		Horizontal load		Horizontal load		Horizontal load		Horizontal load		Horizontal load	
ID	Upstream elevation (m)	Eq1 (kN)	elevation (m)	Hd1 (kN)	elevation (m)	Md1 (kN)	elevation (m)	Em1 (kN)	elevation (m)	Eqs (kN)	elevation (m)	Hds (kN)	elevation (m)	Mds (kN)	elevation (m)	Ems (kN)	elevation (m)	Emc (kN)	elevation (m)	Emc (kN)	elevation (m)
1	104.000	-1184.3	105.519					-1184.3	105.519	523.0	105.530					523.0	105.530	-1294.6	105.521		
2	100.000	-2715.7	103.508					-2715.7	103.508	1158.8	103.583					1158.8	103.583	-2952.6	103.520		
3	96.000	-4686.5	101.159					-4686.5	101.159	1904.4	101.380					1904.4	101.380	-5058.6	101.190		
4	92.000	-7050.5	98.743					-7050.5	98.743	2699.3	99.206					2699.3	99.206	-7549.6	98.802		
5	88.000	-9693.5	96.351	-98.7	89.117			-9792.2	96.278	3464.1	97.180	263.6	90.008			3727.7	96.672	-10477.8	96.328		
6	84.000	-12530.8	94.003	-422.7	86.592			-12953.4	93.761	4140.0	95.364	431.5	88.533			4571.5	94.719	-13736.5	93.868		
7	80.000	-15496.5	91.704	-877.6	84.175			-16374.1	91.300	4681.1	93.830	498.3	87.720			5179.4	93.242	-17173.8	91.479		
8	76.000	-18534.7	89.457	-1403.7	81.849			-19938.4	88.921	5048.7	92.690	476.5	88.222			5525.2	92.305	-20689.8	89.167		
9	72.000	-21594.6	87.267	-1967.6	79.595			-23562.3	86.626	5207.4	92.135	375.1	92.137			5582.4	92.135	-24214.6	86.927		
10	68.000	-24629.4	85.140	-2551.9	77.396			-27181.3	84.413	5124.6	92.509	199.7	111.705			5324.3	93.229	-27697.8	84.755		
11	64.000	-27596.6	83.083	-3147.0	75.240			-30743.6	82.280	4770.9	94.494	-46.2	-132.021			4724.7	96.709	-31104.5	82.641		
12	60.000	-30459.6	81.102	-3747.1	73.120			-34206.7	80.228	4120.9	99.645	-360.4	37.067			3760.5	105.643	-34412.8	80.579		
13	56.000	-33188.4	79.204	-4347.7	71.031			-37536.1	78.258	3153.6	112.452	-741.6	47.797			2412.0	132.332	-37613.5	78.556		
14	52.000	-35759.7	77.394	-4944.0	68.977			-40703.7	76.371	1852.9	153.546	-1188.7	50.112			664.2	338.652	-40709.2	76.561		
15	48.000	-38155.6	75.675	-5531.7	66.962			-43687.3	74.572	206.4	980.159	-1700.4	50.066			-1494.0	-78.429	-43712.8	74.576		
16	44.000	-40362.8	74.054	-6106.5	64.990			-46469.2	72.863	-1795.3	-61.463	-2275.2	49.029			-4070.4	0.297	-46647.2	72.585		
17	40.000	-42371.0	72.536	-6665.3	63.063			-49036.2	71.249	-4159.1	-2.688	-2911.3	47.486			-7070.4	17.971	-49543.3	70.566		
18	36.000	-44172.8	71.129	-7206.4	61.182			-51379.2	69.734	-6890.4	13.422	-3606.4	45.653			-10496.8	24.496	-52440.5	68.498		
19	32.000	-45764.3	69.840	-7729.9	59.342			-53494.2	68.323	-9991.8	19.797	-4357.5	43.640			-14349.3	27.037	-55385.3	66.361		
20	28.000	-47145.9	68.674	-8237.1	57.536			-55383.0	67.017	-13463.0	22.419	-5160.7	41.514			-18623.7	27.710	-58430.4	64.133		
21	24.000	-48323.3	67.635	-8730.1	55.755			-57053.4	65.817	-17300.0	23.206	-6011.4	39.316			-23311.4	27.360	-61632.1	61.800		
22	20.000	-49306.7	66.726	-9211.3	53.992			-58518.0	64.722	-21495.8	22.965	-6904.1	37.075			-28400.0	26.395	-65045.5	59.356		
23	16.000	-50107.6	65.949	-9682.5	52.241			-59790.1	63.729	-26042.4	22.094	-7833.5	34.811			-33875.9	25.034	-68720.0	56.804		
24	12.000	-50730.3	65.312	-10144.3	50.501			-60874.6	62.844	-30905.1	20.817	-8795.5	32.533			-39700.6	23.413	-72676.4	54.170		
25	8.000	-51169.2	64.839	-10596.1	48.774			-61765.3	62.083	-36011.2	19.281	-9788.4	30.246			-45799.7	21.625	-76893.2	51.505		
26	4.000	-51421.8	64.551	-11038.1	47.062			-62459.9	61.460	-41358.8	17.562	-10811.2	27.952			-52170.0	19.715	-81381.5	48.834		
27	Base	-51496.5	64.461	-11473.6	45.351			-62970.1	60.979	-46941.7	15.710	-11863.4	25.649			-58805.1	17.715	-86158.4	46.179		

Table A.2 CADAM Loads (continued-6)

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	104.000	-202.5	65.900												
2	100.000	-491.3	65.900												
3	96.000	-912.4	65.900												
4	92.000	-1484.8	65.900												
5	88.000	-2208.2	65.900	-11.4	51.917										
6	84.000	-3082.9	65.900	-52.1	50.050										
7	80.000	-4108.7	65.900	-122.6	48.183										
8	76.000	-5285.7	65.900	-222.7	46.317										
9	72.000	-6613.9	65.900	-352.5	44.450										
10	68.000	-8093.3	65.900	-512.0	42.583										
11	64.000	-9723.8	65.900	-701.1	40.717										
12	60.000	-11505.5	65.900	-919.9	38.850										
13	56.000	-13438.4	65.900	-1168.3	36.983										
14	52.000	-15522.4	65.900	-1446.4	35.117					-32.2	26.699			-32.6	53.200
15	48.000	-17757.7	65.900	-1754.2	33.250	-27.9	107.902			-132.3	24.934	-28.0	49.200	-116.2	50.800
16	44.000	-20144.1	65.900	-2091.6	31.383	-115.3	109.669			-286.0	23.154	-99.8	46.800	-228.9	48.400
17	40.000	-22681.7	65.900	-2458.8	29.517	-249.7	111.451			-489.0	21.365	-196.5	44.400	-364.4	46.000
18	36.000	-25370.4	65.900	-2855.5	27.650	-427.7	113.241			-738.9	19.570	-313.0	42.000	-519.5	43.600
19	32.000	-28210.4	65.900	-3282.0	25.783	-647.0	115.039			-1034.2	17.769	-446.1	39.600	-691.9	41.200
20	28.000	-31201.5	65.900	-3738.1	23.917	-906.4	116.841			-1373.8	15.965	-594.2	37.200	-880.1	38.800
21	24.000	-34343.7	65.900	-4223.8	22.050	-1204.9	118.647			-1756.9	14.157	-755.8	34.800	-1082.7	36.400
22	20.000	-37637.2	65.900	-4739.3	20.183	-1541.8	120.456			-2182.6	12.347	-929.8	32.400	-1298.9	34.000
23	16.000	-41081.8	65.900	-5284.4	18.317	-1916.4	122.268			-2650.5	10.534	-1115.5	30.000	-1527.8	31.600
24	12.000	-44656.4	65.932	-5425.3	17.850	-2328.5	124.082			-2774.1	10.081	-1312.0	27.600	-2100.6	26.719
25	8.000	-48308.9	66.062	-5425.3	17.850	-2777.4	125.898			-2774.1	10.081	-1519.0	25.200	-2813.7	22.478
26	4.000	-52037.0	66.276	-5425.3	17.850	-3262.9	127.717			-2774.1	10.081	-1735.8	22.800	-3557.8	19.028
27	Base	-55840.6	66.561	-5425.3	17.850	-3784.7	129.536			-2774.1	10.081	-1962.0	20.400	-4331.7	15.984

Table A.2 CADAM Loads (continued-7)

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	104.000	-592.1	105.519					-592.1	105.519	261.5	105.530					261.5	105.530	-647.3	105.521		
2	100.000	-1357.8	103.508					-1357.8	103.508	579.4	103.583					579.4	103.583	-1476.3	103.520		
3	96.000	-2343.2	101.159					-2343.2	101.159	952.2	101.380					952.2	101.380	-2529.3	101.190		
4	92.000	-3525.3	98.743					-3525.3	98.743	1349.7	99.206					1349.7	99.206	-3774.8	98.802		
5	88.000	-4846.7	96.351	-49.4	89.117			-4896.1	96.278	1732.1	97.180	131.8	90.008			1863.9	96.672	-5238.9	96.328		
6	84.000	-6265.4	94.003	-211.3	86.592			-6476.7	93.761	2070.0	95.364	215.7	88.533			2285.7	94.719	-6868.2	93.868		
7	80.000	-7748.2	91.704	-438.8	84.175			-8187.1	91.300	2340.6	93.830	249.1	87.720			2589.7	93.242	-8586.9	91.479		
8	76.000	-9267.3	89.457	-701.8	81.849			-9969.2	88.921	2524.3	92.690	238.3	88.222			2762.6	92.305	-10344.9	89.167		
9	72.000	-10797.3	87.267	-983.8	79.595			-11781.1	86.626	2603.7	92.135	187.5	92.137			2791.2	92.135	-12107.3	86.927		
10	68.000	-12314.7	85.140	-1275.9	77.396			-13590.6	84.413	2562.3	92.509	99.8	111.705			2662.1	93.229	-13848.9	84.755		
11	64.000	-13798.3	83.083	-1573.5	75.240			-15371.8	82.280	2385.5	94.494	-23.1	-132.021			2362.4	96.709	-15552.3	82.641		
12	60.000	-15229.8	81.102	-1873.6	73.120			-17103.4	80.228	2060.4	99.645	-180.2	37.067			1880.2	105.643	-17206.4	80.579		
13	56.000	-16594.2	79.204	-2173.8	71.031			-18768.1	78.258	1576.8	112.452	-370.8	47.797			1206.0	132.332	-18806.8	78.556		
14	52.000	-17879.8	77.394	-2472.0	68.977			-20351.9	76.371	926.5	153.546	-594.3	50.112			332.1	338.652	-20354.6	76.561		
15	48.000	-19077.8	75.675	-2765.8	66.962			-21843.6	74.572	103.2	980.159	-850.2	50.066			-747.0	-78.429	-21856.4	74.576		
16	44.000	-20181.4	74.054	-3053.2	64.990			-23234.6	72.863	-897.6	-61.463	-1137.6	49.029			-2035.2	0.297	-23323.6	72.585		
17	40.000	-21185.5	72.536	-3332.6	63.063			-24518.1	71.249	-2079.6	-2.688	-1455.6	47.486			-3535.2	17.971	-24771.7	70.566		
18	36.000	-22086.4	71.129	-3603.2	61.182			-25689.6	69.734	-3445.2	13.422	-1803.2	45.653			-5248.4	24.496	-26220.3	68.498		
19	32.000	-22882.1	69.840	-3865.0	59.342			-26747.1	68.323	-4995.9	19.797	-2178.8	43.640			-7174.6	27.037	-27692.6	66.361		
20	28.000	-23572.9	68.674	-4118.5	57.536			-27691.5	67.017	-6731.5	22.419	-2580.4	41.514			-9311.9	27.710	-29215.2	64.133		
21	24.000	-24161.6	67.635	-4365.0	55.755			-28526.7	65.817	-8650.0	23.206	-3005.7	39.316			-11655.7	27.360	-30816.0	61.800		
22	20.000	-24653.3	66.726	-4605.7	53.992			-29259.0	64.722	-10747.9	22.965	-3452.1	37.075			-14200.0	26.395	-32522.7	59.356		
23	16.000	-25053.8	65.949	-4841.3	52.241			-29895.1	63.729	-13021.2	22.094	-3916.7	34.811			-16938.0	25.034	-34360.0	56.804		
24	12.000	-25365.2	65.312	-5072.1	50.501			-30437.3	62.844	-15452.5	20.817	-4397.8	32.533			-19850.3	23.413	-36338.2	54.170		
25	8.000	-25584.6	64.839	-5298.1	48.774			-30882.7	62.083	-18005.6	19.281	-4894.2	30.246			-22899.8	21.625	-38446.6	51.505		
26	4.000	-25710.9	64.551	-5519.0	47.062			-31229.9	61.460	-20679.4	17.562	-5405.6	27.952			-26085.0	19.715	-40690.8	48.834		
27	Base	-25748.2	64.461	-5736.8	45.351			-31485.0	60.979	-23470.9	15.710	-5931.7	25.649			-29402.6	17.715	-43079.2	46.179		

Table A.2 CADAM Loads (continued-8)

