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STATE-OF-THE-ART REVIEW

Review on Dynamic Behaviour of Earth Dam and Embankment During an Earthquake

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Abstract The earth dam analysis under the strong seismic load like a destructive earthquake is one of the major topics with respect to the dynamical assessment. Damage control and the structural behaviour during an earthquake is a very important issue for an earthen dam. In this study, a comprehensive review is presented based on literature for dynamic analysis of earth dams. In this context, some significant factors are discussed such as plane stress, plane strain, data monitoring, application of finite-element method or finite-difference method, reinforcement, free vibration analysis, seismic cracks, liquefaction on dams, utilization of shaking table and centrifuge tests based on the small-scale physical modelling in order to validate any numerical analysis. To explain these parameters, case studies are discussed. It is observed that the earth dam structures had the integrated response to increasing the acceleration or displacement at the crest. Consequently, the interaction between the dam and reservoir also the foundation was a very effective factor to establish the nonlinear behaviour. It seems that the reinforced techniques are an essential approach to improve the structural response during an earthquake.

Keywords Earth dam · Earthquake · Numerical analysis · Seismic cracks · Acceleration · Displacement

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1 Introduction

Dams and embankments are considered an important infrastructure for the development of any nation. It serves to irrigations, controlling floods, and producing electricity along with storage of waters and transportation. It is considered as one of the oldest civil engineering structures that serve mankind for their developments. Earthen dams and embankments are adopted, due to ease of construction and locally available chief materials and widely used around the world. However, the failure of dams causes devastating effects in the past and caused losses of lives and properties. A dam is considered stable if the resultant forces or moments do not cause of dam movement (Seed et al. 1969). Dam failure occurs due to the specific performance like crack, internal erosion, piping, toppling, and failure of foundation, spillways, and slopes under static and dynamic loading conditions. Based on literature, overtopping and internal erosion are the most frequent reasons for the earthen dam failures (Cristofano 1973; Wahl 1998; Jandora and Ríha 2008). The failure of the Augusta dam was first reported in the literature, due to the famous earthquake Charleston (South Carolina) in 1886 (Raja 2014). An inclusive summary of the recognized failure or damage in 58 earth dams due to the earthquake was prepared by Ambraseys (1960) revised by Basudhar et al. (2010) and Raja (2014). Due to the earthquake, dam failure always attracts researchers to study about devastating nature. The catastrophic failure of the Lower San Fernando dam during the earthquake in 1971 attracted the attention of regulatory agencies and design professionals to apply more recent advanced dynamic analysis tools to evaluate the seismic stability of the dam (Seed et al. 1971). The initial studies on the dam under the earthquake force started with the pseudo-static method (Terzaghi1950; Sarma 1979). This method was applicable to observe the factor of safety by calculating the minimum seismic loads to make the instability slope of the dam. Furthermore, dynamic analysis also established using the sliding block methods (Newmark 1965; Ambraseys and Sarma 1967; Ambraseys and Menu 1988). It was focused on the calculating the seismically induced permanent deformations instead of a factor of safety. In parallel, some significant advances in dynamic analysis also have been achieved by shear beam technique. This method was based on the considering earthquake vibrations in different directions such as transverse (Ambraseys 1960; Gazetas 1982), vertical (Gazetas 1981a) and longitudinal (Abdel-Ghaffar and Koh 1981; Gazetas 1981b). Apart from the earthquake, ground vibration can also induce by blasting of explosion, which can develop dynamic stresses in the soil and rock mass (Verna and Singh 2010). The shear beam method extended for dynamic analysis of the dam in terms of the effects of canyon geometry (Dakoulasand Gazetas 1987) and heterogeneous dam materials (Abdel-Ghaffar and Koh 1981; Gazetas 1982; Dakoulas and Gazetas 1985). The most recent and accurate methods for dynamic analysis of the dam are numerical methods based on the finite element method, finite difference method, and finite volume methods. Numerical methods like the finite element method (FEM), the finite difference method (FDM), the finite volume method (FVM), the boundary element method (BEM), and the distinct element method (DEM) are becoming increasingly popular for slope stability analysis of the dam and embankment in conditions where the failure mechanism is not controlled completely by discrete geological structures (Monjezi and Singh 2000; Singh and Verma 2007; Singh et al. 2010; Verma and Singh 2010a, b; Faizi et al. 2013). Body cracks are the major cause of damage in the earthen dam. However, some significant phenomena like overflow, piping, and failure mechanism can be observed step by step after the crack development in the dam body. Besides, the construction of earth dams is a gradual procedure in terms of the layers compacted. As known, static settlement in the dam can be completed with the consolidation time. However, the initial step of dynamic load depends on static displacement. In terms of data monitoring based on dam instrumentation, the comprehensive review in this area indicates a few case studies in this field. Due to the development technology, the numerical method is significantly used. As a matter of fact, numerical analysis is one of the best approaches to evaluate the structural behaviour during an earthquake. Also, two types of numerical analysis have been developed like plane stress (3D) and plane strain (2D). It is worth to mention that, both methods depend on valley shape exclusively. Significantly, the lateral strain value is zero in the case of two-dimensional analysis. In addition, on some occasions for physical modelling test, shaking table device and centrifuge are used to estimate the structural behaviour during the seismic load (Gordan et al. 2016a). In terms of the dam failure, it can be controlled by factor of safety and resistance of the shear strength during the construction in each layer, but should control in the initial phase of the design. In terms of reinforcement, some techniques have been used to improve the structural behaviour during an earthquake. It is also important to mention that among all available techniques for solving slope and dam problems, artificial intelligence has been successfully used and suggested in this area and geotechnical engineering (Singh et al. 2004, 2005; Verma and Singh 2013; Gordan et al. 2016b; Mahdiyar et al. 2017; Koopialipoor et al. 2019; Zhou et al.

2015, 2016a, b, 2019; Cai et al. 2020; Tang et al. 2020).

The primary purpose of this study is to review the dynamic analysis for earth dams. Therefore, some branches of dynamic assessment have been reviewed including plane strain (2D) or plane stress (3D) methods, data monitoring by tools instrumentation, reinforcement techniques, the dominant frequency concerning free vibration, case study and liquefaction, and small scale modelling using shaking table device.

2 Methods to Analyse Earth Dams Under Dynamic Condition

2.1 Plane Stress and Plane Strain Method

The in-depth review suggested that numerical analysis is particularly accomplished by two methods such as plane stress and plane strain. It is very important to select appropriate methods to analysis dam to observe the distributed lateral displacement. Some noteworthy studies reviewed for explanation. A comparison of 2D and 3D dynamic analysis in earth dams has been carried out by Mejia and Seed (1983). It was observed that the ratio between 3D and 2D analysis results were always more than one. The dominant frequency of the dam decreased while the ratio between lengths to heights of the dam increased. Furthermore, the natural frequency ratio between 3D and 2D significantly affected by the valley configuration. This ratio is higher for triangular shaped valley compared to the rectangular canyons. Besides, the stress distribution in the maximum cross-section has been compared in both methods during the strong earthquake. The greatest stress in the Oroville dam was observed using 3D analysis. Furthermore, the results from the maximum cross-section indicated that 2D analysis was more stressed than 3D over one-third of the dam base. Figures 1 and 2 show the 3D and 2D finite-element method (FEM) of two sections, for a length quarter and maximum section of the Oroville dam.

Table 1 presents the ratio between 2D and 3D for both sections referred to reanalysis of Oroville earthquake. It can be seen that the ratio of τ(max) xy2D/τ (max) xy 3D was 4.00 at the base of the model for the maximum section. The ratio at maximum and quarter sections for different crest length and dam height ratios also compared for two earthquake events. Moreover, the highest predominant frequency of response in the plane strain model was observed for lower plane stress (3D). In all cases, the effects of the nature of input motion were consequently smaller than canyon geometry.

The three-dimensional seismic analysis of the La Villita dam has been evaluated using 3D approach (Elgamal 1992). Two seismic records and bedrock observation have been practically analysed using numerical analysis. Figures 3 and 4 show the three-dimensional (3D) comparative study indicated the good agreement between observed and numerical method analyses results. Maximum acceleration was located in the middle of the crest for UD (upstream to downstream) while it was very close to the abutment for vertical and longitudinal directions. It has also excellent agreement with the results of sliding-block method.

The elastic seismic response of earth dams by canyon interaction was studied (Papalou and Bielak 2001) which indicated that numerical analysis is a suitable approach, because the classification of results at different points are possible in comprehensive model. However, this research has been particularly carried by a linear method and emphasized that the increase of shear wave velocity in the rock led to an increase in shear strain and displacement. It also focused on the linear method for interaction effect about canyon and dam. In brief, the peak ground acceleration recorded exactly in the middle of the crest. Figure 5 shows five key points on the crest, in order to compare results. Figure 5 also shows the distributed acceleration in the main points and maximum acceleration in the middle of the crest (See-C). Figure 6 shows the acceleration as distributed in different points like A, B, C, D, and E. It is worth noting that the interaction between dam and canyon is the significant factor.

Stability of rock-fill dam considering the influence of the geometry has been evaluated (Yu et al. 2005). As observed, the dimension ratio between length and height (η) emphasised the adequate influence in the 3D analysis. It is worth noting that, factor of safety can be reduced when this ratio increased. The relationship between both ratios has been considered based on Eq. 1. In brief, the plane strain (2D) method exhibited little effect on both ratios. The relations between length to height and safety factor is in the below equation.

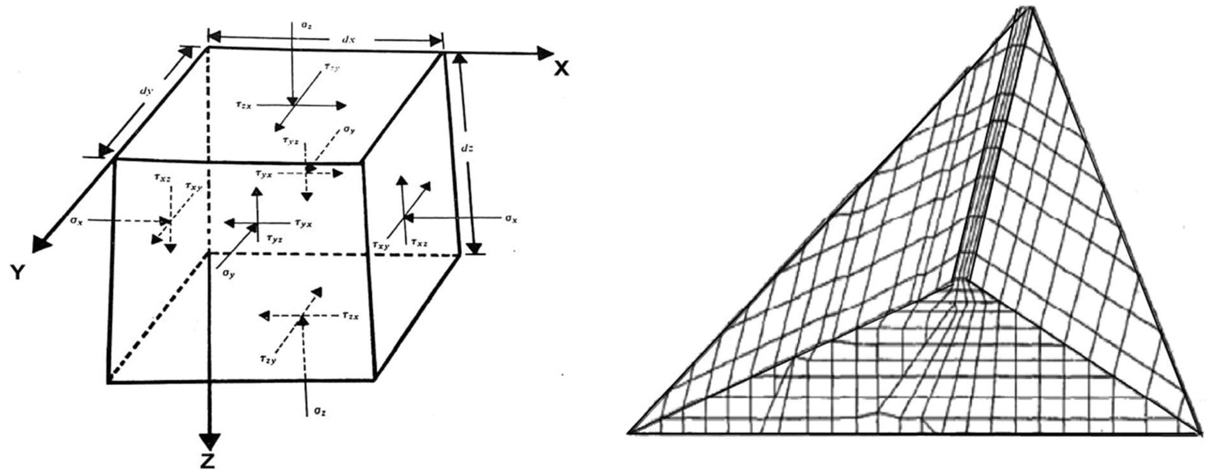

Fig. 1 3D finite element mesh with 591 elements and 705 nodal points (Mejia and Seed 1983)

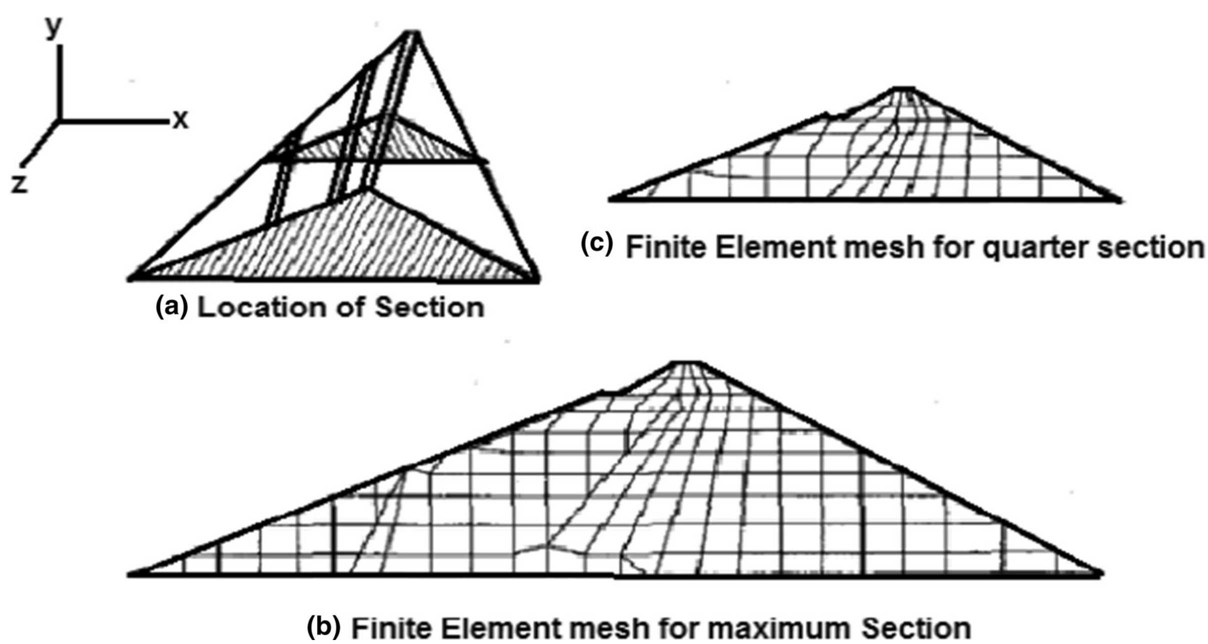

Fig. 2 2D finite element mesh for a quarter and a maximum section from the Oroville Dam (Mejia and Seed 1983)

