EARTHQUAKE INDUCED LIQUEFACTION ANALYSIS OF TENDAHO EARTH-FILL DAM

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ABSTRACT

This paper presents earthquake induced liquefaction analysis of Tendaho earth-fill dam, which is part of Tendaho Dam and Irrigation Project; the largest irrigation project in Ethiopia to date. The dam is located in the most seismic part of Ethiopia and was originally designed to be founded on potentially liquefiable alluvium foundation. The analysis presented in this paper is the first of its kind in the country and has resulted in significant changes in the original design of Tendaho dam. Geological formations of the dam site, behavior of different materials used for building the dam, and site specific earthquake have been incorporated in the analysis. The dynamic analysis results revealed that the loosely deposited alluvium foundation would completely liquefy under earthquake loading, endangering the stability of the dam. Hence, following the recommendations made in this study, the 6 to 10 m thick alluvium foundation under the dam seat has been completely removed prior to placement of the dam. Construction of the dam has been completed recently.

Keywords: *Embankment dams, Earthquake, dynamic analysis, DBE, MCE, Pore pressure, Liquefaction.*

INTRODUCTION

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. The possible effects of earthquakes on the safety of dams were first taken into account by the engineering profession as early as the middle 1920s. In the early 1930s, design practice usually considered earthquake effects by simply incorporating, in the stability or stress analysis for a dam, a static lateral force intended to represent the inertia force induced by the earthquake. This method of approach is termed as the pseudostatic analysis and was the only method used to assess the seismic stability of dams until the late 1960s. In the 1960s and early 1970s a number of earthquake-induced dam failures occurred, which led to an increasing concern that the pseudostatic method of analysis could not always predict the safety of dams against earthquake shaking, as a result of which increasing emphasis was given to the use of dynamic analysis methods where appropriate. At about the same time, new tools for making improved analyses of seismic response had become available (finite element method and high-speed computers).

Current methods of analysis generally use the pseudostatic method for reasonably well-built dams on stable soil or rock foundations and if estimated peak ground accelerations are less than 0.2g. However, in areas where peak ground accelerations may exceed 0.2g and for dams involving embankment or foundation soils that may lose a significant fraction of their strengths under the effects of earthquake shaking, a dynamic analysis should be performed [1]. The principal objectives of a dynamic analysis of embankment dams are assessment of liquefaction potential of susceptible materials and determination of permanent deformations. This paper deals with the first objective, which is earthquake induced liquefaction analysis.

Tendaho dam is a part of Tendaho Dam and Irrigation Project, which at present is the largest irrigation project in the country. The purpose of the project is to harness the inflows of river Awash for sugar cane plantation in an area of 60,000 hectares. The project will help in setting up sugar factories having target production of 500,000 tones of sugar per annum, as part of development plan for lower Awash Valley [2]. The Tendaho earth-fill dam is located in the most seismic part of Ethiopia. Based on seismic studies previously conducted by the Department of Earth Sciences at Addis Ababa University as well as those indicated in the Tendaho dam design report [2], peak ground acceleration at the dam site is estimated to be as high as 0.3g. It was, therefore, essential to carry out a dynamic analysis prior to the execution of the dam. The analysis was particularly crucial for Tendaho dam, where the original design was found to be unsafe during dynamic loading and following the recommendations made in this study a major foundation design change has been proposed and implemented during construction.

EARTHQUAKE DATA

Peak Ground Acceleration (PGA)

Following the guidelines recommended by the International Commission on Large Dams (ICOLD), two different earthquakes – the Maximum Credible Earthquake (MCE) and the Design Base Earthquake (DBE) – have been used [3]. The MCE is the largest reasonably conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework. The DBE is the earthquake which is expected to occur at least once during the expected life period of the dam. ICOLD suggests that under the DBE condition, structure of the dam should not be significantly impaired and should remain operational, even though some deformation is acceptable. ICOLD also indicates that for embankment dams, the MCE should not cause the dam [4]:

a) to lose its free board.

b) to fail due to liquefaction of material in the dam or its foundations.

c) to collapse due to movement at a slip surface in the slope or through the foundation.

PGA data (corresponding to MCE and DBE) from seismic studies previously conducted by the Department of Earth Sciences at Addis Ababa University and additional studies, all referred from the Tendaho dam design report [2] are considered for the analyses. Table 1 below summarizes the design horizontal and vertical PGA used in the analyses. Based on international design standards [5], the vertical PGA has been taken as half of the horizontal component.

Table 1: Design PGA

	Horizontal	Vertical
Maximum Credible Earthquake (MCE)	0.3g	0.15g
Design Base Earthquake (DBE)	0.18g	0.09g

Acceleration Time History (ATH)

The dynamic analysis of the dam has been carried out by a Finite Element Method based state of the art computer program QUAKE/W from GeoSlope International Ltd. [6]. Horizontal and vertical acceleration time histories are key input parameters for QUAKE/W analysis. Therefore, site specific horizontal and vertical ATH for Tendaho dam should be produced using the peak accelerations and records of actual earthquakes. However, because there are no ATH records near the dam site, actual accelerographs recorded elsewhere have been used. The following three ATH data have been considered for the analysis [7].

