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Back analysis of the Chang dam section in the Kachchh region of Gujarat, India
Retour de l'analyse Chang barrage dans la section Kachchh région du Gujarat, en Inde

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ABSTRACT

On 26th January, 2001, the Kachchh region of Gujarat, India faced devastating earthquake of magnitude 7.6 (Mw) popularly known as "Bhuj" earthquake. Various man made facilities including earthen dams faced varying level of damages during the earthquake and one of the major causes of failure is attributed to wide spread liquefaction of foundation soil. In the present study, a typical Chang dam sections located in the vicinity of epicenter is considered for the back analysis using dynamic numerical analysis with the help of commercially available software FLAC 5.0. The software has the capacity to model advanced non-linear constitutive material behavior under dynamic loading along with the pore pressure generation capabilities. The results of the back analysis of the dam section indicated that the presence of liquefiable soil beneath the foundation not only caused large deformation but also modified the failure pattern i.e. from slope to a base type failure, the feature which was also observed during field investigations. Dynamic numerical analysis is performed using the acceleration – time history record of the Bhuj earthquake developed by Iyengar and Raghukanth (2006) involving analytical procedures.

RÉSUMÉ

Le 26 Janvier, 2001, Kachchh la région de Gujarat, en Inde face terrible tremblement de terre de magnitude 7.6 (Mw) populairement connu sous le nom de "Bhuj" tremblement de terre. Plusieurs installations de l'homme, y compris les barrages en terre face variant de dommages pendant le tremblement de terre et l'une des principales causes de l'échec est attribué à une large diffusion de la liquéfaction des sols de fondation. Dans la présente étude, un type de barrage Chang sections situées à proximité de l'épicentre est considéré, pour l'arrière en utilisant l'analyse dynamique, analyse numérique, avec l'aide de logiciels disponibles dans le commerce FLAC 5.0. Le logiciel a la capacité de modèle non-linéaire des matériaux constitutifs de comportement sous charge dynamique avec les capacités de génération de pression de pore. Les résultats de l'analyse de retour de la section du barrage a indiqué que la présence de liquefiable sol sous la fondation non seulement causé de grandes déformations, mais aussi modifié le modèle c'est-à-dire l'échec de la pente à un type de base de l'échec, la fonctionnalité qui a également été observée au cours des enquêtes sur le terrain. Dynamique de l'analyse numérique est réalisée à l'aide de l'accélération - le temps record de l'histoire de tremblement de terre de Bhuj développé par Iyengar et Raghukanth (2006) concernant les procédures analytiques.

Keywords : Dynamic, numerical analysis, constitutive model, pore pressure, dams

1 INTRODUCTION

The Bhuj earthquake in India that occurred on 26th January 2001, caused wide spread damage to man made structures as well as sacrifice to human lives. The state department (Government of Gujarat) estimated that 15.9 million people (50% population of Gujarat) were affected, directly or indirectly, and more than 20,000 cattle were reportedly killed. The estimated economic loss was $5.0 billion. Through post-earthquake field reconnaissance reports findings, it was revealed that several earthen dams located in the region either failed or showed varying levels of damages during the earthquake due to presence of thick liquefiable alluvium beneath the foundation (Singh et al. 2005). The Chang dam, Kaswati dam, Tapar dam and Rudramata dam are few to mention. The liquefaction of foundation soil was relatively localized in the earthquake affected area and since the earthquake struck in the middle of a prolonged dry season, the liquefaction of the foundation soil beneath majority of earthen dams was limited (Singh et al. 2005). However, the presence of loose alluvium contributed to the reduction of the strength of the foundation soil.

The present study investigates the effect of presence of this layer on the failure mechanism as well as on the stability of the earthen dams based on field studies and investigations carried out after the earthquake by providing the representative insitu soil properties for the embankment materials. In order to investigate the stability of the earthen dams sections, a fully coupled non-linear dynamic numerical analysis of a typical Chang dam section is performed because of the several aspects which are either not considered directly in the previous methods developed for the seismic stability analysis procedures or if considered it is only approximate and in an unconventional way.