Table 1 Comparison result between $\tau_{xy}2D/\tau_{xy}3D$ in the base of the dam

Section	Maximum section $\tau_{xy}2D/\tau_{xy}3D$	Quarter section $\tau_{xy}2D/\tau_{xy}3D$
L/H = 2 Reanalysis earthquake	4.0	1.4
L/H = 2 Oroville Earthquake	4.0	1.00
L/H = 7 Reanalysis earthquake	1.2	1.1
L/H = 7 Oroville Earthquake	1.2	1.00

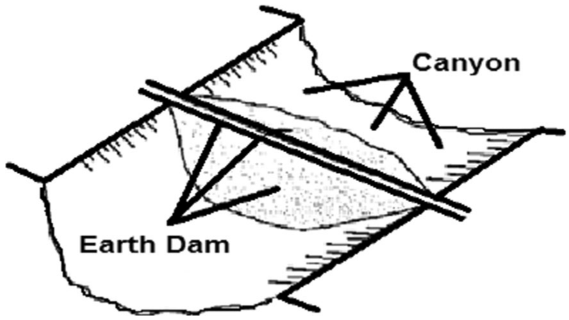

Fig. 3 3D condition from Villita Dam (Elgamal 1992)

very important while using 3D analysis. Besides, the gradient of downstream and upstream and dam height was a greater impact on safety factors in the 3D situation. Hence, it can be concluded that this study was successful for comparison of safety factors in between 2D and 3D analyses. It is also revealed that the economic design depends significantly on the shape of the valley. In terms of accurate design, 3D analysis and 2D analysis have been recommended for narrow and wide valleys respectively

The importance of the above studies indicated that there are two types of numerical methods applied in engineering programs like the Finite-Element Method (FEM) and Finite-Difference Method (FDM). These two methods are required to evaluate the dynamic analysis considering plane strain in 2D and plane stress in 3D. Using these methods, the specific conditions of the valley shape and structural configuration can be analyed. In fact, the dam behaviour

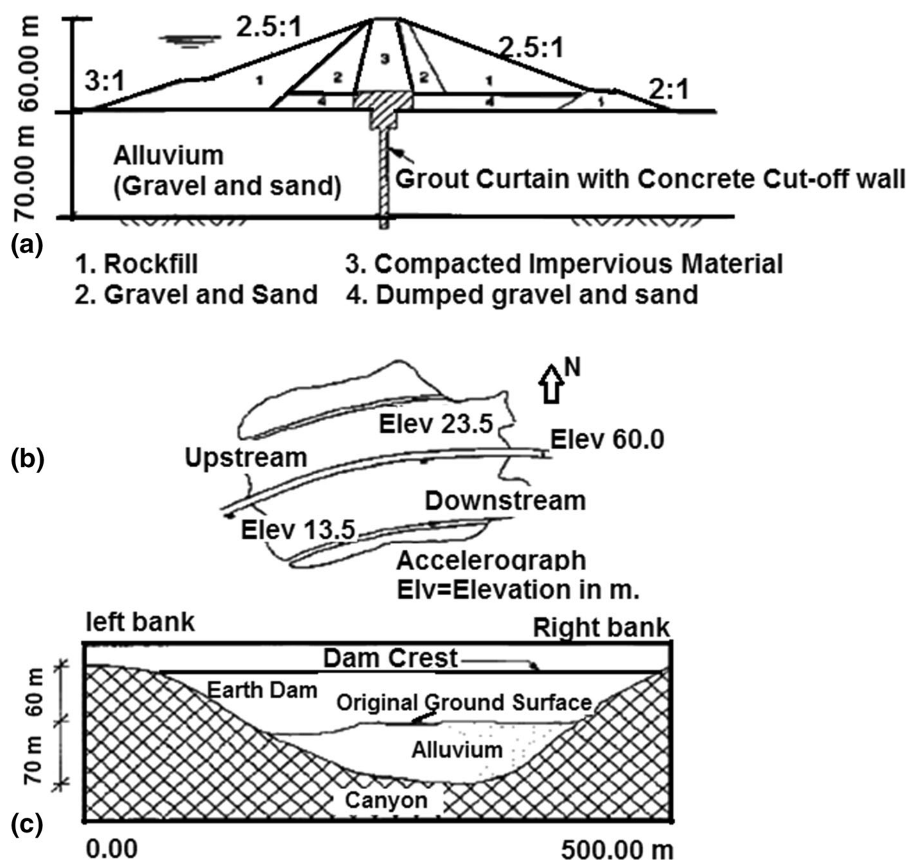

Fig. 4 Lavillita Dam: **a** Maximum cross section, **b** Plan view, **c** Geological profile (Elgamal 1992)

$$\lambda = \frac{(F_{s_{3-}}F_{S_2})}{F_{S_2}} \quad \eta = \frac{L}{H} \tag{1}$$

Figures 7 and 8 show the dam perspective regarding configuration and comparison of the relationship between safety factor and η respectively. The major effects directly accessed using 3D analysis method. It was found that the shape and size of the canyon was

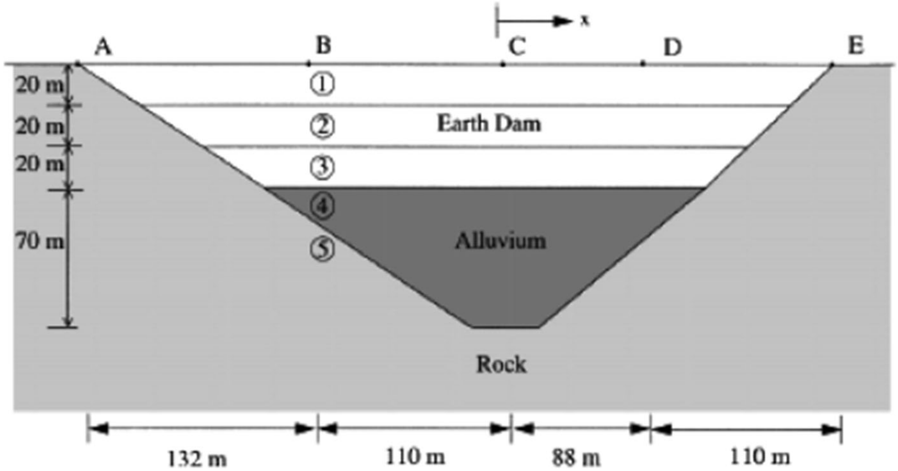

Fig. 5 Main points at the crest of the La Villita dam with geometry (Papalou and Bielak 2001)

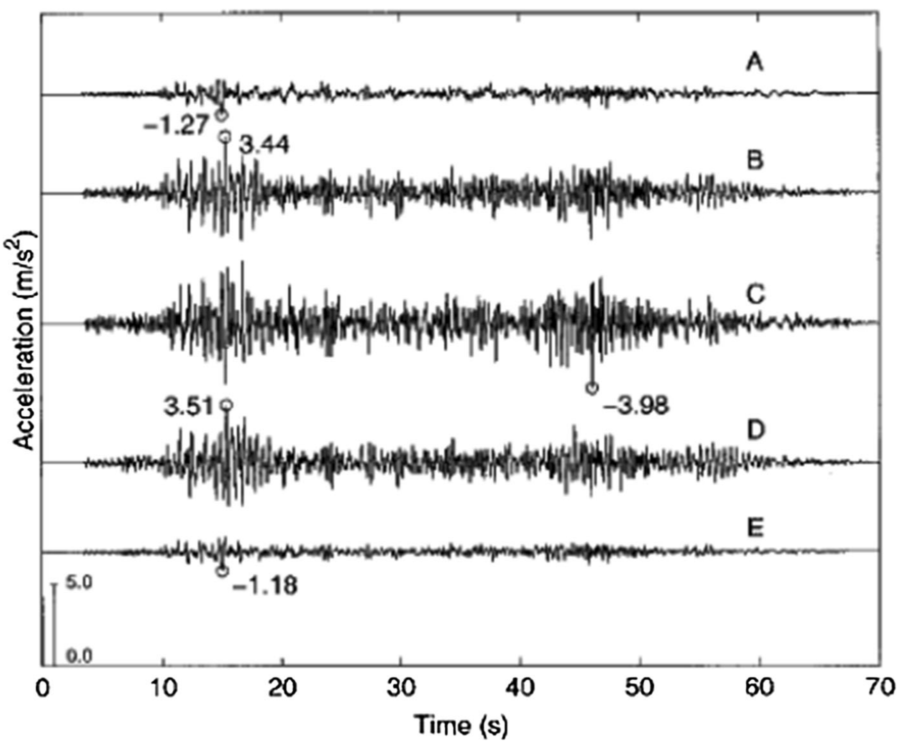

Fig. 6 Acceleration distribution at the crest of the La Villita dam (Papalou and Bielak 2001)

during an earthquake was observed using linear and nonlinear analysis using 2D and 3D analysis to compare the results. The factor of safety between 2D and 3D analysis indicated that the suitable performance of the earth dam is influenced by the U or V valley shapes, respectively. In plane strain (2D), upstream to the downstream direction (UD) was at the best situation. Also, maximum acceleration and displacement was at the middle of the crest for UD.

2.2 Data Monitoring

This section focused on a review data monitoring in earth dams. In this category, the analysis of earth dam

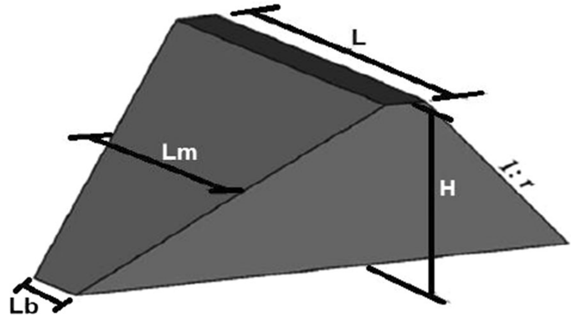

Fig. 7 Parametric modelling of dams (Yu et al. 2005)

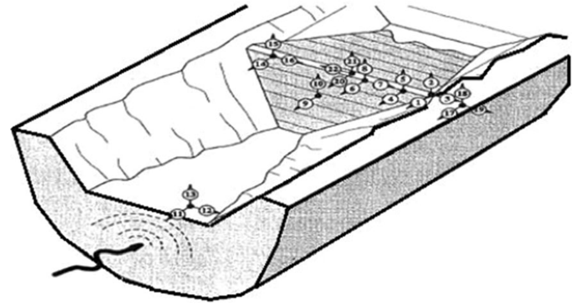

Fig. 9 Arrangement of sensors to record data on long valley dam (Zeghal and Ghaffar 1992)

behaviuor using strong-motion was evaluated by (Zeghal and Ghaffar 1992). The scope of this study was concentrated on the nonlinear behavior of the soil in the earth dam for a long valley in California under six different earthquakes in 1980. According to data monitoring, constitutive hysteric models exhibited an insufficient approach for dissipation mechanisms. Figure 9 shows the sensor distribution in the long valley dam as a case study, in which the model response was evaluated in three directions.

The study of distributed settlement of the Morons dam in Greece included the results of numerical modelling and geodetic monitoring have been recently reviewed (Gikas and Sakellariou 2008). The optimum response was located upstream to downstream in compare to the longitudinal and vertical directions. Hence, the lack of sensors installation at the boundary was dramatically exposed. The identification system was used in this study to detect the structural behaviour, with or without retrofitting. This study was conducted with applicable seismic records with respect to sensor installation. It is worth noting that this method is a useful approach in obtaining valuable information about earthen dam behaviour during a destructive earthquake.

Figures 10 and 11 show the distributed sensor and vertical displacement in different stations (See S20 and S28) as located at the middle and the tail-end of the crest. As a result, the vertical displacement in the middle of the crest was sensibly maximized based on a comparison. It was important to note that, the good agreement obtained between field measurements and numerical analysis. The study indicated that monitoring data is a suitable approach to record earth dam behaviour during the dynamic loading. A few case studies are recorded in this field. Based on observation, dam behaviour has been subjected to the nonlinear performance. In terms of dam response, an excellent response is observed by the upstream to the downstream (UD). However, maximum static

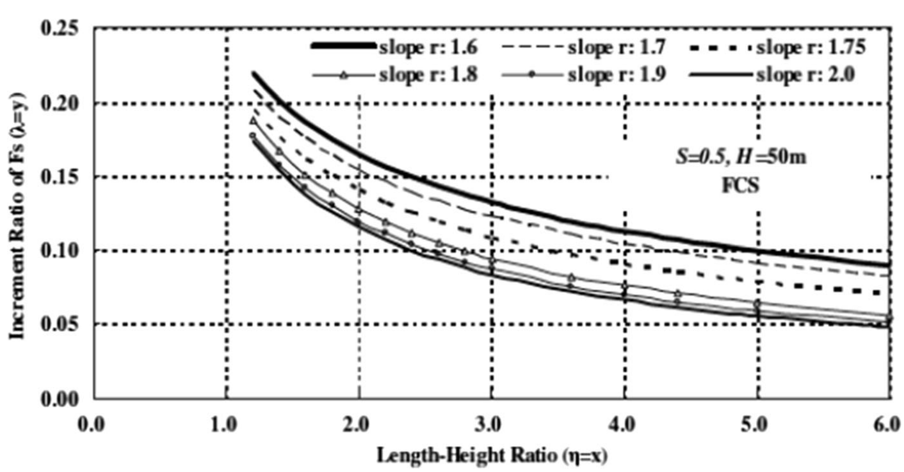

Fig. 8 Relationship between Fs and η with a different gradient of slopes (Yu et al. 2005).

