Seismic Design of a Hydraulic Fill Dam by Nonlinear Time History Method

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SUMMARY:
Time history analyses conducted for the seismic stability and deformation evaluation of Kairakkum Dam is presented. The dam body constructed as hydraulic fill rests on an alluvium layer of 10 m overlying the bedrock. Considering the high ground acceleration values up to 0.78 g, it is found necessary to perform detailed time-history analyses capable of modelling the nonlinear behaviour of soil, the coupled relationship between stress and pore pressure and the reduction in shear strength of the material related to liquefaction. The material parameters are based on the results of geotechnical and geophysical investigations at dam site. Obtained permanent deformations showed that for OBE, the freeboard is sufficient and the dam is safe. For MDE loading, the obtained displacements are greater than allowable limits indicating the unsafety of the dam. In this regard, an improvement of the dam body is found to be necessary and preliminary remedial measures are suggested.

Keywords: Embankment dam, hydraulic fill, seismic

1. INTRODUCTION

Kairakkum Dam is located in Tajikistan, on the Syr-Derya River that comes from Uzbekistan and return to Uzbekistan after passing Kairakkum Reservoir. The dam with a maximum height of 27 m, crest length of 1200 m, reservoir volume of 3131 million m^3 has been constructed with hydraulic fill technique in years between 1952 and 1959. The middle part of the dam body is composed of fine sand and silty material in order to decrease seepage by providing sealing and coarser granular fills take part at the sides to provide support to middle area; besides rockfill buttresses are constituted at both upstream and downstream slopes. The dam body rests on an alluvium layer of about 10 m thickness overlying the bedrock.

The seismic stability of Kairakkum Dam has been examined within the scope of “Consulting Services for Improvement of the Kairakkum Dam and Reservoir Safety and Operations”. Since the dam is located in a region with a very high seismicity, advanced methods are considered as necessary to assess the earthquake safety, and for this purpose time history analyses have been conducted by utilizing FLAC, that is a widely used and accepted computer program to study the dynamic response of embankment dams. The pseudo-static and equivalent linear methods, which have been used in collaboration with time history analyses in the seismic safety assessment of the dam are excluded from the scope of this paper, and detailed information and results are presented for the time history calculations only.

2. PROBLEM DESCRIPTION

2.1. Geometry and Model

The typical cross-section of the dam and the finite difference grid that has been constructed by considering the maximum height of the dam are given in Figures 1 and 2 respectively. Five different material zones are taken into account according to the design as hydraulic fill, foundation alluvium, buttress of granular fill, buttress of rock fill and bedrock. The riprap layer in the upstream has been neglected in the numerical model for simplification, considering that it will not substantially affect the results due to its small thickness. For the dam with a crest length of 30 m, the gradient of the slopes have been designed as 1V/4H, 1V/3H and 1V/2H for the hydraulic fill, buttress of granular fill and buttress of rock fill layers respectively. The middle part of the dam constructed as hydraulic fill rests on an alluvium layer with about 10 m thickness. It is provided for the buttresses in the upstream and downstream sides to rest on bedrock by excavation of the alluvium at those regions. The maximum height of the dam is about 27 m from the foundation level.

