Addis Ababa University
Addis Ababa Institute of Technology
School of Civil and Environmental Engineering (Major in Hydraulic Engineering)

Dynamic Analysis of Concrete Faced Rock Fill Dams (A Case Study on Lower Awash Multi-Purpose Dam Project)

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Presented in partial fulfillment of the requirements Degree of Masters of Science in Civil Engineering, Major in Hydraulic Engineering

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Addis Ababa University
Addis Ababa Institute of Technology
School of Civil and Environmental Engineering

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By: - Belay Biset

This is to Certify that the thesis prepared by Belay Biset, entitled: Dynamic Analysis of Concrete Faced Rock Fill Dam (A Case Study on Lower Awash Dam and Irrigation Project) submitted in partial fulfilment of the degree of masters of science (Civil and Environmental engineering) (Major in Hydraulic Engineering) complies with the regulation of the university and meets the accepted standards with respect to originality and quality.

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DECLARATION

Belay Biset Gebayaw, declare that this thesis is my own original work that has not been presented and will not be presented by me to any other university for similar or any other degree award.

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Date of Submission June 2017.

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List of symbols and Abbreviations

CFRD- Concrete FacedRock Fill Dam

MCE-Maximum Credible Earthquake

DBE- Design Basis Earthquake

Ha- Hectare

UTM-Universal Transverse Mercator

g-gravity

m.a.s.l –meters above sea level

Km^2-kilometre square

N-Northing

E-Easting

WWDSE-Water Works Design and Supervision Enterprise

MER-Main Ethiopian Rift Valley

ICOLD-International Commission on Large Dams

FEM-Finite Element Method

LEM-Limit Equilibrium Method

PGA- Peak Ground Acceleration

SEE-Safety Evaluation Earthquake

ECRDs-Earth Core Rock Fill Dams

OBE-Operating Basis Earthquake

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ABSTRACT

This research work presents a Dynamic Analysis of Concrete Faced Rock Fill Dam located in a high seismic zone in Afar national regional state in the Lower Awash Sub-basin, Ethiopia. The Analysis considered dam configuration of 48m high.The concrete Face Slab will be lined along the dam upstream slope face and extend to top foundation level.

The Dynamic Analyses of Concrete Faced Rock Fill Dam is investigated through numerical analyses of Geo-studio 2007 software packages using both Limit equilibrium and Finite element method.

The Dam was originally designed with Zoned Earth-Rock Fill Dam with Central Clay Core.This research work on Concrete Faced Rock Fill Dam was aimedto provide a feasible dam design alternative under clay core material constraint for zoned earth-rock fill dam with central clay core under seismic load of maximum credible earthquake (MCE) and safety evaluation earthquake (SEE).

The horizontal accelerations generated under the influence of a set of base rock motions are computed for reservoir levels at normal pool level and ground motion intensities. The analyses of Concrete Faced Rock Fill Dam was conducted using three different earthquake (Elcentro, Hachinohe and Kobe) records of similar modified maximum acceleration and duration as base excitation are compared. Applying these seismic shocks,the horizontal and crest settlements are 140mm and 150mm respectively, which means the induced deformations, do not require additional extension of dam or cumber height.

The upstream slope of the dam is stable in all the three earthquake loading conditions against sliding, however, downstream slopes showed a little lower factor of safety than minimum recommended factor of safety against sliding.The thick alluvium foundation of the dam is completely liquefied even for Kobe earthquakes records of short duration for both MCE and SEE, which indicates a potential failure surface if the dam is founded on the thick loose alluvial deposit.

Keywords:Maximum Credible Earthquake, Design Base Earthquake, Finite Element Method, Limit Equilibrium Method, Permanent Deformation and Liquefaction.

1. Introduction

1.1 Background

Concrete Faced Rock fill Dam, CFRD, is a kind of embankment dam designed with an impervious element of concrete face slab constructed on the upstream face of underlying rock fill body in order to achieve water tightness. Dams are designed and constructed to withstand various natural forces and events that have occurred in the past or may be expected to occur in the future. Anticipating the effects of earthquake that would cause the dam to fail – is one of the most important parts of the process of designing these structures, which generally are expected to serve society for 100 years or more.

In evaluation of seismic stability in embankment dams, the focuses are pore water pressure development, excessive deformation, slumping settlement, cracking and planar or rotational failure of embankment. The primary requirement for the earthquake-resistant design of dam is to protect public safety, property and life (ICOLD Bulletin, 072). During earthquake, the previous loading and stress condition of the embankment material will be changed. The new load mode will be dynamic which changes stress state that might result shear or tensile failure of material. (USBR, 2012).

The potential consequences of a dam failure should be understood for planned dams and for those, which are already built. It is important to understand the concerns associated with existing and future downstream developments, especially the risk associated with loss of life and property if a dam fails. Therefore, this study is to carry out the dynamic analysis of concrete face rock fill dams under earthquake loading condition. The scope of the study is limited to the estimation of the deformation, foundation liquefaction and stability of dams due to cyclic loading.

Freeboard allowance for earthquake-induced settlements may exceed 1% of the dam‟s height. In the current state of the art, this allowance cannot be significantly reduced by design efforts and is weakly dependent on the dam‟s geometry and materials. The goal is to advance in the development of simple methods to estimate earthquake-induced deformation of dams at design stage. Due to dam zonification, numerical methods appear to be the natural choice.

Due to the uncertainties inmaterial properties, simple constitutive models should be adopted.

The time-dependent deformation, especially separation between concrete face slab and cushion layer or concrete face supporting layer, seems one of the most significant problems of high concrete-faced rock fill dams. Therefore, selection of rock fill material should be made with care to prevent unpredicted settlement during water impounding. Well-graded rock fill or gravel (with free drainage) is commonly adopted as an excellent material in practice of the developed countries. Some design engineers state that weathered rock fill or dirty gravel can also be used, if zoning is taken into account and compaction for rock fill material is well provided.

1.2 Statement of the problem

The type of dam selected is located in seismic zone, just on top of the active rift floor, Eastern African Rift Valley, in which a sudden earthquake might happen and shake the lithosphere, the upper surface layer.

The expected earthquakes hypo central depth is very shallow ranging 5-7km and the proposed dam site is planned to rest on a foundation, which is about 25m thick alluvium deposits which are suspected for liquefaction. Dams with more than 45m high and reservoir capacity of 120 hectare -meter cubes, and peak ground acceleration equal or greater than 0.25g need dynamic analysis (ICOLD bulletin, 1989).Unwell seismic analysis of high dams in earthquake zone could lead to a disastrouscollapse and limited serviceability. In other end, if the analysis ended with amplifiedremedial measures which are not necessary, the project will end being uneconomical.

The proposed dam that is selected by the project office, WWDSE, for the site is an earth- rock fill dam with central clay core. However, there is shortage of clay materials near the dam site for the central clay core, and even the proposed borrow site for clay core has high to very high degree of expansion and high compressibility.

In addition to Earth Fill Dam with Central Clay Core and Zoned Earth- Rock Fill Dam with Central Clay Core other alternative dam types considered i.e. Concrete faced Rock fill Dam, Asphalt Concrete Faced Rock fill Dam with Clay Core, Asphalt Faced Rock fill Dam with Asphalt core, etc. Based on the multi- criteria analysis conducted Concrete Faced Rock fill Dam was selected to conduct dynamic analysis for this research work.

Various criteria are involved in the process of alternative dam type selection. Of which suitability to the site, foundation requirement, construction material availability, diversion requirement, seismicity, cost, and method of construction and requirements of foreign currency were the major points considered in the multi-criteria analysis.

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So, concrete faced rock fill dam is preferred due to difficulties in finding core material and which has relative advantages than other types of dams such as speed of construction, steep slopes, parts `above water are easy to repair, can be constructed after completion of the rock fill section ,can be used as slope protection, etc.Concrete face rock fill dam is also inherently safe against potential seismic damage (Sherard and Cooke, 1987). This is because earthquake cannot cause pore water pressure build up and strength reduction of the free draining compacted rock fill and therefore will have small deformation.

This research work aims to assess the seismic behavior of a Concrete Faced Rock Fill Dam at Lower Awash sub-basin as an alternative to an Earth Rock Fill Dam with Clay Core.

1.3 Objective

1.4.1 General objective

The main objective of this research work is to investigate the settlement and stability behaviour of a Concrete Faced Rock fill Dam for Lower Awash Dam and Irrigation Project subjected to earthquake loading by using method of dynamic analysis.

1.4.2 Specific objectives

The specific objectives of this research works are:

- To evaluateseismic character of the dam site.
- To determine the extent of permanent deformations of the damunder maximum credible earthquake (MCE) and safety evaluation earthquake (SEE).
- To determine post-earthquake slope stability undermaximum credible earthquake (MCE) and safety evaluation earthquake (SEE).
- To assess earthquake induced liquefaction of the dam foundation

1.4 Structure of the Thesis

This thesis contains six chapters organized as follows:

In Chapter one a general introduction to the study with its background, objective, problem statement and a brief description of the study area.

In Chapter two, design characteristics of concrete face rock fill type of dams developed up to now, are discussed. Since the design of this specific type of dam is mainly dependent on the experience of precedent, former design features, their improvement by time and the reasons forcing these improvements are investigated. Results of these improvements and recent design features are discussed.

In Chapter Three,Alternative Dam Design Selection criteria

In Chapter four, the material used and methodology adopted for the study are introduced.

Chapter five Analysis result and discussion of Lower Awash Dam and Irrigation project has been done.

In Chapter Six, the analysis ends with the conclusions and recommendations by the study.

2. Literature Review

2.1 Dam Type Selection

2.1.1 General

There are factors, which must be considered in selection of dam type, such as topography, availability of construction materials, spillway requirement, diversion arrangements, experience of the contractor, geologic condition, etc. The availability of
construction materials can limit the choice of dam type in economic wise, but provided
the difference in cost is small, the choice between a concrete and an embankment dam is often influenced by previous and local practice and preference of the designer. The dam engineer is required to synthesize design solutions, which, without compromise on safety, represent the optimal balance between technical, economic and environmental considerations (Novak et al., 2007). A material whose composition is satisfactory for use in embankment construction is a suitable material. Embankment dams, i.e. rock fill or earth fill dams with impervious core, induce lower stresses in the foundation than concrete dams, and can be built over a wide range of foundation materials, on rock and on soil foundations. A well-designed embankment dam makes the best use of local materials to fit site conditions. Quality construction is
necessary to transform the design concepts into a successful project.

2.1.2 Embankment Dams

Nowadays, embankment dams exist in excess of 300 meters high with volumes of manymillions of cubic meters of fill. Development of soil mechanics, study of behavior of earthdams, and the development of better construction techniques have all been helpful increating confidence to build higher dams with improved designs and more details thatare ingenious. The result is that the highest dam in the world today is an earth dam. Thehighest earth/rock fill dams in the world are Roguni U.S.S.R (335 m) Nurek, U.S.S.R.(300 m) Mica, Tehri India (260 m) Canada (244 m) and Oroville, U.S.A. (235 m).Thousands of embankment dams exceeding 20 meters in height have been constructedthroughout the world. Currently, China is the leader in embankment dam construction (USSD, 2011).

For a realistic design of an earth dam, it is necessary that the foundation conditions and materials of construction be thoroughly investigated. It is also necessarythat controlled methods of construction are used to achieve necessary degree ofcompaction at predetermined moisture.

The embankment dam is popular because:

❖Materials available within short haul distances are used;

❖The suitability of the type to sites in wide valleys and relatively steep sidedgorges alike;

❖Subject to satisfying essential design criteria, the embankment design is extremely flexible in its ability to accommodate different fill material e.g. earth fill and/or rock fill if suitably zoned internally;

❖Properly designed, the embankment can safely accommodate an appreciable degree of settlement deformation without risk of serious cracking and possible failure;

❖The embankment dam can accommodate a variety of foundation conditions, and;

❖Often, the embankment dam is least costly when compared to other dam types.

Depending on the available materials at the proposed dam site, embankment dams can be either earth fill or rock fill. An embankment may be categorized as an earth fill dam if compacted soils account for over 50% of the placed volume of material. On the other hand, if over 50% of the fill material is granular then it is classified as rock fill. I.e. coarse-grained frictional material, the ideal choice will be rock fill type. In conditions of high dams such as Lower Awash high dam, the dam body must be constructed as heterogeneous fill, having impervious and pervious zones. The impervious zone may be either on the upstream face or within the embankment, in which case it is called core and it may be made of masonry, concrete, geomembrane, steel sheet piles, timber or other material. The shell zone, which is made of granular materials, is provided mainly to support the impervious zone. Present embankment dam design practice preserves both principles. Compacted fine-grained silty or clayey earth fill, or in some instances manufactured materials, e.g. asphalt or concrete, are employed for the impervious core. Subject to their availability, coarser fills of different types ranging up to coarse rock fill are compacted into designated zones within either shoulder, where the characteristics of each can best be deployed within an effective and stable profile.

2.2 Concrete Faced Rock Fill Dams (CFRD)

Concrete faced rock-fill dam is the type of dam that has a dam body of either rock-fills orgravel fill that is compacted in layers and an anti-seepage system of concrete face slab. The facing concrete slab acts as an impervious layer while the rock-fill body with a highpermeability, gives support to the concrete face slab and the overall stability of the dam.The main advantage is utilization of less volume of rock fill consequently considerableeconomy is saved. Some of the disadvantages are design for leakage through openedjoints and tension cracks, large compression cracks can occur for considerable height innarrow valleys, needs competent foundation, and cannot provide storage duringconstruction. As in the case of Lower Awash dam site, the foundation condition shows thatfresh and very sound rock is located at a maximum depth of 15m; use of upstream faced concrete would require removal of the foundation material from the dam seat area and use of a non-compressible rock fill shell.

The first dumped rock fill dam with a concrete face slab, Chatworth Park in California constructed in 1895, was the first of the American CFRD series followed by 84 m high Dix River in Kentucky and 100 m high Salt Springs Dam which had been in service since 1931 in California (ICOLD, 1989).

The development period of rock fill can be divided into three main stages such as; early, transition and modern periods and CFRDs evolved from traditional design of early period to present design of modern period(Kleiner, 2005-b).

The early period started with the gold miners in 1895 and wooden or concrete face dumped rock fill dams were commonly constructed until1940‟s. Operating dams were suffering significant leakage problems caused by the unbearable amount of joint movements resulting from high compressibility of dumped rock fill. During the construction and the first impoundment, dumped rock fill that was underlying and supporting the face slab, were compacted and settled under gravity and reservoir loading, guiding the face slab to deform in the same trend. Deforming grid of vertical and horizontal joints of face slab provided the leakage way (ICOLD, 1989-a). Even in some occasions, articulated structure of faces lab could not tolerate the rock fill settlement and cracked yielding an increase for leakage way. Despite the fact that there had been no stability and safety problems in CFRDs suffering from leakage, this type of dams became unfeasible because of the remedial operation costs required to prevent unavoidable leakage.

A transition period started as the Earth Core Rock fill Dams, ECRDs, came into the scene by 1940‟s. Highly compressible dumped rock fill was admitted to be more compatible to the earth core and its filters. The difficulties of impervious material supply increased the construction cost of ECRDs but the remedial activities required for excessive leakage on operation of CFRDs made the ECRDs first choice of engineers until the mid of 1950‟s (ICOLD, 1989-a).

Since mid-1950‟s by the invention of vibratory roller, compacted rock fill as a result of developing technology and improving construction techniques, modern period started and CFRDs came back as an alternative for most of the sites (Kleiner, 2005-b). Design of CFRDs is empirical and intensely based on precedent, thus keeping the inherent safety features of traditional design, there is an ongoing progress in design features and construction technology challenging design engineers for further developments to achieve successfully operating higher dams (Cooke,2000-a).

2.2.1 Development of Modern Concrete face rock fill dams

Following the successful performance of first trial on Quoich Dam in 1955, compacted rock fill is accepted to be an efficient material for concrete face rock fill dams. However, vibratory roller compactors used to be a brand-new technology which was very costly to afford, hence it was after 1960‟s that compacted rock fill started to be commonly used for construction of higher concrete face rock fill dams (ICOLD, 1989-c).

Other than compaction of rock fill, traditional design goes under many changes but three main features are kept with small revisions. i) The cut-off wall is taken over by a toe slab called plinth, ii) main structure of rock fill is revised by compaction and appropriate zoning with an increasing size gradation towards downstream and iii) the reinforcement ratio decreases as the face slab got thinner and the details of vertical joints improved against openings while establishment of horizontal joints are avoided unless necessary.

2.2.2 Current Design Characteristics of Concrete face rock fill dams

Current design of CFRDs consists of three inherited primary elements; face slab, plinth and the zoned rock fill also have some secondary elements, such as parapet wall and extruded curbs. Current design characteristics of a conventional CFRD constructed on an appropriate type rock foundation are given in Figure 1.

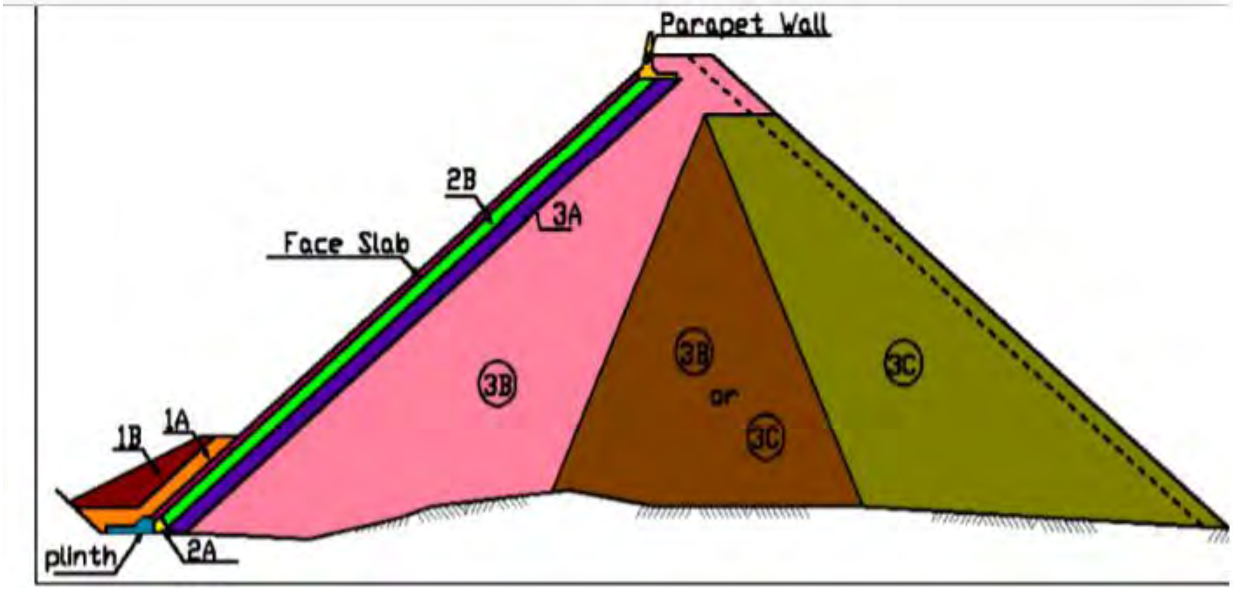


Figure 1: Current cross-section of conventional CFRD (Kleiner, 2005-C)

1A-Cohesion less fine material zone	3A-Transition Zone
1B –Random fill zone	3B-Rock fill zone
2A-Perimetre filter zone	3C-Rock fill zone
2B-Filter support zone	

A. Plinth

Plinth, which connects face slab to the foundation preventing seepage through, is the modern design version of the cut-off trench. It also serves as concrete cap for grouting applied on the underlying foundation. Once the layout alignment is determined, design of plinth cross-section concerns about the selection of width, thickness, confirmation of stability under reservoir loading and impermeability treatment of the foundation.

Plinth segment located on the riverbed is called horizontal plinth because of the levelled foundation while tilted plinth segments on the abutments are called sloping plinths.

The experience of design thickness between 0.3 – 0.4 m was satisfactory in recently constructed dams. The maximum value determined for the river cross-section, is applied as a constant thickness all along the perimeter joint for simplification of construction (Cooke, 2000-b).

Thicker cross-sections are formed across very erodible-fissured formations requiring over excavations or a fault zone, through acute depressions of rock surface, road cut excavations or any other irregularity of rock surface causing abrupt local dents (ICOLD, 1989-c). Plinth of Mohale Dam (145 m) in Lesotho, the largest of South Africa, Shiroro Dam (110 m) in Nigeria is another example for usage of regional thicker plinth cross-sections, the reason of which is the existence of a fault zone and weak zones weathered deep to even 10-15 m in patches throughout the plinth alignment (Kleiner, 2005-d). Yedigöze Dam (140 m) in Turkey currently under construction has some thick plinth cross-sections because of the irregularity of the fresh rock surface of foundation.

Plinths mounted on thick concrete backfills require special design procedure. Plinth slab is fixed to the underlying concrete and work as a monolithic structure. Stability analysis is a casual procedure to be conducted which is not influent on overall design of a conventional plinth placed on a sound rock, but it is a critical step for thicker plinths (Cooke, 2000-b).

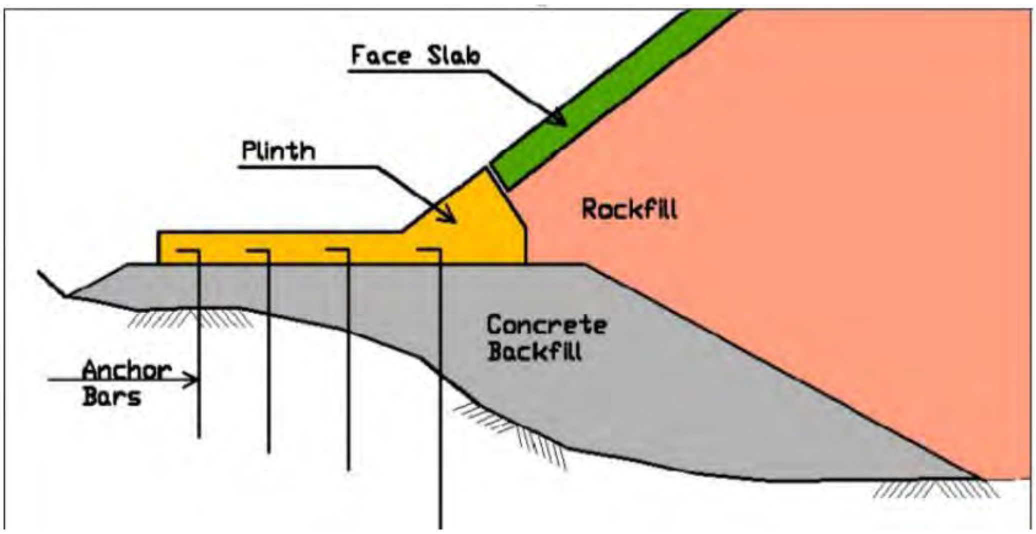


Figure 2: Typical cross-section of thick plinth (Kleiner, 2005-d)

There are many CFRDs constructed on deep alluvial soils, such as Puclaro Dam (80 m) in Chile founded on 113 m deep alluvium and Santa Juana Dam (113 m) again in Chile of 30 m deep alluvium.

The main purpose of the perimeter joint is to connect the toe slab that is fixed to the underlying foundation, to the face slab whose deformation is dependent on the underlying rock fill embankment. The connection of the perimeter must be qualified for imperviousness under maximum water head of reservoir loading and for safely tolerating the differential deformations of the face slab and plinth (Hedien, 2005-a).

B. Face Slab

For concrete face, rock fill dams face slab, which is normally 95~99% submerged in reservoir for operation conditions, constitutes the main part of the water barrier by being exposed to reservoir water directly. Disappointing leakage performance of precedent CFRDs put face slab on the focal point of revision studies. Anticipated displacements and deformations are given in Figure 3-A, B and C for dam cross-section, in the plane of face slab and relative to the plinth respectively.

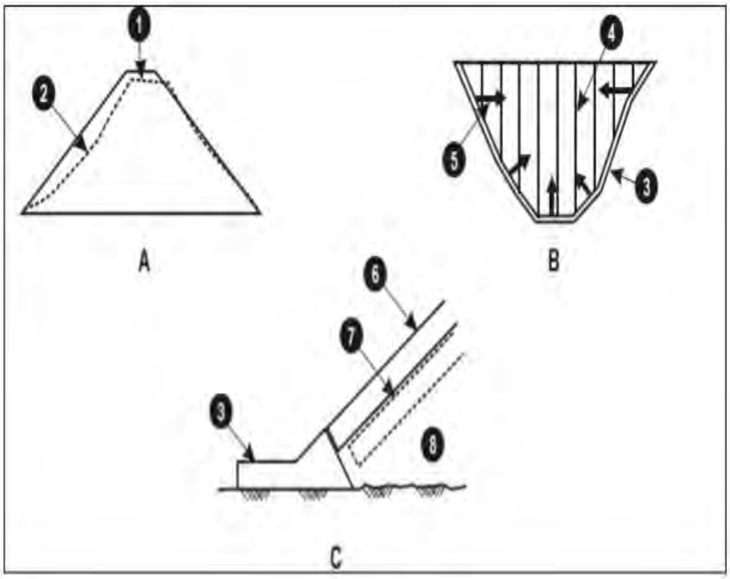


Figure 3: Embankment and face slab behaviour (Hedien, 2005-b)

A) Embankment deformations under water load
B) Movements in the plane of face slab
C) Face displacement at perimeter joint
1. Crest settlement 2. Face settlement
3. Plinth 4. Face joints
5. Direction of movements 6. Face slab
7. Face position after water load8. Rock fill

Concrete slab covering the upstream slope consists of main vertical panels, which are constructed with slip forming. The width of the panel mainly depends on the equipment characteristics.

C. Parapet Wall

Parapet wall application on the crest is one of the main advantages of concrete face rock fill dams, which significantly reduce the rock fill embankment volume especially for high dams. This economic saving, cannot be disregarded if the embankment material is supplied from rock quarry instead of using excavated materials. In common, the parapet wall compensates design practice, flood volume. Even though CFRDs are not as vulnerable as ECRDs against overtopping, top elevation of parapet wall is determined in order to prevent overtopping during probable maximum flood (Sundaram and Kleiner, 2005). In Figure 4 below parapet wall details of Mohale Dam (145 m) in Lesotho are given.

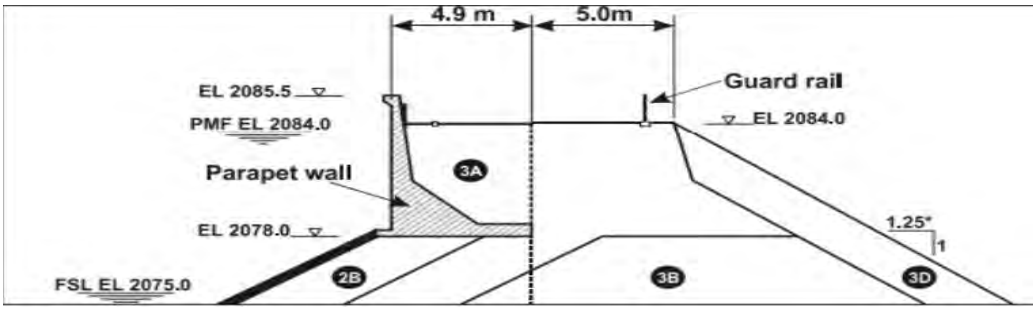


Figure 4: Parapet wall of Mohale Dam (Sundaram and Kleiner, 2005)

2.2.3 Performance of Precedence Concrete Face Rock Fill Dams

2.2.3.1 General

Concrete Face Rock fill Dam design is empirical in nature and is based on the precedent performance and experience (Cooke, 2000-a). However, adding to the precedent experience, invention of new construction equipment"s and development of construction technique supports the design progress of CFRDs, such as invention of vibratory roller compactor, slip form and extruded concrete curb equipment. Concrete Face Rock fill Dams have some inherent safety features, enabling the empirical design and allowing new design trials, which are (Cooke, 2000-a):

-The zoned rock fill allows flow passing through the dam as free draining without slope failure.

