COMPARISON OF PSEUDO-STATIC AND DYNAMIC RESPONSE ANALYSIS OF SIBILU DAM

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TABLE OF CONTENTS

Page

Acknowledgement

Abstract...1

1. Introduction..3

1.1. Background..3

1.1.1. General 3

1.1.2. Seismic hazards 3

1.1.3. Seismicity of Ethiopia 4

1.2. Purpose 5

1.3. Scope and Organization..6

2. Literature Review..7

2.1. Input Earthquake Motion..7

2.1.1. Deterministic approach 8

2.1.2. Probabilistic approach 8

2.2. Material Behavior during Strong Ground Shaking.........................9

2.2.1. Dynamic stress-strain behavior 10

2.2.2. Dynamic shear strength...11

2.2.3. Liquefaction 11

2.2.3.1. Liquefaction susceptibility..12

2.2.3.2. Initiation of liquefaction 14

2.2.3.3. Effects of liquefaction..15

2.3. Embankment Performance during an Earthquake 15

2.3.1. Field performance 16

2.3.2. Analytic models..17

2.3.2.1. Equivalent static forces approach 17

2.3.2.2. The response analysis 18

2.4. Stability Analysis 20

2.4.1. Limit equilibrium 20

2.4.2. Constitutive relations..22

2.5. Modes of Failure and Prevention Measures..................................23

2.5.1. Possible ways of failure...23

2.5.2. Protection measures..24

3. The Sibilu Dam..25

3.1. Location 25

3.2. Foundation Condition 25

3.3. Dam Type and Geometry...26

3.4. Material and Design Parameters 27

3.5. Purpose and Ownership 28

4. Earthquake Parameters 29

4.1. Conception...29

4.2. Data Collection 29

4.2.1. Historical seismicity data 29

4.2.2. Geologic data..30

4.3. Data Assessment..31

4.3.1. Analysis of historical seismicity data....................................32

4.3.2. Analysis of geologic data 33

4.4. Result 34

5. Pseudo-Static Slope Stability Analysis 36

5.1. Selection of Seismic Coefficient 36

5.1.1. Use of empirical coefficients..37

5.1.2. Rigid body response analysis 37

5.1.3. Elastic response analysis 38

5.1.4. Visco- elastic response analysis 38

5.1.5. Seismic coefficient for Sibilu Dam analysis 39

5.2. Pseudo-Static Analysis..40

5.2.1. General 40

5.2.2. The program RAEME 41

5.2.3. The Sibilu Dam model 42

5.3. Cases Analyzed and their Results...43

6. Dynamic Response Analysis..48

6.1. General 48

6.2. Dynamic Response Analysis...49

6.2.1. Linear response analysis...51

6.2.2. Equivalent linear response analysis....................................54

6.2.3. Response spectrum analysis 57

6.3. Evaluation of Embankment Performance 62

7. Conclusions and Recommendations..68

8. References...70

LIST OF TABLES

TABLES **PAGE**

1.1. Changes in Soil Properties With Shear Strain and Corresponding Models....10

3.1.Design Parameters Adopted 28

4.1. Computed and Allowable Safety Factors Obtained by Pseudo-Static Analysis 44

6.1. Results of the Response Analysis 59

6.2. Case1- at the End of Construction for Operating Basis Earthquake (0.09g) 66

6.3. Case2- at the End of Construction for Maximum Credible Earthquake (0.21g) 66

6.3. Case3- Steady State Seepage for Operating Basis Earthquake (0.09g) 66

6.5. Case4- Steady State Seepage for Maximum Credible Earthquake (0.21g) 67

6.6.Case5- Sudden Draw Down for Operating Basis Earthquake (0.09g) 67

6.7.Case6- Sudden Draw Down for Maximum Credible Earthquake (0.21g) 67

LIST OF FIGURES

FIGURES	PAGE
2.1. Methodology for Defining Seismic Risk at the Dam Site	7
2.2. Variation of Seismic Coefficient Along Height of the Dam; Equivalent Static Forces Constant versus Dynamic Variation	19
3.1. The Proposed Sibilu Dam Drawing, not to Scale	27
4.1. Earthquake Activity in the Vicinity of Sibilu Dam	33
5.1. Stability Analysis by Simplified Bishop Method	41
5.2. The Sibilu Dam Geometry Used in the Analysis, not to Scale	43
6.1. Variation of maximum Acceleration Ratio with Depth of Sliding Mass	48
6.2. Horizontally Layered One Dimensional System	51
6.3. Dam Section in Shear Beam Analysis	54
6.4. Shear Modulus and Damping Characteristics Used in the Response Calculation	54
6.5. Normalized Acceleration Response Spectra	55
6.6. Variation of Yield Acceleration with Average Normalized Displacement	60
6.7. Typical Dam Section	61

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ABSTRACT

Dynamic loads induced by earthquakes are often major factors in the design of earth dams. Earthquake induced stresses are major factors in determining the angles of the dam slopes and significantly influence the selection of materials, the zoning of the dam, and the construction method. Awareness about more rigorous seismic stability analysis is also growing during recent years.

In Ethiopia dams are being constructed for various purposes. The analysis and design of dams has been carried out by taking many factors in to account. Among these factors earthquake effect is considered mainly for major dams. One of these dams is the Sibilu Dam, which is under design for the Addis Ababa water supply. Until the present, in Ethiopia, seismic effect is commonly considered by the pseudo-static analysis method. But representation of the complex, transient and dynamic effects of earthquake shaking by constant pseudo-static forces is obviously not accurate. In this thesis an attempt was made to compare the pseudo-static and dynamic response analysis by taking the Sibilu Dam as a case study. The main purpose of the study is to show the shortcomings of the pseudo-static analysis method as compared to the dynamic response analysis when applied to the analysis of earth/ rock fill dams.

The thesis is organized as follows: initially a background is given on seismicity and seismic hazards in general and Ethiopia in particular. The second chapter covers summary of previous works related to this study. In the third chapter some description is given about the Sibilu Dam, the dam under the case study. In chapter four, the definition of the expected seismic excitation for the dam site is presented. The historical records of earthquake events are collected and geotechnical data of the dam site are reviewed

paying special attention to the location of active faults. The obtained data set is analyzed to give the required earthquake parameter for two different return periods.

In fifth chapter, the pseudo-static slope stability analysis is done with the conventional limit equilibrium method by using the design ground motion as an input. The analysis is performed for the upstream and down stream slopes of the dam by varying the possible critical cases. In chapter six, analysis and design is done based on permissible deformation. The dynamic response analysis is done by varying the material behavior. Non-linearity is considered by the Equivalent Linear Method. Finally, comparison of the different analysis methods is made and conclusions and recommendations are drawn.

CHAPTER-1 INTRODUCTION

1.1. Back ground:

1.1.1. General :

The seismic resistant analysis of an earthdam requires knowledge of seismology and earthquake engineering. Earthquakes are vibrations caused by movement of base rocks along fault surfaces. Most earthquakes occur when the energy stored by elastic deformation in the rocks on both sides of a fault is enough to rupture the rocks or to overcome the friction on an existing fault plane. The deformation is understood as being caused by internal forces such as of convectional, gravitational and magnetic origins.

The energy of the earthquake, generated at the fault is radiated outwards by means of elastic waves. As these waves travel through and along the crust of the earth they shake the earth in all directions with varying degree of intensity and the pattern of oscillation changes by refraction, reflection and superposition of one type of wave on others. Generally the magnitude of these waves decrease with distance.

The size of an earthquake depends on the amount of energy released. This can be measured by earthquake magnitude. The amount of energy released in turn can be related to the size of the geologic offset, fault parameters and to the consequences of the seismic hazard on people and their environment.

1.1.2. Seismic hazards:

The practice of earthquake engineering involves the identification and mitigation of seismic hazards. Fault movements, ground shaking and landslide can induce seismic hazard on embankment dams. These in turn result in deformation, liquefaction, slope instability and overtopping of the water of the dam.

The displacement of a fault running through the foundation of the dams is likely to result in severe damage or even collapse of the dam. Hence, detailed geological investigations are to be carried out to identify potentially active faults. Dams should be located at a safe distance from such features.

At a given site the major part of the ground motion during an earthquake is due to the upward propagation of body waves from an underlying rock formation. Among the different waves that are generated at a site shear waves are the major contributors to seismic hazard. The shear waves induce shear stress which may result in irreversible shear deformation of the soil skeleton or an increase in pore pressure. The extent of this deformation and excess pore pressure may dictate slope failure, liquefaction and movement of blocks of rock on the dam foundation and abutments.

The strength and duration of shaking at a particular site during earthquake depends on the size and location of earthquake and on the characteristics of the site. Soil deposits tend to act as filters to seismic waves by attenuating motion at certain frequencies and amplifying it at other frequencies. Strong earthquakes often cause landslides. Some earthquakes induced landslide resulting from liquefaction phenomena, but many others simply represent the failures of slopes that were marginally stable under static conditions. Seismic shaking may also trigger rock falls from potentially unstable slopes, water waves in the reservoir and hazards on the spillway, diversion weir, intakes and outlet structures.

1.1.3. Seismicity of Ethiopia:

The seismicity of Ethiopia and neighboring countries has been studied by Gouin (1979), Kebede (1991), Asfaw (1996) and others. Gouin (1979) compiled a catalogue of earthquake and produced the first seismic hazard map of Ethiopia. Several potentially damaging earthquakes have occurred. Examples are the 1906 Main Ethiopian rift earthquake (M=6.8), the 1961 Kara Kore (western margin of afar depression) earthquake sequence (M=6.6), and the 1969 Central Afar earthquake sequence (M=6.3).

To understand better the geographic distribution of earthquakes and the time for their occurrence in the region, review of plate tectonic is essential. The broad picture of the distribution of earthquakes in the horn of Africa would be determined by the relative motion at boundaries of plates in the immediate region. Which involves the collision process between Africa/ Arabia plate on one hand and Eurasia plate on the other.

During the collision Africa and Arabia plates move north, with the Arabia plate moving faster than Africa plate, generating tension and causing faulting and subsidence at the sites of the Red Sea and Gulf of Aden (Hempton, 1987). The sustained relative motion between Africa and Arabia plates resulted in the formation of the Gulf of Aden and the Red Sea. Later on the East African Rift System was formed.

The relative motion of Africa plates and Arabia plate is responsible for the seismicity of the Red Sea and the Gulf of Aden while the relative motion of Somalia plate and Africa plate is the main cause of seismicity in the East African rift system. In the Horn of Africa, all the three rift systems of the Red Sea, Gulf of Aden and East Africa rift systems meet in the Afar Depression forming a triple junction and a hot spot connecting the deep mantle to the crust.

1.2. Purpose of the Study

Dams are used for water supply, irrigation, hydropower and many other purposes. The failures of dams; not only have sever economic consequences, but also adversely affects the environment and quality of life. In addition, the populations are at risk in locations down streams of major dams.

Generally, the aseismic design of embankment dams is done either by the pseudo-static or by the dynamic response analysis methods. The pseudo-static method of analysis has many known deficiencies like uneconomical design due to the selection of maximum seismic coefficient at any depth for any mode that leads to conservative estimate of the inertia forces developed. Moreover, it predicts a safe condition for dams that are known to have had major slides, which threatens guarantee. Where as the dynamic response analysis incorporates the true dynamic behavior of the embankment. Therefore, this thesis has a specific purpose of comparing pseudo-static and dynamic response analysis by considering the Sibilu Dam as a case study.

1.3. Scope and Organization

The thesis describes the comparison of pseudo-static and dynamic response analysis of Sibilu Dam, which is under design for the Addis Ababa water supply purpose. The main body of the thesis is divided in to eight chapters each dealing with a different aspect of the analysis procedure. At first, a background is given on the seismic hazards in general and seismicity of Ethiopia. Chapter 2 is a literature review of related works. Chapter 3 describes some aspects of the dam under study. Then in the next chapter the characterization of earthquake excitation expected on the site is presented. In chapter 5, the expected performance of the dam for the input design ground motion is analyzed by the conventional pseudo-static analysis. Different safety factors are obtained by varying the critical cases to be considered. The sixth chapter covers application of Makdisi and Seeds' simplified procedure for the dynamic response analysis. Finally, conclusions and recommendations are made in chapter 7 and chapter 8 consists the list of reference materials.