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

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

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

The 1940 Elcentro record appears to have the closest resemblance with the earthquake records reported for the Tendaho dam site area. However, in order to represent other possible earthquakes with different magnitude, time duration and frequency content, the 1995 Kobe JMA record (with shorter duration, big pulse) and the Hachinohe record (with longer duration) have also been considered.

In order to remove site and path effects, deconvolved ATH data have been used in the analyses for all the three cases [7]. Also, the three ATH data have all been scaled to PGA values of 0.3g horizontal and 0.15g vertical corresponding to site specific MCE, and 0.18g horizontal and 0.09g vertical corresponding to site specific DBE. Figure 1 to 3 below show the deconvolved ATH curves used for the analyses. In order to save some paper space, only the ATH curves corresponding to the horizontal MCE are presented here.

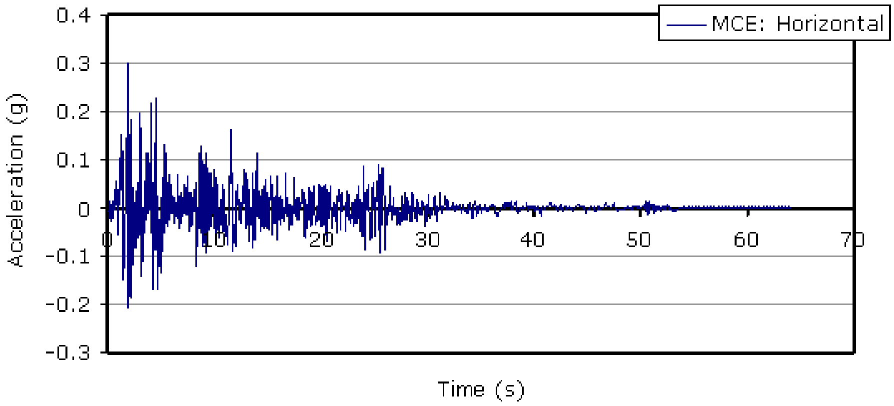

Figure 1 Horizontal maximum credible earthquake – 1940 Elcentro record

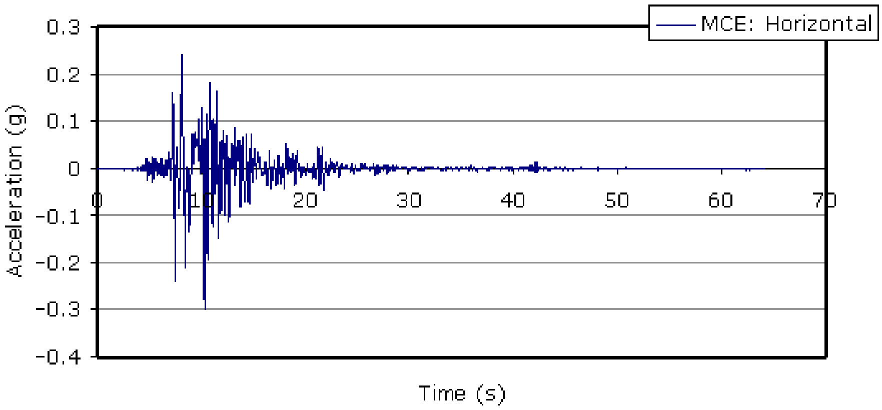

Figure 2 Horizontal maximum credible earthquake – 1995 Kobe JMA record

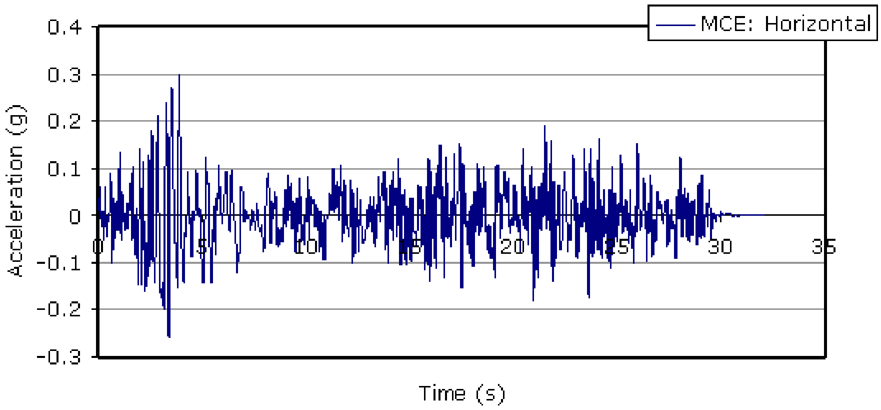

Figure 3 Horizontal maximum credible earthquake – 1968 Hachinohe record

LIQUEFACTION ANALYSIS

Liquefaction Potential Based on Grain Size

Liquefaction is one of the major effects of earthquakes, in which water saturated cohesionless soils temporarily lose strength and fail during shaking. The mechanism for this is, during strong shaking with no or limited drainage, cyclic shear stresses produce a progressive buildup of pore water pressures that significantly reduce the effective stress, which controls the strength of the soil. This pore water pressure development primarily depends on particle shape, size, and gradation. Most liquefaction is observed in clean sands. Well-graded soils are generally less susceptible to liquefaction than poorly graded soils. According to Kramer [5], most liquefaction failures in the field have involved uniformly graded soils. The first step to evaluate the potential of liquefaction is, therefore, identification of grain size distribution of the soil.