-The whole embankment volume is not directly exposed to the reservoir water.

-Under normal operation conditions, pore pressure is not anticipated but in case of any leakage through face cracks the rock fill is zoned and designed for self-drains and leakage is not critical for dam stability and safety

-For satisfactory operation of grout curtain, uplift pressures are not involved for concrete face rock fill dams since embankment is not in direct contact with the reservoir water

-Rock fill behind the face slab is very stable under high seismic loading

-Shear strength of the rock fill is very high and steeper slopes are safely adopted for various concrete face rock fill dams.

2.2.3.2 Post –construction settlement

Available methods (Sowers et al 19365, Parkin (1977), Soydemir and Kjaernsli (1979), Clements (1984), Pinto and Filho Marques (1985) , Sherard and Cooke (1987) , Hunter (2003) , Hunter and Fell (2002) for prediction of post- construction crest settlement of rock fill embankments are empirical and are generally based on historical records of similar embankment types and similar methods of construction.

An important aspects of the deformation behaviour of rock fill related to its stress- strain characteristics is that relatively large deformations occur on application of stresses above these not previously experienced (such as due to fluctuation in the reservoir level) the rock fill modulus is very high and resultant deformation limited. Collapse deformation on initial wetting can result in relatively large deformations.

Consideration for the post- construction deformations of rock fill in CFRDs therefore include:

- Events where stresses are likely to exceed these previously experienced most notably first filling.
- Ongoing, time dependent (or creep) deformation of rock fill.
- Collapse type deformations due to wetting from leakage or tail water impoundment.

For embankments constructed of rock fills of medium to high intact rock sources , the total magnitude of settlement at 10 years is on average approximately twice that of quarried, very high strength rock fills. The long –term creep rate is also significantly higher for the weaker rock fills 2 to 10 times that of quarried, very high strength rock fills.

For the well compacted gravels (Crotty and Golillas), the post- construction deformation is less than that for the well-compacted, very high strength quarried rock fills. This is likely to be due to several reasons, but is considered to be mainly because of the rounded shape of the gravels. The point area of contact between particles of rounded gravel will be significantly greater than that of angular quarried rock fill. Hence, the contact stresses will be significantly less resulting is less particle breakage.

2.3 Theoretical review of earthquakes

2.3.1 Origin and Classification of earthquakes

Anearthquakeismainlyaresultofareleaseofstrainenergybyaruptureofrockatplateboundaries.Thestrainenergystoringprocessisaresultofplatetectonics.Plate tectonics(platemovement)isalarge-scalemotionoftheearth‟slithosphere.Energy released radiates in all directions from its source (focus) and propagates in the form of seismic waves. The release of the accumulated strain energy by the sudden rupture of the fault is due to earthquake shaking and shown below.

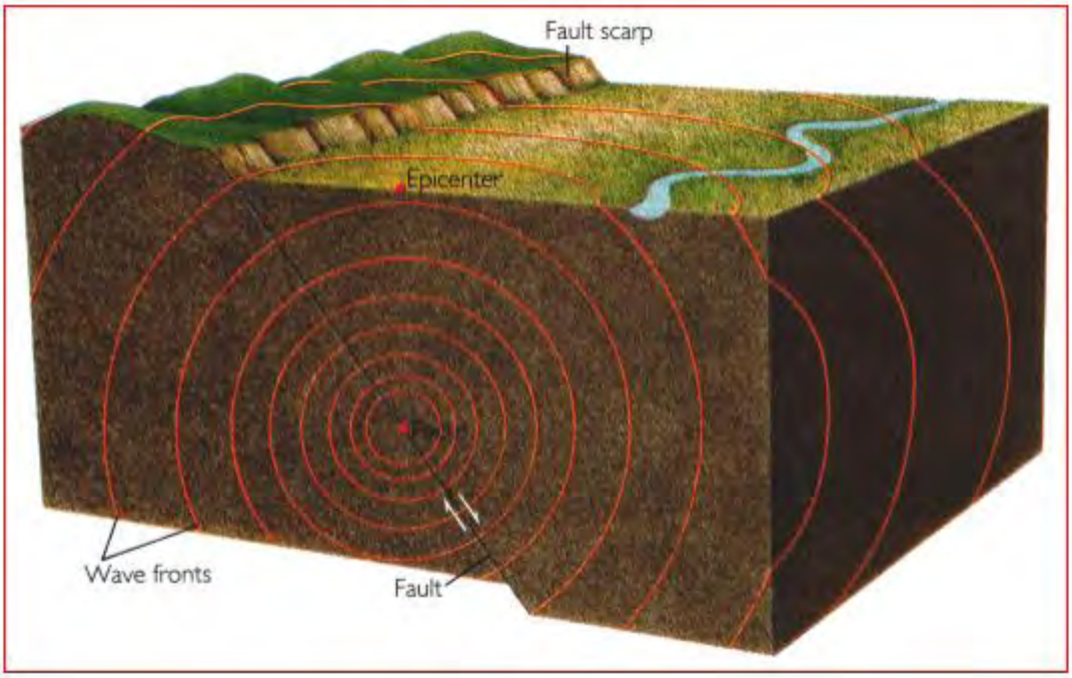


Figure 5: The release of the accumulated strain energy by the sudden rupture of the fault

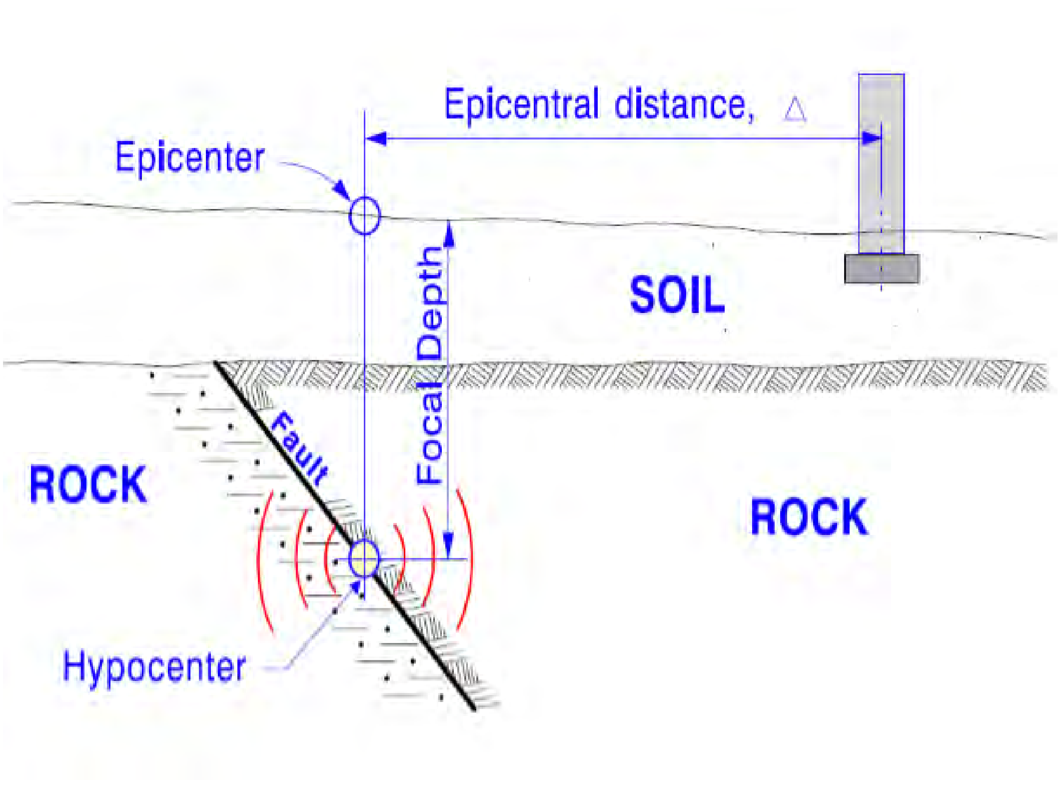


Figure 6: Hypo central and Epicentral distance from the proposed site

AFocusorHypocenteristhepointofplateboundarieswherestrainenergyisreleased. Epicenterisapointonthesurfaceverticallyabovethefocuspoint.Epicenterdistance andHypocenterdistancearedistancefromtheobservationpointtotheepicenterand hypocenter, respectively.

When an earthquake occurs, differenttypeofseismicwavesisreleased.Bodywaves and surfacewavesarethemaincategoryofthesewaves.Bodywavesarereleasedfrom thehypocenter.Underbodywaves,thetwomainwavesareprimarywavesand secondarywaves.Primarywavesarefastandhaveacompressioneffecton the ground.Secondarywavesareslowerinspeedascomparedtotheprimarywaves,and haveashearingeffect.Surfacewavesarewavesemanatingfromtheepicenterlikea waterwaveemanatinginaponddueto athrownstone.Theyoccurbecauseofinteractionbetwee nbodywavesandthesurfaceandlayersoftheearth.Theireffectis swaying and rolling the ground.

Earthquakeresultswhenthestresseswithintheearthbuildupoveralong perioduntiltheyexceedthestrengthoftherock,whichthenfailsand displacementalongafaultresults.Onceruptured,thefaultisaweakness, which can resultan earthquake again in the future.

As a result, if the project is located in highly active seismic zone earthquakes may affect embankment dams in various ways. Seismic forces may be transmitted directly from the foundation to dam.

2.3.2 Size of earthquake

Sizeofanearthquakecanbedefinedbyitsintensity,magnitudeandenergy. Earthquakeintensityisanobservationalevaluationofearthquakefromitsdamage resultandintensitytobefelt.Earthquakemagnitudeisrelatedtothemotionresulted fromearthquake.ThemostknownmagnitudedefinitioncomesfromCharlesRichterin 1935.Hedefinesmagnitudeasthelogarithmofthemaximumtraceamplituderecorded onaWood-Andersonseismometer, locatedat100kmfromtheepicenterofthe earthquake.Earthquakeenergyisthetotalenergyreleasedduringitsoccurrence.Inmost equations developed, it is relatedwith the earthquake momentmagnitude.

Whencometothemotionontheground createdbytheearthquake,thethree characters, which define earthquake motion, are:

- Amplitude
- Frequency
- Duration of motion

Theamplitudeisthemaincharacterofdefinition.Itisameasureofacceleration, velocityordisplacementofthegroundbecauseoftheshake.Ifoneofthese parametersisrecordedintheformoftimehistorydata,theotherscanbecomputed. Fromtheaccelerogramofthecomponent,thelargestvaluesofhorizontalacceleration areobtainedforpseudo-staticanalysis.However,verticalforcesandaccelerationareignored mostlybecausethestaticsafetydesignissufficienttoreactagainstdynamicinstability inthisdirection.Thefrequencydescribeshowtheamplitudeofthegroundmotionis distributed in different frequency. Since loads are cyclicin earthquake, the inverse of the timetakentoaccomplishonecycle,canprovidethefrequencyofthemotion.The durationofthemotionisthetimeorperiodoftheearthquake.Inloosesaturatedsand porewaterpressuredevelopmentuntilliquefactionoccurrenceisnotonlyafunctionoftheamplit udeofthecyclicload,butalsothedurationofthecyclicload.Strong amplitudeearthquakemightnotresultdamageifitsdurationisshort.

Astheareaand length of fault rupture increases, the durationof the motion increases. The shaking strength of an earthquake depends up on:

- Sizeofthe earthquake
- Location of the earthquake
- Sizecharacter ofthe earthquake.

2.3.3 Dams and earthquake load

Ifanyfaultcrossesadamaxisbeneath,designershouldtakeintoaccountamotion andloadofearthquakeforthedamdesign.Theloadinganddamcharacteristicsthatmakeseism icpotentialfailuremodestooccurmorelikelyarelistedbelow.(ICOLD,2001)

Peak Horizontal Acceleration> 20% acceleration of gravity Capable fault beneath the embankment (Active fault which have received movement in the last 10,000 years)

- Hydraulic fill embankment
- Sand embankment
- Loose saturated alluvial foundation
- Fine-grained soils susceptible of cyclic failure
- Thin impervious cores
- Thin filter zones
- Conduit embedded in embankment
- History of seismic damage
- Earth embankment-concrete section interface

Inevaluationofseismicstabilityinconcretedams,thefocusesareanalysisonitsability toresistinducedlateralforcesandmovementsandexcessivecrackduetooverstress ontheconcrete.Onembankmentdams,thefocusesareporewaterpressure development,excessivedeformation,slumpingsettlement,crackingandplanaror rotationalfailureofembankment.Embankmentdamshaveavibrationrangebetween 0.5-

1.5seconds.(ICOLD,1989).

Duringearthquake,thepreviousloadingandstressconditionoftheembankment materialwillbechanged.Thenewloadmodewillbedynamicwhichchangesstressstatethat might result shear or tensile failure of material. (USBR, 2012)

Themainbroadissuesthatneedtoberesolvedinassessingtheseismicperformance of earth dams under earthquakesare:

- Stability:- This deals with the capabilityor thestrength ofthe dam against collapse.
- Deformation:-Thisdealswithitsserviceabilityperformanceafteraccommodating some damage. (Visible or invisible)

Indealingwithearthstructuresduringearthquakethemajorcategoriesofissuestobe considered are:

- The motion, movement and inertial forces that occur during the shaking,
- Thegenerationofexcesspore-waterpressures,thepotentialreductionofthesoil shears strength,
- Theeffectonstabilitycreatedbytheinertialforces,excesspore-waterpressures and possible shear strength loses, and
- Theredistributionofexcesspore-waterpressuresandpossiblestrainsofteningofthe soil after the shaking has stopped.

To make it more clear and practical,the four mainissues to consider in earthquake are:

- Thegeneraldesignofthedamparticularlytheprovisionoffilters,topreventor controlinternalerosionofthedamandfoundation,andprovisionofzoneswith good drainage capacity. (E.g.,free draining rock fill).
- The stability of embankment duringand immediatelyafter the earthquake.
- Deformationsinducedbytheearthquake(settlement,cracking)anddamfree board.
- Thepotentialforliquefactionofsaturatedsandyandsiltysoilsandsomegravel withasandandsiltmatrixinthefoundation,andpossiblyintheembankment,andhowthisaffectsstabilityanddeformationsduringandimmediatelyafterthe earthquake.

2.3.4 Static and Dynamic loading

Instaticloadingafterassessingthestrengthofthestructurebycomparingitwith externaldestabilizingforce,themajorconcernistoevaluatethefactorofsafetyagainst failure.Failureinsoiloccursatafewpercentofstrainlevel.Therefore,astaticproblemdeals withafewpercentofstrainlevel,whichcanoccurduetocompressionorconsolidation. Thestrainlevelisinorderof10^{-3}orgreater.Indynamicproblems,thesoilisinmotion andthelargeimpactofinertialforceduetovelocitychangeisconsidered.Asthe durationoftimefordeformationsbecomeshorter,theroleofinertiabecomeslargerand larger.Insuchacyclicmotionevenifthestrainlevelissosmall,theinertialforcewill increaseinproportiontothesquareofthecyclicfrequency.Duetothisfact,uptoa strainlevel of 10^{-6}such consideration must be given for dynamic loading.

The other issue differentiating dynamic problem with that of static is the rapidity of load. Problem where the load is applied for more than tens of seconds is cited as static. If the load application time is less than this, the problem is of dynamic type. In addition, of the time of application, the other factor differentiating dynamic and static problem is the loading repetition in dynamic loading. The period of impulse in earthquake is from 0.1-3 seconds.

Instaticloadingsincewedealwithastrainlevelof10^{-3}orgreaterwhichisafailure state,thedeformationcharacteristicisnotdependentonshearstrains.However,indynamic loading,thedeformationcharacteristicisdependentonshearstrain.Thebelowtable showtherelationbetweenstrainlevelandmechanicalproperties,withexpected phenomenato occur.

Table 1: Variation ofsoilpropertieswithstrain

10^{-6} 10^{-5} 10^{-4}	10^{-3}	10^{-2}	10^{-1}(strain)
Wave propagation, vibration		Cracks, differential settlement	Slide,compaction, liquefaction
Elasticity		Elasto-plasticity	Rupture/Fracture

In many practical cases, the ground response under seismic loading is evaluated using the well-known equivalent linear method in which compatible values of shear modulus and damping ratio are chosen according to the shear strain level in soil deposit. In this simplified approach, the developed pore water pressure and the residual soil displacements cannot be calculated. However, if a given problem do not involve large strains ($\gamma > 10^{-2}$) the equivalent linear method can be considered acceptable in a practical point of view. This work deals in the range of small to medium strain levels ($10^{-6} \leq \gamma \leq 10^{-2}$).The use of this method of analysis needs reliable strain-dependent shear modulus and damping ratio curves.

In this paper, only the factors affecting the shear modulus of soils are treated and discussed in detail. The text is essentially divided into two parts. In the first part, the stiffness of soil at small strain level ($\gamma \approx 10^{-6}$) is analyzed.

In the second part, the investigation proceeds with the study of the shear modulus degradation at higher strain level until 10^{-2}.

2.3.5 Initial Shear Modulus

In recent years, many studies were performed to investigate the behaviour of soil at small strain level. The initial shear modulus G_0 (for $\gamma \approx 10\text{-}^{6}$) is a very important parameter not only for seismic ground response analysis but also for a variety of geotechnical applications.

A considerable number of empirical relationships have been proposed for estimating initial shear modulus for different kind of soils: [Hardin and Black, 1969], [Iwasaki and Taksuoka, 1977], [Marcusson and Wahls, 1972; Kokusho, 1972; Kokusho and Esashi, 1981 and Nishio et al., 1985 in Ishihara, 1996] and [Biarez et al., 1999].

All of these relationships are based on two experimental evidences: the shear wave velocity (vs.) is a linear function of void ratio (e) and depends on the mean effective stress (p') with a power of n/2, as proposed originally by [Hardin and Richart, 1963]:

2.3.6 Shear Modulus Reduction Factor

In recent years, due to the improvements in laboratory testing (local deformation measurement and stress-path control tests), many reliable experimental data has accumulated allowing a considerable advance in the knowledge of the stress-strain behaviour of soils.

It has been demonstrated that the behaviour of soils can be described using the concept of kinematic regions in stress space. In the simplify scheme proposed by [Jardine, 1992] the current effective stress point is surrounded by two sub-yield surfaces Y1 and Y2. Inside the surface Y2 (Zone II) the response is non-linear but fully recoverable (elastic). The response is linear elastic in that inside Zone II a small region (Zone I-inside surface Y1) can exist. The surface Y2 defines the threshold at which drained or undrained cyclic loading starts to affect the soil significantly. When the Y2 surface is reached significant plastic straining begins to occur until the Y3 yield surface (Zone III).

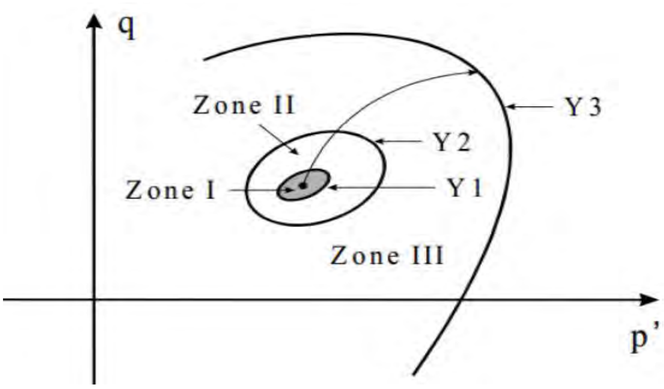


Figure 7: Scheme of multiple yield surfaces (Jardine, 1992).

2.2.7 Seismic activity in Ethiopia

In East Africa, there are three main zones of seismic weaknesses in the crustal segments: the East African rift system, the Gulf of Aden, and along the Red Sea (Haile, 2004). These three zones make up the Afar Triangle (Ayele & Kuthanek, 2007). Movement in these zones has made the region an active tectonic and volcanic zone (Haile, 2004). In Ethiopia, 90 per cent of the seismicity and volcanic activity is related to the East African rift system. The East African Rift System is a 50km to 60km wide zone of volcanoes and faults that extend north to south in Eastern Africa for more than 3000km (1864 miles), from Ethiopia in the north to the Zambezi in the south. It cuts through Ethiopia in a NE-SW direction (ibid; Ayele & Kuthanek, 2007).

The active Great Rift Valley makes Ethiopia susceptible to two types of seismic hazard: earthquakes and volcanic eruptions. As a landlocked country, it is not at risk from tsunamis. Using data from one of the best known disaster databases – the EM-DAT database – Table 3 shows that from 1900 to 2013 there were a total of ten earthquakes and eruptions – leading to a total of 93 deaths, 165 injured, 420 homeless and affecting 11,000 people. These are estimated to have an economic cost of more than US$7 million (Siân Herbert, 2013).

It is important to note that data in these areas is often limited and is expected to be highly under reported.

Table 2: Number and impact (human and economic) of earthquakes and volcanos in Ethiopia (1900-2013)

Disaster type	Number of disasters	Number of people killed	Number of people injured	Number of people affected	Number of homeless	Total number of people affected	Total economic damage (US$ '000)
Earthquake	7	24	165	0	420	585	7070
Volcano	3	69	0	11000	0	11000	0
Total	10	93	165	11,000	420	11,585	7,070

Source: Data downloaded from EM-DAT database on 17 January 2013. [7]

Other sources of data and literature report different figures – for example Kinde (2002) explains that Gouin estimates 15,000 tremors occurred in Ethiopia and the Horn of Africa in the 20th century, while another study by Kebede identified a total of 16 recorded earthquakes of magnitude 6.5 and higher in Ethiopia in the same period.

The seismic hazard map is divided into 5 zones, where the ratio of the design bed rock acceleration to the acceleration of gravity.

Table 3: The value of bedrock acceleration ratio for sites in different seismic zones of Ethiopia.

Zone	5	4	3	2	1	0
$\alpha_o = a_g/g$	0.20	0.15	0.10	0.07	0.04	0

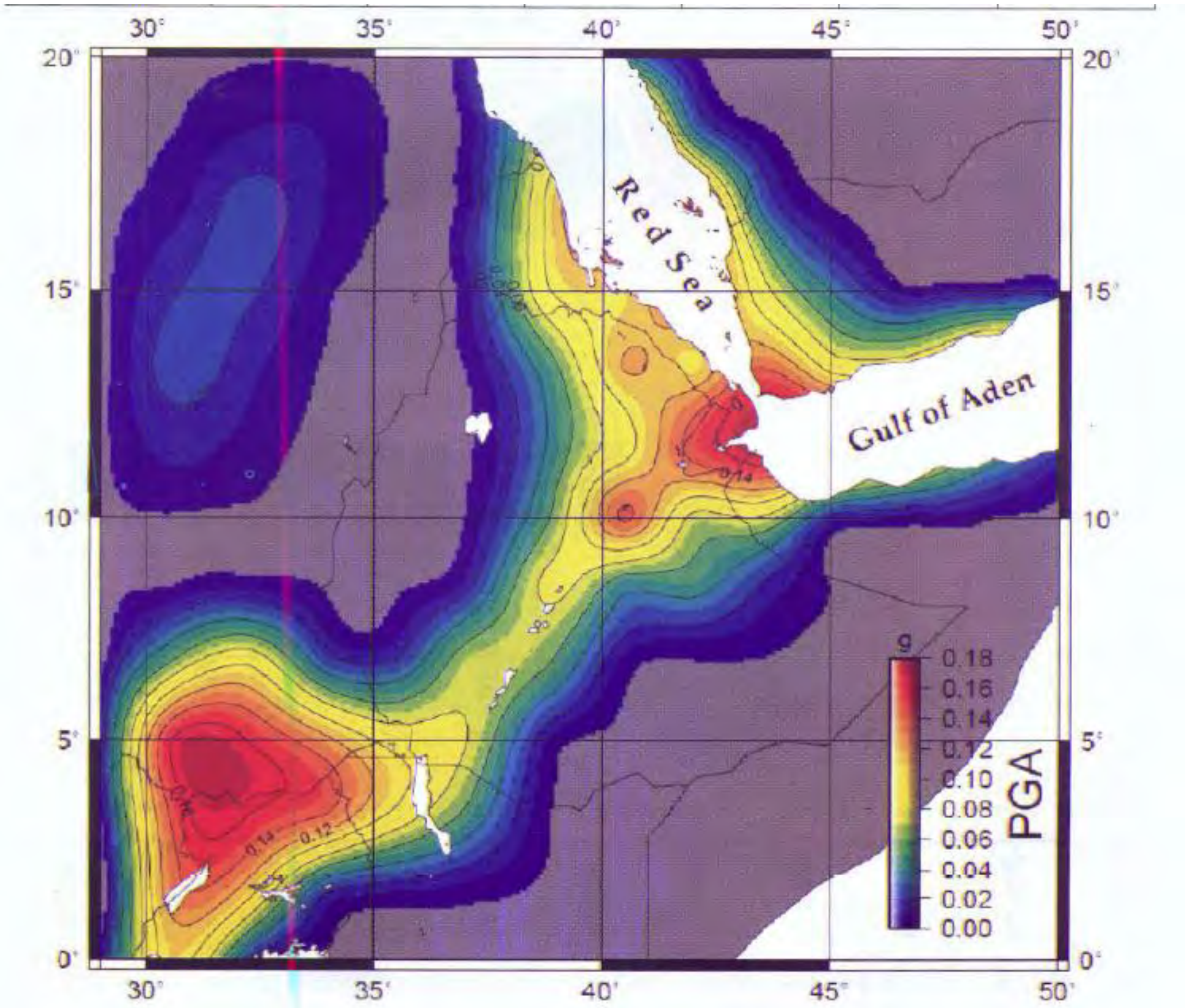


Figure 8: Seismic hazard map along the horn of Africa

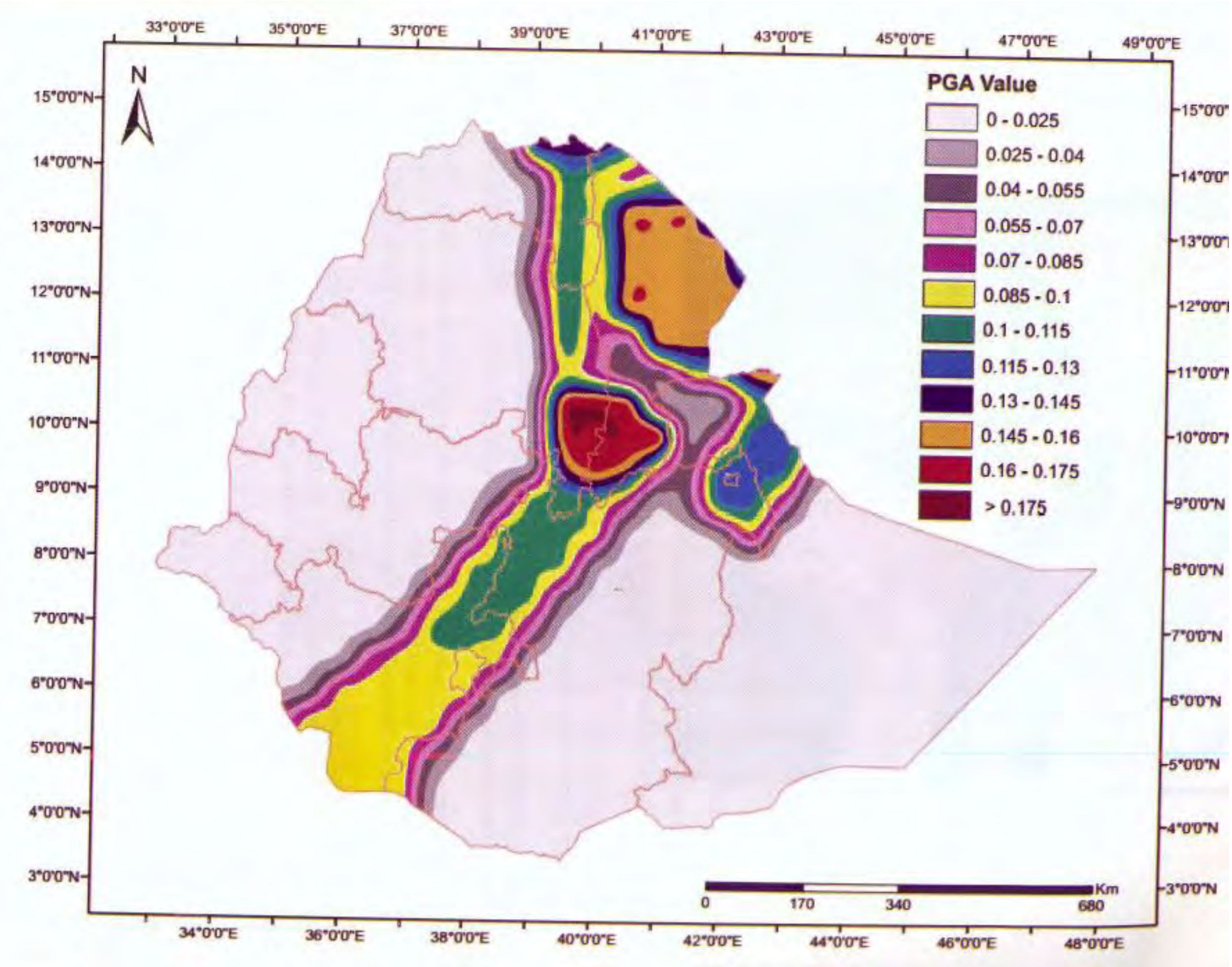


Figure 9: Seismic zone map of Ethiopia in terms of peak ground acceleration (EBCS draft report, 2014)

The seismic hazard can be defined as the probability that a ground motion at a specific site will be exceeded during a time period, i.e. return period, T (years). Alternatively, it is the probability that a ground motion will be exceeded with an annual frequency of 1/T. In general, seismic hazard due to natural earthquakes is, rather haphazardly, presented as the probability that a PGA will be exceeded in T= 475 years (Bommer and Pinho, 2006; GSHAP, 1999). An alternative interpretation is a 10% probability that this PGA will be exceeded in 50 years.