2. LITERATURE REVIEW

2.1. Input Earthquake Motion

To evaluate the expected earthquake response of a dam under an expected earthquake, it is necessary to establish the intensity and frequency characteristics of earthquake motions to which the dam might be subjected. These properties depend on the seismicity of the dam site. The following methodology presented in the form of a flow chart may be adopted to determine input earthquake motion.

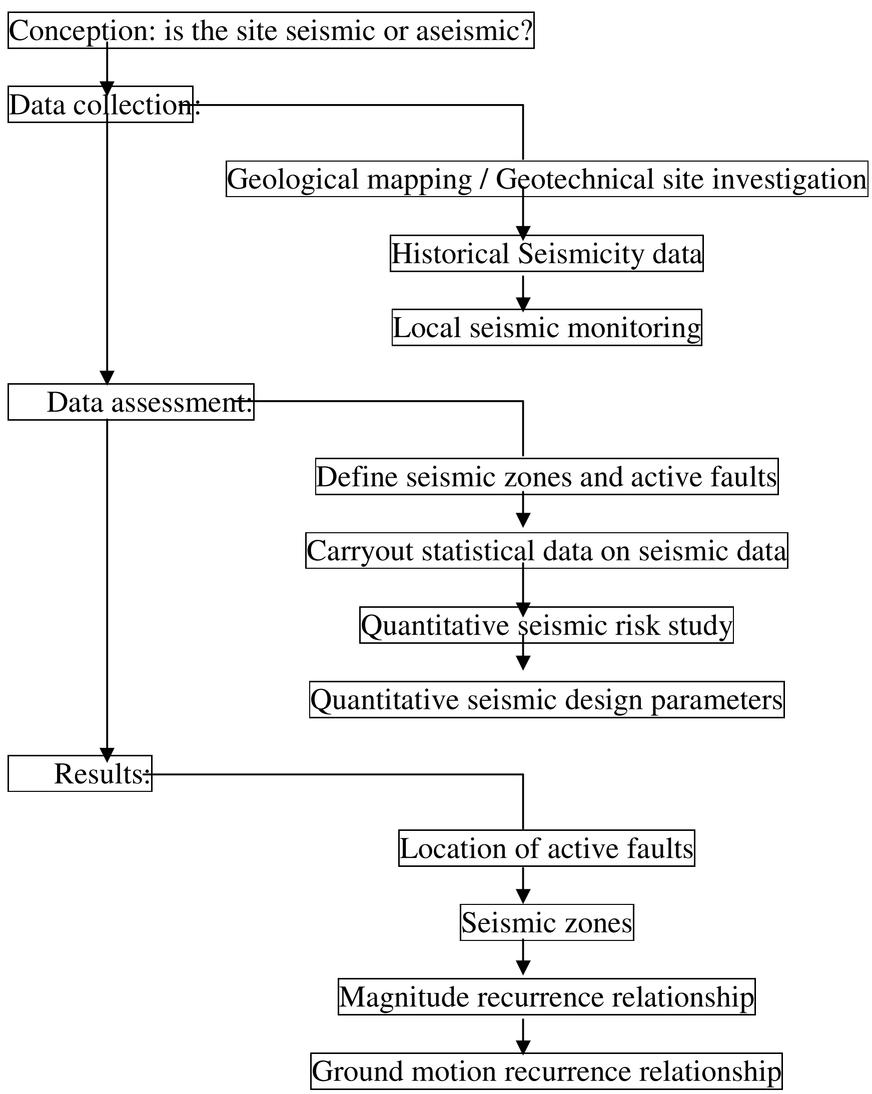


Figure 2.1. Methodology for defining seismic risk at the dam sites (Chaplow, 1981).

Basically there are two methods to determine the most probable design earthquake for a structure on a particular site. These are deterministic and probabilistic methods.

2.1.1. Deterministic approach

This method starts by defining the boundaries of seismotectonic region and the location of active faults. Secondly, a review of the seismic history of the area is made to locate the epicenter and magnitude of major earthquakes and to relate these epicenters to faults or seismotectonic regions.

Based on these results, a group of conceivable design earthquakes is postulated by selecting the most sever earthquake along each fault and in each seismotectonic region. These events are then moved along the fault and with in the seismotectonic region to a point closest to the site. Using applicable attenuation curves, the site magnitude is determined from the maximum postulated events and the maximum site magnitude is chosen as the design earthquake.

Although the deterministic procedure can be used to establish the design earthquake, it does not provide the likelihood of the occurrence of such an event. Therefore, in many cases, a probabilistic analysis is preferable to quantify the risk and to permit the selection of a design earthquake that is more representative of the overall risk embodied in the design of the structure.

2.1.2. Probabilistic approach

In this approach, as in the deterministic one, the seismotectonic regions comprising the site area, as well as capable faults are established first. Next, the available earthquake data are utilized to obtain the recurrence rates (statistical study) for events in each seismotectonic region and along each potential fault. Based on these recurrence rates and the applicable attenuation characteristics, the contributions from several seismotectonic regions in the vicinity of the site are considered to obtain the return period of an event with a certain magnitude.

The uncertainties in input parameters and the need for sensitivity studies ensure that ultimately a high level of engineering judgement must be incorporated in to all earthquake studies. Such earthquake studies are therefore regarded as a means of comparing earthquake between different sites rather than being a deterministic method of establishing absolute value.

The probabilistic analysis rests on two basic assumptions. First, the seismic activity within any seismotectonic region or along capable faults has a uniform spatial distribution. Second, the recurrence rates for earthquakes of a given intensity in any seismotectonic region remains the same as observed in the past. Since the projected life of a dam is very small compared to geologic time, it is reasonable to assume that earthquakes will occur in the near future with the same recurrence rate as they have in the past.

2.2. Material Behavior During Strong Ground Shaking

Once the seismic hazard level of a given site is established, another important aspect to be considered is the dynamic behavior of the material used for dam construction. The main mechanical properties that are to be considered under cyclic loading are dynamic stress- strain relationships and dynamic shear strength. These again depend on soil properties such as: dynamic moduli (stiffness), damping, poison's ratio, density, etc. of which the first two are the most important. Some of these soil parameters are best measured or studied in the field, others are obtained in the laboratory and some can be measured both in the laboratory and in-situ.

2.2.1. Dynamic stress-strain behavior

It is very important to characterize the most important aspects of cyclic stress-strain behavior as accurately as possible with simple rational models. The point at which the conflicting requirements of simplicity and accuracy are balanced depends on many factors, the key one being cyclic shear strain amplitude. Many combinations of stress-strain models have been proposed. The shear strain ranges of practical interest are illustrated in table 1.1.

Table1.1 Changes in soil properties with shear strain and corresponding models. (Kramer, 1996)

Shear strain	Small strain	Medium strain	Large strain	Failure strain
Elastic				
Elasto-plastic				
Failure				
Stress-strain model	Linear elastic model	Visco-elastic model	Load history tracing type model	

2.2.2. Dynamic shear strength:

The shear strength of an element of soil is typically defined as the shear stress mobilized at the point of failure. In the field failure is usually associated with deformations that exceed some serviceability limit. Since deformation results from the integration of strains over some volume of soil, the point of failure of an element of soil is often defined in terms of a limiting strain.

Basically, there are two ways to measure the strength of a cyclically loaded soil. The 'cyclic strength' which is based on a limiting value of cyclic and/ or average strain during cyclic loading. The 'monotonic strength', which is the ultimate static strength that can be mobilized after cyclic loading has ended. As pointed out by Castro and Christian (1976), the ultimate (residual, high strain) undrained shear strength of a saturated soil is controlled by its void ratio and soil structure.

Makdisi and Seed (1978) defined the dynamic yield strength of soils that exhibit small changes in pore pressure under undrained loading as 80% of the undrained strength of the soil. The yield strength is defined as the maximum stress level below which the material exhibits a near elastic behavior. The dynamic shear strength of cohesionless soils is strongly related to the phenomenon of liquefaction.

2.2.3. Liquefaction

A loose saturated sand and as observed from recent studies other soil formations like silt deposits, tends to compact and decrease in volume when subjected to vibration. If drainage is unable to occur, the pore water pressure increases. If the pore water pressure in the cohesionless deposit is allowed to build up by continuos vibration, a condition will be reached where the over burden pressure will be equal to the pore water pressure.

Based on effective stress principle, $\sigma' = \sigma - u$

Where σ' is the effective stress, σ is the total over burden pressure, and u is the pore water pressure. If σ is equal to u, σ' is zero. Under this condition, the soil doesn't posses any shear strength, and it changes into a liquefied state.

Liquefaction phenomena that result from this process can be divided into two main groups; Flow liquefaction which occur when the shear stress required for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state. And cyclic mobility that occurs when the static shear stress is less than the shear strength of the liquefied soil.

The three most critical aspects of liquefaction hazard evaluation are; *susceptibility, initiation, and effects, i.e.*

1. Is the soil susceptible to liquefaction?
2. If the soil is susceptible, will liquefaction be triggered?
3. If liquefaction is triggered, will damage occur?

2.2.3.1. Liquefaction susceptibility

Liquefaction susceptibility can be judged from historical, geologic, compositional, and state criteria.

i. Historical criteria:

Liquefaction case histories can be used to identify specific sites, or more general site conditions, that may be susceptible to liquefaction in future earthquakes. Youd (1991) described a number of instances where historical evidence of liquefaction has been used to map liquefaction susceptibility. Post earthquake field investigations have also shown that liquefaction effects have been confined to a zone within a particular distance of the seismic source.

ii. Geologic criteria:

Geologic processes that sorts soils into uniform grain size distributions and deposit them in loose states produce soil deposits with high liquefaction susceptibility. Consequently river, gravity and wind deposits when saturated are likely to be susceptible to liquefaction. The susceptibility of older soil deposits to liquefaction is generally lower than that of newer deposits.

iii. Compositional criteria:

Compositional characteristics associated with high volume change potential tend to be associated with high liquefaction susceptibility. These characteristics include particle size, shape, and gradation.

For many years, liquefaction-related phenomena were thought to be limited to sands. Recently, Liquefaction of non-plastic silts has been observed (Ishihara, 1984). Coarse silts with bulky particle shape, which are non-plastic and cohesionless, are fully susceptible to liquefaction (Ishihara, 1993).

Well-graded soils are generally less susceptible to liquefaction than poorly graded soils.

Soils with rounded particle shapes are known to densify more easily than soils with angular grains. Consequently, they are usually more susceptible to liquefaction than angular-grained soils.

iv. State criteria:

Since the tendency to generate excess pore pressure of a particular soil is strongly influenced by both density and initial stress conditions, liquefaction susceptibility depends strongly on the initial state of the soil.

Casrto (1969) showed a unique relationship between void ratio and effective confining pressure at large strains. The locus of points describing this relationship, named steady-state line (SSL) (Kramer, 1996) can be used as the boundary for liquefaction susceptibility. A soil whose initial state lies above the SSL will be susceptible to flow liquefaction only if the static shear stress exceeds its steady state (or residual) strength. Cyclic mobility, on the other hand, can occur in soils whose state plot above or below the SSL. In other words, cyclic mobility can occur in both loose and dense soils.

2.2.3.2. Initiation of liquefaction

The occurrence of liquefaction requires a disturbance that is strong enough to initiate, or trigger it. Evaluation of the nature of that disturbance is one of the most critical parts of a liquefaction hazard evaluation.

A number of approaches have been developed over the years to evaluate the potential for initiation of liquefaction. The most common of these is cyclic stress approach, the cyclic stress approach is conceptually quite simple: the earthquake induced loading, expressed in terms of cyclic shear stresses, is compared with the liquefaction resistance of the soil which is also expressed in terms of cyclic shear stresses. At locations where the loading exceeds the resistance, liquefaction is expected to occur.

2.2.3.3. Effects of liquefaction

Liquefaction phenomena can affect dams in many different ways. Liquefaction can also influence the nature of ground surface motions. Flow liquefaction can produce massive flow slides and contribute to the failure of dams. Cyclic mobility causes slumping of slopes, settlement of crest and cracks. If damage appears likely and the anticipated level of damage is unacceptable, the site must be abandoned or improved or on site structures strengthened. Substantial ground oscillation, ground surface settlement, sand boils and post earthquake stability failures can develop at level ground sites. The effects of liquefaction can be better appreciated by studying well-documented case histories.

After the material properties of a dam are well established, how a dam behaves when vibratory earthquake load acts on it is studied by examining the embankment's performance. These include actual field performance of constructed dams and predicted performance of analytical models.

