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Seismic Stability and Deformation of Waba Dam

Submitted by

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A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfillment
of the requirements for the degree of Master of Applied Science

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Abstract

The stability and deformation of Waba Dam following new seismic requirements is evaluated using the prevailing methods of pseudo-static analysis and dynamic non-linear analyses. Waba Dam is a low but wide earthdam of clayey materials overlying a deep deposit of sensitive clay in Eastern Ontario with moderate seismicity. To assess stability, the instantaneous safety factor was calculated based on the conventional and a proposed method considering dynamic forces. While the dam is considered problematic from the pseudo static analysis, it is acceptable based on the dynamic analysis. Review of the various factors for the discrepancies shows that the force-based dynamic analysis is more reliable for dynamic stability analysis.

Deformation analyses show that there would be zero permanent deformation from the pseudo-static methods but some definite permanent deformations from an elasto-plastic analysis but within tolerable limits. Parametric studies have been conducted to cover possible ranges of soil parameters and ground motions. The results show that the performance of the dam will be acceptable.

to
the Baha'i youth in Iran
who are deprived from higher education

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List of Symbols

Symbol	Description
a_h	horizontal acceleration
a_{max}	peak ground acceleration
a_v	vertical acceleration
a_y	yield acceleration
d	horizontal distance
f_0, f_l, f_m and f_p	nested yield surfaces
f_{max}	maximum frequency component of accelerations
h	vertical distance
k_h	horizontal seismic coefficient
$(k_h)_{average}$	average of seismic coefficients
$[k_n(y)]_{max}$	maximum seismic coefficient for move n
k_v	vertical seismic coefficient
k_y	horizontal yield coefficient
$k(y)$	maximum seismic coefficient
$\bar{k}(y)$	SRSS of seismic coefficients
l	spatial element size
n	number of segments
p	annual probability of exceedance
r	radius of slip circle
u	estimated downslope movement caused by earthquake
v_{max}	maximum shear wave velocity
A_{loop}	area of the hysteretic loop
$[C]$	damping matrix
F_h	horizontal force act through the centroid of potential sliding mass
E	Young modulus
E_{ip}	equivalent Young modulus
E_u	undrained modulus of elasticity

$\vec{F}_j$	force vector acting on the j^{th} node
F_{min}	minimum safety factor through the earthquake
FS_{min}	minimum pseudo-static safety factor
FS_{sr}	safety factor by strength reduction technique
F_t	instantaneous safety factor
F_{tmin}	lowest instantaneous safety factor at a given instant
F_v	vertical force act through the centroid of potential sliding mass
G	shear modulus
G_{max}	maximum shear modulus
G_{sec}	secant shear modulus
G_{tan}	tangent shear modulus
$\vec{H}_j$	vector of distance between center of the slip circle and j^{th} node
I_a	arias intensity
$[K]$	stiffness matrix
M	earthquake moment magnitude
$[M]$	mass matrix
M_M	motivating moment
M_R	resisting moment
N_d	required acceleration for downhill movement
N_u	required acceleration for uphill movement
$[Q_1]$ and $[Q_2]$	constant matrices for plane stress analysis
S	actuating forces
S	shear strength of the soil
S_{ij}	deviatoric stress tensor
S_u	undrained shear strength of the soil
R	resisting forces
T_θ	first natural period of the embankment
V_s	shear wave velocity
W	weight of potential failure mass
W_D	dissipated energy
W_S	maximum stored energy

γ_{ave}	average shear strain
γ_c	shear strain amplitude
γ_r	residual shear strain
γ_{yield}	yielding strain
θ	angle between segment and horizontal direction
ε_{i-D}	drained axial strain before earthquake
ε_{i-UD}	undrained axial strain before earthquake
ε_i	axial strain before earthquake
ε_p	axial strain after earthquake
λ	wavelength associated with highest frequency component
ν	Poisson's ratio
ξ	damping ratio
ρ	density
σ_{3i}	confining pressure before earthquake
σ_{di}	deviatoric stress before earthquake
σ_x	normal stress acting through the x-coordinate direction
σ_y	normal stress acting through the y-coordinate direction
τ_{xy}	shear stress acting through the x- and y-coordinate directions
τ_{av}	average of shear strength of segments
τ_c	shear stress amplitude
τ_n	mobilized shear stress
τ_{max}	appropriate shear strength
τ_r	residual shear stress
ΔL_i	length of segment i
$\Delta \sigma_d$	incremental dynamic deviatoric stress
$[\Delta x]$	incremental relative displacements matrix
$[\Delta \dot{x}]$	incremental velocities matrix
$[\Delta \ddot{x}]$	incremental accelerations matrix

1. Introduction

1.1 Statement of Problem

In sites where concrete dams are either uneconomic or unsuitable, embankment dams consisting of compacted earthfill and/or rockfill are constructed. As the suitable sites for concrete dams are becoming scarce, embankment dams are becoming more common. Many of the dam sites are located in seismically active zones and are vulnerable to earthquake damage. For instance, many earthdams are located in parts of Eastern Canada with moderate seismicity. Therefore, their performance during a potential major earthquake should be evaluated. In addition, important earthdams need to be reassessed due to updated provisions for seismic loading.

The 2005 edition of the National Building Code of Canada (NBCC) contains significant changes in the requirements for earthquake loading. Some of these changes include updated hazard assessment in spectral format, increase in return period, explicit recognition of higher mode effects and placing dynamic analysis as the default method of seismic analysis (Heidebrecht, 2003). The impact of such changes on the seismic performance of embankment dams should be evaluated.

Embankment dams can be broadly classified into two groups in terms of performance under seismic conditions. The first are embankment dams that do not have seismically liquefiable soils in the dam body or its foundation and the second are those having

liquefiable soils. Past records show that the first group of dams can survive major earthquakes without sliding failure even though some cracks and deformation were formed. The second group may suffer catastrophic sliding failure even under moderate earthquakes. In this study, the seismic performance of Waba Dam belonging to the first group of embankment dams will be evaluated following new design ground motions (developed using similar methodology as in the NBCC but for events having probability of 1 in 10,000 years).

Waba Dam, which is under the jurisdiction of Ontario Power Generation (OPG), is located in Eastern Ontario and was constructed in 1976. The dam is a relatively low but wide embankment constructed over a deep deposit of sensitive clay. The seismic performance of the dam has been evaluated three times before: in 1985, 1988 and 1992. In this study, in addition to the use of new design ground motions, updated methods of seismic evaluation are implemented.

The seismic slope stability and earthquake-induced permanent deformation are two main aspects in the seismic assessment. Pseudo-static methods are the most common ways to evaluate these aspects. Before the near failure of the Lower San Fernando Dam (1971) the pseudo-static limit equilibrium was almost the only way to evaluate the seismic stability of earthdams. Afterwards, the inadequacy of pseudo-static method for dams consisting of liquefiable materials was recognized. But it is still commonly used for dams constructed of cohesive materials. However, if the pseudo-static method points to inadequacy of a dam, it is not well understood whether or not the dam will suffer seismic instability. Similarly, it is not clear how precise the pseudo-static methods are in the

prediction of permanent displacement. For instance, when a pseudo-static method of permanent deformation (Newmark-based method) suggests no displacement while, at the same time, pseudo-static stability analysis (Seed, 1977) indicates a large deformation, what meaning can be inferred. This research aims to clarify these ambiguities in the case of Waba Dam.

1.2 Objectives and Scope

The main goal of this study is to evaluate the seismic performance of Waba Dam as a part of a proposed research aiming to evaluate the seismic performance of typical embankment dams in Ontario.

The specific objectives of this study are as follows:

- Develop a rigorous method to incorporate the dynamic stress generated by an earthquake for assessing the pseudo-static method.
- Evaluate the seismic slope stability of Waba Dam under the design ground motions defined by a 10,000-year earthquake.
- Evaluate the earthquake-induced permanent deformation of Waba Dam and compare the results with those of the Newmark-based method.
- Conduct a sensitivity analysis to examine the effects of different inputs and to compare the results from the method developed in this research project with those from the Seed's pseudo-static method.

1.3 Outline of Chapters

The thesis reviews the literature in Chapter 2. Three aspects of geotechnical earthquake engineering discussed in this chapter are: (a) case studies on seismic performances of embankment dams, aiming to show the seismic behaviour of both groups of earthdams mentioned above; (b) pseudo-static, dynamic and elasto-plastic analysis methods for evaluation of seismic stability and permanent deformation of earthdams; (c) characteristics of earthquakes in Eastern Canada and new provisions of the 2005 edition of the NBCC for seismic design.

Chapter 3 commences with the description of three aspects of Waba Dam: geometry, soil profile and site-dependent ground motions. Then, a brief history of the previous studies on the dam is presented. The rest of this chapter is devoted to describing the pertinent methods of seismic analysis, including the new method developed for this study. In terms of the seismic stability, current pseudo-static methods and the dynamic nonlinear analysis utilizing the concept of instantaneous safety factor are described. Two dynamic methods are used for obtaining the instantaneous safety factor. The first one is a conventional stress-based method. The second method is newly developed here based on the dynamic forces generated during the earthquake. In terms of the permanent deformation, the conventional Newmark-based method and the elasto-plastic analysis using a 2-dimensional numerical scheme of Fast Lagrangian Analysis of Continua (FLAC) are described.

Chapter 4 presents the results of the stability analyses of Waba Dam using the prevailing method of pseudo-static analysis and dynamic non-linear analysis. The safety factors from the pseudo-static analysis were found to be significantly lower than those from the dynamic analyses. Hence, while the dam is considered problematic from the pseudo static analysis, it is acceptable based on the dynamic analysis. Review of the various factors for the discrepancy shows that the force-based dynamic analysis is more reliable for dynamic stability analysis. A parametric study is conducted to cover possible ranges of soil parameters and ground motions. According to the results, it can be concluded that Waba Dam will be stable under the 10,000-year earthquake. Part of the work in this chapter has been published in a technical conference (Law et al. 2005).

The results of the seismic permanent deformation are presented in Chapter 5. Both the Newmark-based and the elasto-plastic method by the FLAC are used in the analyses to assess the seismic deformations of the Waba Dam. The analyses show that there would be zero permanent deformation from the Newmark-based method but some definite permanent deformations from the FLAC. The main reasons for the difference in the results are discussed in the chapter. A parametric study is performed to cover possible ranges of soil parameters and ground motions. The results show that some permanent deformations are expected but they are within tolerable limits under the given earthquakes. Part of the work in this chapter has also been published in another conference (Law et al. 2005)

Chapter 6 discusses the problems of the pseudo-static method and the advantages of the proposed method for analyzing seismic performance of earthdams not vulnerable to liquefaction.

Chapter 7 presents the major conclusions from the study and presents a few suggestions on the future work.

2. Literature Review

The seismic behaviour of embankment dams has been the subject of many studies since the mid-1960s. Major advances in understanding the performance of soil under cyclic loading, dynamic response of earthdams to earthquake, and numerical methods to perform dynamic analysis for evaluating seismic stability have been achieved. Since the failure of the lower San Fernando Dam (Seed, 1982), which accelerated advancement in this subject, many studies have been performed on earthdams using dynamic analysis techniques.

In this chapter, case studies on the seismic performance of earthdams during earthquakes are reviewed, intending to determine the difference between the behaviour of embankment dams consisting of non-liquefiable soils and those dams consisting of soils, which are vulnerable to liquefaction. Afterwards, pseudo-static methods of slope stability analysis and deformation prediction are discussed. Nonlinear methods of dynamic analysis and elasto-plastic constitutive models are summarized in the next section. The interest of this study is methods dealing with total stress analysis appropriate for cohesive embankments; therefore, those approaches associated with porewater generation are not in the scope of this chapter. Finally, in the last section, characteristics of the earthquakes in eastern Canada are reviewed and changes in the new 2005 edition of the National Building Code are discussed.

2.1 Performance of Earthdams during Earthquakes

Since the late 1930s, the importance of seismic design of earthdams has been recognized. Construction of high dams started and consequences of any kind of failure became important. Some particular events increased interest and concern with earthquake resistance ability of earthdams. For example, the extensive failure of natural slopes in Anchorage, Seward, and Valdez as a result of the Alaskan earthquake of 1964 showed that slope failure is a major problem during earthquakes. Another event, which attracted great concern, was the occurrence of the major slide in the upstream slope of the lower San Fernando Dam caused by the San Fernando earthquake of February 9, 1971. At that time, 80,000 people were living downstream of the dam. The failure of the dam could have been a catastrophic event, since the water table level was 10.7-m below the crest and only 9.1-m of soils were taken out by the failure.

A small number of earthdams have failed during earthquakes though some have been significantly damaged. However, only a few major dams have been subjected to strong ground motions during earthquakes.

Diverse effects of earthquakes on earthdams have been recorded in literature. However, from the geotechnical perspective, two cases received considerable attention. The first one is the magnificent performance of the San Andreas Dam during the 1906 San Francisco earthquake of magnitude 8.3. Earthquake resistance considerations had not been considered in the design of this dam (consisting of two embankments). The second case is the failure of San Fernando Dams due to 1971 San Fernando earthquake of

magnitude 6.6. It is noticeable that these dams (Lower and Upper San Fernando Dams) were considered to have sufficient stability against an earthquake of such magnitude. (Seed, 1982)

The San Andreas Dam was severely affected by the 1906 earthquake. As described by Ambraseys (1960), the knoll between the two embankments passed through by the fault and spread. The fence and road over the dam were offset by about 2 meters. The dam was severely deformed for a distance of about 50 metres. Cracks parallel to the crest and in a transverse direction were developed near the central abutments. A longitudinal crack about 7.5-cm wide extended all along the centerline of the dam.

The San Andreas Dam was badly deformed, but survived the violent 1906 earthquake. After resurfacing the dam, which was the only required remediation, it was in constant service until the 1960s. Besides this dam, there were many other dams in the San Andreas Fault region that survived the 1906 earthquake without serious damage.

In spite of the satisfactory behaviour of the San Andreas Dam against earthquake-induced shaking, the San Fernando Dam showed a disappointing performance during the 1971 earthquake measured at 6.6 on the Richter scale. Seed (1979) describes the behaviour of the Lower San Fernando Dam as follows:

“As a result of the earthquake a major slide occurred in the upstream shell of the dam, taking out the crest and the upper 30-ft of the downstream slope. At the time of the earthquake, the water table level in the reservoir was about 35-ft below the

crest so that, after sliding only about 4 or 5 feet, badly cracked material remained above the water level. While warnings of the potentially dangerous effects of earthquakes had been voiced before the San Fernando event, the seismic stability of the dam had been evaluated only 5 years before the earthquake, and a reputable design agency, review agency, and consulting board had concluded that it was safe against any earthquake to which it might be subjected.”

The San Fernando event contradicted the general belief that earthdams are inherently stable against earthquake-induced shakings. Seed, after extensive investigation, indicated that the slide movements of the San Fernando Dam were initiated by liquefaction occurring in the sandy-fill material of the dam, due to porewater pressure build up during the earthquake that led to strength loss of the fill material. Since then, much research has focused on liquefaction and its effects on slope stability.

Regarding the events mentioned above, it might be concluded that earthdams are quite stable against earthquakes unless the dam materials are vulnerable to strength loss. However, seismic behaviour of earthdams is complicated, and many factors are involved in the response of a dam to ground motions. Factors such as the dam material properties, construction methods, design considerations and foundation properties on one hand, and the site condition and the earthquake characteristics at the site on the other, make it very difficult to predict the exact performance of a dam during an earthquake.

Although just a few main dams have been subjected to earthquakes, a reasonable number of case records are available. Reviewing them may help to make a rational conclusion of classification of earthdams in terms of their seismic behaviour. However, before studying

the summary of case records, possible damages and mechanisms of failure are overviewed.

2.1.1 Possible Reasons for Failure of Earthdams during Earthquakes

Possible factors causing failure of earthdams during earthquakes were summarized by Seed (1973) as follows:

1. Major fault movement in the foundation of the dam causing disruption in the dam.
2. Slope instability caused by ground motions.
3. Decrease of freeboard due to differential tectonic ground movements.
4. Loss of freeboard because of compaction or slope failure.
5. Weakness of foundation causing sliding of the dam.
6. Failure due to piping through cracks occurred by ground shakings.
7. Failure due to overtopping caused by seiches in the reservoir due to ground motions.
8. Failure due to overtopping because of rockfalls or slides into water reservoir
9. Failure due to overtopping caused by failure of the spillway.

Precautions in design to prevent the possibility of the above failures often involve the use of judgment and the incorporation of the following measures (Seed, 1973):

1. The avoidance of locating the dam over active faults.
2. The consideration of sufficient freeboard to allow some loss due to settlement or slope sliding.

3. The use of chimney drains near the central portion of the embankment and providence of ample drainage zones for possible flow of water through cracks
4. The use of wide core of materials, which are capable of recovering the possible developed cracks (“self-healing”).
5. The use of sufficient transitional layers for filter materials, which are not susceptible to cracking.
6. Stability analysis of upstream slopes and provision of overtopping caused by seiches or slope failure.

Cracks are the most common type of seismic damage. In China in-situ observations show that an earthquake with a magnitude of 7.0 will normally generate cracking in earthdams. The number of cracks decreases significantly with the increase of epicentral distance. Generally, longitudinal and transverse cracks do not threaten the stability and the functionality of earthdams. However, severe transverse cracks may lead to leakage, erosion and consequently, failure of an embankment dam. For example, 7 of 146 earthdams in Japan, which suffered from the Niigata earthquake of 1964 failed within a few hours to one day after the earthquake due to cracks that led to leakage, piping, erosion and the eventual collapse of the dams. (Japanese Society of Electricity and Civil Technology, 1980)

Settlement caused by an earthquake may be due to densification, tectonic subsidence and shear deformation. Shear deformation, which is the problem of this study, can occur in a form of uniform settlement over the entire length of a dam. This settlement in a

homogeneous dam may cause lack of free board and overtopping of the dam. Differential settlement in a core dam may be accompanied by wide cracks and failure of the dam. Failure of the Sheffield Dam is an example of this type. (Nunn, 1925)

Permanent deformation of a dam after an earthquake may be in a pattern of incipient sliding. Incipient sliding in a dam shows the occurrence of temporary instability of the slope of the dam. This deformation normally remains stable after the earthquake. Depending on the magnitude and duration of the earthquake, incipient slide can become a real slide.

Sliding in a dam can occur due to a seismic shock with a single well-defined slip surface. Or it can start at a small region and extend to an entire failure surface accompanied by large deformation, settlement and lateral displacement. Sliding may occur even a few hours after earthquakes.

Liquefaction has been associated with most of the failures of earthdams. Generation of porewater pressure may cause loose sands to liquefy and lose their strength. Besides failure of the lower San Fernando Dam, there are a number of well-documented cases associated with liquefaction, some of which are studied in the next section.

2.1.2 Case Studies: Seismic Performance of Earthdams

Ambraseys (1960) summarized the available information on the performance of embankments under strong earthquakes, and Seed et al. (1978) continued his initial review with the consideration of embankment and foundation materials, construction and

compaction procedures, the geometry of the embankments and their performance during earthquakes in terms of slope failures, cracks and major settlements. Most of the available data are come from five major earthquakes: The San Francisco (1906), the Kern County (1952) and the San Fernando (1971) earthquakes in California; the Fallon earthquake (1954) in Nevada; and the Ojika (1939) and Takachi-Okii (1967) earthquakes in Japan. There is also some information on the performance of individual embankments under other earthquakes. A brief summary of this study (Seed et al. 1978) is as follows:

Performance of Embankment Dams in the United States of America

At the time of the San Francisco earthquake in 1906, 33 earthdams, ranging in height from 4.6-m to 43-m were within 60-km of the ruptured fault. The earthquake magnitude was 8-1/4, duration of significant shaking about 60-sec, and the base acceleration experienced at these dams exceeded 0.6g. 31 of the 33 dams were composed mainly of clayey soils or mixed clayey sands and sandy clays, with only two consisting of somewhat sandy soils. Only five of the embankments were reported to have suffered any form of damage. However, 32 dams survived without significant damage. In this survey, there does not appear to be any correlation between the slopes of the embankments (slopes varied from 1 on 2 to 1 on 3) or their methods of compaction (most of the dams were compacted by moving livestock or teams and wagons) and the stability of the embankments during the earthquake. The only parameter found common and appeared to be a major factor contributing to the survival of the embankments was the nature of the clayey body and foundation of the embankments with low to medium plasticity. Two embankments, consisting of sandy soils under unsaturated conditions, also showed

adequate stability during the earthquakes. It seems that materials exhibiting little tendency to loose strength during cyclic load showed excellent performance.

In 1952, Kern County (California) with seven embankments, ranging in height between 6-m to 27-m located within 80 km of the epicenter, were shaken by an earthquake that registered 7.6 in magnitude. In this study, while no correlation between the performance of the dams during earthquake and their slopes and heights was found, it was noticed that compacted clayey embankments located at relatively short epicentral distances performed much better than sandy embankments at large distances from the fault.

About 30 fully operational embankment dams, ranging in height between 9-m to 58-m and located within 40-km of the epicenter, were subjected to the 1971 San Fernando earthquake with a magnitude of 6.6. Twenty five of these dams, which were rolled-compacted clayey dams, weren't damaged while two of the five hydraulic sandy fill dams, including Lower and Upper San Fernando Dams, suffered extensive damages and major slides. Liquefaction and loss of strength happened to be the main reasons for these failures.

Performance of Embankment Dams in Japan

A large number of embankment dams were shaken by the Ojika earthquake having a magnitude of 6.6 in 1939. A survey undertaken to investigate the damage showed that 74 embankments, ranging in height between 1.5-m and 18-m, were severely damaged and 12 of them failed totally. Detailed surveys of 58 damaged embankments and 12 that suffered

no significant damage show the occurrence of slope failure and cracks in 80% of the dams. No relationship between construction method and damage pattern was found. Most of the failures occurred up to 24 hours after the earthquake. While the majority of failures occurred in embankment dams consisting of sandy soil, no complete failure were reported in clayey earthdams even when they were located at short epicentral distances.

The Tokachi-Oki earthquake, with a magnitude of 7.8, damaged a large number of embankment dams with heights ranging between 4.6-m to 18-m. 93 damaged dams were investigated. These embankments consisted mostly of loose volcanic sand. Slope failures had been reported in embankments higher than 9-m. Slides in some clayey-cored dams occurred, but the failure did not occur in the cores.

Summary

Seed (1979), reporting on the above events and some other individual records, concluded that the slope failures in these damaged embankments during earthquakes were due to the phenomenon of “liquefaction” or strength loss associated with cyclic loading effects in loose saturated sands. As a matter of fact, dams which had suffered total failure or slope sliding in consequence of seismic motion had been constructed with “saturated sand shells” or “on saturated sand foundations.” The main cause of damage or failure for them is porewater pressure built-up leading eventually to loss of shear strength.

Dams constructed with clay soils on clay or rock foundation have survived even extremely strong earthquakes with magnitude 8.25 and shaking ranging from 0.35g to 0.8g with no noticeable damage.

In the following sections the pseudo-static methods for seismic stability and deformation assessment of cohesive embankment dams are broadly discussed. Although some of the methods might be applicable to earthdams made of any type of soils, the focus is on embankment dams consisting of materials which are not susceptible to liquefaction.

2.2 Pseudo-static Methods of Seismic Assessment of Earthdams

The main advantage of pseudo-static methods is their simplicity and relatively low computational cost. Two major aspects of embankment dams examined by pseudo-static methods are slope instability and earthquake-induced deformation. Pseudo-static methods may not be able to represent the exact deformation or failure pattern of embankments. However, they may provide valuable indexes to predict the potential damages.

Due to their simplicity now-a-days, pseudo-static methods are widely used by geotechnical engineering practitioners for seismic stability and deformation assessment. Seed's criteria (1979) have provided an easy guideline to assess the slope stability of embankments, which are not vulnerable to seismic liquefaction. Newmark's method (1965), which is in the connection with the pseudo-static analysis, provides the basis for more advanced methods to estimate the earthquake-induced deformation.

2.2.1 Pseudo-Static Analysis of Slope Stability

The pseudo-static method has been the standard method of seismic slope stability assessment of dams since the 1950s. In the analysis of slope stability, the effects of ground motion are represented by equivalent horizontal and vertical accelerations, which cause inertial forces proportional to the sliding mass. These forces F_h and F_v act through the centroid of the potential failure mass. In Equation 2.1 and 2.2, k_h and k_v are dimensionless horizontal and vertical seismic coefficients and W is the weight of potential failure mass. This method is attributed to Terzaghi (1950).

$$F_h = \frac{a_h \cdot W}{g} = k_h \cdot W \quad [2.1]$$

$$F_v = \frac{a_v \cdot W}{g} = k_v \cdot W \quad [2.2]$$

Considering the vertical component of induced-acceleration equal to zero, safety factors can be obtained from following equation (See Figure 2.1):

$$FS = \frac{S \cdot l}{d \cdot W + h \cdot F_h} \quad [2.3]$$

where S is shear strength of the soil and l is the arc length of the slip surface

Freeman (1932) has led to some approximate values for horizontal seismic coefficient k_h in soil structures: $k_h = 0.1$ for “severe” earthquakes $k_h = 0.25$ for “violent and destructive” earthquakes, and $k_h = 0.5$ for “catastrophic” earthquakes. Terzaghi (1950) suggested the same values for slope stability analysis of embankments. However, at that time, there was not possible to judge these values.

Figure 2.1 Forces acting on potential sliding mass in pseudo-static slope stability analysis

These forces increase the moment tending to rotate the slice of mass above the potential slip surface. Hence, the earthquake reduces the factor of safety of the slope with respect to k_h and k_v . In practice, the vertical seismic coefficient, k_v , is considered zero and k_h varies with the earthquake magnitude. These equations are based on the simplifying assumptions that horizontal acceleration, a_h , acts on the slope material in one direction constantly during the earthquake; therefore, the concept it conveys is inaccurate. Based on this method, a factor of safety less than one means slope failure, but in reality a slope may remain stable having a safety factor of less than one and may be unstable with a safety factor above one. In fact, the character of the slope-forming material, which is important, is ignored in this method. *“The most stable materials are clays with a low degree of sensitivity in a plastic state, dense sand either above or below the water table. The most sensitive materials are slightly cemented grain aggregates such as loose or partially submerged loose sand...”* (Terzaghi and Peck, 1948).