Figure 4 shows grain size distribution boundaries separating liquefiable and non-liquefiable soils proposed by Tsuchida [9] and widely used by geotechnical engineers worldwide. As shown in Figs. 5 and 6, Tsuchida's boundaries are used for the assessment of liquefaction susceptibility of the Tendaho dam shell materials and alluvium foundation, respectively.

The liquefaction susceptibility on the basis of average grain size distributions (Fig. 5) shows that the Tendaho dam shell materials are well graded and 50 to 75% of the materials lie outside the boundaries for potentially liquefiable soils.

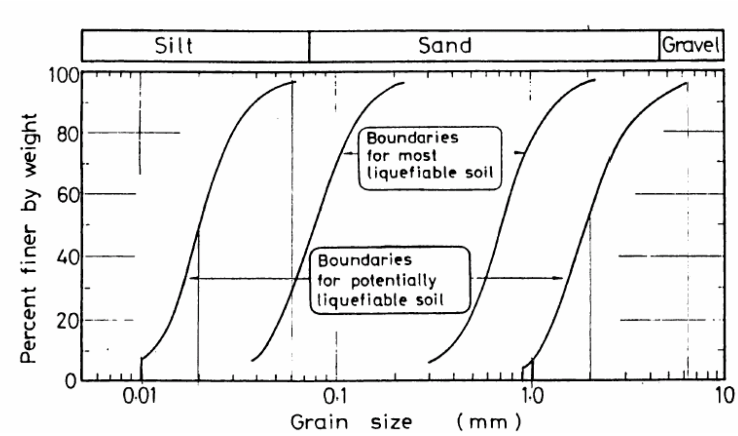

Figure 4 Boundaries separating liquefiable and non-liquefiable soils [9]

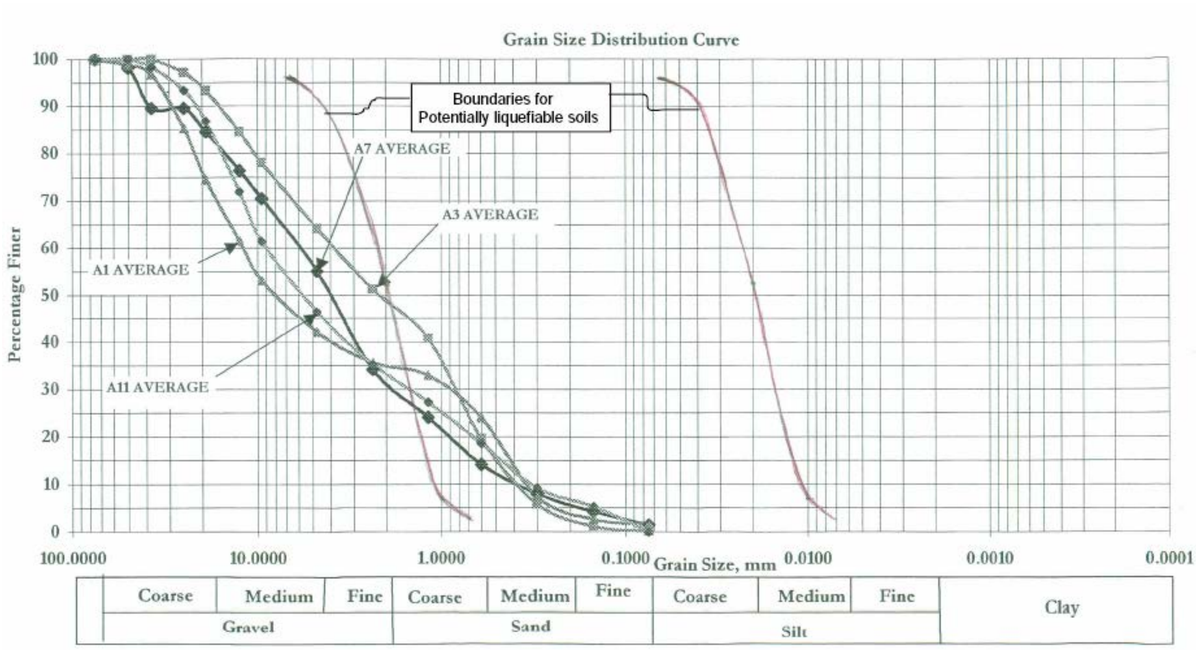

Figure 5 Average gradation curves from different shell material borrow areas [2]

Alluvium Foundation

Based on the observations made during a site visit to the Tendaho dam site and subsequent laboratory tests, it was found that the alluvium foundation under the dam seat is largely comprised of lose sand and silty sand soils.

As shown in Fig. 6, the liquefaction susceptibility on the basis of grain size distribution indicates that over 85% of the alluvium foundation soils lie within the boundaries for potentially liquefiable soils. Therefore in the dynamic analysis of the Tendaho dam, the alluvium foundation has been considered to be comprised of potentially liquefiable soils.