2.3. Embankment Performance during an Earthquake

Performance of an embankment dam during an earthquake could be studied by making observation of its field performance or by using analytical models. This aspect deals with modal characteristics, amplification theories and energy dissipation of the dam structure.

2.3.1. Field performance

The study of the performance of embankments during strong earthquake have shown two distinct types of behaviors; these are:

1. The behavior associated with loose to medium dense sandy embankments, susceptible to rapid increases in pore pressure due to cyclic loading. This in turn is associated with reductions in shear strength and potentially large movements leading to almost complete failure.
2. The behavior associated with compacted cohesive clays, dry and dense sands. Here the potential for build up of pore pressure is much less. The resulting cyclic strains are usually quite small, and the material retains most of its static undrained shearing resistance so that the resulting post earthquake behavior is limited permanent deformation of the embankment.

The comprehensive summary of the field performance of earth dams complied for over 20 years is presented by Professor H. B. Seed in 1978. It is reproduced below:

1. Virtually any well built dam on a firm foundation can withstand moderate earthquake shaking, say with a peak acceleration of about 0.2g from earthquakes with magnitudes up to about 7, with no detrimental effects.
2. Dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking ranging from 0.35g to 0.8g from a magnitude 8.25 earthquake with no apparent damage.
3. For dams constructed of or on loose or medium dense saturated cohesionless soils and subjected to strong shaking, a primary cause of damage or failure is the build–up of pore water pressures in the cohesionless soils and the possible loss of most of its strength which may result from this pore pressure increase.
4. In many rock fill dams the rate of pore pressure dissipation is so great that pore pressures could never build up to any significant extent during an earthquake.

2.3.2. Analytic models

The seismic performance of embankment dams is also evaluated by using an analytical model of the dam based on numerical techniques. Theoretical analysis as a shear beam, the Finite Element Method and model tests are some of the methods used for analyzing dynamic behavior of embankments. Both methods of analysis are coupled with an appropriate characterization of the earthquake. In general, the evaluation is performed by assuming ‘equivalent’ static forces or by using dynamic response analysis.

2.3.2.1. The "equivalent" static forces approach

Before the advancement of the knowledge of earthquake response, seismic effects on dams were characterized in terms of "equivalent" static forces. The amplification of acceleration through response of the dam was assumed to be either negligible or improbable. Equivalent static forces for seismic conditions were simply added to forces determined for true static loading conditions. The analytical results for combined loading conditions including earthquake effects were not evaluated any differently from the one for static loading conditions.

The main limitation of this approach is:

The maximum acceleration will be developed in an embankment for only a short period of time, with the effect not equivalent to those produced by applying inertial static force that is as if it were acting for an unlimited period of time.

2.3.2.2. The response analysis

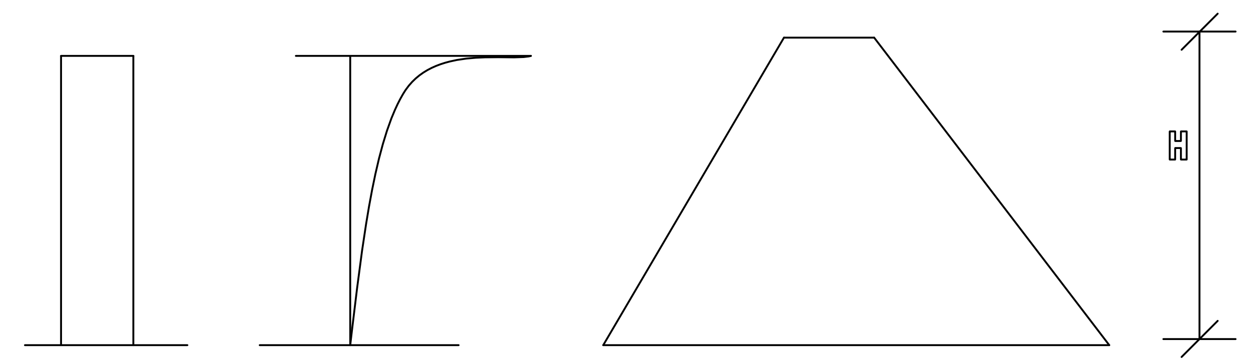
As the knowledge of dynamic response analysis was developed, it was realized that substantial amplitudes of earthquake ground motions could occur at frequencies well within the frequency range of response for dams, and the resulting amplification should not be ignored. The dynamic response behavior of embankment dams was recognized as a key factor in correctly understanding and evaluating the seismic performance of dams. Recognition of this fact leads to more appropriate methods of analysis by using linear or non-linear methods of analysis.

i. Linear response analysis

Recently the elastic properties as well as the mass of the dam materials have been incorporated into the mathematical models, and as a result the vibration properties of dams (the modeshape and frequencies) have been identified to have a controlling influence on the earthquake response behavior. For linearly elastic structures the dynamic aspect of the response is indicated by the response spectra and by the dynamic displacement patterns, which are conveniently expressed in terms of free vibration mode shapes.

In linear analysis earthquake forces may be expressed as the product of a seismic coefficient and the unit weight of the dam material. The seismic coefficient depends on the earthquake response spectrum, the vibration period of the mode, and the shape of the mode. The seismic coefficient associated with force in the fundamental mode of the dam varies with height, as shown in figure2.2.

One of the results of assuming a height wise uniform seismic coefficient is that the biggest calculated forces in dams are found to be at the base of the dam. This misleads the concept of amplification towards the top.



'Equivalent' static forces approach

Fundamental vibration mode

Embankment dam

Figure 2.2. Variation of seismic coefficient along height of a dam: 'Equivalent' static forces approach, constant versus dynamic variation (Housner, 1990).

ii. Non- linear analysis

Although linear response analysis provide great insight into the earthquake performance of dams, it is evident that a rigorous estimate of the seismic safety of a dam can be obtained only by a non-linear analysis, especially when big earthquake force is expected. For important structures and for big intensity earthquakes, it is recommended that full non-linear dynamic analysis be carried out.

A non- linear response analysis requires considerably greater computational effort than a linear analysis. Here a much more detailed analytical modeling is required to express the non-linear response mechanisms. However, the greatest impediment to effective non- linear analysis at present is not the computational procedures, it is the lack of adequate knowledge about the non- linear properties of the dam material. It is well known that the mechanical properties of dam material vary with time. Both the strength and the stiffness vary with the rates at which the load is applied. Thus, the properties associated with earthquake damage may be quite different from those measured by typical slow speed laboratory tests. It is essential that major dynamic soil testing be carried out to determine material properties suitable for use in non-linear seismic safety evaluations.

This predicted earthquake performance by linear and non-linear (damage) analysis is checked against a suitable performance criteria that is set to provide adequate safety. These are different factor of safeties or permissible deformations for the linear and non-linear analysis range. These criteria reflect both the desired level of safety and the nature of the design and analysis procedures. In addition, the criteria depend on recurrence interval of the expected earthquake input, dynamic strength parameters and ultimate failure mechanisms. This is done by the stability analysis.

2.4. Stability Analysis

In order to analyze the stability of an earthdam during an earthquake, the following approaches may be adopted; limit equilibrium and constitutive relations.

2.4.1. Limit equilibrium

Generally, the method ignores the deformability characteristics of the material and concentrates on the collapse mechanism. Here, only the strength properties need to be considered, and with a good estimate of these, reasonably accurate predictions of complete collapse can be found. This is done in order to consider various forms of failure, and to consider the equilibrium of the external loads and internal forces acting at the moment of failure. Once the forces are established then the dam is proportioned so that these loads are greater by some factor than the loads that will act on the structure. In terms of the limiting equilibrium principle, a factor of safety less than one represent failure.

However, under earthquake conditions, it may be possible to allow the factor of safety to drop below one, as this state will exist only for a short time. There are two possible ways by which the factor of safety may be reduced to one or below one during an earthquake. These are the increase of inertia forces and decrease of total strength due to pore water pressure rise. In either case, when the dynamic load exceeds the resistance the dam suffers deformation.

New mark (1965) proposed the model of a sliding block to compute the displacement of the failed mass. Goodman and Seed (1966) showed that this model gives reasonable results for dry cohesionless soil. Finn and Miller (1973) used a finite element model to compute the displacement. As pointed out by Seed (1978), the displacement depends on determining the critical acceleration as accurately as possible.

Therefore, in the design of earth dams and embankments under earthquake loading condition, displacement criteria offer a better criterion than a factor of safety on shear strength. The displacement depends on shear strength, inertia forces and pore pressures generated during the earthquake.

2.4.2. Constitutive relations

In the dynamic problem, like earthquake analysis, it is necessary to adopt constitutive methods because of the following reasons:

1. The loading is itself dependent on the deformations and cannot be predicted beforehand.
2. The material strength is often dependent on the rate of strain and on its deformation history
3. The duration of loading is often so short that even if failure occur the total deformation of the dam may be insignificant.

In order to apply the constitutive relation based analysis the following are necessary:

1. A suitable modeling of the material behavior.
2. Establishing the general behavior pattern and governing differential equations of the problems.
3. Establishing the appropriate numerical discretization process and their possible computer solutions.

It is difficult to model and derive the governing differential equation even under static loads. The situation becomes more complex when fluctuating loads occur, such as may be expected in the earthquake response of earthdam. Here two new phenomena are encountered. Referring to drained behavior of the soil we note that;

i. Permanent shear strains occur after each cycle of stress application
ii. Permanent volume contraction (densification) occurs after each cycle of loading (except for extreme dense materials).

From the two effects, the second one is of greater importance in practice where saturated undrained or partially drained behavior predominates. It is this densification phenomenon that accounts for the large displacements occurring when the pore pressures build up to the value of the mean effective compressive stress. When this happens incipient liquefaction and loss of strength is reached and the material is at a point of yielding. Some dilatancy that occurs in sands during plastic flow counterbalances at this stage the pore pressure increase phenomena and complete liquefaction follows only when such dilatancy is exhausted after a considerable shearing strain.

2.5. Modes of Failures and Prevention Measures

2.5.1. Possible ways of failures

Possible ways in which an earthquake may cause the failure of embankment dams are given below.

a) Disruption of the dam by major fault movements in the foundation
b) Loss of freeboard due to differential tectonic ground movements
c) Slope failures induced by ground motions
d) Loss of freeboard due to slope failures or soil compaction
e) Sliding of the dam on weak foundation materials
f) Piping failure through cracks induced by ground motions

g) Overtopping of the dam due to water waves in the reservoir

h) Overtopping of the dam due to earth or rock slides in the reservoir

i) Failure of spillway or outlet structures

2.5.2. Protection measures

Applying simple protection measures indicated below can solve many of the potential problems. Protective measures of embankment dams against earthquake shaking are:

a) Allow ample freeboard in order to reduce the detrimental effects of settlement, slumping or fault movements

b) Use wide transition zones of material not vulnerable to cracking

c) Use chimney drains near the central portion of the embankment

d) Provide ample drainage zones to allow for a possible flow of water through the cracks

e) Use wide core zones of plastic materials not vulnerable to cracking

f) Use a well graded filter zone upstream from the core to serve as a crack stopper

g) Provide crest details which will prevent erosion in the event of overtopping

h) Flatten the embankment core at the abutment

i) Locate the core in order to minimize the degree of saturation of the materials

j) Stabilize slopes around the reservoir rim to prevent slides in to the reservoir

k) Provide special details, should there be danger of fault movements in the foundation

Most of the failure modes and protection measures listed above are qualitative methods, which defy theoretical analyses. But quantitative checkups can be made for limited cases like slope sliding and crest settlement. Commonly, the upstream and downstream dam slope failure is checked, not because it is amenable to theoretical analysis but also because it is the predominant type of failure. In Ethiopia there are some dams under seismic risk. Hence, seismic safety of important projects like Sibilu Dam, under design for the Addis Ababa water supply purpose need to be analyzed for seismic safety.

3. THE SIBILU DAM

3.1. Location

Sibilu dam is found 45km north of Addis Ababa, near Chancho town on the Addis Ababa Bahir Dar Road. It is near the northwest edge of the seismically active Ethiopian Rift Valley. The dam site is located north of the rift margin in the western highlands.

3.2. Foundation Condition:

The geotechnical investigations carried out at the Sibilu Dam site includes geological reconnaissance and mapping, geophysical surveys, boreholes, testpits and augerholes (hand), all completed within and adjacent to the proposed footprint of the dam, along the proposed spillway and within several potential borrow areas. Where possible packer permeability testing and peizometric installations were also carried out to assess the insitu hydraulic conductivity of the undisturbed natural soils in the area.