Selection of seismic coefficient

Seed (1982) reviewed different methods for determining the seismic coefficient. There were three approaches for choosing the seismic coefficients at that time: empirical values, maximum bedrock acceleration, and viscoelastic response analysis.

In the United States, empirical values of seismic coefficients were in the range of 0.05 to 0.15. The Japanese National Committee on Large Dams in 1957 established criteria for earthdams and depending on the location, type, and downstream hazard risk of the dam suggested a range from 0.12 to 0.25. These values of seismic coefficient may lead to a safe design but there was no explanation behind them.

The idea that the maximum acceleration of the bedrock is taken for obtaining the seismic coefficient has some limitations. First, apart from low stiff embankments in narrow canyons, there is considerable evidence from field tests that force-vibrated earthdams do not behave as a rigid body. Second, the maximum acceleration will only be developed for a short period of time, and its effects are not like those of the static force acting for an unlimited period of time. However, the upper limit of seismic coefficients seems to be a_{max}/g . For only small masses it can be assumed that all the earthquake-induced accelerations are in the same direction at a time during earthquake. Therefore, the use of $k_h = a_{max}/g$ for a small slope mass may be reasonable. Examples would include small rockfalls or shallow stability analyses (Day, 2002).

In the shear-beam method, to define the design seismic coefficient based on viscoelastic response analysis, the embankment is considered to be composed of a series of infinitely

thin horizontal slices. The slices are connected by linear shear springs and viscous damping devices. The responses to the base motion are determined at different levels.

Hatanaka (1955) showed that when the length of the dam is about four times of the height, the effect of end-restraint of the dam on the natural frequencies and amplitudes of vibration in the central region are negligible for all practical purposes.

Based on the viscoelastic response analysis, Ambrases (1960) proposed that the horizontal force acting on a horizontal slice can be determined as the square root of the sum of squares of the seismic coefficients for peak response for the first four modes. The seismic coefficient, at any depth, should be taken as the maximum value for any one of the modal distributions.

$$\bar{k}(y) = \left\{ \sum_{n=1}^{n=4} [k_n(y)]_{\max}^2 \right\}^{1/2} \quad [2.3]$$

$$k(y) = [k_n(y)]_{\max} \quad [2.4]$$

Application of this approach and other similar approaches lead to a depth-decreasing seismic coefficient with the maximum values at the crest of the embankment and zero at the bedrock. Ambraseys and Sarma (1967) developed curves for determination of the seismic coefficient by directly computing the inertial forces acting on the potential sliding mass.

Using shear-beam models (shear slice method), Seed and Martin (1966) and Dakoulas and Gazetas (1986), showed that the value of k_h for earthdams depends on the size of the

potential sliding mass. For instance for a deep-seated failure surface, k_h is considerably less than that for a shallow failure surface not extended far below the dam crest (see Table 2.1).

Table 2.1 Results for embankments of different heights and materials based on viscoelastic response. (After Seed and Martin, 1966)

Equivalent Seismic Force Series		$v_s = 300$ feet per second			$v_s = 1,000$ feet per second		
		Height of dam, in feet			Height of dam, in feet		
		100	300	600	100	300	600
Number of significant force cycles		10	5	3	15	12	7
Predominant frequency of force cycles		1.25	0.4	0.3	3.3	1.25	0.7
Equivalent maximum seismic coefficient operative over different portions of embankment	top quarter	0.35	0.20	0.10	0.40	0.36	0.24
	top half	0.30	0.15	0.07	0.35	0.28	0.16
	top three-quarters	0.22	0.10	0.04	0.30	0.22	0.11
	full height	0.16	0.08	0.03	0.25	0.16	0.08
Natural period of embankment, in seconds		0.87	2.61	5.22	0.26	0.78	1.57

This type of dynamic response analysis is used for determining the maximum inertial force likely to be mobilized in an earth-dam at any instant during an earthquake. The following limitations can be encountered for this method:

1. Selection of the maximum seismic coefficient for any mode at any depth leads to a conservative estimation.

2. Even when the safety factor drops below unity the embankment will not necessarily fail. It may just cause some deformation.
3. This type of methods suffers from the limitations of the shear-slice method that was mentioned before, which only takes into account horizontal shear stresses, whereas what have been indicated by Clough and Chopra (1966), other stress components may also cause permanent deformations within the earth-dam.
4. The assumption of dissipation of energy due to viscous damping is not correct because hysteric damping is more important than viscous damping.

Seed (1973) indicated that there were insufficient well-documented case histories available to present a basis for quantitative evaluation. Furthermore, the pseudo-static method was insufficient to explain the mechanism of a number of embankment failures such as the Sheffield Dam, Anchorage landslides, etc.

Comparing different approaches for determining the seismic coefficient, Seed (1973) showed that there was a variety of values for the seismic coefficient that made it difficult for one to take a proper decision. In addition to the variety of these coefficients, Seed (1973) clearly showed the uncertainty of the other aspects concerned with the pseudo-static method, which influenced the results of the safety factor, overshadowed the choice of a seismic coefficient. In Fig. 2.2 different values of the seismic coefficient are shown.

Seed (1979) proposed that deformations of earthdams constructed of soils that do not show more than 15% strength loss upon cyclic loading with crest acceleration less than

0.75g would be acceptable if the factor of safety due to pseudo-static analysis is not less than 1.15. According to his proposed method, k_h varies from 0.1 (for Magnitude=6.5) to 0.15 (for Magnitude=8.5) and k_v is assumed zero. Seed's criteria also allow the use of pseudo-static accelerations as small as 13 to 20% of the peak crest acceleration.

Figure 2.2 Comparison of the seismic coefficient resulting from different approaches.
(Seed, 1973)

Marcuson (1981) suggested that for dams with $k_h = 0.33a_{\max}/g$ to $0.50a_{\max}/g$. The possibility of amplification or deamplification of the seismic shaking due to the dam geometry shall be considered. For slopes of moderate size, Taniguchi et al. (1986) and Krinitzsky et al. (1993) suggested that a value of $k_h = 0.65a_{\max}/g$ to be used.

According to the criteria suggested by Hynes-Griffin and Franklin (1984), “dangerously large” deformation would not develop if the pseudo-static safety factor is greater than 1.0 using k_h equal to $0.5a_{max}/g$. Based on a study of the earthquake records from more than 350 accelerograms, using the seismic coefficient equal to $0.5a_{max}/g$, it was concluded that earthdams will not be subjected to “dangerously large” earthquake deformations (one meter settlement) when the factor of safety is not less than one.

Kramer (1996) indicated that the study on earthdams by Hynes-Griffin and Franklin (1984) would be appropriate for most slopes. It was also stated that there were no firm rules for the decision of the seismic coefficient, which should be based on the actual anticipated level of acceleration in the failure mass (including any amplification or deamplification effects).

2.2.2 Pseudo-Static Analysis of Earthquake-Induced Deformation

Sliding Block Method Proposed by Newmark (1965)

This method is principally based on the concept proposed by Newmark (1965). He proposed that the effects of earthquakes on embankments should be assessed in terms of the deformations produced rather than the safety factor. In this approach, a presumed time history of the acceleration acts on an embankment during an earthquake. Rigid plastic materials are assumed in Newmark’s approach. In this method, displacement of the critical sliding block occurs when the acceleration exceeds the yield acceleration, which is the minimum acceleration required for stability. Computation of the displacements as explained later yields permanent displacement of the sliding block.

In the Newmark's method it is assumed that the slope will deform only during those portions of the earthquake when the inertial forces acting on a potential failure mass cause the factor of safety to drop below 1.0. When this occurs, the failure mass will no longer be in static equilibrium and it will accelerate downslope. The longer that the slope is subjected to a safety factor below 1.0 and the less the safety factor, the greater is the slope deformation.

Obviously, this method is valid only for those cases in which the pseudo-static safety factor is less than 1.0. Fundamentally, the peak ground acceleration, $a_{\max}$, must be greater than the horizontal yield acceleration a_y so that this method can yield any result.

Figure 2.3 illustrates the basic premise of Newmark's method. Figure 2.3a shows the potential failure mass during an earthquake. N , in the figure, is the seismic acceleration, which varies during the earthquake. Forces acting on the failure mass are shown in Figure 2.3b. When the shear strength of the soil is totally mobilized and equilibrium is not obtained, the resultant of the forces cause the potential failure mass to accelerate the downslope. Using mechanics of a block on an inclined surface, the situation can be illustrated in Figure 2.3c.

Figure 2.3 Basics of the sliding block analysis. The horizontal acceleration required to cause the block to slide downhill is N_d , and N_u is the required acceleration for the uphill movement of the block. When the inclined plane is subjected to an acceleration pulse exceeding N_d , the block will move down relative to the plane. On the other hand when an acceleration pulse in the opposite direction greater than N_u is applied, the block will slide uphill. (Hynes-Griffin and Franklin, 1984)

The total displacement depends on the amount by which the yield acceleration is exceeded and the length of time during this period. In other words, the displacement caused by a single pulse is related to both the amplitude and frequency content of that pulse. Since, during an earthquake, acceleration can exceed the yield acceleration a number of times, it may produce a number of increments of displacement. Therefore, the

duration of the strong motion as well as its amplitude and frequency content will influence the total displacement. As shown in Figure 2.4, the relative velocity and displacement of the block can be obtained by integrating the relative acceleration once and twice, respectively.

Figure 2.4 Development of permanent deformation. (a) Earthquake acceleration highlighted when yield acceleration, a_y , is exceeded, (b) relative velocity for the darkened portions of the acceleration pulses, and (c) the corresponding downslope displacement in response to the velocity pulses. (after Wilson and Keefer 1985)

Permanent slope displacements are very sensitive to the yield acceleration (a_y). Small differences in yield acceleration may create large variations in estimated slope displacement. It is clear that if the earthquake-induced accelerations never exceed the

yield acceleration, no permanent displacement is produced. The relationship between slope displacement and a_y/a_{max} has been studied by several researchers such as Sarma (1975), Franklin and Chang (1977), Makdisi and Seed (1978) and Ambraseys and Menu (1988). One of the equations is as follows (Ambraseys and Menu 1988):

$$\log u = 0.90 + \log \left[\left(1 - \frac{a_y}{a_{max}} \right)^{2.53} \left(\frac{a_y}{a_{max}} \right)^{-1.09} \right] \quad \sigma_{\log u} = 0.30 \quad [2.5]$$

where u is the estimated downslope movement caused by the earthquake in centimetres, a_y is the yield acceleration, defined as the horizontal earthquake acceleration that results in a pseudo-static factor of safety equal to 1.0, and a_{max} is the peak ground acceleration of the design earthquake.

Using the probabilistic approach, Yegian *et al.* (1991) developed an expression for the median normalized displacement. Figure 2.5 shows probabilities of exceeding various displacements at different acceleration ratio. Using the same method, slope displacements are presented by Constantinou and Gazetas (1984) and Lin and Whitman (1986). Jibson (1994), in another approach, used arias intensity to predict sliding block displacements as follows:

$$\log u = 1.460 \log I_a - 6.642 a_y + 1.546 \quad \sigma_{\log u} = 0.409 \quad [2.6]$$

where u is in cm, I_a is in m/sec and a_y is in g.

In addition to the yield acceleration, distributions of earthquake acceleration have great effects on predicted permanent displacements. Even input motions with the same amplitudes, frequency contents and durations can generate extensively different predicted

slope displacement (Kramer, 1996). Originally, in Newmark's method, the potential failure mass is assumed to be rigid. Therefore, the input motion would be considered the ground motion at the level of slip surface. However, actual slopes may deform during earthquake shaking. Therefore, dynamic response of the slopes will depend on their stiffness and characteristics of the earthquake.

Figure 2.5 Variation of normalized permanent displacement with ratio of yield acceleration. (After Yegian et al., 1991)

As shown in Figure 2.6a, in slopes composed of stiff soils subjected to a low frequency shaking, the mobilized dynamic accelerations propagated in the potential slip mass can be nearly in phase. However, a slope consisting of soft soils under a high frequency motion may experience dynamic forces in opposite directions (Figure 2.6b). This reality makes Newmark's method unreliable, particularly for shaking associated with relatively thick failure masses.

Figure 2.6 Influence of frequency of ground motion in propagation of acceleration through potential failure mass. (a) Rigid soil subjected to low frequency waves. (b) Soft soil subjected to high frequency waves. (Kramer, 1996)

Makdisi and Seed's Method (1978)

Utilizing the dynamic finite element analysis, the effects of embankment response on the forces acting on the slip surface can be analyzed. In this method, which is developed by Chopra (1966), the horizontal components of the mobilized dynamic stress acting on a slip surface are integrated over the slip surface to generate a time-varying resultant force acting on the sliding mass. Dividing this force by the mass, the average acceleration is calculated. Since the response of the dam is accounted for in this method, the average acceleration would be more appropriate than the input ground motion to be used in Newmark's method. (Fig. 2.7)

Makdisi and Seed (1978) used the average acceleration mentioned previously to produce a simplified procedure to predict permanent deformation of earthdams after earthquakes. The concept of their procedure is based on Newmark's method but the dynamic response of embankments was calculated using a finite element analysis. Therefore, the

assumptions of a well defined slip surface and elastic behaviour at stress level below failure and perfectly plastic behaviour above yield of material are retained.

Figure 2.7 Finite element analysis of embankment dams to produce time-varying average acceleration used in Newmark's method (Kramer, 1996)

The following steps are involved in the Makdisi and Seed (1978) procedure:

1. A yield acceleration (a_y) related to the critical slip surface is determined.
2. Earthquake induced accelerations in the embankment (a_h) are calculated using dynamic response analysis.
3. Permanent displacement is estimated when a_h exceeds a_y by double integration procedure.

The yield acceleration is calculated by stability analysis using conventional pseudo-static limit equilibrium method. It is obtained from the seismic horizontal coefficient that results a minimum safety factor equal to one. This yield acceleration and the corresponding slip surface are used in Newmark-based methods.

The strength of the material, in the conventional slope stability analysis, is considered equal to either the peak strength from an undrained test, or the shear stress that causes a certain axial strain (10%). However, under the cyclic loading conditions, the behaviour of

a material would be different from its behaviour under the static conditions. The soil element in an embankment dam stressed to the static yield level or even higher will develop some permanent deformation before reaching total failure. Therefore, in this method the yield strength is defined as the maximum stress level below which the material shows almost an elastic behaviour under cyclic loading conditions. Figure 2.8 shows this concept. The material in the figure has a yield strength equal to 90% of its static undrained strength. When the material is subjected to 100 cycles of stress equal to 80% of its strength, very little permanent displacement is measured. However, when the material is subjected to 10 cycles equal to 95% of the soil strength it causes significant deformation.

Based on the experimental data and the shear strength calculated from earthquake response analyses, Makdisi and Seed (1978) suggest that in most cases of clayey materials, cyclic yield strength would be 80% or more of the static undrained strength. However, for compacted cohesive soils they find it reasonable to assume very little reduction as a result of “strong earthquake loading”. For the compacted clayey material “static failure strength” values usually range between 3%-10% depending on whether the material was compacted on the dry or wet of optimum.

After determining the yield acceleration, the time history of average acceleration induced by earthquake must be determined. The procedure proposed by Chopra (1966) is adopted in this method.

Figure 2.8 The concept of dynamic yield strength (Makdisi and Seed, 1978)

The forces acting along the slip surface are calculated at each time-step. As shown in Figure 2.9, these forces are due to horizontal components of normal and shear stresses of the finite elements along the boundary of the potential sliding mass. In the two-dimensional finite element model, equivalent linear properties (strain-dependent Rayleigh damping and shear modulus) were employed.

Makdisi and Seed (1978) showed the variation of the maximum average acceleration and the ratio of maximum average acceleration with the depth of the sliding surface. According to the results of analyses, the deeper the sliding surface, the lower are the average acceleration and the acceleration ratio. Figure 2.10 shows the variation of the maximum acceleration with the depth of the sliding mass on the basis of both the shear-beam (Seed and Martin, 1966) and finite element method (Makdisi and Seed, 1978).

However, they found similarity in the frequency component of the maximum average acceleration for different depths in their results.

Figure 2.9 Average acceleration calculation, using finite element analysis (After Chopra, 1966)

Figure 2.10 Variation of the maximum average acceleration with the depth of potential slip surface, based on both the shear-beam method and the finite element method (Makdisi and Seed, 1978).

After the determination of the yield acceleration and the time history of the average acceleration, the permanent deformation of the potential sliding mass can be computed. Depending on soil type the yield acceleration can be considered constant during an earthquake or can be rate-dependent such as strain-hardening or strain-softening (Lemos and Coelho, 1991 and Tika-Vassilikos *et al.*, 1993).

Makdisi and Seed (1978) assume that, the movement of the sliding mass will be in a horizontal direction. They explained that in many cases this mode of displacement, which causes longitudinal cracks, occurred. Makdisi and Seed (1977) concluded that the factor of direction of motion had little effect on the predicted displacement.

Figure 2.11 Variation of acceleration ratio with normalized permanent deformation in the horizontal direction. Summary of all data is on the left, and average normalized displacement is on the right. T_0 is the first natural period of the embankment (Makdisi and Seed, 1978).

Figure 2.11a shows a summary of the results of the normalized permanent deformation for different earthquake magnitudes. The average curves for the normalized displacement are shown in Figure 2.11b. Using this graph, the permanent displacement may be predicted for any embankment height and earthquake magnitude.

It is obvious that by assuming a horizontal sliding plane there would be no vertical component of displacement. Therefore, vertical settlement cannot be estimated by using this approach. Furthermore, the yield acceleration is calculated on the basis of shearing of the sliding wedge over the slip surface. Therefore, when the average acceleration exceeds the yield acceleration, the sliding block will slide along the slip surface, not along any arbitrary plane. In fact, the yield acceleration causing sliding on a horizontal plane could be much different from the yield acceleration of causing sliding along a circular slip surface.

In practice the procedure proposed by Makdisi and Seed (1978) for determining the average acceleration of the sliding mass may not be followed. Sometimes the average acceleration is computed by averaging accelerations at nodal points of the finite element mesh within the sliding block. This procedure generally yields a more conservative estimate for the average acceleration (Finn, 1988).

Finn (1994) indicated that displacements estimated from the acceleration data using Makdisi and Seed's approach do not signify the deformation patterns of earthdams under earthquake motions. Therefore, the estimated displacements should not be used to meet displacement criteria. However, they may present a reasonable index of potential

deformations. If the result points to large displacements, one should estimate large deformations using a suitable nonlinear finite element analysis.

Seed's Alternative Approach

An alternative method was developed by Seed (1966 and 1979) involving the following steps:

1. Computation of dynamic stresses acting on soil elements within an embankment both before and during an earthquake, and converting them to equivalent uniform stress cycles.
2. In the laboratory, applying stress cycles to typical soil samples in similar consolidated states of corresponding elements in the embankment and observing the resulting deformations.
3. Estimating the potential deformations of the slope from the measured set of strains of the soil elements in the lab.

This approach thus deals with the time history of the stresses mobilized in the field or slopes during an earthquake, the soil performance under simulated earthquake shaking, and the estimation of embankment displacements rather than a safety factor.

Apart from equivalent stress cycles, no further parameter is involved in this method. However clearly, this method requires a lot of laboratory tests and input works which make it difficult to conduct a parametric study that is a common practice in geotechnical researches.

Lee's Permanent Deformation Method

The concept of the method suggested by Lee (1974) is to represent the effects of the earthquake on an embankment dam by stiffness softening of the soil. This method contends that since there is no change in the load condition before and after ground motion, the only consequence of the seismic disturbance would be softening of the soil leading to permanent deformation.

In the numerical analysis, it is assumed that there is no change in the load matrix right before and after the earthquake. Hence, the matrix of load remains constant during analysis, and the only change takes place in the shear modulus. The value of Poisson's ratio is also constant in the analysis. Earthquake-induced deformation is computed as a result of change in stiffness.

There are two simple steps in the analysis: (a) the displacements prior to and after an earthquake are computed and (b) the permanent deformation is obtained by the difference of these two displacements. Figure 2.12 shows the basic concept of Lee's method for an axially loaded sample. ε_i and ε_p are determined by a simple shear test or triaxial test. E_{ip} is the equivalent Young's modulus.

These methods evaluate the overall seismically-induced permanent deformation. A major disadvantage of this method is the determination of E_{ip} , which is obtained directly by laboratory tests. Therefore, for practical purposes, this method involves a lot of laboratory tests and labour cost.

Figure 2.12 Lee's method for estimating permanent deformation (Lee, 1974)

Pseudo-Static Method Proposed by Serff et al.

Another semi-empirical method for the prediction of the permanent deformation is the method suggested by Serff et al. (1976). The concept of this method is to represent the effect of earthquake as equivalent nodal point forces. As shown in Figure 2.13, it is assumed that the stress-strain relationship is represented by a nonlinear hyperbolic curve based on consolidated undrained tests. This relationship will be dependent on the confining pressure, σ_{3i} , in the pre-earthquake state.

The first step of this method is to determine pre- earthquake state ($\sigma_{di}, \epsilon_{i-D}$) using an incremental finite element approach based on the drained stress-strain relationship. Then, the corresponding strain level at the same stress is identified from the undrained curve

$(\sigma_{di}, \varepsilon_{i-UD})$. In the second step, incremental dynamic deviatoric stress, $\Delta\sigma_d$, corresponding to the pseudo-static nodal force is applied to calculate the potential strain, ε_p , from the undrained curve. Eventually earthquake-induced deformation is calculated using a nonlinear finite element method.

Figure 2.13 True pseudo-static approach (Serff *et al.*, 1976)

There are some other pseudo-static methods such as residual strain method suggested by Stamatopolous *et al.* (1991), which are more appropriate for saturated sands. The next section of this chapter is devoted to equivalent linear, full nonlinear, and elasto-plastic dynamic methods.

2.3 Dynamic Analysis of Soil Structures

In the previous discussion, pseudo-static methods were reviewed. By the advancement of computational facilities, utilization of more sophisticated approaches has become possible. There are several 2-D and 3-D finite element programs to model the earthquake shaking and the nonlinearity of the soil. These programs may be based on equivalent linear, fully nonlinear, or elasto-plastic dynamic methods.

Dynamic analysis of soil structures such as earthdams and soil-structure interaction systems has been the major challenge in soil dynamics. A critical review of methods of computing seismic permanent deformation was conducted by Marcusan et al. (1992) and Finn (1994). The methods range from the relatively simple equivalent linear model to complex elasto-plastic models. This review presents the general concepts of methods used in current practices.

2.3.1 Equivalent Linear Method

The basic assumption of equivalent linear method (Seed, 1973 and Lismar *et al.*, 1975) is that the dynamic response of the soil is approximated by a damped elastic model, which is characterized by a hysteretic loop. The properties of that model should be chosen properly by an iterative process. For instance, the stress-strain properties of the soil may be defined by the Poisson ratio, strain-dependent shear modulus and equivalent viscous damping ratio.

Figure 2.14 Hysteretic loop in equivalent linear model (after Seed, 1973)

Figure 2.14 shows a typical hysteretic in equivalent linear method. Generally, two major characteristics of the strain-stress relationship are the secant shear modulus, G_{sec} , and the damping ratio, ξ . The inclination of the hysteretic loop depends on the stiffness of the soil. However, since the tangent shear modulus, G_{tan} , varies throughout a cycle of loading, the average value over the entire loop is approximated by the secant shear modulus given by Equation 2.7:

$$G_{sec} = \frac{\tau_c}{\gamma_c} \quad [2.7]$$

where τ_c and γ_c are the shear stress and shear strain amplitude, respectively. The width of the hysteretic loop is related to the dissipation of energy, which is corresponding to the area of the loop and can be defined by the damping ratio (Kolsky, 1963) for small damping

$$\xi \approx \frac{W_D}{4\pi W_S} = \frac{1}{2\pi} \frac{A_{loop}}{G_{sec} \gamma_c^2} \quad [2.8]$$

where W_D and W_S are dissipated energy and maximum stored energy, and A_{loop} the area of the hysteretic loop. G_{sec} and ξ are called equivalent linear material properties. The maximum shear modulus, G_{max} , which is related to the shear wave velocity, v_{max} , and the material density is given by Equation 2.9:

$$G_{max} = \rho \cdot v_{max}^2 \quad [2.9]$$

There are several empirical relationships developed for determining modulus reduction (G_{sec}/G_{max}) and damping ratio (ξ) based on standard in-situ and laboratory test indexes.

Figure 2.15 Strain-dependency of damping and shear modulus in equivalent linear method (after Makdisi and Seed, 1978)

In the equivalent linear analysis for the first iteration, γ_{ave} is calculated using initial estimates of shear modulus (G) and damping (ξ). In the next iteration, G and ξ will be

adjusted to be compatible with the calculated γ_{ave} . Analysis is repeated until strain-compatible G and ξ are obtained. Figure 2.15 shows shear modulus and damping curves used by Makdisi and Seed (1979).

A major application of the equivalent linear method is the seismic-induced displacement analysis of earth and rockfill dams. However, since the method is on the basis of elastic relationship and total stresses, the earthquake-induced permanent deformation and porewater generation cannot be calculated directly by the analysis. Because of this reason, the computed stresses and accelerations may be overestimated in this method. However, when fill material is not vulnerable to the strength loss caused by porewater pressure build-up during shaking, the deformation may be estimated from acceleration data using Newmark-based methods such as Makdisi and Seed's (1978).

The equivalent linear model is based on the strain-dependent modulus and the viscous damping ratio. Practicing engineers are familiar with these properties, which are measured by standard laboratory procedures. In addition, a large amount of data is available in the literature on these properties. Computer programs based on this method are efficient. Therefore, this method has been used both for general designs and parametric studies. (Marcuson *et al.*, 1992)

2.3.1 Cyclic Nonlinear Methods

Soil, on the basis of observed behaviour in cyclic loading tests, is a nonlinear and hysteretic material. In the nonlinear approach, the nonlinear hysteretic stress-strain

behaviour of the soil under cyclic loading and unloading is obtained by modeling the actual stress-strain path using tangent modulus analytically.

Dynamic models are mostly obtained by modifying static plasticity rules to include cyclic loading effects. There are two approaches for this modification. In the direct approach, the basic set of formulation is replaced by one that is capable of modeling the effects of dynamic loading. In the second approach, the main structure is retained with a new parameter added to take into account the water pressure response. An example for the first approach is the model suggested by Carter *et al.* (1982), and for the second approach is the critical state model with additional kinematical hardening surface proposed by Mroz *et al.* (1978).