2 CHARACTERISTICS OF BHUJ EARTHQUAKE

The Kachchh region of Gujarat in India has a long history of earthquakes. The Bureau of Indian standards i.e. BIS:1893-2002 considers this region as a highly seismic active zone. The Allah Bund earthquake in 1819 (M>7.0), Anjar earthquake in 1956 (M = 7.0), and Bhuj earthquake in 2001 (M = 7.6) are the few major earthquakes occurred in the past. The details of the 26th January 2001 Bhuj earthquake that occurred in the Kachchh region of Gujarat, India, are given below.

Date:	26th January, 2001
Epicenter:	13 kilometers NW of Bhachau (Gujarat), India
Latitude:	23.40^o N ; Longitude: 70.32^o E
Origin Time:	08:46:41 IST
Magnitude:	Mw 7.7 (USGS)
Focal Depth:	18 km; 70 km east of city Bhuj

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In the absence of strong ground motion record, Iyengar and Raghu Kanth (2006) used analytical methods for the estimation of ground motions parameters of the Bhuj earthquake and in their study, a set of three aftershock records have been used as empirical Green's functions to simulate ground acceleration time history and 5% damped response spectrum at Bhuj City. It is estimated that the main shock PGA at Bhuj City between 0.33 g–0.37g. These available informations are utilized in the present study to perform the dynamic numerical analysis of the dam sections.

3 DYNAMIC NUMERICAL ANALYSIS OF THE CHANG DAM SECTION

The first step for the dynamic numerical analysis of the dam sections involves the establishment of initial stresses and pore pressure distribution in the embankment dam body and foundation soil under static condition. The second step involves dynamic numerical analysis using acceleration-time history record of the earthquake data. In the present work, two cases are considered i.e. Case (a) Absence of liquefiable layer beneath the foundation and Case (b) presence of liquefiable layer beneath the foundation in order to study the effect of presence of liquefiable layer beneath the foundation and on the stability of the dam sections. The analysis is also performed taking two different constitutive behaviors of the embankment materials i.e. Mohr-Coulomb model and Finn-Byrne pore pressure generation models and results are compared and discussed.

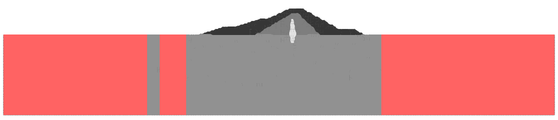

Figure 3(a) Numerical model of the old Chang dam section

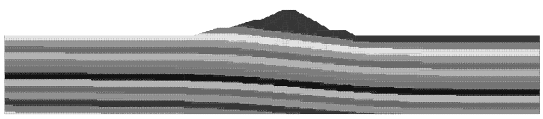

Figure 3(b) Distribution of pore water pressure distribution under static condition

Table 1 Input soil properties of the old Chang dam section

Properties	F. soil	Core	Shell	Wall	L. soil
Mass density (kg/m^3)	1440	1410	1530	2400	1480
Elastic Modulus (MPa)	112	97	108	150	86
Poisson's ratio (ν)	0.36	0.38	0.32	0.15	0.34
Cohesion (c) kPa	5.0	2.5	1.5	-	1.0
Angle of internal friction (ϕ)	35°	30°	32°	-	20°
Shear wave veloci(C_s) m/sec	169	158	163	-	153

As indicated earlier, for the dynamic numerical analysis, a static equilibrium calculation always precedes the dynamic analysis. For the first case (a), the dam sections have been analyzed without considering the presence of liquefiable layer beneath the foundation. Fig. 3a shows the numerical model of the old Chang dam section without any presence of liquefiable layer beneath the foundation which is considered for the case (a) analysis and Fig. 3b shows the distribution of pore pressure at the end of static loading.

As provided in Table 1, for the case of old Chang dam section, the lowest shear wave velocity is calculated as 153 m/sec for the liquefiable soil. In the present case, the maximum allowable frequency content (cut off frequency, fc) for the accurate transmission of the input motion for a grid size of 1m ×1m (a smaller grid size required high memory and more computational time) will be 15.03 Hz. Fig. 4(a) shows the power spectrum of the raw acceleration time history record of the Bhuj earthquake data. It can be seen that almost all the power of the Bhuj earthquake data exist within the frequency content of 17 Hz. Fig. 4(b) shows the power spectrum of the filtered (fc = 15Hz) x-acceleration time history record of Bhuj data. Twice integration of filtered x-acceleration time history record shows a constant drift in the end of the input motion. it is required that the velocity time history record should be baseline corrected so that there is no constant drift or displacement at the end of the numerical simulation.