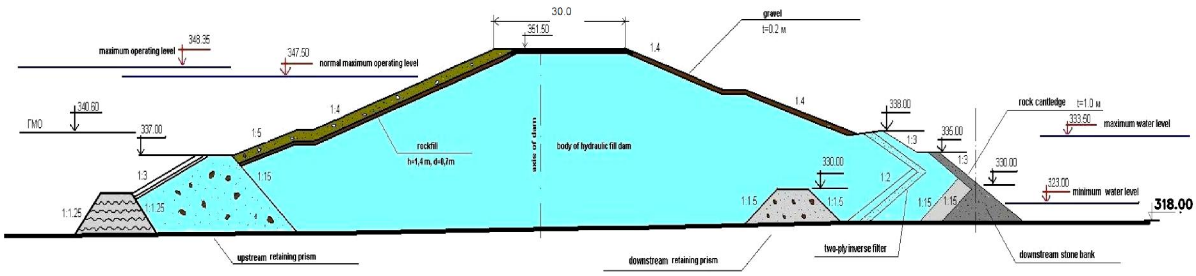

Figure 2.1. Typical Cross-Section of the Dam

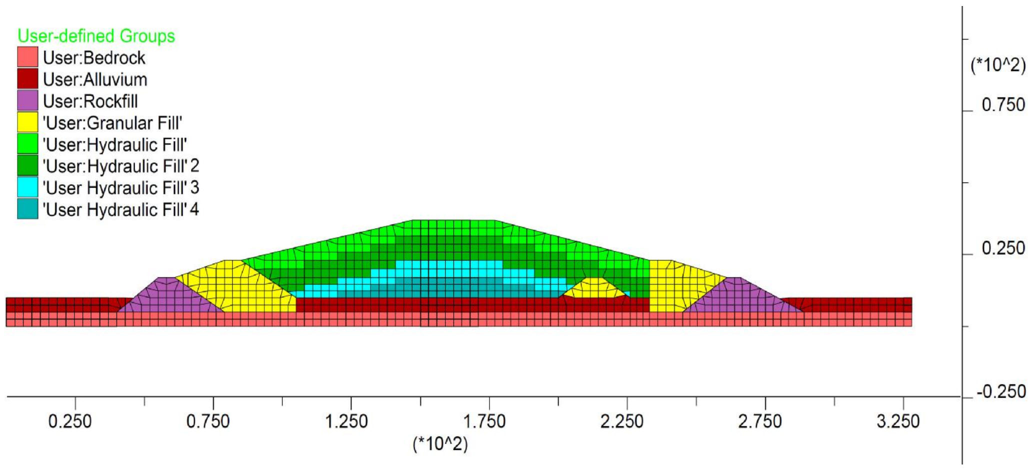

Figure 2.2. Finite Difference Grid

2.2. Material Properties

The material parameters used in the finite difference calculations are summarized in Table 1. The unit weight and the angle of friction for the hydraulic fill are determined according to the average results obtained from the geotechnical investigations. The maximum shear modulus (G_{max}) for the bedrock and hydraulic fill is designated based on the shear wave velocities measured at the site. In the conducted geophysical study, the average shear wave velocity is determined as V_s = 300 m/s for the hydraulic fill. In order to take into account the increase of shear wave velocity with depth, the hydraulic fill zone is divided to four layers and the shear wave velocities are assumed as V_s = 225,275,325 and 375 m/s from up to bottom providing the same average velocity. The shear wave velocity of the bedrock is taken as V_s = 550 m/s according to the geophysical investigation results. The shear wave velocities of the remaining zones are designated by considering typical values in literature for similar soils.

In-situ tests show that the permeability of the hydraulic fill varies between 3.1 and 4.3 m / day. To determine the permeability coefficients for the seepage calculations, the average permeability coefficient of k = 4 m / day is assumed and the potential anisotropy in the fill is taken into account by taking 25 as the ratio of horizontal to vertical permeability, as suggested by Sherard et al.(1963) for uniform soil deposits.

The nonlinear stress-strain behaviour of the embankment dam is considered by using an elastic-perfect plastic material model with Mohr-Coulomb failure criterion for all of the material zones. Moreover, the hysteretic damping – modulus reduction method, dynamically induced pore water pressures and the related liquefaction modelling used in the analyses are presented in Section 2.4.

Table 1. Material Parameters

	γ_{dry}	γ_{sat}	c'	ϕ'	V_s	G_{max}	ν	k_h	k_v
	(kN/m^3)	(kN/m^3)	(kPa)	(deg.)	(m/s)	(MPa)	-	(cm/s)	(cm/s)
Hydraulic Fill	16	20,0	-	30	225 ~375 (ave. 300)	81~225	0,25	2 x 10^{-2}	9 x 10^{-4}
Alluvium	17	20,5	-	32	400	725	0,33	10^{-2}	10^{-2}
Granular Fill	18	21,0	-	36	450	970	0,33	5 x 10^{-2}	5 x 10^{-2}
Rockfill	20	22,5	-	42	500	1330	0,33	10^{-1}	10^{-1}
Bedrock	22	23,5	50	30	550	1600	0,20	10^{-5}	10^{-5}

2.3. Load Cases

The seismicity of the dam site and the seismic design parameters for different earthquake return periods are have been examined with the conducted probabilistic seismic hazard analysis. Accordingly, the peak ground acceleration (PGA) values have been determined as 0.25 g, 0.67 g and 0.78 g corresponding to 150, 5 000 and 10 000 year return periods. For Operating Basis Earthquake (OBE), a 150-year earthquake; and for Maximum Design Earthquake (MDE), 5 000-year and 10 000-year earthquakes have been taken into account. In 1985, when the reservoir level was 13 m below the crest, an earthquake with a magnitude of Mw=6.5 has occurred at the dam site, causing damages like cracks due to shear in the reinforced concrete members in the power plant, and overturning of the transformers.