Streeter *et al.* (1974) used a cyclic nonlinear model for the first time and solved the equations of motions using the method of characteristics. Their method, applicable to the ground level and based on total stresses, was incorporated into a computer program named CHARSOIL.

The porewater pressure generation model for cyclic loading was developed by Martin *et al.* (1975) and the effective stress method incorporated into the computer program (DESRA-2) was presented by Finn *et al.* (1976) and Lee and Finn (1978). The stress-strain behaviour of the soil in shear was represented by a hyperbolic skeleton curve. The unloading and the reloading paths were defined by the Masing criterion (1926).

Martin and Seed (1978) and Ishihara and Towhata (1982) developed 1-D nonlinear effective stress methods. In 1986, 2-D version of the computer program DESRA called TARA-3 was released (Finn *et al.*, 1986). This program was the subject of detailed verification studies by Finn (1988) using data from simulated earthquake tests on centrifuged models.

The nonlinear method of analysis has been validated on element tests with homogeneous stress and strain fields, as well as on inhomogeneous stress fields associated with soil structure interaction problems. Equivalent linear and direct nonlinear methods demand the same length of computational time.

Basic Formulation of Nonlinear Analysis

The main procedure of this type of analysis usually includes: (1) the modeling of soil behaviour by a backbone strain-stress curve, (2) developing rules that govern the unloading-reloading behaviour and the stiffness degradation, (3) deriving suitable numerical processes and computer implementation. A simple nonlinear model has a simple backbone curve with a few basic rules. Many additional rules are required to present effects such as densification, pore pressure generation, or other properties.

As an illustration, a very simple example described by Finn (1988) incorporated a hyperbolic backbone curve represented in the following.

An incremental approach is adopted using tangent shear modulus (G_t) and bulk modulus (B_t). The incremental dynamic forces $[\Delta P]$ are given by the following equation of motion

$$[M] \cdot [\Delta \ddot{x}] + [C] \cdot [\Delta \dot{x}] + [K] \cdot [\Delta x] = [\Delta P] \quad [2.10]$$

where $[M]$, $[C]$ and $[K]$ are the mass, damping and stiffness matrices, respectively. $[\Delta x]$, $[\Delta \dot{x}]$ and $[\Delta \ddot{x}]$ are incremental relative displacements, velocities and accelerations. Very small amounts (up to 1%) of Rayleigh-type viscous damping may be used to control any high frequency oscillations that may arise from numerical integration. It should be considered that in nonlinear method hysteretic damping is automatically included when hysteretic stress-strain loops are carried out during the analysis.

In the nonlinear analysis the stiffness matrix varies by time increment, and is a function of current G_t during analysis. The elasticity matrix is defined by:

$$[D] = B_t \cdot [Q_1] + G_t \cdot [Q_2] \quad [2.11]$$

where $[Q_1]$ and $[Q_2]$ are constant matrices for the plane stress conditions usually considered.

Soil behaviour in shear is assumed hysteretic and nonlinear following the Masing criterion (1926) during unloading and reloading. Hysteretic damping is provided by Masing behaviour.

Figure 2.16 Hyperbolic backbone curve (Kramer, 1966)

The relationship between the shear stress τ and shear strain γ for either the drained or undrained loading conditions is assumed hyperbolic as shown in Figure 2.16 and defined by:

$$\tau = f(\gamma) = \frac{G_{\max} \cdot \gamma}{(1 + (G_{\max} / \tau_{\max}) \cdot |\gamma|)} \quad [2.12]$$

where $G_{\max}$ and $\tau_{\max}$ are the maximum shear modulus and appropriate shear strength, respectively. The unloading-reloading curve based on Masing behaviour from a point (γ_r, τ_r) is given by

$$\frac{\tau - \tau_r}{2} = f\left(\frac{\gamma - \gamma_r}{2}\right) \quad [2.13]$$

Dynamic analysis in each element of the model starts from the static stress-strain conditions. This leads to accumulating permanent deformations in the direction of the smallest residual resistance to deformation. Practically, initial static strains can be ignored and analysis starts from the origin of the stress strain curve in all elements of the model. The ability to represent permanent deformation is one of the main advantages of the cyclic nonlinear method over the equivalent linear method

The cyclic nonlinear model employs low-strain modulus and shear strength as the model parameters. These parameters may be determined from correlations with in-situ index tests such as the SPT or CPT, or they can be determined using standard laboratory or other in-situ test procedures. Permanent deformation is calculated directly, and the porewater pressure generation can be modeled permitting an effective stress analysis. All the nonlinear methods give similar results for total stress analysis. (Marcuson *et al.*, 1992)

2.3.2. Elasto-plastic Constitutive Models

The most general and precise methods for representing soil behaviour are on the basis of advanced constitutive models. By using basic principles of mechanics, these models are capable of simulating initial stress conditions, rotating principle stress axes, different stress paths, monotonic or cyclic loading, low or high strain rates, and drained or undrained conditions. Such models can be described by three major aspects (Kramer, 1996):

- 1) a yield surface to define the limiting stress level that material exhibits elastic behaviour,
- 2) a hardening law to define changes in yield surface in plastic state, and
- 3) a flow rule to define the correlation between increments of stress and plastic strain.

The classic Cam-Clay and modified Cam-Clay models (Roscoe and Burland, 1968) were the first constitutive models. Development of multi-yield surface models (Mroz, 1967 and Prevost, 1978) and bounding surface models (Dafalias and Popov, 1979, Mroz *et al.*, 1979 and Bardet, 1986) have resulted in improvement of shear strain prediction. Detailed critical assessments of advanced constitutive models have been done by a number of researchers, including Finn (1988), Wood (1991) and Marcuson *et al.* (1992). Fewer dynamic methods have been established for clayey structures. Constitutive models were generally developed to study liquefaction and deformation of sandy soils. In this section the basic concept of the multi-yield surface model (Prevost, 1978) is presented.

Nested Yield Surface Method

The Multi-yield Surface model was proposed by Prevost (1977) for describing the undrained stress-strain behaviour of clay under monotonic and cyclic loading conditions. In such models, yielding is assumed to be controlled by deviatoric stresses and occurs at constant plastic volume. A Von-Mises yield criterion is used to define the yield surface. This yield surface is represented by a circle in the deviatoric plane, which is normal to the hydrostatic line in stress-space. A kinematical hardening rule and the associative flow rule (Zaretsky-Lombardo 1983) are used to compute plastic stresses and strains.

Figure 2.17 shows the main elements of Nested Yield Surface models. This model is defined in stress-space by a set of nested yield surfaces $f_0, f_1, \dots, f_p$ with respective sizes. Each surface defines a region of constant shear modulus. The larger the surface section, the lower is the shear modulus. All yield surfaces may be transformed by the stress point

over the stress path without changing in form or in orientation. The surfaces one after another touch and push each other but do not intersect. As it is shown in the Figure 2.17, all the yield surfaces will be tangent to each other at the point M, when the stress point reaches the yield surface f_m .

Elasto-plastic constitutive models are relatively complex and incorporated some parameters not very familiar to practicing engineers. The model considers soil as a two-phase material using coupled equations for soil and water. Complex constitutive models demand heavy computational time. Validation studies show that when the loading path is similar to the calibrating stress paths, the predictions of response are good. However, by deviating from the calibrating stress path the results become less reliable. All the constitutive models are path-dependent, a problem in the current stage of developing constitutive models for soils. No one constitutive model is recommended for all cases. For any case the validation test should be undertaken to find out the reliability of the chosen model. (Marcuson *et al.*, 1992)

A hierarchy of methods is available for modeling earthdams to assess their stability against possible earthquakes. The models range widely in complexity and accuracy. While a model is appropriate for one problem it may not be appropriate for another. It depends on the case, material properties, site condition and seismic hazard of the site to choose the appropriate method.

Figure 2.17 Elements of Nested Yield Surface on the basis of Von-Mises failure criteria (Provost, 1977). S_{ij} is the deviatoric stress tensor.

2.4 Characteristics of Earthquakes in Eastern Canada

In the past, the general belief has been that major earthquakes in the United States and Canada are restricted to the west coast. Consequently the California-type of earthquakes has been used to represent the standard for designs. However, studies since the 1980s have shown that both characteristics and mechanisms of the earthquakes in Eastern North America (ENA) are different from those in California. In fact due to lower attenuation, earthquakes in eastern Canada will affect a much larger area than earthquakes in California with similar magnitudes. The density of population in Eastern Canada is also higher than California.

In California most large earthquakes occur on faults and are directly due to motions between lithosphere plates. The earthquakes along the eastern continental margin are related to the Mesozoic rift falls. Somewhat like the eastern Asian earthquakes, they are not directly related to the plate motions and caused by secondary stresses set up at plate boundary (Scholz, 1977).

First, eastern ground motions are prosperous in high frequency components. In spite of western earthquakes that exhibit a rapid decrease of ground motion amplitudes at frequencies above 15-Hz; amplitudes for ENA events do not reduce rapidly until frequencies of about 40-Hz. The high frequency cut-off in ENA earthquakes may be attributed to both their competent crustal conditions and their high-frequency component.

Figure 2.18 Seismicity map of Canada to 2001. (Adams and Atkinson, 2003)

Some major results obtained by studies on ENA earthquakes by Atkinson (1986; 1988) are described in the following:

Another major discrepancy between ENA earthquakes and their western corresponding events is that they decay more slowly with distance than earthquakes with the same magnitude in California. This may be attributed to the relatively stable and incorporated crust in ENA. Figure 2.18 shows the seismicity of Canada to 2001. The broken lines show the limits of the west, the east and the stable regions throughout the country.

Instead of the scaling standard spectral shapes of California, a new technique called stochastic model, has been utilized for estimating ground motions in ENA. This theoretical model, which has been proved to better match actual ground motions, differentiates between eastern and western ground motions by differences both in the value of high frequency cut-off, maximum frequency, and inelastic attenuation. Using this model, reliable approximations of response spectra can be obtained for moderate earthquake. These estimated spectra can be used as input for seismic hazard analyses to determine response spectra for specific rock sites.

However, the preliminary hazard maps of Eastern North America provided protection against major earthquakes in only a few locations where large earthquakes had occurred historically. They did not provide protection to all locations (Basham, 1988).

2.4.1 Changes in Provisions of 2005 NBCC

Unceasing advancement in the knowledge of seismic hazard and its geographical allocation throughout Canada makes it necessary to revise seismic requirements from time to time. Since the first edition of the National Building Code of Canada (NBCC) in 1941, seismic knowledge has moved from a general quantitative sense of seismicity to the expression of hazard using probabilistic peak ground acceleration and velocity. The main things that led to the 2005 edition of the NBCC were generally studying and learning from the damage due to the world's major earthquakes, research conducted in Canada and around the world, and comparison of our provisions with those in the codes of other countries. For instance, the dynamic amplification of ground motions due to soft

soil observed in the 1989 Loma Prieta earthquake improved code provisions for considering those effects of soft-soil sites (Borcherdt 1994).

Comparing the former editions of NBCC (1990-1995), the 2005 edition contains very important changes in the seismic hazard provisions. Change in probability of exceedance, hazard in spectral format, period-dependent site factors, recognition of higher mode effects, dynamic analysis requirement and other changes are described by Heidebrecht (2003). Some of the changes that may affect design of earthdams are as follows:

- In the previous editions of the NBCC (1985, 1990, and 1995), seismic hazard was described in terms of peak ground acceleration and velocity at a probability of 10% in 50 years. Then, these parameters were amplified to obtain the period-dependent variation of seismic forces. In the 2005 NBCC, seismic hazard is in the form of uniform hazard spectra (UHS) at particular geographical locations shown in Figure 2.19, which provides a much better period-dependent specification of earthquake effects on structures.
- It has been common in the past decades to represent seismic hazard information at a probability of 10% in 50 years. However, to provide a more uniform margin of failure, it is necessary to specify seismic hazard at a lower probability of exceedance, which is much closer to the probability of failure or collapse. Regarding this fact, the seismic code provisions in the 2005 NBCC are on the basis of hazard values at a probability of 2% in 50 years.
- It has been known that amplification of seismic motions from rock to soft soil sites can be significant. As noted before, research by Borcherdt (1994) and others

has made quantification of main seismic response effects possible, including classification of soil profiles using quantitative properties (shear wave velocity, standard penetration resistance, or undrained shear strength), period dependence, and effects of the intensity of underlying rock motions. These provisions provided the basis of the site factors in the 2005 NBCC, as discussed by Finn and Wightman (2003).

Figure 2.19 Typical contour map of seismic hazard used as the basis for the 2005 NBCC.
(Adams and Atkinson, 2003)

For Eastern Canada, because of the rareness of observations in the magnitude–distance range of engineering interest earthquakes, ground motion relations are a major source of uncertainty in seismic hazard estimation. Therefore, eastern ground motion relations may

change considerably as new events are recorded. The Atkinson-Boore (1995) set of uncertainty relationships was adopted to cover this problem that represents the available ground motion relationships. There is, however, substantial argument in this field. The Atkinson-Boore set of relationships was originally developed to fit observational data on hard-rock sites and so needs adjustment to represent the ground motions on the firm ground. A typical contour map of distribution of hazard in Canada is shown in Figure 2.19. (Adams and Atkinson, 2003)

2.4 Conclusions

Based on the available records of the seismic performance of earthdams, it is reasonable to categorize earthdams into two major groups. The first group, which is the majority, consists of embankment dams built of materials not vulnerable to liquefaction due to seismic shaking. The dams in this group have shown adequate strength even under major earthquakes. Although they may suffer some minor damages, they survive. The second group consists of earthdams built of liquefiable materials such as saturated loose sands. Even moderate earthquakes may cause significant damages or failures to these dams.

Conventional pseudo-static methods are available to assess seismic adequacy of earthdams particularly those in the first group. Parameters used in pseudo-static methods are generally easy to obtain. Among the stability analysis methods, Seed's criteria are the most common way to assess the seismic slope stability of embankments. In case of necessity, Newmark's method can be used to estimate the earthquake-induced displacement. However, the accuracy of pseudo-static methods is not clear. They usually

provide some conservative indexes rather than clear answers. Therefore, when seismic adequacy of an earthdam is rejected by applying such methods, the dam may not be inadequate in reality. In such a case, the use of more advanced methods is required.

A hierarchy of nonlinear dynamic methods is available for modeling of earthdams subjected to earthquake accelerations. The models range widely in complexity and accuracy. Most of them involve many parameters, which may not be easy to obtain. The most important aspect of such models is their strong path-dependency that makes them sensitive to input parameters particularly the stress-strain relationship. While a model is appropriate for one problem, it may not be appropriate for another. The main concern of most of these models is porewater generation that makes them inappropriate for undrained analysis. It seems that a simple elasto-plastic approach is adequate in order to model such a case.

In chapter 3, a method is proposed to assess seismic performance of Waba Dam belonging to the first group. The dam consists of clay material, not vulnerable to liquefaction. Regular pseudo-static methods indicate inadequacy of the seismic stability of the dam. Therefore, further assessment is required. A new technique is proposed and applied to calculate the instantaneous safety factor to clarify the stability of the dam during earthquakes. The permanent deformation is also calculated directly by taking into account the elasto-plastic behaviour of the material. Far-field and near-field ground motions are applied to the dam, which are prepared on the similar basis used in the NBCC, but for 10,000 year return period.

3. Waba Dam and Methods of Seismic Analysis

3.1 Introduction

As it was reviewed in Section 2.1, in terms of seismic performance, embankment dams can be largely classified into two groups. The first are embankment dams that do not have seismically liquefiable soils in the dam body or its foundation, and the second are those having liquefiable soils. The first group of dams can survive major earthquakes without sliding failure even though some fractures and deformation were formed. The second group may suffer catastrophic sliding failure even under moderate earthquakes. This study is a part of a research on the seismic performance of the first group of embankment dams under new design ground motions developed using similar methodology as in the NBCC, but for events having probability of 1 in 10,000 years.

The specific objective of this research is to evaluate the seismic performance of the Waba Dam under the jurisdiction of the Ontario Power Generation (OPG). Waba Dam is a relatively low but wide embankment dam consisting of cohesive materials overlying a deep deposit of soft, firm to stiff clay located in Eastern Ontario. These soils are not vulnerable to liquefaction and therefore landslide failure of the dam is not likely during earthquakes.

Ground motions of major earthquakes are considered in this study. These ground motions are developed using seismic hazard evaluation methodology similar to that used for the

National Building Code of Canada (NBCC). While the NBCC considers seismic events of 2,500-year return period, the events studied in this paper correspond to a return period of 10,000 years. Both far-field and near-field ground motions, which are based on the design uniform hazard spectrum at the site, are applied in the analyses.

To assess the slope stability of the dam, the safety factor computed from the traditional pseudo-static method is compared to the instantaneous safety factor, F_t , during earthquake. This F_t can be determined based on the conventional method involving the shear stresses mobilized along the failure plane (Chopra 1966). It can also be evaluated on the basis of dynamic forces due both to gravitational acceleration and to the seismic vibration, which is a new approach proposed in this study. The mobilized shear stresses or dynamic forces can be computed from numerical analyses.

To assess the permanent deformation of the Waba Dam, Newmark-based methods (such as by Makdisi and Seed, 1978) are compared to a nonlinear dynamic analysis based on the 2-dimensional Fast Lagrangian Analysis of Continua (FLAC). Both types of methods consider the same bilinear stress-strain relationship for modeling the soil behavior.

3.2 Waba Dam

Constructed in 1976, Waba Dam is a relatively low embankment dam founded on a deep sensitive marine deposit in Eastern Ontario, where there is moderate seismicity. Large berms were built to provide the needed stability and redistribution of the stresses caused

by the embankment. Figure 3.1 shows an aerial view of the dam taken a few years after its construction. The wide berm at the upstream side is visible in the picture.

Figure 3.1 Waba Dam in Eastern Ontario (Law, 1987)

3.2.1 Geometry and Soil Profile

The most critical section of the dam simplified for this study is shown in Figure 3.2. The foundation is divided into an upper and lower layer of clay. The dam is built over 66.4-m of soft to stiff sensitive marine clay followed by bedrock. The design reservoir level is at +99.06-m. The detailed dimensions of the section of the dam are shown in Appendix A.

Two different cases for soil properties are used in this study. The difference of the two cases lies in the strength assumed for the dam material. The profiles of the undrained shear strength for both cases of the dam material and the marine subsoil are shown in Figure 3.5. Table 3.1 is a summary of soil parameters used in this study. In both cases the undrained shear strength for the dam foundation is taken from piezocone tests (Konrad and Law, 1987) shown in Figure 3.3 (Cragg 1988 and Wu 1992). In the first case, the strength of the dam material is taken as equal to the minimum strength of the subsoil. In the second case, however, 80% of the measured undrained shear strength of the dam material is taken. The focus of the study is on Case2. Case 1 is not used in analyses unless it is specified.

Figure 3.2 Waba Dam cross section. (Dimensions in meter)

Figure 3.3 Undrained shear strength profile from piezocone tests (Cragg 1988 and Wu 1992)

Figure 3.4 Results of the field vane shear test from the phase II soil investigation.

(Ontario Power Generation)

Figure 3.5 Undrained shear strength profile: Case 1 and Case 2. In all the analyses performed for this study Case 2 is the default soil profile. In this study, Case 1 is not used in the analysis unless it is specified.

Table 3.1 Material properties of the Waba Dam used in the analyses

Elevation (m)	Material	Case1 S_u (kPa)	γ_{sat} (kg/m ³)	ν	E_{Case1} (kPa)	Case2 S_u (kPa)	E_{Case2} (kPa)
102.6	Dam	23.54	1900	0.45	2.12E+04	96.00	8.64E+04
91.4	material	23.54			2.12E+04	96.00	8.64E+04
91.4	Marine	23.54	1600		2.12E+04	33.33	3.00E+04
77.4	Subsoil	43.14			3.88E+04	33.33	3.00E+04
77.4		43.14			3.88E+04	43.14	3.88E+04
54.9		74.64			6.72E+04	74.64	6.72E+04
54.9		92.91	8.36E+04		92.91	8.36E+04	
31.9		131.09	1800		1.18E+05	131.09	1.18E+05
31.9		147.77			1.33E+05	147.77	1.33E+05
25		153.23			1.38E+05	153.23	1.38E+05

An elasto-plastic (bilinear) stress-strain relationship is assumed to approximate the non-linear behaviour of the dam material and the subsoil. For the elastic region, a constant undrained modulus E_u is assumed and for the plastic region, a zero value is taken. Since the undrained modulus starts at a high initial value and decreases steadily with increasing shear strain in the actual behaviour, the constant E_u is taken as somewhere between the initial and the failure values. After surveying clay behaviour in the region (e.g., Cragg 1988 and Bozozuk 1972), the undrained modulus, E_u , is expressed in Equation 3.1:

$$E_u = 900.S_u \quad [3.1]$$

where S_u is the undrained strength of the soils. The Poisson's ratio is taken as 0.45.

By applying the elasto-plastic stress-strain relationship, a certain degree of damping is encountered when the soil is sheared into the plastic state. The damping at this state is a function of the magnitude of the shear strain. However, in the elastic state the model assumes no damping. For the present study, 2% mass proportional Rayleigh damping is applied to reduce the oscillations after the earthquake stops. This value is taken from the results of resonant column tests for the low shear strain (Figure 3.6). In Case 1, however, no damping is assumed. Undrained analysis is considered here since the duration of earthquake is too short for the clay material to undergo any consolidation.

Figure 3.6 Resonant column tests (Cragg, 1988)

3.2.2 Previous Studies

Wai (1985) performed a dynamic analysis on Waba Dam in a research project within Ontario Hydro (now Ontario Power Generation). The finite element computer program used in that analysis was FLUSH developed at the University of California, Berkeley (Lysmer, 1975). The study indicates no formation of any continuous slip surface. However, some overstressed isolated zones were observed in brief moments during strong ground motions. Because there was no appropriate earthquake record from Eastern Canada at that time, an acceleration time history from a Californian earthquake was used in the study. The time history was scaled to a maximum acceleration of $0.1g$. In addition to the unrealistic input motion, insufficient stress-strain behaviour of the soil and other dynamic properties increase the uncertainties of the study.

A further study, which was a significant improvement in the study of Waba Dam, was conducted by Wai in 1988. In that study, a number of far-field and near-field computer-generated input motions were used, which were specified for the dam site (Atkinson, 1986). Material properties were also on the basis of piezocone tests (Cragg, 1988), resonant column tests and shear wave velocity measurements. In the study, far-field ground motions were found to produce higher amplification in the response of the dam than near-field input motions. The study suggested that the Waba Dam would survive seismic events of 1000-year return period.

A supplementary study on the seismic response of the Waba Dam was performed in 1992 (Wu, 1992). A number of input motions were used in the study for annual probability of

0.0021 in accordance with the 1985 NBCC. One of the new aspects of this study was the consideration of seismic response of the dam to the vertical component of ground motions. The important aspect of this study was the scaling of the maximum acceleration to 0.2g compared with 0.1g in the previous studies. In the study, an elasto-plastic model was developed based on laboratory tests and applied to the dam to calculate permanent settlement. The maximum permanent settlement encountered by the study was 15.4 and 34.5-cm for the near-field and the far-field input motion respectively.

In the current study, a new method, utilized for the seismic stability analysis of cohesive embankment dams, is used to examine the slope stability of Waba Dam during seismic events of 10,000-year return period. In addition to the stability assessment, the earthquake-induced permanent deformation of the dam is estimated by the elasto-plastic analysis. Comparative analyses are also conducted to show possible weaknesses of conventional pseudo-static methods and superiority of nonlinear approaches.

3.3 Ground Motions

Ground motions generated with the method used by the NBCC-2005 are used in the study. The time histories generated by Atkinson (2004) are compatible with the Uniform Hazard Spectrum (UHS) for the dam site. The UHS is prepared based on a probabilistic hazard analysis performed for Ontario Power Generation (OPG) dam sites in 1998 (Atkinson, 1988). The UHS values for the 5% damped horizontal-component of pseudo-acceleration on hard rock are provided by OPG. The annual probability considered for this study is 0.0001. Some analyses are also performed by the application of time

histories with the annual probability of 0.0004, which is recommended by the 2005 edition of NBCC. Figure 3.7 shows the Uniform Hazard Spectrum of the site for the annual probability of 0.0001.

Figure 3.7 Uniform Hazard Spectrum and spectra of simulated records. $p=0.0001$ p.a.
horizontal component on rock

The simulation methodology is as described by Atkinson and Boore (1995). Since motions are generated for horizontal-component on hard-rock sites, soil response should be considered in the analysis. Therefore, input motions are applied to the bedrock and not to the ground level. The generated motions typically include nearby moderate

earthquakes (Radius =10-km & Magnitude = 5.5) and large earthquakes further away (Radius =40-km & Magnitude = 7). (Atkinson, 2004)

One far-field and one near-field ground motion among 20 generated records are chosen. Examination of these records leads to the choice of Record M70R40#8 for the far-field and Record M55R10#1 for the near-field on the basis that they have the largest peak acceleration and velocity. Table 3.2 and Figures 3.8 and 3.9 summarize the characteristics of these ground motions used as input horizontal motion at the bedrock. In addition to these records, a near-field ground motion with a return period of 2500 years is also applied to the analysis. This additional record shown in Figure 3.10 is also based on the UHS of the dam site and is chosen among 10 records based on the same criterion. However, the focus of the analysis will be on the records with 10,000-year return period. All the records are available in the Appendix B.

Table 3.2 Characteristics of Input ground motions

Trial	p	Dist. (km)	M	PGA g	PGV cm/s	PGD cm	Duration sec	f_{max} Hz
M70R40#8	0.0001	40	7	0.325	9.87	4.022	15.2	25
M55R10#1	0.0001	10	5.5	0.678	12.53	0.855	1.7	25
M55R16#7	0.0004	16	5.5	0.378	7.86	0.719	2.6	25

Note: Dist. = hypocentral distance; M = moment magnitude; PGA = peak ground acceleration; PGV = peak ground velocity; PGD = peak ground displacement; f_{max} = maximum frequency component of accelerations; p = annual probability

The focus of this study is on the ground motions with 10,000-year return period. M55R16#7 is not used unless notified otherwise.

Figure 3.8 Far-field record, M70R40#8 with 10,000-year return period.

Figure 3.9 Near-field record, M55R10#1 with 10,000-year return period.