Finite Element Model

Figure 7 shows the finite element model used for the dynamic analysis of Tendaho dam. The model is prepared using the QUAKE/W program for the maximum cross-section of the dam. Both structured and unstructured meshes are used. The bottom boundary of the model has been taken at the surface of the basalt bed-rock underlying a 6 m alluvium and a 20 m lake sediment stratum as shown in Fig. 8. In order to minimize the disturbance due to the boundary wave reflection in the dynamic analyses, the side boundaries were extended by 150 m (about 3 times the dam height) on both left and right directions. To account for damping of the soils on left and right boundaries, both horizontal and vertical dumping boundary conditions have been applied on left and right ends of the models.

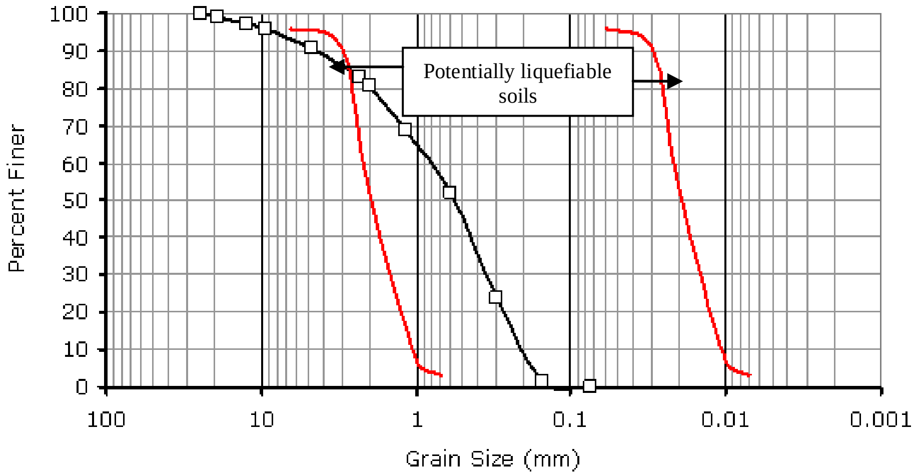

Figure 6 Liquefaction susceptibility of alluvium foundation

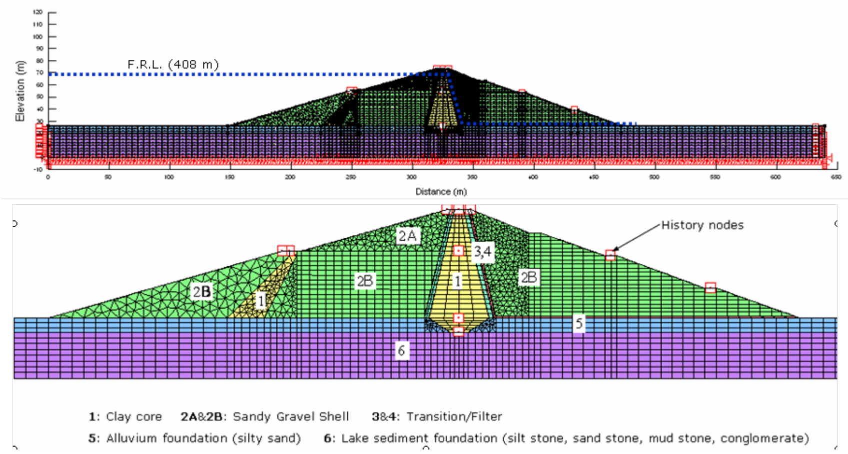

Figure 7 Numerical model used for the dynamic analysis

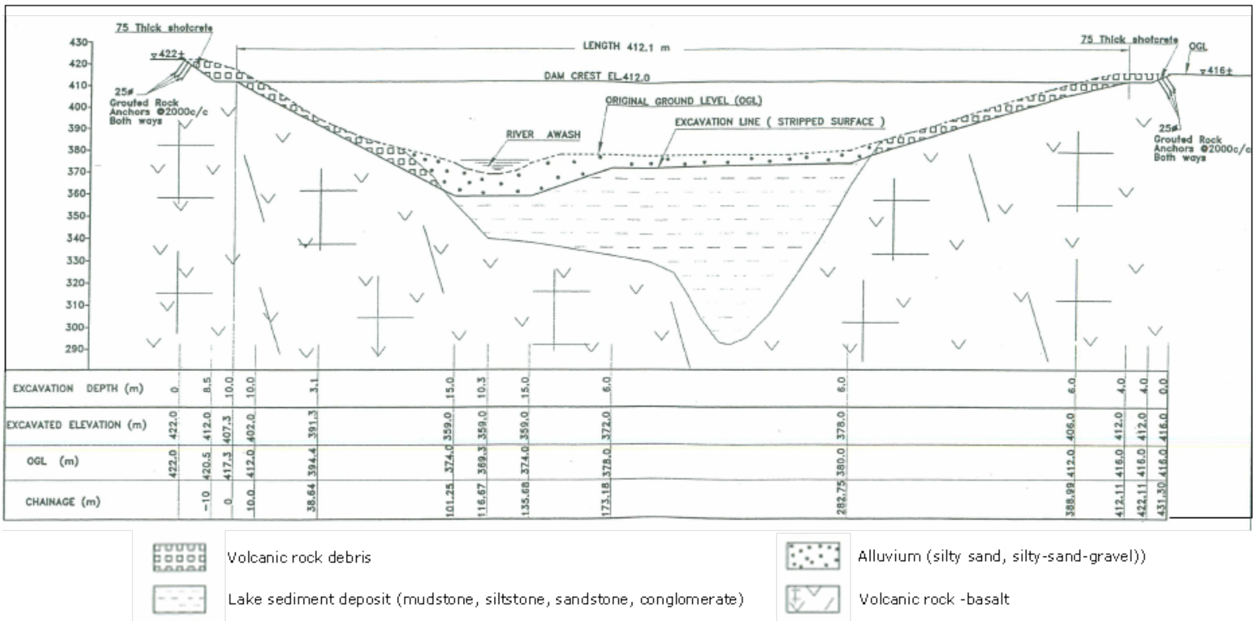

Figure 8 Geological profile along Tendaho dam axis [2]