The Lower Awash Multipurpose Dam project sites are located at the most active rift floor or zone 5, in Ethiopia. Hypo-central depths of well-constrained events are 5–7 km from modelled earthquakes in the Afar region, which is the approximate elastic plate thickness in Afar and the main Ethiopian rift, possibly indicating the depth to the brittle–ductile transition zone in this part of the rift. Theshallownessofthedepthestimates agreewiththemacro-seismicreportsavailablefromawideareareportedforHosanna andYirgalemearthquakesin SouthEthiopia.(WWDSE,2016).

The design ground acceleration for each seismic zone corresponds to a reference return period of 100 years. To this period, an importance factor equal to 1 is assigned.

Peak ground acceleration:-The most commonly used measure of amplitude of a particular motion is the peak horizontal/ground acceleration (PHA/PGA).Proper design of earthquake resistant structures or facilities requires estimation of the level of ground shaking, to which they will be subjected, whose level of shaking is most commonly and conveniently described in terms of PGA.

Inevaluatingsoilmaterialsunderearthquakeload,determinationofdynamicproperties ofsoilisafundamentalpartofthesolution.Whenthestrainlevelislessthan10^{-6},` it indicateselasticpropertiesrepresentedbywavepropagation.Forthesakeofsimplicity inapplyingmathematicaltheoryofelasticity,soilisgenerallyconsideredasalinear massandwithsuchassumptions,thedynamicsoilpropertiescanbetheoretically handledeasily.Uptoastrainlevelof10^{-6},beyondwhichthenon-linearbehavior becomesprominent,anapproximateanalysisiscarriedoutusingthesocalled „equivalentlinearizationmethod‟,whichtakesintoconsiderthechangesindeformation coefficientanddampingratio.Recently,soilhasbeenrepresentedbyanElasto-plastic model to enable the realistic analysis of failure phenomenon.

Thesoilpropertiesthatinfluencewavepropagationandotherlowstrainphenomena includestiffness,damping,poisonratioanddensity,inwhichthefirsttwoarethemostinfluential. Themainmechanicalcharactersofanearthmaterialunderdynamicloadare dynamicstress-strainrelationshipanddynamicshearstrength.Thesemechanical charactersarehighlydependentoninitialrelativedensityordegreeofcompaction, degree of saturation, and rate of loading.(P Bertacchi, 1981).

Thismechanicalcharacterofthematerialsneedsspecialfocusbecauseofthebelow tworeasonsthathappensduringearthquakeloading.(M.NoseandDr.EngK.BABA,1981)

- Permanentshearstrainsoccuraftereachcycle ofstressapplication andthat
- Permanentvolumecontraction (densification)occursaftereachcycleofloading (except for extremely dense material).

Duringearthquake,thetwobroadcategoriesofdynamicpropertiesaresoilstiffnessand generationofexcessivepore-pressure.(Quake/W,2007).

- Dampingisaresistanceofvibration.Itistheenergyabsorbingcapacityofasoil. Dampingratioisinfluencedbymanyfactors.Dampingratioofhighplasticsoilislower thanthoseoflowplasticitysoil.Dampingratioisalsoinfluencedbyeffectiveconfining pressureparticularlyforsoilsoflowplasticity.Dampingratiodecreaseswithconfining pressure,voidratio,andgeologicage.Dampingratioincreaseswithcyclicshearstrain. Dampingismostlyrate-independentandofhystereticinnature.Ingeneral,forsands, dampingratioincreasewithincreasingshearstraintoavalueof25%oftheinitial damping when the shear strain increases by 0.5%.
- Shearmodulusisaratioofshearstressbyshearstrain.Modulusratio(G/G_O)isaratio ofshearmodulusatatimeandtheinitialshearstress.Theinitialshearstressslopeis alwaysgreaterthanorequaltoanyshearstressatatime.Shearmodulusratioisa dynamicsoilproperty.Itincreaseswithconfiningpressure,voidratio,plasticityindex andgeologicage.Itdecreaseswithcyclicstrainandnumberofloadingcycles.Ingeneral,for sands,theshearmodulusdecreaseswithincreasingstraindowntoaboutone tenth of the initial value, if the strain increases to a level of 0.5%.

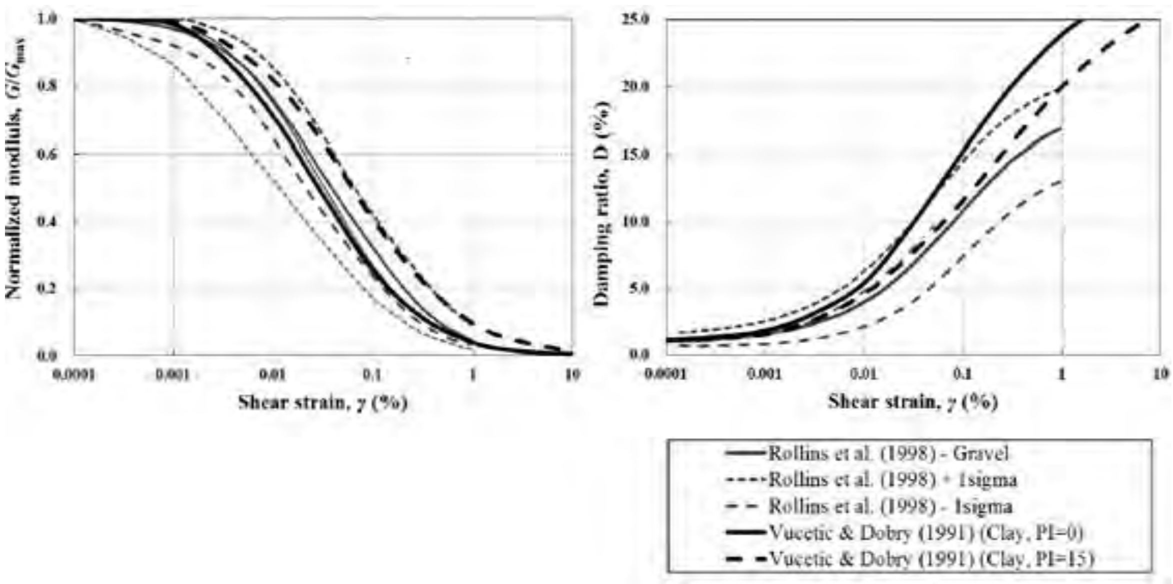


Figure 10: G/Gmax and damping ratio curves dependent on shear strain by different researchers.

2.3.7 Rock-fill and concrete material behavior during earthquake

Rock-filldamshavebetterperformancethanearthenfilldamsduetotworeasons consideringearthquakeload.Thefirstreasonistheflexibilityofsteepeningslopes, makingiteconomicalbyachievingstabilityconditionssettingotherfactorsthesame.Inimpervio ussaturatedsoils,earthquakestressesareappliedunderun-drainedcondition. Thisisachievedwithnodrainageordissipationofexcessiveporewaterpressure.However,inhi ghlyperviousmaterialslikegravelandrockfill,thereisadissipationofporewater pressureeveniftheduratonoftheshakingisforshorttime.Thiseliminatesthe susceptibilityofsuchsectionsandpartsofthedamforliquefactionpotential.(P Bertacchi, 1981).

Coming to concrete material, when identifying the dynamic behavior of concrete, we are confronted with a series of typical problems for each high-speed behavior identification test. Some of these problems are increased by the nature of concrete, which is the reason why we prompt the reader to be very cautious when using experimentation signals or results. Due to its structure in aggregates, where it is mixed with sand and hardened cement paste, concrete can be a highly heterogeneous material, and the size of a representative sample is not always an easy thing to state.

2.3.8 Liquefaction

It has been learned that solid ground can turn into mush under conditions that are not so rare. Soil that had supported structures before could suddenly become fluid; anything on it can slip or sink. The soil loses its strength and the soil transfers from solid state to liquefied state. Here soil liquefaction is as a result of increased pore water pressure and hence reduced effective stresses due to dynamic loading. When a loosely packed, saturated fine uniform deposit of sand is subjected to a shearing stress at a very fast rate, for example due to earthquake shocks, pile driving, explosive blasting or even rapid draw down in dams, there is a tendency of rapid decrease in the volume of soil. Since the soil volume contraction cannot occur within such a short period, the soil mass behaves like an undrained system. Hence, there is a sudden increase in the pore water pressure and consequently, an equal decrease in the effective stress.

If this reduction is such that the effective stress almost reduces to zero, the soil in that zone will be transferred to a fluid-like mass with hardly any shear strength.

Since soil strength is proportional to the vertical effective stress (σvo'), the reduction of effective stress from increased pore pressures (u) will lead to strength loss in a soil deposit. The pore water pressure in the soil will be a combination of initial in-situ pore water stress (uo) and the shear- induced pore water pressure (Δu). When the pore water pressure ($u = \Delta u + uo$) equals the total overburden stress (σvo), the effective stress ($\sigma vo' = \sigma vo - u$) will go to zero causing initial liquefaction (Seed & Lee, 1966).

Soils composed of particles that are all about the same size are more susceptible to liquefaction than soils with a wide range of particle sizes. In a soil with many different size particles, the small particles tend to fill in the voids between the bigger particles thereby reducing the tendency for densification and pore water pressure development when shaken by various dynamic loads.

Thus a uniformly graded soil is more susceptible to soil liquefaction than a well-graded soil due to the reduced volumetric strain, hence decreases the amount of excess pore pressure that can develop under undrained conditions.

The geologic process of soil deposits that are either fluvial, colluvial or Aeolian deposits tend to sort particles into uniform grain sizes and produce rounded particles. Historically, sands were considered to be the only type of soil susceptible to liquefaction, but liquefaction has also been observed in gravel and silt. Strain softening of fine-grained soils can produce effects similar to those of liquefaction.

Fine-grained soils are susceptible to this type of behavior if they satisfy the criteria shown below (Wang, 1979)

- Fraction finer than 0.005 mm< 15%
- Liquid Limit, LL < 35%
- Natural water content > 0.9 LL
- Liquidity Index < 0.75

The type of geologic process that created a soil deposit has a strong influence on its liquefaction susceptibility. Saturated soil deposits that have been created by sedimentation in rivers and lakes (fluvial or alluvial deposits), deposition of debris or eroded material (colluvial deposits), or deposits formed by wind action (Aeolian deposits) can be very liquefaction susceptible. These processes sort particles into uniform grain sizes and deposit them in loose state, which tends to densify when shaken by earthquakes. The tendency for densification leads to increasing pore water pressure and decreasing strength.

There are case histories indicating that liquefaction has occurred in loose gravelly soils (Seed, 1968; Ishihara, 1985; Andrus, et al., 1991) during severe ground shaking or when the gravel layer is confined by an impervious layer. The space between the two curves farthest to the left reflects the influence of fines in decreasing the tendency of sands to densify during seismic shearing. Fines with cohesion and cementation tend to make sand particles more difficult to liquefy or to seek denser arrangements. However, non-plastic fines such as rock flour, silt and tailing slimes may not have as much of this restraining effect. Ishihara (1985) stated that clay or silt-size materials having a low plasticity index value would exhibit physical characteristics resembling those of cohesion less soils, and thus have a high degree of potential for liquefaction.

Walker and Steward (1989), based on their extensive dynamic tests on silts, have also concluded that non-plastic and low plasticity silts, despite having their grain size distribution curves outside of Tsuchida's boundaries for soils susceptible to liquefaction, have a potential for liquefaction similar to that of sands and that increased plasticity will reduce the level of pore pressure response in silts. This reduction, however, is not significant enough to resist liquefaction for soils with plasticity indices of five or less.

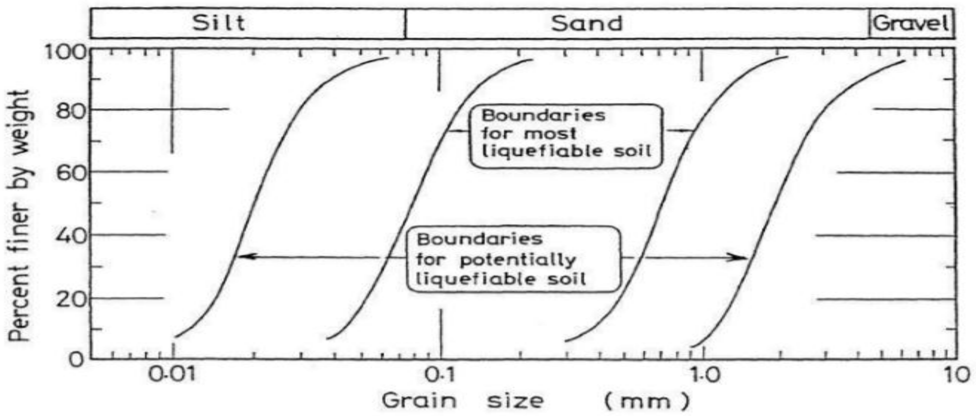


Figure 11: Limits in the gradation curves separating liquefiable and non-liquefiable soils (Tsuchida, 1970).

2.3.9 Deformation Behavior of Rock fills Dams

Rock fill dams are composed of material having particle sizes up to 1 m in diameter. Therefore, it is very difficult to carry out laboratory shear strength tests on rock fill materials. Based on very limited laboratory triaxial test data available in the literature, it is concluded that rock fill material exhibit nonlinear, inelastic stress-strain behavior (Marsal, 1967; Marachi et al., 1972; Duncan et al., 1980, Saboya and Byrne, 1993) as can be seen in Figure 17 below.

To represent this behavior Duncan and Chang"s (1970) hyperbolic model is frequently used in the literature (Ozkuzukiran et al. 2006, Unsever 2007).

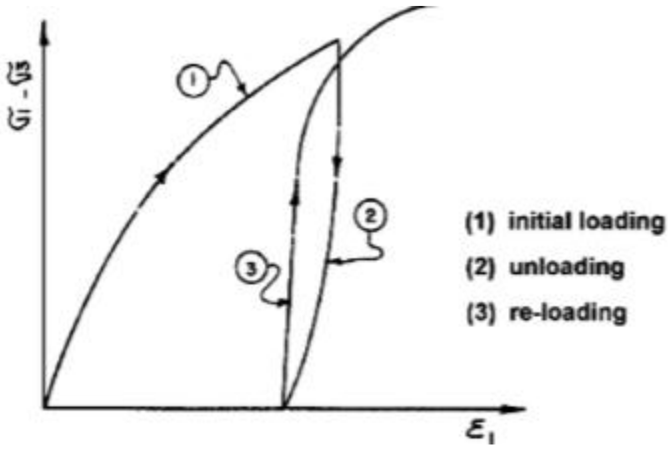


Figure 12: Typical stress-strain behaviour of rock fill from a triaxial compression test (Mori and Pito 1988)

Instead of laboratory tests, it is often more practical to look at the data collected from deformations observed in constructed rock fill dams. In this section, collected data in the literature on the vertical and lateral deformation of rock fill dams, and modulus of rock fill material will be reviewed. Rock fill dams continue to deform long after their construction is completed, although at a decreasing rate. According to Hunter and Fell (2003) compressibility characteristics of rock fill are influenced by:

Degree of compaction of the rock fill, applied stress conditions and stress path, particle shape and particle size distribution, intact strength of the rock and the susceptibility of the rock fill to collapse upon wetting.

U.S. Bureau of Reclamation recommended, for the design of rock fill dams, a maximum crest settlement that is equal to 1%H (plus any deformation due to the settlement of the foundation), for rock fill dams with heights less than 15 m. It is noted in the literature that better compaction and sluicing decreases the crest settlements.

2.3.10 Earthquake time- history data, period and peak ground acceleration modification

In earthquake dynamic analysis, time history data is fundamental but it is also the mostly unavailable data input. Time -history data of earthquake amplitude is the value of the velocity, acceleration or displacement in a constant time interval between records for some period of the earthquake shaking. Time history data describes the set of the three earthquake motion characters in one graph: amplitude, frequency and duration.

The cyclic shear stress in a dynamic analysis is calculated as an inertial force in every time step. To calculate this inertial force oscillating in fraction of seconds during earthquake shaking, the time- history data of acceleration usually in every 0.02 seconds must be provided for the computation. However, in most places including Ethiopia, it is hard to find a recorded time- history data of earthquake. If it was available even in far places from where the dam in dynamic analysis is available, a deconvolution technique (i.e. changing the recorded surface ground motion to the source bedrock motion) can be done and applied to the bedrock under the dam in analysis. However, the recently recorded time- history data amplitude in Addis Ababa is so small that it undermines the probable earthquake to happen. So, following the approach of the Army Corps of Engineers recommendation, for selecting time history data, elsewhere recorded time history data compared considering this points of resemblance with the site, which is on analysis.

- Tectonic environment
- Earthquake magnitude and type of faulting
- Earthquake source to site distance

After selection of the time history data, modifying its amplitude to the design base and maximum credible earthquake amplitude of the dam area is done to make it fit to the site condition.

For embankment dams, the frequency range is 0.5-5 hertz, which means the period, is from 0.2-2 second. Taking the 0.2 sec period ground acceleration will make the analysis conservative. For Maximum Design Earthquake take ground motion which has a 10 percent chance of being exceeded in a 100-year period. (1000 years of return period).

For Design Base Earthquake, ground motion, which has 144 years of return period for the 100 year of project service life, is taken. (Army Corps of Engineers, 2003). The Ambrasey‟s equation for determination of period of a structure is adopted to determine the dam‟s period.

$T=2.61H/V_s$…Equation2:Ambraseysequationofdam‟speriod

Where, T= period in second, H= the dam height and = Vs the shear wave velocity, which is a function of maximum shear modulus and density

3. Lower Awash Dam Alternative Design

3.1 Description of Existing Dam

Thelayoutoftheprojectconsistsofa48m high damwithacrestlengthof335m acrossAwashRiverwithasidechannelspillwayontheleftabutmentandcombined intakeanddiversionstructureontherightabutment.Thereservoirareaiswithinthe valley, which apeculiarattentionwasgiventoexclude awashfallwithintheinundation area.Thereservoiriscontainedwithintheupperrollinghillcatchmentandinthevalley of the Awash River.

Upstream and downstream faces are inclined with 3.0H: 1V and 2.75H: 1V respectively. The geometry of the dam at the highest cross section is shown in Figure. (The Section below is selected for detail design by the consultant (Construction design and supervision works corporation –Water and energy design and supervision sector- premises -WWDSE) which is zoned earth-rock fill dam with central clay core.

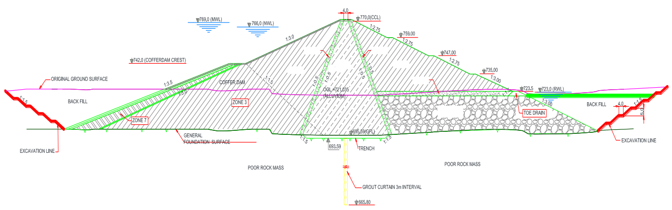


Figure 13: Dam geometry at its maximum Section (Chainage 0+180) (Source: WWDSE, 2016)

Table 4: Zoning of Earth-Rock Fill Dam with Central Clay Core

Zone	Function
1	Core
`2A	Fine filter
2B	Coarse filter
2C	Transition
3	Granular Shell
4	Rock Fill
5	Rip-Rap
6	Toe rock
7	Clay Blanket
	Foundation bed rock

3.1.1 Site Seismic Hazard Assessment

Results of the hazard assessment for the Lower Awash Multipurpose dam site are summarized in Table below.

Table 5: Ground motion amplitude in percentage of gravity (%g) for rock and soil sites at different periods of motion and for different return periods at the Lower Awash Multipurpose Dam site (WWDSE, 2016).

Return Period in years	Ground Motion Amplitude in % of g for Boore-Joyner-Fumal (1993,1997					
	Period =0.2sc		Period =1.0sc		Period =2.0sc	
	Rock	Soil	Rock	Soil	Rock	Soil
50	13.01	13.83	4.20	4.82	2.72	3.03
100	17.71	18.84	5.49	6.32	3.45	3.87
200	23.94	25.45	7.17	8.28	4.37	4.98
500	35.15	37.39	10.21	11.84	5.98	6.84
1,000	43.97	45.97	13.34	15.52	7.59	8.75
5,000	66.73	71.11	24.54	28.47	13.35	15.70
10,000	78.70	84.22	31.13	36.07	16.70	19.75

Note: For the soil site, an average thickness of 30 meters with shear wave velocity 310 m/s is adopted for all periods considered (Boore et al., 1997).

3.1.2Ground Acceleration Period

The Geotechnical investigation around the dam axis and dam seat confirms that the dam body is going to rest partly on soil foundation and partly rock foundation mainly on the river valley bottom. Therefore, for further analysis the PGA values estimated for soil are considered to be the worst scenario and thus adapted accordingly (WWDSE, 2016).

As shown in Table 12, the PGA is indicated with respect to three periods. The site-specific seismic hazard did not indicate which duration would be appropriate for this project. However, considering the location of the project and the possible damaging effects, the PGA values reported for 0.2second are considered. Therefore, following the above discussion the peak ground acceleration that will be considered further are shown in the following Table:

Table 6: Magnitude of PGA for different return periods and design earthquake considerations (WWDSE, 2016)

Designation	Foundation	Period (Sec.)	Return period	PGA as fraction of gravity (g)
SEE	Soil	0.2	5,000	0.7111
OBE	Soil	0.2	1000	0.4597

Direction of the peak ground acceleration was assumed horizontal since nothing has been mentioned in the site-specific hazard assessment report concerning the direction of peak ground acceleration. Moreover, because the report did not specify the magnitude of the peak ground acceleration in vertical direction, the magnitude of the peak vertical ground acceleration was estimated to be 50% of the peak horizontal ground acceleration.

Table 7: Adopted PGA for the design of Logia Dam during earthquake conditions (WWDSE, 2016)

Designation	Foundation	Period (Sec.)	Return period	Adopted Horizontal PGA as fraction of gravity (g)	Adopted Vertical PGA as fraction of gravity (g)
SEE	Soil	0.2	5,000	0.36	0.18
OBE	Soil	0.2	1000	0.23	0.12

3.1.3 Liquefaction and Deformation Analysis of Loose Foundation Soils

The geotechnical investigation at the Logia dam foundation indicates the existence of loose cohesion less soil deposits with or without some soft cohesive layers, which is the most susceptible soil for risks due to earthquake loadings. Pore-water pressure build-up and partial or complete liquefaction of the saturated sand layers are possible during earthquake loading. Instability in the dam foundation may also result from the developments of hydraulic gradients causing erosion and loss of strength under the downstream portion of the dam (hydraulic instability). This fact entails the need for appropriate measure in order to minimize the risks associated with an earthquake loading.

3.2 Alternative Dam Selection

If suitable earth fill materials are available with in economic haul distance of the selected dam site, it is ideal to be selected for the construction of an embankment dam. If suitable earth fill is not available, then the construction of dam will be made with rock materials with appropriate impervious membrane or concrete. Of course, the latter is not economical as compared to embankment dams due to their huge volume concrete requirement and other foundation requirements.For embankment dam project site, there are several different design options;e.g., a dam with earth core; upstream facing of reinforced concrete or asphalt orsynthetic geomembrane; or asphalt core.

Various criteria are involved in the process of alternative dam type selection for embankment dams. Of which topographic condition and spillway and outlet work arrangement, valley shape, dam height, geologic stability of abutment, foundation condition and availability of construction material are very important. From site to site and case to case, one or the other factor may stand out as the main criteria influencing the final dam type selection. In addition to the cost, these mentioned factors affect the choice of dam type. The advantage and possible challenges of the different options and alternatives were explained. Therefore, to select the most feasible dam type, multi-criteria analysis has adopted as indicated below

The major points considered in the multi-criteria analysis are

- Suitability of the site for the proposed dam type;
  - ✓ Morphological condition of the site;
  - ✓ Foundation Requirement of the proposed dam type;
  - ✓ Availability of suitable construction materials for the proposed dam type;
  - ✓ Risk of damage in relation to diversion facilities;
  - ✓ Adaptability of the dam type for the seismic site conditions;
  - ✓ The economic factor (i.e. cost of the proposed dam type);
  - ✓ Method of Construction;
  - ✓ Construction Technology (i.e. the capacity of local construction industry, the availability of equipment/machinery, the requirement of foreigncontractors/consultant, etc.);
  - ✓ The need for specialized quality control;

- ✓ Social and environmental impact;
- ✓ The requirements of Foreign Currency are the major criteria considered.

Taking in to consideration the above points, it is better to select the most feasible dam type for dynamic analysis of Lower awash dam and irrigation project.

Five alternative dam types has been presented below:

a. Earth fill dam with central clay core as dam type 1
b. Zoned earth-rock fill dam with central clay core as dam type 2
c. Asphalt face rock fill dam as dam type 3
d. Rock fill dam with asphaltic coreas dam type 4
e. Concrete face rock fill damas dam type 5

1. Construction material availability

The most economical type of dam will often be the one for which materials are to be found in sufficient quantity within a reasonable haul distance from the site.

The laboratory tests on clay material, exhibit very large percentage of finer particles than (0.075mm). The soil is highly compressible with high values of liquid limit. The test results indicate that the average value of linear shrinkage is 21%.In addition to the above problems there is also shortage in quantity for clay core (WWDSE, 2016). Earth fill dam with central clay core and Zoned earth-rock fill dam with central clay core has been neglected. Asphalt shall be imported and transported to the site. Inputs for concrete face slab i.e. cement, aggregate, sand etc. shall be processed on site or within a country. Concrete face rock fill dam shall be better than asphaltic face rock fill dam in accordance with hauling distance.

2. Diversion requirement

The damage risk in relation to diversion facilities in terms of dam types.