Analyzing the results, the geologic units within the Sibilu area consists of quaternary sediments of alluvial deposit and residual soil overlying tertiary rocks mainly basalt. The soil thickness ranges from 2-2.5m throughout the valley bottom, and it has highly plastic consistency. The bedrock is basalt with different texture and joints with secondary infillings of predominantly quartz. The results of field permeability testing show the requirement of grout curtain.

From the conditions found on site, the following foundation treatment works have been recommended at the pre-design phase. The site will be stripped, with all clayey soils and weathered material removed to expose sound bedrock. At the abutments, steeper slopes will be shaped to provide a better transition for the dam section and reduce the susceptibility of cracking. In the core contact zone, the exposed bedrock surface will be cleaned of all loose material with all joints and openings cleaned out and slush grouted. In addition, all cavities, depressions and irregularities either existing or created due to the removal of rock fragments will be filled with low strength dental concrete to create a suitable surface for fill placement. Grouting will also be required beneath the core zone, near or immediately upstream of the dam centerline to a depth of approximately 15m from the natural ground.

3.3. Dam Type and Geometry:

The proposed dam is an earth and rock fill type with central clay core. It consists of rock fill as a shell material, central impermeable clay core and a downstream sand and graded material transition filter. For economic reason, much of the dam body is the clay core due to the abundance of clay in the vicinity of the dam site. It has a height of 32m, upstream slope of 1V: 1.5H and downstream slope of 1V: 2H.

A cofferdam consisting of a down stream rock fill zone with a slope of 1H: 1V, and an upstream earth fill zone in slope of 3.5H: 1V constructed during initial stages of foundation preparation. The filter materials will be placed to form upstream and down stream slopes of 1H: 1.3V.

Based on storage and yield requirement the specifications for the embankment are shown here in Fig 3.1.

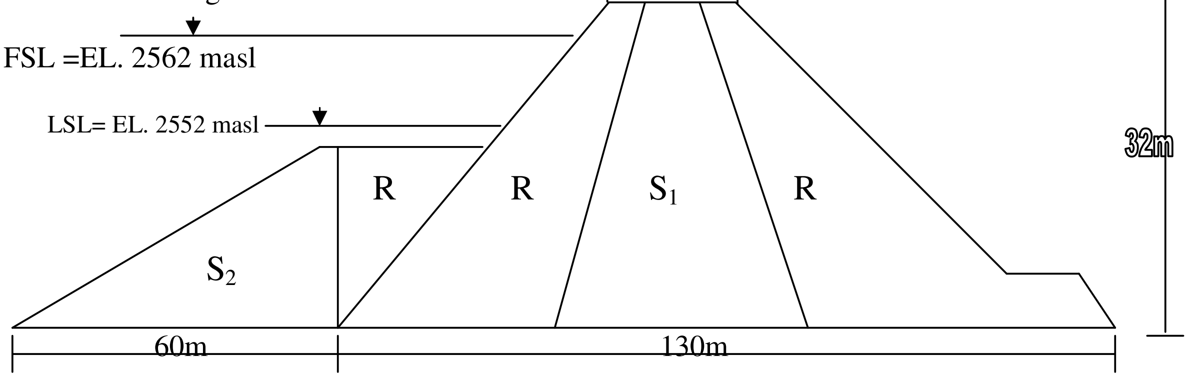


Figure 3.1. The Proposed Sibilu Dam X- section.

Where:
S_1= impervious clay core

S_2= random excavation

R= rock fill shell

3.4. Material and Design Parameter:

The clay material available in the vicinity of the dam will provide a highly impermeable fill for construction of the dam core. The clay core is of low shear strength and of highly plastic consistency. Due to the high clay content of the proposed clay core fill, several filter zones or a multi stable filter will be required to prevent piping in to the rock fill shells. Different laboratory and field tests have been carried out to estimate shear strength parameters.

The results are summarized in the following table. (AGRA, 1997)

Material	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Angle of internal friction (degrees)
Clay core	18	20	15
Rock fill	23	0	35

Table 3.1. Soil -design parameters adopted

As described earlier, the overburden soil on the valley bottom is suggested to be removed to reach to the rock foundation.

3.5. Purpose and Ownership:

The Sibilu Dam is to be constructed for Addis Ababa water supply purpose. Hence, Addis Ababa Water Supply and Sewerage Authority owns the project. As the failures of Sibilu Dam causes not only sever economic consequences but also adversely affect the environment and quality of life, it is mandatory to make a proper earthquake resistance design. The purpose of which is to design the dam that can withstand a certain level of shaking (the design ground motion) without excessive damage.

4.0. EARTHQUAKE PARAMETERS

4.1. Conception:

The first step in the seismicity study of a dam site is to define whether seismic loading of the structures must be incorporated in to the design or not. The usual basis for this initial assessment is the map of seismic activity. The absence of any record of an earthquake with in 400km of the proposed site is regarded as sufficient justification for regarding it as aseismic (Sarma, 1975). The presence of earthquake record with in limited distance indicates that the Sibilu Dam site is seismic.

4.2. Data Collection:

The scope of data collection depends on the status of the investigation (whether it is feasibility or project design), on the scale of proposed structures and upon the assessment of likely seismic risk. Data collection commences with inquiring if an appropriate earthquake code exists. Until now the available code to be referred to is the Ethiopian Building Code of Standards.

4.2.1. Historical seismicity data

Collection of historical seismicity data around the site is carried out (Jeffery, 1997). Lists of teleseismic events recorded around the site are obtained from either the international seismological center or records from Ethiopia. The information available from teleseismic and historical records on the frequency and distribution of earthquakes around a dam site is often very incomplete. The only feasible way, in which such information can be supplemented with in the time scale of a dam site investigation is to install and operate a local network of seismic monitoring stations. But such observation has cost consequences and was not done.

Epicenters of earthquake with in 500 km of Addis Ababa from the National Oceanic and Atmospheric Administration (NOAA) catalogue are collected (Jeffery, 1997). Siliceous and basaltic volcanoes of Quaternary age and the boundaries of the East African Rift Valley were referred from map (Kazmin, 1972). The volcano locations and the Rift Valley boundaries were based on information on the1: 2,000,000-scale geological map of Ethiopia (Kazmin, 1972). All of the siliceous volcanoes and most of the earthquake epicenters appear to be associated with the Northeast- trending Rift Valley of Ethiopia.

Evaluation of earthquake activity for the Sibilu Dam site was based on the earthquake catalogue obtained from the National Geographical Data Center that is part of the NOAA in Boulder, Colorado, USA. This catalogue consisted of all M> 4 earthquakes with in 500 km of a point located north of Addis Ababa (9.15^0N Latitude, 38.6^0E Longitude). The NOAA database was also searched for earthquakes of all magnitudes with in 200-km radius of Addis Ababa.

4.2.2. Geologic data

Regional geology and earthquake source

Geological mapping and the implementation of geotechnical site investigation proceed routinely as part of the preliminary works at the dam site in seismic areas. These are done with particular attention to such aspects as the location of active faults and to identifying those aspects of ground conditions which may be affected by, or themselves affect, seismic shaking.

Published geologic maps were reviewed and a stereoscopic aerial photograph of the region around the Sibilu Dam was examined. Kebede Thehayu (Jeffery, 1997) conducted a systematic lineament analysis. Limited trenching was performed to evaluate a lineament in the Sibilu reservoir area. Evidence of faulting was studied to indicate that the most recent movement was pre-quaternary.

4.3. Data Assessment:

This stage of defining input earthquake motion includes both the establishments of the seismic forces likely to act at the site and the site response. The assessment of both aspects is invariably based upon incomplete data, particularly in relation to the range of possible interpretations of geotechnical conditions, ground response characteristics and earthquake frequency and distributions.

Magnitude recurrence relationship based on incomplete and inhomogeneous teleseismic records and the empirical attenuation relationships are regarded as two input parameters subject to significant uncertainty. Therefore, it is always considered essential to check earthquake assessments for a range of these parameters.

Depending on the variability of the character of seismic sources and in the amount and kinds of data available, there are different methods used to scale earthquake size. Among these historical seismicity method is adopted below because it is best fit to the available data.

4.3.1. Analysis on historical seismicity data

The earthquake catalogue within 200 km of Addis Ababa was analyzed by accumulating the number of earthquakes interval of 0.1 magnitudes each. The logarithm of the cumulative number was taken and divided by the number of years in the record. The earliest earthquake in the catalogue within 200 km of Addis Ababa occurred in 1841. The catalogue is considered to be complete through 1995, making the number of years in the record 154. The completeness of the catalogue for moderate and small magnitude earthquakes before 1968 is doubtful.

Figure 4.1, is a plot of earthquake activity with in 200 km of the Sibilu Dam site. Regression analysis was performed on the data for magnitudes 5.9 and higher earthquakes. The equation describing the earthquake activity is

$$\text{Log } N = 3 - 0.7247M \ldots \quad 4.1 \text{ (Jeffery, 1997)}$$

Where N is the cumulative number and M is earthquake magnitude. This equation produces a straight line on semi-log paper as shown in figure 4.1.

The East Africa Rift has some similarities to the Rio Grande rift in the Western United States where the maximum earthquake considered possible is magnitude 7.5 (Jeffery, 1997). This was the maximum magnitude considered reasonable for the region around the Sibilu Dam site. The non-linear equation for earthquake activity is

$$AF = 0.053 * 10^{-0.724/(M-5.9)-0.00367} \ldots \quad 4.2 \text{ (Jeffery, 1997)}$$

Where AF is annual frequency and M is earthquake magnitude.

A circle with a radius of 200km has an area of 125,664 km^2. The earthquake activity can be normalized to an area of 1000 km^2 by dividing AF in equation 4.2 above by 125,664. The line for the normalized curve is shown in figure 4.1 and indicates that an earthquake of magnitude 5 should be expected on average once every 500 years with in an area of 1000km^2. Similarly, an earthquake of magnitude 6.6 should be expected on average once every 10,000 years. The radius of a circle with an area of 1000km^2 is approximately 17.8km. Therefore, this distance was used in attenuation of relationships for estimating ground motion as the site. Since any single technique of the magnitude estimation approach is subject to some uncertainty, the above analysis is supplemented with source characterization approach.

Fig. 4.1 Earthquake activity in the vicinity of the sibilu dam site (Jeffery, 1997).

4.3.2. Analysis on geological data

Some effort was spent on searching for evidence of quaternary faulting in the East African Rift valley near Debrezeit in the vicinity of a prominent alignment of young volcanoes and crater lakes. Faults with scarps in young alluvial deposits that are common in the arid part of the seismically active Western United States could not be found in the Addis Ababa region. Either scarps were not produced by large magnitude earthquakes in the East Africa Rift near Addis Ababa or scarps that are produced are eroded rapidly.

Hence, it appears that earthquake sources in the vicinity of Sibilu Dam site will not be recognized as faults with surface expression in young alluvial deposits.

The background earthquake may be one that can be generated with in a region with out producing surface fault rapture. The background earthquake is expected to be no larger than approximate magnitude of 6.5 because surface faulting should accompany larger earthquakes. (Jeffery, 1997)

4.4. Results:

The assessments being carried out with the available data, the boundaries of seismic zones and location of active faults are identified. From the statistical study on seismic data, magnitude recurrence relationships or maximum credible earthquakes are obtained. Outputs of quantitative seismic earthquake study are envelopes of ground motion recurrence relationships. Quantitative seismic design parameters such as peak ground acceleration are the final results to be used as an input earthquake motion in the stability analysis.

Site-specific acceleration values were calculated for the Sibilu Dam site based on the earthquake activity shown on figure 4.1 and the attenuation relationship published by Campbell and Bozorgnia (1994). The magnitude of the earthquake with an average recurrence of 10,000 yeas was taken to be the maximum earthquake for design of the dam. The earthquake has a magnitude of 6.6 and would be expected to occur at an approximate distance of 17km from the site. The site condition at a dam is projected to be soft rock (shear-wave velocity of approximately 360 to 750m/s in the upper 30m) (Jeffery, 1997). As the Sibilu Dam is important structure, where by its failure could endanger public safety, two levels of earthquake are used for the analysis.