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	
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	104.000	-97.5	65.900												
2	100.000	-236.6	65.900												
3	96.000	-439.3	65.900												
4	92.000	-714.9	65.900												
5	88.000	-1063.2	65.900	-5.5	51.917										
6	84.000	-1484.4	65.900	-25.1	50.050										
7	80.000	-1978.3	65.900	-59.0	48.183										
8	76.000	-2545.0	65.900	-107.2	46.317										
9	72.000	-3184.5	65.900	-169.7	44.450										
10	68.000	-3896.8	65.900	-246.5	42.583										
11	64.000	-4681.8	65.900	-337.6	40.717										
12	60.000	-5539.7	65.900	-442.9	38.850										
13	56.000	-6470.3	65.900	-562.5	36.983										
14	52.000	-7473.8	65.900	-696.4	35.117					-27.3	15.922			-16.8	53.200
15	48.000	-8550.0	65.900	-844.6	33.250	-14.1	107.900			-105.8	15.677	-14.5	49.200	-60.0	50.800
16	44.000	-9699.0	65.900	-1007.1	31.383	-57.9	109.663			-220.8	15.038	-51.5	46.800	-118.2	48.400
17	40.000	-10920.8	65.900	-1183.8	29.517	-125.1	111.442			-367.7	14.208	-101.5	44.400	-188.3	46.000
18	36.000	-12215.4	65.900	-1374.9	27.650	-213.6	113.229			-544.3	13.260	-161.7	42.000	-268.4	43.600
19	32.000	-13582.8	65.900	-1580.2	25.783	-322.5	115.023			-749.1	12.228	-230.5	39.600	-357.4	41.200
20	28.000	-15022.9	65.900	-1799.8	23.917	-451.0	116.822			-980.9	11.133	-307.0	37.200	-454.6	38.800
21	24.000	-16535.9	65.900	-2033.7	22.050	-598.7	118.625			-1238.9	9.988	-390.4	34.800	-559.3	36.400
22	20.000	-18121.6	65.900	-2281.9	20.183	-765.2	120.431			-1522.3	8.802	-480.3	32.400	-671.0	34.000
23	16.000	-19780.1	65.900	-2544.3	18.317	-950.1	122.240			-1830.7	7.582	-576.2	30.000	-789.2	31.600
24	12.000	-21501.2	65.932	-2612.2	17.850	-1153.4	124.051			-1911.7	7.272	-677.8	27.600	-1076.6	26.719
25	8.000	-23259.8	66.062	-2612.2	17.850	-1374.6	125.865			-1911.7	7.272	-784.7	25.200	-1433.2	22.478
26	4.000	-25054.8	66.276	-2612.2	17.850	-1613.7	127.680			-1911.7	7.272	-896.7	22.800	-1805.2	19.028
27	Base	-26886.2	66.561	-2612.2	17.850	-1870.5	129.497			-1911.7	7.272	-1013.6	20.400	-2192.2	15.984

Table A.3 CADAM Results

C A D A M 2 0 0 0 - R e s u l t r e p o r t	
Project:	Project engineer:
Dam:	Analysis performed by:
Owner:	Date:
Dam location:	Units: Metric

L O A D C O M B I N A T I O N F A C T O R S					
	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)	1.000	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					
Post-tensioning					
Applied forces					
Seismic (horizontal)			-1.000	-1.000	
Seismic (vertical)			-1.000	-1.000	

Table A.3 CADAM Results (continued-1)

USUAL COMBINATION (STRESS ANALYSIS)													
Joint		**Cracking**				**Stresses**							
		Upstream		**Downstream**		**Normal stresses**		**alowable 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	104.000					-74.988	-74.988	0.000	-1998.000				
2	100.000					-142.164	-142.164	0.000	-1998.000				
3	96.000					-183.665	-183.665	0.000	-1998.000				
4	92.000					-229.129	-229.129	0.000	-1998.000				
5	88.000					-246.939	-274.179	0.000	-1998.000	172.858	-88.235	49.119	191.925
6	84.000					-267.774	-318.382	0.000	-1998.000	187.442	-91.085	48.504	222.867
7	80.000					-291.326	-364.073	0.000	-1998.000	203.928	-91.352	48.012	254.851
8	76.000					-315.688	-411.639	0.000	-1998.000	220.982	-90.007	47.558	288.148
9	72.000					-340.129	-460.944	0.000	-1998.000	238.090	-87.605	47.118	322.660
10	68.000					-364.366	-511.738	0.000	-1998.000	255.056	-84.485	46.689	358.216
11	64.000					-388.303	-563.778	0.000	-1998.000	271.812	-80.864	46.271	394.645
12	60.000					-411.916	-616.856	0.000	-1998.000	288.341	-76.887	45.868	431.799
13	56.000					-435.220	-670.796	0.000	-1998.000	304.654	-72.652	45.480	469.557
14	52.000					-459.931	-724.631	0.000	-1998.000	321.952	-67.995	45.156	507.242
15	48.000					-488.726	-748.702	0.000	-1998.000	342.108	-57.425	45.322	524.091
16	44.000					-519.443	-765.838	0.000	-1998.000	363.610	-47.122	45.629	536.087
17	40.000					-551.925	-785.246	0.000	-1998.000	386.348	-38.785	45.945	549.672
18	36.000					-586.066	-806.371	0.000	-1998.000	410.246	-32.093	46.268	564.459
19	32.000					-621.745	-828.806	0.000	-1998.000	435.222	-26.793	46.595	580.164
20	28.000					-658.841	-852.254	0.000	-1998.000	461.188	-22.686	46.920	596.578
21	24.000					-697.234	-876.490	0.000	-1998.000	488.064	-19.608	47.242	613.543
22	20.000					-736.814	-901.344	0.000	-1998.000	515.770	-17.429	47.557	630.941
23	16.000					-777.478	-926.686	0.000	-1998.000	544.235	-16.036	47.865	648.680
24	12.000					-836.500	-941.568	0.000	-1998.000	0.000	-4.444	7.565	659.098
25	8.000					-899.891	-953.862	0.000	-1998.000	0.000	-1.306	4.231	667.704
26	4.000					-1182.903	-988.022	0.000	-1998.000	0.000	-0.472	2.544	691.615
27	Base					-1239.241	-1026.190	0.000	-1998.000	0.000	718.333	100.000	718.333

Table A.3 CADAM Results (continued-2)

USUAL 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	104.000	> 100	> 100	> 100	> 100	> 100	-749.9	0.0	0.0	50.000		
2	100.000	> 100	> 100	> 100	> 100	> 100	-1819.7	0.0	0.0	50.000		
3	96.000	> 100	> 100	> 100	> 100	> 100	-3379.4	0.0	0.0	50.000		
4	92.000	> 100	> 100	> 100	> 100	> 100	-5499.1	0.0	0.0	50.000		
5	88.000	> 100	74.108	24.163	12.107	16.177	-7712.6	60.1	1988.9	50.871	508.2	0.000
6	84.000	> 100	21.588	13.294	6.674	8.967	-10316.3	275.9	5225.4	51.439	1294.9	0.000
7	80.000	70.928	11.900	10.037	5.036	6.810	-13370.1	648.7	10091.5	51.850	2301.4	0.000
8	76.000	45.819	8.267	8.490	4.248	5.783	-16874.0	1178.4	17215.0	52.199	3527.7	0.000
9	72.000	33.471	6.447	7.597	3.787	5.188	-20827.9	1865.1	27223.6	52.514	4973.7	0.000
10	68.000	26.326	5.378	7.020	3.484	4.800	-25231.8	2708.8	40745.2	52.804	6639.4	0.000
11	64.000	21.741	4.683	6.619	3.269	4.529	-30085.8	3709.4	58407.7	53.072	8524.9	0.000
12	60.000	18.580	4.198	6.325	3.110	4.329	-35389.8	4867.0	80839.0	53.320	10630.1	0.000
13	56.000	16.285	3.843	6.102	2.987	4.176	-41143.8	6181.5	108666.7	53.550	12955.1	0.000
14	52.000	14.524	3.567	5.927	2.891	4.057	-47382.5	7668.9	141173.0	53.724	15499.8	0.000
15	48.000	13.025	3.280	5.090	2.738	3.713	-52961.9	9323.6	158744.9	53.502	19523.9	0.000
16	44.000	11.918	3.065	4.397	2.601	3.404	-58608.8	11039.6	170781.4	53.195	24379.8	0.000
17	40.000	11.096	2.916	3.938	2.496	3.181	-64719.1	12811.9	182189.6	52.908	29675.3	0.000
18	36.000	10.465	2.811	3.612	2.415	3.013	-71292.8	14640.6	192505.0	52.637	35410.2	0.000
19	32.000	9.968	2.737	3.369	2.350	2.884	-78329.8	16525.5	201263.0	52.379	41584.6	0.000
20	28.000	9.569	2.683	3.181	2.298	2.781	-85830.2	18466.9	207999.0	52.133	48198.5	0.000
21	24.000	9.243	2.646	3.033	2.255	2.698	-93793.9	20464.5	212248.4	51.898	55251.9	0.000
22	20.000	8.973	2.621	2.912	2.220	2.629	-102221.0	22518.5	213546.8	51.674	62744.8	0.000
23	16.000	8.747	2.605	2.812	2.190	2.572	-111111.5	24628.9	211429.5	51.459	70677.1	0.000
24	12.000	8.440	2.565	2.724	2.171	2.530	-119041.7	26795.6	156981.6	50.985	77828.4	0.000
25	8.000	8.135	2.521	2.647	2.154	2.494	-126704.0	29018.6	84046.0	50.485	84820.0	0.000
26	4.000	8.404	2.793	3.012	2.615	3.015	-151422.0	31297.9	-316035.9	48.504	75145.8	0.000
27	Base	2.878	1.630	2.937	2.548	2.961	-161185.4	36002.3	-359510.0	48.433	82206.1	0.000
	Required:	3.000	1.500	1.200	1.200	1.200						

Table A.3 CADAM Results (continued-3)

FLOOD COMBINATION (STRESS ANALYSIS)															
Joint		Cracking				Stresses									
		Upstream		Downstream		Normal stresses		alowable 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	104.000					-74.988	-74.988	300.000	-3000.000						
2	100.000					-121.314	-140.176	300.000	-3000.000	84.919	-41.652	48.760	98.123		
3	96.000					-138.224	-179.950	300.000	-3000.000	96.757	-38.088	47.551	125.965		
4	92.000					-158.437	-227.472	300.000	-3000.000	110.906	-33.307	46.394	159.231		
5	88.000					-179.255	-279.167	300.000	-3000.000	125.479	-28.147	45.324	195.417		
6	84.000					-200.140	-333.293	300.000	-3000.000	140.098	-22.882	44.370	233.305		
7	80.000					-220.966	-388.948	300.000	-3000.000	154.676	-17.614	43.533	272.264		
8	76.000					-241.710	-445.622	300.000	-3000.000	169.197	-12.381	42.800	311.935		
9	72.000					-262.375	-503.009	300.000	-3000.000	183.663	-7.194	42.157	352.106		
10	68.000					-282.973	-560.912	300.000	-3000.000	198.081	-2.056	41.592	392.639		
11	64.000					-303.514	-619.203	300.000	-3000.000	212.460	3.037	41.091	433.442		
12	60.000					-324.007	-677.791	300.000	-3000.000	226.805	8.087	40.647	474.454		
13	56.000					-344.460	-736.612	300.000	-3000.000	241.122	13.099	40.249	515.629		
14	52.000					-366.553	-789.928	300.000	-3000.000	256.587	19.477	40.001	552.950		
15	48.000					-392.226	-809.708	300.000	-3000.000	274.558	32.340	40.234	566.796		
16	44.000					-420.283	-831.685	300.000	-3000.000	294.198	42.977	40.567	582.179		
17	40.000					-450.533	-855.252	300.000	-3000.000	315.373	51.775	40.977	598.677		
18	36.000					-482.781	-879.986	300.000	-3000.000	337.947	59.020	41.441	615.990		
19	32.000					-516.840	-905.581	300.000	-3000.000	361.788	64.936	41.939	633.907		
20	28.000					-552.535	-931.815	300.000	-3000.000	386.775	69.697	42.454	652.270		
21	24.000					-589.711	-958.524	300.000	-3000.000	412.798	73.445	42.975	670.967		
22	20.000					-628.225	-985.588	300.000	-3000.000	439.757	76.298	43.491	689.912		
23	16.000					-667.949	-1012.916	300.000	-3000.000	467.564	78.355	43.995	709.041		
24	12.000					-720.777	-1033.109	300.000	-3000.000	0.000	723.176	100.000	723.176		
25	8.000					-776.348	-1051.769	300.000	-3000.000	0.000	736.238	100.000	736.238		
26	4.000					-1104.702	-1097.172	300.000	-3000.000	0.000	768.020	100.000	768.020		
27	Base					-1158.463	-1117.146	300.000	-3000.000	0.000	782.002	100.000	782.002		

Table A.3 CADAM Results (continued-4)