To be used in the seismic design, elastic design spectra have been developed by the approach suggested by Newmark & Hall (1982), considering the peak ground accelerations for each of the earthquake return periods given above. The damping ratio for the respective design spectra is assumed as 10%, considering the type of the embankment dam. The earthquake loading is carried out by applying acceleration-time histories to the base of the model in the utilized finite difference program. For this reason, it is adopted to use ground motions which are in accordance with the developed elastic design spectra. So, three selected original ground motions as San Fernando – 1971, Kushiro – 1993 and Kocaeli – 1999 are modified in order to provide the frequency content of the ground motions matching the design spectra. As an example, the modified ground motion records and the acceleration response spectra of these with the design spectrum are shown in Figures 3 and 4.

In the nonlinear analyses conducted for embankment dams, the initial stress conditions and the pore water pressures have to be determined correctly prior to the application of the earthquake motion. For this purpose, the results obtained from the separate two analyses of static stress and seepage have been combined and the phreatic line and effective stress distribution has been attained before the seismic loading. After this stage, the earthquake loading has been carried out by means of applying the acceleration-time records to the base of the model.

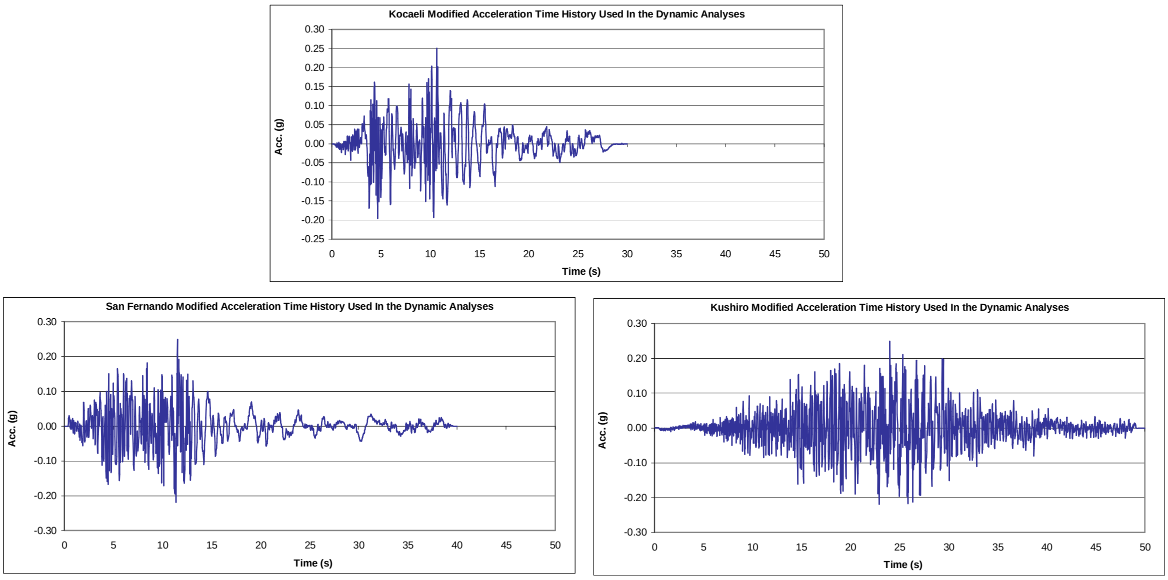

Figure 2.3. Acceleration-Time Histories Used in the Analyses (OBE)