The lower level acceleration corresponds to the earthquake that could reasonably be expected to occur during the lifetime of the dam. For this acceleration, the stresses should be within the elastic range of deformation, as the dam should be designed to remain operational during and after the earthquake. For this condition a mean peak horizontal acceleration of 0.09 would be expected from an earthquake of magnitude 5.

The higher-level acceleration corresponds to the maximum earthquake that could reasonably be postulated to occur at the dam site. For this acceleration level, some inelastic deformation of the dam is permitted since partial damage or loss of function is acceptable as long as public safety is not endangered. For this condition a mean peak horizontal acceleration of 0.21 would be expected from an earthquake of magnitude 6.5.

The application of these design earthquake parameters depends on the method of analysis employed. While in the pseudo-static analysis it is represented by constant acceleration coefficients, in dynamic response analysis it is defined by means of acceleration time histories or spectra related to the seismicity of the area, to the seismic geological environment and to the geotechnical characteristics of the site.

5.0 PSEUDO-STATIC SLOPE STABILITY ANALYSIS

The pseudo-static approach has a number of attractive features. The analysis is relatively simple, straightforward and its similarity to the static limit equilibrium analysis routinely conducted by geotechnical engineers makes its computations easy to understand and perform. It produces a scalar index of stability (the factor of safety) that is analogous to that produced by static stability analyses. It must always be recognized, however, that the accuracy of the pseudo-static approach is governed by the accuracy with which the simple pseudo-static inertia forces represent the complex dynamic inertial forces that actually exists in an earthquake. Difficulty in assignment of appropriate pseudo-static coefficients and in interpretation of pseudo-static factors of safety, coupled with the development of more realistic methods of analysis, have reduced the use of pseudo-static approach for seismic slope stability analysis purposes.

5.1. Selection of Pseudo-Static Coefficient

One of the major problems encountered in using this type of approach is that of selecting the value of the seismic coefficient to be used for design purposes. Methods that are commonly used include:

1. The empirical rules
2. The assumption of rigid body response
3. The elastic response analyses
4. The use of visco-elastic response analysis

5.1.1. Use of empirical rules

In the pseudo-static method of seismic stability analysis, some empirical values are adopted for the design seismic coefficient; typically this lies in the range of 0.05-0.15. There is good reason to adopt some value of this type as a means of differentiating between the seismicity and foundation conditions at different sites or for studying the advantages of different sections. On the other hand, there appears to be no published justification for using values in the range of 0.05to 0.15 as a basis for selecting or approving final design slopes. It appears that continued use of these empirical values has given them some resemblance of an authoritative design criterion.

5.1.2. Rigid body response analyses

If an embankment is assumed to behave as a rigid body, the accelerations will be uniform throughout the section and equal at all times to the ground accelerations. Thus it is sometimes argued that the design seismic coefficient should be equal to the maximum ground acceleration.

Low and stiff embankments or embankments in narrow canyons may respond essentially as rigid structures. However, there is considerable evidence that all erathdams don't behave as rigid bodies. In this method, there is nothing to guide the design engineers how this value might appropriately be *modified for different structures and ground motions*.

5.1.3. Elastic response analyses

The deficiencies in the use of empirical rules or the assumption of rigid body response have led a number of investigators to propose the use of elastic response solutions for the determination of design seismic coefficients.

The approach assumes elastic deformation in the soil mass with energy being dissipated in terms of viscous damping. However, soil deformations under higher stresses are inelastic and some energy is dissipated by hysteretic damping. Thus considerable care is required in selecting equivalent moduli and viscous damping factors for use in the analysis in order to obtain meaningful estimates of embankment response.

5.1.4. Visco-elastic response analysis:

The response of dams is in good agreement with that predicted by visco-elastic response analysis. This type of analysis also indicates that the accelerations induced in an embankment by an earthquake increases with height above the base. This behavior is in good agreement with that observed by seismographs installed at the base and crest of some dams in California and Japan.

Thus it would appear that visco-elastic response analyses can provide a reasonable approach for assessing the dynamic forces induced in an embankment by an earthquake. But the merits of the approach are not used to full advantage by incorporating the maximum dynamic forces, as static forces, in pseudo-static methods of analysis.

Considering the advantages and disadvantages of the methods mentioned above, a seismic coefficient is selected for the proposed project. Experiences show that the usual trend in Ethiopia for the dam design is the adoption of the rigid body response coefficient.

5.1.5. Seismic coefficient for Sibilu Dam analysis:

In view of the different approaches described above for the determination of seismic coefficients, it is good to compare their preference for specific site. Theoretical explanation verifies the improvement of the analysis from empirical coefficient to the visco-elastic method in their order of description above. Model tests on shaking tables and results of actual field observation also confirm this.

As the validity is not checked locally, the empirical coefficient has no rational basis to be used. The geometry of the dam site and the dam section are not similar to those expected to show rigid response behavior. For rigorous stability analysis the ultimate resisting capacity and the resulting inelastic strain range need to be established.

With these facts in mind the visco-elastic approach has better preference.

Apart from the rational basis of which it is derived, the common trend applied in the region is considered for comparison purpose. Locally, past and current practice in the selection of seismic coefficient for embankment stability analysis against earthquake force is done by the adoption of rigid body response coefficient.

This coefficient which assumes that the accelerations will be uniform through out the section and equal at all times to the ground accelerations is selected for Sibilu Dam analysis. The value obtained in chapter 4.0 from magnitude attenuation relationship, 0.21g, is used for the analysis.

5.2. Pseudo-Static Analysis

5.2.1. General:

In their most common form, pseudo-static analyses represent the effects of earthquake shaking by pseudo-static accelerations that produce inertial forces, F_h and F_v that act through the centroid of the failure mass. The magnitudes of the pseudo-static forces are;

$F_h = a_h W/g = K_h W$… 5.1

$F_v = a_v W/g = K_v W$…5.2

Where a_h and a_v are horizontal and vertical pseudo-static accelerations, K_h and K_v are dimensionless horizontal and vertical pseudo-static coefficients, and W is the weight of the failure mass. The magnitudes of the pseudo-static accelerations should be related to the severity of the anticipated ground motion.

The horizontal pseudo-static force mainly decreases the factor of safety. It reduces the resisting force (for $\Phi>0$) and increases the driving force. The vertical pseudo-static force has less influence. As a result, the effects of vertical accelerations are frequently neglected in pseudo-static analyses. The pseudo-static approach can be used to evaluate pseudo-static factors of safety for planar, circular, and noncircular failure surfaces. Many commercially available computer programs for limit equilibrium slope stability analysis have the option of performing pseudo-static analyses. Among such programs available with its derivation and coding is RAEME (rotational equilibrium analysis of multi layered embankments) by Huang (1981) described below. It is adopted for the pseudo-static analysis considering some assumptions.

5.2.2. The program RAEME:

The RAEME computer program is used to determine the factor of safety of a slope based on a cylindrical failure surface (Huang, 1981). Slopes of any configuration with a large number of different soil layers can be handled. Seepage can be considered by specifying a peizometric surface or a pore pressure ratio. To compute the static or seismic factor of safety the program applies either simplified Bishop or other method of stability analysis.

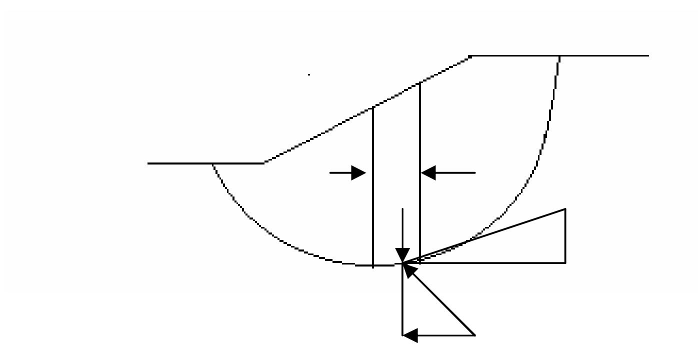


Figure 5.1 Stability Analysis by Simplified Bishop Method (Huang, 1981).

The limiting plastic equilibrium equation used to compute the factor of safety; when the seepage is specified in terms of pore pressure ratio will be:

$$F=\frac{\sum_{i=1}^{n}\left[cb_i\sec\theta+(1-r_u)w_i\cos\theta\tan\phi\right]}{\sum_{i=1}^{n}(w_i\sin\theta+c_s w_i a_i/R)}$$ …5.3 Fellenius method (Huang, 1981)

$$F=\frac{\sum_{i=1}^{n}\frac{cb_i+(1-r_u)\gamma h_i b_i\tan\phi}{\cos\theta+(\sin\theta\tan\phi)/F}}{\sum_{i=1}^{n}(w_i\sin\theta+c_s w_i a_i/R}$$ …5.4 simplified Bishop method (Huang, 1981)

When a phreatic surface is specified for the seepage the above equations become:

$$F=\frac{\sum_{i=1}^{n}\left[cb_i\sec\theta+(1-\gamma_w h_{iw})w_i\cos\theta\tan\phi\right]}{\sum_{i=1}^{n}(w_i\sin\theta+c_s w_i a_i/R)}$$ … 5.5 Fellenius method (Huang, 1981)

$$F=\frac{\sum_{i=1}^{n}\frac{cb_i+(1-\gamma_w h_{iw})\gamma h_i b_i\tan\phi}{\cos\theta+(\sin\theta\tan\phi)/F}}{\sum_{i=1}^{n}(w_i\sin\theta+c_s w_i a_i/R}$$ … 5.6 simplified Bishop method (Huang, 1981)

Where c_i = the cohesion value of each slice

b_i = the width of the slice

θ = the inclination of the slice with the horizontal

γ = unit weight, w_i= weight of the slice

h_i =height of the slice R= radius of trial failure surface

As factor of safety appears on both sides of the equation, a method of successive approximation must be used to solve it. A very effective procedure, the Newton's method of tangents, is applied in the program.

5.2.3. The Sibilu Dam model:

The Sibilu Dam cross-section used for this analysis is taken at the maximum height position. The strength parameter values for the clay core and rock were determined from the laboratory test results as given in chapter 3.

As the dam body rests on basaltic rock, the foundation material is taken as bedrock. The geometry of the dam is simplified to facilitate the slope stability analysis using the computer software REAME. The filter material used between the rockfills and the clay core is neglected.

From the storage and yield requirement the geometry used in the analysis is shown here.

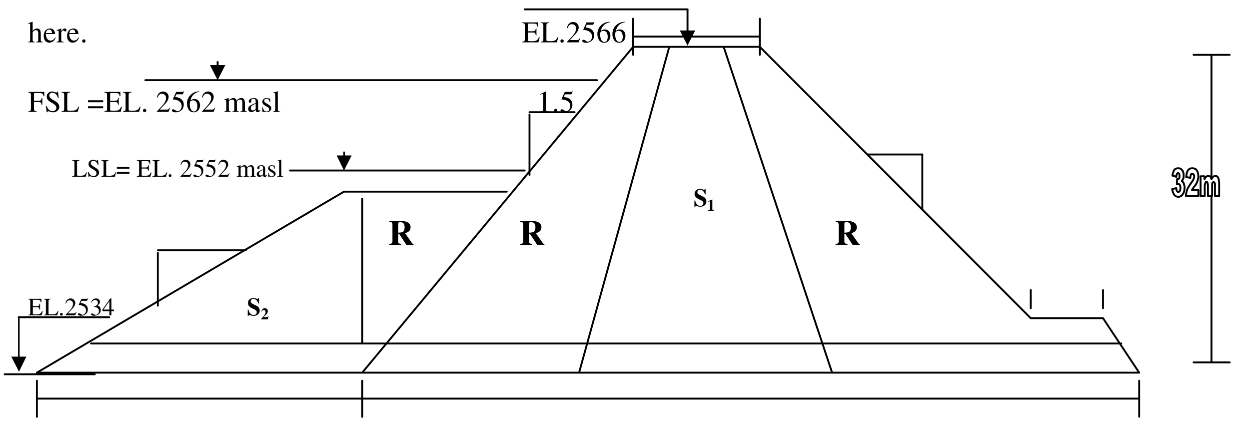


Figure 5.2. The Sibilu Dam Geometry used in the pseudo-static Analysis.

Where:

S_1= impervious clay core

S_2= random excavation

R= rock fill shell

For the Sibilu Dam analysis the earthquake effect is considered in the programme by providing a constant horizontal seismic coefficient of 0.21g.

The strength parameters used are the summarized test results in table 3.1.