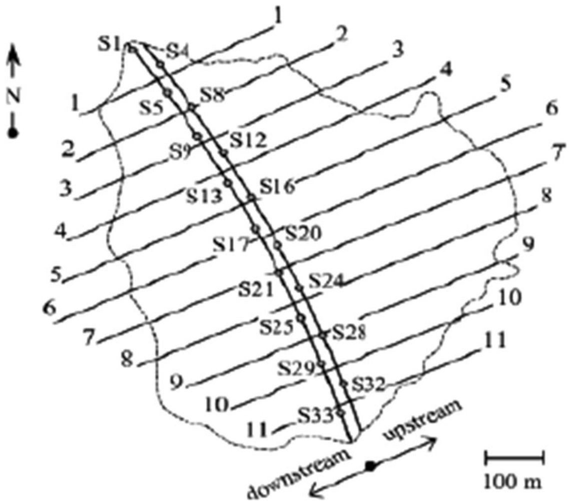

Fig. 10 Arrangement of sensors to record data of the dam at the crest (Gikas and Sakellariou 2008)

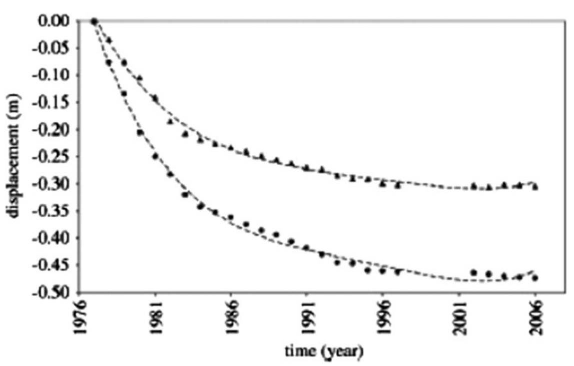

Fig. 11 Distribution of vertical displacement during the time for S20 (middle) and S28 (toward the tail-end) at the crest (Gikas and Sakellariou 2008)

movement was at the central cross-section. Good agreement was found between field measurements and numerical analysis.

2.3 Finite Element Method

Commonly used methods to examine the dynamic response of the earthen dams divided into three types: simplified, empirical, and numerical methods. Numerical methods firstly used for the dynamic analysis of an embankment dam, Clough et al. (1966), followed by main developments are done by Ghaboussi (1967) and Li et al. (1992). The two most important numerical methods are finite element and finite difference methods briefly discussed below.

Any complex structure such as dams need the stress analysis to examine stress and displacement within the structure at equilibrium under external difficult to determine using traditional methods, Raja (2014). A new modern method is introduced naming finite element method (FEM) is computer-oriented, more effective in the idealization of any complex structure of the arbitrary shape, Chopra (1966) and Chopra et al. (1969). This method is commonly used in seismic analysis of dams (Raja 2014). The basic idea of this method is to find the solution of any arbitrary complex structure by dividing them into discrete elements or component whose nature can easily be understood, such elements form mesh and interlinked with joints called nodes having a different degree of freedom (Raja 2014). FEM can be expressed by integral or differential equations and provide a numerical solution to any practical problem. It provides a fairly accurate solution because the finite element is allowed to have a modest spatial variation, commonly expressed by polynomial expressions (x^2, xy, and y^2) whereas real variation is more complex. The governing equation of FEM can be, idealized using a mass-spring-damper system, expressed as:

$$[M]\{\ddot{r}\} + [C]\{\dot{r}\} + [K]\{r\} = \{R(t)\} \quad (2)$$

where [M] is the mass matrix, [K] is the stiffness matrix, [C] is the damping matrix, {R(t)} is the time-dependent load matrix. {r}, $\{\dot{r}\}$ and $\{\ddot{r}\}$ are the nodal displacement, velocity and acceleration respectively. The FEM can be classified as (1) flexibility or force method in which internal forces considered to be unknown are used to find governing equations by applying initial equilibrium conditions and later on using compatibility conditions to obtained other equations, and (2) stiffness or displacement methods in which nodal displacement is considered unknown to find governing equations by using equilibrium conditions and force-displacement relations (Zienkiewicz 1977; Cook 2002).

The stability analysis of earthen dams exposed to earthquake loads can be analysed by different methods; Gazetas (1982) elaborated the historical advances of theoretic techniques for calculating the seismic response of earthen dams subjected to the ground shaking and defined their key features, advantages, and limitations. After the San Fernando dam failure during an earthquake in 1971, the most important developments reached in the understanding of seismic forces on the dams USCOLD (1992). Dynamic behavior of the earthen dam using elastic-plastic soil model examined by finite element method (Zeghal and Abdel-Ghaffar et al. 1992). The dynamic response of the Marana Capacciotti earthen dam was compared using two numerical methods (Cascone and Rampello 2003).

2.4 Finite Deference Method

The finite difference method (FDM) is a discretized approximate method that provides a numerical solution of partial differential equations through discretizing the continuous physical domain into a distinct finite-difference network, approximating the distinct partial derivatives by algebraic finite difference approximations to solve the resulting equations for the dependent variable (Hoffman 2001). This method approximates complex linear/non-linear ordinary and partial differential equations into a system of linear/non-linear equations in matrix form which suits the modern computer and hence widely used in numerical analysis (Grossmann et al. 2007). In other words, FDM is used to discretize the domain and replace the differential equations with simple definite increments. When more than one variable exists then the method is applied separately to each variable (Cuminato and Meneguete 1999).

The finite-difference approximations were developed using Taylor's series for the dependent variable at several neighbouring grid points and assuming base grid point (i, j), then merging these Taylor series to solve for the anticipated partial derivatives (Kermani and Barani 2012). The 2-D governing equation obeying Darcy's law for FDM describing steady-state seepage in a porous medium can be written as follows-

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial z}\left(K_z\frac{\partial h}{\partial z}\right)=0 \tag{3}$$

where $K_x$and $K_z$are hydraulic conductivities in the horizontal and vertical directions and h is the available water head. For homogeneous soil, Eq. (3) can be modified as Eq. (3) and replacing derivatives in 2-D seepage equation, Eq. (4) can be rewritten using the second-order centred difference approximations at grid point (i,j) to obtained Eq. (4) as follows:

$$K_x\frac{\partial^2 h}{\partial x^2}+K_z\frac{\partial^2 h}{\partial z^2}=0 \tag{4}$$

$$K_{x_{i,j}}\left(\frac{h_{i+1,j}-2h_{i,j}+h_{i-1,j}}{\Delta x^2}\right)+K_{z_{i,j}}\left(\frac{h_{i+1,j}-2h_{i,j}+h_{i-1,j}}{\Delta z^2}\right) \tag{5}$$

Equation 5 can be used to obtain the five-point approximation of Laplace equation as follows:

$$h_{i+1,j}+\beta^2h_{i,j+1}+h_{i-1,j}+\beta^2h_{i,j-1}-2\left(1+\beta^2\right)h_{i,j}=0 \tag{6}$$

where $\beta=\frac{\Delta x}{\Delta z}\sqrt{\frac{K_x}{K_z}}$. Solving Eq. 5 for $h_{i,j}$ gives:

$$h_{i,j}=\frac{h_{i+1,j}+\beta^2h_{i,j+1}+h_{i-1,j}+\beta^2h_{i,j-1}}{2\left(1+\beta^2\right)}. \tag{7}$$

2.5 Case Studies and Liquefaction

In terms of technological development in the context of applying engineering programs for numerical purposes, there are many case studies in this domain have been explained. An evaluation of the response of the SURGU dam during the May 5, 1986 earthquake was studied (Özkan et al. 1996). In this study, the primary purpose is to focus on the evaluated stability and deformation during the earthquake. As a result, an excellent agreement obtained with measurements while the dam coupled with the lowest magnitude of the earthquake.

Figures 12 and 13 show key nodes and acceleration as distributed in the dam during ten earthquakes respectively. The vibration period exhibited higher stirring acceleration at the crest. The amplified acceleration at the dam crest was two to three times greater than the acceleration at the bedrock. Figure 13 shows the maximum acceleration along the vertical axis of the dam body for all ten earthquakes.

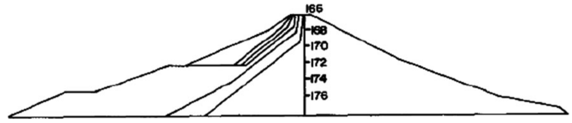

Fig. 12 Positions of node number along with the centreline of the dam (Özkan et al. 1996)

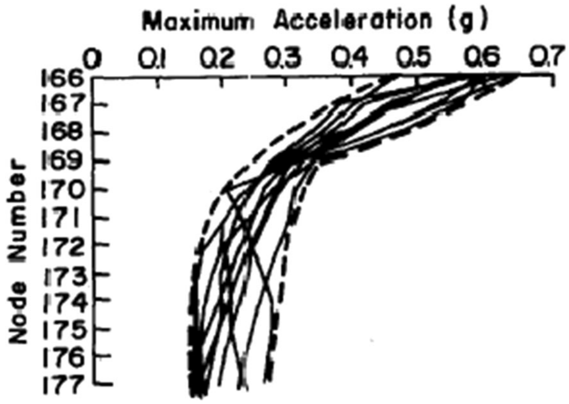

Fig. 13 Maximum acceleration along the vertical axis of the dam body (Özkan et al. 1996)

Earthquake-induced deformation of earthen dams also has been investigated (Siyahi and Arslan 2008). Thus, the dynamic analysis of the Alibey dam depends on the geological characteristics and geotechnical information of the site. Moreover, given the failure modes such as liquefaction, landslide, piping, and type of cracks, it's worth noting that both geology and design engineers should work together in one engineering group. The finite-element method and experimental results from this study described that the dam has not experienced excessive movement under excitation such as strong earthquakes. In addition, the maximum displacement of the horizontal movement was 400 mm at the middle of the dam crest, and it occurred around the 20th second of the earthquake.

Parish and Abadi in 2009 has been also investigated the dynamic behaviour of earth dams for variation of earth material stiffness. This study was entirely performed by numerical analysis in order to use FLAC 3D program. The real record from the KOCAELI earthquake with the "equivalent material" method was used. In the case of elastic analysis, the lateral velocity increased with distance from the base of the dam, and the horizontal velocity in this direction was decreased. The dynamic behaviour of earth dams with variation of earth material stiffness has been investigated (Parish and Abadi 2009). This study was entirely performed by numerical analysis in order to use FLAC 3D program. The real record from KOCAELI earthquake with "equivalent material" method was used. In the case of elastic analysis, the lateral velocity increased with distance from the base of the dam, and the horizontal velocity in this direction was decreased.

Figure 14 shows deformation at the end of the earthquake. The velocity was dramatically amplified three times at the crest due to soft soil, as seen in Fig. 15. However, increases of shear modulus in the core led to a change of amplification velocity at the crest with a moderate rate (See Fig. 16). Moreover, the sharp increase of the amplification of the seismic velocity was revealed while the shear modulus in the shell was increased. The increase in the fundamental frequency was consequently observed.

The tailing earth dam was evaluated under seismic conditions (Chakraborty and Choudhury 2009). Comparing results between FLAC 3D and SEEP/W (Geo Slope) indicated the similar behaviour. Furthermore, it was evident that the greatest displacement occurred at the crest. The displacement was nineteen times more than the static state after the earthquake, which was significant. Maximum permanent displacement was at 66.7 cm, but slightly higher than Makdisi-Seed method. During the amplification process, the peak of the acceleration placed three times more input data than on the bedrock.

Figure 17 input acceleration at the bedrock with 0.15 g. This value was 0.50 g at the crest. The minimum factor of safety was calculated using the FLAC 3D program and Seed's method were 0.89 and 1.15 respectively. Therefore, an unsafe situation under the dynamic loading was observed. A numerical study in the arrangement of clay core for seismic performance of the earth dam was evaluated (Berhe et al. 2010). As a result, depending on usage, inclined clay core upstream can reduce excessive deformation of the shell part, as shown in Fig. 18. The destructive earthquake (El CENTRO) with the peak ground acceleration equal to 0.3 g as input data in FLAC 3D was utilized.