Figure 3.10 Near-field record, M55R16#7 with 2,500-year return period. In this study, this record is not used in the analysis unless specified.

3.4 Slope Stability Analysis

To examine the slope stability of the dam, the results of conventional pseudo-static methods (such as Seed's method, 1979) are compared to the instantaneous safety factor (F_t) during earthquake. This F_t is evaluated by using the conventional stress-based method (Chopra 1966). It can also be determined a new approach based on dynamic forces

3.4.1 Pseudo-Static Limit Equilibrium Analysis

In the pseudo-static analysis of slope stability, the effects of ground motion are represented by two equivalent horizontal and vertical forces, which are proportional to the sliding mass. These forces F_h and F_v act through the centroid of the potential failure mass. In Equation 3.2 and 3.3, k_h and k_v are dimensionless horizontal and vertical seismic coefficient and W is the weight of the potential failure mass.

$$F_h = \frac{a_h \cdot W}{g} = k_h \cdot W \quad [3.2]$$

$$F_v = \frac{a_v \cdot W}{g} = k_v \cdot W \quad [3.3]$$

As mentioned in Chapter 2, Seed (1979) suggests that deformations of earthdams constructed of soils that do not show more than 15% strength loss upon cyclic loading with crest acceleration less than $0.75g$ would be acceptable if the factor of safety due to pseudo-static analysis is not less than 1.15. According to this method k_h varies from 0.1 (for Magnitude=6.5) to 0.15 (for Magnitude=8.5). Seed's criteria also allow the use of

pseudo-static accelerations as small as 13 to 20% of the peak crest acceleration. In Seed's method, k_v is assumed equal to zero.

According to the criteria suggested by Hynes-Griffin and Franklin (1984), "dangerously large" deformation would not develop if the pseudo-static safety factor is greater than 1.0 using k_h equal to $0.5a_{max}/g$.

For the pseudo-static analysis, the conventional limit equilibrium method by Morgenstern and Price (1965) is used. SLOPE/W is the commercial computer program used in this study program to calculate the safety factor. Seed's criteria (1979) and Hynes-Griffin and Franklin method (1984), which are currently used, are also used to perform pseudo-static analysis.

3.4.2 Instantaneous Safety Factor during Earthquake

The instantaneous safety factor (F_t) is defined as the safety factor for the slope at a particular instant during an earthquake. It takes into account the effects of soil strength, gravitational acceleration of the moving soil mass and the acceleration due to the earthquake motions. Therefore, F_t depends on location and time during the earthquake. The lowest instantaneous safety factor at a given instant is denoted by F_{tmin} , while the overall minimum instantaneous safety factor throughout the entire earthquake is denoted by F_{min} . F_{min} can be obtained in two main steps. First, F_{tmin} is obtained by examining a sufficiently large number of potential slip surfaces at any given instant. Second, the first step is repeated for every instant throughout the earthquake. F_{tmin} for each instant are

compared and the lowest of all F_{imin} is taken as F_{min} and the potential slip surface associated with this F_{min} is taken as the critical slip surface. This approach can be applied to different zones of the dam to ensure the unambiguous location of the critical slip surface.

Circular slip surfaces are assumed for evaluating F_t . Each circular slip surface is assumed to be composed of a chain of linear segments connected at the boundary of adjoining elements (Figure 3.11).

Figure 3.11 An arbitrary slip surface in a general form.

The centre of the slip surface can be located within a grid system and the slip surfaces are bounded by two parallel lines as shown in Figure 3.11 (Geo-Slope 2002). Different zones of the dam may need a different set of grid and parallel lines.

During the earthquake excitation, the safety factor and the location of the critical slip surface may vary. Under ideal conditions it is possible to determine the most critical slip surface and F_{min} for each time-step of the input ground motions. In practice, it is very time consuming to calculate F_{min} for each time-step. It is a common practice to determine F_{min} for time increments larger than the time-steps of the input motions (e.g., once in each 10 or 20 time steps).

Two-dimensional, plane-strain, fully dynamic analyses are carried out using FLAC, a commercial computer program for static and dynamic analyses capable of treating general non-linear behaviour including elasto-plastic relationship. The calculation is based on an explicit finite difference scheme to solve the complete equations of motion, using lumped grid point masses derived from the density of surrounding zones (Itasca 2002). In this study, elasto-plastic behaviour is considered for the soil model. The failure envelope for this model corresponds to a Mohr-Coulomb criterion (shear yield function) with tension cut-off (tensile yield function).

Stress-based method

The evaluation of F_t has been obtained using two different methods. The first is the conventional (stress-based) method that compares the resisting moment to the motivating moment via the shear stress induced along the potential slip surface and the strength mobilized there (Chopra 1966). Mathematically F_t can be expressed by Equation 3.4 to 3.9.

$$F_t = \frac{M_R}{M_M} \quad [3.4]$$

$$M_R = R \cdot r \quad [3.5]$$

$$M_M = S \cdot r \quad [3.6]$$

$$R = \int_A^B \tau_{av} \cdot dL \cong \sum_{i=1}^n \tau_{av_i} \cdot \Delta L_i \quad [3.7]$$

$$S = \int_A^B \tau_n \cdot dL \cong \sum_{i=1}^n \tau_{n_i} \cdot \Delta L_i \quad [3.8]$$

where, M_M and M_R are motivating and resisting moments, respectively, S and R the actuating and resisting forces, r the radius of the slip circle. n the number of segments, τ_{av} the shear strength, τ_n the mobilized shear stress and ΔL_i the length of the segment (Figure 5).

The shear stress τ_n developed in the i^{th} segment can be computed as Equation 3.9:

$$\tau_n = \tau_{xy} \cdot \cos(2\theta) - \frac{(\sigma_x - \sigma_y)}{2} \cdot \sin(2\theta) \quad [3.9]$$

where, θ is the angle between the i^{th} segment and the horizontal direction. σ_x , σ_y and τ_{xy} are normal and shear stresses acting through the x- and y-coordinate directions. These stresses are determined for any time-step during the dynamic analysis.

Force-based method

In this study, a new approach (force-based) is developed for calculating F_t , which takes into account both the static and the dynamic forces that exist in the potential sliding mass

during the earthquake. The motivating moment now consists of the net effect of the static (gravitational acceleration) forces and the dynamic force seismically induced in the potential sliding mass (Equation 3.10)

$$M_M = \sum \vec{F}_j \times \vec{H}_j \quad [3.10]$$

where, $\vec{F}_j$ is the force vector, including both dynamic and static forces acting on the j^{th} node located in the potential sliding mass, and $\vec{H}_j$ is the vector of distance between the center of the slip circle and the j^{th} node. In this case, the resisting moment remains the same as in the stress-based method. (See Figure 3.12)

Figure 3.12 Nodal forces acting through potential failure mass.

All these shear stresses, forces, accelerations and instantaneous safety factor are calculated through routines developed for this study to post process the results from the finite difference program.

A few computer functions (written by a built-in computer language available in FLAC called FISH) are written to facilitate implementation of these methods for the Waba Dam: A FISH function to provide required information for different sets of slip surfaces, a FISH function to save the output stresses required for the calculation of the instantaneous safety factor, a function written by QBASIC to calculate the minimum safety factor at each time step and an EXCEL macro to read and summarized all the output files and draw required graphs.

3.5 Permanent Deformation

In this study, the permanent displacement of the Waba Dam caused by earthquakes is first determined using the Newmark-based methods, as they are the most commonly used in current practice. This is followed by nonlinear dynamic analyses incorporating the selected stress-strain relationship for modeling the soil under study.

3.5.1 Newmark-Based Method by GEO-SLOPE

This group of methods is principally based on the concept proposed by Newmark (1965). When a dam is subjected to an earthquake, its stability will largely depend on the shear strength of the soil and the average coefficient of horizontal acceleration k_h . The coefficient of yield acceleration (k_y) is defined as a coefficient that when $k_h=k_y$, the safety factor of the potential sliding mass located above the most critical slip surface will be equal to unity. When the dam is subjected to a pulse of acceleration of k_h that exceeds k_y , the potential sliding mass will move. The displacement of the sliding mass during the

earthquake can then be determined by double integration of the acceleration in excess of k_h .

The acceleration time-history for the above calculation is based on the response of the dam during the earthquake. This response can be obtained by means of a dynamic finite element analysis in which the horizontal components of dynamic stresses acting on a potential failure surface are integrated along the slip surface. Then, the resultant horizontal force is divided by the potential sliding mass to produce the average coefficient of horizontal acceleration k_h .

In this group of methods, one of the more commonly used is by Makdisi and Seed (1978), in which k_y is calculated on the basis of a dynamic yield strength equal to 80% of the undrained shear strength of the soil.

The value of k_h , in practice, may be computed once for every 10 or 20 time-steps during an earthquake. This time history of k_h can be used to determine a time-varying safety factor and the accumulated permanent displacement. The sum of the durations with the safety factor less than unity corresponds to the total duration for the permanent movement to occur.

In this study, some components of GEO-SLOPE, commercially available software, are used to perform the above procedure. An equivalent linear method (Seed, 1973) is used to perform the dynamic stress analysis of the dam. Then the time-varying stress results are

post-processed to determine the time-varying safety factor and permanent displacement of the critical sliding mass of the dam.

3.5.2 Nonlinear Analysis by FLAC

The dynamic analysis consists of two-dimensional, plane-strain, fully dynamic analysis with FLAC. The calculation is based on an explicit finite difference scheme to solve the full equations of motion, using lumped grid point masses derived from the density of surrounding zones (Itasca 2002). In the finite difference method, every derivative in the set of governing equations is replaced directly by an algebraic expression written in terms of the field variables (e.g., stress or displacement) at discrete points in space. These variables are undefined within elements. This procedure first invokes the equations of motion to derive new velocities and displacements from stresses and forces. Then, strain rates are derived from velocities, and new stresses from strain rates. One time-step is taken for every cycle around the loop (Figure 3.13)

In initial static analysis, boundaries of the model are fixed. However, in the dynamic analysis, free-field boundaries are invoked along the left and right boundaries to absorb energy. The seismic input is represented by plane waves propagating upward through the underlying material. The boundary conditions at the sides of the model must account for the free-field motion, which would exist in the absence of the structure. In some cases, elementary lateral boundaries may be sufficient. For example, if only a shear wave was applied on the horizontal boundary, AC, shown in Figure 3.14, it would be possible to fix the boundary along AB and CD in the vertical direction only. These boundaries should be

placed at sufficient distances to minimize wave reflections and achieve free-field conditions. For soils with high material damping, this condition can be obtained with a relatively small distance (Seed et al. 1975). However, when the material damping is low, the required distance may lead to an impractical model. In this study a free-field boundary condition is applied along the left and right of the model.

Figure 3.13 Basic explicit calculation of motion in FLAC (Itasca, 2002)

The approach used in FLAC is in fact similar in concept to other nonlinear dynamic analyses proposed by a number of researchers, such as Prevost (1981), Pande and Zeinkiewicz (1982), and comprehensive critical reviews have been given by Finn (1988) and Marcuson et al. (1992). This type of analyses accommodates the development of plastic zones that produce permanent deformations in different parts of the dam body and foundation. In the present analysis of Waba Dam, the selected elasto-plastic behaviour for modeling the clayey soil behaviour is characterized by recoverable deformation during the loading and the unloading phases that are below the shear strength, S_u , of the

soil and by irrecoverable (permanent) deformation during the plastic straining. Fig. 3.15 shows schematically the typical stress-strain path and development of permanent deformation during an earthquake excitation in the analysis.

Figure 3.14 Model of seismic analysis for surface mesh (Itasca, 2002)

The model has a built-in constitutive function corresponding to a Mohr-Coulomb failure criterion (shear yield function) with tension cut-off (tensile yield function). The shear flow rule is non-associated and the tensile flow rule is associated (Zaretaskiy-Lombardo 1983). The tensile strength is considered to be zero. The friction angle and dilation angle are also assumed to be zero. The model is simple to apply and is realistic for describing the soil behaviour.

Figure 3.19 Typical stress-strain path for an elasto-plastic soil during an earthquake excitation with development of permanent deformation

Unlike the Newmark-based methods where the permanent deformation is calculated from stress, the present analysis calculates the permanent deformation on the basis of strain.

The topic of the next two chapters is the stability and deformation of Waba Dam during earthquakes. Modeling of the dam, stages of analysis and stability analysis results are presented in Chapter 4. Parametric studies covering possible ranges of input parameters are conducted to examine the sensitivity of the dam to the input parameters. Permanent deformation of the dam is the topic of Chapter 5. The results of conventional methods are compared to the results of somewhat new methods in both chapters.

4. Seismic Stability Assessment of Waba Dam

4.1 Introduction

In this chapter, pseudo-static and dynamic analysis results of the seismic stability assessment of Waba Dam are presented. The results of conventional pseudo-static methods are compared with the instantaneous safety factor of dynamic analysis. Both cases of soil strength profiles are used in the analyses. However, the focus of the study is on Case 2 while it represents more realistic characteristics of the dam fill material than the other case. Parametric studies are also performed to study the sensitivity of the dam response to the input parameters in the dynamic analysis.

In the pseudo-static limit equilibrium analysis, two conventional methods are applied. Slice-based limit equilibrium utilized by SLOPE/W provides the basis for the analysis. Safety factor in static mode is also calculated to verify the accuracy of programs developed for the computation of the instantaneous safety factor.

In the dynamic analysis section, stages of the finite different analysis of the dam are reviewed. In the initial static state, the static safety factor of the dam using different techniques is determined. Then, the responses of the dam to the far-field and the near-field input motions are presented, and finally, the instantaneous safety factor based on both the conventional stress-based method and the proposed force-based method is derived.

The comparison study aims to show the shortcomings of the pseudo-static approach. Furthermore, observations of different responses of the dam during dynamic analysis yield a better understanding of the seismic behaviour of embankments. Generation of irrevocable plastic zones and counter acceleration vectors are some of these responses to be discussed in this section. A new equivalent seismic coefficient is defined here for comparison with the seismic coefficient used in the pseudo-static analysis.

The purpose of the parametric studies is to investigate the sensitivity of the input parameters to increase the reliability of the study. Parameters such as input acceleration, undrained shear strength and shear modulus have been considered.

4.2 Pseudo-static Limit Equilibrium Analysis

The simplified section of the Waba dam is modeled with SLOPE/W, one of the components of GEO-SLOPE Office. Among the available limit equilibrium methods, the Morgenstern-Price method is chosen. The static safety factor is also calculated based on the finite element stresses computed using QUAKE/W, another component of GEO-SLOPE. This finite-element-stress based safety factor is comparable with that of FLAC in initial static mode. The analysis is performed for both cases of soil profiles.

Pseudo-static analysis is carried out using SLOPE/W with the horizontal seismic coefficients (k_h) based on Seed's criteria (1979) and Hynes-Griffin and Franklin's method (1984).

Waba Dam as modeled by SLOPE/W is shown in Figure 4.1. In all the following figures of the dam, the upstream side is on the right hand side. The horizontal radius lines, which indicate the depth of slip surfaces, are placed from reservoir level to the lower foundation in three-meter interval. It should also be considered that the radius lines exist at the boundary of soil layers. All the slip surfaces are considered circular. Limited multi-linear slip surfaces are also examined in the study.

Figure 4.1 Waba Dam Modeled by SLOPE/W. (Dimensions in meter)

4.2.1 Safety Factor in Static Mode

Figures 4.2 to 4.4 show the results of the static limit equilibrium analysis. In these figures the contour lines show the variation of the safety factor of the slip surfaces in the dam. The centers of the slip surfaces are located within the contour lines. For Case 2, which is our main study, the critical slip surface with a safety factor of 2.2, is located on the downstream berm (Figure 4.2). For Case 1, in which the shear strength of the dam material is assumed equal to the minimum subsoil strength, the critical safety factor corresponds to the sliding of the crest of the dam based on Morgenstern-Price method. The resulting safety factor is equal to 1.58 (Figure 4.4). The dam has also been analysed using QUAKE/W, which is a finite element program capable of performing dynamic analysis, and the computed stresses can be exported to SLOPE/W. The safety factor

resulted from the finite element stresses is 2.08 (Figure 4.3) for Case 2 and 1.52 for Case 1. The finite element stress method is just applied to provide a safety factor comparable to that of FLAC in static mode, in order to verify the subroutines written for FLAC.

Figure 4.2 Static analysis by the Morgenstern-Price method performed by the SLOPE/W. The minimum safety factor is 2.23.

Figure 4.3 Static analysis by the finite element stress method performed by the SLOPE/W. The minimum safety factor is 2.08.

Figure 4.4: Static analysis by the Morgenstern-Price method. The minimum safety factor is 1.58. (Case 1)

4.2.2 Pseudo-Static Safety Factor

Seed (1979) proposed that deformations of an earth-dam constructed of soils that do not show more than 15% strength loss upon cyclic loading and crest acceleration less than 0.75g would be acceptable if the safety factor using the pseudo static analysis is not less than 1.15. It is also suggested through this study that $k_h = 0.1$ (for $M=6.5$) to $k_h=0.15$ (for $M=8.5$) and k_v is assumed equal to zero. Seed's criteria also allow the use of pseudo-static accelerations as small as 13 to 20% of the peak crest acceleration.

The results of pseudo-static analysis on Waba Dam for Case 2 are shown in Figures 4.5 to 4.8. As shown in Figure 4.5, for k_h as low as 0.11 corresponding to earthquake magnitude equal to 7 ($M = 7$), the safety factor is significantly less than 1.15. The minimum pseudo-static safety factor, FS_{min} , is associated with deep-seated failures. According to Seed's criteria, these results imply that there would be unacceptably large deformation. In Figure 4.6, which is for k_h as low as 0.1, FS_{min} is again less than 1.15.

Figure 4.5 Seed's pseudo-static method analysis using $k_v = 0$ $k_h = 0.11$ ($M=7$)

Figure 4.6 Seed's pseudo-static method analysis using $k_h = 0.1$ ($M=6.5$)

If one uses the criterion suggested by Hynes-Griffin and Franklin (1984), “dangerously large” deformation might be concluded under the given ground motions. According to this criterion earthdams with pseudo-static safety factors greater than 1.0 using

$k_h=0.5a_{max}/g$ would not develop “dangerously large” deformations. Figures 4.7 and 4.8 show the results of pseudo-static analysis for the far-field and the near-field acceleration records. The calculated FS_{min} is significantly less than 1.0 for both records. Such results imply that “dangerously large deformations” will take place based on the Hynes-Griffin and Franklin criteria.

Figure 4.7 The Hynes-Griffin and Franklin method. $k_h = 0.1935$ (50% of a_{max} of the near-field ground motion acceleration record, 0.387g)

Figure 4.8 The Hynes-Griffin and Franklin method. $k_h = 0.1625$ (50% of a_{max} of the far-field ground motion acceleration record)

Variation of the minimum safety factor with the seismic coefficient for Case 2 of soil material is shown in Figure 4.9. Each curve in the figure is related to a specific downstream zone of the dam. The berm, the entire dam and the crest are referred to Zone 1, Zone 2 and Zone 3, respectively. Figure 4.10 shows the different zones of the dam. Similarly, variation of the minimum safety factor with the seismic coefficient for similar upstream zones is shown in Figure 4.11. The upstream berm, the entire dam and the crest, are referred to Zones 4, 5 and 6, respectively.

Figure 4.9 Variation of the minimum safety factor with k_h for the different zones of the downstream part of the dam

Figure 4.10 Different potential sliding zones of Waba dam

Figure 4.11 Variation of the minimum safety factor with k_h for the different zones of the upstream part of the dam

Figures 4.5 to 4.8 show that the deeper and wider the slip surface, the lower is the safety factor. This is the problem of the pseudo-static analysis for soft soils when the strength-increase with depth is small. As the theoretical slip surface goes deeper, the effect of k_h will outweigh the effect of strength increase, leading to smaller factor of safety. If this exercise is carried to the extreme, one would obtain a safety factor of less than 1.0 for a level ground. Hence, the pseudo-static analysis produces unrealistic results for this type of earth structures as noted earlier by Wu et al. (1991).

Owing to the fact that the water of the reservoir is acting on the dam body as a stabilizing force, the safety factor of the upstream part of the dam is slightly larger than the downstream part. Figure 4.12 shows a comparison of the minimum safety factor in both sides of the dam. It seems that the occurrence of a sliding failure in the downstream part of the dam is more probable than in the upstream part.

Figure 4.12 Variation of the minimum safety factor with k_h for the downstream and upstream part of the dam

Figures 4.13 shows the results of the pseudo-static analyses on Waba Dam for Case 1 soil profile. In the same manner as in Case 2, the results of the pseudo-static analyses for this case indicate inadequacy of the dam. For k_h as low as 0.1, the critical slip surface is quite deep-seated.

Figure 4.13 Seed's pseudo-static analysis using $k_h=0.1$ ($M=6.5$). (Case 1)

4.3 Nonlinear Dynamic Analysis

4.3.1 Finite Difference Modeling of Waba Dam

A very fine mesh with optimum element size is developed to facilitate wave transmission. Kuhlemeyer and Lysmer (1973) show that for accurate representation of wave transmission through a model, the spatial element size, l , must be smaller than one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave - i.e., where λ is the wavelength associated with the highest frequency component, f_{max} , that contains appreciable energy. Using Equations 4.1 to 4.3, λ can be expressed.

$$l < \lambda/10 \quad [4.1]$$

$$\lambda = V_s / f_{max} \quad [4.2]$$

$$V_s = \sqrt{G/\rho} \quad [4.3]$$

where V_s is the speed of the s -wave propagation through a medium, and G and ρ are shear modulus and mass density, respectively. The shear modulus (G) and the bulk modulus (K) can be determined via the Young's modulus (E) and the Poisson's ratio (ν) ratio using Equations 4.4 and 4.5.

$$G = \frac{E}{2(1+\nu)} \quad [4.4]$$

$$K = \frac{E}{3(1-2\nu)} \quad [4.5]$$

The highest frequency component of the input wave that contains appreciable energy is 25Hz. Therefore, considering the undrained shear strength profile and Equation 3.1, one can choose a vertical element size smoothly varying from 55 to 20-cm starting from bedrock to the crest.

The horizontal lengths of the elements are optimized along the width of the dam. A concentration of a number of elements in the downstream side of the dam makes it possible to obtain fairly precise results for calculating the instantaneous safety factor. A total of 200 columns and 275 rows of elements were used to model the dam and the subsoil resulting in a total node number of 55,000 (Figure 4.14).

Figure 4.14 Finite difference mesh for the Waba Dam model

The stress-strain relationship of the soil is assumed to be elasto-plastic. Before yielding the behaviour is assumed elastic. Once reaching the undrained strength, the behaviour is considered plastic resulting in some permanent deformation. After yielding, any unloading due to changing acceleration brings the soil back to perform elastically. Table 4.1 shows the input parameters for the soil material.

Table 4.1 Input material parameters for the Mohr-Coulomb constitutive model

S_u (kPa)	E (kPa)	ν	Damping Ratio
Case 2	$900.S_u$	0.45	2% (For Elastic State)
Case 1			No Additional Damping

Note: In all the analyses Case 2 is used unless it is specified.

The input files for the FLAC modeling of Waba Dam plus the subroutines and programs developed for this study to calculate the instantaneous safety factor are given in Appendix C.

4.3.2 Initial Static Analysis

Prior to performing a dynamic simulation, an equilibrium state must be obtained. This consists of a few stages. First, the foundation is set in place and brought to equilibrium. Then the dam is created in a single placement of material. In the next stage, by applying a mechanical pressure to the upstream face of the dam, the dam responds mechanically to the reservoir filling. Eventually the dynamic simulation may be conducted.

In this study, due to the fact that it has been constructed for a relatively long time, the foundation and the dam body are modeled and brought to the equilibrium state at the same time. Afterwards, water pressure due to reservoir filling is applied to reach a new equilibrium state. In order to fulfill initial static analysis, all the displacements in this stage of modeling are set to zero, but the stresses are kept as they are. Figures 4.15 and

4.16 show the deviatoric stress (principal stress difference) contours and the vertical stress at the initial static state, respectively.

Figure 4.15 Deviatoric stress contours in the initial static state. (unit is in kN/m^2)

Figure 4.16 Vertical stress contours in the initial static state. (unit is in kN/m^2)

At this stage, the static safety factor can be calculated. Two different methods are used to calculate the safety factor in the static mode. The first method is based on the strength reduction technique and the second method is based on the same methodology explained in Section 3.4.2 for the instantaneous safety factor, but for the initial static mode.

Strength Reduction Technique

The strength reduction technique is applied by Matsui & San (1988), Griffiths & Lane (1999), Dawson and Roth and Drescher (1999), and others. According to this method, which is based on the strain failure criterion, the shear strength of the soil material is

reduced until failure occurs, i.e., when a numerical solution no longer exists. At the time of failure ($t_{failure}$) the safety factor, FS_{sr} , can be expressed by Equations 4.6 to 4.7.

$$S(t) = S_{base} \times f(t) \quad [4.6]$$

$$FS_{sr} = \frac{S_{base}}{S(t_{failure})} = \frac{1}{f(t_{failure})} \quad [4.7]$$

where $S(t)$ is the actual shear strength (S_{base}) reduced by factor $f(t)$.

The reduction factor, $f(t)$, can be applied to the cohesion and/or friction angle parameter. When the safety factor is less than 1.0, $f(t_{failure})$ will be greater than unity. In this case, the strength of the material is increased until numerical solutions occur. Then $f(t_{solution})$ will define the safety factor in the same manner. The main advantage of this method over the conventional methods is that a pre-defined slip surface is not required.

FLAC incorporates the use of the strength reduction method. Using this method for Case 2, the initial safety factor is 2.00, which is associated with the downstream berm (Zone 1) shown in Figure 4.17. The FS_{sr} for Case 1 of soil material is equal to 1.51. Figure 4.18, the deformed shape of the potential failure in the downstream part of the crest (Zone 3).