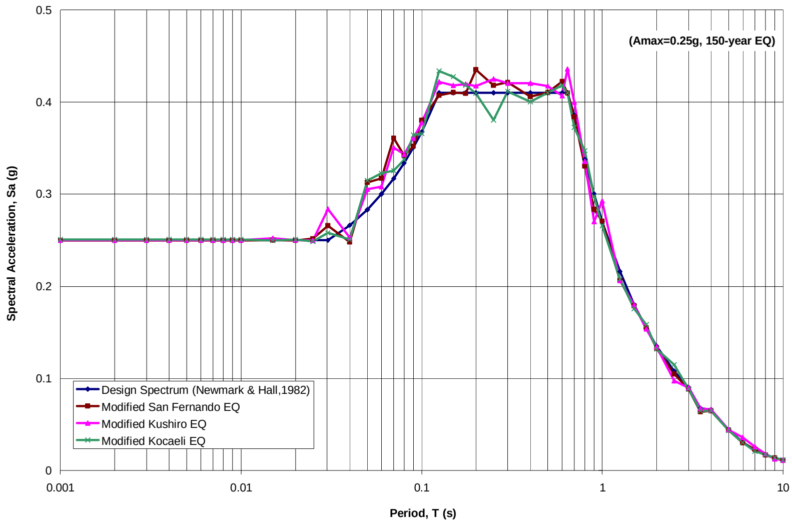

Figure 2.4. Acceleration Response Spectra of the Ground Motions (OBE)

2.4. Damping and Dynamic Pore Pressure Generation

The hysteretic damping option, that considers the increase of damping and decrease of the shear modulus with increasing shear strain, is utilized in the analyses. The hysteretic damping is only active within the linear elastic region. As the yield is reached, the hysteretic damping is switched off and the damping is essentially controlled by the plastic strains based on the constitutive model. The reader is referred to Itasca (2008) for a more detailed information on the damping.

For all of the dam zones, coupled stress-pore pressure relationship has been taken into account. The generation and dissipation of the pore water pressures are considered by the seepage calculations based on the given permeability's, concurrently during the earthquake.

The pore water pressures are related to the pore volume change resulted by the mechanical loading in FLAC program. A semi-coupled liquefaction approach, Finn-Byrne model has been utilized for the hydraulic fill, appraised as potentially liquefiable. In the subject model, the reduction in volumetric strain, , $\Delta\varepsilon_{vd}$, related to the engineering shear strain, γ, is obtained by the below formula,

$$\frac{\Delta e_{vd}}{g} = C_1 \exp(-C_2 \frac{\Delta e_{vd}}{g}) \tag{2.1}$$

where C_1 and C_2 are the model coefficients. C_2 is given as $0.4 / C_1$, and C_1 is suggested as given below based on the relative density of the material.

$$C_1 = 7600(D_R)^{-2.5} \tag{2.2}$$

The above formulas allow the calculation of the excess pore water pressures induced by the cyclic shear strains, and the related decrease in the effective stresses and accordingly the shear strength of the material, using only the relative density of the material. The relative density of the hydraulic fill has not been able to be determined precisely with the conducted in-situ and laboratory tests, so two different relative densities of 40% and 50% are considered in the evaluation. The reader is referred to Itasca (2008) for more detailed information about the dynamic pore water pressures.

3. ANALYSES RESULTS

3.1. Initial Conditions

Initial stress distribution has been obtained before the earthquake loading in the analyses where the dam zones are modelled as elastic-plastic materials having strength relied on the effective stress. The phreatic line has been calculated considering the levels of 347.5 and 327.5 m for upstream and downstream respectively, where the crest level is 351.5 m. Attained pore water pressures are combined with the static stress analysis results in order to determine the effective stresses prior to seismic shaking. Initial pore water pressures are shown in Figure 3.1.