Consequently, pore water pressure at the upstream was observed along with the seismic behaviour of the

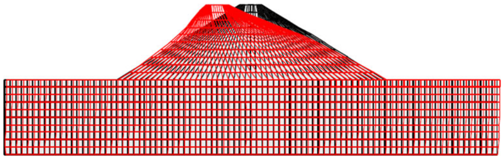

Fig. 14 Dam deformation at the maximum excitation under Kocaeli record (U_{max} = 0.30 m at the crest) (Parish and Abadi 2009)

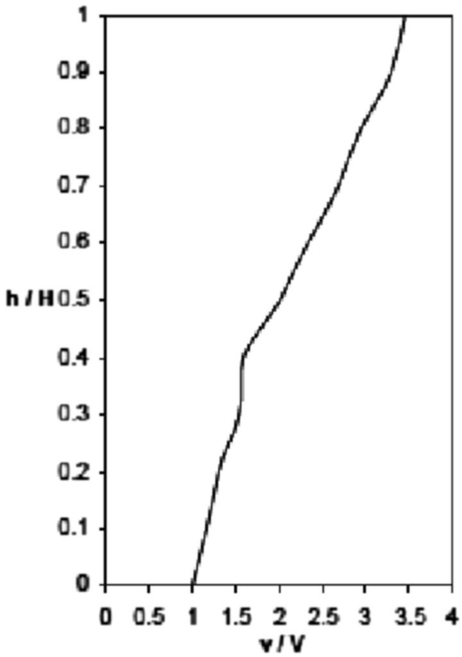

Fig. 15 Velocity amplification in the dam axis (Parish and Abadi 2009)

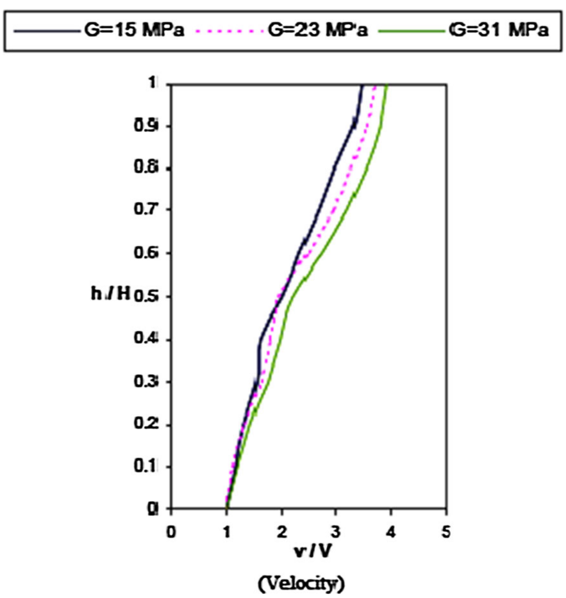

Fig. 16 Influence of the core stiffness on the velocity response under Kocaeli record (Parish and Abadi 2009)

earthen dam, and it was found that the impermeable clay core has oriented towards the upstream face which indicates substantially more stability. Finally, it was observed that the location of core clay strongly

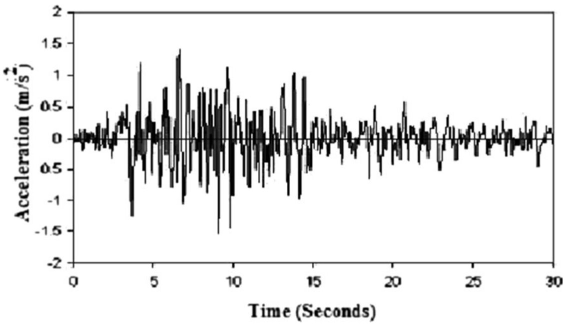

Fig. 17 PGA = 0.15 g at the base (Chakraborty and Choudhury 2009)

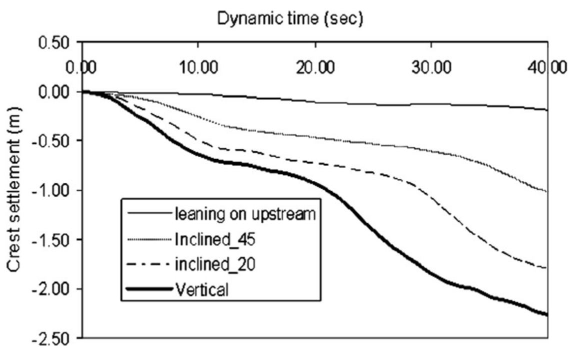

Fig. 18 Crest settlement as a function of time for different core arrangements (Berhe et al. 2010)

affected dam behaviour with respect to the earthquake impact. Furthermore, the fully coupled numerical analysis of a repeated shake-consolidation process of earth embankment on the liquefiable foundation was studied (Xia et al. 2010). In the static analysis, a soil–water coupled scheme (Akai and Tamura 1978) was adopted with the displacement increment of soil Δu and pore water pressure p were the unknown variables to be determined. For the dynamic analysis, a fully coupled two-phase field theory with u–p formulation (Oka et al. 1994) was studied with the total displacement of the soil u and the pore water pressure p were the primary unknowns could be determine using following equations-.

1. Equilibrium equation

$$\rho \ddot{u}_i^s = \frac{\partial \sigma_{ij}}{\partial x_j} + \rho b_i \tag{8}$$

where ρ and u are the soil density and displacement and b_i is the body force respectively.

2. Continuity equation:

$$\rho^f \ddot{\epsilon}_{ii}^s - \frac{\partial^2 p_d}{\partial x_i \partial x_i} - \frac{\gamma_w}{k}\left(\frac{\dot{\epsilon}_{ii}^s}{n} - \frac{1}{K^f}\dot{p}_d\right) = 0 \tag{9}$$

where ρ^f is fluid density, γ_w is unit weight of the fluid, $\dot{p}_d$ is the pore pressure, k is the coefficient of permeability, K^f and n indicate the volumetric compressibility of the fluid and soil porosity respectively.

Based on results after the first dynamic loading, silt layer was dense, with increasing resistance to liquefaction. Figure 19 illustrates crest settlement in both situations such as main shock and aftershock. It was found that the consolidation process created aftershock. Consequently, the rate of excess pore water pressure (EPWPR) after the shock decreased if the deformation was relatively small. Finally, they found that, greater emphasis on the effect of several dynamic analyses in the earthen dam was needed.

Furthermore, the failure mechanism of the Zipingpu Dam studied under seismic load. Several reflections of seismic design also found in the highest rock-fill dam (Kong et al. 2010). Based on this case study, some suggestions have been technically suggested for damage control in CFRD during dynamic load. In general, specific attention was necessary for the high stresses at the top of the dam. Finally, controlling of the whiplash effect was a significant factor for additional stress on the slab.

Fig. 20 Concrete slab dislocation Zpingpu dam during the Wenchuan earthquake (Kong et al. 2010)

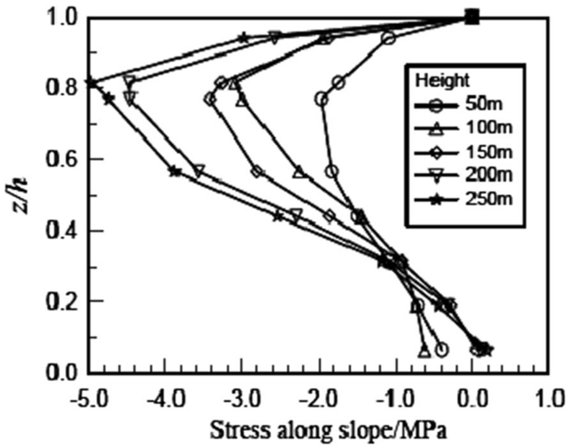

Fig. 21 Distribution of stress along the concrete slab slope (Kong et al. 2010)

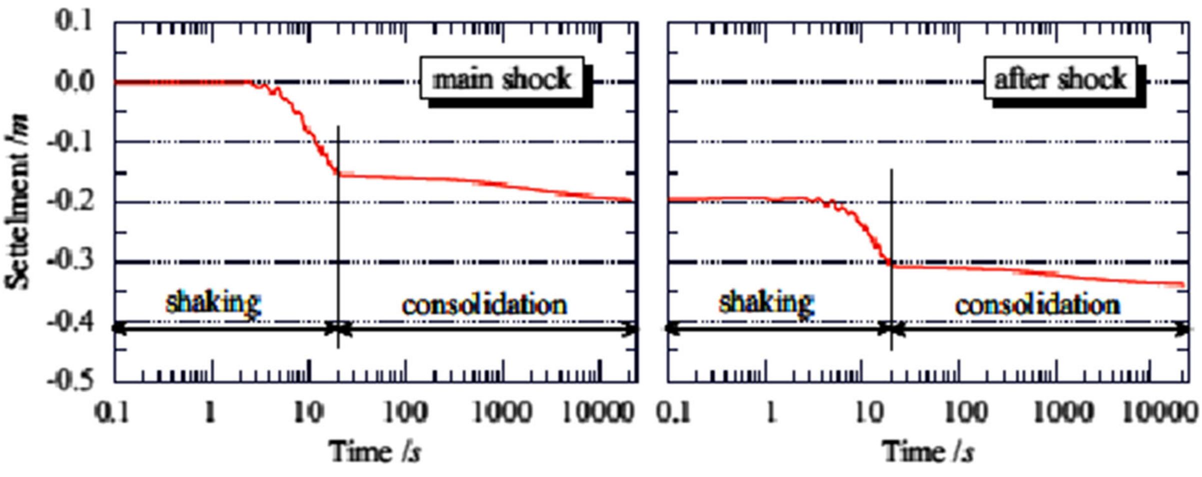

Fig. 19 The settlement at the crest (Xia et al. 2010)

Figures 20 and 21 show the crack process on the slab and distribution of the stress in the slab, respectively. It is worth noting that in order to avoid any tension on the slab, the dynamic deformation can be accurately controlled in the dam. In fact, the tension zone caused a complete on the slab surface during the earthquake. As well, a precise analysis of the material and the effect of the slab could be seriously considered in this category. As a consequence, the shift in the peak and a strong acceleration was excellent. Therefore, according to Fig. 22, the Nail-panel system and related techniques of reinforcement at the level 4/5 of the dam were suggested.

The interface impact on concrete slab rock-fill dam (CFRD) in the earthquake performance has been recently investigated (Bayraktar et al. 2011). Results emphasized that stress and displacement increased while the linear analysis was carried out. This study evaluated by one of the best finite element program (ANSYS).

Figure 23 shows the regular mesh method that was used in the model with respect to the strong ability of the program. An accurate interface was used regarding face to face connection in different zones. However, severe damage was obtained according to the linear analysis for each condition of the reservoir. Moreover, the low shear stiffness of the interface element was an effective factor for the hydrodynamic condition. In brief, nonlinear time history analysis indicated a safe condition.

An example (GEO-SLOPE International Ltd), showed how the results from a QUAKE/W dynamic analysis could be used in SIGMA/W to compute the permanent plastic strain and deformation that may occur when an earth dam is struck by an earthquake. This evidence encouraged engineering designers to use this program due to its strong ability (GEO-SLOPE 2016).

Figures 24 and 25 respectively show the vertical displacement and factor of safety during an earthquake. This report was a suitable approach to evaluate deformation and factor of safety during an earthquake.

Earth dam liquefaction and deformation analysis using numeric modelling has been investigated (Risheng Park Piao et al. 2006). Results indicated that the FLAC program has a specific ability to analyse with two-dimensions using the Finite-volume method to evaluate deformation, as shown in Figs. 26 and 27.

It was found that the assumption of allowing deformation was a very economical design compared to convergence. Moreover, buttress with a shear key method would resist during the earthquake, as seen in Fig. 28. Moreover, acceptable deformations were revealed. Briefly, reductions in the budget of the project with the shortened construction schedule were carried out using this program.

In terms of slope stability, the significant purpose is to establish a safe and economical design of structures such as earth dams, embankments, excavations, and landfills. The literature review indicated that there are some critical steps at the static condition of slope stability in artificial slopes like embankment and earth dam. The limit equilibrium methods (LEM) based on slice discrete of soil mass above the assumed failure surface with a circular slide line e.g., (Fellenius 1936; Bishop 1955; Spencer 1967) with a general slide line, e.g., (Janbu 1973; Morgenstern and Price 1965) and inclined slices, e.g., (Sarma 1973, 1979), were significantly carried out. Moreover, the finite element method (FEM) has been developed based on the

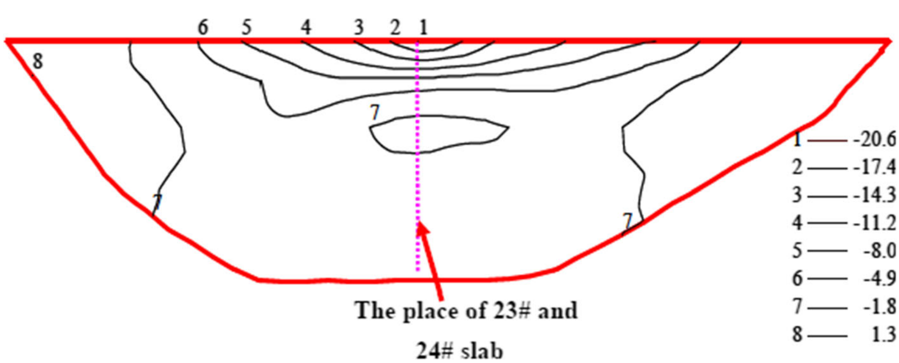

Fig. 22 Dam-axial concrete slab stress (MPa) after the earthquake (3D FEM) (Kong et al. 2010)

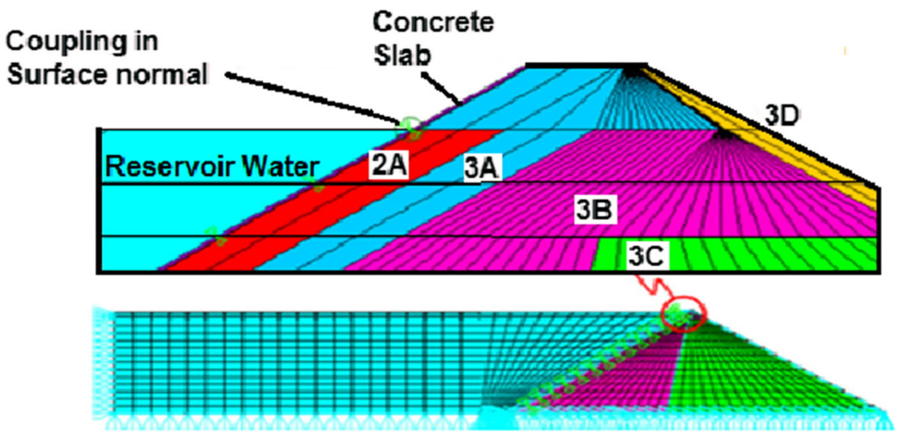

Fig. 23 Regular mesh method with reservoir interaction (Bayraktar et al. 2011)

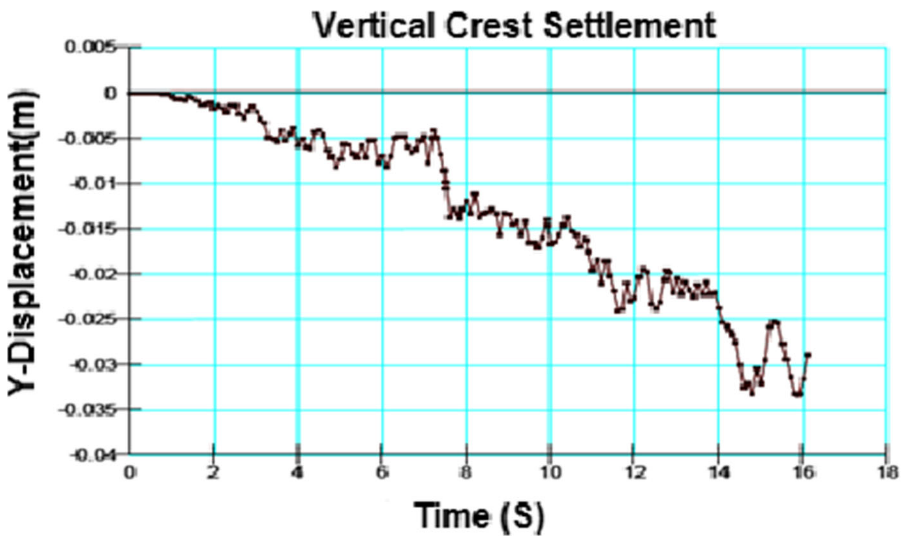

Fig. 24 Vertical displacement at crest during loading (GEO-SLOPE 2016)

displacement approach (Raja 2014, 2015). It is used to evaluate safety factors with shear strength reduction" SSR or "phi-c reduction" techniques (Maosong and Cang-Qin 2009; Baker 2006). Some combined methods based on probabilistic approaches of slope analysis and design with an aid of LEM, and FEM were investigated (El-Ramly et al. 2002; Li and Lumb 1987; Griffiths and Fenton 2004; Raja and Maheshwari 2014; Raja and Maheshwari 2016). Recently, an artificial neural network with built-in optimization techniques based on the self-optimizing genetic algorithms (GA) was considered (Sung Eun Cho 2009). In addition, the crossover and mutation probabilities method were adapted (Xie et al. 2008). Furthermore, due to structural evolution and parameter optimization, a two-stepped self-optimizing method with respect to a genetic algorithm (GA) was studied (Yang et al. 2004). This thesis tried to evaluate the safety factor using Geostudio 2007 program with Bishop's method. Soil properties such as the angle of internal friction, cohesion, relative density, and different geometry with respect to gradient and embankment height at the end of construction were assessed.