Both asphaltic face rock fill dam and concrete face rock fill dam shall perform better than earth fill dam with central clay core and zoned earth-rock fill dam with central clay core.

3. Seismicity

Seismicity may dictate the type of dam suitable for the site. The dam site located at very high seismic zone of Ethiopian main rift valley.

Seismic resistance of asphaltic face rock fill dam and concrete face rock fill dam shall be weaker than earth fill dam and zoned earth -rock fill with central clay core. However, comparison has to be made on asphaltic face rock fill dam and concrete face rock fill dam. Concrete face rock fill dam can be easily repaired with local materials if crack will happened and can reduce the damage following the seepage quantity, but maintenance of asphaltic face will take time since which is imported from other country and the level of damage will be very high till maintenance.

4. Cost

Cost is the main governing factor for dam type selection.

Project cost of concrete face rock fill dam has been compared with alternative dam types made by the consultant i.e. earth-rock fill dam with central clay core, earth fill dam with central clay core ,asphalt concrete face rock fill dam.

Table 8: Cost Estimation of alternative dams

Dam type	Cost estimated (Birr)
Earth fill dam with central clay core (WWDSE, 2016)	1,781,469,054.00
Earth-rock fill dam with central clay core (WWDSE, 2016)	1,557,367,422.00
`Asphaltic concrete face (WWDSE, 2016)	1,512,533,488.00
Concrete face rock fill dam	1,510,216,668.02

Note: Project cost is estimated for the dam only.

5. Method of construction

Technology (local contractors capacity, availability of equipment / machinery, the requirements of foreign contractors and quality control (the need for specialized quality control) shall be considered. Although both concrete faced rock fill dam and Asphaltic face rock fill dam needs foreign contractors and specialized quality control, but, asphaltic face rock fill dams requires specialized quality control than concrete face rock fill dams.

Table 9: Multi-Criteria Analysis for the Selection of the Most Feasible Dam Type

No	Criteria	Weight (%)		Dam type 1	Dam type 2	Dam type 3	Dam type 4	Dam Type 5
		Total	Sub-weight					
1	**Suitability to the site**	**35**						
1.1	**Morphology** (Narrow stream flowing between high, rocky walls would naturally suggest a rock fill dam. Conversely, the low, rolling plains would suggest an earth fill dam).	`	5	4.5	4	3.5	3.5	3.5
1.2	**Foundation Requirement:** ➢ Competent rock foundation is suitable for all dam types, and overall cost will be the ruling factor for such foundations. ➢ Gravel foundation, if well compacted, are suitable for earth fill or rock fill dams.		11	10.5	9.5	8.5	8.5	8.3
	Silt or fine sand foundations can be used for the support of							

No	Criteria	Weight (%) Total	Weight (%) Sub-weight	Dam type 1	Dam type 2	Dam type 3	Dam type 4	Dam Type 5
	earth fill dams if properly designed, but they are generally not suitable for rock fill dams. Clay foundations can be used for the support of earth fill dams but requires flat embankment slope because of relatively lower foundation shear strength. It is usually not economical to construct a rock fill dam on such foundations.							
1.3	**Construction Material availability** (the most economical type of dam will often be the one for which materials are to be found in sufficient quantity within a reasonable haul distance from the site. • Clay, shell , sand and quarry material shall be obtained locally • Asphalt shall be imported and transported to the site • Concrete material shall be processed on site.		8	6	7	5	5	7.5

No	Criteria	Weight (%)		Dam type 1	Dam type 2	Dam type 3	Dam type 4	Dam Type 5
		Total	Sub-weight					
1.4	**Diversion Requirement** (the damage risk in relation to diversion facilities in terms of dam types).		3	1.5	2	2.5	2.5	2.55
1.5	**Seismicity** (Site seismicity may dictate the type of dam suitable for the site. The dam site located at very high seismic zone of Ethiopia Main Rift Valley.		8	7	6.5	5.55	5.5	6
2	**Project Cost (Billion Birr)**	**25**		**1.781**	**1.557**	**1.512**	**1.532**	**1.510**
			25	21.2	24.24	24.97	24.64	25.00
3	**Method of Construction**	**20**						
3.1	Technology (Local contractors capacity, availability of equipment/ machinery, the requirement of foreign contractors/consultants has to be considered here)		12	10.5	10	7	6.5	8
3.2	Quality control (the need for specialized quality control should be considered).		8	7.5	6.8	5.5	5.25	6
4	**Social and Environmental Impact**	**10**						
4.1	Job opportunity for skilled and unskilled labours		5	4.8	4.6	4.25	4.25	4.25
4.2	Environmental Impact		5	3.4	3.8	4	4	43.5
5	**Requirements of Foreign Currency**	**10**	10	8.5	8.25	6.5	6.4	6.3
	Total Points	**100 %**		**85.4**	**86.69**	**77.27**	**76**	**81.4**
	Rank			**2**	**1**	**4**	**5**	**3**

Based on multi-Criteria analysis Zoned Earth-Rock Fill Dam with Central Clay Core and Earth Fill Dam with Central Clay Core could be better than other dam types. However, the proposed borrow site for clay core has high to very high degree of expansion and high compressibility, which will liquefy during earthquake shaking.

In addition to the above points, Concrete Faced Rock Fill Dam alternative design will have the following advantages:

- ✓ Speed of construction
- ✓ Steep slopes
- ✓ Parts above water are easy to repair
- ✓ Can be constructed after completion of the rock fill section
- ✓ Can be used as slope protection ,etc.

4. Materials and Methods

4.1 Materials Used

Geostudio, 2007 was used to carry out the modeling, including seepage analysis, stability analysis and deformation analysis under static and dynamic conditions.

For the cross-section dam at its maximum section can be done using AutoCAD 2007.

4.2 Description of the study area

The Lower Awash Multipurpose Dam and Irrigation Development Project are located in Zone 1 of Afar National Regional State, at about 700 km northeast of Addis Ababa. The proposed multi-purpose dam is located on Logia River approximately about 100km upstream from the bridge on Logia River of main road to Djibouti. The command area of the project lies on the left side of the Logia River.

Basin wise, the project is located in Awash River Basin with geographic coordinates of the proposed dam site is 1308800 UTM Northing and 652575 UTM Easting. The location map of project area is given in Figure 14 below. The target gross command area of the proposed Lower Awash Flood Control and Irrigation Development Project is approximately about 4500 ha of land.

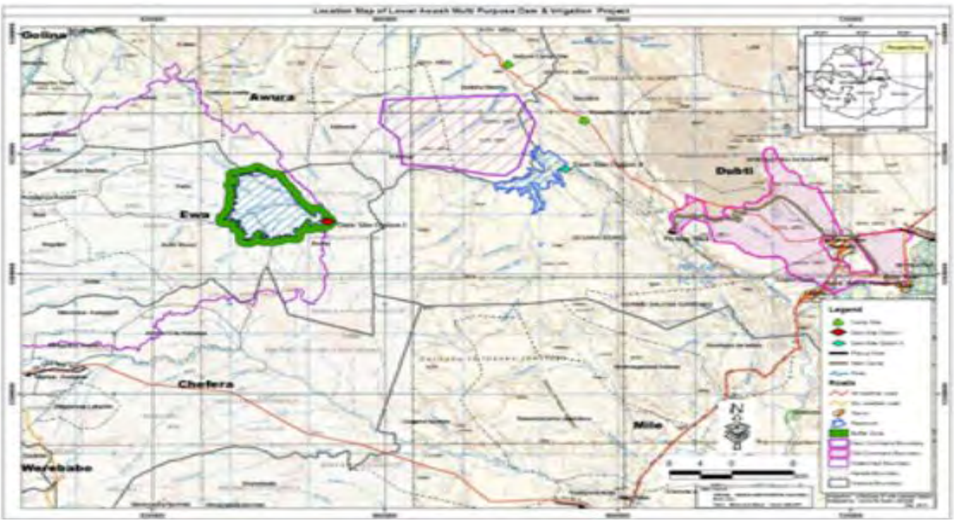


Figure 14: Location map of Lower Awash Dam (WWDSE, 2016)

Logia River is often known to create a flooding hazard to the downstream infrastructure and community. There are three towns, large scale state irrigation scheme, factory and a number of economic and social infrastructures downstream of the river including the Ethio-Djbouti main access road and bridge just before its confluence with Awash River at downstream. The protection of these economical important infrastructures and communities from frequent flooding is crucial.

The position for the dam center line was selected based on topography, effect of thickness of alluvial deposit on the valley floor and location of geological structures particularly along the ridge on both flank geotechnical investigations were conducted at the following locations: along the dam center line; to provide founding levels of dam; along the proposed alignment for a spillway on the upper left flank. Subsurface explorations (drill holes) were located along the centerline of the dam and at the spillway location.

The depth of the subsurface exploration was designed to locate and determine the extent and properties of all soil and rock strata that could affect the performance of the dam and appurtenant structures and to verify the suitability of encountered rock for use as a foundation and/or construction material, specially the spillway site investigation. The dam site is composed of a narrow river valley bounded by volcanic ridges at the right and the left sides. These ridges slope 42 to 45 degree in to the valley.

TheLowerAwashMulti-purposeDamProjectislocatedintheMainEthiopianRift(MER) of Afar Depression (Figure 4). The rift is one of the well-developed continental rift segments in East Africa that marks the boundary between Nubia, Arabia and Somalia plates. Riftingisevidentfromtopographicexpression,geology,volcanismandseismicity.The narrowriftvalleytopographyofMERisprimarilycausedbysubsidenceoffaultbounded sedimentarybasinsand upliftsofthe adjacentriftflanks.(WWDSE,2016)

Damsmorethan45mhighandwhichareinseismiczones,whereanactivefaultisin 10kmradiusarelocatedinhazardousregion(ICOLD,bulletin072).Seismicanalysisofdams startedbeforeafewdecades.Thepreviousmostdominantanalysistypeofearthquakeload wasstaticorpseudo-staticseismicanalysis.Nowdayswiththe advancementofnumericbasedcomputerapplications,dynamicanalysisofadamis becomingcompulsoryforhighandseismicregiondams.Deformationsduetoearthquakesa ndothercharactersofadamduringearthquakes, which cannot be captured by pseudo-static analysis,canbedoneusingdynamicanalysiswiththehelpofcomputer tools.

Duetoseismicload,afewmillimetersofdeformationuptoatotalcollapseofaproject couldhappen.Thisisdeterminedbythecharacteroftheearthquake,soilbehaviorofthesite, dammaterialanddamgeometry.Usingdevelopedmodelstorelateinteraction betweenthesefactors,adynamicanalysis ofLowerAwashconcrete face rock fill dams resting on thick alluvium deposit is conducted on this research to evaluate the dam"s performanceofserviceabilityand safetyduringearthquake.

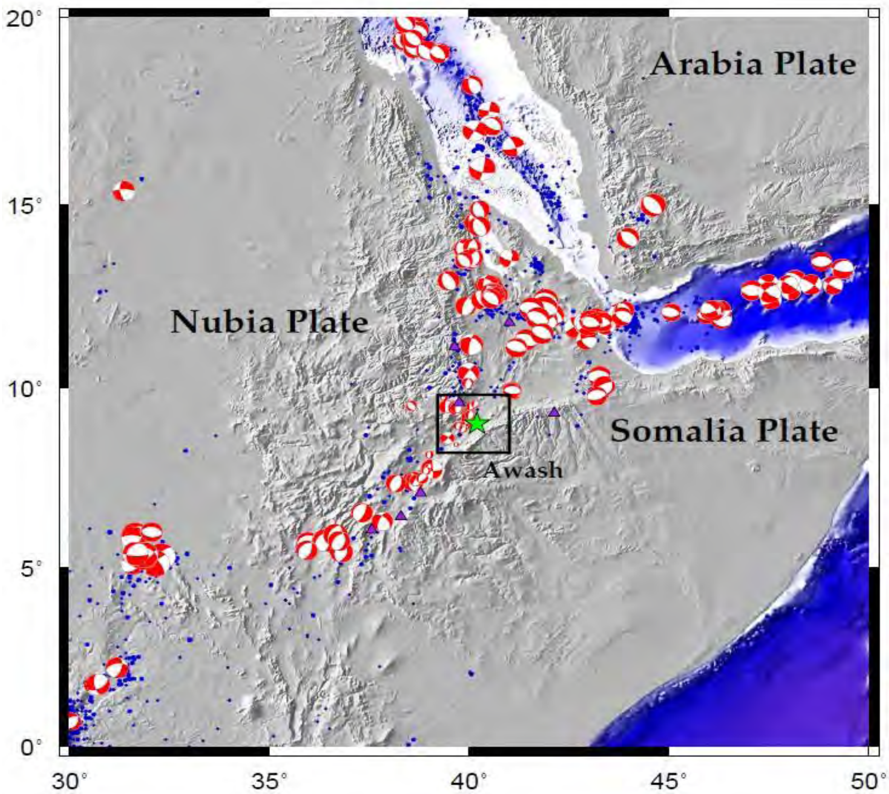


Figure 15: Seismicity data for the Horn of Africa (WWDSE, 2011)

Intheabovefigure,bluecirclesrepresentearthquakesthatoccurredforthelast110yearsinth eregionandsizeofthecirclesisproportionalwithmagnitude.Thegreenstarshowsthe locationoftheLowerAwashDamProjectsite

4.3 Data type and data sources

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 and from literatures, because, there is no Code of standards related to water structures, in Ethiopia.
This research mainly relies on secondary data sources. The secondary data sources needed for dynamic analysis of embankment dam are as follows:

- Geometrical section of the dam (Zoning of rock fill dams and concrete face sab)
- material property of the dam and
- Earthquake data inputs are the major categories of the necessary data inputs. These data are collected from project office (ECDSWC) & reviewed literature.

In general, the following data types has been collected from project office and reviewed.

- Project location site Geology
- Site Specific seismic hazard assessment report at the dam site or in the near proximity of the dam site
- Dam design and geotechnical report
- Flood hazard Assessment of the project site

4.4 Method of analysis

The analysis is performed by the finite element method using the Geo-Studio 2007" software packages.

- ✓ SEEP/W for seepage analysis,
- ✓ SIGMA/W for stress-deformation analysis,
- ✓ QUAKE/W for dynamic analysis and
- ✓ SLOPE/W for slope stability.

4.4.1 Analysis of Dam project

4.4.1.1 Configuration and setup

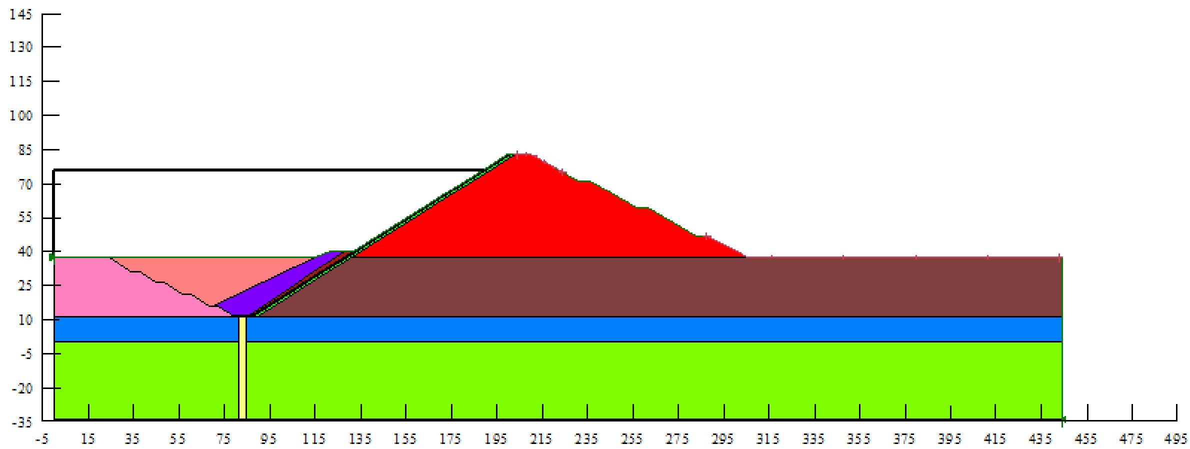


Figure 16: Configuration of dam at its maximum section (the thick alluvium deposit under the dam foundation and large cross-section -Chainage 0+180)

4.4.1.2. Model Range

According to the characteristics of seepage, stress-deformation, static and dynamic slope stability analysis and the actual situation of the project, the two-dimensional finite element model is established. The model analysis is selected at its maximum section (Chainage 1+180) and the direction of X-axis is along the stream (river), Y is along the dam axis. The upstream boundary is 173m away from dam axis, the downstream is 185m away from dam axis, the bottom boundary is the bottom level of curtain wall and the top boundary is the dam crest. The finite element model contains the main boundary and geologic stratification, which may influence the seepage, the stress-deformation, and static and dynamic slope stability field. The shape and material partition of the dam are simulated in the model. Then, the finite element is obtained under the technology of automatic meshing. Based on the model of super element, the finite element model is established with 4319 nodes and 4196 elements.

The finite element model is shown in Fig.17 below

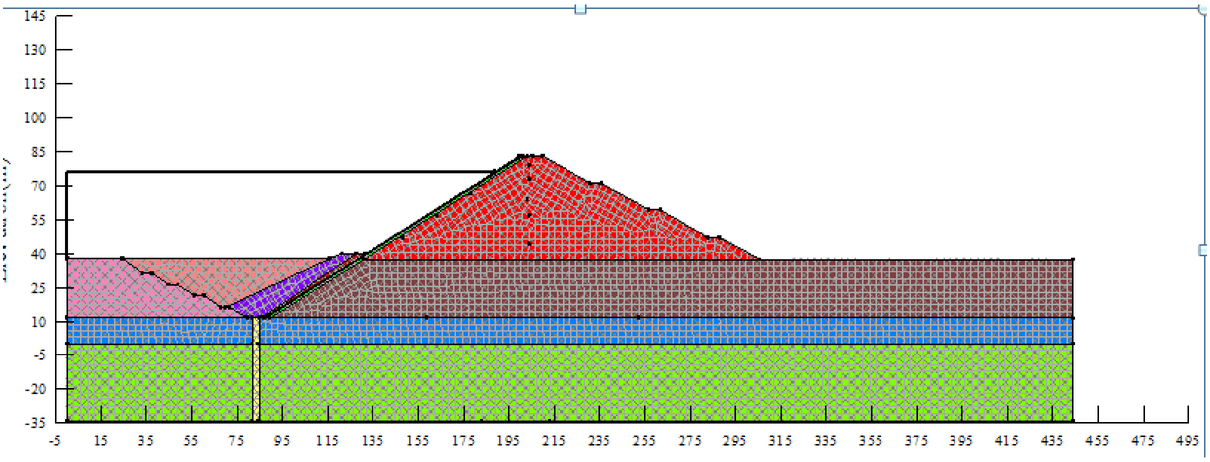


Figure 17: Analysis model at its maximum section (the thick alluvium deposit under the dam foundation and large cross-section)

4.4.1.3. Salient features of Dam and reservoir

- Dam height above river bed level: 48 meter
- Normal water level: 766 m.a.s.l
- Maximum water level: 769 m.a.s.l
- Dead storage level: 728 m.a.s.l
- Crest width: 10 meter
- Crest length: 335 meter
- Upstream slope: 1.5
- Downstream slope: 1.75
- Alluvium deposit (Loose Sand) thickness below the dam: 25 meter.

4.4.1.4. Zoning of Dam Body of Concrete Faced Rock Fill Dam Section Selected For Analysis

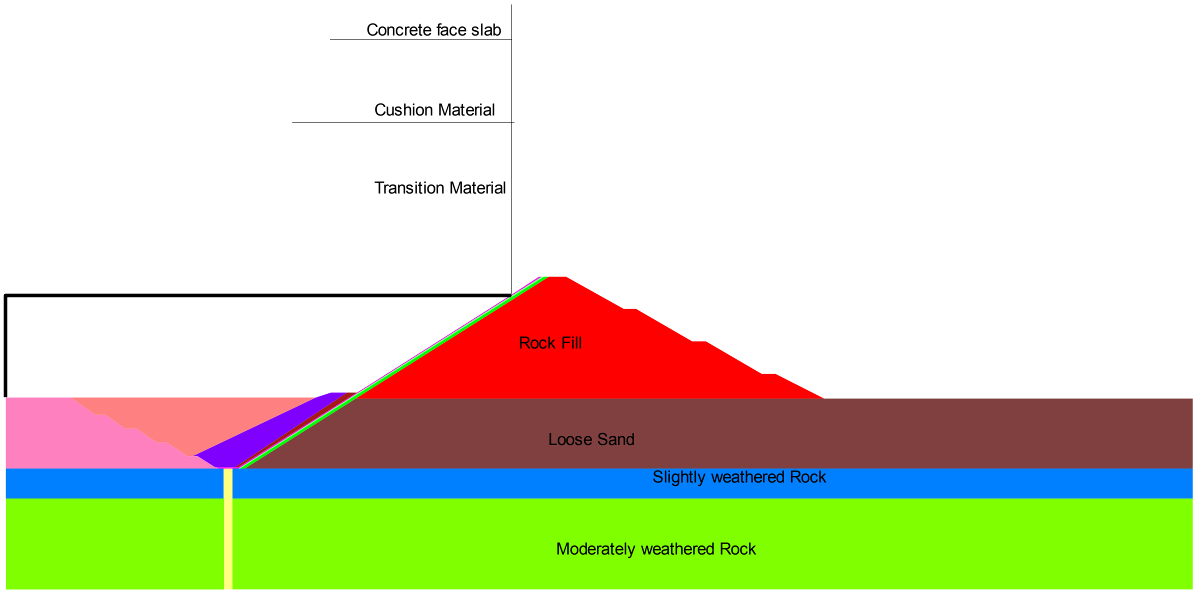


Figure 18: Zoning of Concrete Faced Rock Fill Dam

The principles of zoning of dam body: the permeability of each material zone shall successively increase from the upstream to the downstream; the deformation of each material zone due to water pressure shall be coordinated and minimal, the division of material zone shall be as simple as possible for reducing construction difficulties.

The dam rock fill zones are cushion zone or concrete face supporting zone, transition zone, and main rock fill zone and rubble slope protection are the principal or inherited zones. In addition to this zonification, downstream rock fill zone (secondary rock fill zones) and special cushion zones are provided. For this research work downstream rock fill, (secondary rock fill) zones and special cushion zones are not incorporated for analysis. The slope ration between the cushion zone and transition zone is 1:1.5, and the horizontal width of the two zones is 3m and 4m respectively. The main rock fill zone with a top elevation of 771m is located at the downstream side of the transition zone.

On the upstream of the facing with an elevation, less than 728m (Invert level of bottom outlet), “clay + rock ballast” or random fill cover shall be provided for the purpose of auxiliary anti-seepage function. For clay cover, its top shall be of 4.00m wide, and the upstream surface slope ratio is 1:1.6; the upstream of the clay cover is rock ballast mixture zone, the top width is of 6.00m, and the upstream surface slope ratio of which is 1:2.0.

Design of Dam body Materials shall incorporate and discuss as shown below:

1. Cushion Material or concrete face supporting zone

As the key material of concrete facing rock fill dam, cushion material shall be of semi-permeability, low compressibility, high strength and high seepage stability, and shall be with the function of reversed filtering against the fly ash. Thus, the horizontal width of the cushion material shall be 3m, and processed and slightly weathered rock fill materials with good gradation are to be used. Main design parameters: unit weight is ≥19 kN/m3; porosity is ≤18%; maximum diameter is 80mm; content of particles with a diameter less than 5mm is 40%~50%; content of particles with a diameter less than 0.075mm is no more than 8%; and the gradation is continuous. Rolling shall be conducted layer by layer, and the layer thickness is 400mm.

2.Transition Material

The transition material, with a horizontal width of 4m, is located between the cushion material and main rock filling material, and it is required to have a function of filtering for the fine materials of the cushion zone. Thus, the fresh and processed slightly weathered rocks, which can meet the requirements of gradation, shall be adopted. The transition material shall be of low compressibility, high shearing strength and free drainage after it is compacted. Main design parameters: dry density is ≥19kN/m3; porosity is ≤20%; maximum diameter is 300mm; content of particles with a diameter less than 5mm is less than 15%; and the gradation is continuous. Compacting shall be conducted layer by layer, and the layer thickness is 400mm.

3. Main rock fill Material

The main rock fill material is the main supporting body of the dam, thus, it shall be with sufficient density and necessary deformation modulus. The relatively fresh granite will be blasted and exploited as parent rock to be the main rock fill materials.

Main design parameters: dry density is ≥22 kN/m3; porosity is ≤23%; maximum diameter is 600mm; content of material with a diameter less than 5mm is less than 10%; content of particles with a diameter less than 0.075mm is less than 5%; and the gradation is continuous. Compacting shall be conducted layer by layer, and the layer thickness is 1000mm.

4 Concrete Facing Design

In order to ensure the anti-seepage function of the concrete facing, its thickness shall be determined according to the following equation: t=0.3+0.003H (I COLD Bulletin 070 or 141) , Where t is the facing thickness (m);

H is the vertical height from the section to the facing top (m).

The facing top is with an elevation of 771m and a thickness of 0.45m and shall apply constant thickness from its top to bottom for analysis.

(5) Concrete Toe Slab Design

The dam, with a maximum dam height of 48m, is high dam. The toe slabs on the riverbed shall be put on the upper part of moderately weathered rock mass after removing 25m thick alluvium deposit.

Analyses of the Lower Awash Dam are performed by using Geostudio software packages. Total stresses, displacements, pore water pressures, static and dynamic slope stability are calculated by two dimensional plain strain finite element analyses. Described below is the computation analysis of the concrete face rock fill dam under static and seismic loading.

4.4.2 Dam Seepage Analysis

The proposed dam type seepage analyses through the dam andthe foundation have been conducted using the state-of-the-art Finite Element Methodbased computer program – SEEP/W from Geo-Studio international, 2007.

After a prolonged storage of reservoir water, water percolating through an embankment dam will establish a steady-state condition of seepage. The upper surface of seepage is called the phreatic line. It is general practice to analyze the stability of the downstream slope of the dam embankment for steady-state seepage (or steady seepage) conditions with the reservoir at its normal operating pool elevation (usually the spillway crest elevation) since this is the loading condition the embankment will experience most.

The material model used in the seep/W component is saturated/unsaturated for all the materials.

Table 10: hydraulic conductivity values for Seepage Calculation Zoning Material (Source: WWDSE, 2016).

Dam Material	Hydraulic conductivity(cm/s)
Main Rock fill Material	0.012
Random fill	1.00×10-4
Clay blanket	5.08 × 10-8
Transient Material	5.0×10-4
Cushion Material or face supporting layer	3.15×10-4
Concrete Facing	1.00×10-9
Impermeable Curtain	1.74×10-7
Rock Mass Above Relatively Impermeable Stratum (Slightly weathered)	1.00×10-7
Rock Mass Above Relatively Impermeable Stratum (Moderately weathered)	3.00×10-9

4.4.3 Analysis of Stress and Deformation of Dam- Initial static stress

Theinitialstaticstressisdonebeforethedynamicanalysis.Theconstitutivematerial modelselectedfortheanalysisiselastic-plastic.Intheanalysis,apore-waterpressure contributesfor the strength of the soil by altering the matric suction.

Failure of embankment dams, except for failures caused by unanticipated catastrophic events such as earthquakes or overtopping, are usually preceded by warning signals such as increased rate of deformation, cracking, leakage, etc. It is important to recognize that all embankment dams in service deform and settle. In general, deformations of concrete face rock fill dams may result in aesthetically unacceptable surficial appearance. However, excessive deformations indicate internal distress of the dam, and can result in:

- Reduction or loss of freeboard, and/or
- Internal and/or external loads.

The greatest section (Chainage 0+180) has been selected to conduct stress-deformation analysis of the dam under the effect of dead load and water load during different periods,

Table 11: Embankment fill and foundation material characteristics for initial Insitu stress- deformation and dynamic analysis (Source: WWDSE, 2016).