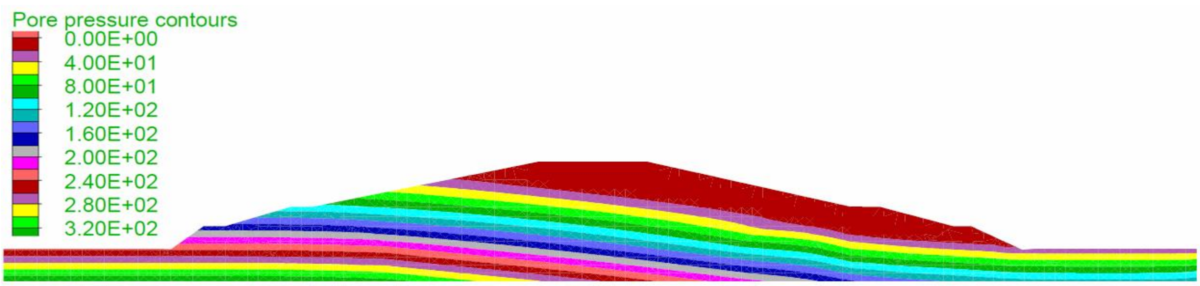

Figure 3.1. Initial Pore Water Pressure Distribution

3.2. OBE Loading

According to the results obtained from the analyses carried out using three different earthquake records for OBE loading, permanent deformations have occurred especially in the middle part of the dam composed of hydraulic fill but the stability of the dam has been retained at the end of the earthquake. Since it is not possible to present all of the results within the scope of this paper, the critical results of them all, obtained for Kushiro ground motion and Dr = 50% relative density, are presented as example. The time variation of crest settlement during the earthquake given in Figure 3.4 indicate the displacements have been resulted by reaching of strength limits during earthquake, but the increase in the displacements have been ended and the system reaches to a stable condition at the end of earthquake motion. The crest settlement has been calculated as between 1.3 – 1.6 m and between 1.1 – 1.25 m for Dr = 40% and Dr = 50% relative densities respectively. Here, the same strength parameters have been used for the two cases of different relative density; it is observed that the relative density of the embankment have led to different settlements though it only affects the dynamic pore water pressures. This indicates the effect of using a liquefaction model for the hydraulic fill.

It can be seen from the permanent deformation vectors shown in Figure 3.2 that during the cyclic motion in the upstream and downstream slopes, plastic deformations take place in both directions and the settlement of the dam is mainly induced by these outward displacements. Figure 3.3 that the shear strains at the end of the earthquake are shown indicate that the yielding at different parts and at various instants of the earthquake cause a somewhat failure mechanism generated to both directions.

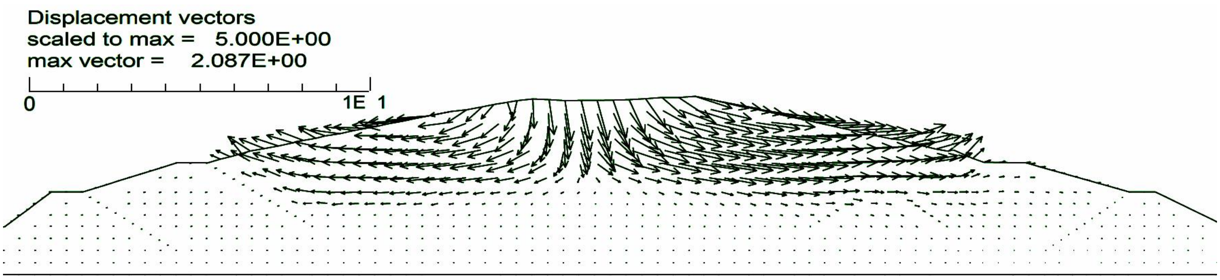

Figure 3.2. Permanent Deformation Vectors (OBE)

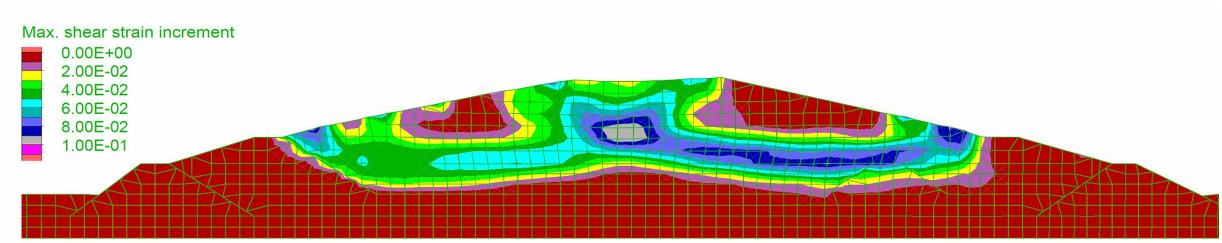

Figure 3.3. Plastic Shear Strains After Earthquake (OBE)