In terms of liquefaction, Liquefaction-induced ground deformation resulting from seismic activity are a main threat to the stability of earth dams comprising of, or underlain by, loose saturated granular soils. As a matter of fact, the pore water pressure is

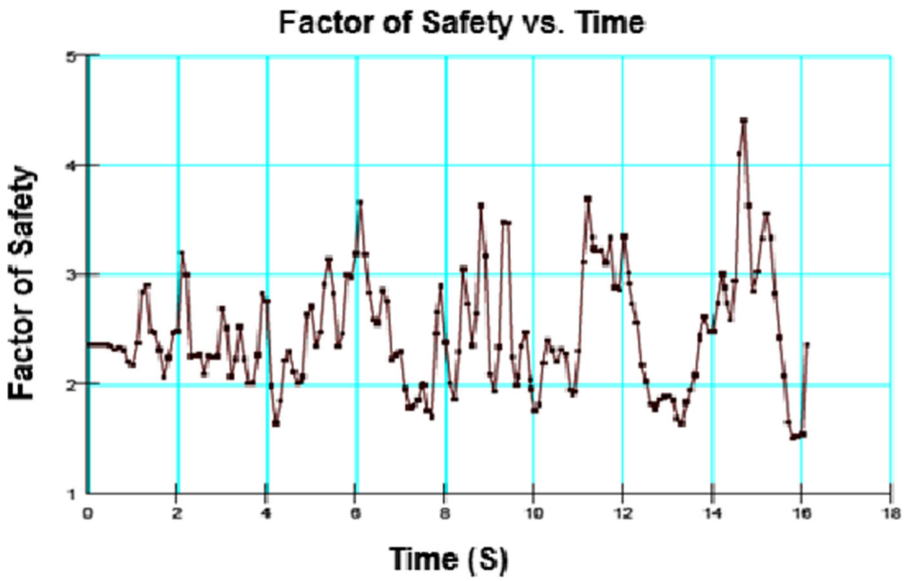

Fig. 25 Safety factor during an earthquake (GEO-SLOPE 2016)

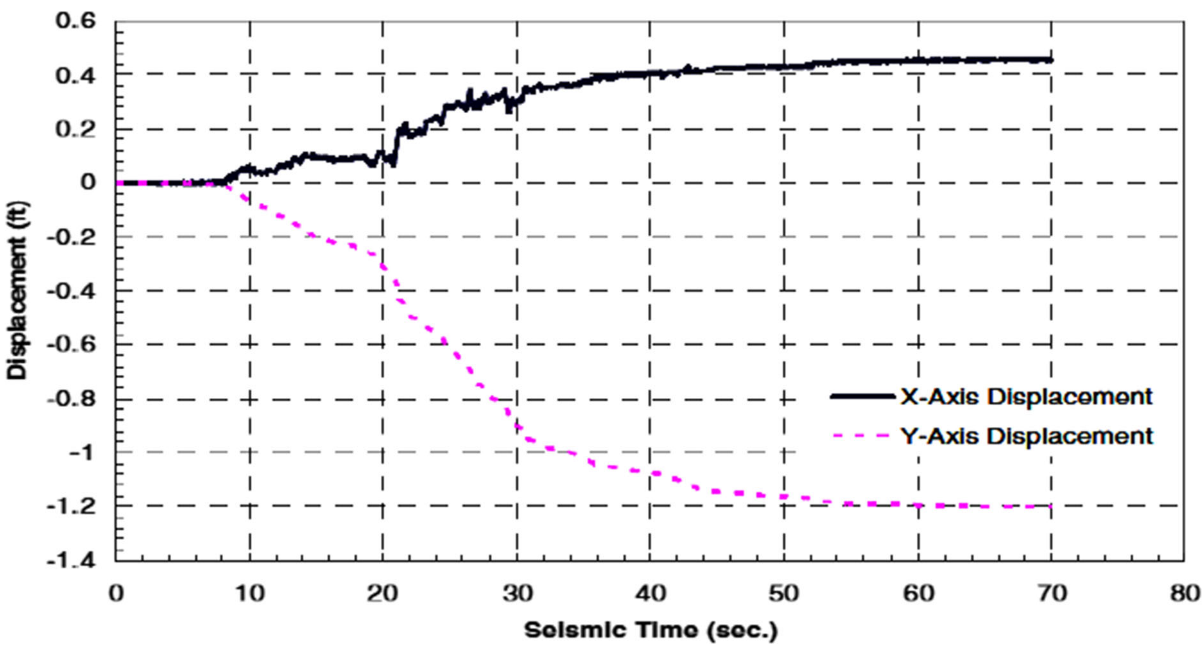

Fig. 26 Horizontal and vertical displacements during the earthquake (Risheng Park Piao et al. 2006)

increased during an earthquake. In parallel, the effective stress value is declined while pore pressure is increased, consequently. Shear stress can also decrease when effective stress is minimized. After that, failure is possible in this state as described. Several liquefaction forced earth dam failures or near-failures have been stated around the world during different earthquakes (Seed et al. 1990; Japanese Geotechnical Society and 1996; Adalier 1996). Such embankment damages were mostly destructive when the underlying saturated granular soils liquefied, resulting in cracking, settlement, lateral spreading, and slumping of the embankment (Duke et al. 1963;

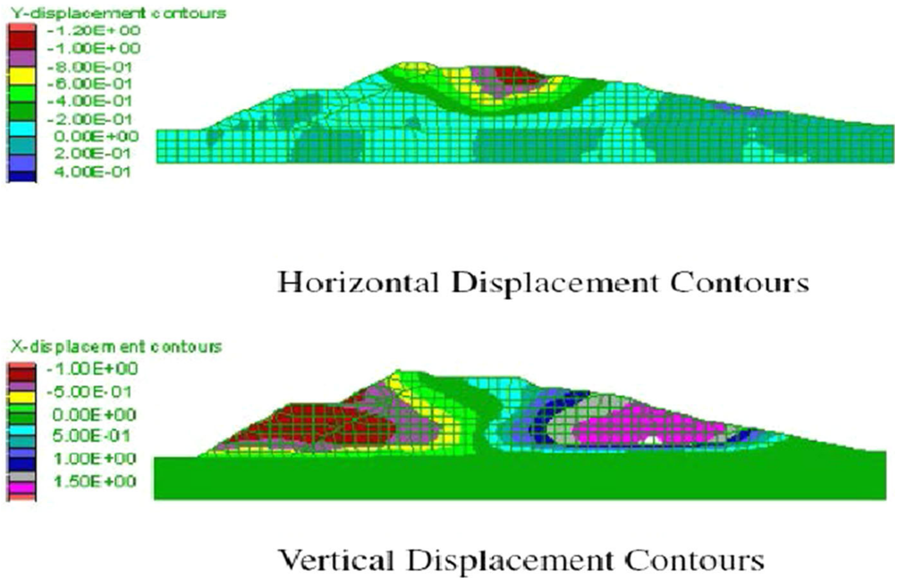

Fig. 27 Horizontal and vertical displacements contours during an earthquake in each substep (Risheng Park Piao et al. 2006)

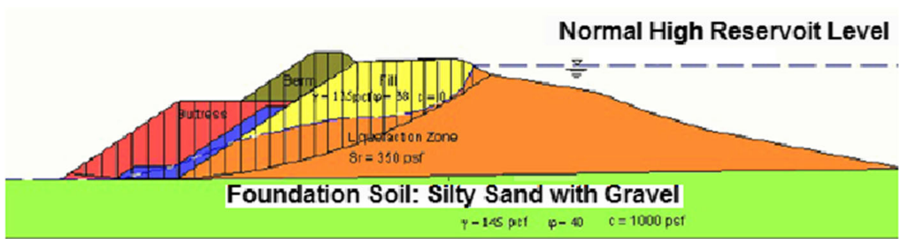

Fig. 28 Typical buttress and dam cross section (Risheng Park Piao et al. 2006)

Yokomura 1966; McCulloch 1967; Seed 1968; Tani 1996; Krinitzsky et al. 2002).

According to this part, it can be concluded that the physical property of the site is very important. Cooperation of Geotechnical engineering and geologist engineering together is necessary. Moreover, strong vibration is the main problem for dam performance. Acceleration at the top of the dam was two to three times compared to input data in the bedrock. Furthermore, a variation of the shear modulus in the core clay corresponded to a moderate effect on the amplification process. In addition, the significant rate of the amplification was featured using high shear modulus in the shell. Maximum displacement was located at the crest after the earthquake. However, the amplification process was very important and acceleration was three times more than bedrock. The behaviour of Concrete Face Rock-fill Dam (CFRD) exposed that specific attention to high stress in the upper part of the dam and control of the "whiplash effect" was very important. Moreover, additional stress and active deformation in the slab according to the tension zone are necessary for controlling. Briefly, the evidence in this context encouraged engineering designers to use programs to evaluate and estimate the dynamic deformation. Finally, Liquefaction should control during the design and after reservoir using tools instrumentation.

3 Earth Dam Reinforcement

There are some methods to improve the dam behaviour during an earthquake as are explained below. Back analysis of Geosynthetic reinforced well done for embankments on soft soils (Palmeria, 1998). It was a comprehensive report, as reviewed six previous methods. Each of them had a different situation on the geometry, material property, and Geotextile location. As a result, common assumption about horizontal force reinforcing was methodical. The relationship between strain rate and tensile strength was completely found in the analysis. In addition, the good agreement and proximity got to the estimated safety factor close to the unity. As well, the limit equilibrium method to assess the minimum safety factor has indicated a suitable approach while this value implied was greater than 1.2. In add, control of the safety factor was a major aspect of the dynamic study of earth dam and embankment. Also, three-dimensional analysis of embankments on soft soils has been investigated using the vertical drains by the finite-element method (Borges 2004). Results demonstrated that the use of vertical drain led to a significant reduction of the consolidation process by a factor of almost 10.

Figure 29 shows this detailed on the soft soil. It is worth noting that dissipation of the pore pressure was fast during and after construction according to increased stress level and maximum settlement at the end of construction. Finally, this effect indicated a kind of a “hardening” effect of the soil, a decrease of the long-term settlement, as well as the reduction of long-term horizontal displacement. Moreover, numerical analysis of embankments on PVD improved soft clays was evaluated (Yildiz 2009). Results exposed that the use of a PVD system in order to drain can dramatically improve dam behaviour in both MCC and S-CLAY1S models (See Figs. 30, 31).

However, the MCC model showed less value than other models. In addition, the maximum value recorded for MMC and S-CLAY1S model was 0.77 and 1.21 m, respectively over a period of 27 years. It was revealed that 3D finite element analysis of embankment with PVD improved flexible clay, and the matching technique was in good agreement with results using real boundary conditions. Finally, both methods like INDARATNA and REDANA for 2D analysis by PLAXIS software are suitable options. Embankments reinforced by piles and Geosynthetics-numerical and empirical studies dealing with the transfer of load on the soil embankment was recently studied (Le Hello and Villard 2009). In this study, the effect of reinforcing the slab on piles to improve the soil was considered.

As can be seen in Fig. 32, the main goal was the soft soil improving. It is also worth noting that after analysis, the experimental test and analytical methods were perfectly converged. In addition, the arching function has been found clearly in an empirical condition in compare to the analytical analysis. However, using Geosynthetics between the embankments and piles to improve this system was a very suitable method based on membrane behaviour. Figures 33 and 34 show the membrane approach observed based on the experimental and analytical methods. Finally, this study needs more research about the cohesion factor of soil because it does not been considered yet.

A simplified method for analysis of a piled embankment reinforced with Geosynthetics using numerical and experimental studies dealing with the transfer of load on the soil embankment has been recently evaluated (Abusharar et al. 2009). To conclude, the distributed stress of the Geotextile layer depends on a complex interaction of the soil characteristics, the property of reinforcing material, and ground flexibility.