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	104.000	> 100	> 100	> 100	> 100	> 100	-749.9	0.0	0.0	50.000		
2	100.000	> 100	27.021	16.165	8.069	10.872	-1673.5	35.8	257.5	51.202	169.5	0.000
3	96.000	80.147	7.675	8.579	4.256	5.841	-2927.2	220.2	1177.2	52.186	604.7	0.000
4	92.000	42.436	4.761	6.844	3.359	4.676	-4630.9	561.6	3313.7	52.982	1259.6	0.000
5	88.000	28.742	3.696	6.115	2.968	4.179	-6784.7	1059.9	7294.9	53.632	2134.3	0.000
6	84.000	21.891	3.160	5.725	2.752	3.908	-9388.4	1715.2	13748.5	54.160	3228.7	0.000
7	80.000	17.837	2.842	5.485	2.616	3.739	-12442.2	2527.5	23302.5	54.590	4542.8	0.000
8	76.000	15.176	2.633	5.325	2.522	3.624	-15946.1	3496.7	36584.6	54.945	6076.7	0.000
9	72.000	13.303	2.485	5.212	2.453	3.541	-19900.0	4622.9	54222.7	55.240	7830.3	0.000
10	68.000	11.917	2.376	5.127	2.401	3.479	-24303.9	5906.1	76844.7	55.489	9803.7	0.000
11	64.000	10.852	2.292	5.061	2.360	3.430	-29157.9	7346.2	105078.4	55.702	11996.8	0.000
12	60.000	10.008	2.225	5.010	2.327	3.392	-34461.8	8943.2	139551.6	55.886	14409.7	0.000
13	56.000	9.323	2.171	4.968	2.300	3.360	-40215.9	10697.3	180892.1	56.046	17042.3	0.000
14	52.000	8.735	2.116	4.838	2.270	3.302	-46259.2	12622.9	225800.0	56.101	20090.9	0.000
15	48.000	8.179	2.026	4.219	2.193	3.070	-51442.8	14663.1	254920.2	55.789	24856.2	0.000
16	44.000	7.760	1.967	3.803	2.134	2.899	-57089.7	16759.7	285150.5	55.477	30061.0	0.000
17	40.000	7.436	1.929	3.506	2.089	2.770	-63200.0	18912.7	316026.4	55.166	35705.3	0.000
18	36.000	7.182	1.907	3.283	2.054	2.670	-69773.7	21122.0	347083.3	54.858	41789.0	0.000
19	32.000	6.979	1.896	3.111	2.027	2.590	-76810.7	23387.6	377856.6	54.555	48312.3	0.000
20	28.000	6.814	1.893	2.973	2.004	2.525	-84311.1	25709.6	407881.8	54.259	55275.0	0.000
21	24.000	6.680	1.897	2.861	1.986	2.472	-92274.8	28087.8	436694.3	53.970	62677.3	0.000
22	20.000	6.570	1.905	2.768	1.972	2.428	-100701.9	30522.5	463829.5	53.691	70519.0	0.000
23	16.000	6.480	1.917	2.689	1.960	2.391	-109592.4	33013.4	488822.9	53.421	78800.2	0.000
24	12.000	6.314	1.906	2.620	1.952	2.363	-117422.7	35560.8	466654.3	52.968	86169.5	0.000
25	8.000	6.140	1.890	2.558	1.944	2.339	-124951.8	38164.4	428896.4	52.511	93335.5	0.000
26	4.000	6.496	2.172	2.998	2.409	2.925	-153580.7	40824.4	-12211.2	49.943	79791.6	0.000
27	Base	2.388	1.353	2.904	2.369	2.862	-161909.6	43540.7	-69720.1	49.697	86937.5	0.000
	Required:	2.000	1.300	1.100	1.100	1.100						

Table A.3 CADAM Results (continued-5)

SEISMIC #1 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)													
Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable 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	104.000					-27.745	-81.737	545.400	-5454.000	0.000	44.993	50.000	0.000
2	100.000					-13.389	-194.170	545.400	-5454.000	9.373	135.919	100.000	135.919
3	96.000					-17.944	-250.207	545.400	-5454.000	12.561	175.145	100.000	175.145
4	92.000					-25.794	-308.734	545.400	-5454.000	18.056	216.114	100.000	216.114
5	88.000					-2.987	-368.159	545.400	-5454.000	2.091	257.711	100.000	257.711
6	84.000					21.393	-429.421	545.400	-5454.000	-14.975	300.595	100.000	300.595
7	80.000					45.773	-493.755	545.400	-5454.000	-32.041	345.628	100.000	345.628
8	76.000					70.951	-560.845	545.400	-5454.000	-49.666	392.592	100.000	392.592
9	72.000					97.030	-630.163	545.400	-5454.000	-67.921	441.114	100.000	441.114
10	68.000					123.926	-701.237	545.400	-5454.000	-86.748	490.866	100.000	490.866
11	64.000					151.513	-773.693	545.400	-5454.000	-106.059	541.585	100.000	541.585
12	60.000					179.673	-847.242	545.400	-5454.000	-125.771	593.070	100.000	593.070
13	56.000					208.302	-921.665	545.400	-5454.000	-145.812	645.165	100.000	645.165
14	52.000					237.243	-996.779	545.400	-5454.000	-166.070	697.746	100.000	697.746
15	48.000					264.559	-1042.358	545.400	-5454.000	-185.191	732.330	94.874	729.651
16	44.000					290.828	-1079.684	545.400	-5454.000	-203.579	768.766	89.640	755.779
17	40.000					316.216	-1118.695	545.400	-5454.000	-221.351	810.037	86.085	783.086
18	36.000					340.691	-1159.055	545.400	-5454.000	-238.484	853.758	83.537	811.338
19	32.000					364.239	-1200.466	545.400	-5454.000	-254.967	898.700	81.636	840.326
20	28.000					386.869	-1242.686	545.400	-5454.000	-270.808	944.194	80.172	869.881
21	24.000					408.606	-1285.529	545.400	-5454.000	-286.024	989.861	79.015	899.870
22	20.000					429.482	-1328.844	545.400	-5454.000	-300.638	1035.481	78.081	930.191
23	16.000					449.537	-1372.517	545.400	-5454.000	-314.676	1080.927	77.314	960.762
24	12.000					480.276	-1434.084	545.400	-5454.000	0.000	1053.303	82.192	1003.859
25	8.000					519.084	-1505.455	545.400	-5454.000	0.000	1121.402	80.289	1053.818
26	4.000					341.176	-1601.717	545.400	-5454.000	0.000	1195.300	80.065	1121.202
27	Base					387.834	-1680.465	545.400	-5454.000	0.000	1264.980	79.068	1176.325

Table A.3 CADAM Results (continued-6)

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	104.000	*28.496*	*1.054*	*4.148*	*2.564*	*3.704*	-547.4	300.0	449.9	58.219		
2	100.000	*15.893*	*1.054*	*4.489*	*2.075*	*3.704*	-1328.4	727.9	2468.3	64.516		
3	96.000	*12.714*	*1.054*	*4.484*	*2.080*	*3.704*	-2467.0	1351.8	6552.9	64.436		
4	92.000	*10.554*	*1.054*	*4.466*	*2.102*	*3.704*	-4014.3	2199.6	13581.1	64.096		
5	88.000	*8.634*	*0.939*	*3.863*	*1.808*	*3.014*	-5493.0	3378.8	26662.5	66.398	508.2	0.000
6	84.000	*6.813*	*0.831*	*3.487*	*1.626*	*2.621*	-7181.3	4991.4	46548.0	68.414	1294.9	0.000
7	80.000	*5.559*	*0.752*	*3.263*	*1.516*	*2.399*	-9138.8	7017.1	74843.3	70.073	2301.4	0.000
8	76.000	*4.690*	*0.694*	*3.118*	*1.443*	*2.258*	-11365.5	9449.5	113352.6	71.494	3527.7	0.000
9	72.000	*4.063*	*0.651*	*3.018*	*1.391*	*2.161*	-13861.5	12284.8	163860.9	72.733	4973.7	0.000
10	68.000	*3.594*	*0.618*	*2.946*	*1.352*	*2.091*	-16626.6	15520.8	228141.0	73.822	6639.4	0.000
11	64.000	*3.234*	*0.593*	*2.891*	*1.322*	*2.038*	-19660.9	19155.6	307957.9	74.784	8524.9	0.000
12	60.000	*2.949*	*0.572*	*2.849*	*1.298*	*1.996*	-22964.4	23187.9	405070.1	75.638	10630.1	0.000
13	56.000	*2.719*	*0.555*	*2.815*	*1.279*	*1.963*	-26537.1	27616.5	521231.2	76.400	12955.1	0.000
14	52.000	*2.526*	*0.540*	*2.788*	*1.262*	*1.935*	-30381.5	32489.0	658145.4	77.078	15499.8	0.000
15	48.000	*2.323*	*0.508*	*2.618*	*1.233*	*1.849*	-33289.8	37845.6	798021.1	78.005	19523.9	0.000
16	44.000	*2.145*	*0.477*	*2.447*	*1.205*	*1.765*	-35971.9	43573.5	949929.0	78.956	24379.8	0.000
17	40.000	*1.999*	*0.452*	*2.315*	*1.182*	*1.699*	-38840.0	49642.1	1120454.9	79.802	29675.3	0.000
18	36.000	*1.877*	*0.432*	*2.211*	*1.165*	*1.647*	-41900.2	56042.3	1310497.7	80.544	35410.2	0.000
19	32.000	*1.775*	*0.415*	*2.127*	*1.151*	*1.604*	-45156.2	62768.4	1520892.6	81.186	41584.6	0.000
20	28.000	*1.689*	*0.402*	*2.058*	*1.139*	*1.569*	-48610.4	69816.7	1752445.8	81.735	48198.5	0.000
21	24.000	*1.615*	*0.391*	*2.001*	*1.130*	*1.540*	-52264.6	77184.2	2005945.4	82.198	55251.9	0.000
22	20.000	*1.550*	*0.382*	*1.952*	*1.122*	*1.516*	-56120.2	84868.5	2282166.7	82.585	62744.8	0.000
23	16.000	*1.494*	*0.374*	*1.910*	*1.116*	*1.495*	-60178.3	92867.8	2581875.2	82.902	70677.1	0.000
24	12.000	*1.413*	*0.361*	*1.884*	*1.108*	*1.480*	-63857.5	102003.8	2860246.7	83.451	77828.4	0.000
25	8.000	*1.332*	*0.349*	*1.865*	*1.100*	*1.468*	-67418.4	111661.9	3152695.3	84.209	84820.0	0.000
26	4.000	*1.480*	*0.418*	*2.022*	*1.204*	*1.634*	-87922.7	121556.1	3150765.1	75.689	75145.8	0.000
27	Base	*0.501*	*0.254*	*1.997*	*1.188*	*1.613*	-91970.7	131683.9	3490132.0	76.668	82206.1	0.000
	Required:	1.300	1.000	1.100	1.100	1.100						

Table A.3 CADAM Results (continued-7)

SEISMIC #1 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)													
Joint		**Cracking**				**Stresses**							
		Upstream		**Downstream**		**Normal stresses**		**alowable 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	104.000					-51.741	-78.737	545.400	-5454.000	0.000	22.496	50.000	0.000
2	100.000					-78.488	-168.878	545.400	-5454.000	54.941	-3.509	40.932	118.215
3	96.000					-101.723	-217.855	545.400	-5454.000	71.206	-4.492	40.982	152.498
4	92.000					-128.607	-270.077	545.400	-5454.000	90.025	-5.391	41.194	189.054
5	88.000					-126.364	-322.545	545.400	-5454.000	88.455	0.677	38.441	225.782
6	84.000					-124.878	-375.512	545.400	-5454.000	87.415	11.972	35.416	262.858
7	80.000					-124.764	-430.767	545.400	-5454.000	87.335	24.823	32.217	301.537
8	76.000					-124.666	-488.342	545.400	-5454.000	87.266	37.709	28.758	341.839
9	72.000					-124.163	-547.902	545.400	-5454.000	86.914	49.730	25.024	383.531
10	68.000					-123.155	-609.086	545.400	-5454.000	86.208	60.286	21.018	426.360
11	64.000					-121.655	-671.584	545.400	-5454.000	85.159	68.928	16.746	470.109
12	60.000					-119.712	-735.147	545.400	-5454.000	83.799	75.297	12.213	514.603
13	56.000					-117.383	-799.578	545.400	-5454.000	82.168	79.081	7.420	559.704
14	52.000					-114.775	-864.849	545.400	-5454.000	80.343	80.081	2.183	605.395
15	48.000					-114.084	-901.040	545.400	-5454.000	79.859	630.728	100.000	630.728
16	44.000					-114.698	-929.689	545.400	-5454.000	80.288	650.782	100.000	650.782
17	40.000					-116.640	-960.254	545.400	-5454.000	81.648	672.178	100.000	672.178
18	36.000					-119.951	-992.247	545.400	-5454.000	83.966	694.573	100.000	694.573
19	32.000					-124.606	-1025.306	545.400	-5454.000	87.224	717.714	100.000	717.714
20	28.000					-130.549	-1059.161	545.400	-5454.000	91.384	741.413	100.000	741.413
21	24.000					-137.708	-1093.614	545.400	-5454.000	96.396	765.529	100.000	765.529
22	20.000					-146.006	-1128.514	545.400	-5454.000	102.204	789.960	100.000	789.960
23	16.000					-155.364	-1163.748	545.400	-5454.000	108.755	814.624	100.000	814.624
24	12.000					-170.113	-1202.004	545.400	-5454.000	0.000	841.403	100.000	841.403
25	8.000					-183.440	-1243.711	545.400	-5454.000	0.000	870.598	100.000	870.598
26	4.000					-414.884	-1308.838	545.400	-5454.000	0.000	916.187	100.000	916.187
27	Base					-423.026	-1355.116	545.400	-5454.000	0.000	948.581	100.000	948.581

Table A.3 CADAM Results (continued-8)