DYNAMIC MATERIAL PROPERTIES

The dynamic characteristics of the dam materials have not been investigated by means of dynamic triaxial tests. Therefore, the material properties required for the dynamic analysis were estimated with the help of the geotechnical literature as will be explained in the next sections.

Stiffness as a Function of Depth

The soil stiffness is generally a function of the stress state. As the confining stress increases, the soil stiffness increases. QUAKE/W uses the following relationship to describe the soil stiffness as a function of depth [6].

$$G = K_G(\sigma'_m)^n \qquad (1)$$

where G is the shear modulus, K_G is a soil modulus, σ'_m is the mean effective stress, and n is a power exponent (generally n is taken as 0.5). To determine G_{max} and the corresponding soil modulus K_G, the following widely used empirical equation, which was developed by Seed and Idriss [10], has been utilized.

$$G_{\max} = 220 \cdot K_{2\max}(\sigma'_m)^{0.5} \text{ (in kPa)} \qquad (2)$$

From Eqs. (1) and (2), we get,

$$K_G = 220 \cdot K_{2\max} \qquad (3)$$

According to Seed et al. [11], the magnitude of K_{2max} for gravels ranges between 80 to 180. The K_{2max} values for Tendaho dam materials are determined based on the curves published by Seed and Idriss [10]. The Tendaho shell materials are composed of sandy gravel soils, thus a K_{2max} value of 90 is used. Tables 2 summarize the K_{2max} values and the corresponding K_G values used in the analyses. Assumed Poisson's ratios (υ) for each material are also shown in these tables. The K_{2max} value for the clay core is determined based on the publication by Malla et al. [12].

Table 2: K_{2max} and K_G values based on Eq. (3)

Material	K_{2max}	K_G	υ
Clay Core	50	11000	0.4
Sandy Gravel Shell	90	19800	0.3
Transition/Filter	70	15400	0.3
Alluvium Foundation	70	15400	0.3
Lake Sediment Foundation	90	19800	0.3

Shear Modulus Reduction and Damping Ratio Functions

As the dynamic shear strain increases, the effective dynamic shear modulus becomes smaller than the maximum value G_{max}. At the same time, the nonlinear response at higher dynamic strains leads to a higher rate of energy dissipation, which is represented by a damping ratio that increases at higher strain levels. The strain-dependent dynamic shear modulus and damping ratio values for different soils and rock published by Sun et al. [13] and Idriss [14] have been used for Tendaho dam dynamic analysis.

Pore Pressure Function

The pore pressures developed during earthquake shaking are a function of the equivalent number of uniform cycles N for a particular earthquake and the number of cycles N_L, which will cause liquefaction for a particular soil under a particular set of stress conditions. According to Seed et al. [15], N = 10 for earthquake magnitude M = 7.0 (corresponding to MCE), and N = 6 for M = 6.5 (corresponding to DBE). The ratio of N/N_L is then related to a pore pressure parameter r_u [5, 6]. Lee and Albaisa [8] and DeAlba et al. [16] found that the pore pressure function can be described by the following equation:

$$r_u = \frac{1}{2} + \frac{1}{\pi}\sin^{-1}\left[2\left(\frac{N}{N_L}\right)^{1/\alpha} - 1\right] \qquad (4)$$

The above equation is used to estimate the pore pressure function in QUAKE/W. For saturated sand α = 0.7 [17]. As described above, the alluvium foundation is largely comprised of recently deposited lose sand and silty sand soils and has been assumed to be a potentially liquefiable soil. Therefore, the pore pressure function shown in Fig. 9 (obtained using Eq. (4), for α = 0.7) has been used for the alluvium foundation of Tendaho dam.

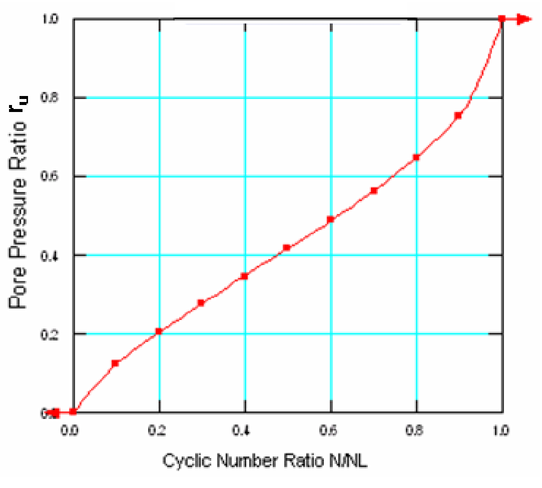

Figure 9 Pore pressure function used for alluvium foundation

As stated earlier, assessment of the liquefaction susceptibility on the basis of grain size distribution for Tendaho Sandy gravel shell shows that 50 to 75% of the shell materials lie outside the boundaries for potentially liquefiable soils. Moreover, the shell materials are in most cases well graded and suitable for achieving good compaction (or densification) which minimizes liquefaction susceptibility. Therefore, the Tendaho shell materials are considered to be non-liquefiable soils but with a potential for some pore water pressure build up during earthquake shaking. To account for the pore water pressure build up, a constant r_u value of 0.3 has been used. This value has been determined based on the publication by Malla et al. [12], which deals with the dynamic analysis of a 75 m earth-fill dam in India. For the clay core r_u = 0.35 has been used based on Tendaho dam design report [2].