Rock Fill material sub-zone	Unit Weight (KN/m3)	Poissons ratio, V	Damping ratio ,D	Modulus of Elasticity,E (KPA)	Maximum Shear Modulus , G_{max} (KPA)
Cushion zone	19	0.3	0.12	40000	57692.5
Transition zone	19	0.3	0.3	40000	60000
Main rock fill zone	22	0.15	0.1	45000	47670
Curtain Wall	24	0.167	0.16	20000000	100000
Clay blanket	18	0.3	0.1		17857.5
Concrete face	24	0.167		30000000	2.68×10^{10}
Random fill material	15	0.3	0.12	40000	35715
Alluvial	18	0.4	0.1	20000	416667
Backfill (granular shell)	22	0.25	0.334	40000	38465
Foundation	22	0.22	0.1	45000	320000
Relatively impermeable bed rock	Bed rock	0.16	0.12	2100000	800000

4.4.4 Dynamic Analyses and Earthquake Records Used

Dynamic analysis of the dam is performed using Quake/W (2007) software utilizing equivalent linear analysis technique. The earthquake motion is applied at the base of the dam where the nodes are fixed in both directions. In order to conduct the effect of stored water on the dynamic behavior of the dam dynamic analyses are performed for full reservoir conditions. The computed values of accelerations, displacements and stresses at various points on the dam body are compared.

When an earthquake occurs, the effects of earthquake-induced ground shaking is often sufficient to cause failure of slopes that were marginally to moderately stable before the earthquake. The resulting damage from slope instability can range from insignificant to catastrophic depending on the geometric and material characteristic of the slope.

Therefore, evaluation of seismic slope stability is one of the most important activities.

Earthquake magnitude and source distance are two of the most important factors, which determine ground motion or base rock motion at any particular location. Base rock motion in the form of an acceleration-time relationship or record is the most important seismic data for seismic analysis. Often, however, the use of the whole of the record may be cumbersome. Therefore, one or more ground motion parameters, derived from the record, are used for assessments of stability and deformation. Typical parameters are peak ground acceleration (PGA), duration of motion and predominant period. Seismically triggered failures may be associated with reduction in factor of safety, large deformations and liquefaction phenomena.

Earthquake ground motions are capable of inducing large destabilizing inertial forces, of a cyclic nature, in slopes and embankments. In addition, the shear strength of the soil may be reduced due to the transient loads (i.e., cyclic strains) or due to the generation of excess pore water pressures.

The combined effect of the seismic loads and the changes in shear strength will result in an overall decrease in the stability of the affected slope. In this research paper it is essential to discuss the dynamic response of embankment dam‟s earthquake loading conditions using three earthquake records - Elcentro, Kobe and

Hachinohe (in Ethiopia there is no long time historical record earthquake data for dynamic analysis of embankment dams).

The model requires the following parameters:

- Shear modulus reduction function is a function, which the reduction in shear modulus is expressed in terms of the strain, occurred. These functions for different soils and materials of the dam are obtained from the Shake (2000) software sample functions published by Seed and Idriss (1990) and Gazetas-Soil dynamics and Earthquake Eng. (1992).

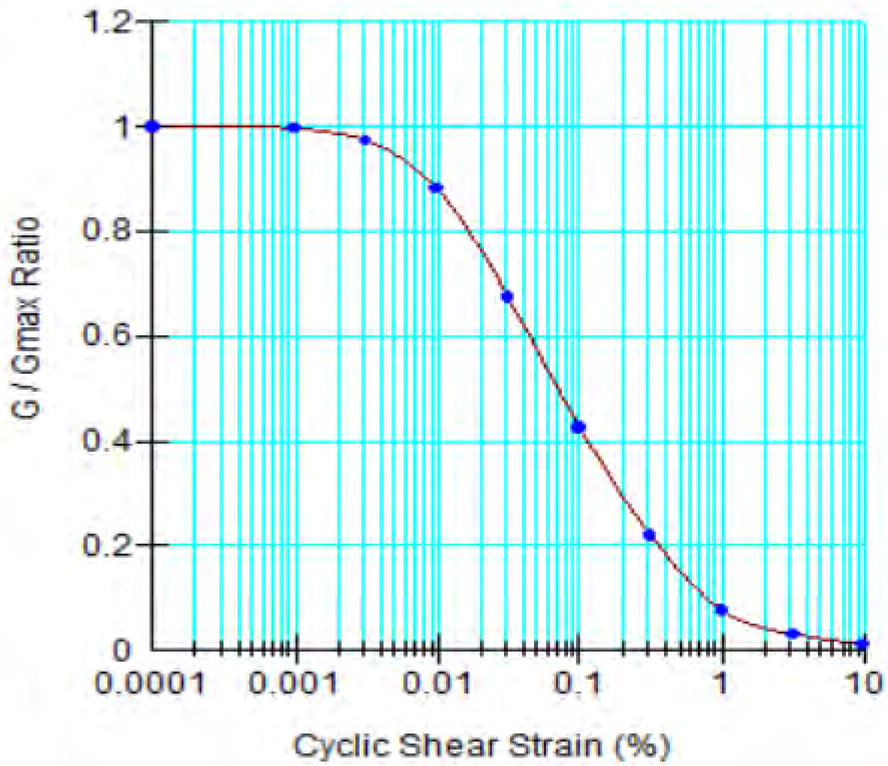


Figure 19: Typical shear modulus reduction for clay plasticity index between 20-30 developed by Seed and Idriss

- The damping ratio function is a function, which explains the increasing response of the soil to dissipate energy as the strain gets higher. These functions for different soils and materials of the dam are obtained from the Shake (2000) software sample functions published by Seed and Idriss (1990), Gazetas-Soil dynamics and Earthquake Eng. (1992)

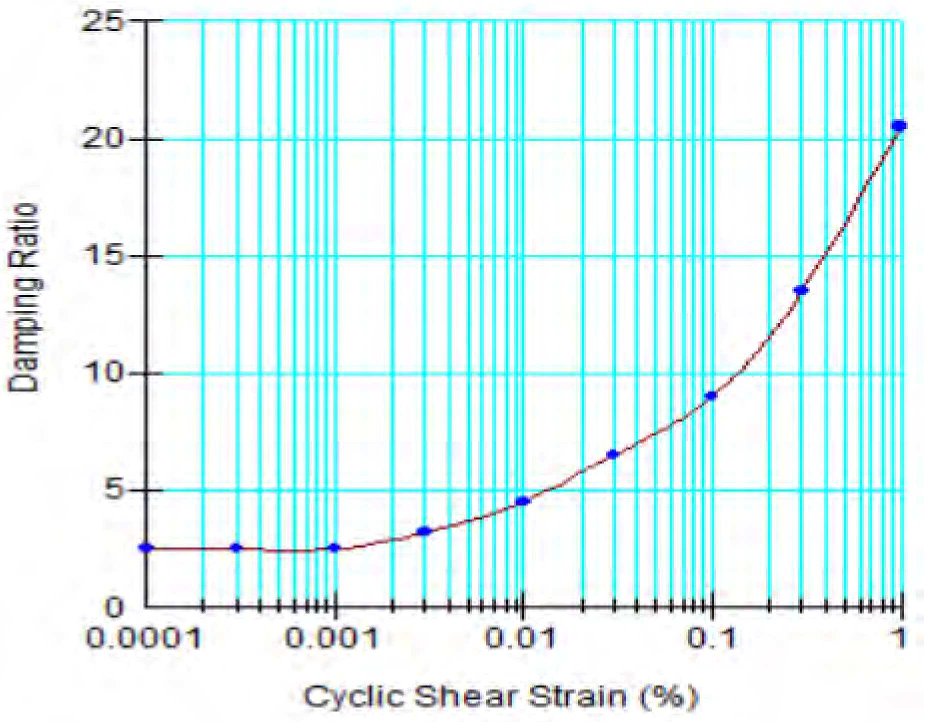


Figure 20: Typical damping ratio function

- The maximum shear modulus is a function of overburden stress. Soil, which is under athick soil layer, exhibits a greater shear modulus as compared to the same soil, butoverburdened by a thin layer of soil.

In the dynamic analysis of Lower awash dam and irrigation project, earthquake motions recorded at base rock level for Elcentro, Hachinoe and Kobe are used based on the earthquake magnitude and epicentral distance. Characteristics of the earthquake motions used are summarized in Table 9 below, and the records are shown in Figure below.

Table 12: Properties of earthquake records used in the analysis

Record	Maximum Acceleration (g)	M_w (magnitude)
1940 Elcentro earthquake record	0.35	6.7
1995 Kobe JMA earthquake Record	0.0005	7.2
1968 Hachinohe earthquake Record	0.22265	7.9

A).The 1968 Hachinohe record, Japan (M=7.9, H=0km, R= 200km)

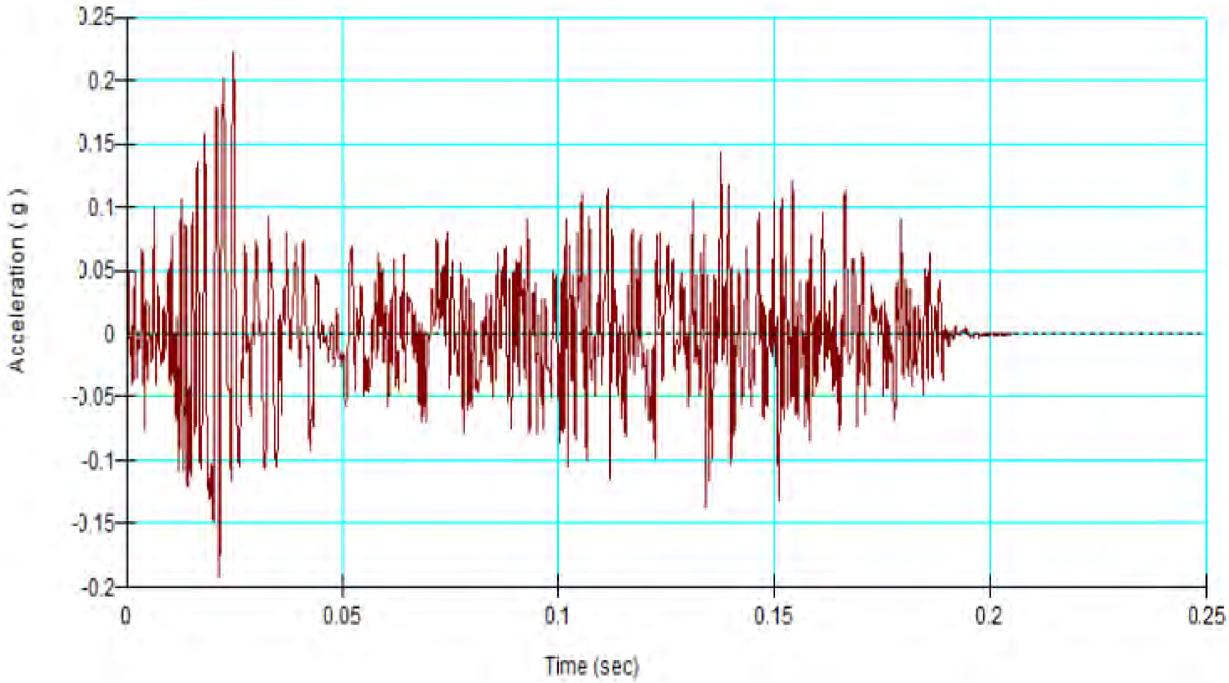


Figure 21: Maximum credible earthquake-Horizontal

B). The 1995 Kobe JMA record, Japan (M=7.2, H=14.3km, R= 19km)

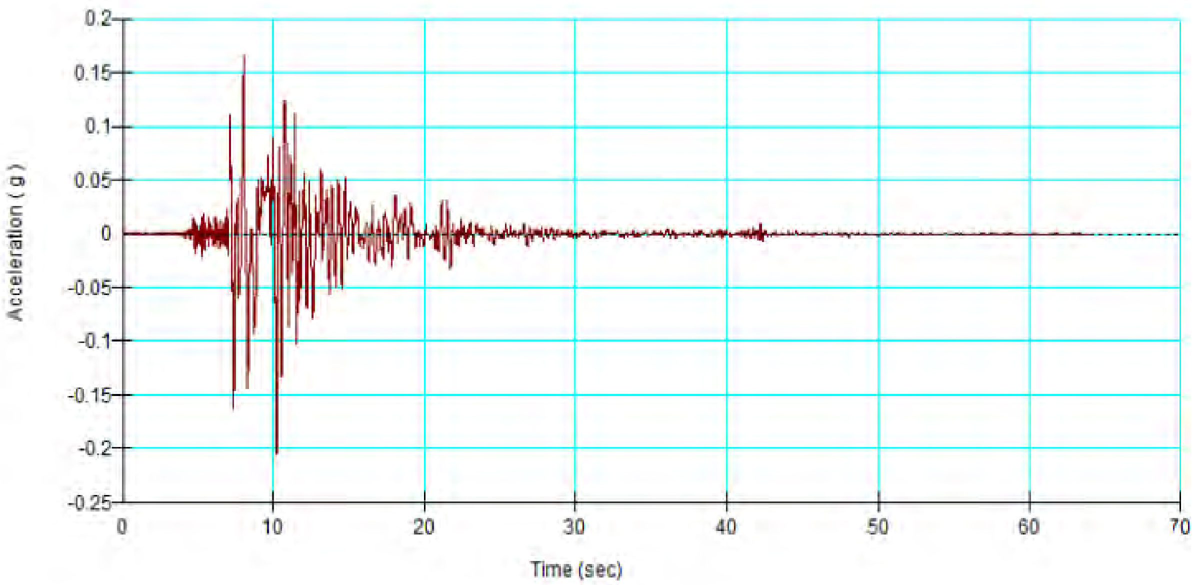


Figure 22: Maximum credible earthquake-Horizontal

The 1940 Elcentro record, Japan (M=6.7, H=11km, R= 11.5 km)

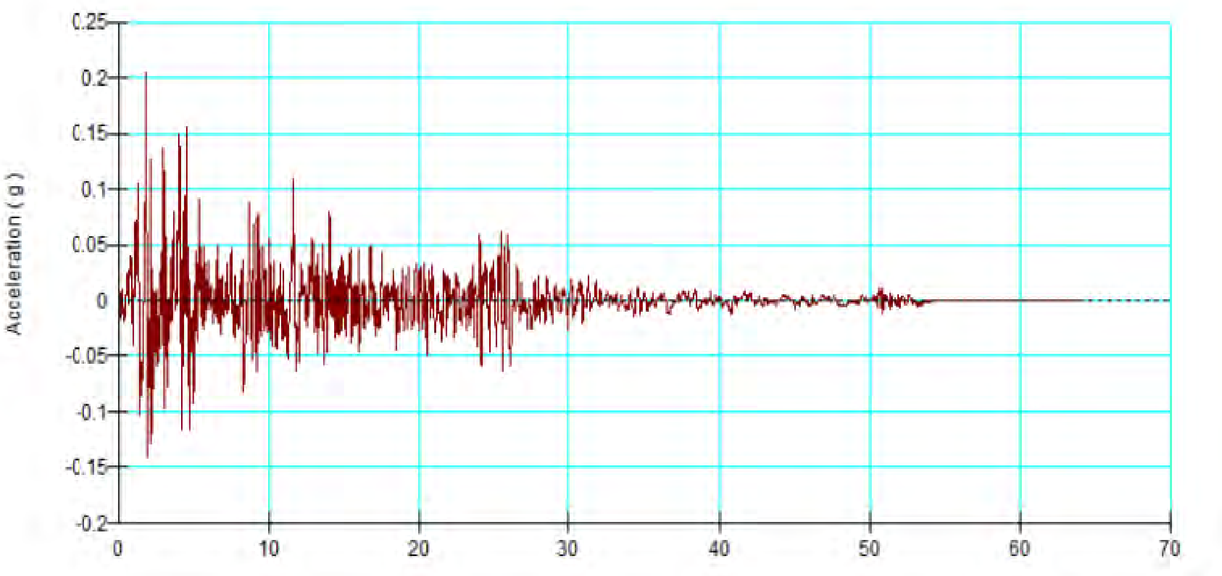


Figure 23: Maximum credible earthquake-Horizontal

In order to evaluate the dynamic behavior of the dam and understand the effect of various Parameters, the values of horizontal acceleration and horizontal displacements computed through dynamic analysis.

4.4.4.1 Site Seismic Hazard Assessment of Lower Awash Dam Project Calculation principle and its result

4.4.4.1.1 Selection of Earthquakes for Analysis

A) Return Periods

Earthquakes must be defined for analytical purposes so that appropriate seismic evaluation parameters canbe selected. According to ICOLD Bulletin 72, seismic hazard of a specific project at interest can be studied either through the deterministic or probabilistic approach.

ICOLD Bulletin 72 has defined the Safety Evaluation Earthquake (SEE), which is a replacement of Design Basis Earthquake (DBE), Operating Basis Earthquake (OBE) and Maximum Credible Earthquake (MCE).

SEE: It is that level of shaking for which damage can be accepted but for which there should no uncontrolled release of water from the reservoir. The SEE is normally characterized by a level of motion equal to that expected at the dam site from the occurrence of a deterministically evaluated maximum credible earthquake or of theprobabilistically evaluated earthquake ground motion with a very long return period for example 10,000 years.

OBE: The operating Basis Earthquake represents the level of ground motion at the dam site for which only minor damage is acceptable. As stated in ICOLD Bulletin 72, it is appropriate to choose a minimum return period of 145 years. Since the consequences of exceeding the OBE are normally economic, it may be justified to use a more severe or less event for OBE.

Dams with more than 45 meter height and with a reservoir capacity of 120 hectare-meter cubes need dynamic analysis. The rate of seismic hazard is related with site conditions in the below table. (ICOLD, 1989).

Table 13: Rate of seismic hazard

Rate of seismic hazard	
Condition	Hazard
PGA<0.1gravity	Low
0.1gravity <= PGA< =0.25gravity	Medium
PGA>0.25gravity(but no active fault in 10km radius)	High
PGA>0.25gravity(with active fault in 10km radius)	Extreme

Seismic hazard due to natural earthquakes is, rather haphazardly, presented as the probability that a PGA will be exceeded in T= 475 years (Bommer and Pinho, 2006; GSHAP, 1999). The seismic hazard evaluation report of Lower Awash Project highlighted that the PGA values of 500 and 1000years return periods can be considered as OBE (Operation Basis Earthquake) and MDE (Maximum Design Earthquake) respectively.

In conclusion the Lower Awash Project is located in a very tectonically sensitive area and any potential failure might because of earth is not allowed since its prime purpose is to protect the settlement and important infrastructure downstream of the dam.

Therefore, this calls for adapting very conservative values and therefore 1:5,000 years return period is considered for SEE and 1:1000 years return period for OBE considering the recommendation of ICOLD Bulletin 72 and site-specific facts.

4.4.5Analysis of Dam Slope Stability

The stability analysis has been conducted in order to determine the factors of safety for various slip surfaces.Carry out analysis on dam slope stability using both limit equilibrium method and finite element method (Morgenstern-Price analysis method) and Geo-slope procedure.

Table 14: Embankment fill and foundation material characteristics for Slope Stability analysis (Source: WWDSE, 2016).

Rock Fill material sub-zone	Unit Weight (KN/m3)	C‟(KPa)	ϕ‟
Cushion zone	19	0	32
Transition zone	19	0	34
Main rock fill zone	22	0	42
Clay blanket	18	28	10
Concrete face	24	35	15
Random fill material	15	2.5	25
Alluvial	18	2.5	32
Backfill (granular shell)	18	0	34
Foundation	22	2.8	43.1
Relatively impermeable bed rock	Bed rock	10-9	

5. Result and Discussion

5.1 Results of Seepage Analysis

The Finite Element Models used in the analyses and the computed discharges areshown on respective Figures below. As can be seen from SEEP/W results, phreatic levelfollows the upstream slope face slab down indicating that the use of concrete face slab as a water barrier material isquite effective. It is one of the tasks of design and construction to make the structurefunctional in the sense that the water is properly drained away and the quantity ofdrained water is tolerable and small. The computed seepage for the dam and its foundation is $1.0424*10^{-7} m^3/s$ and $1.4146*10^{-5} m^3/s$, per meter width of the dam respectively.

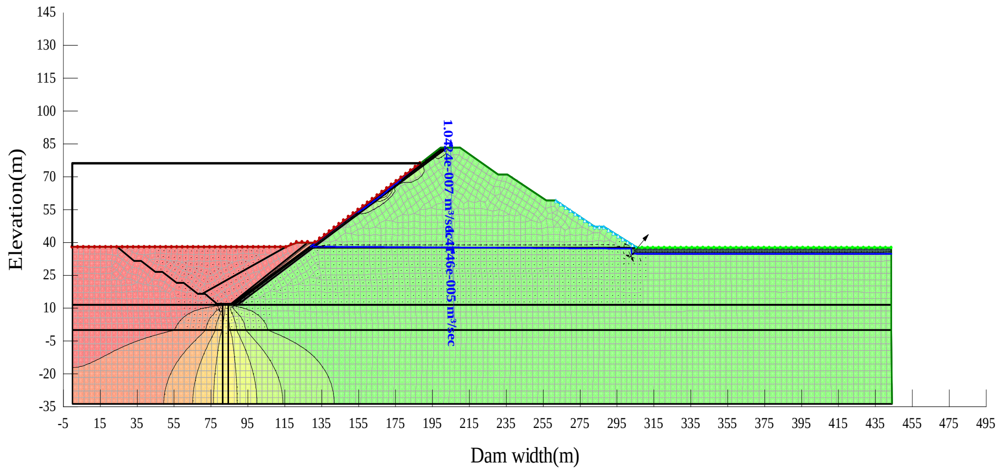


Figure 24: Results of Seepage Analysis

5.2 Results of Initial- Insitu stresses

The Results from this analysisare used to` determine the total stress and effective stress under reservoir loading at normal pool level and self-weight of the dam itself before earthquake loading.

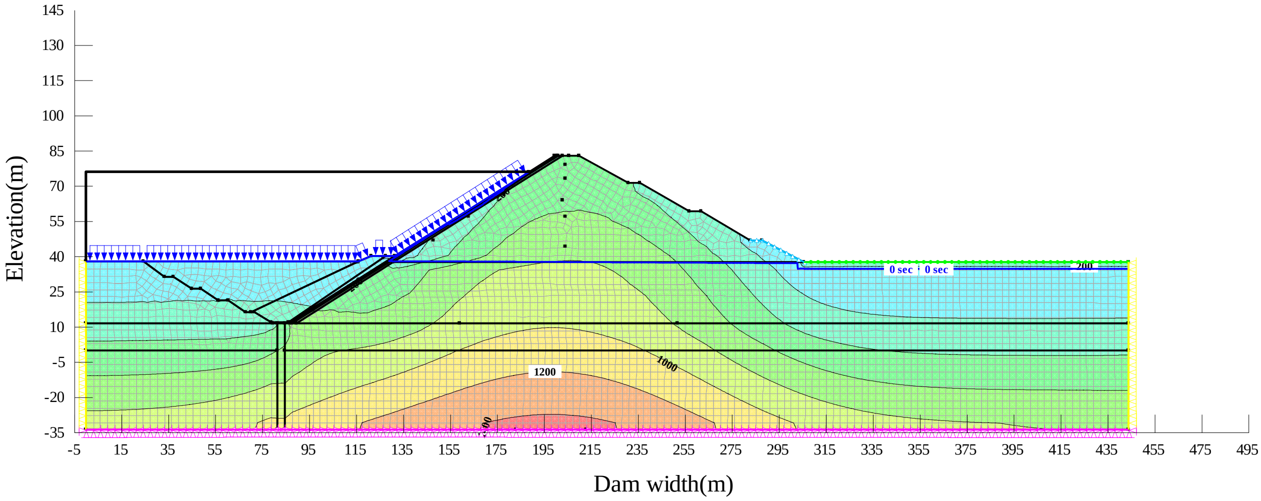


Figure 25: Vertical effective Stress contours

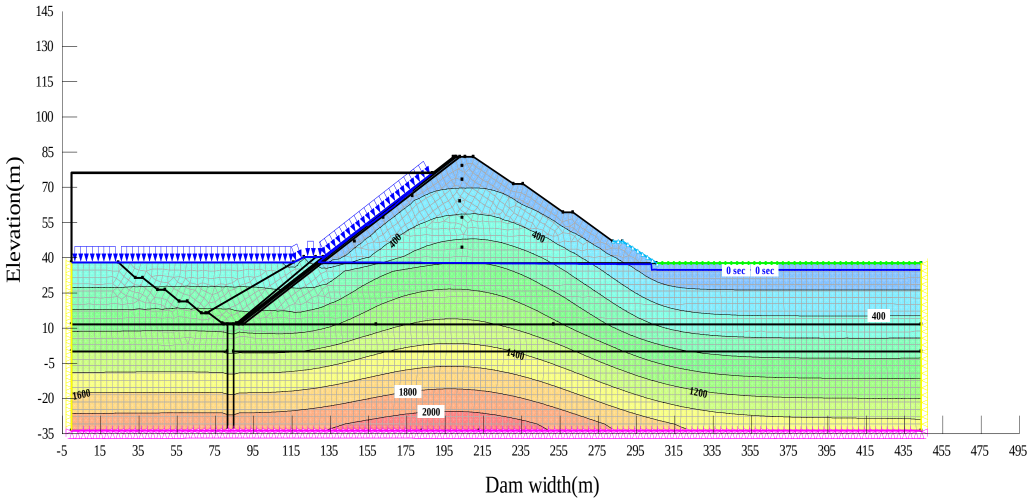


Figure 26: Total Vertical Stress contours

As the graphic result presents, maximum effective stress is recorded at the base of the foundation (1400kpa). Effective stress is computed by minimizing pore water pressure from total stress. The maximum stress computed is 2000kpa. The Insitu site investigation result shows that the maximum bearing capacity of the foundation is 2690.5kpa and thus the foundation can withstand the load from self-weight of the dam and the static reservoir loading.

5.3 Results of Dynamic Analysis

Once the Insitu static stresses have been established, the next step is to do the dynamic or shaking analysis. The deformation, stability and liquefaction assessment has been computed after earthquake load.

5.3.1 Pore-pressure functions

To make compute excess pore-pressures that may arise due to the shaking, it is necessary as a minimum to define two functions. They are called the Pore-Pressure Ratio function and the Cyclic Number function. For this analysis the sample functions has been used as shown in Figure 31 and Figure 32 (these sample functions are included within the software). The Cyclic Number function in Figure 32 is the sample function for loose sand.