Figure 35 shows the distributed stress on the Geotextile (Low et al. 1994). However, Geotextile was a sufficient approach with respect to the excellent compressibility of ground flexibility. Furthermore,

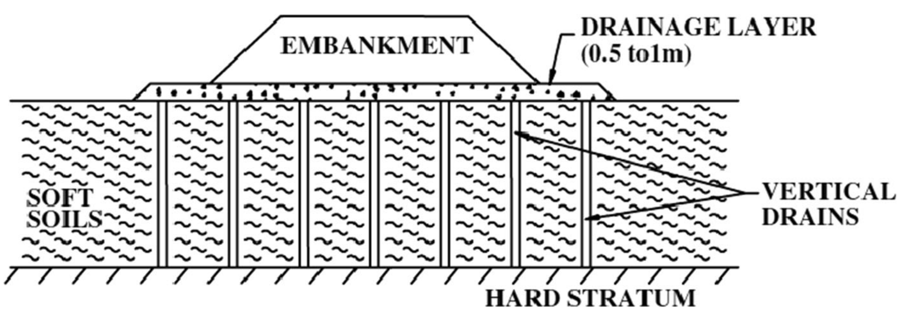

Fig. 29 Vertical drains on soft soil (Borges 2004)

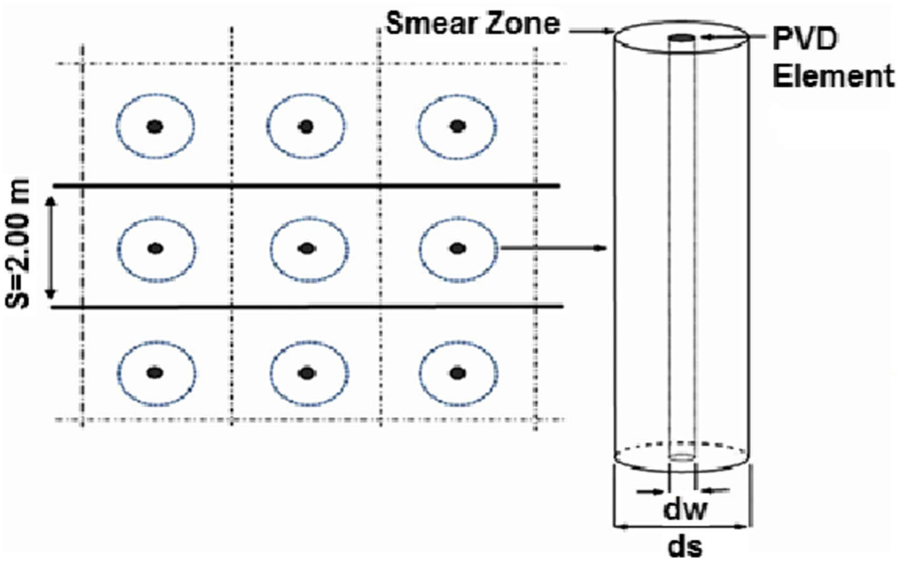

Fig. 30 Drain installation pattern (Yildiz 2009).

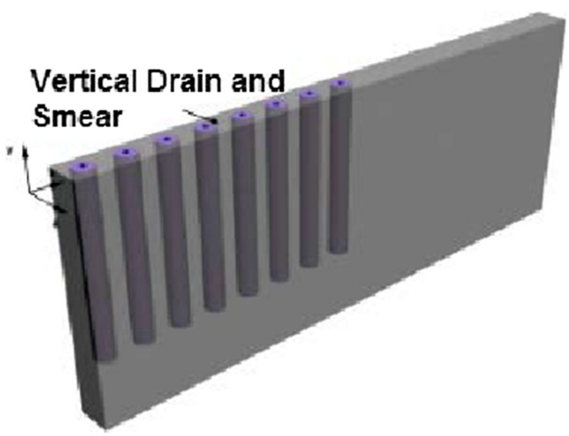

Fig. 31 3D model of vertical drains (Yildiz 2009)

this method was useful to improve soft ground (Low et al. 1994). In addition, to validate the theoretical model, the use of the full scale or prototype centrifuge was necessary. Finally, this method exposed a significant reduction of relative settlement in the embankment. In recent years this topic has been extremely developed.

Seismic displacement analysis of embankment dams with the reinforced cohesive shell have been recently performed (Noorzad and Omidvar 2010; Gordan and Adnan 2013). As a result, the movements of both ridges were declined based on reinforcement method. Also, the distributed strain reduced along with the height. Retention in the strengthening of the

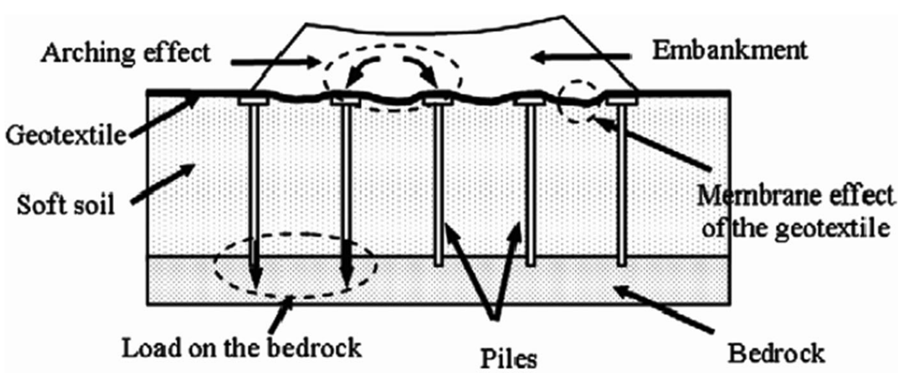

Fig. 32 Mechanism of the pile with Geotextile (Le Hello and Villard 2009)

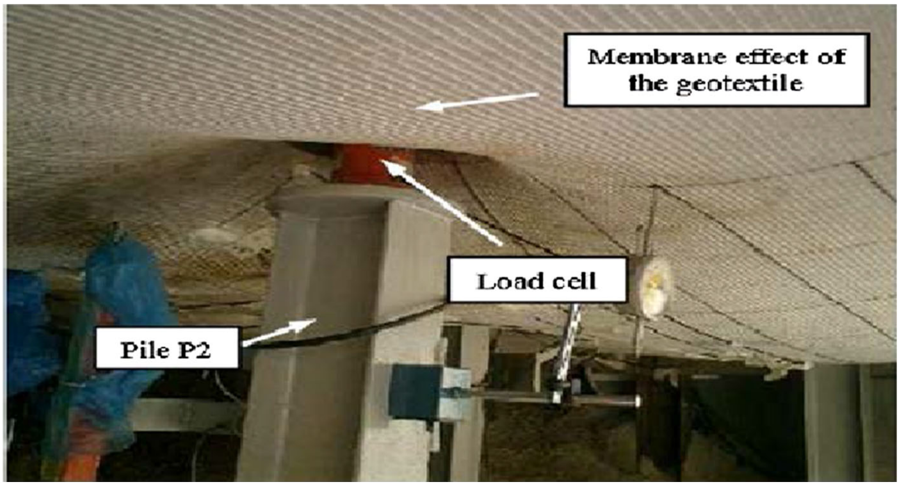

Fig. 33 Membrane effect observed over the pile (Le Hello and Villard 2009)

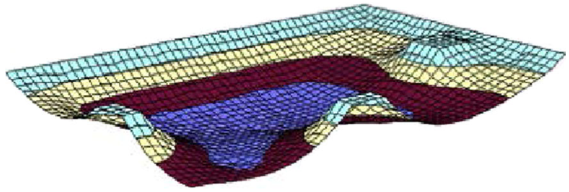

Fig. 34 Membrane behaviour of the Geotextile (Le Hello and Villard 2009)

embankment caused an increase in elastic response and resulted in higher amplification in compare to an unenforceable dam at the peak. Briefly, the maximum tension corresponded to a third, and half of the height reinforced zone for the upstream and downstream area, respectively.

Finally, in terms of seismic improvement during an earthquake, some methods are used such as Geosynthetic, vertical drains in soft soil; PVD system on soft clays, embankments reinforced by piles and Geosynthetics, and reinforced cohesive shell. They are successful methods but not economical. In fact, they are very suitable to reduce both displacements at the crest.

4 Fundamental Frequency

Some researchers have focused on assessing the dominant frequency using free vibration analysis. This analysis was very useful in order to collect input data for nonlinear and linear analysis of dam. Methods such as Time-history analysis and response spectrum analysis depend significantly on the distributed frequency in different mode shape. Moreover, the resonance phenomena can be controlled with dominant frequency.

Motion characteristic of compacted earth dams under earthquake excitations in Taiwan have been evaluated (Hwang 2008). This research introduced two methods such as RFRS and RRS to compute the fundamental frequencies during a short earthquake and indicated similar results in both.

Figure 36 shows the spectral ratio in three directions for both methods. The acceleration was dramatically increased at the crest based on the amplification process as the same in theory one or two dimensions of the dam. Correlations of the PGA, PGV, and PGD for four earth dams were not the same statistically, regarding results of other researchers based on recordings of free-field movements derived from minor to large earthquakes. This could be related to the small record as used in this study or the specific vibration properties of earth dams.

In other research, the effect of a core on dynamic responses of the earth dam have been studied (Tsai, 2009). As a result, the initial natural frequency was close to 2.5 Hz for 3D analysis while the length-height ratio (η) was more than 4 so this influence could be neglected.

Figures 37 and 38 show the initial frequency as respectively changed with length-height and width-height ratios. It is reduced when both ratios increased.

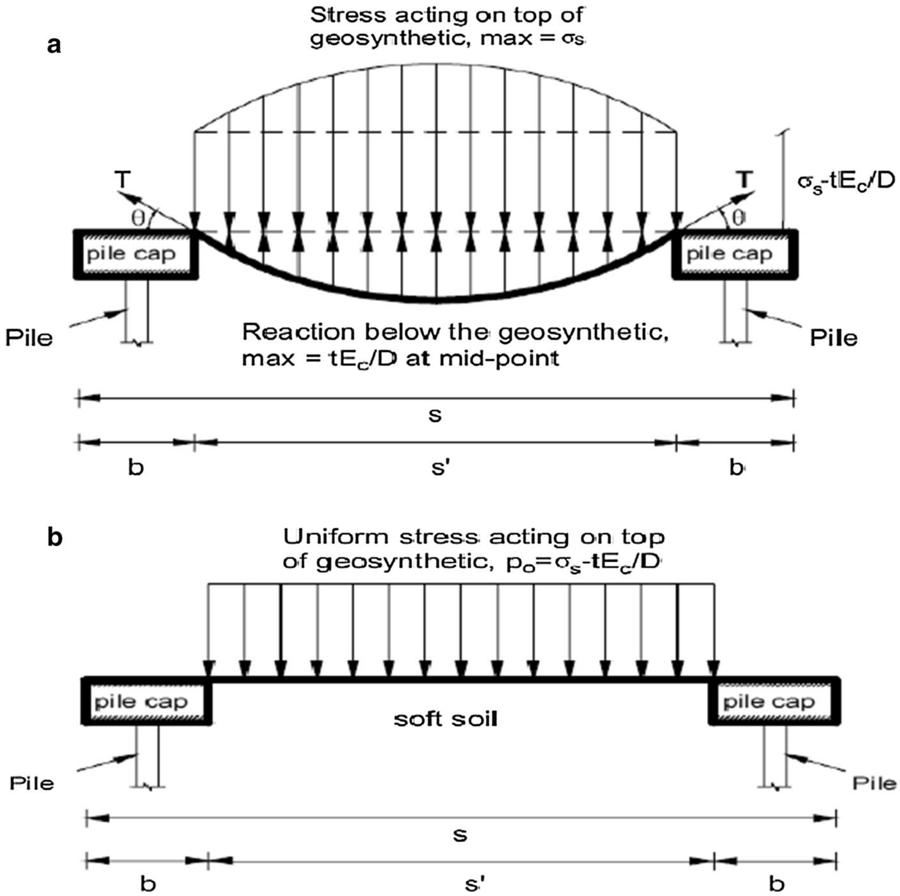

Fig. 35 Idealized stress distribution on geotextile based on Low et al. (1994)

Briefly, the natural frequency is enhanced slightly after the seepage phase. However, It is declined at the end of dynamic loading due to the reduction of the stiffness in the dam.

Finally, dominant frequency is a major function to analyse dam under earthquake. Both dynamic analyses such as nonlinear and linear are based on the frequency in different vibration modes. Time-history and response acceleration spectra analysis are a nonlinear and linear method, respectively. In addition, damping coefficients are important input data for time history analysis, as can be computed by results from free vibration analysis. In the case of response spectral analysis, the combination modes depend on frequency in different vibration modes. In other words, the resonance spectra method is another significant analysis that can be controlled by the dominant frequency.

5 Shaking table and centrifuge test

In case of verification, two methods like shaking table and centrifuge test are used in this context. This section tried to present some researches in this area.