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	104.000	57.692	2.511	8.154	5.263	7.692	-652.4	150.0	225.0	53.448		
2	100.000	32.487	2.511	8.507	4.238	7.692	-1583.1	363.9	1234.1	56.090		
3	96.000	26.129	2.511	8.503	4.248	7.692	-2940.1	675.9	3276.5	56.057		
4	92.000	21.807	2.511	8.484	4.294	7.692	-4784.2	1099.8	6790.5	55.914		
5	88.000	17.628	2.230	6.428	3.196	5.213	-6643.9	1720.2	14323.8	57.284	508.2	0.000
6	84.000	14.023	1.929	5.373	2.649	4.140	-8806.9	2636.1	25878.8	58.348	1294.9	0.000
7	80.000	11.459	1.705	4.810	2.358	3.612	-11332.8	3837.6	42448.7	59.180	2301.4	0.000
8	76.000	9.648	1.543	4.467	2.178	3.301	-14221.8	5321.2	65248.3	59.888	3527.7	0.000
9	72.000	8.338	1.424	4.238	2.055	3.098	-17473.7	7085.3	95482.6	60.508	4973.7	0.000
10	68.000	7.358	1.334	4.077	1.967	2.956	-21088.5	9128.4	134350.2	61.060	6639.4	0.000
11	64.000	6.605	1.264	3.958	1.900	2.851	-25066.4	11449.8	183045.7	61.554	8524.9	0.000
12	60.000	6.011	1.209	3.866	1.848	2.770	-29407.2	14048.6	242760.4	61.999	10630.1	0.000
13	56.000	5.532	1.164	3.795	1.806	2.707	-34110.9	16924.3	314682.8	62.400	12955.1	0.000
14	52.000	5.131	1.125	3.738	1.771	2.654	-39185.0	20109.2	400039.6	62.761	15499.8	0.000
15	48.000	4.738	1.062	3.415	1.710	2.496	-43447.3	23621.3	480525.7	62.921	19523.9	0.000
16	44.000	4.409	1.005	3.112	1.654	2.347	-47624.0	27351.2	564886.6	63.006	24379.8	0.000
17	40.000	4.142	0.962	2.891	1.610	2.233	-52121.7	31280.5	658738.4	63.056	29675.3	0.000
18	36.000	3.922	0.929	2.722	1.575	2.144	-56944.5	35404.7	762224.3	63.072	35410.2	0.000
19	32.000	3.738	0.903	2.590	1.546	2.074	-62095.2	39720.8	875480.6	63.055	41584.6	0.000
20	28.000	3.583	0.882	2.483	1.524	2.017	-67575.5	44226.8	998642.3	63.009	48198.5	0.000
21	24.000	3.449	0.866	2.396	1.505	1.970	-73386.8	48921.3	1131843.3	62.939	55251.9	0.000
22	20.000	3.334	0.853	2.323	1.490	1.931	-79530.0	53802.9	1275216.5	62.848	62744.8	0.000
23	16.000	3.233	0.843	2.262	1.477	1.898	-86006.1	58870.7	1428893.5	62.741	70677.1	0.000
24	12.000	3.084	0.822	2.214	1.467	1.875	-91863.3	64527.8	1541749.4	62.534	77828.4	0.000
25	8.000	2.936	0.799	2.175	1.456	1.856	-97545.8	70471.9	1651097.2	62.382	84820.0	0.000
26	4.000	3.028	0.907	2.401	1.650	2.131	-120229.6	76562.2	1449713.7	58.644	75145.8	0.000
27	Base	1.056	0.556	2.351	1.625	2.095	-126514.8	82797.7	1572847.3	58.737	82206.1	0.000
	Required:	1.300	1.000	1.100	1.100	1.100						

Table A.3 CADAM Results (continued-9)

SEISMIC #2 COMBINATION - PEAK ACCELERATIONS (STRESS ANALYSIS)													
Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable 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	104.000					63.381	-172.863	545.400	-5454.000	0.000	194.193	50.000	0.000
2	100.000					276.797	-484.357	545.400	-5454.000	-193.758	384.535	78.097	339.050
3	96.000					331.239	-599.390	545.400	-5454.000	-231.867	463.115	79.981	419.573
4	92.000					367.680	-702.209	545.400	-5454.000	-257.376	531.648	81.603	491.546
5	88.000					425.235	-797.148	545.400	-5454.000	-297.665	604.011	81.574	558.004
6	84.000					476.204	-887.194	545.400	-5454.000	-333.342	674.474	81.283	621.036
7	80.000					519.918	-973.910	545.400	-5454.000	-363.943	739.717	81.353	681.737
8	76.000					557.413	-1056.908	545.400	-5454.000	-390.189	798.999	81.763	739.835
9	72.000					589.124	-1135.815	545.400	-5454.000	-412.387	852.469	82.439	795.070
10	68.000					615.392	-1210.480	545.400	-5454.000	-430.774	900.661	83.325	847.336
11	64.000					636.556	-1280.923	545.400	-5454.000	-445.589	944.259	84.382	896.646
12	60.000					652.982	-1347.292	545.400	-5454.000	-457.087	984.000	85.583	943.104
13	56.000					665.047	-1409.816	545.400	-5454.000	-465.533	1020.623	86.903	986.871
14	52.000					673.084	-1468.781	545.400	-5454.000	-471.159	1055.201	88.251	1028.147
15	48.000					675.652	-1494.438	545.400	-5454.000	-472.956	1075.887	87.822	1046.107
16	44.000					673.831	-1508.557	545.400	-5454.000	-471.682	1090.817	87.010	1055.990
17	40.000					668.310	-1521.590	545.400	-5454.000	-467.817	1104.319	86.362	1065.113
18	36.000					659.595	-1533.732	545.400	-5454.000	-461.717	1116.766	85.812	1073.612
19	32.000	71.450	77.166			89.993	-3231.866	545.400	-5454.000	-62.995	3181.302	90.083	2262.306
20	28.000	67.880	77.111			89.998	-2959.302	545.400	-5454.000	-62.998	2885.628	88.936	2071.511
21	24.000	64.219	76.549			89.999	-2738.873	545.400	-5454.000	-62.999	2645.352	87.782	1917.211
22	20.000	60.583	75.608			90.000	-2564.345	545.400	-5454.000	-63.000	2454.944	86.654	1795.042
23	16.000					589.602	-1593.631	545.400	-5454.000	-412.721	1181.448	83.103	1115.541
24	12.000	56.470	75.613			89.997	-2467.311	545.400	-5454.000	0.000	2353.800	85.184	1727.118
25	8.000	57.166	78.145			90.000	-2578.063	545.400	-5454.000	0.000	2511.390	85.153	1804.644
26	4.000					433.365	-1771.688	545.400	-5454.000	0.000	1242.556	95.812	1240.182
27	Base					466.497	-1835.379	545.400	-5454.000	0.000	1290.139	93.937	1284.765

Table A.3 CADAM Results (continued-10)

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	104.000	*4.944*	*0.244*	*5.648*	*1.258*	*3.704*	-547.4	1294.6	1968.7	85.964		
2	100.000	*2.657*	*0.260*	*7.009*	*0.860*	*3.704*	-1328.4	2952.6	10392.3	111.119		
3	96.000	*2.362*	*0.282*	*6.831*	*0.897*	*3.704*	-2467.0	5058.6	26256.2	107.842		
4	92.000	*2.201*	*0.307*	*6.586*	*0.954*	*3.704*	-4014.3	7549.6	51354.7	103.303		
5	88.000	*1.988*	*0.302*	*5.529*	*0.940*	*3.026*	-5504.3	10537.8	89250.3	104.779	508.2	0.000
6	84.000	*1.824*	*0.298*	*4.860*	*0.939*	*2.652*	-7233.4	14012.4	140775.4	105.289	1294.9	0.000
7	80.000	*1.714*	*0.300*	*4.441*	*0.947*	*2.445*	-9261.4	17822.5	207223.8	104.841	2301.4	0.000
8	76.000	*1.641*	*0.306*	*4.152*	*0.959*	*2.315*	-11588.3	21868.2	289630.8	103.865	3527.7	0.000
9	72.000	*1.595*	*0.315*	*3.937*	*0.973*	*2.227*	-14214.0	26079.7	388686.2	102.587	4973.7	0.000
10	68.000	*1.568*	*0.325*	*3.769*	*0.988*	*2.163*	-17138.5	30406.6	504816.4	101.137	6639.4	0.000
11	64.000	*1.555*	*0.338*	*3.633*	*1.004*	*2.116*	-20362.0	34813.9	638238.6	99.596	8524.9	0.000
12	60.000	*1.552*	*0.351*	*3.518*	*1.020*	*2.079*	-23884.2	39279.8	789013.2	98.016	10630.1	0.000
13	56.000	*1.556*	*0.365*	*3.421*	*1.036*	*2.050*	-27705.4	43795.1	957091.5	96.432	12955.1	0.000
14	52.000	*1.564*	*0.380*	*3.335*	*1.051*	*2.025*	-31827.9	48410.6	1142326.6	94.863	15499.8	0.000
15	48.000	*1.546*	*0.380*	*3.084*	*1.056*	*1.936*	-35044.0	53180.6	1325083.5	94.173	19523.9	0.000
16	44.000	*1.525*	*0.379*	*2.839*	*1.058*	*1.847*	-38063.5	58015.4	1512655.3	93.575	24379.8	0.000
17	40.000	*1.512*	*0.379*	*2.647*	*1.062*	*1.778*	-41298.7	62916.2	1709988.7	92.774	29675.3	0.000
18	36.000	*1.503*	*0.380*	*2.494*	*1.068*	*1.722*	-44755.8	67913.6	1916556.1	91.819	35410.2	0.000
19	32.000	*0.992*	*0.383*	*2.370*	*1.074*	*1.678*	-48438.2	73048.9	263183.8	90.756	41584.6	0.000
20	28.000	*1.029*	*0.386*	*2.266*	*1.081*	*1.641*	-52348.5	78371.6	338325.2	89.629	48198.5	0.000
21	24.000	*1.067*	*0.389*	*2.180*	*1.087*	*1.610*	-56488.4	83935.1	428833.6	88.478	55251.9	0.000
22	20.000	*1.101*	*0.391*	*2.106*	*1.094*	*1.585*	-60859.5	89792.7	535268.5	87.339	62744.8	0.000
23	16.000	*1.475*	*0.394*	*2.044*	*1.100*	*1.563*	-65462.7	95992.1	3093669.6	86.241	70677.1	0.000
24	12.000	*1.111*	*0.389*	*2.001*	*1.098*	*1.543*	-69282.8	102884.6	724001.5	86.039	77828.4	0.000
25	8.000	*1.071*	*0.381*	*1.967*	*1.094*	*1.525*	-72843.7	110244.4	762317.9	86.238	84820.0	0.000
26	4.000	*1.551*	*0.457*	*2.123*	*1.200*	*1.701*	-93348.0	117973.0	3575906.0	77.460	75145.8	0.000
27	Base	*0.540*	*0.281*	*2.085*	*1.187*	*1.674*	-97396.0	126085.8	3884278.0	78.026	82206.1	0.000
	Required:	1.300	1.000	1.100	1.100	1.100						

Table A.3 CADAM Results (continued-11)

SEISMIC #2 COMBINATION - SUSTAINED ACCELERATIONS (STRESS ANALYSIS)													
Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable 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	104.000					-6.178	-124.300	545.400	-5454.000	0.000	97.096	50.000	0.000
2	100.000					66.606	-313.971	545.400	-5454.000	-46.624	219.780	100.000	219.780
3	96.000					72.868	-392.446	545.400	-5454.000	-51.008	274.712	100.000	274.712
4	92.000					68.130	-466.814	545.400	-5454.000	-47.691	326.770	100.000	326.770
5	88.000					87.766	-537.045	545.400	-5454.000	-61.436	375.932	100.000	375.932
6	84.000					102.593	-604.410	545.400	-5454.000	-71.815	423.087	100.000	423.087
7	80.000					112.431	-670.856	545.400	-5454.000	-78.702	469.599	100.000	469.599
8	76.000					118.753	-736.383	545.400	-5454.000	-83.127	515.468	100.000	515.468
9	72.000					122.142	-800.735	545.400	-5454.000	-85.499	560.514	100.000	560.514
10	68.000					122.910	-863.710	545.400	-5454.000	-86.037	604.597	100.000	604.597
11	64.000					121.277	-925.199	545.400	-5454.000	-84.894	647.639	100.000	647.639
12	60.000					117.435	-985.169	545.400	-5454.000	-82.204	689.618	100.000	689.618
13	56.000					111.568	-1043.650	545.400	-5454.000	-78.098	730.555	100.000	730.555
14	52.000					103.812	-1100.847	545.400	-5454.000	-72.669	770.593	100.000	770.593
15	48.000					92.220	-1127.078	545.400	-5454.000	-64.554	788.954	100.000	788.954
16	44.000					77.655	-1144.127	545.400	-5454.000	-54.359	800.889	100.000	800.889
17	40.000					60.353	-1161.707	545.400	-5454.000	-42.247	813.195	100.000	813.195
18	36.000					40.546	-1179.597	545.400	-5454.000	-28.382	825.718	100.000	825.718
19	32.000	71.450	77.166			0.000	-1198.013	545.400	-5454.000	0.000	838.609	100.000	838.609
20	28.000	67.880	77.111			-5.296	-1216.101	545.400	-5454.000	3.707	851.271	100.000	851.271
21	24.000	64.219	76.549			-30.570	-1234.874	545.400	-5454.000	21.399	864.412	100.000	864.412
22	20.000	60.583	75.608			-56.867	-1254.221	545.400	-5454.000	39.807	877.955	100.000	877.955
23	16.000					-83.775	-1274.361	545.400	-5454.000	58.643	892.053	100.000	892.053
24	12.000	56.470	75.613			-107.912	-1303.223	545.400	-5454.000	0.000	912.256	100.000	912.256
25	8.000	57.166	78.145			-128.459	-1336.910	545.400	-5454.000	0.000	935.837	100.000	935.837
26	4.000					-367.144	-1394.029	545.400	-5454.000	0.000	975.820	100.000	975.820
27	Base					-382.062	-1432.793	545.400	-5454.000	0.000	1002.955	100.000	1002.955