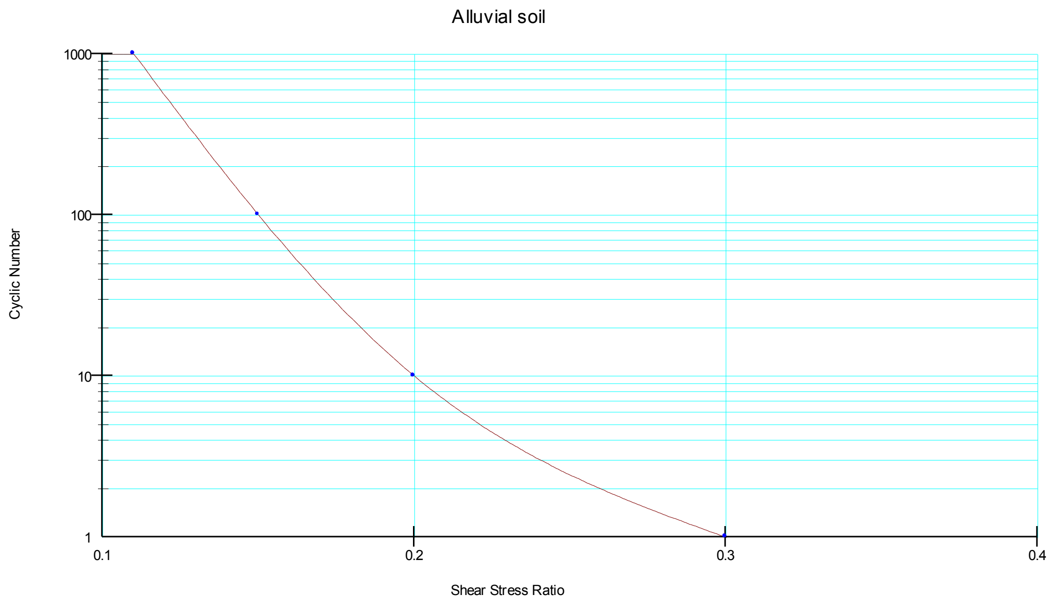


Figure 27: Cyclic number function for loose sand (foundation-25m thick alluvial deposit)

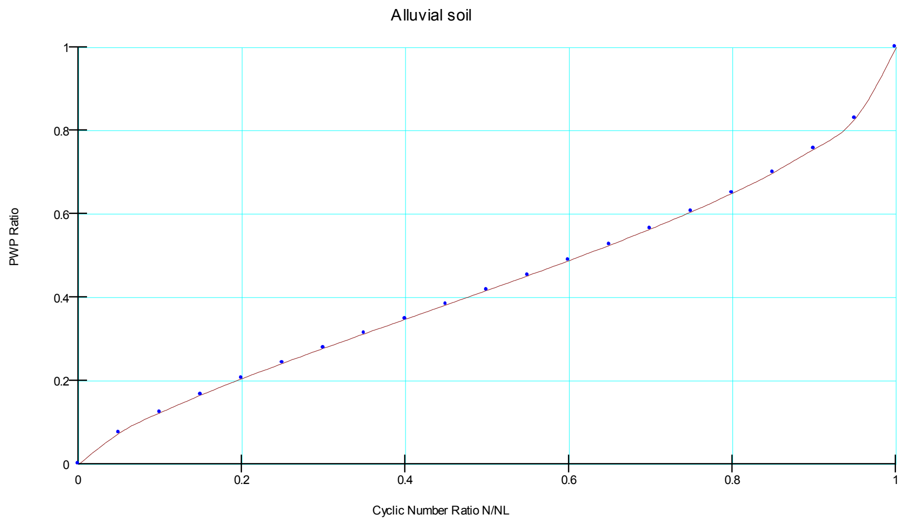


Figure 28: Pore pressure ratio function for loose sand (foundation-25m thick alluvial deposit)

QUAKE/W is formulated on the basis of a time integration scheme.. In this analysis, the earthquake record data points have a time interval of 0.02 seconds, making 1500 data points for the 30-second duration. This means there will be 1500 time steps, or 1500 finite element analyses.

5.3.2 Response of the analysis dam section for different earthquake records at the base and crest of the dam.

A History Point at the crest of the dam and base of the dam has been defined. With the contour graph command, it has been created graphs specific to the History Points. The amplification of motion occurred at the crest and base of the dam has been done for each time-history data (Elcentro, Hachinohe and Kobe).

- **Results of Dynamic Analysis with Elcentro Earthquake Record**

The maximum horizontal acceleration values computed because of dynamic analysis using the 1940 Elcentro Record, USA (M=6.7, H=11km, R=11.5km) as base excitation are shown below.

Table 15: Maximum Horizontal Acceleration values at the crest and base of the dam

Points	Full reservoir condition
	$a_{max(g)}$
Dam crest	0.46
Dam base	0.34

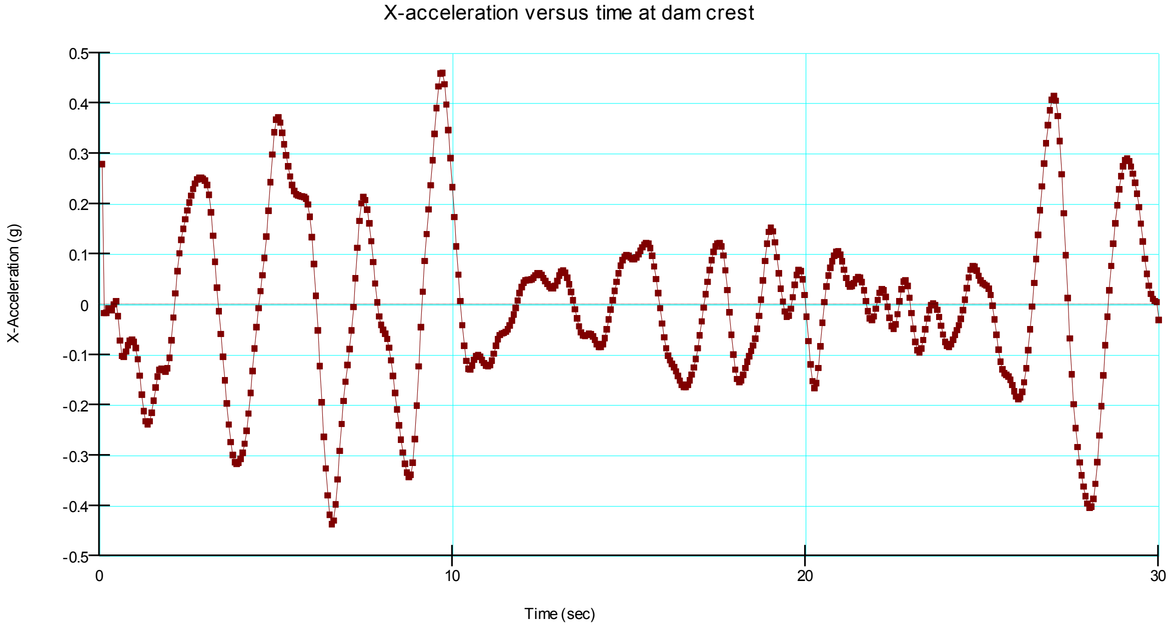
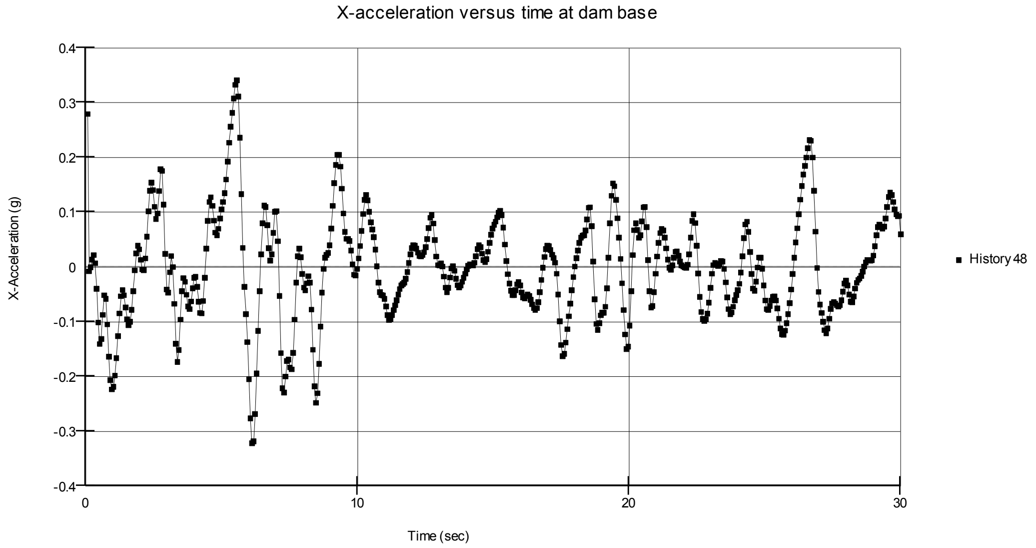


Figure 29: Horizontal acceleration- time history at dam crest for MCE



Figure

Figure 30: Horizontal acceleration- time history at dam base for MCE

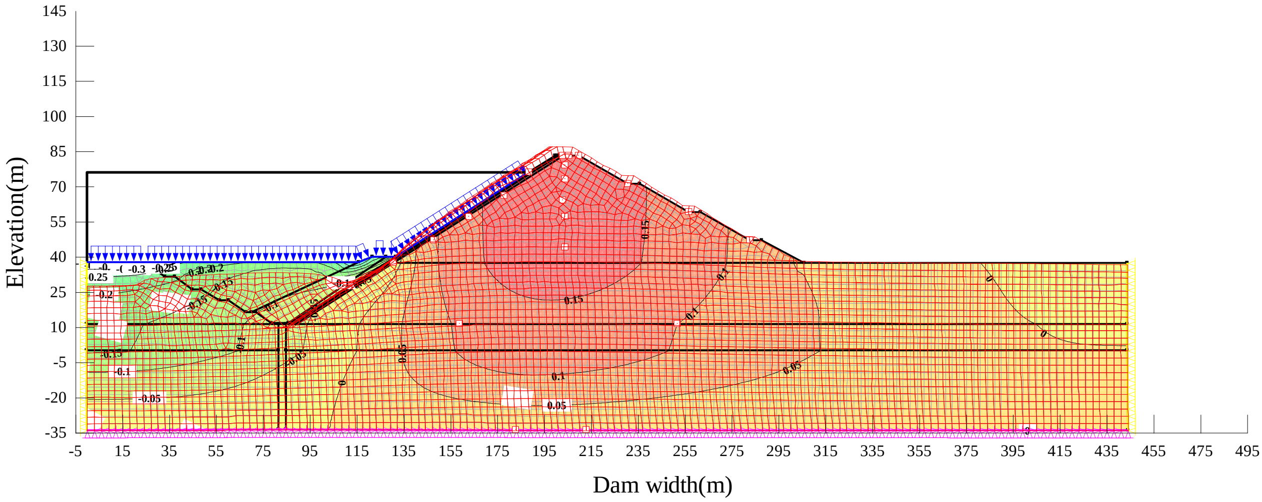


Figure 31: Computed vertical deformation For MCE

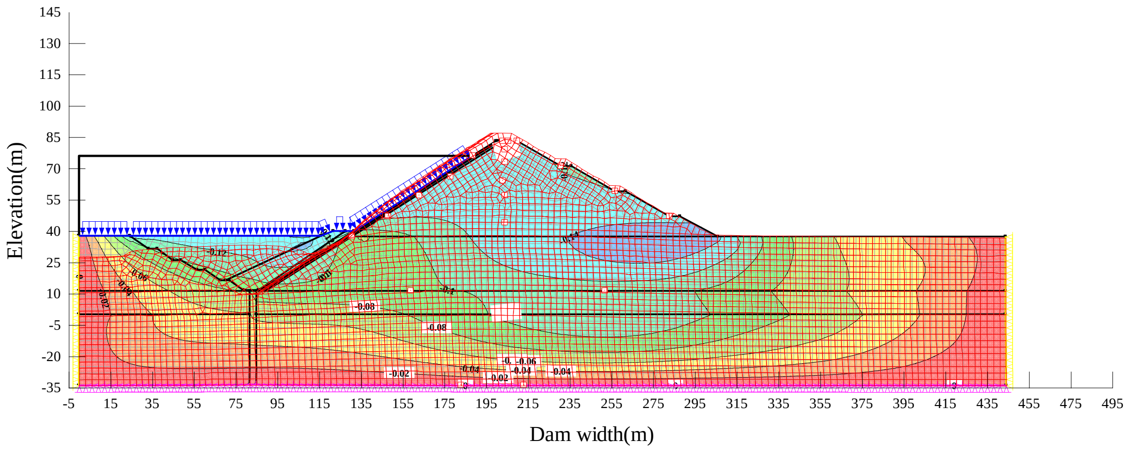


Figure 32: Computed horizontal deformation for MCE

The magnitude and distribution of the vertical and horizontal displacements are shown

in Figures above for the maximum cross -section and full reservoir condition.

As shown in Fig.35 and Fig.36, in the upstream and downstream parts, the computed maximum horizontal displacement is -140mm (directional upstream). The maximum vertical displacement is 150mm, which is approximately located in the middle of dam height and is about 0.50% of dam height. The horizontal displacement of dam is zero at the central line of dam section and increases towards both upstream and downstream faces. The maximum computed horizontal displacement at the upstream face of dam could reach 0.15m. The results of deformation showed that no additional extension of dam or cumber height.

- **Results of Dynamic Analysis Using the 1995 Kobe JMA Record, Japan (M=7.2,**

H=14.3km, R=19km)

The maximum horizontal acceleration values computed as a result of dynamic analysis using the 1995 Kobe JMA Record, Japan (M=7.2, H=14.3km, R=19km) as base excitation are discussed below .

Table 16: Maximum Horizontal Acceleration values at the crest and base of the dam.

Points	Full reservoir condition
	$a_{max(g)}$
Dam crest	0.19
Dam base	0.2

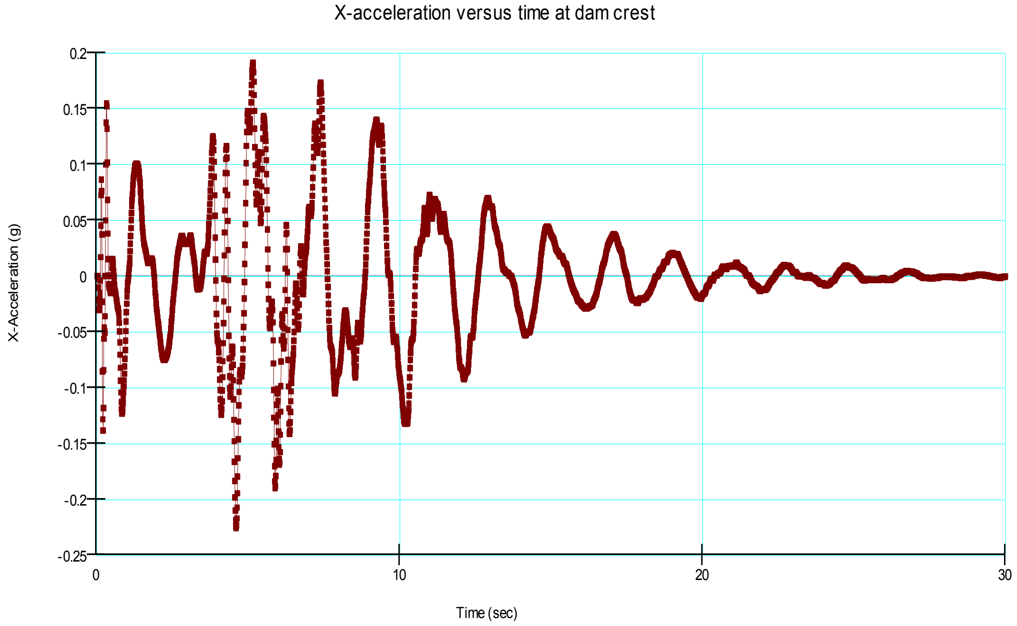


Figure 33: Horizontal acceleration- time history at dam crest for MCE

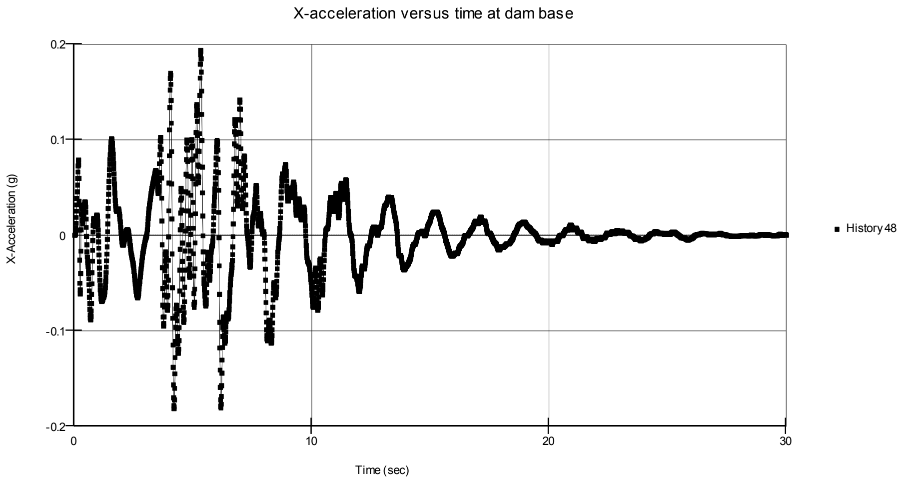


Figure 34: Horizontal acceleration- time history at dam base for MCE

The Kobe earthquake record contour shows no horizontal and vertical displacements at all due to its loading.

- **Results of Dynamic Analysis with Hachinohe Record**

The maximum horizontal acceleration values computed as a result of dynamic analysis using the1968 Hachinohe Record, Japan (M=7.9, H=0km, R=200km) record are discussed below.

Table 17: Maximum Horizontal Acceleration values at the crest and base of the dam

Points	Full reservoir condition
	$a_{max(g)}$
Dam crest	0.27
Dam base	0.28

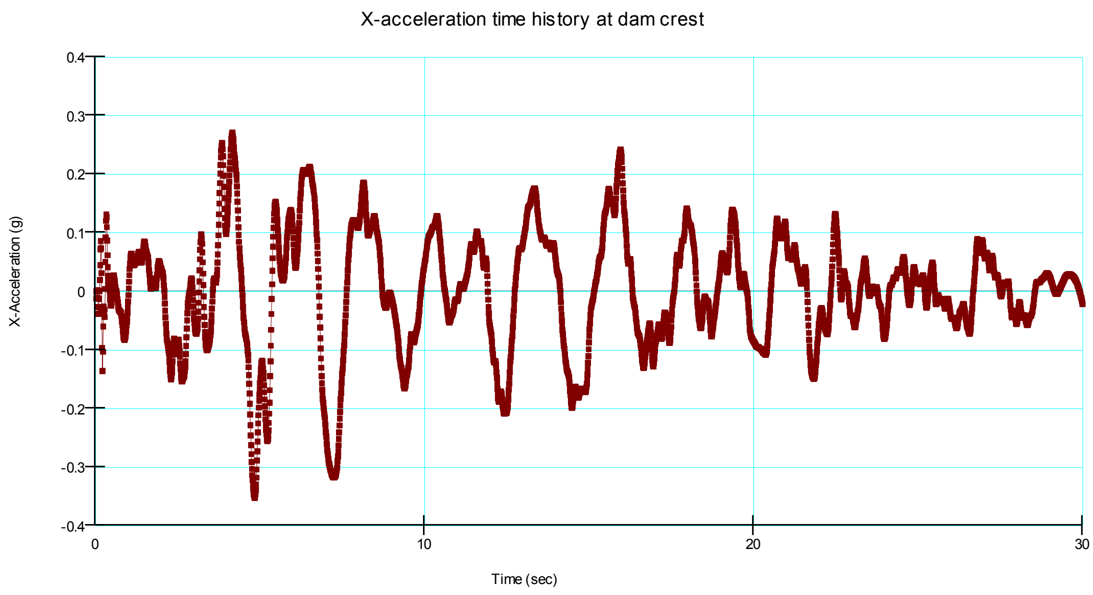


Figure 35: Horizontal acceleration- time history at dam crest for MCE

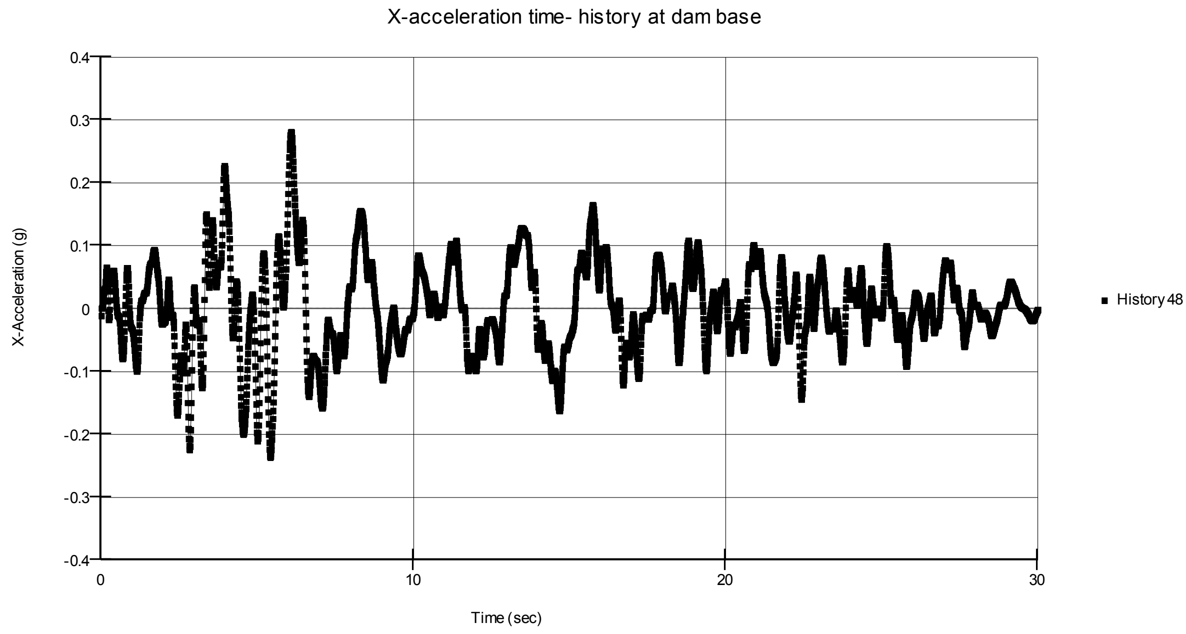


Figure 36Horizontal acceleration time- history at dam base for MCE

5.3.3 Liquefaction potential numerical analysis result for the three earthquake records

Rock fills are neglected for liquefaction potential assessment because of its free draining material as no pore water pressure development. Alluvial deposits of the foundation comprise thick loose sand granular deposits with minor fine proportions.

Its thickness at the riverbed reaches to a maximum of 25m.

In an Equivalent Linear analysis, a key parameter obtained is the Cyclic Stress Ratio (CSR). This number is used together with the Cyclic Number function to indicate the possibility of liquefaction.

Liquefaction during MCE:-

In the three cases of earthquake during Maximum Credible Earthquake, the analysis shows that liquefaction will occur in all the parts of the alluvium foundation. The yellow hatched region in the below figure illustrate the area of liquefaction.

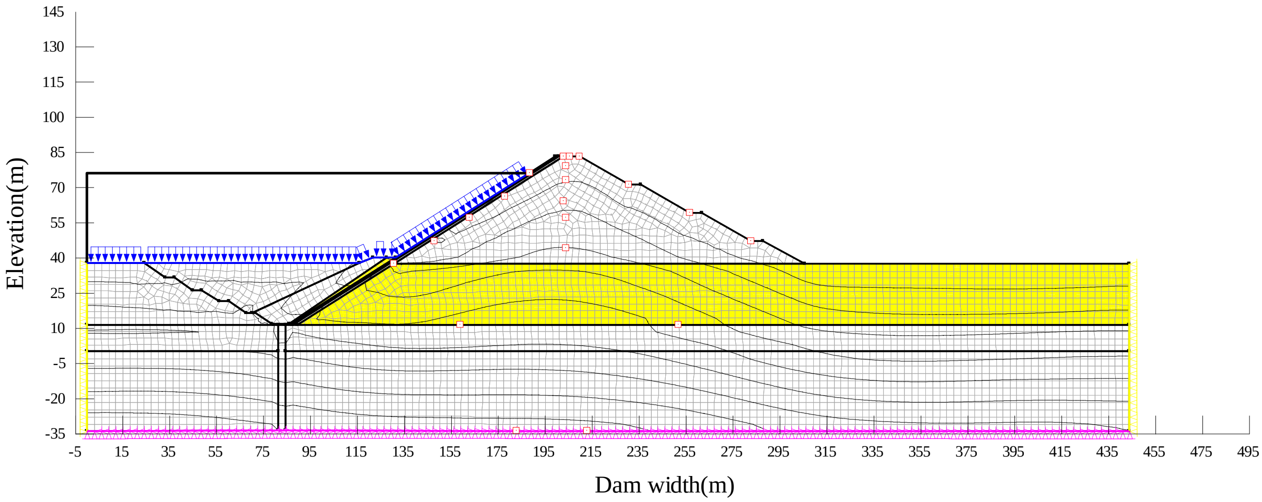


Figure 37: Zones of potential liquefaction of loose sand (Yellow color)-Hachinohe, Elcentro and Kobe Earthquake record- For maximum credible earthquake (MCE).

Liquefaction during DBE:-

In design base earthquake, the same result with the above MCE case is determined. i.e., liquefaction will most likely occur in the all parts of the alluvium foundation except some region at the center of the dam. See the below figure with a yellow hatched area of liquefied zone.

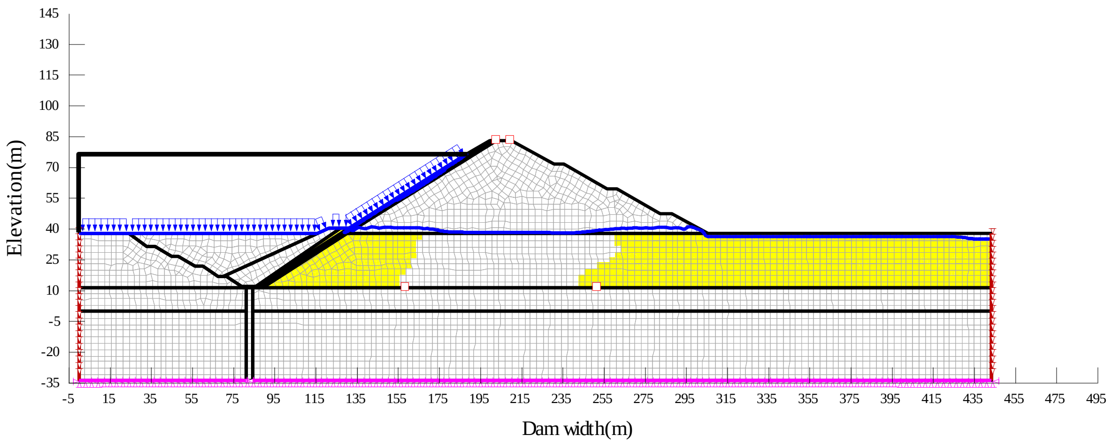


Figure 38: Zones of potential liquefaction of loose sand (Yellow color) - Kobe Earthquake record- For design basis earthquake (DBE).

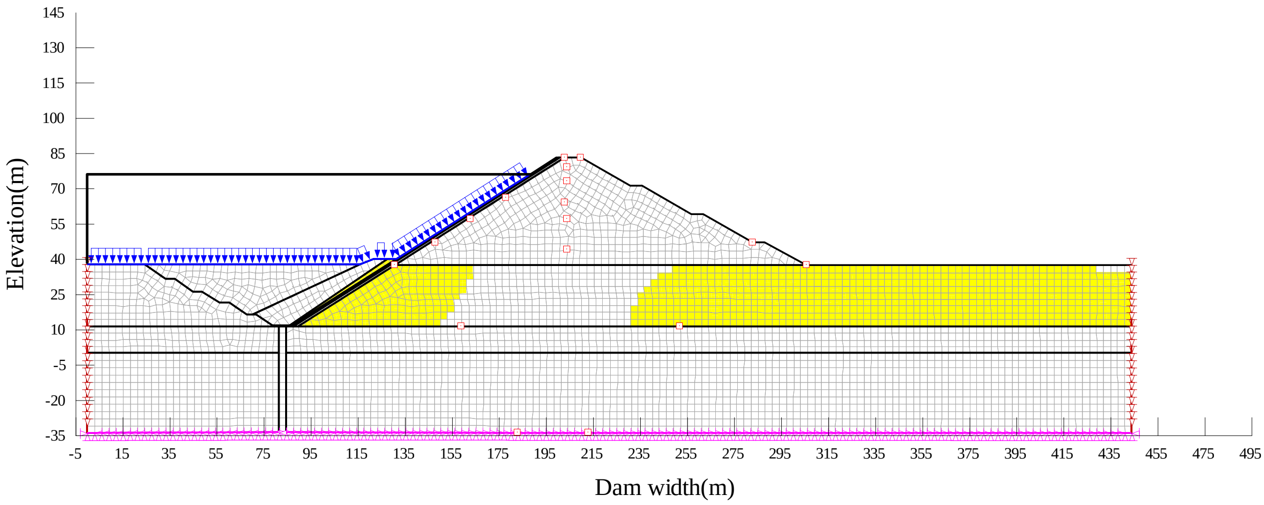


Figure 39: Zones of potential liquefaction of loose sand (Yellow color) – Hachinohe Earthquake record- For design basis earthquake (DBE).