Cyclic Number Function

A Cyclic Number Function must be attached to the Pore Pressure Function so that N_L is defined. For high shear stress ratios (defined as the ratio of cyclic deviatoric stress to initial static effective vertical stress), only a few cycles may be required to cause liquefaction, while for low ratios, a larger number of cycles are required. The cyclic number function specifies this relationship. DeAlba et. al. [18] and USNRC [19] have published cyclic number function curves obtained from shaking table tests on sand. Based on these publications, the Cyclic Number Function shown in Fig. 10 (corresponding to D_r = 70%, which was considered in the dam design report) has been used for the alluvium foundation of the Tendaho dam.

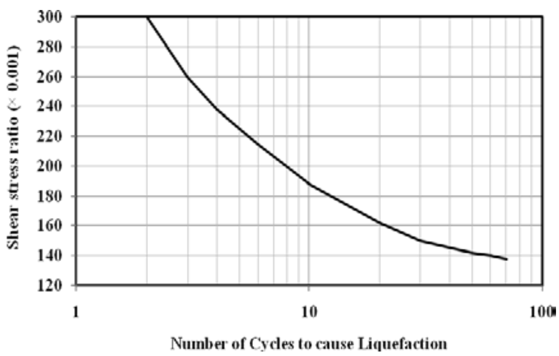

Figure 10 Cyclic number function used for the analysis [18, 19]

The number of cycles required to cause liquefaction (N_L) can be corrected for overburden and initial static shear stresses by attaching K_a and

$K_{\sigma\square}$ correction functions to the cyclic number function which are discussed below.

Overburden Pressure Correction Function, $K_{\sigma\square}$

The cyclic shear stress required to trigger liquefaction increases as the confining stress increases [5]. In QUAKE/W a $K_{\sigma\square}$ function is specified to account for this. Marcuson et al. [20] reported variation of correction factor $K_{\sigma\square}$ with effective overburden pressure for different soils. The $K_{\sigma\square}$ correction function is attached to the cyclic number function and is specified as part of the cyclic number function data. The overburden correction factor influences N_L and therefore has an effect on the pore-water pressure value that is computed.

Based on the work of Marcuson et al. [20], the $K_{\sigma\square}$ function shown in Fig. 11 (corresponding to the estimated average curve for sand) has been used for the alluvium foundation of the Tendaho dam.

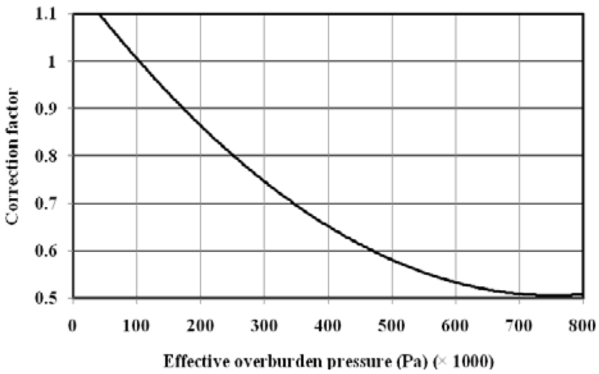

Figure 11 $K_{\sigma\square}$ correction function used for the analysis [20]

Shear Stress Correction Function, K_a

The initial in-situ static shear stresses also influence the cyclic stress required to trigger liquefaction [5]. This function is dependent on density of the soil. Seed and Harder [21] reported the shear stress correction function for different relative densities. Accordingly, the K_a function shown in Fig. 12 (corresponding to D_r = 70%) has been used for the alluvium foundation of Tendaho dam.

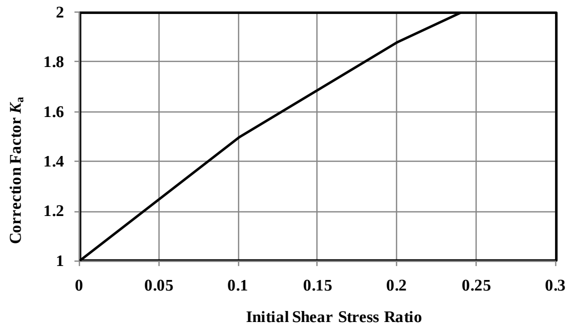

Figure 12 K_a correction function used for the analysis [21]

Initial Static Stress Analysis

Prior to the dynamic analyses, it is essential to establish initial static stress conditions. The initial static stress of the Tendaho dam subjected to the force of gravity was computed by a separate step within the QUAKE/W computer program. The computed initial static stress results were counter checked using manual calculations and the results were in very good agreement. Then, the computed results of the initial static stress and initial pore water pressure were then imported to the dynamic analysis part of the QUAKE/W program.