Table A.3 CADAM Results (continued-12)

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	104.000	13.367	0.582	9.712	2.548	7.692	-652.4	647.3	984.4	65.088		
2	100.000	6.795	0.619	11.124	1.736	7.692	-1583.1	1476.3	5196.1	75.642		
3	96.000	6.071	0.671	10.940	1.811	7.692	-2940.1	2529.3	13128.1	74.267		
4	92.000	5.706	0.732	10.685	1.926	7.692	-4784.2	3774.8	25677.3	72.363		
5	88.000	5.096	0.724	7.941	1.760	5.232	-6649.3	5299.0	45619.6	73.178	508.2	0.000
6	84.000	4.606	0.714	6.529	1.660	4.178	-8832.0	7144.1	73000.4	73.481	1294.9	0.000
7	80.000	4.260	0.712	5.760	1.607	3.662	-11391.9	9235.6	108657.7	73.378	2301.4	0.000
8	76.000	4.017	0.718	5.277	1.577	3.360	-14329.0	11523.3	153422.9	73.076	3527.7	0.000
9	72.000	3.846	0.729	4.945	1.560	3.163	-17643.4	13972.4	207954.9	72.666	4973.7	0.000
10	68.000	3.725	0.744	4.701	1.551	3.025	-21335.0	16557.7	272780.6	72.197	6639.4	0.000
11	64.000	3.640	0.761	4.514	1.547	2.924	-25403.9	19261.7	348323.0	71.695	8524.9	0.000
12	60.000	3.580	0.781	4.365	1.546	2.846	-29850.1	22073.4	434925.7	71.178	10630.1	0.000
13	56.000	3.539	0.801	4.243	1.548	2.785	-34673.4	24988.3	532878.8	70.657	12955.1	0.000
14	52.000	3.508	0.821	4.142	1.551	2.734	-39881.4	28040.3	642485.0	70.137	15499.8	0.000
15	48.000	3.442	0.818	3.750	1.531	2.571	-44291.9	31254.5	744519.4	69.637	19523.9	0.000
16	44.000	3.387	0.813	3.386	1.509	2.415	-48631.1	34532.9	846841.8	69.094	24379.8	0.000
17	40.000	3.351	0.813	3.118	1.493	2.297	-53305.5	37873.4	954250.0	68.493	29675.3	0.000
18	36.000	3.330	0.815	2.913	1.482	2.205	-58319.4	41290.9	1066176.8	67.853	35410.2	0.000
19	32.000	1.972	0.820	2.752	1.474	2.132	-63675.4	44806.1	1128134.2	67.191	41584.6	0.000
20	28.000	2.035	0.827	2.621	1.469	2.073	-69375.3	48443.7	1302115.9	66.522	48198.5	0.000
21	24.000	2.097	0.834	2.514	1.466	2.024	-75420.5	52230.3	1425959.8	65.861	55251.9	0.000
22	20.000	2.156	0.841	2.424	1.463	1.984	-81811.9	56192.6	1554069.8	65.221	62744.8	0.000
23	16.000	3.196	0.847	2.348	1.461	1.950	-88550.5	60354.4	1687075.8	64.611	70677.1	0.000
24	12.000	2.175	0.841	2.290	1.456	1.923	-94475.4	64888.1	1785914.7	64.118	77828.4	0.000
25	8.000	2.110	0.830	2.241	1.450	1.899	-100158.0	69683.0	1881848.1	63.745	84820.0	0.000
26	4.000	3.139	0.950	2.468	1.645	2.184	-122841.8	74690.6	1665286.9	59.718	75145.8	0.000
27	Base	1.110	0.588	2.409	1.623	2.144	-129127.0	79918.6	1773047.3	59.649	82206.1	0.000
	Required:	1.300	1.000	1.100	1.100	1.100						

Table A.3 CADAM Results (continued-13)

POST-SEISMIC COMBINATION (STRESS ANALYSIS)													
Joint		Cracking				Stresses							
		Upstream		Downstream		Normal stresses		alowable 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	104.000					-74.988	-74.988	400.200	-4002.000				
2	100.000					-142.164	-142.164	400.200	-4002.000				
3	96.000					-183.665	-183.665	400.200	-4002.000				
4	92.000					-229.129	-229.129	400.200	-4002.000				
5	88.000					-246.939	-274.179	400.200	-4002.000	172.858	-88.235	49.119	191.925
6	84.000					-267.774	-318.382	400.200	-4002.000	187.442	-91.085	48.504	222.867
7	80.000					-291.326	-364.073	400.200	-4002.000	203.928	-91.352	48.012	254.851
8	76.000					-315.688	-411.639	400.200	-4002.000	220.982	-90.007	47.558	288.148
9	72.000					-340.129	-460.944	400.200	-4002.000	238.090	-87.605	47.118	322.660
10	68.000					-364.366	-511.738	400.200	-4002.000	255.056	-84.485	46.689	358.216
11	64.000					-388.303	-563.778	400.200	-4002.000	271.812	-80.864	46.271	394.645
12	60.000					-411.916	-616.856	400.200	-4002.000	288.341	-76.887	45.868	431.799
13	56.000					-435.220	-670.796	400.200	-4002.000	304.654	-72.652	45.480	469.557
14	52.000					-459.931	-724.631	400.200	-4002.000	321.952	-67.995	45.156	507.242
15	48.000					-488.726	-748.702	400.200	-4002.000	342.108	-57.425	45.322	524.091
16	44.000					-519.443	-765.838	400.200	-4002.000	363.610	-47.122	45.629	536.087
17	40.000					-551.925	-785.246	400.200	-4002.000	386.348	-38.785	45.945	549.672
18	36.000					-586.066	-806.371	400.200	-4002.000	410.246	-32.093	46.268	564.459
19	32.000					-621.745	-828.806	400.200	-4002.000	435.222	-26.793	46.595	580.164
20	28.000					-658.841	-852.254	400.200	-4002.000	461.188	-22.686	46.920	596.578
21	24.000					-697.234	-876.490	400.200	-4002.000	488.064	-19.608	47.242	613.543
22	20.000					-736.814	-901.344	400.200	-4002.000	515.770	-17.429	47.557	630.941
23	16.000					-777.478	-926.686	400.200	-4002.000	544.235	-16.036	47.865	648.680
24	12.000					-836.500	-941.568	400.200	-4002.000	0.000	-4.444	7.565	659.098
25	8.000					-899.891	-953.862	400.200	-4002.000	0.000	-1.306	4.231	667.704
26	4.000					-1182.903	-988.022	400.200	-4002.000	0.000	-0.472	2.544	691.615
27	Base					-1243.952	-1001.942	400.200	-4002.000	0.000	701.359	100.000	701.359

Table A.3 CADAM Results (continued-14)

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	104.000	> 100	> 100	> 100	> 100	> 100	-749.9	0.0	0.0	50.000		
2	100.000	> 100	> 100	> 100	> 100	> 100	-1819.7	0.0	0.0	50.000		
3	96.000	> 100	> 100	> 100	> 100	> 100	-3379.4	0.0	0.0	50.000		
4	92.000	> 100	> 100	> 100	> 100	> 100	-5499.1	0.0	0.0	50.000		
5	88.000	> 100	74.108	24.163	12.107	16.177	-7712.6	60.1	1988.9	50.871	508.2	0.000
6	84.000	> 100	21.588	13.294	6.674	8.967	-10316.3	275.9	5225.4	51.439	1294.9	0.000
7	80.000	70.928	11.900	10.037	5.036	6.810	-13370.1	648.7	10091.5	51.850	2301.4	0.000
8	76.000	45.819	8.267	8.490	4.248	5.783	-16874.0	1178.4	17215.0	52.199	3527.7	0.000
9	72.000	33.471	6.447	7.597	3.787	5.188	-20827.9	1865.1	27223.6	52.514	4973.7	0.000
10	68.000	26.326	5.378	7.020	3.484	4.800	-25231.8	2708.8	40745.2	52.804	6639.4	0.000
11	64.000	21.741	4.683	6.619	3.269	4.529	-30085.8	3709.4	58407.7	53.072	8524.9	0.000
12	60.000	18.580	4.198	6.325	3.110	4.329	-35389.8	4867.0	80839.0	53.320	10630.1	0.000
13	56.000	16.285	3.843	6.102	2.987	4.176	-41143.8	6181.5	108666.7	53.550	12955.1	0.000
14	52.000	14.524	3.567	5.927	2.891	4.057	-47382.5	7668.9	141173.0	53.724	15499.8	0.000
15	48.000	13.025	3.280	5.090	2.738	3.713	-52961.9	9323.6	158744.9	53.502	19523.9	0.000
16	44.000	11.918	3.065	4.397	2.601	3.404	-58608.8	11039.6	170781.4	53.195	24379.8	0.000
17	40.000	11.096	2.916	3.938	2.496	3.181	-64719.1	12811.9	182189.6	52.908	29675.3	0.000
18	36.000	10.465	2.811	3.612	2.415	3.013	-71292.8	14640.6	192505.0	52.637	35410.2	0.000
19	32.000	9.968	2.737	3.369	2.350	2.884	-78329.8	16525.5	201263.0	52.379	41584.6	0.000
20	28.000	9.569	2.683	3.181	2.298	2.781	-85830.2	18466.9	207999.0	52.133	48198.5	0.000
21	24.000	9.243	2.646	3.033	2.255	2.698	-93793.9	20464.5	212248.4	51.898	55251.9	0.000
22	20.000	8.973	2.621	2.912	2.220	2.629	-102221.0	22518.5	213546.8	51.674	62744.8	0.000
23	16.000	8.747	2.605	2.812	2.190	2.572	-111111.5	24628.9	211429.5	51.459	70677.1	0.000
24	12.000	8.440	2.565	2.724	2.171	2.530	-119041.7	26795.6	156981.6	50.985	77828.4	0.000
25	8.000	8.135	2.521	2.647	2.154	2.494	-126704.0	29018.6	84046.0	50.485	84820.0	0.000
26	4.000	8.404	2.793	3.012	2.615	3.015	-151422.0	31297.9	-316035.9	48.504	75145.8	0.000
27	Base	3.062	1.729	2.911	2.564	2.944	-159795.3	33633.6	-408377.9	48.204	82206.1	0.000
	Required:	2.000	1.100	1.100	1.100	1.100						

APPENDIX B

DETERMINISTIC ANALYSES

❖ Actual cross-section:

Crest elevation	: 272.00 m
Crest thickness	: 10.00 m
Elevation of foundation	: 165.00 m
Upstream face slope	: 0.70
Downstream face slope	: 0.70
Bottom width	: 142.30 m

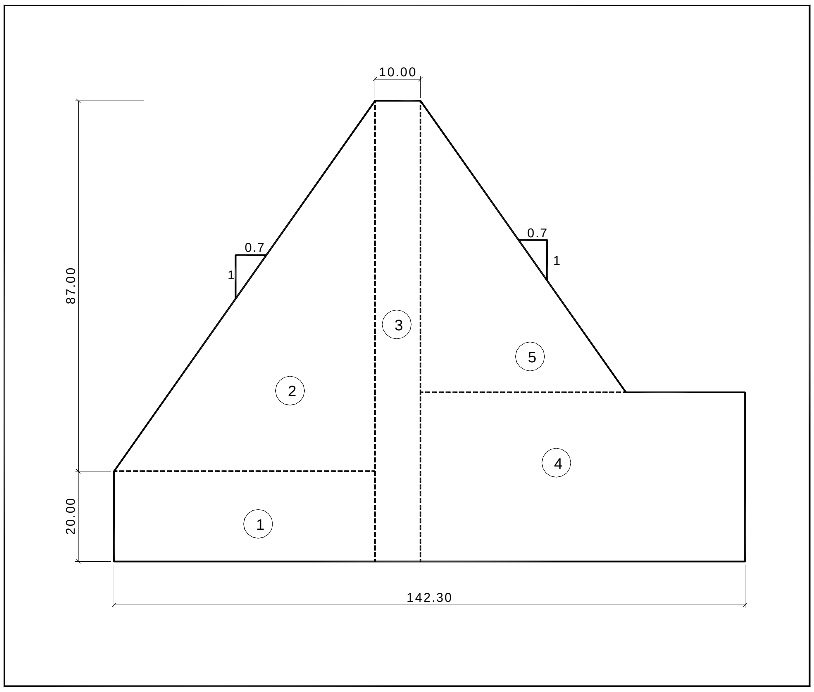


Figure B.1 Actual Cross-section of the Dam

Table B.1 Geometry of the Dam Body

Section	Area (m^2)	x (m)	y (m)	A.x	A.y
1	913.50	111.85	7.50	102174.98	6851.25
2	2649.15	101.70	44.00	269418.56	116562.60
3	1070.00	76.40	53.50	81748.00	57245.00
4	3748.50	35.70	26.25	133821.45	98398.13
5	857.59	59.85	69.00	51326.61	59173.54
TOTAL	9238.74			638489.59	338230.51
			X_0 (m)	69.11	
			Y_0 (m)		36.61

Usual Combination:

- Dam body

$F_{y1} = area \ x \ \gamma_{hardfill}$

$= 9238.74 \text{ x } 25 = 230968$ kN/m

$M = F_{y1} \ x \ xo_1$

$= 230968 \text{ x } 69.11 = 15962198$ kN.m/m (R)