5.4 Results of Slope Stability

As discussed in Chapter 4 in materials and methods, the Lower Awash Multi-Purpose Project is located within the rift segments of Afar regional state and there by an appropriate design measures are required considering earthquake effects.

Therefore, the major concern for this Concrete Faced Rock Fill Dam will be dynamic analysis but at the same timestatic analysis for steady state loading conditions havebeen analyzed and are presented.

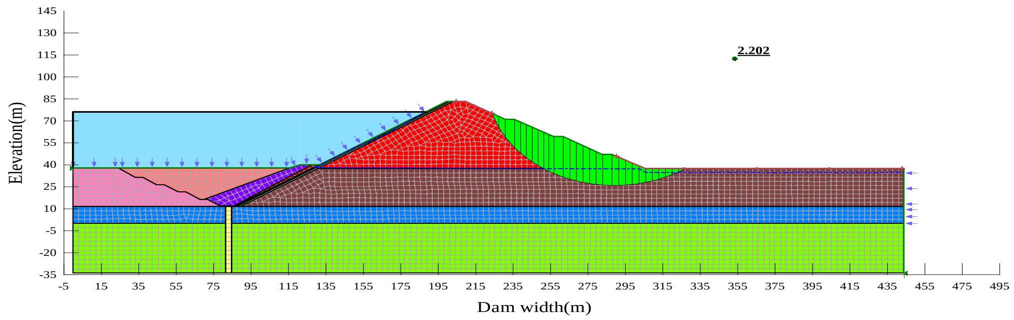


Figure 40: Factor of safety and critical slip surface (Steady state seepage stability analysis without earthquake for downstream slope)

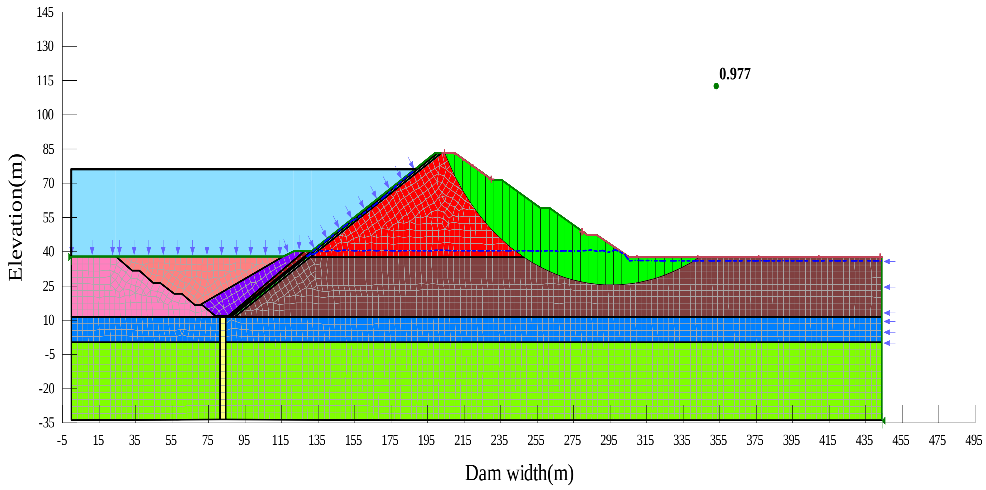


Figure 41: Factor of safety and critical slip surface (Steady state seepage stability analysis with earthquake for downstream slope-Elcentro (MCE))

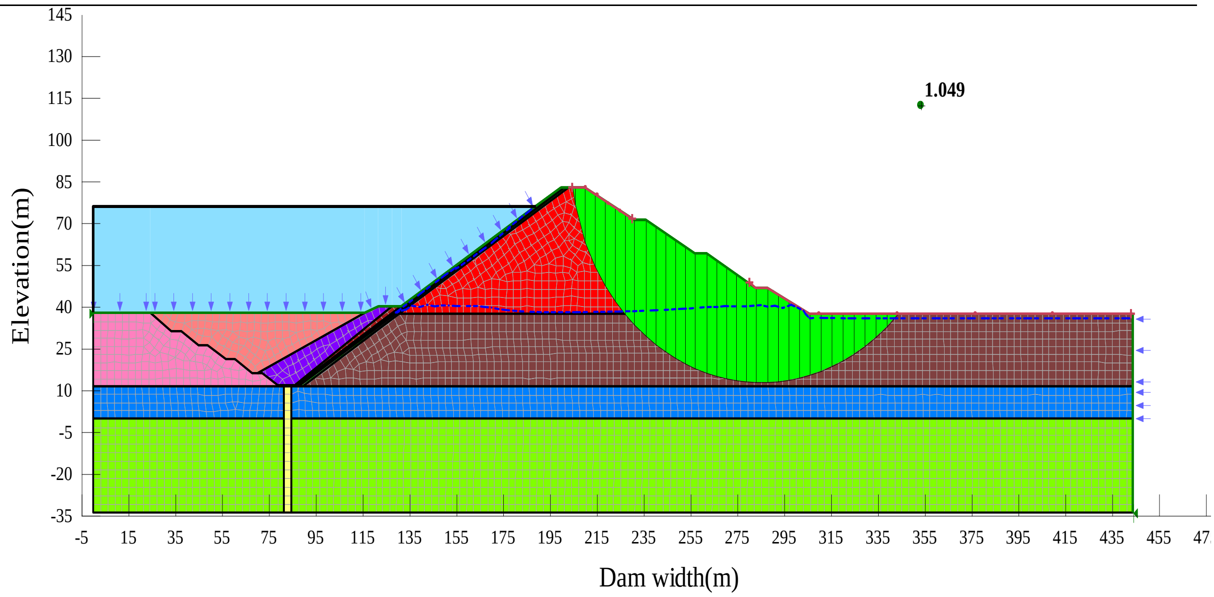


Figure 42: Factor of safety and critical slip surface (Steady state seepage stability analysis with earthquake for downstream slope-Kobe (MCE))

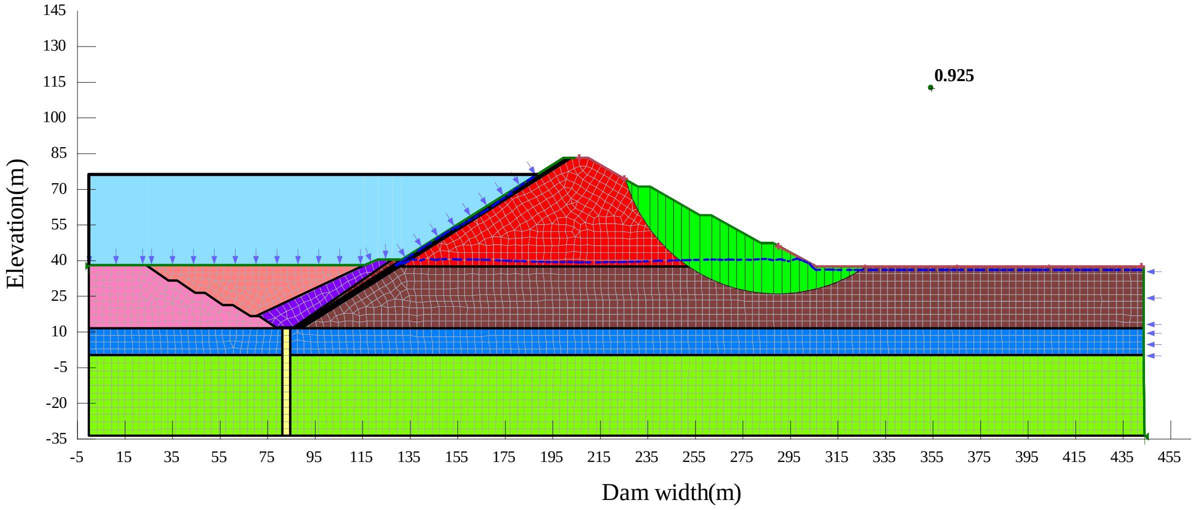


Figure 43: Factor of safety and critical slip surface (Steady state seepage stability analysis with earthquake for downstream slope-Hachinohe (MCE))

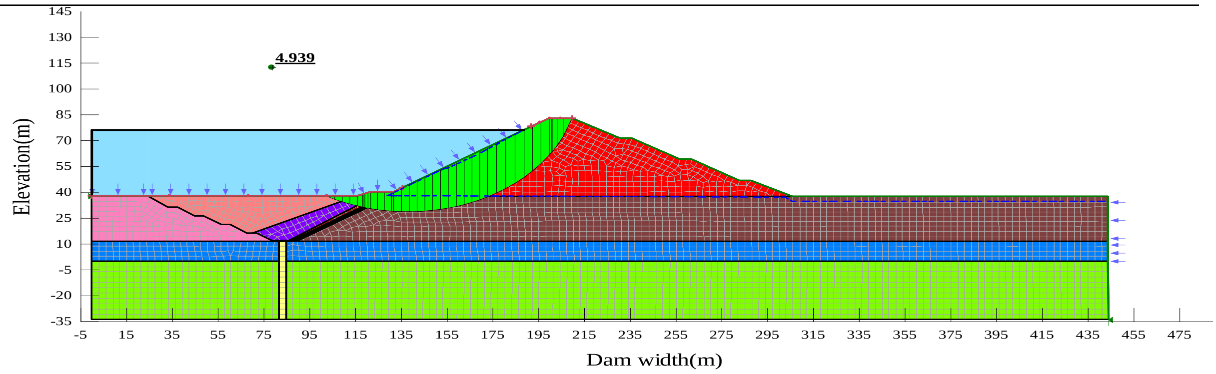


Figure 44 : Factor of safety and critical slip surface for upstream slope without earthquake.

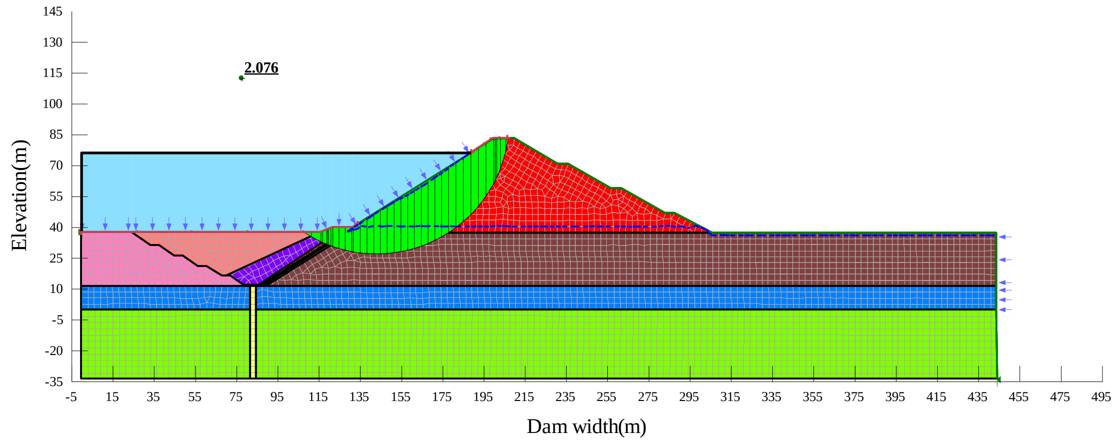


Figure 45: Factor of safety and critical slip surface for upstream slope with earthquake- Elcentro (MCE).

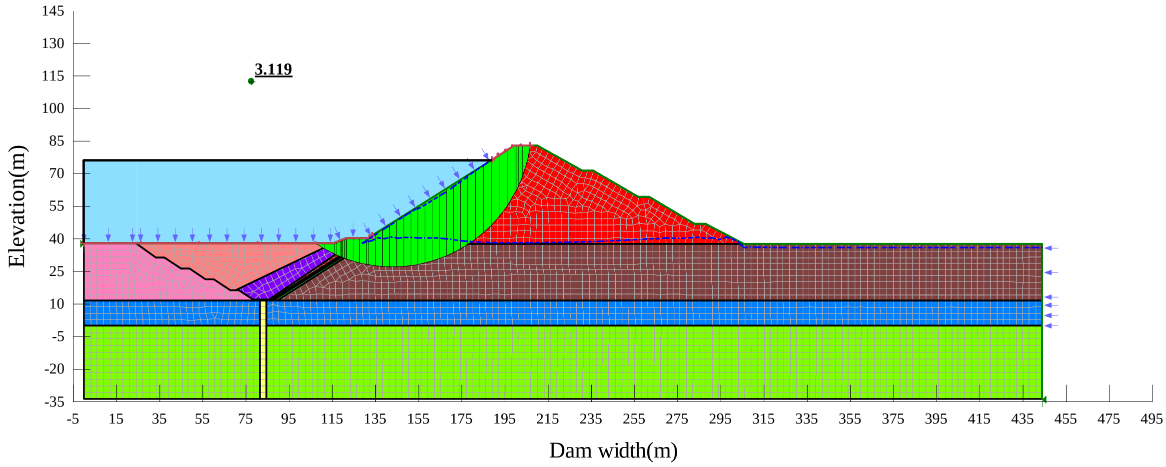


Figure 46: Factor of safety and critical slip surface for upstream slope with earthquake- Kobe (MCE).

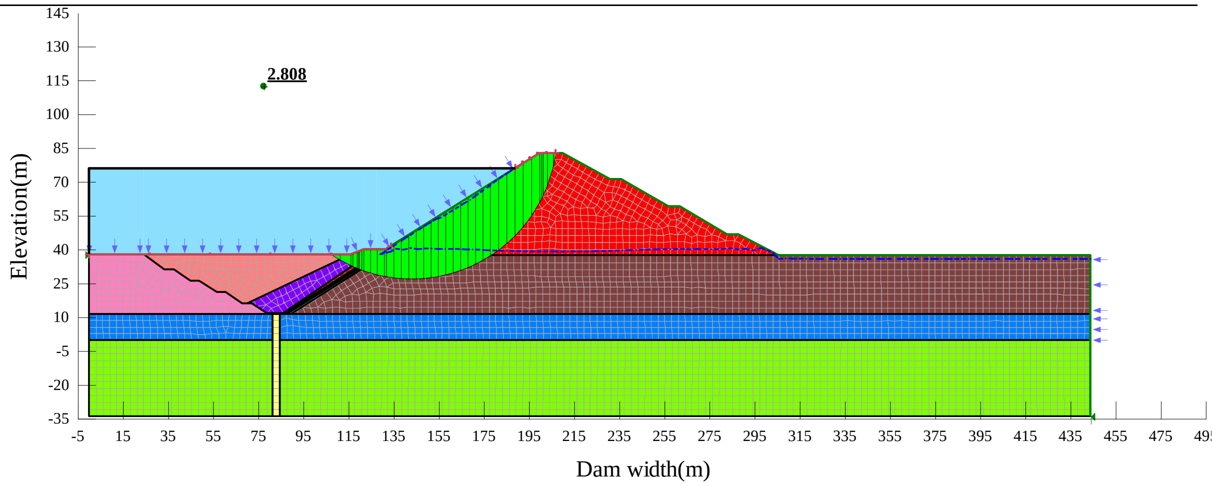


Figure 47: Factor of safety and critical slip surface for upstream slope with earthquake- Hachinohe (MCE).

Summary of factor of safety calculation after earthquake analysis are shown below the table:

Table 18: Slope stability factor of safety after earthquake (Post-earthquake analysis)

Slope stability after earthquake					
Earthquake Type	Earthquake considered	LEM		FEM	
		Upstream	Downstream	Upstream	Downstream
DBE	Elcentro	2.477	1.03	5	1.13
	Hachinohe	3.277	0.932	5	0.988
	Kobe	3.141	1.092	5	1.181
MCE	Elcentro	2.076	0.977	5	1.115

	Hachinoe	2.808	0.925	4.487	0.883
	Kobe	3.119	1.049	5	1.043

5.4.3 Newmark Deformation Analysis

SLOPE/W can use the results from a QUAKE/W dynamic analysis to examine the stability and permanent deformation of the dam subjected to earthquake shaking using a procedure similar to the Newmark method. SLOPE/W can use these stresses to analyze the stability variations during the earthquake and estimate the resulting permanent deformation. This dynamically driven mobilized shear is divided by the total slide mass to obtain an average acceleration. This average acceleration for the entire potential sliding mass represents one acceleration value that affects the stability at an instance in time. The shaking of the slope is 30 seconds long and is modeled with 1500 time steps.

The Newmark deformation analysis for Kobe, Hachinohe and Elcentro earthquake showed no deformation at all.

- Newmark Deformation slope stability of Hachinohe for MCE

For different slip surfaces, different factor of safety and permanent deformation recorded for MCE.The below figures presented calculated safety factor for sample slip surfaces using LEM.

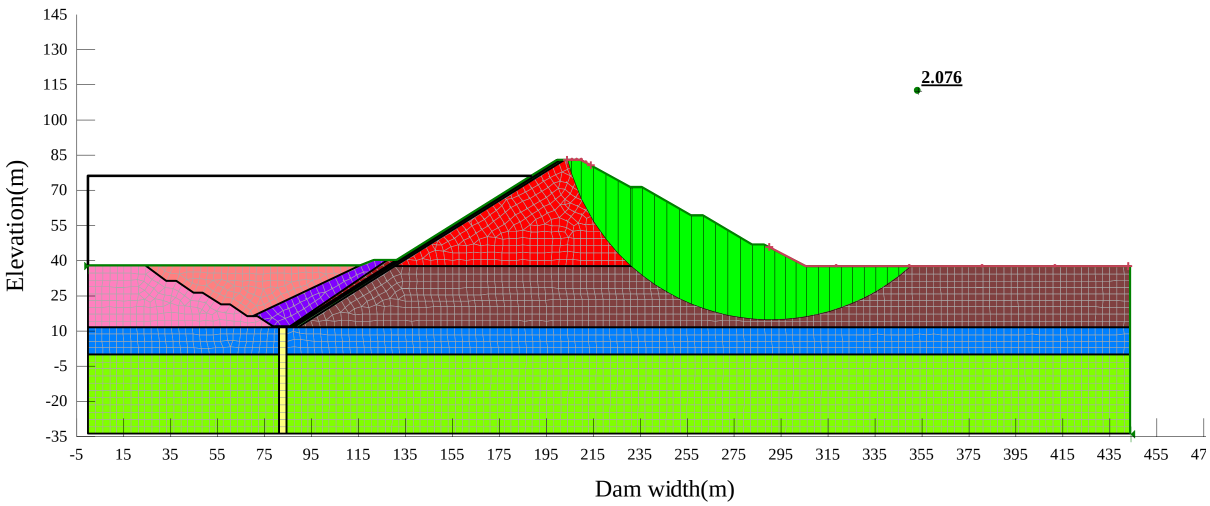


Figure 48: Factor of safety using Newmark deformation for d/s slope

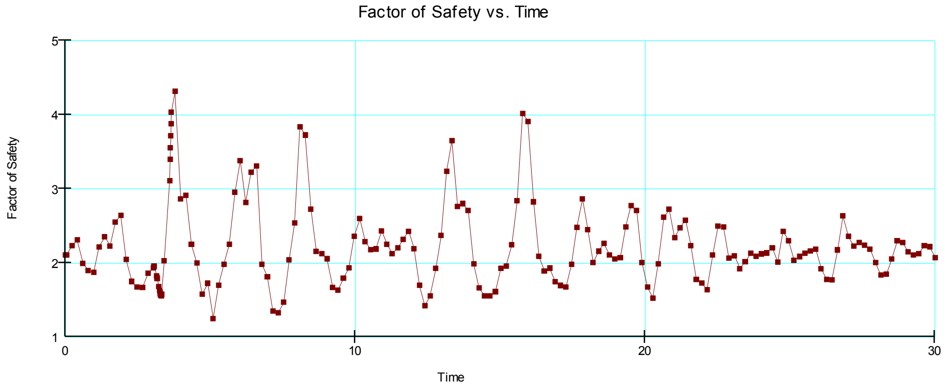


Figure 49: Factor of safety vs time during the 30 shaking.

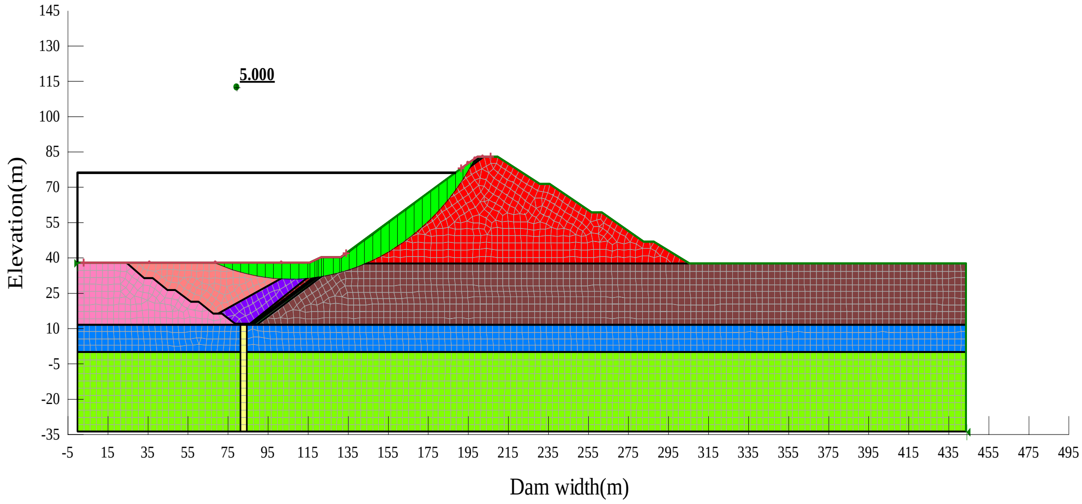


Figure 50: Factor of safety using Newmark deformation for u/s slope

Figure below shows the computed factor of safety versus the average acceleration of the slip surface.

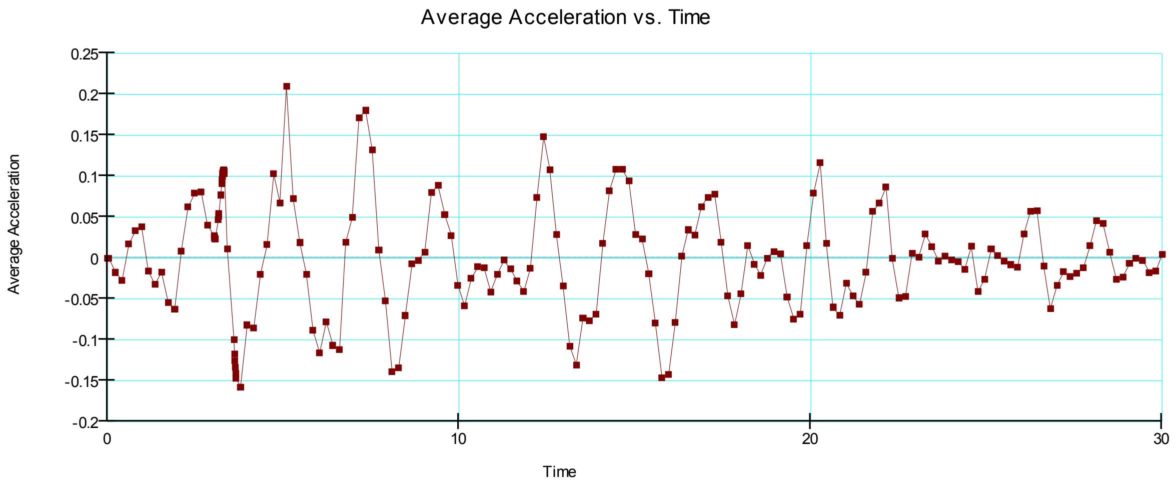


Figure 51: *Factor of safety versus average acceleration*

The Newmark deformation analysis for Hachinoe, Kobe and Elcentro earthquake showed no deformation at all.

The main intention is to determine reduction in free board. From the above analysis result, there is no reduction of more freeboard and thus recommended camber height is sufficient.

In general, for the selected dam type by the consultant, i.e. zoned Earth-rock fill dam with central clay core seepage analyses through the dam and its foundation have been conducted using the state-of-the-art Finite Element Methodbased computer program – SEEP/W from Geo-Studio international, 2007.

The computed seepage for zoned earth –rock fill dam with central clay core and its foundation is $3.85*10^{-6}m^3/s$ and $0.48*10^{-6}m^3/s$, per meter width of the dam respectively. Therefore, Concrete Faced Rock Fill Dam with central clay core shall perform better than Zoned Rock Fill Dam with Clay Core in terms of controlling seepage quantity.

Zoned Earth-Rock Fill Dam with Central Clay Coreinitial static stress have been conducted using SIGMA/W– from Geo-Studio international, 2007.The computed vertical stress obtained is 1600 kpa and the analysis result showed that the stress level for Zoned Earth-Rock Fill Dam with Central Clay Core should be better than Concrete Faced Rock Fill Dam (2000kpa).
In general, the analysis result showed that the stress level for Zoned Earth-Rock Fill Dam should be better than Concrete Faced Rock Fill Dam (2000kpa).

Dynamic analysis of Zoned Earth-Rock Fill Dam with Central Clay Core has been computed using QUAKE/W, 2007. The main purpose of thedynamic analysis is to determine the excess pore-pressures that may develop, and identifyzones where the soil may have liquefied or where the soil strength may have dropped downto the steady-state strength for maximum credible earthquake (MCE) and Safety evaluation earthquakes (SEE) . The earthquake time –history records ofElcentro are used as earthquake data inputs.

The analysis result showed that the foundation is completely liquefied even Kobe of earthquake records of short duration. Finite element stress analysis has been carried out for the duration of the earthquakeconsidering the pore water pressure generated as a result of earthquake shaking.

SLOPE/Whas the capability to use the specified steady-state strength along the portion of apotential slip surface that passes through an element that QUAKE/W has marked asliquefied. The post-earthquake slope stability analysis result with pore pressures and specified steady state strength in the liquefied zones, results in very low factor of safety in the case of Zoned earth-rock fill dams.Accordingly, the computed minimum factor of safety for the downstream slope at the end ofthe earthquake duration is 0.323 whereas the minimum factor of safety within the earthquakeduration is 0.19.

Where as in the case of Concrete Faced Rock Fill Dam, the computed factor of safety in both Maximum credible earthquake and safety Evaluation earthquake shall be better than Zoned earth-rock fill dam with central clay core.
The thick alluvium foundation completely liquefies for all seismic loading.

6. Conclusions and Recommendations

6.1Conclusion

This paper describes Dynamic Analyses of 48m high Concrete Faced Rock fill Dam as an alternative for Lower Awash Dam and Irrigation project under clay core material constraint for Zoned Earth-rock fill dam with central clay core selected by the project office, WWDSE. The dam is located in high seismic zone in the lower awash sub-basin of Logia river.

The analysis was aimed to investigate the settlement and post-earthquake stability behaviour of the dam subjected to earthquake loading of MCE and SEE using QUAKE/W 2007 software and also conducted using numerical analysis of Limit Equilibrium and Finite Element method. An acceleration-time-history data of Elcentro, Hachinohe and Kobe earthquake records are used as earthquake inputs.

The analysis result showed that computed horizontal accelerations are somewhat larger under the influence of 1940 Elcentro earthquake record with amplifications of 1.15 at the dam crest, The amplification which can be an indication that properties of the earthquake motion other than the maximum acceleration might also be influential on dynamic behavior.