5.3. Cases analyzed and their results

For the pseudo-static analysis the potential failure mechanisms that are commonly assessed are considered.

During construction of an earthdam embankment, shear stresses and pore water pressures on potential failure surface increase as the fill height is raised in successive lifts. Hence stability of upstream and downstream slopes of the dam at the end of construction is one of the critical cases analyzed.

It is an established fact that pore water pressure plays a significant role in the design of earth dams. The critical issues with this aspect are considered after the reservoir is filled to the full supply level. The upstream face is analyzed for sudden draw down conditions and the down stream slope for steady state seepage condition.

Iterating on different trial failure surfaces programme finds out failure surfaces of minimum safety factors and radius for the different critical cases considered.

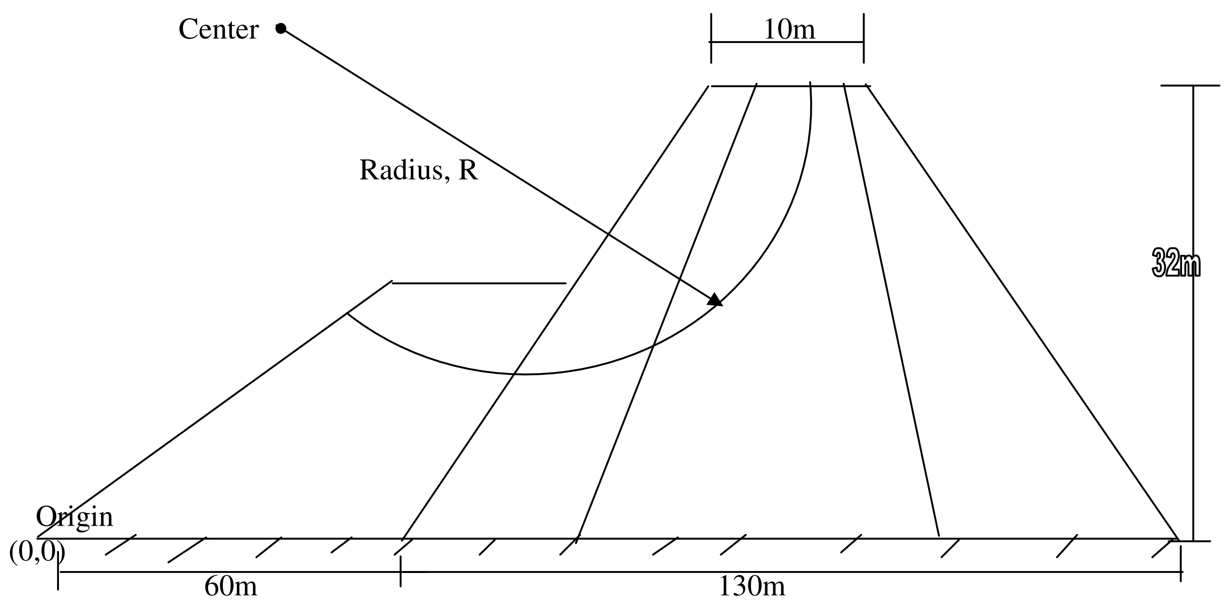


Figure 5.3. Typical Failure Surface of Sibilu Dam.

N.B. Failure surfaces for the different cases analyzed vary from one another as in table 5.1.

Table 5.1 the coordinates of the center and radius of the different failure surfaces

Cases analyzed	With out seismic coefficient			With seismic coefficient		
	Coordinates of center		Radius (m)	Coordinates of center		Radius (m)
	X (m)	Y (m)		X (m)	Y (m)	
At the end of construction U/ S	70.833	77	61.625	70.833	77	61.625
At the end of construction D/ S	171.5	78.4	69.4	91.5	95	78.5
Steady state seepage	133.5	42.5	21.74	138	59	41
Sudden draw down U/ S slope	68.33	72	51.15	68.33	71	54.85

The computed factors of safeties, considering earthquake effects and under static cases are tabulated below.

Table 5.2 computed and allowable safety factors obtained by pseudo-static analysis

Cases analyzed	Factor of safety		Allowable factor of safety*	
	With seismic coefficient	Without seismic coefficient	With seismic coefficient	With out seismic coefficient
At the end of construction u/s slope	0.947	1.432	1.25	1.5
At the end of construction d/s slope	0.941	1.477	1.25	1.5
Steady state seepage d/s	0.922	1.552	1.1	1.5
Sudden draw down u/s	0.881	1.27	1.0	1.2

* ROBIN FELL and et al. (1996)

Nevertheless, the safety factors tabulated above are a crude index of relative stability.

Especially, for materials that are influenced by deteriorating effects (soils that show reduced static strength) or by pore pressure build-ups under cyclic loading, the pseudo-static analysis cannot be applied except for low excitation levels. For silts and sands having a relative density of less than 75%, a few cycles of dynamic load are sufficient to observe significant variations in the pore pressure. With these types of materials the dynamic response analysis performed is more appropriate.

6. DYNAMIC RESPONSE ANALYSIS

6.1. General

Commonly, the pseudo-static method of analysis has been the standard procedure for the evaluation of seismic safety of embankment dams. Through time, both the improvement in the analytical tools to evaluate the response of embankments and the knowledge of material behavior during cyclic loading lead to the development of a more rational approach to the study of stability of embankments during seismic loading.

This method of analyses incorporates the real vibration behaviors such as the period (frequency), the vibration mode and damping characteristics.

They are used to evaluate dynamic stresses and strains for evaluation of liquefaction hazards, and to determine the earthquake induced forces that can lead to instability of earth and earth retaining structures.

The progress of numerical methods has also helped to understand problems of great interest, such as the development of high pore pressures in the embankment or in the foundation, during earthquake shaking. The tri-dimensional response of the dam and even the effect of a fault movement in the dam foundation can be determined. Different levels of modeling an embankment dam response to the earthquake shaking can be handled by numerical methods.

With the assumption that the material does not have a potential for the rise of pore pressure, Makdisi and Seeds' simplified procedure can be used for the dynamic response analysis of Sibilu Dam. The method involves the following steps:

1. Determination of yield acceleration, i.e. an acceleration at which a potential sliding surface would develop a factor of safety of unity. Values of yield

acceleration are a function of the embankment geometry, the undrained strength of a material (or the reduced strength due to shaking), and the location of the potential sliding mass.

2. Earthquake induced accelerations in the embankment are determined using response analyses. From these analyses, time histories of average accelerations can be determined for various potential sliding masses.
3. For a given potential sliding mass, when the induced acceleration exceeds the calculated yield acceleration, movements are assumed to occur along the direction of failure plane and the magnitude of the displacement is evaluated by a simple double integration procedure.

6.2. The Response Acceleration:

From the given earthquake time history the response point acceleration time history can be obtained by different methods. However, these point accelerations that vary with depth are not of much use since they do not occur at the same time or even in the same direction. It is an average seismic coefficient, therefore, that is used to give the net maximum earthquake load with in a potential failure mass. This is obtained by assuming a failure surface and by computing the total load on this mass at different times and then by finding the absolute maximum value of the total load.

On the basis of response computations for embankments subjected to different ground motion records, a relationship for the variation of induced average acceleration with embankment depth has been established. Then a design curve as in Fig. 6.1 of the variation of maximum acceleration ratio, k_{max} / U_{max}, with depth is made for a range of all data considered. It would then be sufficient, for design purposes, to estimate the maximum crest acceleration in a given embankment due to a specified earthquake and use this relationship to determine the maximum average acceleration for any depth of the potential sliding mass.

K_{max}/ U_{max}

Figure 6.1. Variation of Maximum Acceleration Ratio with Depth of Sliding Mass (Seed and et al., 1979)

The response acceleration and there by the inertia forces to be generated during an earthquake will depend on:

i. The geometry of the dam and its foundation

ii. The material properties and

iii. The earthquake time history

I. The geometry of the dam and its foundation:

For earth dams with the following;

- Very long compared to height (L>4H)
- Very flat slopes (<1:1.5) so that the vibrations takes place in shear only

The only geometrical parameters involved in the analysis are the height of the dam (h) and the depth of the foundation (d).

II. The material properties:

The response acceleration can be computed with the assumption of linear, equivalent linear and non-linear material behaviors. Improvement in results is obtained with the shift of the analysis from linear to non-linear material behaviors.

III. The earthquake time history:

Although the peak amplitude, the period and the duration of earthquake motion are the controlling parameters of earthquake motion, the entire time history of acceleration is necessary for the dynamic analysis. For the dynamic analysis of Sibilu Dam, Elcentro NS component recorded time history is used with the peak acceleration re scaled to peak value of 0.09 and 0.21g referred to no damage and no failure design criterion respectively established for Sibilu Dam site.

For the given geometry and input ground motion the maximum crest acceleration of Sibilu Dam is computed by varying the material behavior with different approaches as shown next.

6.2.1. Linear response analysis:

The earthquake-induced acceleration at the crest can be obtained by assuming linear material behavior. Shear modulus and damping factor of soils are both strain dependent, i.e. any soil is non-linear material in terms of the level of strain. However, for the sake of simplicity both the shear modulus and the damping factor are assumed to be constant in linear analysis. This linear analysis has an application to response analysis of soil deposits in connection with microtremors or low amplitude earthquake motions. Moreover, it can serve as an approach to an approximate solution to non-linear problems where large strains are encountered.

For the linear analysis of Sibilu Dam a program based on frequency response function developed by Ohsakhi (1980) is applied. The program uses modal superposition method as described herein. Dynamic equilibrium equation of motion is formulated for a discrete or lumped mass system.

Represented in matrix form:

$$[M]\{\partial^2 x / \partial t^2\} + [C]\{\partial x / \partial t\} + [K]\{x\} = -\{\partial^2 y / \partial t^2\}[M]\{I\} \quad \ldots 6.1$$

Where: [M] = the total mass matrix

[C] = the total viscous damping matrix and

[K] = the total stiffness matrix

{x}= displacement of the mass relative to the base vector

{y}= displacement of the base vector

Let the relative displacement vector {x} be represented by product of modal matrix [u] and normal coordinates { q (t)}; $\{x\}=[u]\{q(t)\}$

Where $\{q(t)\}= e^{i\omega t}$

Then, equation 6.1 becomes;

$$[M][u]\{\partial^2 q / \partial t^2\} + [C][u]\{\partial q / \partial t\} + [K][u]\{q\} = -\{\partial^2 y / \partial t^2\}[M]\{I\} \ldots 6.2$$

The modal matrix [u] and natural frequencies, ω_j, are determined from the eigenequations derived from undamped free vibration of the system.

Applying orthogonality of eigenvectors the mass and stiffness terms are decoupled. After certain manipulations introducing modal damping factors the motion of a multi degree of freedom system is decomposed to behave as if it were a single degree of freedom system with respect to each mode. Then the responses are individually computed with respect to each mode. Finally the vibration of the system is determined by superposing all modal responses.

Based on the dynamic response analysis adopted (Seed and et al. 1978) it is required to calculate the response acceleration at the crest of the Sibilu Dam. For the linear analyses the input data is used so as to get optimized result as formatted in the programme. The maximum dam cross-section (32m height) is considered for the analysis and is divided in to four sub layers for the purpose of accuracy.