Application of cement-mixed gravel reinforced by ground for soft ground improvement was investigated (Matsumaru et al. 2008). This research was carried out using a shaking table test. Results demonstrated the

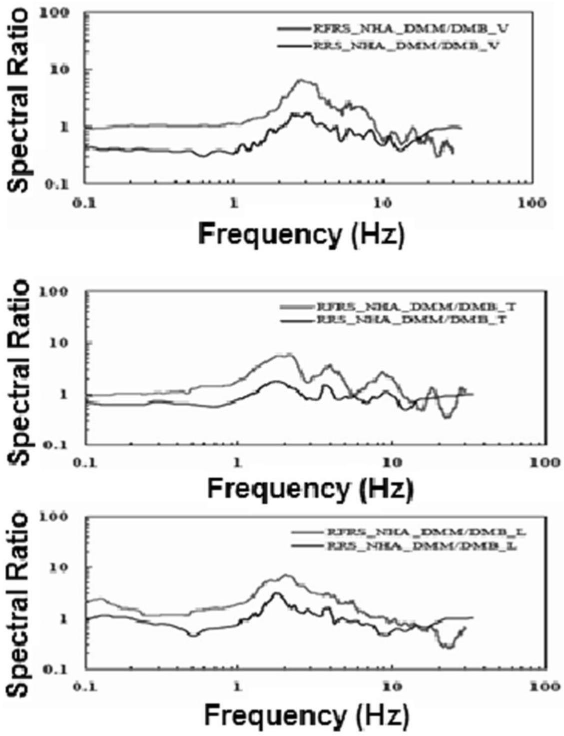

Fig. 36 The Spectral ratio of Nanhua dam during the 16 February 2000 earthquake (Hwang 2008)

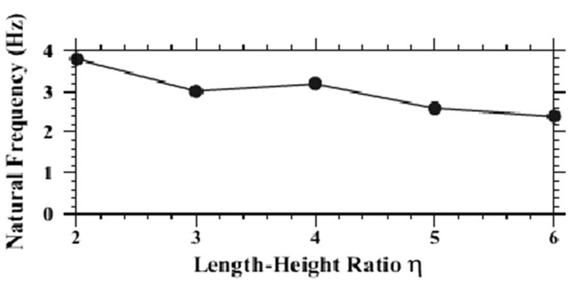

Fig. 37 The first natural frequency verse length-height ratio (Tsai 2009)

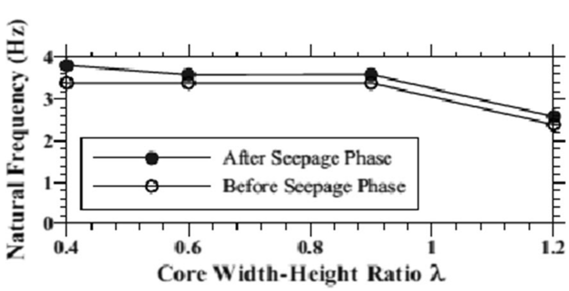

Fig. 38 The first natural frequency verse Width-height ratio (Tsai 2009)

increase of strength in gravel mixed with cement, compaction was clear, and Geogrid was good for cement-mixed in tension part of the bending model. Therefore, three cases were performed. First, column-net method and second countermeasure method with a slab to improve piles with 5 % of the cement ratio. The third case was the same as the second case but piles were not tested.

Figure 39 shows all cases. Figure 40 shows large deformation in case one after shaking. Finally, based on results, settlement control and excess pore water pressure in the second case were better in compare to the first case, and the first case was better than the third case. Figure 41 shows the second case could confine pressure less than other cases.

The seismic evaluation of embankment with shaking table test and the finite-element method has been recently investigated (Namdar and Pelko 2010). To summarize, not only Poisson's ratio obtained with the shaking table test was close to the value of prior works by other scientists but also the comparison of results between analysis modelling and the test indicated good agreement. Figures 41 and 42 show the distribution of pore water pressure and location of sensors, respectively. Moreover, deformation during dynamic loading was recorded. Figure 43 shows the modelling deformation during vibration for physical modelling in the first to four seconds. Consequently, the shaking table test is a suitable approach to validate numerical results. It seems that this test was necessary before the design and construction of a dam on the seismic zone.

Furthermore, the centrifuge model test of Geotextile-reinforced soil embankments during an earthquake have been recently evaluated (Wang et al. 2011). As a result, the strain magnitude of Geotextile and horizontal displacement of the Geotextile-reinforced embankment decreased with increasing Geotextile layers, decreasing water content of the soil, decreasing gradient of the slope, and decreasing amplitude of the earthquake wave. The vertical distribution of peak strain of the Geotextile was also affected by these factors.Figure 44 shows a different view of centrifuge physical modelling. Finally, the centrifuge test is another experimental method to evaluate the reinforced embankment. Furthermore, the performance of the earth dam under seismic effects was assessed by the shaking table test (Torisu et al. 2010). As a result, the deformation of the upstream

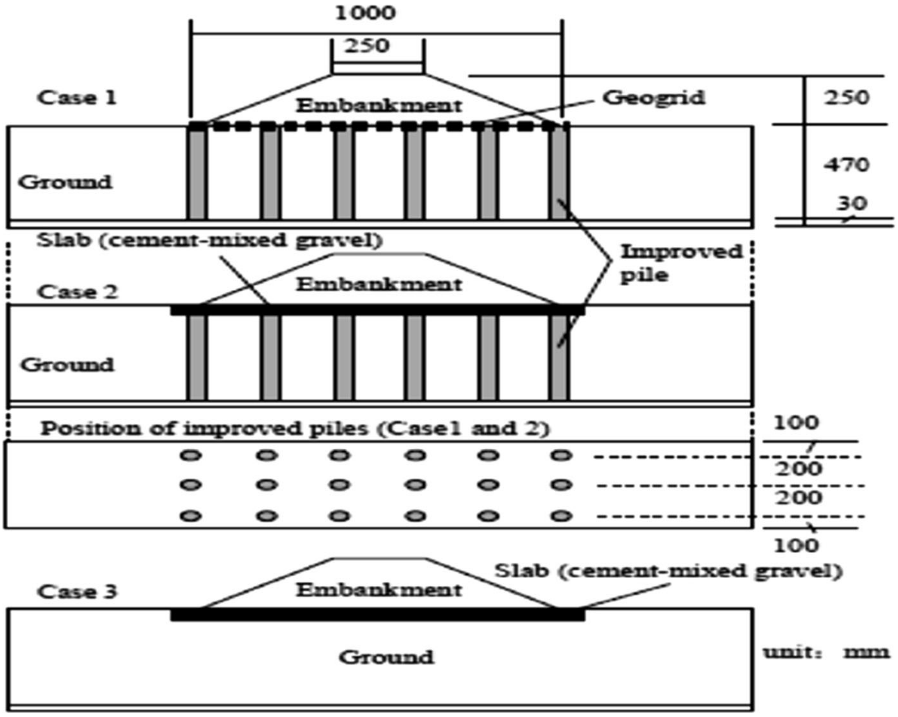

Fig. 39 Cases on the shaking table test (Matsumaru et al. 2008)

was greater than downstream according to reservoir interaction.

Figure 45 shows the reservoir interacted core clay of embankment. Moreover, the membrane model was very good to control deformation in dam and slopes in compare to other models. The increase of relative density led to decreased deformation in the core clay model and deformation of slopes was more than the dam body, as can be seen in Fig. 46. Finally, a comparison of results from this research indicated the best behaviour was lining the surface upstream during earthquakes.

Finally, to verify dynamic analysis using a numerical method, physical small-scale modelling can be used in both methods such as shaking table and centrifuge. Both techniques depend on the scale parameter to convert results from small-scale to prototype. However, a good agreement between the numeric method and experimental approach with high accuracy was obtained. Therefore, both are recommended for designing while the dam site is near to the active fault. In this thesis, the small-scale model using a vibrator table test instead of a shaking table was used.

6 Seismic Cracks in the Dam

The cracks are one of the major causes of dam failure in the past. Cracks may be developed during shrinkage, swelling, heaving or settling processes of the filled materials due to tension as well as due to the earthquake shaking. These cracks may be led to the failure of the dam due to piping, deformation, or excessive settlements in the dam materials. In the past, nearly 20,000 irrigation dams that were demanding repair to strengthen had got damaged during the long-standing use (Kato 2005). If earth-filled dams are not designed for seismic resistance, a considerable crack will be developed at the crest of the dam in the direction of the axis of the dam as shown in Fig. 47. Cracks are also developed in the rock-filled dam which was constructed in 1988 and designed for earthquake resistance. Though the acceleration at crest was not known, as the earthquake-induced the crack up to 3 m

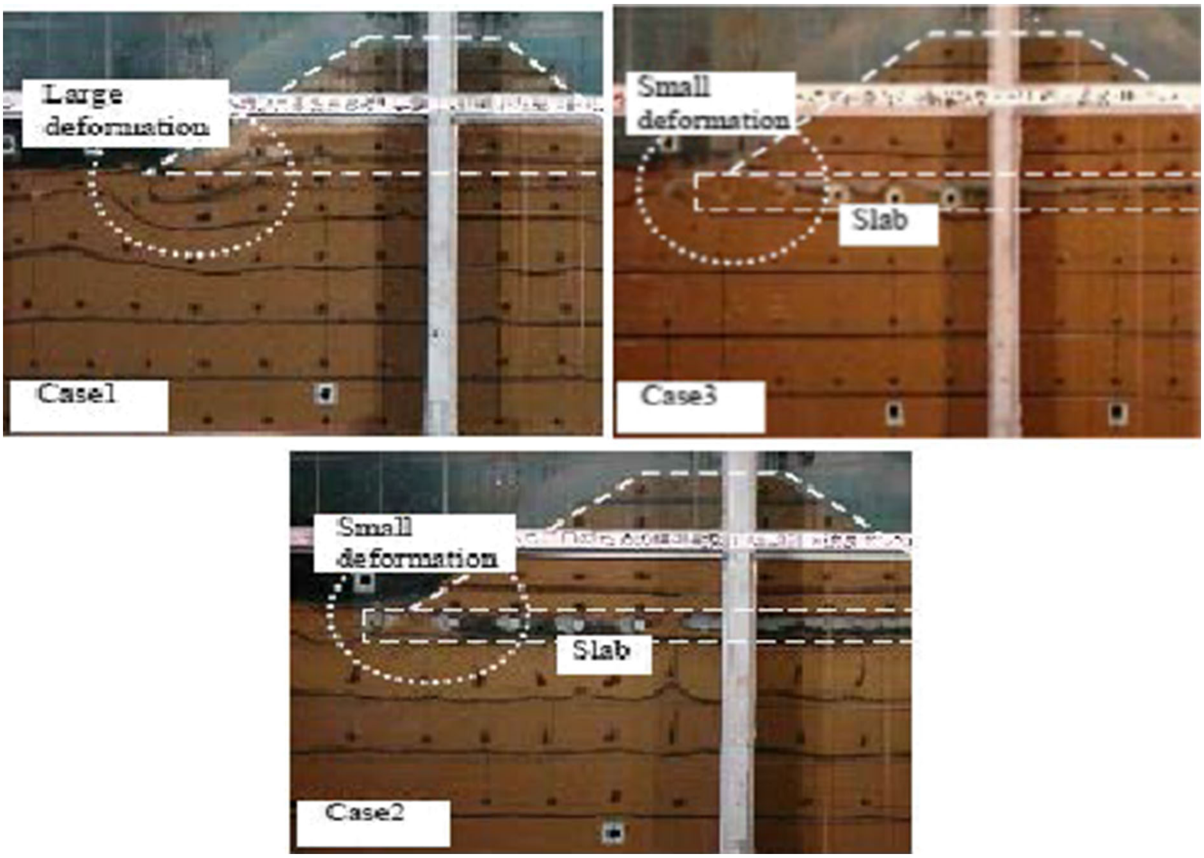

Fig. 40 Photographs after shaking for all cases (Matsumaru et al. 2008)

depth, it had inferred a large acceleration was hit the create of the dam. The cracks induced in the dam was not by sheer failure as the earthquake resistance design was examined by the slip-circle method (Kobayashi 2018). If the mechanism of failure is unknown, repairing the dam cracks is a difficult task.

The shanking table test can be used to determine the behaviour of earthen dams and embankments. It was observed that acceleration at the upper part of the dam has a significant effect on the stability of the slopes (Lin and Wang 2006). The centrifugal test can carry to determine the failure mechanism of the crack in the dam. The crack in the direction of the axis of the dam at the crest was investigated for tensile stresses by the centrifugal test which gets affected by vertical and horizontal vibrations (Masukawa et al. 2004). The relation between failure and natural period was developed (Tsutumi et al. 1975), and the influence of the aspect ratio of the dam on the mode of vibration was also investigated (Masukawa et al. 2008). The crest acceleration response and dam deformation were examined using the centrifugal loading tests for the core rock-fill earthen and concrete faced rockfill dams (Kim et al. 2011), deformation of the dams due to liquefaction in the dam foundation was investigated (Sharp and Adalier 2006). Furthermore, the liquefaction and dynamic response in loose sand dams and embankments were also studied (Ng CWW et al. 2004). The 1-G shaking table test was used to investigate strain distribution in the dam, the results exhibited that the slope felt the large shear strain and the upper part of the dam observed maximum volumetric strain (Miyanaga et al. 2015).

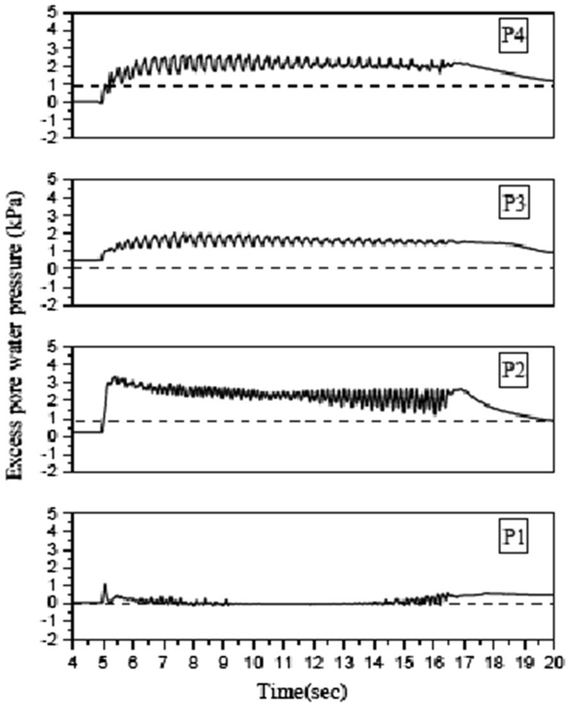

Fig. 41 Excess pore water pressure distribution (Namdar and Pelko 2010)

7 Conclusions

Based on a critical review in this paper, it can be concluded that the behaviour of dams during an earthquake is a complex phenomenon and need to be investigated with greater care and safety factors should be considered to minimize the approximation in the calculation of the dam analysis. In the past, many failures of dams occurred due to improper material selections and improper designs of the dam. Numerical Analysis is a widely accepted method in the dam analysis and FEM and FDM are commonly used methods, proving quite satisfactory results. Deformations, settlements, liquefaction, slope stability, and cracks developed in the crest are a major concern during the dynamic analysis of the dams. In the slope stability analysis, the limit equilibrium method is a basic method to provide an approximate factor of safety. Many software-based on FEM like Geo-Slope can be used to analyse the dynamic behavior of the dam very accurately. The following points can be drawn from the above discussion.