The post-earthquake slope stability analysis ensures stability of upstream slope of 1.115. However,downstream slope stability is somehow undermined (0.977), but Newmark deformation analysis procedure showed no deformation at all for the three earthquakes. Therefore, do not require additional extension of dam height because of the allowed sufficient cumber height of 1.626m under detail design of WWDSE.

Theliquefactionassessmentshowedthatthe25mthickalluviumfoundationunderthe damliquefiesafter theearthquake shakingforallthethreetypesofearthquake inbothSEE andMCEcases.Thus,thealluviumfoundationisveryweakthateven Kobeearthquake,whichhaveashortdurationofpeakacceleration.The soil completely liquefies on average after 5 seconds of earthquake shaking for the three time-history earthquake record.

The adoption of concrete face rock fill dams for Lower Awash dam project allow to optimize the rock fill volume and would reduce the volume limitation of clay core.

6.2 Recommendation

The alluvium foundation below the dam is immediately liquefied following the earthquake shaking. Therefore, it is necessary to replace the alluvium foundation with a selected material.

Further increase of dam height and cumber is not required because the dam‟s SEE and MCE upstream deformations, which includes the crest, can be entertained by the allowed cumber height.
I recommend to do further research work for the site in the future if more research works are done related to this type of dam in international level. Design of this type of dam is still based on engineering judgement and experience (Cooke, 1984; Nunez, 2007b).

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AppendixA

Modifiedtime -historydataofKobe, Elcentro andHachinoheEarthquakes

The1995KobeJMARecord,Japan(M=7.2,H=14.3km,R=19km)

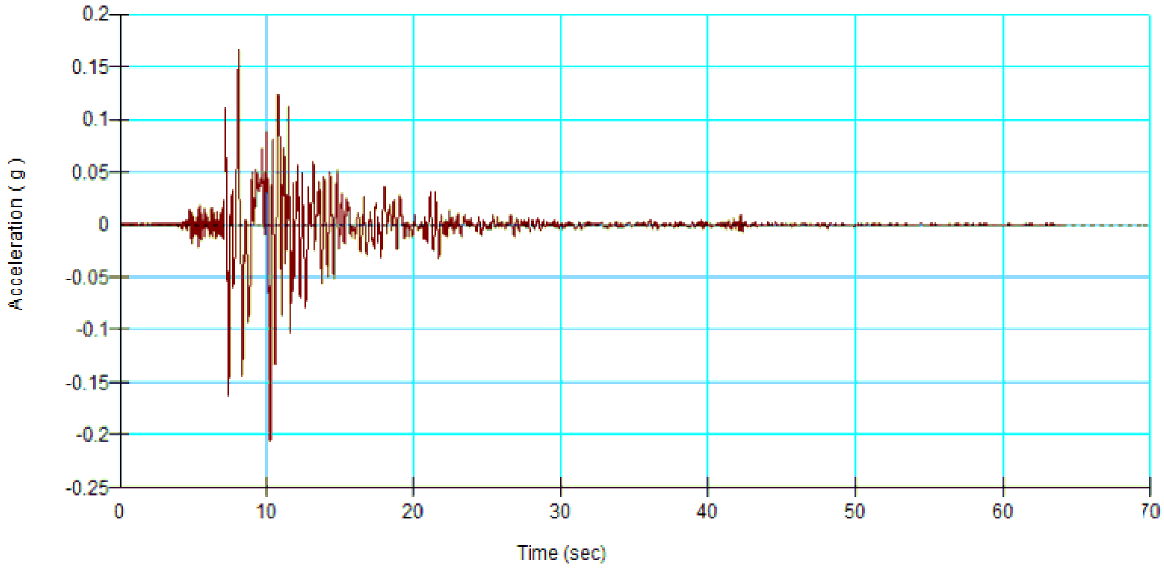

Figure A-1: Maximum Credible Earthquake: Horizontal

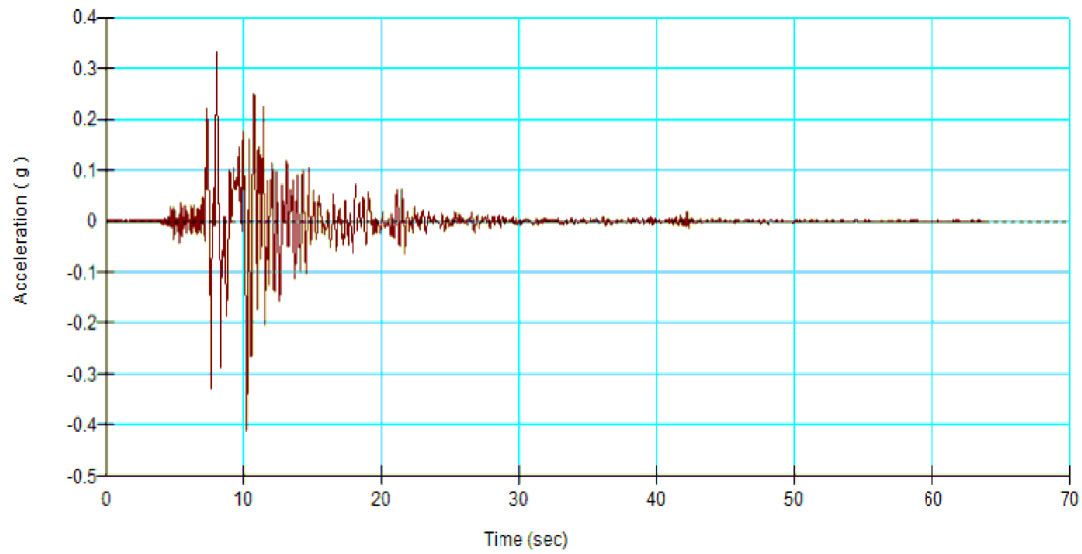

Figure A-2: Maximum Credible Earthquake: Vertical

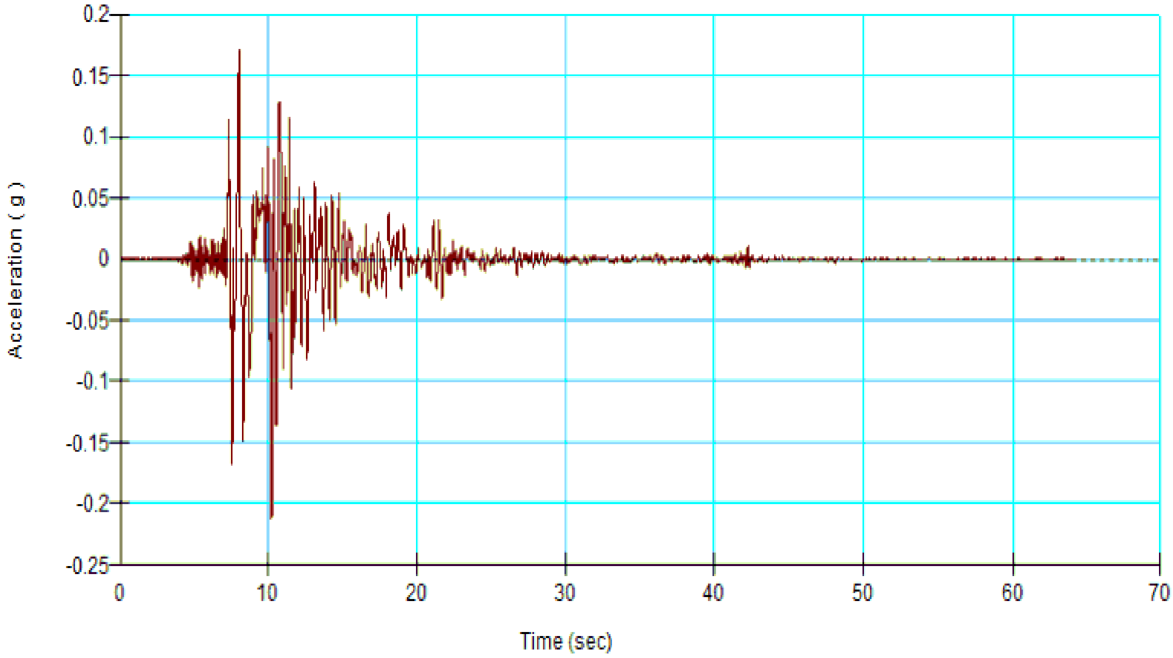

Figure A-3: Design Base Earthquake: Horizontal

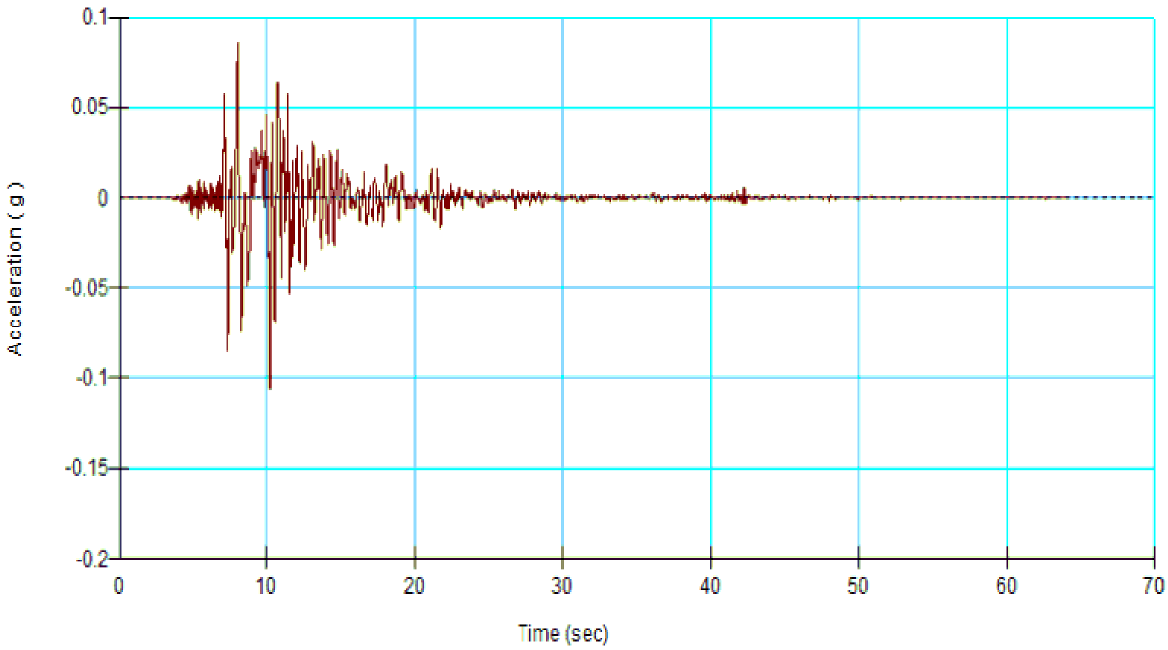

Figure A-4: Design Base Earthquake: Vertical

The1968HachinoheRecord,Japan(M=7.9H=0km,R=200km

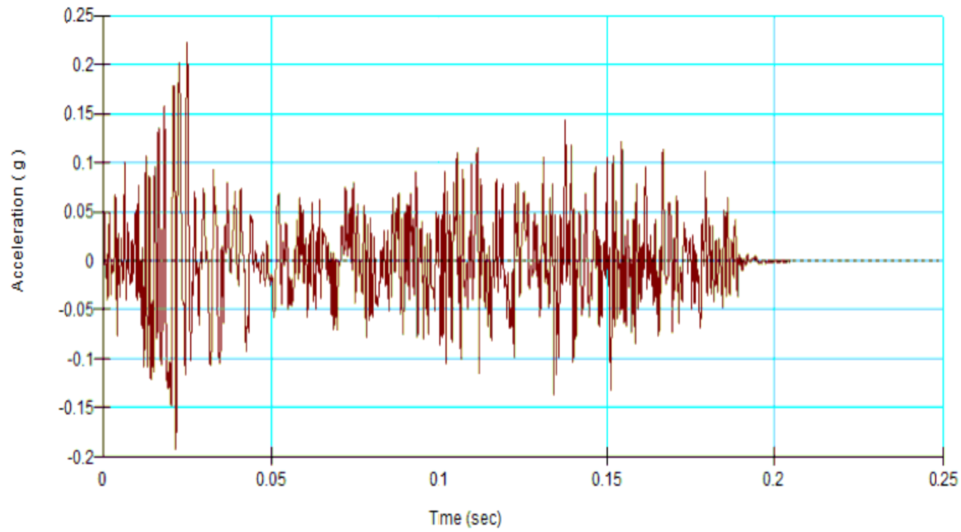

Figure A-5: Maximum Credible Earthquake: Horizontal

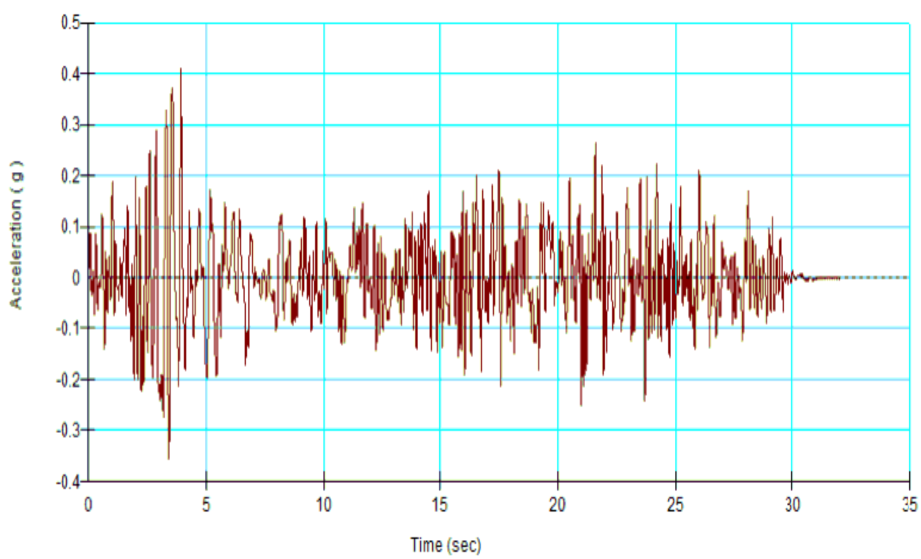

Figure A-6: Maximum Credible Earthquake: Vertical

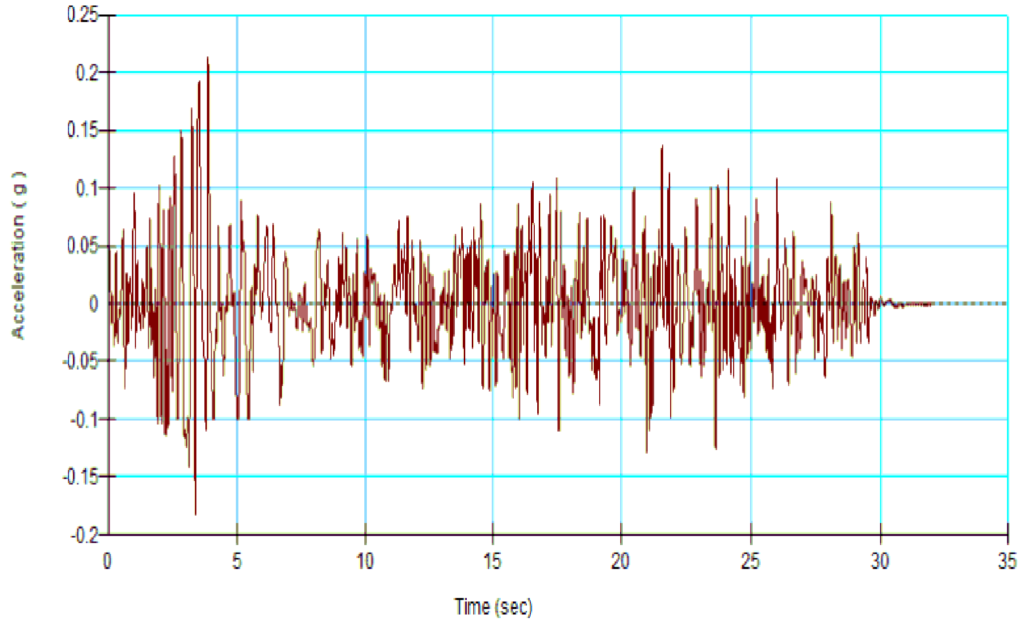

Figure A-7: Design Base Earthquake: Horizontal

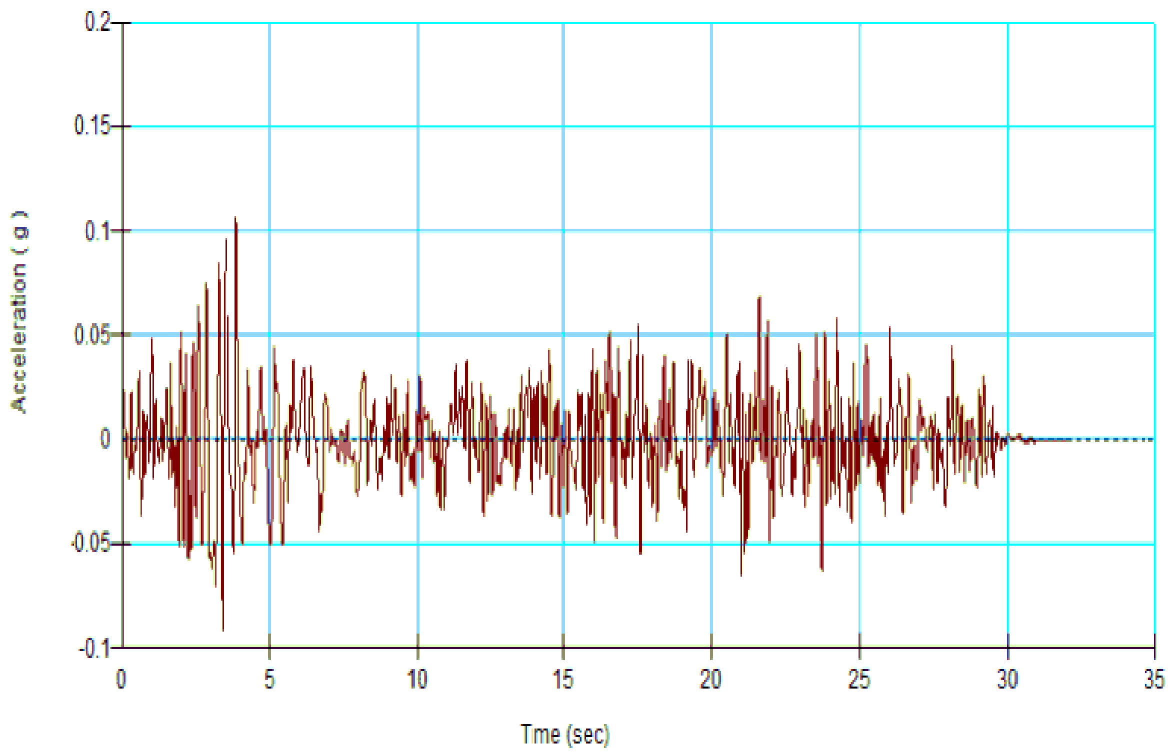

Figure A-8: Design Base Earthquake: Vertical

The 1940 Elcentro Record, USA (M=6.7, H=11km, R=11.5km)

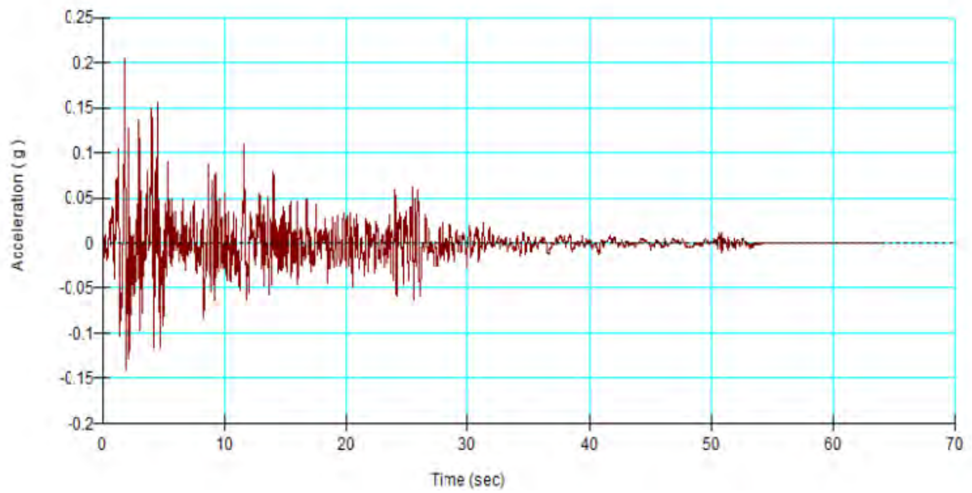

Fig.A-9 Maximum Credible earthquake- Horizontal

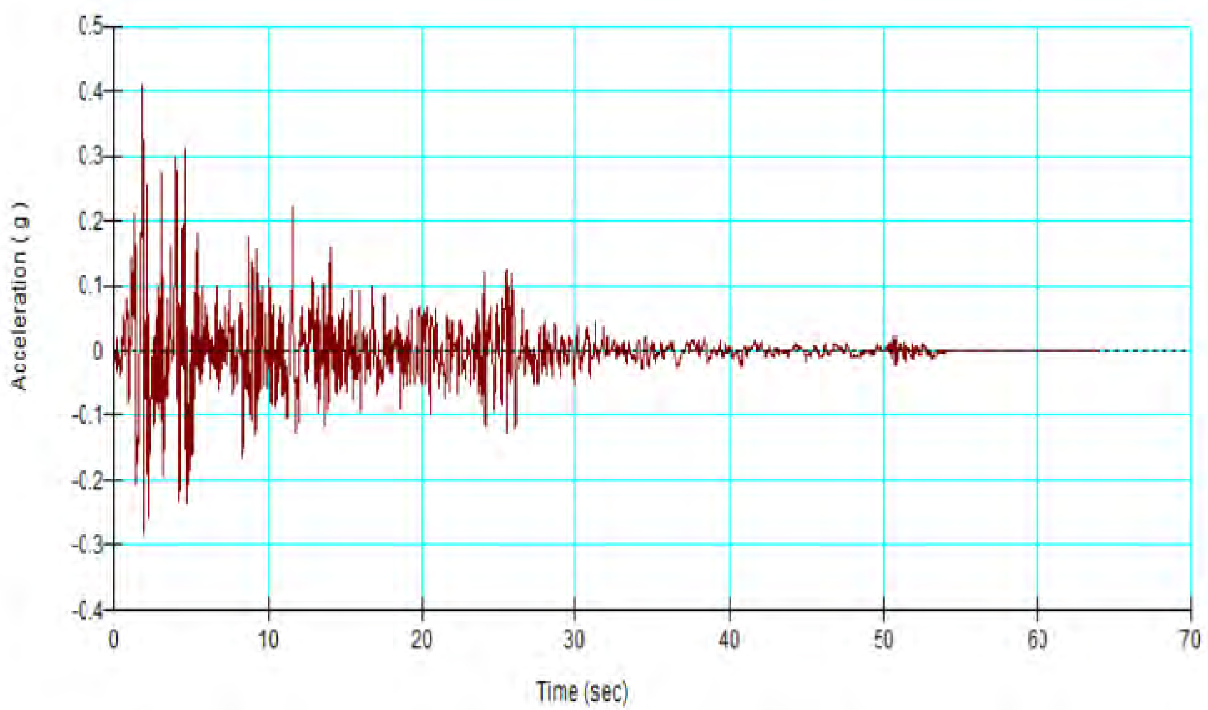

Fig A-10. Maximum Credible earthquake- Vertical

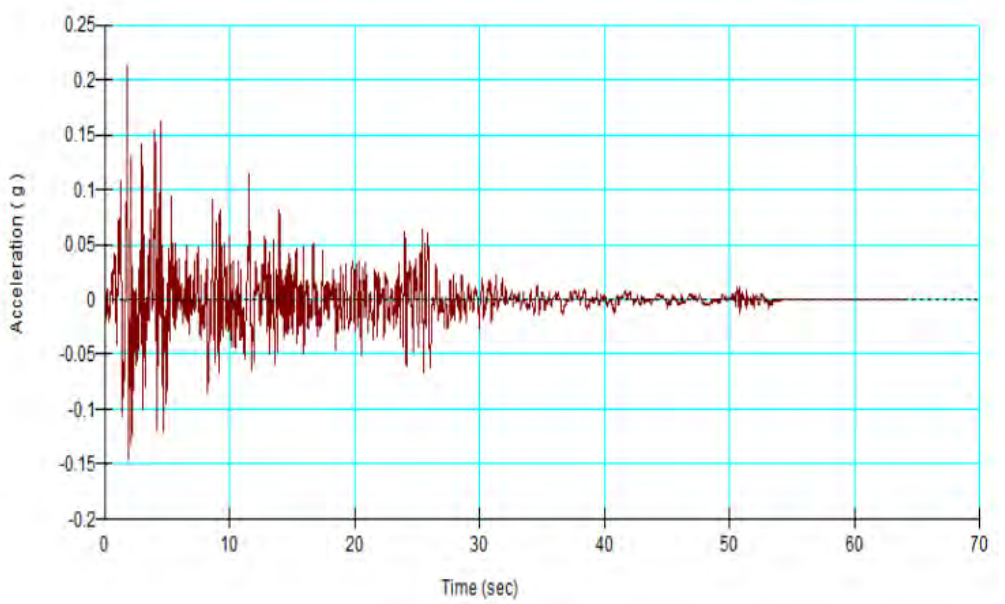

Fig.A-11 Design base earthquake- Horizontal

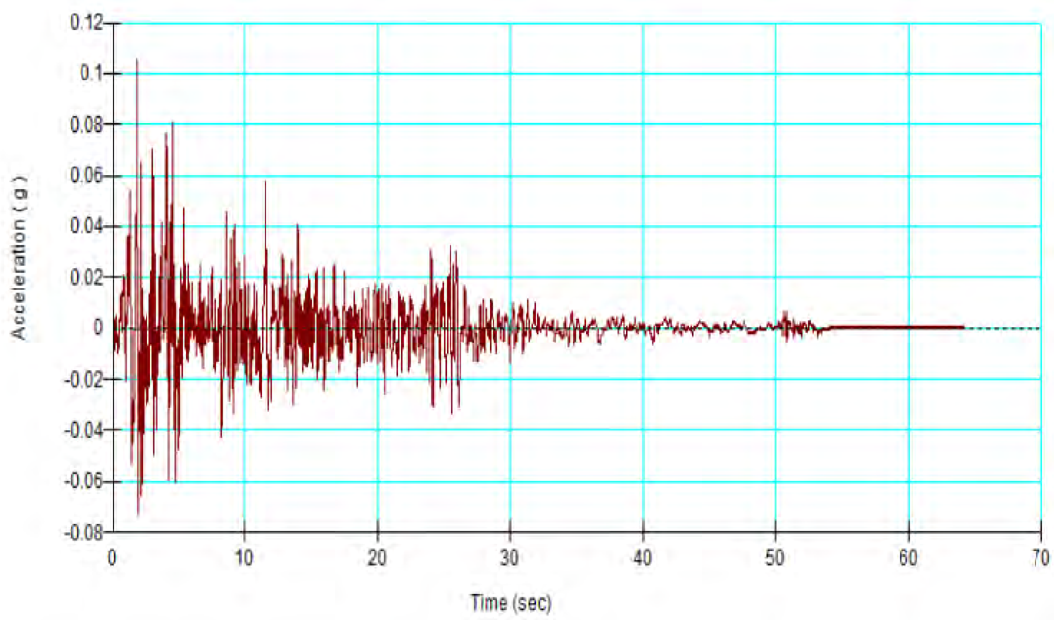

Fig.A-12 Design base earthquake- Vertical