Figure 4.17 Magnified deformation of potential failure of the downstream berm using the strength reduction technique ($FS_{sr} = 2.00$, Case 2)

Figure 4.18 Magnified deformation of potential failure of the downstream crest using the strength reduction technique ($FS_{sr} = 1.49$, Case 1)

Comparison of different method for calculating the static factor of safety

Four different methods of computing the static safety factor are now possible: (1) limit equilibrium analysis using the method of slices; (2) limit equilibrium using the finite stress method via Quake/W and Slope/W; (3) the strength reduction method; and (4) limit equilibrium analysis via stress calculation using FLAC. The last method has been explained in connection with the computation of the instantaneous safety factor proposed in Chapter 3. For obtaining the static safety factor, one can set the time to zero in that computation, i.e., the static safety factor is equal to the initial instantaneous safety factor (F_{i0}) before the dynamic stresses are applied. These four methods are now applied to Waba Dam for Case 1 and Case 2 for comparison. The results are summarized in Table 4.2.

The results of the proposed initial instantaneous safety factor are in general agreement with the other methods. In Case 1, in which the shear strength of the dam material is low, the critical slip surface occurs in the dam body and does not extend to the foundation. The safety factor varies within 6% from the various methods. In Case 2, the critical slip

surface extends to the foundation characterized with a depth-dependent strength. The safety factor in this case varies within 11%. The strength reduction method gives in both cases the lowest safety factor because it automatically seeks the most critical mode of failure. Others seek the critical slip surface with a pre-assumed shape and over specified spacing.

Table 4.2 Summary of safety factor results in initial static analyses

Method	Morgenstern-Price (GEO-SLOPE)	FEM (GEO-SLOPE)	Strength Reduction (FLAC)	Stress-based (FLAC)
Case 1	1.58	1.52	1.49	1.51
Case 2	2.23	2.08	2.00	2.25

4.3.3 Dynamic Analysis

Dynamic analysis needs special considerations. To simulate dynamic analysis, boundary conditions should be changed from those of the static analyses. A lateral free-field boundary condition, which is explained in Section 3.5.2, is applied to both sides of the model. Horizontal accelerations of the far-field and the near-field motions are applied to the base of the model representing bedrock motions. Since when the earthquake stops the dam still continues shaking for a while, the dynamic analysis should be continued until the shaking of the dam is no longer significant. It may take tens of seconds. A model without damping may not stop shaking at all. Horizontal acceleration and velocity responses of the crest of the dam to the far-field (M70R40#8) and the near-field (M55R10#1) input motions are shown in Figures 4.19 to 4.22.

Figure 4.19 Horizontal acceleration response of the crest to the near-field input motion (m/s vs. sec)

Figure 4.20 Horizontal velocity response of the crest to the near-field input motion (m/s vs. sec)

Figure 4.21 Horizontal acceleration response of the crest to the far-field input motion (m/s vs. sec)

Figure 4.22 Horizontal velocity response of the crest to the far-field input motion (m/s vs. sec)

All the dynamic analyses described in this chapter are based on Case 2 of soil profile.

The results of analyses for Case 1 are available in Appendix F.

4.3.4 Instantaneous Safety Factor

Three groups of circular slip surfaces (similar to SLOPE/W) are used in the analysis. Each group is set to examine a specific zone of the dam. The radius lines are placed in 3-meter interval, and the centers are placed in 2- or 3-meter spacing. The location of the mesh of centers of slip surfaces and their radii is determined based on trial and error in preliminary analyses. Also the results of pseudo-static analysis are used as a guideline to locate each group of slip surfaces. Groups of 184, 95 and 184 slip surfaces are used for Zone 1 to 3 respectively. It should be also considered that the center and radius line of the slip surface, which yields the minimum safety factor, is located well within the mesh of centers.

The instantaneous safety factor (F_t) for the two input motions corresponding to the near- and the far-field earthquakes are presented here. While the pseudo-static method gives one single safety factor during the whole earthquake, F_t varies throughout the earthquake. The computation of F_t is conducted at 0.15-second intervals for the stress-based method and 0.05-second for the force-based method. In this study, the minimum safety factor, F_{tmin} , at any time step is obtained by searching for the critical slip surface.

Theoretically, the two methods should give the same safety factor if nowhere in the dam and the subsoil is stressed into the plastic state. At this state, the safety factor is high. When plastic zone begins to exist, there starts a difference in the safety factor computed from the two different methods. The reason for this difference is that for any non-linear analysis, the mobilized shear stress along the slip surface is upper bounded by the shear

strength of the soil. As the stress-based safety factor is defined by the ratio of the mobilized shear stress to the available strength, the safety factor is lower bounded by the value of 1.0. In the force method, however, the forces over the entire sliding body are taken into consideration in defining the safety factor. In fact, when this body starts to move, the usual concept of limit equilibrium is no longer valid as there is a net acceleration involving the motion of the sliding mass. Taking the acceleration or the force at all parts of the sliding body can lead to a safety factor below 1.0. Consequently, the larger the plastic zone, the larger will be the difference between the safety factors computed from the two different methods and the lower will be the safety factor by the force method.

The results of computations on the downstream side of Waba Dam based on these two different methods are shown in Figures 4.23 to 4.28. These Figures show that the variation of F_{min} in different zones of the dam as a function of time during the earthquake for the far- and the near-field motions. The results of the computations are summarized in Table 4.3 which lists the overall minimum safety factors, F_{min} , for the different cases and earthquake motions.

To illustrate the difference of the two methods, Figure 4.29 shows an artificial situation in which the far-field accelerations are doubled in magnitude and the shear strength reduced to half of its original value. In this case, the force-based safety factor drops below unity periodically for short durations, while the stress-based safety factor is lower-bounded by 1.0.

Figure 4.23 Variation of F_{tmin} of Zone 1 under the far-field motion

Figure 4.24 Variation of F_{tmin} of Zone 2 under the far-field motion

Figure 4.25 Variation of F_{min} of Zone 3 under the far-field motion

Figure 4.26 Variation of F_{min} of Zone 1 under the near-field motion

Figure 4.27 Variation of F_{min} of Zone 2 under the near-field motion

Figure 4.28 Variation of F_{min} of Zone 3 under the near-field motion

Figure 4.29 Variation of F_{min} of Zone 1 under the doubled far-field input motion and shear strength reduced by half

Table 4.3 Variation of the overall minimum instantaneous safety factor (F_{min})

Motions	Force-based F_{min}	Stress-based F_{min}
Far-field	1.37	1.41
Near-field	1.52	1.55
2 x far-field, 0.5 x S_u	0.94	1.05

Note: S_u = Undrained shear strength

The first two cases illustrate the point that the instantaneous safety factor is higher based on the stress method. The third and last case corresponds to the artificial situation for

illustration in which the safety factor drops below 1.0 from the force-based method but not from the stress-based method. These results therefore support that the force-based method proposed herein is preferable to the conventional stressed-based method.

In general, the instantaneous safety factor at any time during the two selected ground motions corresponding to events of 10,000-year return period is always above 1.0. This suggests that Waba Dam will unlikely suffer from a landslide type of failure during the selected earthquakes.

4.4 Comparison of Two Methods

In this section analysis results of the pseudo-static method and elasto-plastic method are compared. The results from the pseudo-static and the dynamic methods produce different results for the Waba Dam. For the far-field motions, for instance, the pseudo-static method yields a safety factor of 0.95 while the dynamic method yields a minimum of 1.37 from the force-based approach. In the following, a number of factors are considered to cast light on the inadequacy of the pseudo-static method.

4.4.1 Direction of Seismically Generated Accelerations

The dynamic method not only produces the instantaneous safety factor, it also produces stress, strain and acceleration generated during the earthquake. The vectors of seismically generated accelerations at 10.55 second (one of the critical instants) during the far-field motion are shown in Figure 4.30.

Figure 4.30 Distribution of seismically generated accelerations and plastic zone development at 10.55 second during the far-field motion and the critical slip surface at this time. Each color corresponds to a different state: light gray, elastic; black, yielded in shear; dark gray, elastic but yielded in the past; white, yielded in tension. Each vector in the graph represents the resultant acceleration of 121 nodes of the model.

Figure 4.30 shows that the seismically generated accelerations are pointing to different directions. Consequently, while some of the accelerations are causing the dam to slide down, others in fact are resisting the slide motion at the same instant. This is in contrast to the pseudo-static analysis, which uses a single horizontal acceleration for analysis. The pseudo-static method therefore produces a far smaller safety factor that is not compatible with the actual development of accelerations during the earthquake. At this particular instant for this case, the safety factors from the pseudo-static method and the dynamic method are 0.95 and 1.39, respectively.

4.4.2 Plastic Zone Development

Figure 4.30 also shows the plastic zone developed at this particular instant. The plastic zone is discontinuous and relatively small in extent. This is in contrast to that depicted by the pseudo-static method, which yields a safety factor below 1.0, suggesting that there should be a large and continuous plastic zone.

Looking at the development of plastic zone in Figure 4.30, one may wonder what will be the safety factor of a non-circular slip surface that passes through the plastic zones. To answer this question a FISH subroutine is developed to calculate the stress-based safety factor for multi-linear slip surfaces. For the case shown in Figure 4.30 the instantaneous safety factor for the circular slip surface shown is 1.39 while the minimum safety factor corresponding to multi-linear slip surfaces passing through the plastic zones is as high as 1.70. The main reason for this discrepancy is that although the maximum mobilized shear stress in the plastic zone is equal to the shear strength of the soil, but this maximum shear stress is not necessarily along the multi-linear slip surface. The mobilized shear stress along the specified multi-linear slip surface may be less than the shear strength even in the plastic zone; consequently the overall safety factor is higher than that of the critical safety factor for the circular slip surfaces.

4.4.3 Equivalent Horizontal Seismic Coefficient

The dynamic method also produces results from which the equivalent horizontal acceleration coefficient (k_h) can be determined and used in the pseudo-static method. The equivalent horizontal seismic coefficient is defined here as the coefficient when applied to the pseudo-static analysis yields the same value of safety factor for the same slip surface as the minimum instantaneous safety factor obtained from the force-based method during the entire earthquake. This coefficient has been computed for Waba Dam for the far-field and the near field motions and the results are summarized in Figures 4.31 to 4.32.

Figures 4.31 and 4.32 show the maximum equivalent k_h developed for different potential slip surfaces and Figures 4.33 and 4.34 show the corresponding minimum safety factor for a given slip surface during the entire earthquake. Understandably, the equivalent k_h is dependent on the slip surface chosen for the analysis. For shallow slip surfaces, k_h can reach values as high as 0.5. The critical slip surface is much deeper though, with k_h reaching as low as 0.06 for the far-field and 0.03 for the near-field input motions. This is much smaller than that recommended for this case by the Seed's method, which is 0.11 ($M = 7$).

Figure 4.31 k_h versus potential failure mass associated with different slip surface for the far-field ground motions

Figure 4.32 k_h versus potential failure mass associated with different slip surface for the near-field ground motion

Figure 4.33 Minimum safety factor for given slip surfaces throughout the entire earthquake (F_h) versus potential failure mass associated with different slip surface for the far-field ground motion

Figure 4.34 Minimum safety factor for given slip surfaces throughout the entire earthquake (F_h) versus potential failure mass associated with different slip surface for the near-field ground motion

By comparing the results of F_h (Figures 4.33 and 4.34), it is observed that under the near-field input motion, Zone 1 (the berm) is more critical than Zone 2 (the entire dam and foundation). While under the far-field motion, Zone 2 is the most critical zone. In other words, the far-field earthquake brings more adverse effects to large masses than to small masses. On the other hand, the near-field earthquake is more damaging to smaller masses. It is also observed that, the results of F_h of Zone 1 are almost the same for both the far-field and the near-field input motions, with the computed F_{min} being about 1.52 for both earthquake motions. However, the results of F_h for Zone 1 are lower for the far-field than for the near-field motion. The main reason for this occurrence may lie in the fact that the near field motion contains significant high frequency components of very high accelerations. The high frequency components are associated with short wavelengths that allow changing of acceleration direction over a certain distance. This change of acceleration direction will produce more beneficial effects (opposing dynamic inertia forces balancing each other) on a large mass than on a small mass. This effect is not considered in the pseudo-static approach.

As shown in Figure 4.31 and 4.32, k_h is relatively constant for Zone 2 and the larger masses in Zone 1. Although the safety factor varies with mass in these zones, k_h appears to be constant and insensitive to the mass and the location of the slip surface. F_{min} for the far-field motion is associated with Zone 2. Hence, it seems reasonable to take the average value of k_h associated with Zone 2 as an index to study the variation of the equivalent seismic coefficient (k_h) in the parametric studies. The average equivalent seismic coefficient is 0.059 and 0.035 for the far-field and the near-field motions, respectively.

4.5 Parametric Studies

Parametric studies were carried out to study the effects of two main types of parameters. The first type relates to the soil behaviour that includes the undrained shear strength, S_u , and the modulus of elasticity, E . The second type pertains to the ground motion inputs such as PGA and vertical component of accelerations. The results of the safety factor are based on the force-based method. In this section a brief summary of the results is presented.

Summary of the results of the equivalent seismic coefficient are also presented in this section. As shown in Section 4.4.3, there is a variation of equivalent seismic coefficients in each analysis. The average of the maximum k_h corresponding to Zone 2, $(k_h)_{average}$, is calculated for each analysis. Zone 2, which is the deepest zone, is the critical zone for the far-field input motion, and for each of the analyses, k_h corresponding to the slip surfaces associated with Zone 2 almost remains constant. Therefore, although this zone is not the only critical zone for the far-field input motion, the variation of $(k_h)_{average}$ related to Zone 2 is studied in this section. However, the responses of the dam, are available in Appendix D and the results of the instantaneous safety factor and the equivalent seismic coefficient of the parametric studies are presented in Appendix E.

4.5.1 Undrained Shear Strength

The undrained shear strength used for this study was measured partly with the field vane shear test and partly with the piezocone test which has been shown to produce practically the same value as the field vane test for this type of soil (Konrad and Law, 1978).

Bjerrum (1972) pointed out that the field vane strength may overestimate the actual soil strength for first time static loading mainly because of the strain rate effect and he proposed some reduction curve to remedy the situation. For the present case of dynamic condition in which the loading occurs only for a short duration, it is unlikely that the vane strength will overestimate the actual soil strength. Nevertheless, the effect of strength reduction is studied here. The reduced strength ranges from 100% to 50% of the original strength.

When the shear strength is reduced, the overall safety factor (F_{min}) increases as shown in Figure 4.35. The rate of decrease slightly lowers as the assumed strength drops below 67% of the original. At this strength reduction, the computed F_{min} s are 1.21 and 1.10 for the near-field and the far-field ground motions. When the assumed strength drops below 50%, there is no convergence in the analysis, indicating a lack of system stability. However, such an occurrence is not possible because the clayey materials at the site are not likely to drastically lose the strength during an earthquake.

When the shear strength is reduced, $(k_h)_{average}$ decreases as shown in Figure 4.36. When the assumed strength drops below 67%, $(k_h)_{average}$ decreases more rapidly. At this strength reduction level, the computed $(k_h)_{average}$ s are 0.025 and 0.037 for the near-field and the far-field input motions.

Figure 4.35 Variation of the overall safety factor versus the assumed undrained shear strength with reference to the original value for the far-field and the near-field ground motions

Figure 4.36 Variation of the average maximum equivalent seismic coefficient of Zone 2 versus the assumed undrained shear strength with reference to the original value for the far-field and the near-field ground motions

4.5.2 Modulus of Elasticity

The effects of varying the modulus of elasticity appear to depend on the ground motions. By increasing E_u from its reference value of $900S_u$, the overall safety factor decreases slightly but it decreases sharply then increases for the far-field condition as shown in Figure 4.37. The difference in safety factor is due probably to the interaction between the natural frequency of the dam and the different frequency contents in the two different ground motions.

Figure 4.37 Variation of the overall safety factor with the elasticity modulus for the far-field and the near-field ground motions

Similar to the overall safety factor, variation of the average equivalent seismic coefficient with modulus of elasticity appear to depend on the ground motions. While increasing the

soil modulus increases $(k_h)_{average}$ in the near-field condition, $(k_h)_{average}$ for the far-field condition may go up or down with the increase in the soil modulus (Figure 4.38).

Figure 4.38 Variation of the average maximum equivalent seismic coefficient of Zone 2 with the modulus for the far-field and the near-field ground motions

4.5.3 Peak Ground Acceleration

The peak horizontal ground acceleration has been increased to 150% of the reference value of events of 10,000-year return period to examine the effects of this parameter. The results show that the overall safety factor increases almost linearly with increase in acceleration (Figure 4.39). At peak horizontal acceleration of 150% of the reference value, the safety factor becomes 1.25, still an acceptable value for that exceptionally large earthquake.

The average equivalent seismic coefficient increases slightly and linearly with the increase of peak ground acceleration. Figure 4.40 shows the results.

Figure 4.39 Variation of the overall safety factor with the peak ground acceleration for the far-field and the near-field ground motions

Figure 4.40 Variation of the average maximum equivalent seismic coefficient of Zone 2 with the peak ground acceleration for the far-field and the near-field ground motions.

The variations of F_{min} and $(k_h)_{average}$ with PGA are studied in an artificial condition in which the input shear strength is reduced to 60%. As shown in Figure 4.41, in this case, the increase of the PGA does not change much the overall safety factor or the average

seismic coefficient. The main reason for these results could be the generation of plastic zones that restricts transfer of the s-wave to upper layers. In the plastic state, the tangent shear modulus, which indicates the shear wave velocity, is almost zero. Therefore, transfer of accelerations through the layers of soil is limited. In the same manner, the magnitude of developed dynamic forces is limited. The reduction of the shear strength leads to the growth of the plastic zones. Hence, dynamic forces acting to the mass above does not increase significantly and the safety factor remains almost constant.

Figure 4.41 Variation of the overall safety factor (left) and the average maximum equivalent seismic coefficient of Zone 2 (right) with the peak ground acceleration for the far-field ground motion

4.5.4 Vertical Component of Ground Motion

Normally, only the horizontal components of the ground motions are considered for assessment of dam performance. In reality, there are significant ground motions in the vertical direction. A trial run is made in this study to explore the effect of combining the horizontal and the vertical accelerations on the performance of Waba Dam. The vertical component has been generated by multiplying the corresponding horizontal component

with a factor of $2/3$. The two components are then combined in phase as the ground motions input for the analysis. The results are given in Table 4.4. It is observed that this combination of motions causes a fair impact on the overall safety factor. It is considerable that the factor of safety due to the combination of motions of the near-field is 1.24 and lower than that of far-field, which is 1.31. These overall values are related to Zone 1 for both the far-field and the near-field ground motions.

Table 4.4 Computed overall safety factors ($F_{\min}$) under horizontal accelerations only and under combined horizontal and vertical accelerations

	Minimum Safety factor					
	Zone 1		Zone 2		Zone 3	
	Far Field	Near Field	Far Field	Near Field	Far Field	Near Field
Horizontal	1.52	1.52	1.37	1.63	1.94	2.27
H + Vertical	1.31	1.24	1.33	1.34	1.62	1.78

Figure 4.42 shows the results of k_h for the combination of the horizontal plus vertical input accelerations of the far-field ground motion. It is observed that, unlike in the case for the horizontal acceleration alone, the rate of decrease of the equivalent seismic coefficient for large masses does not reach a limiting value but continues to drop slowly with the increase in the potential sliding mass.

Figure 4.42 k_h versus potential failure mass associated with different slip surfaces for the application of the horizontal plus vertical accelerations of the far-field ground motion

4.5.5 Damping Ratio

The damping ratio chosen for the parametric studies ranges from 1 to 5%. This range is within the values as measured in the laboratory for this soil (Cragg, 1988). Naturally, the safety factor increases with an increase in the damping as shown in Figure 4.43. The increase in the safety factor for the near-field ground motion is quite small while larger increases of safety factors are found with increase in the damping ratio. With the near-field ground motions, the lower end of the damping ratio of 1% produces almost the same safety factor as in the case of 2% used in the main analysis in this chapter. With the far-field ground motion, the safety factor drops to about 1.32 for damping ratio of 1%. This value of safety factor is considered acceptable for the far-field earthquakes of a return period of 10,000 years.

The variation of the average equivalent coefficient with the damping ratio is shown in Figure 4.44. With the increase of the damping ratio, $(k_h)_{average}$ remains almost constant for the near-field, but decreases slightly for the far-field ground motion.

Figure 4.43 Variation of the overall safety factor with the damping ratio for the far-field and the near-field ground motions

Figure 4.44 Variation of the average maximum equivalent seismic coefficient of Zone 2 with the damping ratio for the far-field and the near-field ground motions

4.5.6 Poisson' Ratio

A small range of variation of the Poisson's ratio (0.42 to 0.48) is considered for the essentially undrained condition. Figure 4.45 shows that the overall safety factor decreases with the increase in Poisson's ratio. The rate of change in the safety factor appears to be the same for both the near-field and the far-field ground motions. In both cases, the overall safety factor is still acceptable.

Figure 4.45 Variation of the overall safety factor with Poisson's ratio for the far-field and the near-field ground motions

The average equivalent seismic coefficient increases almost linearly with the increase of Poisson's ratio. The results are shown in Figure 4.46.

Figure 4.46 Variation of the average maximum equivalent seismic coefficient of Zone 2 with Poisson's ratio for the far-field and the near-field ground motions.

4.5.7 Other Parameters and Summary

An elastic analysis is also performed in which the behaviour of the soil material is considered completely elastic instead of elasto-plastic. All the input parameters for this elastic analysis are kept the same as the main reference parameters. The overall safety factors are decreased drastically in this case. The main reason lies in the fact that by elastic consideration of the soil, the stiffness of the model is increased and the damping is dramatically reduced for high strains and hence the resulting acceleration is increased. This in turn leads to a lowering of the computed overall safety factor.

The results of the parametric studies are summarized in Table 4.5.

Table 4.5 Summary of the results of the parametric studies

Variable Parameters	$F_{\min}$ (force-based)						$(K_h)_{average}$	
	Zone 1		Zone 2		Zone 3		Zone 2	
S_u (%)	FF	NF	FF	NF	FF	NF	FF	NF
100	1.52	1.52	1.37	1.63	1.94	2.27	0.059	0.035
67	1.19	1.24	1.10	1.21	1.62	1.83	0.037	0.025
60	1.13	1.18	1.07	1.14	1.60	1.75	0.028	0.021
57	1.10	1.15	1.05	1.11	1.59	1.72	0.024	0.019
50	1.00	1.04	1.01	1.05	1.51	1.62	0.017	0.013
Damping (%)								
5	1.54	1.54	1.43	1.65	2.05	2.34	0.053	0.034
2	1.52	1.52	1.37	1.63	1.94	2.27	0.059	0.035
0.5	1.49	1.52	1.32	1.62	1.94	2.24	0.062	0.035
a_{max} (%)								
100	1.52	1.52	1.37	1.63	1.94	2.27	0.059	0.035
133	1.41	1.45	1.27	1.51	1.80	2.13	0.068	0.042
150	1.37	1.43	1.25	1.45	1.74	2.05	0.072	0.045
E ($\times S_u$)								
500	1.47	1.71	1.23	1.84	2.01	2.48	0.074	0.025
700	1.49		1.37		2.00		0.060	
900	1.52	1.52	1.37	1.63	1.94	2.27	0.059	0.035
1200	1.39		1.14		1.70		0.086	
1500	1.34	1.47	1.11	1.47	1.87	2.05	0.089	0.051
2000	1.37		1.25		1.89		0.074	
ν								
0.42	1.44	1.59	1.43	1.70	1.92	2.11	0.053	0.031
0.45	1.52	1.52	1.37	1.63	1.94	2.27	0.059	0.035
0.48	1.46	1.47	1.28	1.58	1.58	1.69	0.065	0.043
Others								
Reference	1.52	1.52	1.37	1.63	1.94	2.27	0.059	0.035
H + Vertical	1.31	1.24	1.33	1.34	1.62	1.78	0.067	0.057
Elastic	1.13		1.16		1.54		0.078	
$0.6S_u + 1.5a_{max}$	1.10		1.05		1.62		0.029	
$0.6S_u + 2a_{max}$	1.08		1.04		1.55		0.032	
$0.5S_u + 2a_{max}$	0.93		0.94		1.39		0.023	

Note: FF and NF denote the far-field and the near field ground motions, respectively.

Some analysis has also been conducted with the application of a near-field input motion (M55R16#7) with a return period of 2,500 years, which is recommended in the new edition of the NBCC (2005). The results of the analyses are presented in Appendix G.

4.6 Conclusions

The stability of Waba Dam under earthquake events of 10,000 year return period is assessed in this chapter. The pseudo-static method of Seed and the nonlinear dynamic analysis have been applied. The computed stresses, strains and accelerations have been extended to evaluate the instantaneous safety factor using the conventional stress-based approach and a new force-based approach developed for this study. Analysis of the results leads to the following:

1. The results of the pseudo-static method yields a safety factor less than 1.0, suggesting that there would be unacceptably large deformation of the dam under the given earthquake motions.
2. There is some small difference between the two approaches for computing the instantaneous safety factor. Evaluation of these approaches shows that the force-based method is fundamentally more appropriate than the stress-based method.
3. The minimum instantaneous safety factor for Waba Dam throughout the given ground motions have been found to be significantly higher than 1.0, suggesting that Waba Dam will not suffer from landslide type of failure during the earthquakes.
4. Comparison of the pseudo-static and the dynamic methods has revealed some inherent problems with the pseudo-static methods. The problems include:

assumption of direction of acceleration inconsistent with reality, implied plastic zone being too large, and the assumed horizontal acceleration coefficient being too high. In addition, the seismic coefficient is not just a function of input motions but it varies with the soil parameters and geometry of the slip surface.

5. Comparison of the far-field and the near-field ground motions has revealed some inherent differences between the effects of them on the response of the dam. While both ground motions have almost the same impact on relatively small masses, the impact of the far-field ground motion is more significant on large masses.
6. The dynamic analysis is more realistic and therefore more appropriate for the analysis of Waba Dam under earthquakes.

Based on the above observations, Waba Dam will likely perform well under earthquakes of 10,000-year return period.

5. Seismic Deformation of Waba Dam

5.1 Introduction

The last chapter provided a discussion on the seismic stability of Waba Dam. According to the results of the analysis, the dam is stable during earthquakes of 10,000-year return period. In this chapter, the seismic permanent deformation of the dam under the same input ground motions is examined. A conventional method and an elasto-plastic constitutive model are used in the analysis. Finally, parametric studies are conducted to cover the possible ranges of soil parameters and ground motions and also to investigate the sensitivity of the permanent deformation to the input parameters.