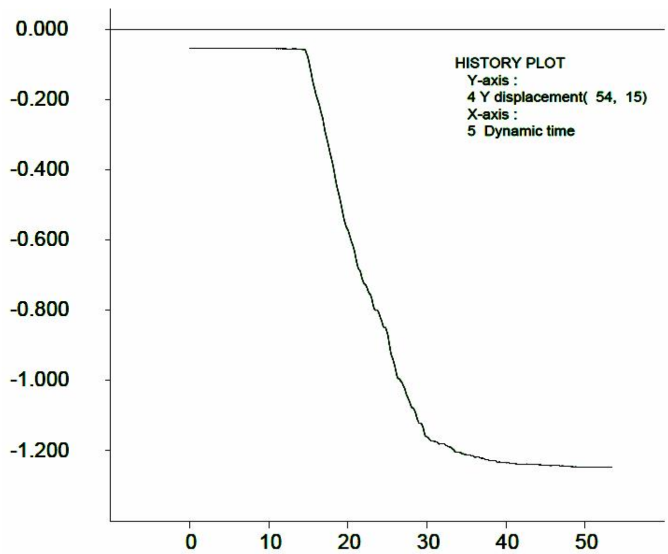

Figure 3.4. Settlement of Crest vs. Time (OBE)

For an earthquake of this level, relatively stronger buttresses have provided the necessary support to the hydraulic fill and prevented the displacements to get larger. The primary reason of the crest settlement and permanent displacements is the temporarily yielding in the weak hydraulic fill zone with the effect of the dynamic pore water pressures.

The permanent settlements are about 1/3 of the freeboard whereas this level of settlement (0 – 5 ft) is considered as acceptable by DSOD (California Department of Water Resources).

3.3. MDE Loading

Earthquakes with return periods of 5 000 and 10 000 years are considered for MDE loading as explained in Section 2.3. For the analysis conducted by using Kushiro ground motion with PGA = 0.67 g corresponding to 5 000-year earthquake and assuming Dr = 40%, the plastic deformations occurred in the downstream of hydraulic fill resulted in excessive distortion of the finite difference grid over the limits and the analysis has stopped. As the displacements and shear strains at this condition has been examined, it has been concluded that the hydraulic fill with a relative density of Dr = 40% loses its stability when subjected to a 5 000-year earthquake. For the same peak ground acceleration and assuming a relative density of Dr = 50% for hydraulic fill, the crest settlement has been obtained between 3.3 – 3.75 m which is very close to the freeboard of the dam as 4 m.

Only the relative density of Dr = 50% has been considered for 10 000-year earthquake loading since the stability has not been retained for Dr = 40% in 5 000-year earthquake loading. The below figures given as example show the results obtained for Kushiro ground motion. It is observed from Figure 3.7 showing the crest settlement during earthquake, the system has reached equilibrium and the stability of the dam has been retained. As indicated by the permanent deformation vectors and plastic shear strains shown in Figures 3.5 and 3.6, the strength of the material has been temporarily reached in the whole dam body including the granular and rock fill buttresses and the plastic deformations have led to large displacements especially to the downstream direction. The plastic shear strains show two different failure mechanisms in the downstream, one within the hydraulic fill and the other passing through the alluvium and under the buttresses.

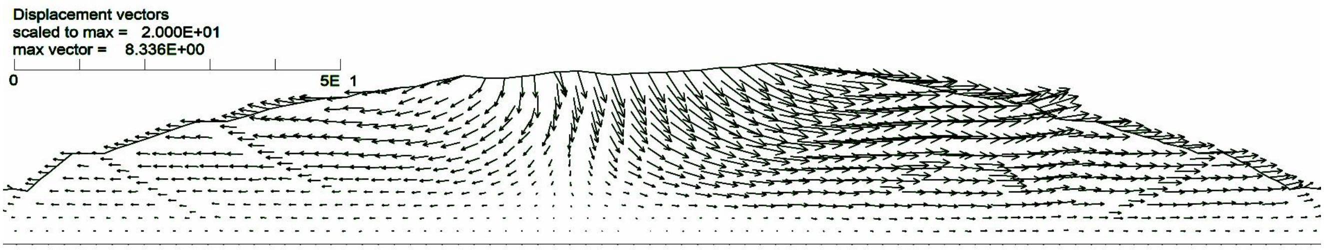

Figure 3.5. Permanent Deformation Vectors (MDE)