- Maximum acceleration in the earth dam during an earthquake is at the crest.
- A maximum settlement is at the centreline of the crest.
- An interface is a very important factor for numerical analysis.
- Plane stress (3D) and plane strain (2D) methods are useful for dam analysis.
- The amplification procedure depends on the soil properties.
- The shape of the valley affects the analysis type (2D-3D).
- Dam data monitoring is not access and available in more case studies.
- FEM and FDM are two methods using in numerical analysis.
- The body cracks are one of the major causes of the failure of dams in the past.
- Shaking table and centrifuge are possible for validating.
- Liquefaction phenomena is essential factor to dam design during an earthquake.

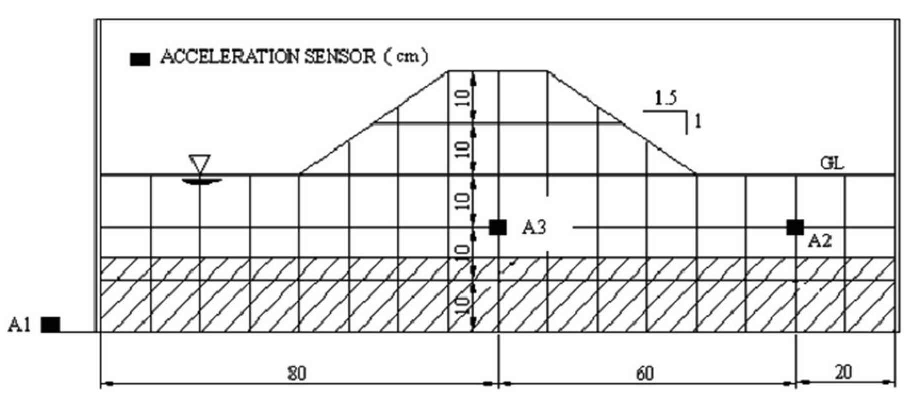

Fig. 42 Location of the pore water pressure sensor (Namdar and Pelko 2010)

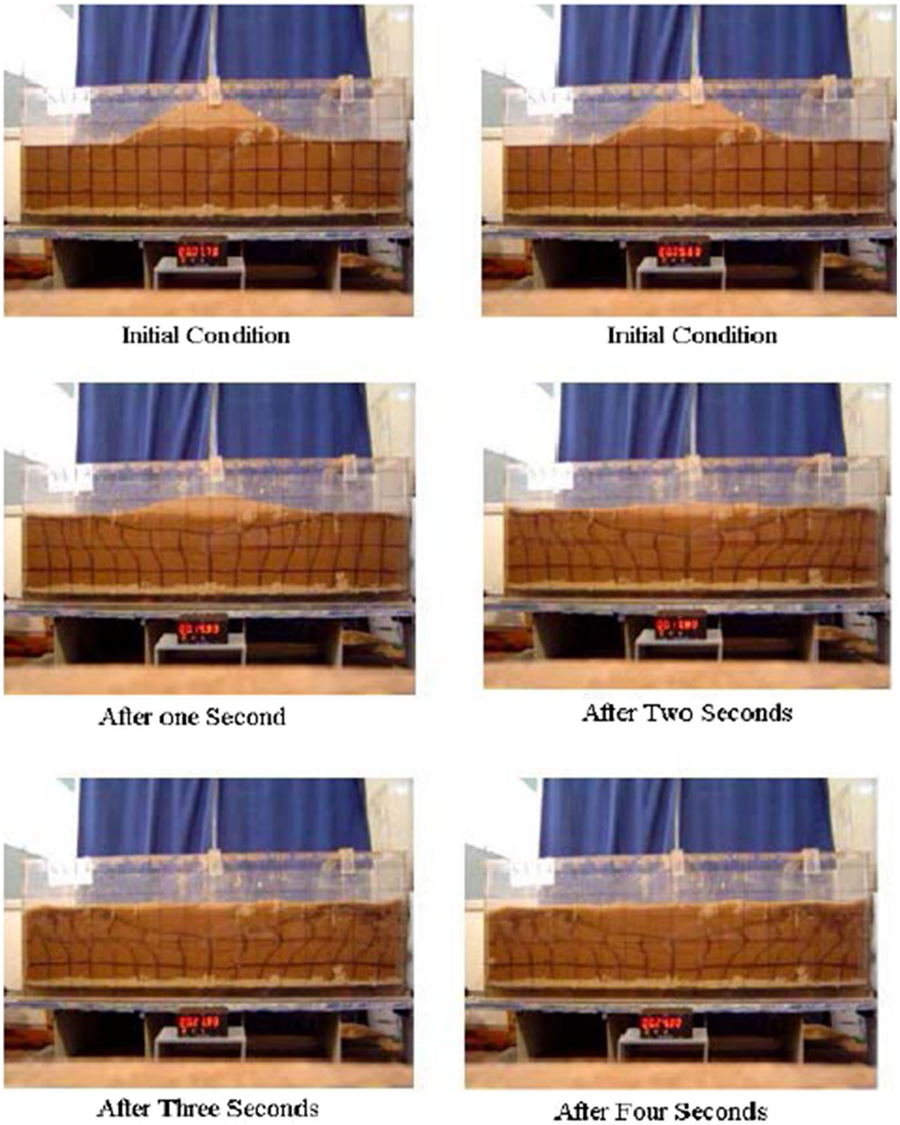

Fig. 43 Deformation shape of the embankment-subsoil system at different instants of time (Namdar and Pelko 2010)

- It is observed that acceleration at the upper part of the dam has a significant effect on the stability of the slopes.
- Mechanism of cracks developed at the dam crest can be understood using shaking table and centrifuge tests.
- Slope stability is very important for the static and dynamic condition.
- Dam reinforcement is possible using flexible materials.

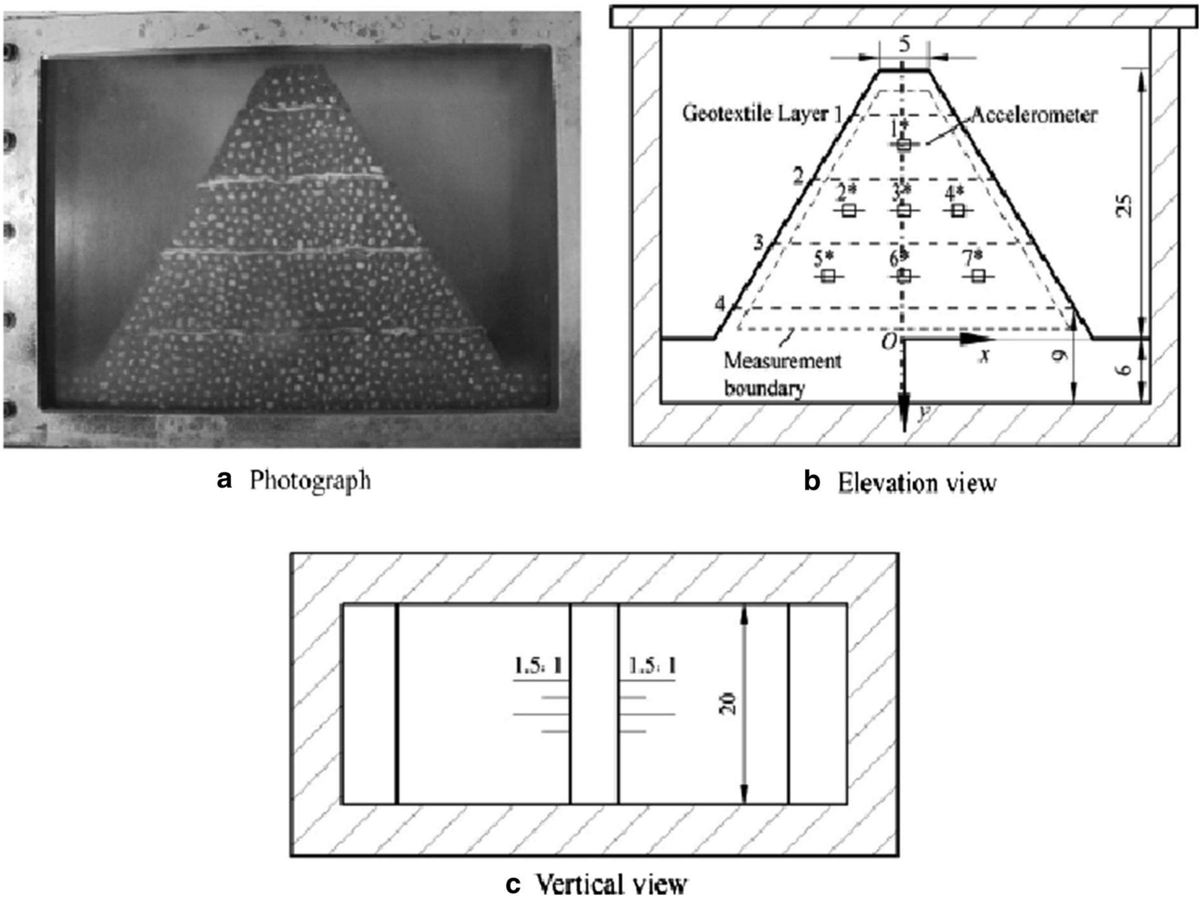

Fig. 44 Primary centrifuge test (Wang et al. 2011)

Fig. 45 An embankment with the reservoir on a shaking table test (Torisu et al. 2010)

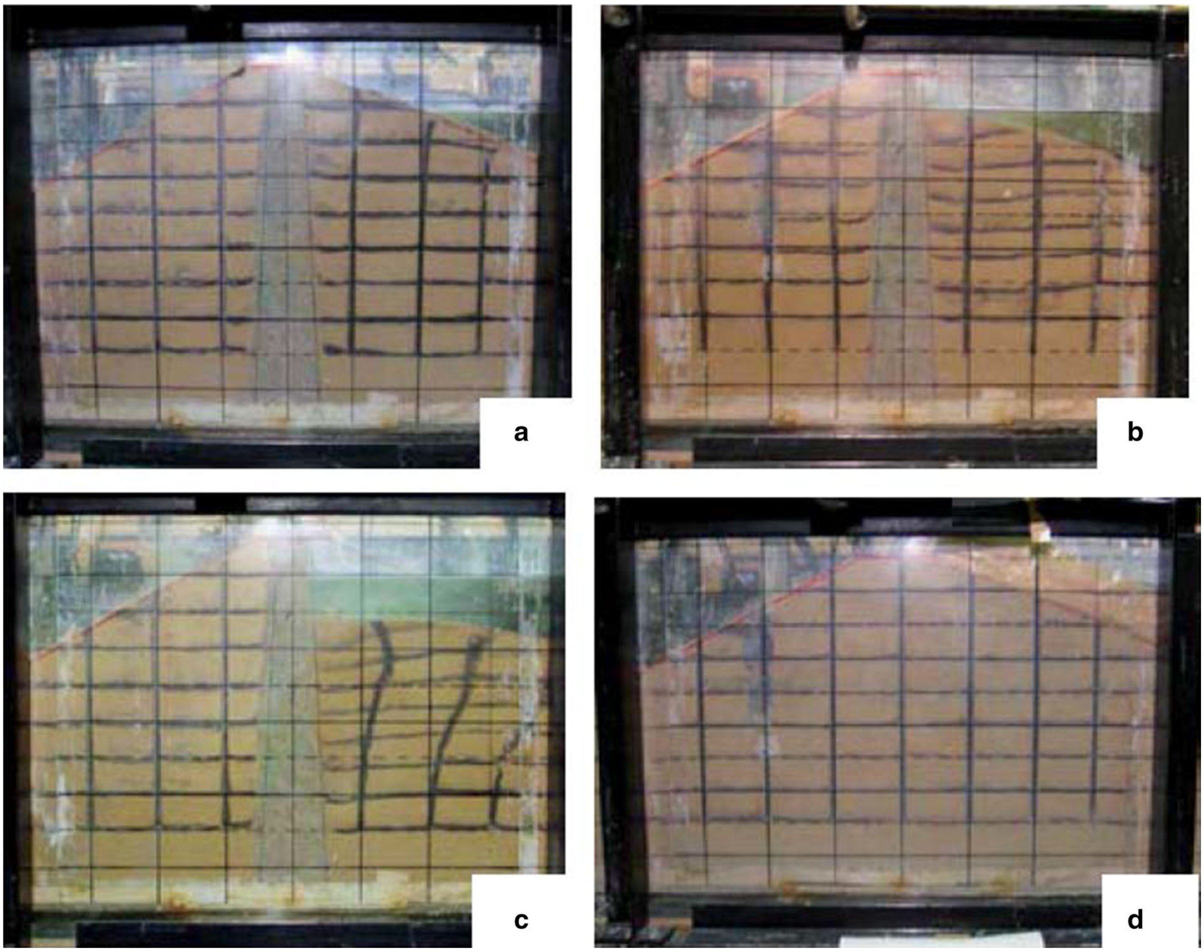

Fig. 46 Central core dam model with a relative density of dam body **a** 70 % **b** 50 % **c** 20 % and **d** dam model with membrane covered face (Torisu et al. 2010).

Fig. 47 Crack at the crest of the earth-filled dam (Kobayashi 2018)

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