To assess the permanent deformation of Waba Dam, Newmark-based methods (such as by Makdisi and Seed, 1978) are compared to a nonlinear dynamic analysis based on the 2-dimensional Fast Lagrangian Analysis of Continua (FLAC). Both types of methods consider the same bilinear stress-strain relationship for modeling the soil behavior. The comparison shows that there will be no permanent deformation as computed by the Newmark-based methods, while there is a definite permanent deformation by the nonlinear dynamic method. The reason for this discrepancy is discussed in light of the difference in the assumptions used in the methods. These findings, coupled with the results of the parametric studies, show that Waba Dam will sustain some tolerable permanent deformations under the design earthquake.

5.2 Newmark-Based Method

The conventional Newmark-based method, which is explained in Chapter 2 and 3, is considered to calculate the earthquake-induced permanent displacement of the dam. For the analysis, two components of GEO-SLOPE Office are used. The first component, QUAKE/W, facilitates the dynamic equivalent linear analysis using finite element method to provide a history of stresses developed in the model during the earthquake. The second component, SLOPE/W, calculates the safety factor for defined slip surfaces based on the history of the stresses provided by QUAKE/W.

5.2.1 Equivalent Linear Analysis

Waba Dam is modeled using QUAKE/W with a relatively fine mesh of finite elements. Due to some limitations of the software, element sizes are somewhat larger than the maximum restricted by Equation 4.1. Figure 5.1 shows the model of the dam. Soil parameters used for equivalent linear analysis are similar to those of the FLAC model mentioned in the previous chapters. However, there are three main differences between the input parameters of the two models including: the average of G_{max} being taken here for each layer of the soil instead of a depth-dependent modulus, a degradation shear modulus curve being defined here, which indicates the same bilinear stress-strain relationship used in FLAC and a strain-dependant damping curve is applied to the model instead of a constant ratio used in FLAC.

Figure 5.1 Mesh of Waba Dam modeled by using QUAKE/W

Figure 5.2 shows the degradation shear modulus curve used in the equivalent linear analysis. This curve is determined in a manner that indicates elasto-plastic (bilinear) behaviour of the material. Based on Equations 3.1, 4.4 and 5.1, the yield strain, γ_{yield} , is given by Equation 5.2 as follows:

$$\gamma_{yield} = S_U / G_{max} \quad [5.1]$$

$$\gamma_{yield} = \frac{2(1+\nu)}{900} \quad [5.2]$$

With Poisson's ratio equal to 0.45, γ_{yield} will be given by:

$$\gamma_{yield} = 0.0032$$

Figure 5.2 Shear modulus degradation curve

To make it compatible with the elasto-plastic analysis, the modulus reduction (G/G_{max}) for shear strains less than γ_{yield} is equal to 1.0 and for strains greater than γ_{yield} is given by Equation 5.3.

$$G/G_{max} = \gamma_{yield} / \gamma \quad [5.3]$$

The cyclic damping curve shown in Figure 5.3 is taken from the results of resonant column tests (Figure 3.6), which is suitable for the equivalent linear analysis.

Figure 5.3 Strain-dependent damping curve

The analysis is performed in two stages. In the first stage, initial static equilibrium is obtained. In the second stage, boundary conditions are changed in a way that the horizontal displacement for lateral boundaries is released but vertical displacement is fixed. Then, horizontal accelerations for both the near-field and the far-field input motions are applied to the bedrock, i.e., the base of the model. During the dynamic analysis the history of mobilized stresses once in each 10 or 15 time steps of input ground

motion is obtained. Figures 5.4 and 5.5 show vertical stress and maximum shear stress contours after initial static analysis.

Figure 5.4 Vertical stress contours in initial static state. (unit is in kN/m^2)

Figure 5.5 Maximum shear stress contours in initial static state. (unit is in kN/m^2)

5.2.2 Permanent Displacement

For the Newmark-based method, the safety factors during the course of the earthquake have to be evaluated first. For this purpose, the variation in the values of safety factor with time is computed using SLOPE/W based on the stress history provided by QUAKE/W as mentioned earlier. These safety factors correspond to the critical slip surface postulated from the analysis of a large number of different slip surfaces. Fig. 5.6 and 5.9 show the variations of the safety factor of the critical slip surface for the far-field and the near-field ground motions. The results show that there is no instant in which the safety factor has slipped below 1.0 under these ground motions, implying that the dam is stable during the earthquakes. These results are confirmed in the previous chapter based

on a more rigorous method that uses the concept of instantaneous factor of safety. As shown in Figures 5.7 and 5.9 the overall safety factor is 1.2 and 1.26 for the near-field and the far-field input motions, respectively.

Since the safety factor always remains above 1.0, the average coefficient of horizontal acceleration k_h would never fall below the coefficient of yield acceleration (k_y). Therefore, there is no occasion for any permanent deformation to occur according to the Newmark's method of analysis. Even by applying the recommendation of Makdisi and Seed (1978), i.e., using 80% of the undrained shear strength as the dynamic shear strength of the soil, the safety factor still stays above 1.0 during most of the course of the earthquake except for a brief moment during which the safety factor drops to 0.96 under the near-field ground motion and the resulting displacement is almost zero. These analyses therefore suggest that no permanent displacement will be developed during the 10,000-year seismic events according to the Newmark-based method.

Figure 5.6 Variation of factor of safety, F , with time, for the near-field motions and the permanent displacement being zero according to the Newmark-based method

Figure 4.7 The safety factor contours at $t = 3.7$ sec when the minimum overall safety factor is obtained (minimum $F = 1.20$)

Figure 5.8 Variation of safety factor, F , with time for the far-field motions and the permanent displacement being zero according to the Newmark-based method

Figure 5.9 The safety factor contours at $t = 9.95\text{sec}$ when the minimum overall safety factor is obtained (minimum $F = 1.26$)

5.3 Nonlinear Dynamic Analysis

As elucidated in Section 4.3 nonlinear dynamic analyses are conducted on Waba Dam based on the elasto-plastic soil behaviour. The horizontal and vertical displacement responses of the dam are presented here.

5.3.1 Responses of the Dam

The horizontal and vertical permanent deformation of Waba Dam at the crest due to the near- and the far-field input motions are shown in Fig. 5.10 through 5.13. For the far-field motions, the horizontal deformation and the vertical settlement are about 12 cm and 8.5 cm, respectively (Figures 5.10 and 5.11). For the near-field motions, the permanent

horizontal deformation and the vertical settlement are about 1.2 cm and 4 cm, respectively (Figures 5.12 and 5.13). These displacements are not considered dangerous in affecting the water retaining ability of this dam, even for earthquakes of such long return period.

Figure 5.10 Horizontal displacement of the crest vs. time during the far-field motion

Figure 5.11 Vertical settlement of the crest vs. time during the far-field motion

Figure 5.12 Horizontal displacement of the crest vs. time during the near-field motion

Figure 5.13 Vertical displacement of the crest vs. time during the near-field motion

5.3 Comparison of Two Models

In the nonlinear dynamic analysis, while the safety factor never drops below 1.0 in the analyses, plastic zones are developed in the dam and in the subsoil during the course of the earthquake. Figure 5.14 shows the plastic zones in the dam and the subsoil developed at a critical instant of the earthquake. These zones vary in location and size throughout the earthquake. Permanent localized deformations are generated in these zones and accumulated during the earthquake. These permanent deformations will affect the overall deformed shape of the dam. Figure 5.15 shows the deformed shape of the upper part of Waba Dam due to the far-field ground motion. Therefore, even if the safety factor for the entire dam never falls below 1.0, permanent deformations can still occur.

Figure 5.14 Distribution of plastic zone development and seismically generated accelerations at 16.70 second during the far-field motion. Each colour corresponds to a different state: light gray, elastic; black, yielded in shear; dark gray, elastic but yielded in the past; white, yielded in tension.

Figure 5.15 Deformed shape of the upper part of Waba Dam due to far-field ground motion (Deformation is magnified 25 times.)

The foregoing analyses on Waba Dam show that the permanent deformation, calculated using the Newmark-based methods, under the input ground motions is always zero because of the assumption that permanent displacement can only be associated with general yielding of the dam and the foundation. Since such yielding does not occur in Waba Dam, the permanent deformation is consequently zero. On the other hand, the nonlinear dynamic analyses using the same stress-strain behaviour of the soil show that plastic zones can be developed, which will produce permanent deformation as a result of irrecoverable plastic deformation. Field observations (Seed, 1979) show that many dams consisting of non-liquefiable materials similar to Waba Dam, sustained some permanent deformations as a result of major earthquakes. The nonlinear dynamic analyses presented here are therefore more consistent with observed dam behaviour under earthquakes.

To verify the results of the equivalent linear analysis by QUAKE/W an analysis with an artificial set of parameters is conducted. In this analysis the shear strength and shear modulus of the soil are reduced to 60% of the original values and the near-field ground motion is applied to the model. Then, the computed stress history is exported into

SLOPE/W and the resulting time-varying safety factor is obtained. In SLOPE/W, instead of the depth-varying shear strength, the average of the shear strength of each layer of the soil is used in order that the parameters are compatible with those used in QUAKE/W.

Figure 5.16 Variation of the factor of safety with time, for the reduced shear strength and shear modulus under the near-field input motion

Figure 5.17 Cumulative displacement along the critical slip surface of Waba Dam with reduced shear strength and shear modulus under the near-field motion

In this artificial case, as shown in Figure 5.16, there are some periods during which the safety factor of the critical slip surface drops below unity. Consequently, some permanent displacement is encountered for the corresponding mass above this slip surface. In SLOPE/W it is assumed that the displacement occurs along the slip surface. The double integration of the excess acceleration results in the cumulative displacement shown in Figure 5.17. In this case, the calculated displacement is almost 4cm.

Figure 5.18 Critical slip surface that yields the maximum permanent displacement under the near-field motion. The shear strength and shear modulus are reduced in this analysis.

The main problem of the Newmark-based methods is that they do not give the vertical settlement directly. SLOPE/W calculates the displacement of the sliding mass along the failure surface. For failure surfaces that do not cross the crest of the dam; therefore, the vertical settlement cannot be easily deduced.

There is also a theoretical problem in the approach of SLOPE/W. The average accelerations, which act through the centroid of the sliding mass, should be scaled to be applicable to the calculation of the displacement along the failure surface. In addition,

since the movement of the circular failure mass is rotational rather than translational, the angular acceleration should be used in the analysis. However, since the displacement is relatively small this error might be negligible.

Figure 5.19 Variations of the shear strength and the shear mobilized shear stress along the critical slip surface under the near-field motion. The shear strength and shear modulus are reduced in this analysis. (Units are in kPa and meter)

A serious problem is also found in the results of stress analysis performed by QUAKE/W. By applying an appropriate equivalent linear stress-strain relationship, it is intended to upper-bound the shear stress by the shear strength. However, the results (Figure 5.19) show that the mobilized shear stress can significantly exceed the shear strength along the slip surface. Although after a few iterations in the equivalent linear analysis, it is indirectly confirmed that the developed shear stress is compatible with the shear strain, the computed shear stress is as high as 75-kPa compared to the shear strength, which is about 32-kPa. As a matter of fact, if the real stress-strain path is

followed by the model, the shear stress can never exceed the shear strength. And as a result, the safety factor based on the mobilized stresses can never drop below unity.

5.5 Parametric Studies

A parametric study has been carried out to study the effects of possible variation of soil behaviour and ground motions on the performance of Waba Dam. The variation in the soil behaviour includes the undrained shear strength, modulus of elasticity, damping ratio and Poisson's ratio. For ground motions consideration, the PGA has been varied and both the vertical and the horizontal components of accelerations have been combined. A summary of results will be presented here. The graphical responses of the dam are available in Appendix D.

5.5.1 Undrained Shear Strength

The effect of undrained shear strength reduction is studied here. The reduced strength ranges from 100% to 50% of the original strength. When the shear strength is reduced, the vertical settlement increases as shown in Figure 5.20. The rate of settlement increases rapidly as the assumed strength drops below 67% of the original. At this strength reduction, the computed permanent settlements are 18.6 cm and 6.6 cm for the near-field and the far-field ground motions. When the assumed strength drops below 50%, there is no convergence in the analysis, indicating a lack of system stability. However, such an occurrence is not possible because the cohesive materials at the site cannot lose that much strength during the earthquake.

Figure 5.20 Variation of vertical permanent displacement at the crest vs. assumed undrained shear strength with reference to the original value for the far-field and the near-field ground motions

Inspired by Figure 5.20 and the strength reduction technique, one can develop a new approach, which defines a settlement-based safety factor for the seismic assessment of cohesive embankment dams. By using this approach coupled with extensive parametric studies, the seismic settlement of earthdams can be estimated based on their geometry and safety factor in the static mode. However, since it is not in the scope of this study to discuss such a method, it is just briefly explained in Appendix H.

5.5.2 Modulus of Elasticity

The effects of varying the modulus of elasticity appear to depend on the ground motions. By increasing E_u from its reference value of $900S_u$, the permanent settlement of the crest does not change much for the near-field condition but it increases sharply then decreases

for the far-field condition, as shown in Figure 5.21. The difference in settlement is probably due to the interaction between the natural frequency of the dam and the different frequency contents in the two different ground motions.

Figure 5.21 Variation of vertical permanent displacement with elasticity modulus for the far-field and the near-field ground motions

5.5.3 Damping Ratio

The damping ratio chosen for the parametric studies ranges from 1 to 5%. This range is within the values as measured in the laboratory for this soil (Cragg, 1988). Naturally, the deformation decreases with an increase in damping as shown in Figure 5.23. The decrease in settlement for the near-field ground motions is quite small, while larger settlement decreases are found with an increase in the damping ratio. With the near-field ground motions, the lower end of the damping ratio of 1% produces almost the same settlement as in the case of 2% used in the main analysis in the foregoing section. With the far-field ground motions, the crest settlement rises to about 10 cm for a damping ratio

of 1%. This magnitude of settlement is still considered tolerable for the far-field earthquakes of a return period of 10,000 years.

Figure 5.22 Variation of vertical permanent displacement at the crest with damping ratio for the far-field and the near-field ground motions

5.5.4 Poisson's Ratio

Figure 5.23 Variation of vertical permanent displacement at the crest with Poisson's ratio for the far-field and the near-field ground motions.

A small range of variation of the Poisson's ratio (0.42 to 0.48) is considered for the essentially undrained condition. Figure 5.24 shows that the vertical settlement decreases with the increase in Poisson's ratio. The rate of change in the settlement at the crest appears to be the same for both the near-field and the far-field ground motions. In both cases, the crest settlement is still within a small value that is less than 10 cm.

5.5.5 Peak Ground Acceleration

The peak horizontal ground acceleration has been increased to 150% of the reference value of events of 10,000-year return period to examine the effects of this parameter. The results show that the settlement increases almost linearly with increase in acceleration (Figure 5.25). At peak horizontal acceleration of 150% of the reference value, the settlement at the crest reaches 16 cm, still a tolerable value for that exceptionally large earthquake.

Figure 5.24 Variation of vertical permanent displacement at the crest vs. maximum peak acceleration of the far-field and the near-field ground motions

5.5.6 Combined horizontal and vertical accelerations

Normally, only the horizontal components of the ground motions are considered for the assessment of dam performance. In reality, there are significant ground motions in the vertical direction. A trial run is made in this study to explore the effect of combining the horizontal and the vertical accelerations on the performance of Waba Dam. The vertical component has been generated by multiplying the corresponding horizontal component with a factor of $2/3$. The two components are then combined in phase as the ground motions input for the analysis. The results are given in Table 5.1. It is observed that this combination of motions causes a fair impact on the settlement at the crest. For both the far-field and the near-field ground motions, this combination almost triples the crest settlement to reach 19 and 11 cm, respectively. Figure 5.26 shows the horizontal displacement under the combination of near-field motions, which is significantly larger than that of the reference case.

Table 5.1 Computed crest deformations under horizontal accelerations only and under combined horizontal and vertical accelerations

	Response at the Crest			
	Settlement (cm)		Horizontal displacement (cm)	
	F-8	N-1	F-8	N-1
H	8.5	4.0	3.1	1.6
H+V	19.3	10.6	12.6	7.4

Note: F-8 = far-field motions, N-1 = near-field motions **H** = Horizontal acceleration
V = Vertical acceleration = $\mathbf{H} \times 2/3$

Figure 5.25 Variation of horizontal displacement with time under the combination of the horizontal and vertical accelerations of the near-field input motion

5.5.7 Summary

A summary of the results of permanent deformations and peak acceleration and velocity of the parametric studies is given in Table 5.2. Some of the results for ultimate situations are not mentioned in the forgoing section. The reported maximum acceleration is not very accurate because some sharp acceleration shocks may not be reflected in the acceleration history, since the acceleration could not be saved in all the time-steps during analysis. The results of the analyses for 2,500-year ground motion are presented in Appendix G.

Table 5.2 Summary of the results of the parametric studies

Variable Parameters	Maximum Response of the Top of Crest							
	Dis _{Verl} (cm)		Dis _{Hori} (cm)		Vel _{max} (cm/s)		Acc _{max} (m/s ²)	
S_u (%)	FF	NF	FF	NF	FF	NF	FF	NF
100	8.47	4.00	3.10	1.60	-38.00	-19.20	12.6	-14.50
67	18.60	6.62	20.00	4.25	-25.60	-15.80	7.88	7.35
60	25.00	8.58	24.50	5.71	-22.83	-15.52	-6.02	8.80
57	30.00	10.00	27.50	7.00	-21.60	-15.20	-6.69	6.20
50	45.00	15.63	38.60	10.70	-18.92	-14.00	7.2	12.40
Damping (%)								
5	6.44	3.69	11.00	1.07	-34.00	-18.60	12.8	-14.80
2	8.47	4.00	3.10	1.60	-38.00	-19.20	12.6	-14.50
0.5	9.96	4.12	12.70	1.26	-39.20	19.55	11.8	-14.50
a_{max} (%)								
100	8.47	4.00	3.10	1.60	-38.00	-19.20	12.6	11.50
133	13.62	5.74	17.00	2.50	-43.00	-22.90	14.3	28.00
150	16.16	6.63	20.00	3.40	-43.00	24.00	14.8	29.00
E ($\times S_u$)								
500	14.24	3.31	15.00	1.50	49.00	21.80	9.7	17.00
700	10.00		12.80		34.60		-8.26	
900	8.47	4.00	3.10	1.60	-38.00	-19.20	12.6	-14.50
1200	27.50		28.70		-33.90		12	
1500	21.73	4.19	20.00	2.40	37.00	-17.40	8.7	17.00
2000	10.00		12.00		-30.80		-9.39	
ν								
0.42	9.50	5.00	11.00	1.70	-31.00	19.60	12.2	10.20
0.45	8.47	4.00	3.10	1.60	-38.00	-19.20	12.6	-14.50
0.48	7.30	3.00	12.70	1.50	-38.40	22.00	8.5	9.72
Others								
Reference	8.47	4.00	3.10	1.60	-38.00	-19.20	12.6	-14.50
$H_+(2/3)V$	19.30	10.60	12.60	7.40	-33.00	17.00	11	15.00
Elastic	0.00		7.60		-45.00		-12.6	
$0.6S_u \& 1.5a_{max}$	45.00		37.00		-26.30		9.3	
$0.6S_u \& 2a_{max}$	65.00		47.00		-29.60		-11	
$0.5S_u \& 2a_{max}$	97.00		73		-28		10	

Note: FF and NF denote the far-field and near field ground motions, respectively.

5.6 Conclusions

The permanent deformation of Waba Dam under the ground motions corresponding to seismic events of 10,000-year return period has been studied in this chapter. The Newmark-based methods and a nonlinear dynamic analysis, both based on the same bilinear relationship, are employed for modelling the dam. The study shows the following:

1. The Newmark-based methods indicate that there would be no permanent deformation under the 10,000-year seismic events. Such results appear to be inconsistent with observed behaviour of non-liquefiable dams during major earthquakes. The main reason for this inconsistency may be attributed to the fact that the Newmark-based methods would yield permanent deformation only when the average horizontal acceleration in the dam mass exceeds the yield acceleration. This situation does not seem to occur in the case of Waba Dam even under the new ground motions. Therefore this type of methods yields zero permanent deformation.
2. The nonlinear dynamic analysis, based on the elasto-plastic soil behaviour yields definite permanent deformation both in the horizontal and vertical directions for the same ground motions used in the Newmark-based methods. The permanent deformation was found to be caused by plastic irrecoverable deformation in some localized areas of the dam and its foundation.
3. Parametric studies have been conducted to cover possible ranges of soil parameters and ground motion characteristics. The results show that within the range of the

parameters studied, the corresponding permanent deformations are within tolerable limits.

Based on the above observations, it is concluded that Waba Dam will perform adequately in terms of permanent deformation under the estimated 10,000-year seismic events.

6. Discussion

In this study, a few general conclusions have been made that may be considered as contributions to the seismic assessment of clayey earthdams similar to Waba Dam. The first part of these contributions is in the light of signifying the limitations of the pseudo-static and Newmark-based methods, and the second part is the proposing of the force-based approach and showing its superiority in the calculation of the instantaneous safety factor.

Comparison of the pseudo-static and the nonlinear dynamic method has revealed some inherent problems with the pseudo-static methods. The assumption of seismic coefficient being just a function of earthquake magnitude or peak ground acceleration is not sufficient. In fact, the seismic coefficient is varied by the variation of failure mass volume and soil parameters as well as ground motion characteristics. Another problem of the pseudo-static methods is in the assumption of the magnitude of the seismic acceleration, which is too high at least in the case of this study (Waba Dam). The distribution of accelerations developed throughout the nonlinear dynamic model shows a variety of directions for the acceleration vectors at a critical time during earthquake. Therefore, the resultant acceleration for a potential failure mass is much lower than the current assumptions of pseudo-static methods.

Evaluation of the conventional stress-based approach and the force-based approaches, which has been developed during the course of this study, has shown that the force-based

method is superior to the stress-based method. In the stress-based approach, the mobilized stresses over the potential failure surface are used to calculate the average acceleration and motivating force in order to calculate the instantaneous safety factor in dynamic analysis. However, due to the fact that following the real stress path the mobilized shear stress cannot exceed the shear strength, the safety factor resulting from the stress-based approach can never take a value less than one. In the force-based approach, however, the motivating force is obtained directly from the dynamic forces developed throughout the model during earthquake excitation. When the initial safety factor is considerably above one, both approaches yield similar results. However, when the initial static safety factor is very close to one, the force-based instantaneous safety factor may periodically drop below one during the earthquake, whereas the stress-based safety factor always remains above one. Therefore, the newly-defined force-based approach is found to be fundamentally rational and practically reliable than the conventional stress-based approach.

Comparison of the Newmark-based methods and the nonlinear elasto-plastic analysis revealed some inherent problems in the Newmark-based methods. The Newmark-based methods yield permanent displacement only when the average horizontal acceleration exceeds the yield acceleration. If the average horizontal acceleration does not exceed the yield acceleration, there will be no permanent deformation in any direction even if there is some local yielding in the dam. In reality, however, occurrence of local yielding leading to irrevocable localized plastic deformations is possible particularly when the

safety factor is low. This may bring about a significant overall permanent deformation for the entire dam.

The conventional Newmark-based methods also suffer from the limitations of the stress-based approach used to compute the average accelerations in these methods. As shown earlier, this method may introduce some overshooting of shear stress in the computation leading to unrealistic stress computation. If this problem is removed with careful simulation of the stress-strain curve by some means, the whole approach will bring about a situation that the instantaneous safety factor will never drop below 1.0 and hence there will be zero permanent deformation.

7. Summary and Conclusions

7.1 Seismic Performance of Waba Dam

The seismic performance of Waba Dam under earthquake events of 10,000-year return period is assessed in this study. Waba Dam is a relatively low dam with wide berms for stability. It is built of clayey soil and founded over a thick deposit of soft to stiff marine clay. The ground motions are generated on the basis of the uniform hazard spectrum of the dam site. Twenty ground motions have been generated for both the far-field and the near-field events. The seismic stability and permanent deformation of the dam have been examined by both conventional pseudo-static methods and the nonlinear dynamic analysis.

To assess the seismic stability of the dam, the pseudo-static method of Seed (1979) and elasto-plastic dynamic analysis have been applied. By developing a few computer functions to suite the results of FLAC, the stresses and accelerations calculate to determine the instantaneous safety factor during earthquake. Two different approaches have been developed in this study to calculate the instantaneous safety factor: the conventional stress-based approach and a new approach based on dynamic forces.

The results of the pseudo-static methods yield a safety factor less than 1.0, suggesting that there would be unacceptably large deformation of the dam under the given earthquake motions. However, the results of the elasto-plastic analyses have

shown that the assumption of the seismic coefficient used in the pseudo-static analysis is too conservative for Waba Dam.

The minimum instantaneous safety factor for Waba Dam throughout the design ground motions, obtained from both the stress-based and the force-based methods, has been found to be significantly higher than 1.0, signifying that the dam will not suffer from landslide-type failure during the earthquakes.

Parametric studies have been conducted to cover possible ranges of soil parameters and ground motions characteristics. The variation of each of the input parameters affects both the overall safety factor and the average seismic coefficient. However, over the possible range of input parameters, the instantaneous safety factor does not drop below unity throughout the given input motions.

To assess the earthquake-induced permanent deformation of Waba Dam the conventional Newmark-based method and elasto-plastic dynamic analysis have been employed. While the elasto-plastic analysis directly yields the permanent deformation of the dam, Newmark-based method yields a displacement along the critical failure surface in a coupled analysis.

The Newmark-based method indicates that there would be no permanent displacement under the 10,000-year seismic events. Apparently, such a result is inconsistent with the observed behaviour of cohesive embankment dams during major earthquakes. This

method would yield permanent displacement only when the average horizontal acceleration of the potential failure mass exceeds the yield acceleration. This situation does not happen in the case of Waba Dam. Therefore, the permanent displacement would be zero using this method.

The nonlinear dynamic analysis based on the elasto-plastic soil behaviour yields definite permanent deformation both in the horizontal and vertical directions for the far-field and the near-field ground motions. The permanent deformation was found to be caused by plastic irrecoverable deformation due to local yielding throughout the dam and its foundation during the earthquake.

The results of the parametric studies show that the permanent deformation is influenced by the variation of the input parameters. However, within the range of parameters studied, the corresponding permanent deformations are within tolerable limit.

Based on the above observations, it is concluded that the nonlinear dynamic analysis is more realistic and hence more appropriate for the seismic analysis of Waba Dam. The results of nonlinear dynamic analysis show that Waba Dam will perform adequately in terms of stability and permanent deformation under the estimated 10,000-year seismic events.