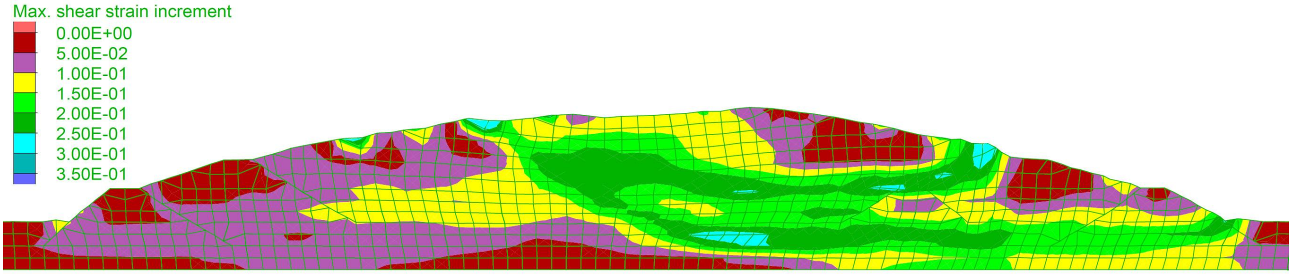

Figure 3.6. Plastic Shear Strains After Earthquake (MDE)

The crest settlement of 3.5 m obtained for Kocaeli ground motion is very close to the freeboard. In the analyses conducted for San Fernando and Kushiro ground motions, the crest settlement is calculated over the freeboard as 4.2 m and 4.6 m, indicating the overtopping and accordingly the failure of the dam failure under operation conditions in case of a 10 000-year earthquake.

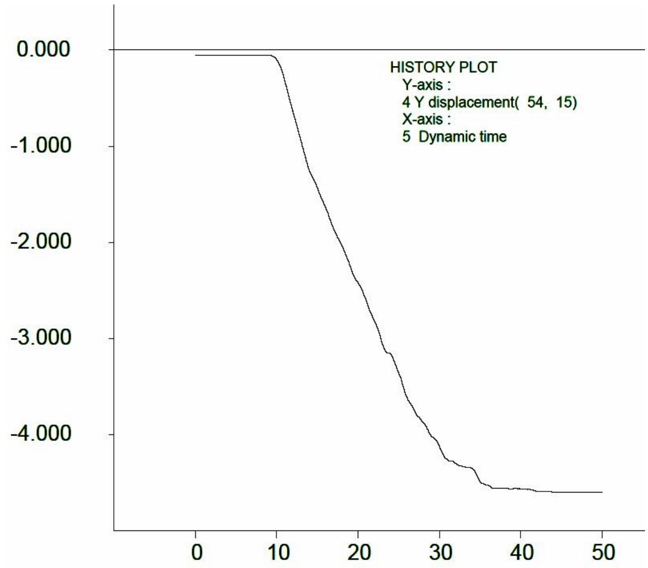

Figure 3.7. Settlement of Crest vs. Time (MDE)

Results presented above show that the dam is not safe for MDE level, therefore improvement and remediation alternatives have been suggested for the upstream and downstream regions of the hydraulic fill.

4. CONCLUSIONS

The seismic stability and permanent displacements of Kairakkum Dam of hydraulic fill type have been evaluated by conducting time history analyses where the nonlinear stress-strain properties of materials, coupled stress-pore pressure relationship and the dynamically induced pore pressures related to liquefaction in the hydraulic fill are taken into account.

Obtained results have shown that the freeboard is sufficient for an earthquake of 150 year return period corresponding to operating basis earthquake (OBE). For the maximum design earthquake (MDE) considered with 5 000 and 10 000 year return periods, obtained permanent deformations have exceeded the acceptable limits and it has been observed that the dam is insufficient in terms of earthquake safety .

In this regard, it has found reasonable to make improvements and remedial works in the dam body and proposal designs have been developed. In the downstream slope, stone columns are suggested with the depth, diameter and spacing's to be determined by detailed calculations. Stone columns will increase the shear strength of the hydraulic fill by means of densifying, and will also function to provide drainage as well.

For the hydraulic fill in the upstream side, alternatives of performing similar stone columns at the whole slope, and to backfill with granular material after excavation up to a certain elevation are presented as suggestions for the detailed remedial works to be studied in the future.

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