Liquefaction Analysis Results

The computed liquefied zones of Tendaho dam are shown in Figs. 13 and 14 for Elcentro MCE and DBE, respectively. As can be seen from the shaded zones, the alluvium foundation is entirely liquefied. The liquefaction results for other ATH records were also similar. An observation made on the computed effective stress time history indicates that the alluvium foundation does not liquefy (where liquefaction means zero effective stress) up to 21 seconds in the case of DBE and up to 11 seconds in the case of MCE after the earthquake shaking starts. The computed liquefaction stages at different time steps shown in Figs. 15 and 16 also reveal the same process.

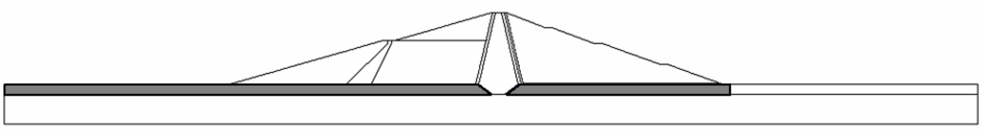

Figure 13 Liquefied zone of Tendaho dam corresponding to MCE

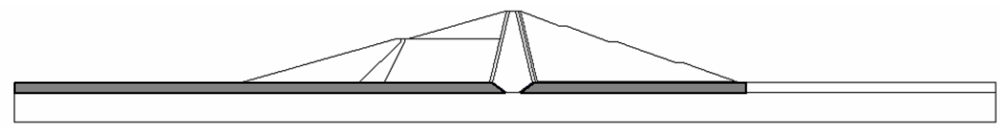

Figure 14 Liquefied zone of Tendaho dam corresponding to DBE

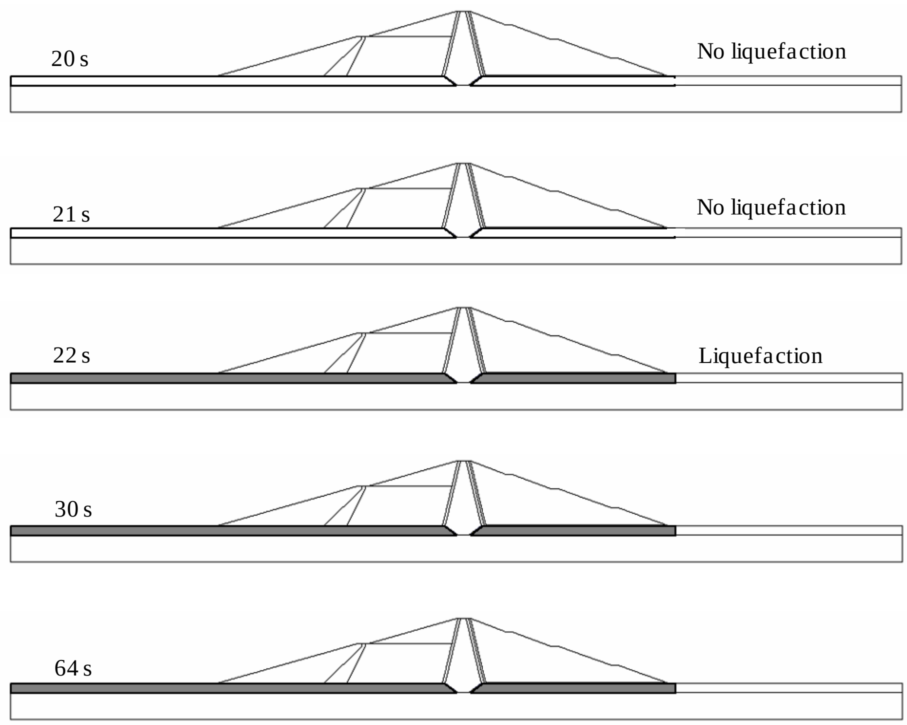

Figure 15 Liquefaction stage at different time steps (DBE)

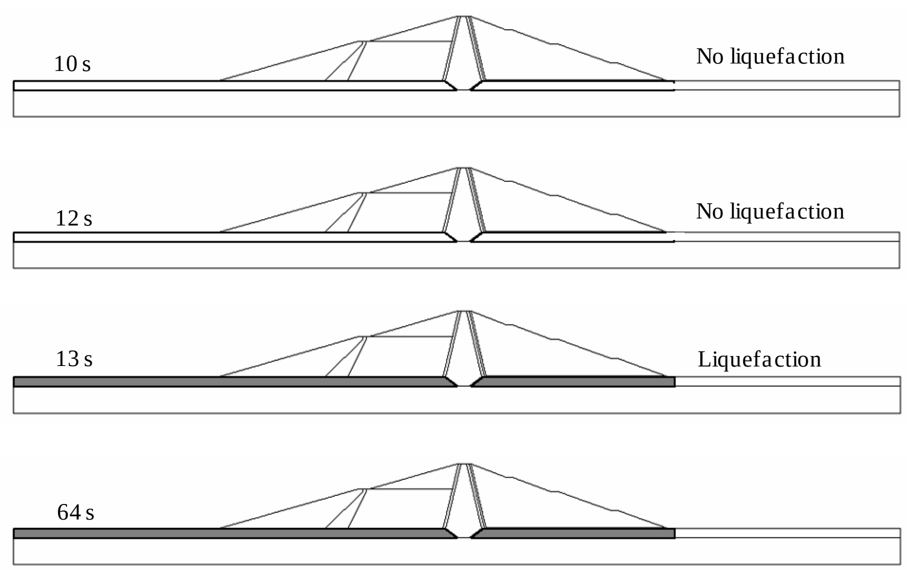

Figure 16 Liquefaction stage at different time steps (MCE)