7.2 Suggestions for Future Works

Analysis results of this study confirmed by past records have shown that cohesive earthdams are unlikely to suffer from landslide-type failures during earthquakes. Therefore, the author believes that the focus of future studies should be mainly on the seismic permanent deformation rather than the slope instability.

Since Waba Dam is exceptional in terms of the dam geometry and the thick subsoils, it is suggested that a study of a generic cohesive dam be conducted along the same scope as that for Waba Dam.

Development of a semi-empirical method for the seismic evaluation of embankment dams in ENA is also suggested. This will consist of extensive parametric studies to gather a database for establishing some statistical correlations between the permanent deformation and the various parameters of dam geometry, engineering behaviour of dam material, subsoil conditions, and earthquake characteristics. Appendix H might be helpful in this regard.

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Appendixes

A Waba Dam Section

B Far-Field and Near-Field Ground Motions

B.1 Far-field Motions

M70R40#4 **M70R40#5** **M70R40#6**

B.2 Near-field Motions

M55R10#2

M55R10#3

M55R10#4

M55R10#5 **M55R10#6** **M55R10#7**

M55R10#8 **M55R10#9** **M55R10#10**

C Input Files and Subroutines for Modelling and Analysis of Waba Dam

C.1 Input file for Optimum Modeling of Waba Dam with FLAC (parametric studies is not included)

```
;Units m & kN
new
config dyn ex 5
grid 275 200
m mohr
gen -200 25 -200 103 -130 103 -130 25 rat 0.98,0.996 i=1,36 j=1,201
gen -130 25 -130 103 20 103 20 25 rat 1,0.996 i=36,186 j=1,201
gen 20 25 20 103 200 103 200 25 rat 1.01,0.996 i=186,276 j=1,201
table 1 -200,91.4 -103.2,91.4
table 1 -79.2,97.4 -18.6,97.4
table 1 -3,102.6 3,102.6
table 1 13.62,99.06 18.6,97.4
table 1 82.3,97.4 106.3,91.4 200,91.4
table 1 200,103 -200,103 -200,91.4
gen table 1
;;;;;; mat_maker must be entered here;;;;;;
model null reg=1,200
set grav=9.81
set dyn=off flow=off
fix x i=1
fix x i=276
fix x y j=1
def wazn_yab
```

```

float wazn
array wazn(276,201)
  loop i (1,igp)
    loop j (1,jgp)
      wazn(i,j) = ybody(i,j) ; yforce(i,j)
    end_loop
  end_loop
end
wazn_yab
;
;;;;;; circle_maker must be entered here for Zone 1 ;;;;;;;;;;
;;;;;; circle_maker must be entered here for Zone 2 ;;;;;;;;;;
;;;;;; circle_maker must be entered here for Zone 3 ;;;;;;;;;;
;;;;;; mat_maker may be re-entered here for with different parameters ;;;
save w8-1.sav
rest w8-1.sav
;
;;;;;; parametric studies should be started from here using w8-1.sav;;;;;;;;;;
;
his 1 unbal
his 2 ydis xdis i=165 j=159
solve
save w8-2.sav
rest w8-2.sav
ini xvel=0 yvel=0 xdisp=0 ydisp=0
apply press 75.15 var 0 -75.15 i=276,180 j=159,186
;set step 30000
solve
save w8-3.sav
rest w8-3.sav
solve fos associated file w8-4.fsv include coh exclude ten int fric

```

```

rest w8-3.sav
his reset
ini xvel=0 yvel=0 xdisp=0 ydisp=0
def state0
  loop iz (1,izones)
    loop jz (1,jzones)
      state(iz,jz) = 0
    end_loop
  end_loop
end
state0
set large, dyn=on ncwrite=50
set dytime=0
fix x y j=1
apply ff
his 1 nstep=47 dytime
his 2 nstep=47 unbal
his 10 read L:\w\ m70r40-8.dat      ; m55r10-1.dat  m70r40-8.dat
his 11 nstep=235 xdis i=165 j=199
his 12 nstep=235 ydis i=165 j=199
his 13 nstep=10 xacc i=165 j=199
his 14 nstep=47 yacc i=165 j=199
his 15 nstep=47 xvel i=165 j=199
his 16 nstep=47 yvel i=165 j=199
his 26 nstep=235 xdis i=89 j=180
his 27 nstep=235 ydis i=89 j=180
his 31 nstep=47 xdis i=165 j=1
his 33 nstep=47 xacc i=165 j=1
his 35 nstep=47 xvel i=165 j=1
,,,,,,,,,,,,, sf_maker must be entered here ,,,,,,,,,,
;

```

```

; set multi on
apply xacc 0.01 his 10 j=1
apply yacc 0 j=1 ; 0.00667 his 10
set dy_damp=rayleigh 0.02 0.5 mass ; 0.35 0.64 0.5 0.42 .57
set clock 200000 step 1000000
solve dyt maxtime
save w8-f8.sav
rest w8-f8.sav
,,,,,,,,, sf_reader must be entered here,,,,,,,,,
,,,,,,,,, circle_reader must be entered here,,,,,,,,,
,,,,,,,,, defo_reader must be entered here,,,,,,,,,

```

C.2 FISH routine developed to define material properties

```

def mat_maker
  red = 1
  p_ratio = 0.45
  loop jz (1,jzones)
    yc = (y(1,jz)+y(1,jz+1))/2
    if yc < 31.9 then
      Su = -0.791 * yc + 173
      ro = 1.80
    else
      if yc < 54.9 then
        Su = -1.66 * yc + 184.04
        ro = 1.80
      else
        if yc < 77.4 then

```

```

    Su = -1.4 * yc + 151.5
    ro = 1.60
else
    if yc < 91.4 then
        Su = 33.33
        ro = 1.60
    else
        Su = 96          ;    23.5 for Case 1
        ro = 1.9
    end_if
end_if
end_if
y_mod = 900 * Su
if yc >= 64
    Su = red * Su
end_if
shear_m = y_mod / (2*(1+p_ratio))
bulk_m = y_mod / (3*(1-2*p_ratio))
command
    prop cohesion=Su dil=0 fri=0 tension=0 j=jz
    prop bulk=bulk_m dens=ro shear=shear_m j=jz
end_command
end_loop
end
mat_maker

```


C.3 FISH routine developed to define characteristics of the slip circles (may be entered a few times)

```
def circle_maker
  float circles3
  array circles3(245,6)
  status=open('circle03.dat',1,1)

;=====
===
; Zone 1 : ( (-97,100) (-79,106) (-79,135) ) & (75-89) h 5 v 8 z 4
;-----
; x1=-102 ; 252
; y1=109
; x2=-70
; y2=117.0
; x3=-85.0
; y3=159
; h=6
; v=5
; y4=84
; y5=69
; z=5.0
; xmin=-145
; xmax=-35
; ylim=91.4
;=====
===
; Zone 2 : (-65.2,180) & z 67 or (-75,150) (-65,150) (-84,195) & z (87,91)
;-----
; x1=-68 ; 105
; y1=146
```

```

; x2=-53
; y2=146
; x3=-67
; y3=213
; h=4
; v=6
; y4=70
; y5=64
; z=2
; xmin=-155
; xmax=15
; ylim=91.4
;=====
===
; Zone 3 : ( (-15,107) (-1,114) (-13,130) ) & z (86-94)
;-----
x1=-18 ; 245
y1=106
x2=-4
y2=108
x3=-24
y3=150
h=6
v=6
y4=97
y5=84
z=4
xmin=-75
xmax=16
ylim=97.4

```

```

=====
===
; Zone 4 : ( (-15,107) (-1,114) (-13,130) ) & z (86-94)
;-----
; x1=20
; y1=19
; x2=29
; y2=22
; x3=26
; y3=32
; h=2
; v=3
; y4=4.5
; y5=0.5
; z=2
; xmin=4.5
; xmax=59.5
; ylim=5
=====
===
indicator=777
float zz x0 y0 r nes x` y` x1 xp x11 y11 cohe len tet
mor= 5000 ; it must be at least 5000
nes=mor/pi
zz=(y4-y5)/z
ccc=1
array cir(4)
array res(6)
array info(5)
jdown=0
iright=1

```

```

ileft=0
ci=0
zo=0
loop while y(1,jdown+1) < y5
    jdown=jdown+1
end_loop
loop while x(ileft+1,1) < xmin-5
    ileft=ileft+1
end_loop
loop while x(iright,1) <= xmax+10
    iright=iright+1
end_loop
info(1)=string(izones)
info(2)=string(jzones)
info(3)=string(jdown)
info(4)=string(ileft)
info(5)=string(iright)
status=write(info,5)
loop io (0,h)
    loop jo (0,v)
        loop k (0,z)
section
            x0 = io*(x2-x1)/h + jo*(x3-x2)/v + x1
            y0 = jo*(y3-y2)/v + io*(y2-y1)/h + y1
            r = y0 - y4 + zz*k
            d1=((x0-xmin)^2+(y0-ylim)^2)^0.5
            d2=((x0-xmax)^2+(y0-ylim)^2)^0.5
            if r > min(d1,d2)
exit section
        end_if
    loop te (1,5000)

```

```

x` = x0 + r*cos(te/nes)
y` = y0 - r*sin(te/nes)
loop ll (ileft,iright)
  loop m0 (jdown,jzones)
    if cohesion(ll,m0) # 0 then
p=m0+1
yp=(x`-x(ll,p))*(y(ll+1,p)-y(ll,p))/(x(ll+1,p)-x(ll,p))+y(ll,p)
  if y` <= yp
yl=(x`-x(ll,m0))*(y(ll+1,m0)-y(ll,m0))/(x(ll+1,m0)-x(ll,m0))+y(ll,m0)
  if y` > yl
p=ll+1
xp=(y`-y(p,m0))*(x(p,m0+1)-x(p,m0))/(y(p,m0+1)-y(p,m0))+x(p,m0)
  if x` <= xp
xl=(y`-y(ll,m0))*(x(ll,m0+1)-x(ll,m0))/(y(ll,m0+1)-y(ll,m0))+x(ll,m0)
  if x` > xl
    uh=1
    if (1000*ll+m0) # zon
      if pic=1
check=1
    len=((x`-x11)^2+(y`-y11)^2)^0.5
    if x'=x11 then
      tet=(abs(y`-y11)/(y`-y11))*pi/2
    else
      tet=pi-atan((y`-y11)/(x`-x11))
    end_if
    res(1)=string(lll)
    res(2)=string(m00)
    res(3)=string(len)
    res(4)=string(tet)
    res(5)=string(cohe)
    res(6)=string(77)

```

```

status=write(res,6)
circles3(ci,4)=circles3(ci,4) + len * cohe
end_if
if cic=0
    ci=ci+1
    cir(1)=string(indicator)
    cir(2)=string(x0)
    cir(3)=string(y0)
    cir(4)=string(r)
    status=write(cir,4)
    circles3(ci,1)=x0
    circles3(ci,2)=y0
    circles3(ci,3)=r
    cic=1
end_if
zo=zo+1
x11 = x`
y11 = y`
pic=1
zon= 1000*ll + m0
lll=ll
m00=m0
cohe=cohesion(ll,m0)
ll=iright
m0=jzones
end_if
end_if
end_if
end_if
end_if
else

```

```

    if ll=iright
    if m0=jzones
    if pic = 1
    if uh=0
    if fir=0
check=2
    len=((x`-x11)^2+(y`-y11)^2)^0.5
    if x'=x11 then
        tet=(abs(y`-y11)/(y`-y11))*pi/2
    else
        tet=atan((y`-y11)/(x`-x11))
    end_if
    res(1)=string(lll)
    res(2)=string(m00)
    res(3)=string(len)
    res(4)=string(tet)
    res(5)=string(cohe)
    status=write(res,5)
    circles3(ci,4)=circles3(ci,4) + len * cohe
    uh=1
    fir=1
exit section
    end_if
    end_if
    end_if
    end_if
    end_if
    end_if
end_loop
end_loop
uh=0

```

```

        end_loop
    end_section
        y11=0
        uh=0
        fir=0
        pic=0
        oops=0
        zon=0
        cic=0
        end_loop
    end_loop
end_loop
info(1)=string(7777)
info(2)=string(ci)
info(3)=string(zo)
status=write(info,3)
status=close
end
circle_maker

```

C.4 FISH routine developed to calculate force-based safety factor and to save computed stresses

```

def sf_maker
    if minsaz=0
ch=9
        float noch ydel xdel d1 mas mxa mya mxad myad mintime stime
        float mxg wyg zone1 zone2 zone3 sf delt t0 vxy accx accy
        float cicoms2 circos3 circoms1 myg iin jin defo
    
```



```

maxtime=17
mintime=1.75
sjump = 0.15
fjump = 0.05
red = 1.0
array accx(276,201)
array accy(276,201)
array vxy(276,201,2)
array zone1(465,7)
array zone2(465,7)
array zone3(465,7)
array sline(6)
array circoms1(252,8)
array circoms2(105,8)
array circoms3(245,8)
array iin(276,201)
array jin(276,201)
array defo(465,5)
xmin=-155
xmax=35
ymin = 64
iright=1
ileft=0
jdown=0
loop zs (1,465)
  zone1(zs,2) = 100
  zone2(zs,2) = 100
  zone3(zs,2) = 100
end_loop
loop i (1,igp)
  loop j (1,jgp)

```

```

    iin(i,j) = x(i,j)
    jin(i,j) = y(i,j)
end_loop
end_loop
loop while jin(1,jdown+1) < ymin
    jdown=jdown+1
end_loop
loop while iin(ileft+1,1) < xmin
    ileft=ileft+1
end_loop
loop while iin(iright,1) <= xmax
    iright=iright+1
end_loop
status= open('str_his.dat',1,1)
sline(1) = string(jdown)
sline(2) = string(ileft)
sline(3) = string(iright)
status=write(sline,3)
sabt = 0
t0=-1
stime = mintime
minsaz=1
indicator=777
end_if
if dytime >= (maxtime - 0.04)
    if cl=0
        sline(1)=string(indicator)
        status=write(sline,1)
    end_if
    cl=1
status=close

```

```

    exit
end_if
While_stepping
ch=0
    delt = dytime - t0
    t0 = dytime
    loop i (1,igp)
        loop j (1,jgp)
            accx(i,j) = (-xvel(i,j) + vxy(i,j,1)) / delt
            accy(i,j) = (-yvel(i,j) + vxy(i,j,2)) / delt
            vxy(i,j,1) = xvel(i,j)
            vxy(i,j,2) = yvel(i,j)
        end_loop
    end_loop
;,,,
    if abs(accx(165,199)) > xacmax
        xacmax = accx(165,199)
    end_if
    if abs(accy(165,199)) > yacmax
        yacmax = accy(165,199)
    end_if
    if abs(xvel(165,199)) > xvemax
        xvemax = xvel(165,199)
    end_if
    if abs(yvel(165,199)) > yvemax
        yvemax = yvel(165,199)
    end_if
    nxac = nxac + 1
    xacsic = xacsic + abs(accx(165,199))
    if dytime >= mintime
        loop i (ileft,iright)

```



```

defo(sabt,5) = accx(165,199)
loop c1 (1,252)
  if circles1(c1,4) > 400
    loop i (ileft,iright)
      loop j (jdown,jgp)
        if jin(i,j) # 0
          xdel= iin(i,j) - circles1(c1,1)
          ydel= circles1(c1,2) - jin(i,j)
          d1=(xdel^2 + ydel^2)^0.5
          if d1 <= circles1(c1,3)
check=2
            germ = (-1 * wazn(i,j)) / 9.81
            mxa = mxa + (germ*(accx(i,j)))
            mya = mya + (germ*(accy(i,j)))
            mxad = mxad + (germ*(accx(i,j)*ydel))
            myad = myad + (germ*(accy(i,j)*xdel))
            if sabt = 1
              mas = mas + germ
              mxg = mxg + (germ * ydel)
              myg = myg + (germ * xdel)
              wyg = wyg + (ybody(i,j)*xdel) + (xbody(i,j)*ydel)
            end_if
          end_if
        end_if
      end_loop
    end_loop
  if sabt = 1
    circles1(c1,6) = 100
    circles1(c1,5) = mas
    circoms1(c1,1) = wyg
    circoms1(c1,2) = mxg / mas
  
```

```

        circoms1(c1,3) = myg / mas
    end_if
sf = (-1*circles1(c1,4)*circles1(c1,3))/(mxad + myad + circoms1(c1,1))
sf = abs(sf) * red
if sf < circles1(c1,6)
    circles1(c1,6) = sf
    circoms1(c1,4) = mxa / circles1(c1,5)
    circoms1(c1,5) = mya / circles1(c1,5)
    circoms1(c1,6) = mxad / (circles1(c1,5) * circoms1(c1,2))
    if circoms1(c1,3) # 0 then
        circoms1(c1,7) = myad / (circles1(c1,5) * circoms1(c1,3))
    else
        circoms1(c1,7) = myad
    end_if
    circoms1(c1,8) = (mxad + myad) / (circles1(c1,5) * circoms1(c1,2))
end_if
if sf < zone1(sabt,2)
check=3
    zone1(sabt,1) = dytime
    zone1(sabt,2) = sf
    zone1(sabt,3) = c1
    zone1(sabt,4) = mxa / circles1(c1,5)
    zone1(sabt,5) = mya / circles1(c1,5)
    zone1(sabt,6) = mxad
    zone1(sabt,7) = myad
end_if
end_if
mas = 0
mx = 0
mya = 0
mxad = 0

```

```

myad = 0
myg = 0
mxg = 0
wyg = 0
end_loop
.....
loop c1 (1,105)
  if circles2(c1,4) > 400
    loop i (ileft,iright)
      loop j (jdown,jgp)
        if jin(i,j) # 0
          xdel= iin(i,j) - circles2(c1,1)
          ydel= circles2(c1,2) - jin(i,j)
          d1=((xdel^2) + (ydel^2))^0.5
          if d1 < circles2(c1,3)
check=4
            germ = (-1 * wazn(i,j)) / 9.81
            mxa = mxa + (germ*(accx(i,j)))
            mya = mya + (germ*(accy(i,j)))
            mxad = mxad + (germ*(accx(i,j)*ydel))
            myad = myad + (germ*(accy(i,j)*xdel))
            if sabt = 1
              mas = mas + germ
              mxg = mxg + (germ * ydel)
              myg = myg + (germ * xdel)
              wyg = wyg + (ybody(i,j)*xdel) + (xbody(i,j)*ydel)
            end_if
          end_if
        end_if
      end_loop
    end_loop
  end_loop

```

```

if sabt = 1
    circles2(c1,6) = 100
    circles2(c1,5) = mas
    circoms2(c1,1) = wyg
    circoms2(c1,2) = mxg / mas
    circoms2(c1,3) = myg / mas
end_if
sf = (-1*circles2(c1,4)*circles2(c1,3))/(mxad + myad + circoms2(c1,1))
.sf = abs(sf) * red
if sf < circles2(c1,6)
    circles2(c1,6) = sf
    circoms2(c1,4) = mxa / circles2(c1,5)
    circoms2(c1,5) = mya / circles2(c1,5)
    circoms2(c1,6) = mxad / (circles2(c1,5) * circoms2(c1,2))
    if circoms2(c1,3) # 0 then
        circoms2(c1,7) = myad / (circles2(c1,5) * circoms2(c1,3))
    else
        circoms2(c1,7) = myad
    end_if
    circoms2(c1,8) = (mxad + myad) / (circles2(c1,5) * circoms2(c1,2))
end_if
if sf < zone2(sabt,2)
check=5
    zone2(sabt,1) = dytime
    zone2(sabt,2) = sf
    zone2(sabt,3) = c1
    zone2(sabt,4) = mxa / circles2(c1,5)
    zone2(sabt,5) = mya / circles2(c1,5)
    zone2(sabt,6) = mxad
    zone2(sabt,7) = myad
end_if

```



```

    end_if
    mas = 0
    mxa = 0
    mya = 0
    mxad = 0
    myad = 0
    myg = 0
    mxg = 0
    wyg = 0
    end_loop
.....
    loop c1 (1,245)
        if circles3(c1,4) > 400
            loop i (ileft,iright)
                loop j (jdown,jgp)
                    if jin(i,j) # 0
                        xdel= iin(i,j) - circles3(c1,1)
                        ydel= circles3(c1,2) - jin(i,j)
                        d1=((xdel^2) + (ydel^2))^0.5
                        if d1 < circles3(c1,3)
check=6
                            germ = (-1 * wazn(i,j)) / 9.81
                            mxa = mxa + (germ*(accx(i,j)))
                            mya = mya + (germ*(accy(i,j)))
                            mxad = mxad + (germ*(accx(i,j)*ydel))
                            myad = myad + (germ*(accy(i,j)*xdel))
                            if sabt = 1
                                mas = mas + germ
                                mxg = mxg + (germ * ydel)
                                myg = myg + (germ * xdel)
                                wyg = wyg + (ybody(i,j)*xdel) + (xbody(i,j)*ydel)

```

```

        end_if
    end_if
end_if
end_loop
end_loop
if sabt = 1
    circles3(c1,6) = 100
    circles3(c1,5) = mas
    circoms3(c1,1) = wyg
    circoms3(c1,2) = mxg / mas
    circoms3(c1,3) = myg / mas
end_if
sf = (-1*circles3(c1,4)*circles3(c1,3))/(mxad + myad + circoms3(c1,1))
sf = abs(sf) * red
if sf < circles3(c1,6)
    circles3(c1,6) = sf
    circoms3(c1,4) = mxa / circles3(c1,5)
    circoms3(c1,5) = mya / circles3(c1,5)
    circoms3(c1,6) = mxad / (circles3(c1,5) * circoms3(c1,2))
    if circoms3(c1,3) # 0 then
        circoms3(c1,7) = myad / (circles3(c1,5) * circoms3(c1,3))
    else
        circoms3(c1,7) = myad
    end_if
    circoms3(c1,8) = (mxad + myad) / (circles3(c1,5) * circoms3(c1,2))
end_if
if sf < zone3(sabt,2)
check=7
    zone3(sabt,1) = dytime
    zone3(sabt,2) = sf
    zone3(sabt,3) = c1

```

```

zone3(sabt,4) = mxa / circles3(c1,5)
zone3(sabt,5) = mya / circles3(c1,5)
zone3(sabt,6) = mxad
zone3(sabt,7) = myad
end_if
end_if
mas = 0
mx = 0
my = 0
mxad = 0
myad = 0
myg = 0
mxg = 0
wyg = 0
end_loop
ch=8
.....
end_if
end
sf_maker

```

C.5 FISH routine developed to read force-based safety factors from the output file

```

def sf_reader
array farsh(7)
status= open('fsf1-f8.dat',1,1)
loop tar (1,465)
if zone1(tar,1) # 0

```

```

loop pood (1,7)
  farsh(pood) = string(zone1(tar,pood))
end_loop
status= write(farsh,7)
end_if
end_loop
farsh(1)=string(7777)
status= write(farsh,1)
status=close
status= open('fsf2-f8.dat',1,1)
loop tar (1,465)
  if zone2(tar,1) # 0
    loop pood (1,7)
      farsh(pood) = string(zone2(tar,pood))
    end_loop
    status= write(farsh,7)
  end_if
end_loop
farsh(1)=string(7777)
status= write(farsh,1)
status=close
status= open('fsf3-f8.dat',1,1)
loop tar (1,465)
  if zone3(tar,1) # 0
    loop pood (1,7)
      farsh(pood) = string(zone3(tar,pood))
    end_loop
    status= write(farsh,7)
  end_if
end_loop
farsh(1)=string(7777)

```

```

        status= write(farsh,1)
    status=close
end
sf_reader

```

C.6 FISH routine developed to read the results of the slip surfaces from the output file

```

def circle_reader
array baft(8)
status= open('circ1.dat',1,1)
loop tar (1,252)
    if circles1(tar,6) # 0
        loop pood (1,6)
            baft(pood) = string(circles1(tar,pood))
        end_loop
        status= write(baft,6)
    loop pood (1,8)
        baft(pood) = string(circoms1(tar,pood))
    end_loop
    status= write(baft,8)
    end_if
end_loop
baft(1)=string(7777)
status= write(baft,1)
status=close
status= open('circ2.dat',1,1)
loop tar (1,105)
    if circles2(tar,6) # 0

```

```

loop pood (1,6)
  baft(pood) = string(circles2(tar,pood))
end_loop
status= write(baft,6)
loop pood (1,8)
  baft(pood) = string(circoms2(tar,pood))
end_loop
status= write(baft,8)
end_if
end_loop
baft(1)=string(7777)
status= write(baft,1)
status=close
status= open('circ3.dat',1,1)
loop tar (1,245)
  if circles1(tar,6) # 0
    loop pood (1,6)
      baft(pood) = string(circles3(tar,pood))
    end_loop
    status= write(baft,6)
    loop pood (1,8)
      baft(pood) = string(circoms3(tar,pood))
    end_loop
    status= write(baft,8)
  end_if
end_loop
baft(1)=string(7777)
status= write(baft,1)
status=close
end
circle_reader

```

C.7 FISH routine developed to read the responses from the output file

```

def defo_reader
; array baft(5)
status= open('defo.dat',1,1)
loop tar (1,465)
if defo(tar,1) # 0
loop pood (1,5)
baft(pood) = string(defo(tar,pood))
end_loop
status= write(baft,5)
end_if
end_loop
baft(1)=string(7777)
status= write(baft,1)
status=close
end
defo_reader

```

C.8 QBASIC program developed to calculate stress-based safety factor (should be entered 3 times)

```

minzone = 10
minhzone = 2
maxdt = 300

```

```

red = 1.00
OPEN "L:/str_his.dat" FOR INPUT AS #1
OPEN "L:/circle01.dat" FOR INPUT AS #2
OPEN "L:/sfs1-f8.out" FOR OUTPUT AS #3
INPUT #2, izones, jzones, jdown, ileft, iright
INPUT #1, jdown, ileft, iright
jneed = jzones - jdown + 1
REM $DYNAMIC
DIM ss1(ileft TO iright, 1 TO jneed) AS SINGLE
DIM ss2(ileft TO iright, 1 TO jneed) AS SINGLE
DIM ss3(ileft TO iright, 1 TO jneed) AS SINGLE
DIM ci(1 TO iright + 2 * jneed, 1 TO 5) AS SINGLE
REM $STATIC
DIM fs(1 TO maxdt, 1 TO 7) AS SINGLE
INPUT #1, dt
ndt = 0
WHILE dt < 777
    ndt = ndt + 1
    dt1 = dt
    fs(ndt, 1) = dt
    fs(ndt, 2) = 100
    WHILE dt = dt1
        INPUT #1, i, j, sxx, sxy, syy
        jo = j - jdown + 1
        ss1(i, jo) = sxx
        ss2(i, jo) = sxy
        ss3(i, jo) = syy
        INPUT #1, dt
    WEND
    INPUT #2, indi
    WHILE indi < 7777