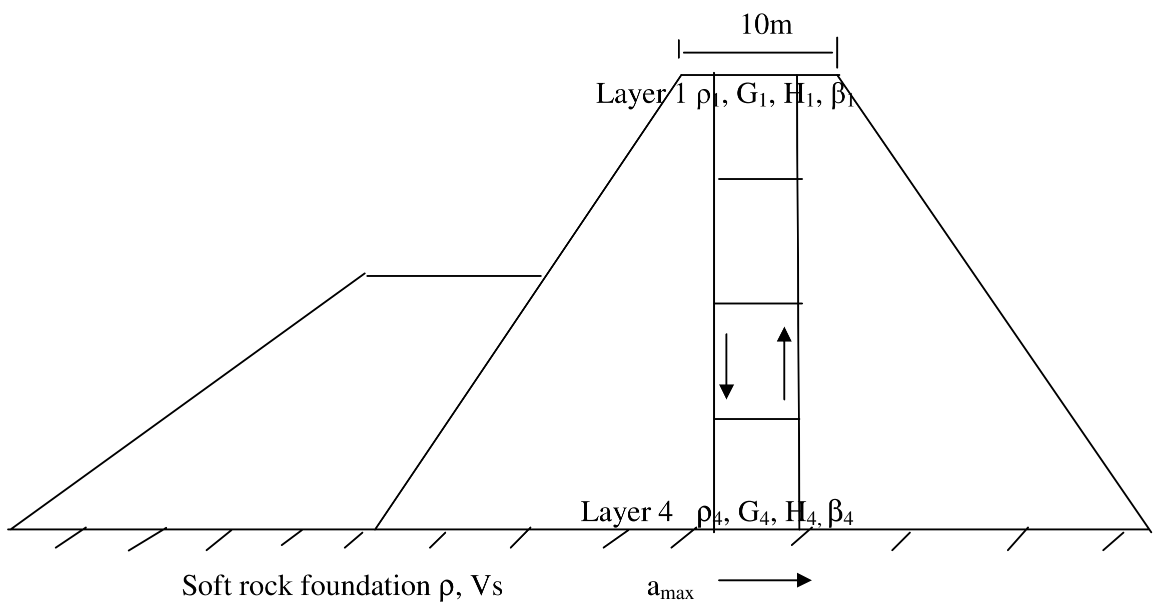


Figure 6.2. Sibilu Dam Model for the Linear Analysis.

The densities, thickness, shear modulus and critical damping ratios are given for each layer.

The density applied is obtained from the test result given in table 3.1.

For linear strain ranges a shear modulus of (500* undrained shear strength) and a critical damping ratio of 0.02 are used (Ohsaki, 1982).

An input earthquake, Elcentro NS component, rescaled to 0,09g and 0.21g is assumed to act at the soft rock foundation level.

6.2.2. Equivalent linear response analysis:

For this analysis the program SHAKE (1985) is applied. The program computes the response in a horizontally layered soil-rock system subjected to vertical travelling shear waves through the linear visco-elastic system shown in Fig. 6.3.

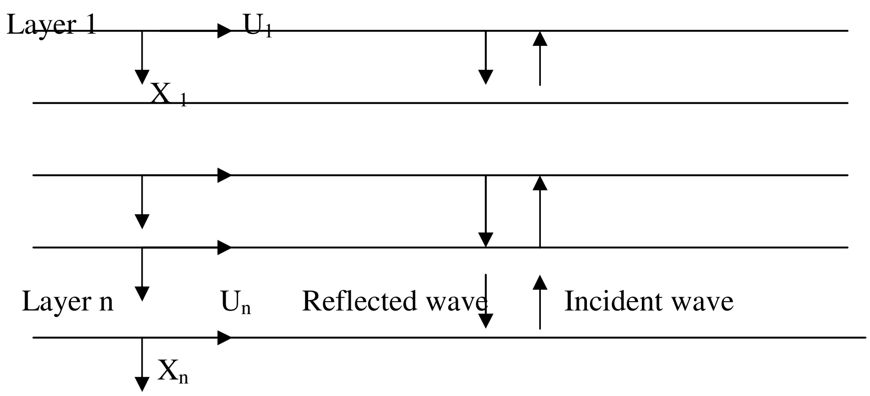


Figure 6.3. Horizontally Layered One Dimensional System (Schanabel, 1972)

This propagation will cause only horizontal displacements: u= u (x, t)…6.3

Which must satisfy the wave equation:

$\rho\, \partial^2 u / \partial t^2 = G\, \partial^2 u / \partial x^2 + \eta\, \partial^3 u / \partial x^2 \partial t$…6.4

Where ρ is density, G is shear modulus and η is viscosity related to critical damping ratio.

Harmonic displacements with frequency ω can be written as: $u(x, t)= u(x)\, e^{i\omega t}$…3

Substituting equation 6.4 in equation 6.3 results in:

$(G+ i\omega\eta)\partial^2 u / \partial x^2=\rho\omega^2 u$…6.5

Which has the general solution: $u(x)= E\, e^{ikx} + F\, e^{-ikx}$…6.6

In which $k^2= \rho\omega^2/ G+ i\omega\eta =\rho\omega^2/ G\,(1+2i\beta)= \rho\omega^2/ G^*$

Where G^* is the complex shear modulus

The solution to the wave equation is:

$u(x,t) = E\, e^{i(kx+\omega t)} + F\, e^{-i(kx-\omega t)}$…6.7

Where the first term represents the incident wave travelling upwards and the second term represents the reflected wave travelling downwards, which is valid for each of the layers.

Applying the continuity of stresses and displacements at all interfaces yields the following recursion formulas for the amplitudes of layer m+1 expressed in terms of amplitudes of layer m:

$E_{m+1} = ½\, E_m(1+\alpha_m)\, e^{ik_m h_m} + ½\, F_m(1-\alpha_m)\, e^{-ik_m h_m}$…6.8

$F_{m+1} = ½\, E_m(1-\alpha_m)\, e^{ik_m h_m} + ½\, F_m(1+\alpha_m)\, e^{-ik_m h_m}$…6.9

Where α_m is the complex impedance ratio,

$\alpha_m = k_m G_m / k_{m+1} G_{m+1} = (\rho_m G_m / \rho_{m+1} G_{m+1})^{½}$

At the free surface, the shear stresses must be zero which gives $E_1 = F_1$.

Repeated use of the recursion formulas resulted in the transfer function $A_{n,m}$ between the displacements at level n and m defined by :

$A_{n,m}(\omega) = \dfrac{e_m(\omega) + f_m(\omega)}{e_n(\omega) + f_n(\omega)}$ …6.10

Where e_m, f_m, e_n, and f_n are the amplitudes of incident and reflected waves in layers m and n respectively.

With this function if the motion is known in any one of the layer in the system, the motion can be computed in any other layer. This method is Kanai's solution to the harmonic wave equation. The theory is extended to transient earthquake motions through the use of Fast Fourier Transform algorithm. The non-linearity of the shear modulus and damping is accounted by the use of the equivalent linear soil properties

(Idriss and Seed, 1968, Seed and Idriss, 1970) an iterative procedure is used to obtain values for modulus and damping compatible with the effective strains in each layer.

Like the linear case this term of analysis is applied to find the response peak acceleration at the crest of the Sibilu Dam, hence a shake column is used as shown below.

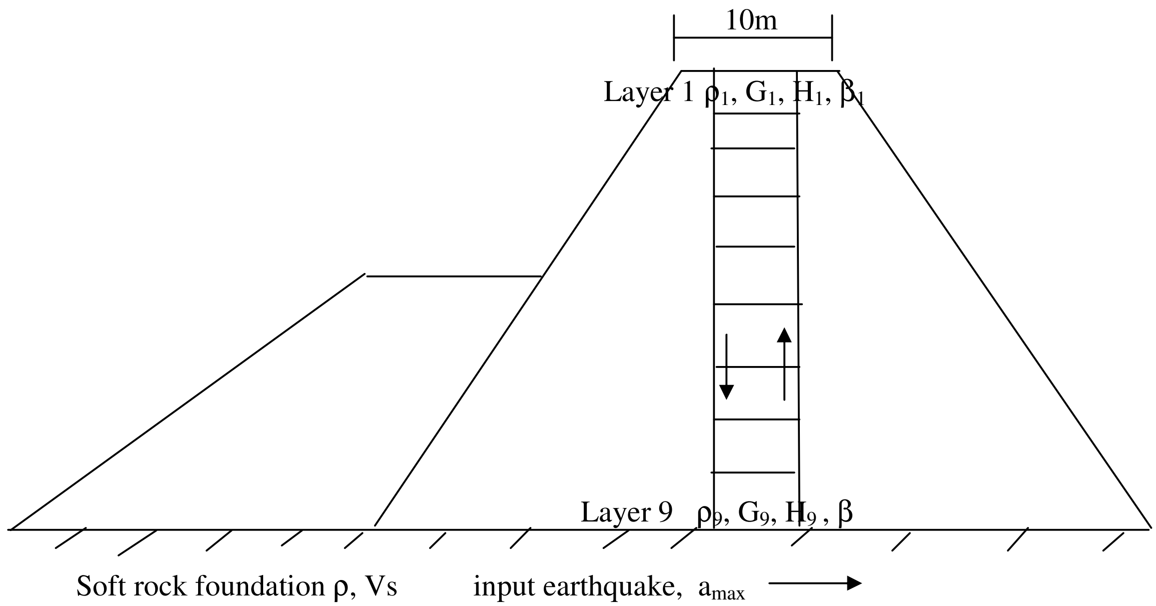


Figure 6.4. Sibilu Dam Model for the Equivalent Linear Analysis.

As the soil properties vary with the variation of strain level induced during shaking the selected column is divided in to sub-layers. The Sibilu Dam in this case is divided in to nine sub layers, which gives sufficient accuracy.

An input earthquake motion is applied at the soft rock base level rescaled to 0.09g and 0.21g-peak acceleration. The initial material properties ρ, G, and β are given as formatted in the programme (Schenabel, 1972).

6.2.3. Response spectrum analysis:

Makdisi and Seed (1979) have prepared simplified procedure for evaluating embankment response based on spectral analysis. The validity has been proven to give sufficient accuracy for many practical purposes. The method also allows the use of strain dependent material properties through iteration procedure. The analysis is based on shear slice theory which considers the dam as a plate obtained by slicing it with two cross sections perpendicular to the dam axis and analyzing the plate as a wedge shaped shear beam. The assumptions in the theory are;

- The dam is a beam with a variable wedge shaped cross section. Displacements of points on the same horizontal cross section are horizontal and uniform.
- Deformation produced in the dam is shear deformation only.

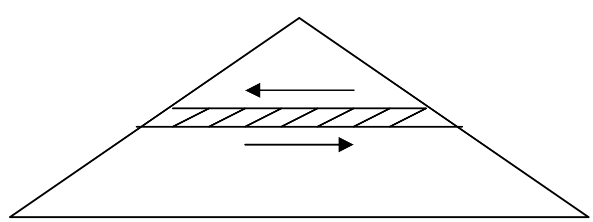


Figure 6.5. Dam Section in Shear Beam Analysis (Shunzo, 1984).

The steps involved in the response computation are described as follow:

1. *Evaluation of initial properties*

For the first iteration of computations, assume any initial value of shear modulus G, and determine the ratio G/G_{max}, for the calculated value of G/G_{max} the corresponding value of shear strain and damping could then be determined.

Figure 6.6. Shear Modulus and Damping Characteristics Used in the Response Computation (Makdisi, 1979)

2. Calculation of maximum acceleration and natural period

From the principle of shear slice theories, considering the first three modes of vibration the natural frequencies are:

$\omega_1 = 2.4\ Vs/h$…*6.11* (Makdisi, 1979)

$\omega_2 = 5.52\ Vs/h$… *6.12* (Makdisi, 1979) *and*

$\omega_3 = 8.65\ Vs/h$…*6.13* (Makdisi, 1979)

Where Vs= shear wave velocity and h= the height of the dam

And the corresponding values of the mode participation factors at the dam crest for the first three modes are given by:

$\phi_1(0)=1.6$; $\phi_2(0)=1.06$; and $\phi_3(0)=0.86$… (Makdisi, 1979)

Therefore, the value of acceleration at the crest in each mode is given by the expression:

$U_{nmax} = \phi_n(0)\ S_{an}$…*6.14* (Makdisi, 1979)

Where S_{an} is the spectral acceleration and is a function of the natural frequency and damping value.

Period- seconds

Figure 6.7. Normalized Acceleration Response Spectra (Makdisi, 1979)

The maximum value of crest acceleration is then approximated by taking the square root of the sum of the squares of the maximum acceleration of the first three modes.

$$U_{max}=[\sum_{i=1}^{3} (U_{max})^{2}]^{1/2}$$

3. Determination of average shear strain

To estimate the strain compatible material properties, an expression for the average shear strain over the entire section should be determined. This is done by considering the shear strain contribution of each mode with shear strain mode participation factor.

$\gamma_{ave} = 0.65 * 0.3 * h / v_s^2 * S_{a1}$…*6.15* (Makdisi, 1979)

Having obtained a new value of average shear strain a new set of modulus and damping values can be determined if these values are different from those assumed a new iteration would be performed.