CONCLUSIONS

This paper presented earthquake induced liquefaction analysis of the Tendaho dam, a major embankment dam located in the most seismic part of Ethiopia. According to assessment results of liquefaction susceptibility on the basis of grain size distribution, most of the Tendaho shell materials could be considered as non-liquefiable soils but with some potential for pore water pressure build up during earthquake shaking. In contrast, the alluvium foundation soils largely lie within the boundaries for potentially liquefiable soils and have, therefore, been considered to be potentially liquefiable. The dynamic analysis results indicate that the alluvium foundation under the dam seat, with a depth ranging from 6 to 10 m would liquefy in the events of both site-specific Maximum Credible and Design Base Earthquakes. Liquefaction of the alluvium foundation would lead to excessive settlement and eventual failure of the dam. Therefore, based on the presented dynamic analysis results, complete removal of the alluvium foundation under the dam seat has been recommended and implemented during execution of the dam.

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REFERENCES

[1] United States National Research Council (USNRC). *Safety of Dams: Flood and Earthquake Criteria*, National Academy Press, Washington, DC, 1985.

[2] WWDSE in cooperation with WAPCOS (India). *Tendaho Dam Final Design Report*, 2005.

[3] ICOLD. *Seismicity and Dam design*, International Commission on Large Dams Bulletin 45, 1983.

[4] Fell, R., MacGregor, P. and Stapledon, D. *Geotechnical Engineering of Embankment Dams*, A.A. Balkema, Rotterdam, 1992.

[5] Kramer, S.L. *Geotechnical Earthquake Engineering*, Prentice Hall, Upper Saddle River, New Jersey, 1996.

[6] Geo-slope International, Ltd., Canada. *Dynamic Modeling with QUAKE/W, QUAKE/W Engineering Workbook*, 2004.

[7] Messele Haile. *Critical Assessment of Site Effect Parameters for Strong Ground Motion Prediction*, Doctoral Thesis, Tokyo Institute of Technology, Japan, 1996.

[8] Lee, K.L. and Albaisa, A. *Earthquake Induced Settlements in Saturated Sands*, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 100, No. GT4, pp. 387-403, 1974.

[9] Tsuchida, H. *Prediction and countermeasure against the liquefaction in sand deposits*, Abstract of the Seminar in the Port and Harbor Research Institute, pp. 3.1-3.33 (in Japanese), 1970.

[10] Seed, H.B. and Idriss, I.M. *Soil Moduli and Damping Factors for Dynamic Response Anaysis*, Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkley, USA, 1970.

[11] Seed, H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K. *Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils*, Journal of Geotechnical Engineering, ASCE, Vol. 112, No. GT11, pp. 1016 – 1032, 1986.

[12] Malla, S., Straubhaar, R., Sorgenfrei, A. and Bhowmik, V.C. *Seismic Analysis and Safety Assessment of Earthfill Dam Founded on Alluvium Soil*, Commission Internationale, Des Grands Barrages, Barcelona, 2005.

[13] Sun, J.I., Golesorkhi, R. and Seed, H. B. *Dynamic Moduli and Damping Ratios for Cohesive Soils*, Report No. UCB/EERC-88/15, Earthquake Engineering Research Center, University of California, Berkeley, 42p, 1988.

[14] Idriss, I.M. *Response of Soft Soil Sites during Earthquakes*, Proceedings, Memorial Symposium to honor Professor Harry Bolton Seed, Berkeley, California, Vol. II, May, 1990.

[15] Seed, H.B, Mori, K., and Chan, C.K. *Influence of Seismic History on the Liquefaction Characteristics of Sands*, Report EERC 75-25, Earthquake Engineering Research Center, University of California, Berkeley, 1975.

[16] DeAlba, P., Chan, C.K. and Seed, H.B. *Determination of Soil Liquefaction Characteristics by Large-scale Laboratory Tests*, Report EERC 75-14, Earthquake Engineering Research Center, University of California, Berkeley, 1975.

[17] Das, B.M. *Principles of Soil Dynamics*, Brooks/Cole, 1993.

[18] DeAlba, P., Seed, H.B. and Chan, C.K. *Sand Liquefaction in Large-scale Simple Shear Tests*, ASCE, Geotechnical Engineering, Vol. 102, No. GT9, pp. 909-927, 1976.

[19] United States National Research Council (USNRC. *Liquefaction of Soils during Earthquakes*, National Academy Press, Washington, DC, 1985.

[20] Marcuson, W.F. III, Handala, P.F. and Ledbetter, R.H. *Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments*, Earthquake Spectra, Vol. 6, No.3, pp. 529-572, 1990.

[21] Seed, H.B. and Harder, L.F. *SPT-based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength*, Seed Memorial Symposium Proceedings, Bi Tech Publishers, 1990.