```



```

good = 0
WHILE good = 0 AND indi < 7777
  INPUT #2, x0, y0, r
  indi = 77
  z = 0
  WHILE indi = 77
    INPUT #2, m, n, l, d, co
    z = z + 1
    ci(z, 1) = m
    ci(z, 2) = n
    ci(z, 3) = l
    ci(z, 4) = d
    ci(z, 5) = co
    INPUT #2, indi
  WEND
  IF z > minzone AND ci(1, 2) > (ci(z, 2) + minhzone) THEN
    good = 1
  ELSE
    good = 0
  END IF
WEND
fav = 0
fdr = 0
FOR zon = 1 TO z
  fav = fav + ci(zon, 5) * ci(zon, 3)
REM PRINT ci(zon, 1), ci(zon, 2) - jdown + 1, ci(zon, 2)
  sxx = ss1(ci(zon, 1), ci(zon, 2) - jdown + 1)
  sxy = ss2(ci(zon, 1), ci(zon, 2) - jdown + 1)
  syy = ss3(ci(zon, 1), ci(zon, 2) - jdown + 1)
  teta = -2 * ci(zon, 4)
  tm = sxy * COS(teta) - ((sxx - syy) / 2) * SIN(teta)

```

```

REM    tm = (((sxx - syy) / 2) ^ 2 + sxy ^ 2) ^ .5
        fdr = fdr + tm * ci(zon, 3)
NEXT zon
fos = ABS(fav / fdr) * red
IF fos < fs(ndt, 2) AND good = 1 THEN
    fos = INT(100 * fos) / 100
    fs(ndt, 2) = fos
    fs(ndt, 3) = x0
    fs(ndt, 4) = y0
    fs(ndt, 5) = r
    fs(ndt, 6) = fav
    fs(ndt, 7) = fdr
END IF
WEND
PRINT dt, fs(ndt, 2)
WRITE #3, fs(ndt, 1), fs(ndt, 2), fs(ndt, 3), fs(ndt, 4), fs(ndt, 5), fs(ndt, 6), fs(ndt, 7)
CLOSE #2
OPEN "L:/circle01.dat" FOR INPUT AS #2
INPUT #2, tem, tem, tem, tem, tem
WEND
CLOSE #1, #2, #3
END

```

C.9 QBASIC program developed to combine output files of the force-based safety factor

```

DIM f(14) AS SINGLE
OPEN "L:/fsf1-f8.dat" FOR INPUT AS #1
OPEN "L:/fsf-f8.out" FOR OUTPUT AS #2

```

```
INPUT #1, f(1)
WHILE f(1) < 7777
FOR i = 2 TO 7
INPUT #1, f(i)
NEXT i
WRITE #2, f(1), f(2), f(3), f(4), f(5), f(6), f(7)
INPUT #1, f(1)
WEND
CLOSE #1
OPEN "L:/ fsf2-f8.dat" FOR INPUT AS #1
INPUT #1, f(1)
WHILE f(1) < 7777
FOR i = 2 TO 7
INPUT #1, f(i)
NEXT i
WRITE #2, f(1), f(2), f(3), f(4), f(5), f(6), f(7)
INPUT #1, f(1)
WEND
CLOSE #1
OPEN "L:/ fsf3-f8.dat" FOR INPUT AS #1
INPUT #1, f(1)
WHILE f(1) < 7777
FOR i = 2 TO 7
INPUT #1, f(i)
NEXT i
WRITE #2, f(1), f(2), f(3), f(4), f(5), f(6), f(7)
INPUT #1, f(1)
WEND
CLOSE #1
CLOSE #2
END
```

C.10 QBASIC program developed to combine the output files of slip surfaces output

```

DIM f(14) AS SINGLE
OPEN "L:/circ1.dat" FOR INPUT AS #1
OPEN "L:/circ.out" FOR OUTPUT AS #2
INPUT #1, f(1)
WHILE f(1) < 7777
FOR i = 2 TO 14
INPUT #1, f(i)
NEXT i
WRITE #2, f(1), f(2), f(3), f(4), f(5), f(6), f(7), f(8), f(9), f(10), f(11), f(12), f(13), f(14)
INPUT #1, f(1)
WEND
WRITE #2, 777
CLOSE #1
OPEN "L:/circ2.dat" FOR INPUT AS #1
INPUT #1, f(1)
WHILE f(1) < 7777
FOR i = 2 TO 14
INPUT #1, f(i)
NEXT i
WRITE #2, f(1), f(2), f(3), f(4), f(5), f(6), f(7), f(8), f(9), f(10), f(11), f(12), f(13), f(14)
INPUT #1, f(1)
WEND
WRITE #2, 777
CLOSE #1

```

```

OPEN "L:/circ3.dat" FOR INPUT AS #1
INPUT #1, f(1)
WHILE f(1) < 7777
FOR i = 2 TO 14
INPUT #1, f(i)
NEXT i
WRITE #2, f(1), f(2), f(3), f(4), f(5), f(6), f(7), f(8), f(9), f(10), f(11), f(12), f(13), f(14)
INPUT #1, f(1)
WEND
WRITE #2, 777
CLOSE #1
OPEN "L:/defo.dat" FOR INPUT AS #1
INPUT #1, f(1)
WHILE f(1) < 7777
FOR i = 2 TO 5
INPUT #1, f(i)
NEXT i
WRITE #2, f(1), f(2), f(3), f(4), f(5)
INPUT #1, f(1)
WEND
WRITE #2, 777
CLOSE #1
CLOSE #2
END

```

C.11 FISH routine developed to show distribution of the accelerations or dynamic forces

```
def acc_dis2
```

```

loop i (1,igp)
  loop j (1,jgp)
    ex_1(i,j) = 0 ; x-force
    ex_2(i,j) = 0 ; y-force
    ex_3(i,j) = 0 ; x-acc
    ex_4(i,j) = 0 ; y-acc
  end_loop
end_loop
i=1
j=5
zn=9
zn2= (zn-1)/2
; while_stepping
loop while j < jgp-zn
  loop while i < igp-zn
    loop ix (i,i+zn-1)
      loop jy (j,j+zn-1)
        wz=wz - wazn(ix,jy)
ex_1(i+zn2,j+zn2)=ex_1(i+zn2,j+zn2)-accx(ix,jy)*(wazn(ix,jy)/9.81)
ex_2(i+zn2,j+zn2)=ex_2(i+zn2,j+zn2)-accy(ix,jy)*(wazn(ix,jy)/9.81)
      end_loop
    end_loop
    if wz#0 then
      ex_3(i+zn2,j+zn2)=ex_1(i+zn2,j+zn2) / wz
      ex_4(i+zn2,j+zn2)=ex_2(i+zn2,j+zn2) / wz
    end_if
    wz=0
    i=i+zn
  end_loop
  j=j+zn
i=1

```

```

end_loop
end
acc_dis2
plot state bloc ex_1,2 ma 200 b ; bou
plot pen state bloc ex_1,2 ma 200 b ; bou
set dxf xy xoff 0 xs 1 yoff 0 ys 1
set plot DXF
set output w8e15-f8.dxf
plot state bloc ex_3,4 ma .50 iw ; dxf waba3.dxf ; bou
plot ex_3,4 ma .5 dxf waba3.dxf w ; bou
pl dxf waba3.dxf w
set back b
movie size 880,660 file w8e15-2.dcx on step 37

```

C.12 FISH routine developed to calculate safety factor for tri-linear slip surfaces

```

def noncirc_sf
ch=0
sf=100
array a(10) b(10)
,,,,,,,,,,,,,; Dam Information ,,,,,,,,,,,,,,;
a(1)=-200.1
b(1)=91.4
a(2)=-103.2
b(2)=91.4
a(3)=-79.2
b(3)=97.4
a(4)=-18.6
b(4)=97.4

```

```

a(5)=-3
b(5)=102.6
a(6)=3
b(6)=102.6
a(7)=18.6
b(7)=97.4
a(8)=82.3
b(8)=97.4
a(9)=106.3
b(9)=91.4
a(10)=200
b(10)=91.4
,,,,,,,,,,,,, Slip Surface Information ,,,,,,,,,,
zabt=0
ox=-50 ;-63 ;-46.6
oy=180
ra1=102.6 ;124
ra2=105
nra=1
htl=80
zlef=30 ;28.8
zrig=45 ;30 ;47.1
,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,
loop 11 (0,nra)
zabt=zabt+1
ch=0
ra0 = ra1 + (11*((ra2-ra1)/nra))
xk1 = ox - (htl/2)
xk2 = ox + (htl/2)
yk1 = oy - ra0
xsad = -180

```



```

ykh = 100
ysad = 91
loop while ykh - ysad >= 0
ch=1.1
xsad = xsad + 0.10
if xsad < xk1 then
  zavr = tan(((180-zrig)*pi)/180)
  ykh = zavr*(xsad-xk1)+yk1
else
  if xsad <= xk2
    ykh = yk1
  end_if
end_if
if xsad > xk2
  zavrl = tan(((zlef)*pi)/180)
  ykh = zavrl*(xsad-xk2)+yk1
end_if
loop i (1,9)
j=i+1
if xsad >= a(i)
if xsad < a(j)
ysad=((b(j)-b(i))/(a(j)-a(i)))*xsad+((b(i)*a(j)-b(j)*a(i))/(a(j)-a(i)))
end_if
end_if
end_loop
end_loop
yk0 = ysad
xk0 = xsad
;,,,,,,,,,
xsad = 40
ykh = 100

```

```

ysad = 91
loop while ykh - ysad >= 0
ch=1.2
xsad = xsad - 0.1
if xsad < xk1 then
  zavl = tan(((180-zlef)*pi)/180)
  ykh = zavl*(xsad-xk1)+yk1
else
  if xsad <= xk2
    ykh = yk1
  end_if
end_if
if xsad > xk2
  zavr = tan(((zrig)*pi)/180)
  ykh = zavr*(xsad-xk2)+yk1
end_if
loop i (1,9)
j=i+1
if xsad >= a(i)
if xsad < a(j)
ysad=((b(j)-b(i))/(a(j)-a(i)))*xsad+((b(i)*a(j)-b(j)*a(i))/(a(j)-a(i)))
end_if
end_if
end_loop
end_loop
yk3 = ysad
xk3 = xsad
,,,,,,,,,,,,,
Sus = 0
tms=0
loop i (0,9)

```

```

ch=1.5
x`=xk0- ((i+0.5)*((xk0-xk1)/10))
y`=(((yk1-yk0)/(xk1-xk0))*x`+((yk0*xk1-yk1*xk0)/(xk1-xk0)))
loop ll (ileft,iright)
  loop m0 (jdown,jzones)
    if cohesion(ll,m0) # 0 then
p=m0+1
yp=(x`-x(ll,p))*(y(ll+1,p)-y(ll,p))/(x(ll+1,p)-x(ll,p))+y(ll,p)
      if y` <= yp
yl=(x`-x(ll,m0))*(y(ll+1,m0)-y(ll,m0))/(x(ll+1,m0)-x(ll,m0))+y(ll,m0)
      if y` > yl
p=ll+1
xp=(y`-y(p,m0))*(x(p,m0+1)-x(p,m0))/(y(p,m0+1)-y(p,m0))+x(p,m0)
      if x` <= xp
xl=(y`-y(ll,m0))*(x(ll,m0+1)-x(ll,m0))/(y(ll,m0+1)-y(ll,m0))+x(ll,m0)
      if x` > xl
sus=sus + cohesion(ll,m0)
      teta = -2 * zav1
tm = (sxy(ll,m0)*COS(teta))-(((sxx(ll,m0)-syy(ll,m0))/2)*SIN(teta))
tms = tms +tm
      end_if
      end_if
      end_if
      end_if
    end_if
  end_loop
end_loop
end_loop
end_loop
Sus1 = (((xk0-xk1)^2+(yk0-yk1)^2)^0.5)*Sus/10
tms1 = (((xk0-xk1)^2+(yk0-yk1)^2)^0.5)*tms/10

```

```

; Mcl1=Sus1*(((xk0-ox)*(yk1-yk0))-((yk0-oy)*(xk1-xk0))) ; (xk1-xk0)*(yk0-oy)-(xk0-
ox)*(yk1-yk0)
;;;;
Sus = 0
tms=0
loop i (0,9)
ch=1.6
x`=xk3- ((i+0.5)*((xk3-xk1)/10))
y`=(((yk1-yk3)/(xk2-xk3))*x`+((yk3*xk2-yk1*xk3)/(xk2-xk3)))
loop ll (ileft,iright)
loop m0 (jdown,jzones)
if cohesion(ll,m0) # 0 then
p=m0+1
yp=(x`-x(ll,p))*(y(ll+1,p)-y(ll,p))/(x(ll+1,p)-x(ll,p))+y(ll,p)
if y` <= yp
yl=(x`-x(ll,m0))*(y(ll+1,m0)-y(ll,m0))/(x(ll+1,m0)-x(ll,m0))+y(ll,m0)
if y` > yl
p=ll+1
xp=(y`-y(p,m0))*(x(p,m0+1)-x(p,m0))/(y(p,m0+1)-y(p,m0))+x(p,m0)
if x` <= xp
xl=(y`-y(ll,m0))*(x(ll,m0+1)-x(ll,m0))/(y(ll,m0+1)-y(ll,m0))+x(ll,m0)
if x` > xl
sus=sus + cohesion(ll,m0)
teta = -2 * zavr
tm = (sxy(ll,m0)*COS(teta))-(((sxx(ll,m0)-syy(ll,m0))/2)*SIN(teta))
tms = tms +tm
end_if
end_if
end_if
end_if
end_if

```

```

    end_loop
end_loop
end_loop
Sus2 = (((xk3-xk2)^2+(yk3-yk1)^2)^0.5)*Sus/10
tms2 = (((xk3-xk2)^2+(yk3-yk1)^2)^0.5)*tms/10
; Mcl2 = Mcl1 + Sus2*(((ox-xk3)*(yk1-yk3))-((oy-yk3)*(xk2-xk3))) ; (xk3-xk2)*(yk3-
oy)-(xk3-ox)*(yk3-yk1)
;;;
Sus = 0
tms=0
loop i (0,9)
ch=1.7
x` = xk3 - ((i+0.5)*((xk3-xk1)/10))
y` = yk1
loop ll (ileft,iright)
loop m0 (jdown,jzones)
if cohesion(ll,m0) # 0 then
p=m0+1
yp=(x`-x(ll,p))*(y(ll+1,p)-y(ll,p))/(x(ll+1,p)-x(ll,p))+y(ll,p)
if y` <= yp
yl=(x`-x(ll,m0))*(y(ll+1,m0)-y(ll,m0))/(x(ll+1,m0)-x(ll,m0))+y(ll,m0)
if y` > yl
p=ll+1
xp=(y`-y(p,m0))*(x(p,m0+1)-x(p,m0))/(y(p,m0+1)-y(p,m0))+x(p,m0)
if x` <= xp
xl=(y`-y(ll,m0))*(x(ll,m0+1)-x(ll,m0))/(y(ll,m0+1)-y(ll,m0))+x(ll,m0)
if x` > xl
sus=sus + cohesion(ll,m0)
tm = sxy(ll,m0)
tms = tms +tm
end_if

```

```

        end_if
    end_if
end_if
end_loop
end_loop
end_loop
Sus3 = (htl*Sus)/10
tms3 = (htl*tms)/10
; Mcl = Mcl2 + (Sus3 * (ra0 * htl))
sf0=-(sus1+sus2+sus3)/(tms1+tms2+tms3)
if sf > sf0
    sf= sf0
    ra = ra0
end_if
end_loop
end
noncirc_sf
pr sf                ; stress-based

```

D Vertical Displacements Graphs (Parametric Studies)

D.1 Responses to the Far-Field Motion

$0.5S_u$

$0.6S_u$

$0.67S_u$

1.33PGA

1.50PGA

$E = 500S_u$

$$E = 700S_u$$

$$E = 1200S_u$$

$$E = 1500S_u$$

$$E = 2000S_u$$

0.5% Damping

5% Damping

$\nu = 0.42$

$\nu = 0.48$

$0.5S_u$ & 2PGA

D.2 Responses to the Near-Field Motion

 $0.5S_u$ $0.57S_u$ $0.67S_u$

1.33PGA

1.50PGA

1.33PGA

1.50PGA

0.5% Damping

5% Damping

$\nu = 0.42$

$\nu = 0.48$

H+V Accelerations

E Results of Analysis (Parametric Studies)

E.1 Instantaneous Safety Factor for Zone 2 Under the Far-Field Motion

1.33PGA

1.5PGA

$E = 500S_u$

$$E = 2000S_u$$

0.5% Damping

5% Damping

E.2 Instantaneous Safety Factor for Zone 2 Under the Near-Field Motion

E.3 Equivalent Seismic Coefficient for the Far-Field Motion

E.4 Equivalent Seismic Coefficient for the Near-Field Motion

1.33PGA

1.5PGA

$E = 500S_u$

E.5 Slip surfaces' Safety Factor for Far-Field Motion

E.6 Slip surfaces' Safety Factor for the Near-Field Motion

F Results of Instantaneous Safety Factor for Case 1

The instantaneous safety factor corresponding to the far-field motion (M70R40#8, $p=0.0001$) and near-field motion (M55R16#7, $p=0.0004$) for Case 1 of soil profile are presented here. Analysis is performed based on the conventional stress method. The minimum overall safety factor for the far-field input motion is 1.08 and for the near-field 1.22, both of which are associated with the failure of the downstream crest. These results suggest the satisfactory stability of the dam with a material strength as low as Case 1. However, these results are based on stress method, which makes them lower-bounded by one. Figures F.1 and F.2 show the instantaneous safety factor related to Zone 3 during the earthquake.

Figure F.1 Variation of stress-based F_{min} of Zone 3 under the far-field motion

Figure F.2 Variation of stress-based F_{min} of Zone 3 under the near-field motion

The same analyses (for Case 1 soil profile) are performed by GEO-SLOPE, but using equivalent linear model instead of elasto-plastic model. Equivalent linear model of the Waba Dam is explained in Section 5.2. Figures F.3 to F.6 show the results of these analyses. Each of the safety factor curves associates with only a single slip surface, which yields the minimum overall safety factor. The associated slip surface at the critical time is shown next to each graph. The safety factor results are different from those of FLAC. Apart from the fact that they are related to a single slip surface, the main reason for discrepancy could lie in differences exist between equivalent linear method and elasto-plastic method. Another possible reason is the size of elements, which is larger in the QUAKE/W model that affects the transfer of high frequency components of s-wave throughout the model. The results of the overall minimum safety factor are summarized in Table F.1.

Figure F.3 Variation of the equivalent-linear-based F_t of Zone 3 under the far-field motion. $F_{tmin}=1.33$.

Figure F.4 Variation of the equivalent-linear-based F_t of Zone 4 under the far-field motion. $F_{tmin}=1.78$.

Figure F.5 Variation of the equivalent-linear-based F_t of Zone 3 under the near-field motion. $F_{tmin}=1.03$.

Figure F.6 Variation of the equivalent-linear-based F_t of Zone 4 under the near-field motion. $F_{min}=1.54$.

Table F.1 Summary of the overall safety factor for Case 1 based on both equivalent linear and nonlinear elasto-plastic stress-based methods

Motions	Equivalent linear By GEO-SLOPE F_{min}	Elasto- plastic (stress- based) F_{min}
Far-field	1.33	1.08
Near-field	1.03	1.22

The computed permanent settlement of the dam based on nonlinear elasto-plastic analysis is 10-cm and 38-cm for the near-field and the far-field input motions, respectively.

G Near-Field Motion with 2,500-Year Return Period

Some analyses have been performed with the application of the near-field input motion (M55R16#7) with the return period of 2,500 years, which is recommended in the new edition of the NBCC (2005). The results of the analyses are summarized in Table G.1. The overall safety factor for each set of parameters are slightly higher than that for Zone 1 and Zone 3 but slightly lower for Zone 2 for the near-field ground motion with 10,000-year return period. The possible reason for this observation is due to the rich high frequency content of ground motion and a closer proximity of the 10,000-year earthquake (epicentral distance of 10 km compared to 16 km in the 2,500-year earthquake).

Table G.1 Summary of F_{min} for the near-field ground motion of 2,500-year return period

Variable parameters	F_{min}			$(k_h)_{average}$
	Zone 1	Zone 2	Zone 3	Zone 2
Reference	1.53	1.52	2.46	0.047
1.5. a_{max}	1.48	1.38	2.36	0.058
H + Vertical	1.48	1.34	1.71	0.062
$E = 500.S_u$	1.82	1.79	2.47	0.027
$E = 1500.S_u$	1.46	1.24	2.09	0.075

The results of the maximum responses of the crest of the dam are summarized in Table G.2. The results of the permanent deformation for each set of parameters are mostly lower than those of the near-field ground motion with 10,000-year return period

Table G.2 Summary of the maximum responses of the crest of Waba Dam to the near-field input motion with 2,500-year return period

Variable parameters	Maximum Response of the Top of Crest			
	Dis _{Vertical} (cm)	Dis _{Horizontal} (cm)	Vel _{max} (cm/s)	Acc _{max} (m/s ²)
Reference	2.8	0.7	20	6.3
1.5. a_{max}	5.6	1.2	25	8.7
H + Vertical	11	6.0	18	8.2
E = 500. S_u	1.8	1.0	16	7.9
E = 1500. S_u	5.3	2.2	25	5.7

H Settlement-Based Safety Factor

Due to the fact that earthdams consisting of non-liquefiable soils, which are not vulnerable to loss of strength under seismic shakings, are not likely to suffer from landslide-type failures, one may suggest a new definition for the seismic safety factor, which is based of the permanent settlement rather than the slope stability. Regarding the strength reduction technique, the settlement-based safety factor, F_{sr} , may be given by Equation H.1

$$F_{sr} = 1 / (k_{sr})_{danger} \quad [H.1]$$

where $(k_{sr})_{danger}$ is the strength reduction factor, which leads to dangerous deformation of the dam under a given earthquake. It is common to define dangerously large deformations based on the freeboard of earthdams (or equal to 1.0-m). However in this approach, the concept of the dangerous deformation is different, and it is related to the natural performance of earthdams rather than to freeboard. The dangerous deformation is defined a deformation which is very sensitive to the variation of the strength of the soil. The magnitude of the dangerous deformation may vary for different dams based on their geometry. To illustrate this concept a series of elasto-plastic analyses have been conducted for a typical earthdam.

A typical earthdam is modeled using FLAC with characteristics given in Table H.1. The mesh of the dam model is shown in Figure H.1. The soil behaviour is considered elasto-plastic similar to the behaviour assumed for Waba Dam. The static safety factor of

the dam using the strength reduction technique is 2.04 (Figure H.2). The strength of the soil is reduced by the strength reduction factor, k_{sr} , ranging from 1.0 to 0.5. Corresponding to each k_{sr} , a static safety factor, SF_{sr} , based on the strength reduction technique can be considered. This safety factor, which is equal to 2.04 multiplied by k_{sr} , ranges from 2.04 to 1.02. For different k_{sr} s, the vertical displacement of the dam under the far-field and the near-field motions are obtained. Figures H.3 and H.4, respectively, show the deformed shape and the vertical displacement of the dam under the far-field input motion.

Table H.1 Soil properties and dam geometry

S_u (kPa)	ϕ	ρ (kN/m ³)	ν	E (kPa)	Slope (v : h)	Height (m)	Foundation Depth (m)
50	18	18	0.45	$1000S_u$	1 : 2	15	5

Figure H.1 Model of the dam (FLAC)

Figure H.2 $SF_{sr} = 2.04$ by using the strength reduction technique

Figure H.3 Deformation of the dam for $k_{sr} = 0.52$ under the far-field motion

Figure H.4 Vertical displacement of the crest (meter) vs. time (sec) for $k_{sr} = 0.52$ under the far-field motion

Figure H.5 shows the variation of the permanent vertical settlement with the static safety factor (SF_{sr}) for the far-field and the near-field ground motions. When SF_{sr} is reduced, the vertical settlement increases. The rate of settlement increases rapidly as SF_{sr} drops below about 1.25. As SF_{sr} becomes lower than 1.25, the rate of settlement significantly increases, and the settlement becomes more sensitive to the variation of the SF_{sr} . Therefore, the settlement corresponded to SF_{sr} less than 1.25 can be considered as dangerous deformation. However, when SF_{sr} is larger than 1.25, the rate of settlement is small. In this range, both the stress-based and the force-based methods yield the same

results for the instantaneous safety factor. It means that in this range, the generation of plastic zones is limited and permanent deformation is controlled by the shear strength of the soil.

Regarding the original input parameters of the dam for which SF_{sr} is 2.04, k_{sr} that results in a SF_{sr} equal to 1.25 will be 0.61 ($1.25/2.04$). Therefore, $(k_{sr})_{dangerous}$ can be considered equal to 0.61. Using Equation H.1, the seismic safety factor (F_{sr}) for the original dam will be equal to 1.64.

The range of SF_{sr} leading to the dangerous deformation may vary for earthdams based on their geometry and soil profile. However, since earthdams are generally constructed with static safety factors (SF_{sr}) above 1.50, F_{sr} might be always above one and the dangerous deformation may not occur to earthdams during earthquakes.

Figure H.5 Settlement vs. SF_{sr} under the far-field and the near-field motions

A set of graphs similar to the graph shown in Figure H.5 can be developed for the prediction of earthquake-induced settlement of clayey earthdams. The effects of most of the soil parameters are inherited in the static safety factor based on the strength reduction technique, SF_{sr} . The settlement can also be normalized to the height of the dam. To take into account the effects of other parameters such as dam geometry and ground motions, extensive parametric studies should be conducted. Depth of subsoils and slopes of dams are the main geometric parameters that should be studied. The impact of the earthquake magnitude and the hypocentral distance to the permanent settlement should also be studied.