For maximum credible earthquake (0.21g) and,

The final iteration results are:

Height=32m

Shear wave velocity=183m/sec

G/G_{max}=0.4 and β=13%… assumed

From G= $\rho\, v_s^{\,2}$, and constant density $V/v_{max} = \sqrt{G/G_{max}} = 0.632$

Using equations 6.11 to 6.13 the values of the modal periods are computed;

ω_1= 2.4*183/32=13.7 rad/sec,

$T_1 = 2 * \pi / \omega_1 = 0.458$ sec

ω_2=5.52*183/32=31.54 rad/sec,

$T_2 = 2 * \pi / \omega_2 = 0.2$ sec

ω_3=8.65*183/32=49.43 rad/sec,

$T_3 = 2 * \pi / \omega_3 = 0.127$ sec

Where ω_1, ω_2 ω_3, T_1, T_2, and T_3 are natural frequencies and periods of the first three modes of vibration of the dam respectively.

Then using equation 6.14, the values of crest acceleration in each mode are;

U_{1max}=1.6*1.4*0.21=0.47 g

U_{2max}=1.06*1.15*0.21=0.256 g

U_{3max}=0.86*1.05*0.21=0.19g

The maximum value of the crest acceleration for the dam is approximated by taking the square root of the sum of the squares of modal accelerations;

$$U_{max} = \left[\sum_{i=1}^{3} (U_{max})^2 \right]^{1/2}$$

$$= 0.568g$$

Average shear strain over the entire section is calculated by using equation 6.15;

$\gamma_{Ave.}$=0.65*0.3*32/183^2*0.294*9.802

= 0.054%…compatible with G/G_{max} and β adopted.

For operation basis earthquake (0.09g) the final iteration results are:

Height=32m

Shear wave velocity=204m/sec

G/G_{max}=0.55 and β=10%

$V/v_{max} = \sqrt{G/G_{max}}$= 0. 74

Applying equations 6.11 to 6.13 the values of the modal periods are computed;

ω_1= 2.4*204/32=15.25 rad/sec,

$T_1 = 2 * \pi / \omega_1 = 0.412$ sec

ω_2=5.52*204/32=35.07 rad/sec,

$T_2 = 2 * \pi / \omega_2 = 0.18$ sec

ω_3=8.65*204/32=54.95 rad/sec,

$T_3 = 2 * \pi / \omega_3 = 0.11$ sec

Where ω_1, ω_2 ω_3, T_1, T_2, and T_3 are natural frequencies and periods of the first three modes of vibration of the dam respectively.

Then using equation 6.14, the values of crest acceleration in each mode are;

U_{1max}=1.6*1.5*0.09=0.216 g

U_{2max}=1.06*1.25*0.09=0.119 g

U_{3max}=0.86*1.05*0.09=0.085g

The maximum value of the crest acceleration for the dam is approximated by taking the square root of the sum of the squares of modal accelerations;

$$U_{max} = \left[\sum_{i=1}^{3} (U_{max})^2 \right]^{1/2}$$

$$= 0.261g$$

Average shear strain over the entire section is calculated by using equation 6.15;

$$\gamma_{Ave.} = 0.65*0.3*32/\ 204^2*0.135*9.802$$

$$= 0.02\% \ldots \text{compatible with } G/G_{max} \text{ and } \beta \text{ adopted.}$$

For input with peak 0.09g and 21g an output with peak 0.148g and 0.345g are obtained by the linear response analysis, and an output with peak 0.217g and 0.447g are obtained by the equivalent linear response analysis. The corresponding values for the response spectrum analyses are 0.261g and 0.568g respectively.

Table 6.1. Results of the response analysis

Input peak horiz. Accele.(g)	Type of analysis	Response acceleration (g)
0.09	Linear	0.148
	Equivalent linear	0.217
	Response spectrum	0.261
0.21	Linear	0.345
	Equivalent linear	0.447
	Response spectrum	0.568

The average value of the response results from the different methods is used for the computation of deformation.

6.3. Embankment Performance:

An important step in the dynamic response analysis is to determine what happen when the dynamic load exceeds the dynamic strength that is covered here for the

Sibilu Dam. Since the instantaneous acceleration during the earthquake may be large enough to reduce the factor of safety below one, surfaces of discontinuity may be produced and displacements may develop along slip surfaces. The displacement of the sliding mass is estimated by using Makdisis' and Seeds' (1978) procedure. This method evaluates dynamic response of the embankment as proposed by Seed and Martin (1966) rather than rigid body behavior. It assumes that failure occurs on a well-defined failure surface and that the material behaves elastically at stress level below failure but develops a perfectly plastic behavior above yield. Determination of yield acceleration (resistance) and calculation of permanent deformation described below are the necessary steps for estimating the earthquake-induced deformation.

Yield acceleration is an acceleration, which is needed to bring the factor of safety of the possible failure surface to one. It is computed for different cases of pore pressure and depth of failure surface as shown in K_y column in the deformation analysis tables.

Therefore, the yield acceleration is computed using the pseudo-static slope stability analysis by the computer program RAEME. The yield strength used is 80% the undrained strength for the clay core material.

The design graphs reproduced from Seed's simplified procedure and used for the deformation analysis are presented below.

Ky/ Kmax Ky/ Kmax

Figure 6.8. Variation of Yield Acceleration with Average Normalized Displacement (Seed and et al., 1979)

The dynamic response of the dam is accounted for by an acceleration ratio that varies with the depth of the potential failure surface relative to the height of the dam (Fig. 6.1).

By subjecting several real and hypothetical dams to several actual and synthetic ground motions Makdisi and Seed computed the variation of permanent displacement with k_y/k_{max}. Normalizing the displacements with respect to the peak base acceleration and the fundamental period of the dam reduced scatter in the predicted displacements. Prediction of permanent displacements by the Makdisi – Seed procedure is accomplished with the charts shown in Figure 6.1 through Figure 6.9.

Assuming the depth of failure surface at quarter, half, three-fourth and full height of the dam, the computed displacements for the different critical cases as presented below (Fig. 6.9), pore pressure conditions and levels of input earthquake motion resulting when the earthquake-induced acceleration exceeds the resisting yield acceleration are given in Tables 6.1-6.6.

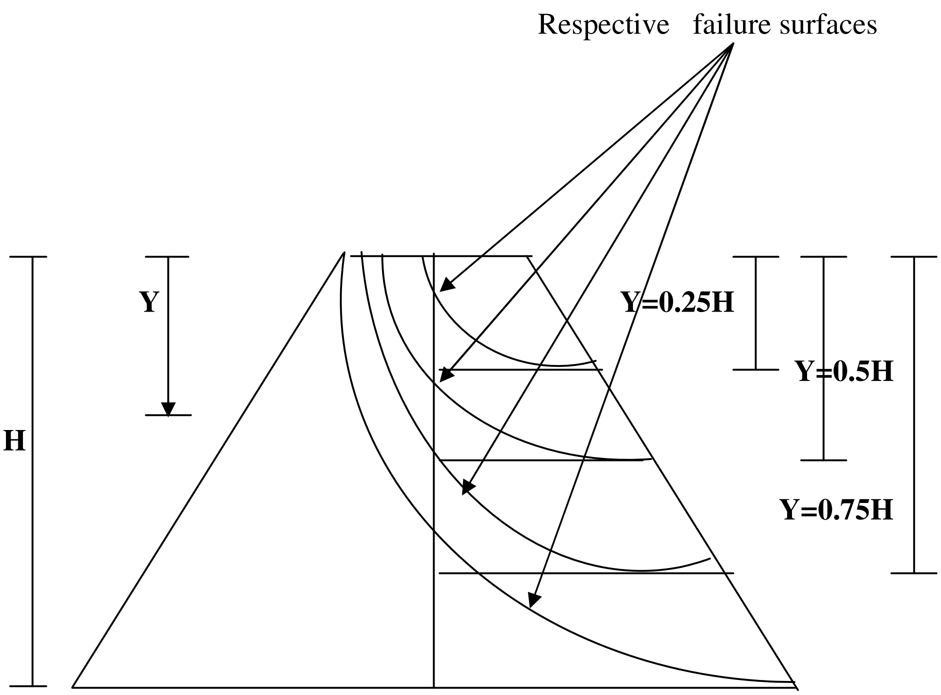


Figure 6.9. Typical Dam Section for Different Failure Surfaces

Table 6.1. Case 1 – at the end of construction for operating basis earthquake (0.09g) where: (k_{max}= maximum average acceleration) (k_y = yield acceleration)

(U= induced deformation) (T_O= fundamental period of the dam)

Y/H	**K_{max}**	**K_y**	**K_y/K_{max}**	**$U/K_{max}gT_o$**	**U (cm)**
0.25	0.167	0.092	0.551	0.010	0.752
0.5	0.104	0.148	1.420	_	_
0.75	0.073	0.180	2.470	_	_
1.00	0.063	0.185	2.940	_	_

Table 6.2. Case 2 – at the end of construction for maximum credible earthquake (0.21g)

Y/H	**K_{max}**	**K_y**	**K_y/K_{max}**	**$U/K_{max}gT_o$**	**U(cm)**
0.25	0.363	0.092	0.253	0.100	16.335
0.5	0.227	0.148	0.652	0.008	0.82
0.75	0.159	0.180	1.132	_	_
1.00	0.136	0.185	1.360	_	_

Table 6.3. Case 3- steady state seepage for operating basis earthquake (0.09g)

Y/H	**K_{max}**	**K_y**	**K_y/K_{max}**	**$U/K_{max}gT_o$**	**U(cm)**
0.25	0.167	0.143	0.857	0.001	0.075
0.5	0.104	0.148	1.420	_	_
0.75	0.073	0.149	2.040	_	_
1.00	0.063	0.150	2.400	_	_

Table 6.4. Case 4 – steady state seepage for maximum credible earthquake (0.21g)

Y/H	K_{max}	K_y	K_y/K_{max}	$U/K_{max}gT_o$	U(cm)
0.25	0.363	0.143	0.394	0.06	9.8
0.5	0.227	0.148	0.652	0.008	0.82
0.75	0.159	0.149	0.937	0.0006	0.04
1.00	0.136	0.150	1.103	–	–

Table 6.5. Case 5- sudden draw down for operating basis earthquake (0.09g)

Y/H	K_{max}	K_y	K_y/K_{max}	$U/K_{max}gT_o$	U(cm)
0.25	0.167	0.092	0.551	0.010	0.752
0.5	0.104	0.110	1.06	–	–
0.75	0.073	0.115	1.575	–	–
1.00	0.063	0.135	2.140	–	–

Table 6.6. Case 6- sudden draw down for maximum credible earthquake (0.21g)

Y/H	K_{max}	K_y	K_y/K_{max}	$U/K_{max}gT_o$	U(cm)
0.25	0.363	0.092	0.253	0.010	16.335
0.5	0.227	0.110	0.485	0.050	5.110
0.75	0.159	0.115	0.723	0.006	0.430
1.00	0.136	0.135	0.993	–	–

7.CONCLUSION AND RECOMMENDATIONS

Analysis was made for the Sibilu Dam, by the pseudo-static method with a horizontal seismic acceleration of 0.21g. Factor of safeties ranging from 0.881 to 0.947 were obtained. The results for all the critical cases analyzed show that the computed safety factors are below the allowable values for pseudo-static analysis.

The dynamic response analysis, with an operating basis earthquake (0.09g) showed that the computed deformation values are below the acceptable value, 3 inch (76.2mm)*. Hence the dam is safe satisfying deformation related, no damage, design criterion. With the maximum credible earthquake (0.21g) as an input, the computed deformations will not cause complete collapse and hence the dam could remain operational after the maximum earyhquake.

Comparison of the pseudo-static and dynamic response analysis showed that the later ensures safety of the dam from deformation criteria while the pseudo-static analysis indicates unsafe condition. This implies that the dynamic response analysis shows an earthquake resistance analysis and design aspect which we can't observe by the pseudo-static one. If the pseudo-static results have to be strictly followed it leads to conservative (uneconomical) design.

The dynamic response analysis shows that the Sibilu Dam will experience a maximum of 16.335cm deformation with the maximum credible earthquake occurrence. This quantitative result is an important basis giving insight to judge on the serviceability of the dam such as crest settlement checked for overtopping and cracks limit requirements for piping. These are the necessary parameters for dam design, which can't be investigated by the Pseudo-static analysis.

* U.S. Army Corps of Engineers

This study showed that the earthquake analysis and design of dams, especial major dams require a more elaborate dynamic analysis as the pseudo-static analysis would not show all safety aspects of a dam in an event of an earthquake. However, the scope of this thesis doesn't exhaust all aspects of dynamic response analysis and hence more rigorous approaches could be adopted for further study.

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