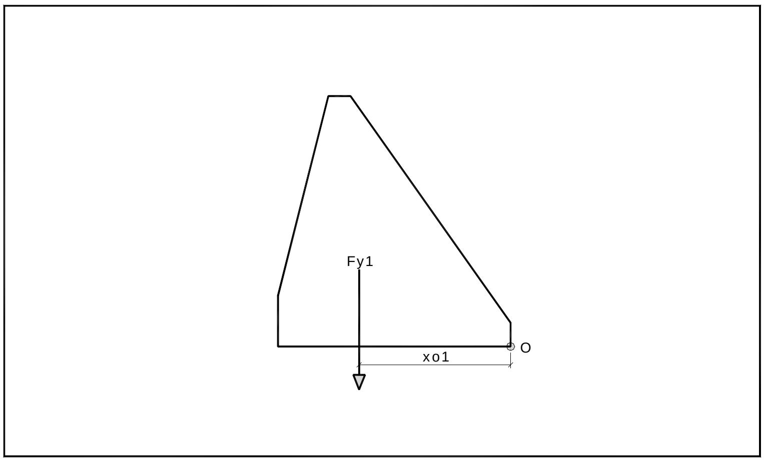


Figure B.2 Dead Load (Weight) of the Dam

- Upstream Reservoir Loads

$F_{x2} = 0.5 \ x \ H^2 \ x \ \gamma_{water}$

$= 0.5 \text{ x } 91.50^2 \text{ x } 9.81 = 41066 \text{ kN/m}$

$M \ = F_{x2} \ x \ yo_2$

$= 41066 \text{ x } 91.50/3 = 1252513 \text{ kN.m/m (O)}$

$F_{y2} = 0.5 \ x \ h \ x \ b \ x \ \gamma_{water}$

$= 0.5 \text{ x } 76.50 \text{ x } (0.7 \text{ x } 76.50) \text{ x } 9.81 = 20094 \text{ kN/m}$

$M \ = F_{y2} \ x \ xo_2$

$= 20094 \text{ x } (142.30-(0.7 \text{ x } 76.50)/3) = 2500698 \text{ kN.m/m (R)}$

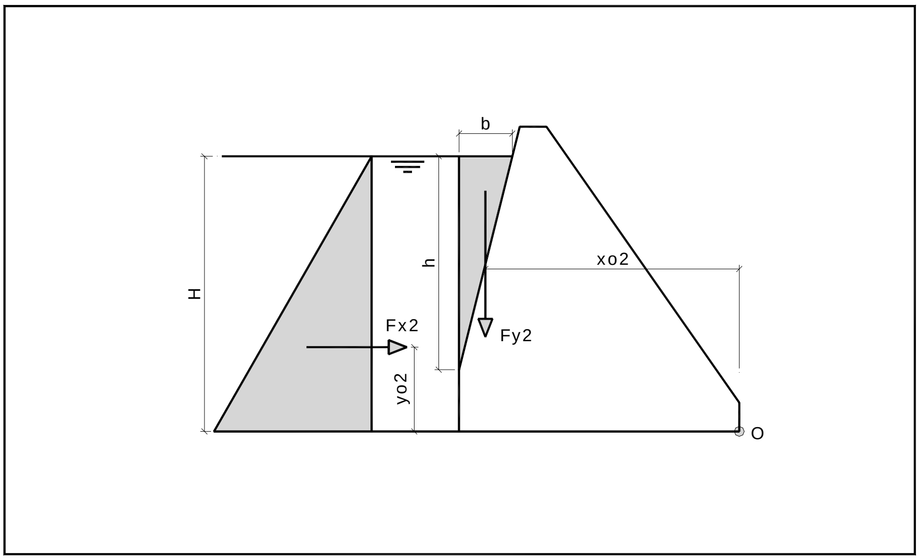


Figure B.3 Hydrostatic Forces in the Upstream

- Downstream Hydrostatic Loads

$F_{x3} = 0.5 \ x \ H^2 \ x \ \gamma_{water}$

$= 0.5 \text{ x } 51^2 \text{ x } 9.81 = 12758 \text{ kN/m}$

$M \ = F_{x3} \ x \ yo_3$

$= 12758 \text{ x } 51/3 = 216886 \text{ kN.m/m (R)}$

$F_{y3} = 0.5 \times h \times b \times \gamma_{water}$

$= 0$ kN

$M = F_{y3} \times xo_3$

$= 0$ kN.m/m (R)

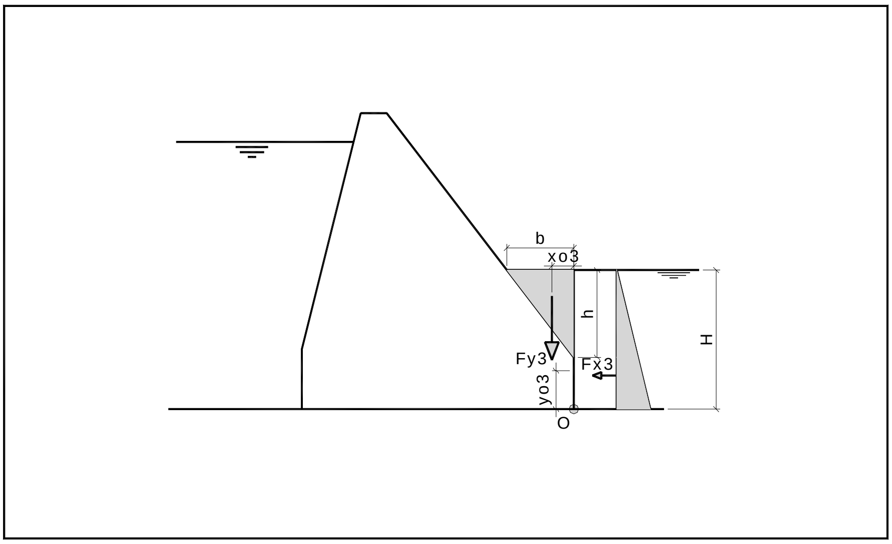


Figure B.4 Hydrostatic Forces in the Downstream

- Upstream Fill Material and Silt Loads

$Fy_{m1} = area \times \gamma_{silt\text{-}sub}$

$= 420 \times 11 = 4620$ kN/m

$M = Fy_{m1} \times xo_{m1}$

$= 4620 \times 131.30 = 606606$ kN.m/m (R)

$Fy_{m2} = area \times \gamma_{fill\text{-}sub}$

$= 140 \times 11.19 = 1567$ kN/m

$M = Fy_{m2} \times xo_{m2}$

$= 1567 \times 137.63 = 215666$ kN.m/m (R)

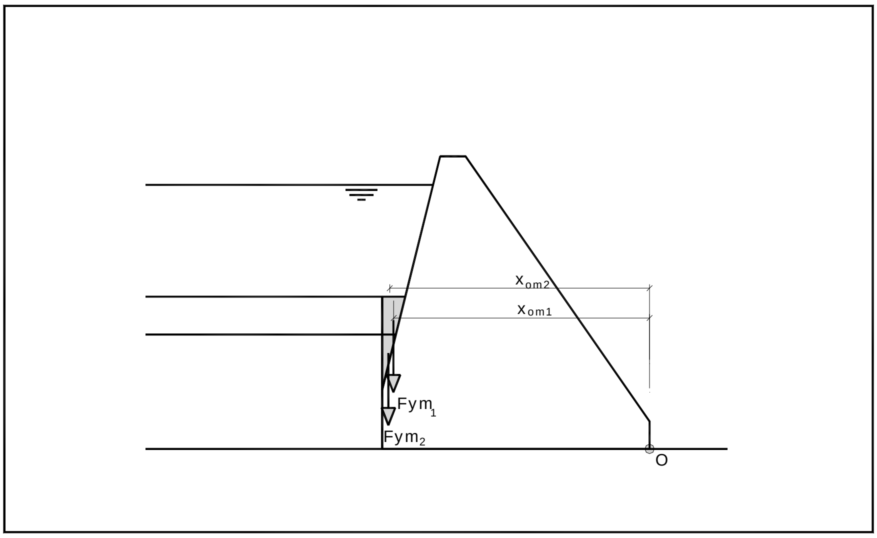


Figure B.5 Weight of the Fill Material and Silt

- Uplift Force is calculated using method proposed by USACE (1995).

U1 = $H_{water\text{-}upstream}$ = 91.50 m

U2 = $H_{water\text{-}downstream}$ = 51.00 m

x = 12.00 m

L = 142.30 m

U3 = U2 + (U1-U2)/3 x (L-x)/L = 63.36 m

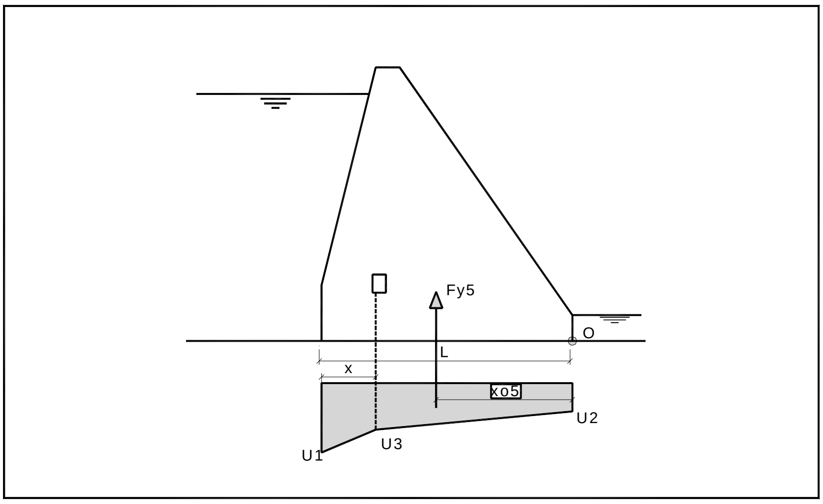


Figure B.6 Uplift Force Acting Under the Base of the Dam

$Fy_5 = area \times \gamma_{water}$

$= 8379.70 \times 9.81 = 82205$ kN/m

$M = Fy_5 \times xo_5$

$= 82205 \times 75.17 = 6179350$ kN.m/m (O)

Table B.2 Summary of the Forces and Moments Acting on the Dam (Usual Combination)

LOAD	Fx (kN/m)	Fy (kN/m)	$M_{Overturning}$ (kN.m/m)	$M_{Resisting}$ (kN.m/m)
Dam		230968		15962198
Hydrostatic $_{upstream}$	41066	20094	1252513	2500698
Hydrostatic $_{downstream}$	-12758	0		216886
Fill and silt		6187		822277
Uplift		-82205	6179350	
TOTAL	28308	175044	7431863	19502059
$FS_{Overturning}$	**2.62**			
$FS_{Sliding}$	**3.89**			

Base Pressures:

$\sigma_{1\ (upst.)} = V/b*(1 + 6*e/b)$

$\sigma_{2\ (downst.)} = V/b*(1 - 6*e/b)$

$X = \sum M / V = 12070196 / 175044 = 68.96$ m

$e = X - b/2 = 68.96 - 142.30/2 = -2.19$ m

⟹ $\sigma_{1\ (upst.)} = 1116.52$ kPa

$\sigma_{2\ (downst.)} = 1343.69$ kPa

$FS_{Overturning} = M_R / M_O = 19502059 / 7431863 = 2.62$

$FS_{Sliding} = F_R / F_S = (c \times L + \sum F_y \times \tan \phi) / F_S = (200\times142.30+175044 \times \tan 25) / 28308 = 3.89$

Seismic-1 (pseudo-static) Combination:

- Dam body

$F_{x1} = F_{y1} \times a_o$

$= 230968 \times 0.20 = 46194$ kN/m

$M = F_{x1} \times yo_1$

= 46194 x 36.61 = 1691162 kN.m/m (O)

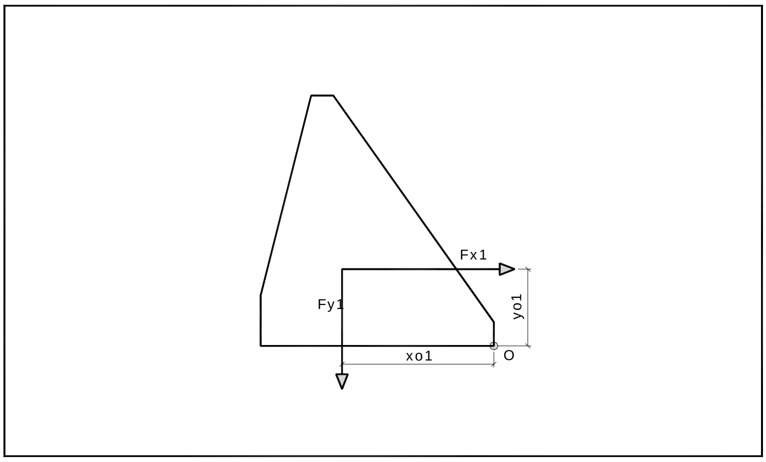


Figure B.7 Earthquake Force on the Dam Body

- Hydrodynamic Forces

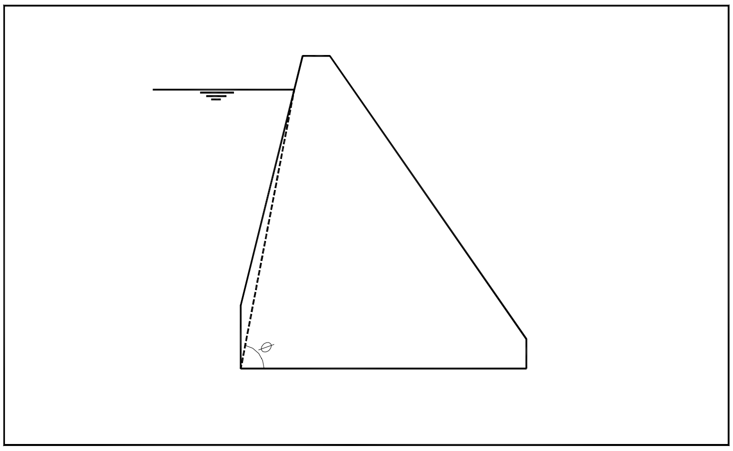


Figure B.8 Hydrodynamic Pressures on Sloping Dams

According to Chawang and Housner (1978), hydrodynamic pressures on sloping dams during earthquakes can be calculated using the following procedure:

$\beta = cot\ \phi$ = (256.50-180)*0.75/(256.50-165) = 0.63

$$\frac{b_o}{h} = \frac{1}{2^{1/2}} \exp\left\{ -\frac{\beta}{(8-\beta^2)^{1/2}} \left[\frac{\pi}{2} - \tan^{-1}\left(\frac{\beta}{(8-\beta^2)^{1/2}}\right) \right] \right\} = 0.52$$

Hydrodynamic coefficients, $C_y = ½ - (b_o/h)^2 = 0.2296$

$C_x = C_y / \beta = 0.3644$

$F_{y2} = a_o \text{ x } C_y \text{ x } H^2 \text{ x } \gamma_{water}$

$= 0.20 \text{ x } 0.22016 \text{ x } (256.50\text{-}220)^2 \text{ x } 9.81 = 575$ kN/m

$M = F_{y2} \text{ x } xo_2$

$= 575 \text{ x } 101.20 = 58190$ kN.m/m (R)

$F_{x2} = a_o \text{ x } C_x \text{ x } H^2 \text{ x } \gamma_{water}$

$= 0.20 \text{ x } 0.37320 \text{ x } (256.50\text{-}220)^2 \text{ x } 9.81 = 975$ kN/m

$M = F_{x2} \text{ x } yo_2$

$= 975 \text{ x } (0.4 \text{ x } (256.50 - 220)+220-165) = 67860$ kN.m/m (O)

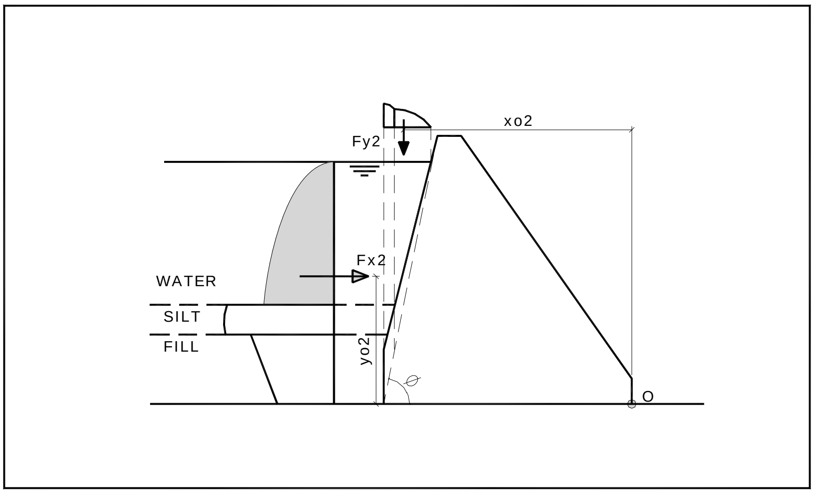


Figure B.9 Hydrodynamic Force

$a = a_o \text{ x } C_y \text{ x } H^2 \text{ x } \gamma_{silt}$

$= 0.20 \text{ x } 0.22016 \text{ x } (256.50\text{-}200)^2 \text{ x } 20.81 = 2925$ kN/m

$b = a_o \text{ x } C_y \text{ x } H^2 \text{ x } \gamma_{silt}$

$= 0.20 \text{ x } 0.22016 \text{ x } (256.50\text{-}220)^2 \text{ x } 20.81 = 1221$ kN/m

$F_{y3} = a - b$

$= 2925\text{-}1221 = 1704$ kN/m

$M = F_{y3} \text{ x } xo_3$

$= 1704 \times 116.93 = 199249$ kN.m/m (R)

$a = a_o \; x \; C_x \; x \; H^2 \; x \; \gamma_{silt}$

$= 0.20 \times 0.37320 \times (256.50\text{-}200)^2 \times 20.81 = 4958$ kN/m

$b = a_o \; x \; C_x \; x \; H^2 \; x \; \gamma_{silt}$

$= 0.20 \times 0.37320 \times (256.50\text{-}220)^2 \times 20.81 = 2069$ kN/m

$F_{x3} = a - b$

$= 4958 – 2069 = 2889$ kN/m

$M \;\; = F_{x3} \; x \; yo_3$

$= 2889 \times 43 = 124227$ kN.m/m (O)

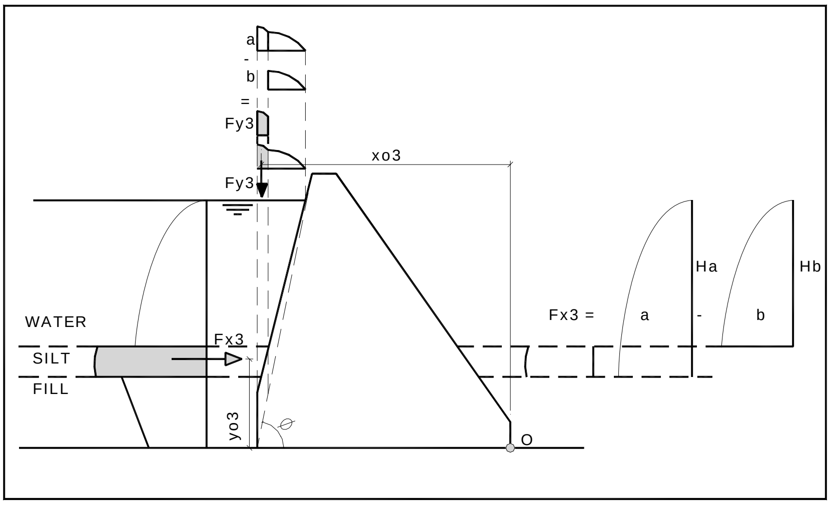


Figure B.10 Dynamic Silt Force

Ka = 0.290 (Coulomb earth pressure coefficient)

Kae = 0.437 (Coulomb earth pressure coefficient during earthquake)

$\Longrightarrow$ K = 0.437 – 0.290 = 0.147

$a = K \; x \; \gamma_{silt\text{-}sat} \; x \; H_{silt} + K \; x \; \gamma_{water} \; x \; H_{water}$

$= 0.147 \times 20.81 \times (220\text{-}200) + 0.147 \times 9.81 \times (256.50\text{-}220) = 113.82$

$b = K \; x \; \gamma_{fill\text{-}sat} \; x \; H_{fill}$

$= 0.147 \times 21 \times (200\text{-}165)^2 = 108.05$

$c = a + b = 221.87$

$F_{x4} = (a + c) \; x \; H / 2$

$= (113.82 + 221.87) \times (200\text{-}165) / 2 = 5874$ kN/m

$M \;\; = F_{x4} \; x \; yo_4$

$= 5874 \times 19.378 = 113831$ kN.m/m (O)

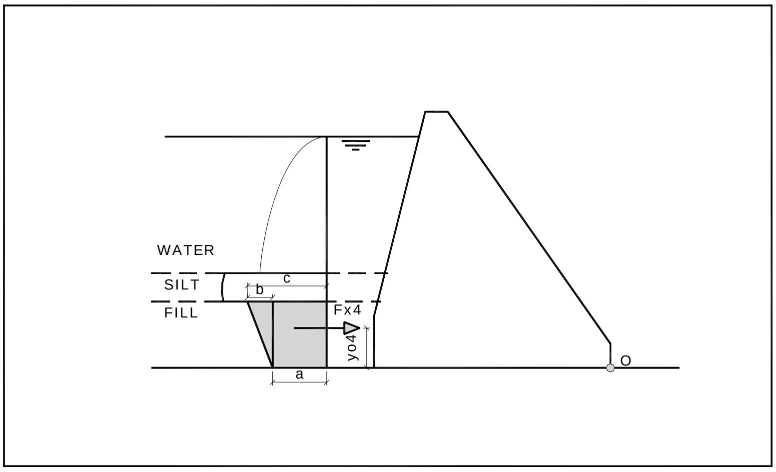


Figure B.11 Earthquake Effect of Fill Material

Table B.3 Summary of the Forces and Moments Acting on the Dam (Seismic-1 Combination)

LOAD	Fx (kN/m)	Fy (kN/m)	M Overturning (kN.m/m)	M Resisting (kN.m/m)
Dam	46194	230968	1691162	15962198
Hydrostatic upstream	41066	20094	1252513	2500698
Hydrostatic downstream	-12758	0	0	216886
Hydrodynamic water	975	575	67860	58190
Hydrodynamic silt	2889	1704	124227	199249
Earthquake fill	5874		113831	
Fill and silt		6187		822277
Uplift		-82205	6179350	
TOTAL	84240	177323	9428943	19759498
FS Overturning	**2.10**			
FS Sliding	**1.32**			

Base Pressures:

$\sigma_{1\,(upst.)} = V/b*(1 + 6*e/b)$

$\sigma_{2\,(downst.)} = V/b*(1 - 6*e/b)$

$X = \sum M / V = 10330555 / 177323 = 58.26$ m

$e = X - b/2$ = 58.26 – 142.30/2 = -12.89 m

$\Longrightarrow$ $\sigma_{1\ (upst.)}$ = 568.85 kPa

$\sigma_{2\ (downst.)}$ = 1923.39 kPa

$FS_{\ Overturning} = M_R / M_O$ = 19759498 / 9428943 = 2.10

$FS_{\ Sliding} = F_R / F_S = (c \times L + \sum F_y \times \tan\phi) / F_S$ = (200x142.30+177323 x tan 25) / 84240 =1.32

Flood Combination:

- Upstream Reservoir Loads

$F_{x2} = 0.5 \times H^2 \times \gamma_{water}$

= 0.5 x 102.70^2 x 9.81 = 51734 kN/m

$M = F_{x2} \times yo_2$

= 51734 x 102.70/3 = 1771027 kN.m/m (O)

$F_{y2} = 0.5 \times h \times b \times \gamma_{water}$

= 0.5 x 87.70 x (0.7 x 87.70) x 9.81 = 26408 kN/m

M = F_{y2} x xo_2

= 26408 x (142.30-(0.7 x 87.70)/3) = 3217463 kN.m/m (R)

- Downstream Hydrostatic Loads

$F_{x3} = 0.5 \times H^2 \times \gamma_{water}$

= 0.5 x 52.50^2 x 9.81 = 13519 kN/m

$M = F_{x3} \times yo_3$

= 13519 x 52.50/3 = 236583 kN.m/m (R)

$F_{y3} = 0.5 \times h \times b \times \gamma_{water}$

= 0 kN

$M = F_{y3} \times xo_3$

= 0 kN.m/m (R)

- Uplift Force

$U1 = H_{water\text{-}upstream}$ = 102.70 m

$U2 = H_{water\text{-}downstream}$ = 52.50 m

x = 12.00 m

L = 142.30 m

$U3 = U2 + (U1\text{-}U2)/3 \times (L\text{-}x)/L$ = 67.82 m

$Fy_5 = area \times \gamma_{water}$

= 8862 x 9.81 = 86936 kN/m

$M = Fy_5 \times xo_5$

= 86936 x 75.86 = 6594965 kN.m/m (O)

Table B.4 Summary of the Forces and Moments Acting on the Dam (Flood Combination)

LOAD	Fx (kN/m)	Fy (kN/m)	M Overturning (kN.m/m)	M Resisting (kN.m/m)
Dam		230968		15962198
Hydrostatic upstream	51734	26408	1771027	3217463
Hydrostatic downstream	-13519	0		236583
Fill and silt		6187		822277
Uplift		-86936	6594965	
TOTAL	38215	176627	8365992	20238521
FS Overturning	**2.42**			
FS Sliding	**2.90**			

Base Pressures:

$\sigma_{1\ (upst.)} = V/b*(1 + 6*e/b)$

$\sigma_{2\ (downst.)} = V/b*(1 - 6*e/b)$

$X = \sum M / V$ = 11872529 / 176627 = 67.22 m

$e = X – b/2$ = 67.22 – 142.30/2 = -3.93m

⟹ $\sigma_{1\ (upst.)}$ = 1035.55 kPa

$\sigma_{2\ (downst.)}$ = 1446.91 kPa

$FS_{Overturning} = M_R / M_O$ = 20238521 / 8365992 = 2.42

$FS_{Sliding} = F_R / F_S = (c \times L + \sum F_y \times \tan \phi) / F_S$ = (200x142.30+176627 x tan 25) / 38215 =2.90

❖ New cross-section: Upstream and downstream slopes are increased to 0.75.

Crest elevation	: 272.00 m
Crest thickness	: 10.00 m
Elevation of foundation	: 165.00 m
Upstream face slope	: 0.75
Downstream face slope	: 0.75
Bottom width	: 151.75 m

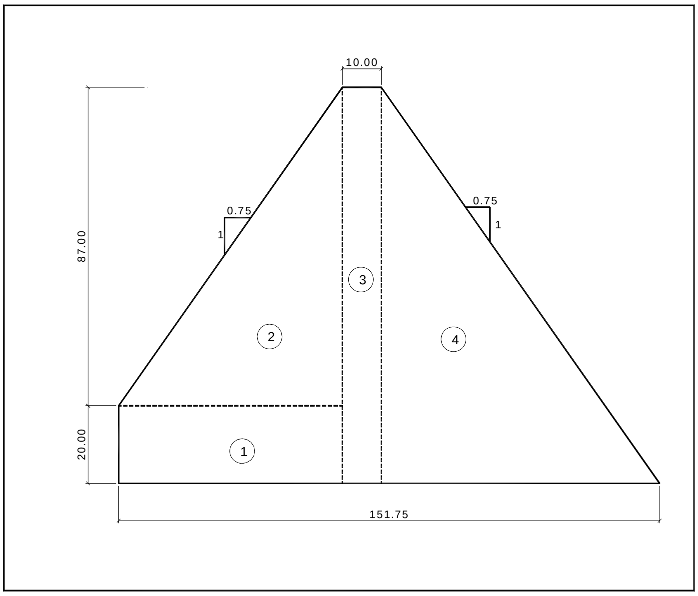


Figure B.12 New Cross-Section of the Dam

Table B.5 Geometry of the Dam Body

Section	Area (m^2)	X (m)	Y (m)	A.x	A.y
1	978.75	119.13	7.50	116593.59	7340.63
2	2838.38	108.25	44.00	307254.09	124888.50
3	1070.00	81.50	53.50	87205.00	57245.00
4	3901.50	51.00	34.00	198976.50	132651.00
TOTAL	8788.63			710029.19	322125.13
			X_0 (m)	80.79	
			Y_0 (m)		36.65

Usual Combination:

- Dam body

$F_{y1} = area \times \gamma_{hardfill}$

$= 8788.63 \times 25 = 219716$ kN/m

$M = F_{y1} \times xo_1$

$= 219716 \times 80.79 = 17750856$ kN.m/m (R)

- Upstream Reservoir Loads

$F_{x2} = 0.5 \times H^2 \times \gamma_{water}$

$= 0.5 \times 91.50^2 \times 9.81 = 41066$ kN/m

$M = F_{x2} \times yo_2$

$= 41066 \times 91.50/3 = 1252513$ kN.m/m (O)

$F_{y2} = 0.5 \times h \times b \times \gamma_{water}$

$= 0.5 \times 76.50 \times (0.75 \times 76.50) \times 9.81 = 21529$ kN/m

$M = F_{y2} \times xo_2$

$= 21529 \times (151.75-(0.75 \times 76.50)/3) = 2855284$ kN.m/m (R)

- Downstream Hydrostatic Loads

$F_{x3} = 0.5 \times H^2 \times \gamma_{water}$

$= 0.5 \times 51^2 \times 9.81 = 12758$ kN/m

$M = F_{x3} \times yo_3$

$= 12758 \times 51/3 = 216886$ kN.m/m (R)

$F_{y3} = 0.5 \times h \times b \times \gamma_{water}$

$= 0.5 \times (216\text{-}165) \times (0.75 \times (216\text{-}165)) \times 9.81 = 9568$ kN

$M = F_{y3} \times xo_3$

$= 9568 \times 12.75 = 121992$ kN.m/m (R)

- Upstream Fill Material and Silt Loads

$Fy_{m1} = area \times \gamma_{silt\text{-}sub}$

$= 450 \times 11 = 4950$ kN/m

$M = Fy_{m1} \times xo_{m1}$

$= 4950 \times 140.08 = 693396$ kN.m/m (R)

$Fy_{m2} = area \times \gamma_{fill\text{-}sub}$

$= 150 \times 11.19 = 1679$ kN/m

$M = Fy_{m2} \times xo_{m2}$

$= 1679 \times 146.75 = 246393$ kN.m/m (R)

- Uplift Force is calculated using method proposed by USACE (1995).

$U1 = H_{water\text{-}upstream} = 91.50$ m

$U2 = H_{water\text{-}downstream} = 51.00$ m

x = 12.00 m

L = 151.75 m

$U3 = U2 + (U1\text{-}U2)/3 \times (L\text{-}x)/L = 63.43$ m

$Fy_5 = area \times \gamma_{water}$

$= 8925.40 \times 9.81 = 87558$ kN/m

$M = Fy_5 \times xo_5$

$= 87558 \times 80.08 = 7011645$ kN.m/m (O)

Table B.6 Summary of the Forces and Moments Acting on the Dam (Usual Combination)

LOAD	Fx (kN/m)	Fy (kN/m)	$M_{Overturning}$ (kN.m/m)	$M_{Resisting}$ (kN.m/m)
Dam		219716		17750856
Hydrostatic $_{upstream}$	41066	21529	1252513	2855284
Hydrostatic $_{downstream}$	-12758	9568		338878
Fill and silt		6629		939789
Uplift		-87558	7011645	
TOTAL	28308	169884	8264158	21884807
$FS_{Overturning}$	**2.65**			
$FS_{Sliding}$	**3.87**			

Base Pressures:

$\sigma_{1\ (upst.)} = V/b*(1 + 6*e/b)$

$\sigma_{2\ (downst.)} = V/b*(1 - 6*e/b)$

$X = \sum M / V$ = 13620649 / 169884 = 80.18 m

$e = X - b/2$ = 80.18 – 151.75/2 = 4.305 m

⟹ $\sigma_{1\ (upst.)}$ = 1310.05 kPa

$\sigma_{2\ (downst.)}$ = 928.94 kPa

$FS_{Overturning} = M_R / M_O$ = 21884807 / 8264158 = 2.65

$FS_{Sliding} = F_R / F_S = (c \times L + \sum F_y \times \tan \phi) / F_S$ = (200x151.75+169884 x tan 25) / 28308 =3.87

Seismic-1 (pseudo-static) Combination:

- Dam body

$F_{x1} = F_{y1} \times a_o$

= 219716 x 0.20 = 43943 kN/m

$M = F_{x1} \times yo_1$

= 43943 x 36.65 = 1610511 kN.m/m (O)

- Hydrodynamic Forces

$F_{y2} = a_o \times C_y \times H^2 \times \gamma_{water}$

$= 0.20 \times 0.2296 \times (256.50\text{-}220)^2 \times 9.81 = 600$ kN/m

$M \ = F_{y2} \times xo_2$

$= 600 \times 107.90 = 64740$ kN.m/m (R)

$F_{x2} = a_o \times C_x \times H^2 \times \gamma_{water}$

$= 0.20 \times 0.3644 \times (256.50\text{-}220)^2 \times 9.81 = 952$ kN/m

$M \ = F_{x2} \times yo_2$

$= 952 \times (0.4 \times (256.50 -220)+220\text{-}165) = 66259$ kN.m/m (O)

$a = a_o \times C_y \times H^2 \times \gamma_{silt}$

$= 0.20 \times 0.2296 \times (256.50\text{-}200)^2 \times 20.81 = 3050$ kN/m

$b = a_o \times C_y \times H^2 \times \gamma_{silt}$

$= 0.20 \times 0.2296 \times (256.50\text{-}220)^2 \times 20.81 = 1273$ kN/m

$F_{y3} = a - b$

$= 3050\text{-}1273 = 1777$ kN/m

$M \ = F_{y3} \times xo_3$

$= 1777 \times 124.66 = 221521$ kN.m/m (R)

$a = a_o \times C_x \times H^2 \times \gamma_{silt}$

$= 0.20 \times 0.3644 \times (256.50\text{-}200)^2 \times 20.81 = 4841$ kN/m

$b = a_o \times C_x \times H^2 \times \gamma_{silt}$

$= 0.20 \times 0.3644 \times (256.50\text{-}220)^2 \times 20.81 = 2020$ kN/m

$F_{x3} = a - b$

$= 4841 – 2020 = 2821$ kN/m

$M \ = F_{x3} \times yo_3$

$= 2821 \times 43 = 121303$ kN.m/m (O)

Ka = 0.290 (Coulomb earth pressure coefficient)

Kae = 0.437 (Coulomb earth pressure coefficient during earthquake)

$\Rightarrow$ K = 0.437 – 0.290 = 0.147

$a = K \times \gamma_{silt\text{-}sat} \times H_{silt} + K \times \gamma_{water} \times H_{water}$

$= 0.147 \times 20.81 \times (220\text{-}200) + 0.147 \times 9.81 \times (256.50\text{-}220) = 113.82$

$b = K \times \gamma_{fill\text{-}sat} \times H_{fill}$

$= 0.147 \times 21 \times (200\text{-}165)^2 = 108.05$

$c = a + b = 221.87$

$F_{x4} = (a + c) \times H / 2$

$= (113.82 + 221.87) \times (200\text{-}165) / 2 = 5874$ kN/m

$M = F_{x4} \times yo_4$

$= 5874 \times 19.378 = 113831$ kN.m/m (O)

Table B.7 Summary of the Forces and Moments Acting on the Dam (Seismic-1 Combination)

LOAD	Fx (kN/m)	Fy (kN/m)	M Overturning (kN.m/m)	M Resisting (kN.m/m)
Dam	43943	219716	1610511	17750856
Hydrostatic upstream	41066	21529	1252513	2855284
Hydrostatic downstream	-12758	9568	-	338878
Hydrodynamic water	952	600	66259	64740
Hydrodynamic silt	2821	1777	121303	221521
Earthquake fill	5874	-	113831	-
Fill and silt		6629	0	939789
Uplift		-87558	7011645	
TOTAL	81898	172261	10176062	22171068
FS Overturning	**2.18**			
FS Sliding	**1.35**			

Base Pressures:

$\sigma_{1\ (upst.)} = V/b*(1 + 6*e/b)$

$\sigma_{2\ (downst.)} = V/b*(1 - 6*e/b)$

$X = \sum M / V = 11995006 / 172261 = 69.63$ m

$e = X - b/2 = 69.63 - 151.75/2 = -6.245$ m

$\Longrightarrow$ $\sigma_{1\ (upst.)} = 854.87$ kPa

$\sigma_{2\ (downst.)} = 1415.46$ kPa

$FS_{Overturning} = M_R / M_O$ = 22171068 / 10176062 = 2.18

$FS_{Sliding} = F_R / F_S = (c \times L + \sum F_y \times \tan \phi) / F_S$ = (200x151.75+172261 x tan 25) / 81898 =1.35

Flood Combination:

- Upstream Reservoir Loads

$F_{x2} = 0.5 \times H^2 \times \gamma_{water}$

= 0.5 x 102.70^2 x 9.81 = 51734 kN/m

$M = F_{x2} \times yo_2$

= 51734 x 102.70/3 = 1771027 kN.m/m (O)

$F_{y2} = 0.5 \times h \times b \times \gamma_{water}$

= 0.5 x 87.70 x (0.75 x 87.70) x 9.81 = 28294 kN/m

$M = F_{y2} \times xo_2$

= 28294 x (151.75-(0.75 x 87.70)/3) = 3673269 kN.m/m (R)

- Downstream Hydrostatic Loads

$F_{x3} = 0.5 \times H^2 \times \gamma_{water}$

= 0.5 x 52.50^2 x 9.81 = 13519 kN/m

$M = F_{x3} \times yo_3$

= 13519 x 52.50/3 = 236583 kN.m/m (R)

$F_{y3} = 0.5 \times h \times b \times \gamma_{water}$

= 0.5 x (217.50-165) x (0.75 x (217.50-165)) x 9.81 = 10140kN

$M = F_{y3} \times xo_3$

= 10140 x 13.13 = 133088 kN.m/m (R)

- Uplift Force

$U1 = H_{water\text{-}upstream}$ = 102.70 m

$U2 = H_{water\text{-}downstream}$ = 52.50 m

x = 12.00 m

L = 151.75 m

$U3 = U2 + (U1\text{-}U2)/3 \times (L\text{-}x)/L$ = 67.91 m

$Fy_5 = area \times \gamma_{water}$

= 9437.30 x 9.81 = 92580 kN/m

$M = Fy_5 \times xo_5$

= 92580 x 80.81 = 7481390 kN.m/m (O)

Table B.8 Summary of the Forces and Moments Acting on the Dam (Flood Combination)

LOAD	Fx (kN/m)	Fy (kN/m)	M $_{Overturning}$ (kN.m/m)	M $_{Resisting}$ (kN.m/m)
Dam		219716		17750856
Hydrostatic $_{upstream}$	51734	28294	1771027	3673269
Hydrostatic $_{downstream}$	-13519	10140		369671
Fill and silt		6629		939789
Uplift		-92580	7481390	
TOTAL	38215	172199	9252417	22733585
FS $_{Overturning}$	**2.46**			
FS $_{Sliding}$	**2.90**			

Base Pressures:

$\sigma_{1\ (upst.)} = V/b*(1 + 6*e/b)$

$\sigma_{2\ (downst.)} = V/b*(1 - 6*e/b)$

$X = \sum M / V$ = 13481168 / 172199 = 78.29 m

$e = X - b/2$ = 78.29 – 151.75/2 = 2.415 m

$\Longrightarrow$ $\sigma_{1\ (upst.)}$ = 1243.11 kPa

$\sigma_{2\ (downst.)}$ = 1026.40 kPa

$FS_{\ Overturning} = M_R / M_O$ = 22733585 / 9252417 = 2.46

$FS_{\ Sliding} = F_R / F_S = (c \times L + \sum F_y \times \tan \phi) / F_S$ = (200x151.75+172199 x tan 25) / 38215 =2.90