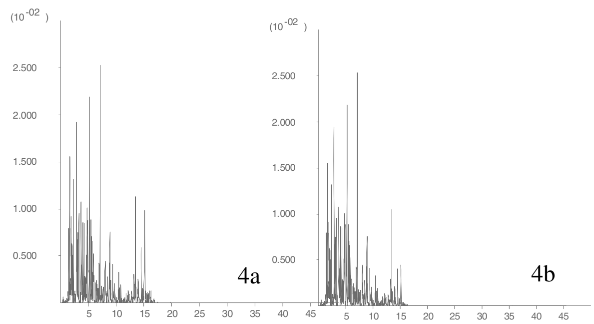

Figure 4. Power spectra of the raw (4a) and power spectra of the filtered (f_c = 15 Hz) x– acceleration-time history record of the Bhuj data (4b) as analytically developed by Iyengar and Raghu Kant (2006)

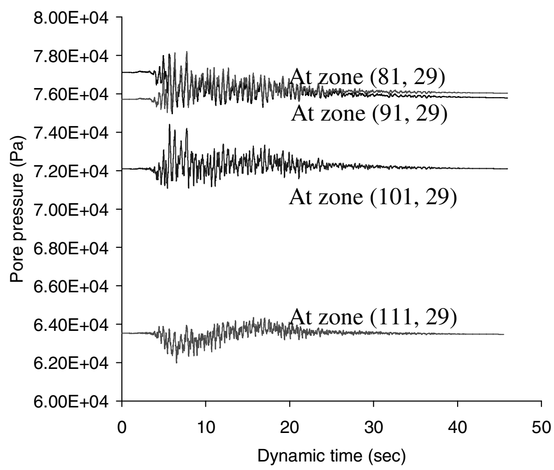

Figure 5 Pore pressure generation time history record with at four different locations beneath the foundation (Mohr-Coulomb material model)

It should be understood that the applied input motion is as a stress boundary, hence it is required that the x-velocity time history record at the base of the dam section should be monitored during the dynamic loading to ensure that the correct input motion is provided at the base. Fig 5 shows the pore pressure generation time history record at four different locations beneath the foundation and it can be observed that the change in pore pressure is not significant when the soil is assumed to behave as Mohr-Coulomb material. The pore fluid simply responds to changes in pore volume caused by the mechanical dynamic loading.

Fig. 6 shows shear stress (τ_{xy}) vs. shear strain (ε_{xy}) time history record in a particular zone beneath the foundation. It can be seen that the model well captures the hysteretic type of damping and also the shear stress and shear strain loops shows accumulation of shear strains with the passage of time history of the input motion indicating a permanent shear deformation in that zone.

Fig. 7(a) shows the relative x-displacement –time history record of the crest of the dam section showing maximum x-displacement of 18.0 cm and Fig. 7(b) shows x-displacement contours within the body of the dam section at the end of the dynamic loading and it clearly indicates the possibility of slope failure in the absence of liquefiable layer beneath the foundation.

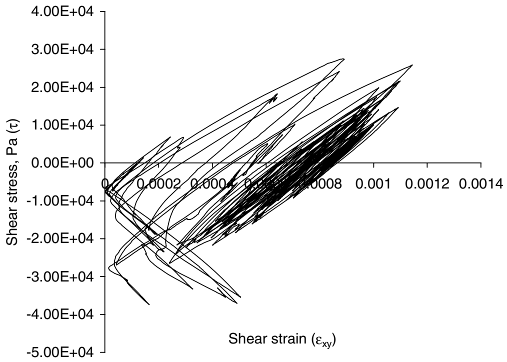

Figure 6 Shear stress (τ_{xy}) vs. shear strain (ε_{xy}) time history record with in a particular zone (110, 29) beneath the foundation

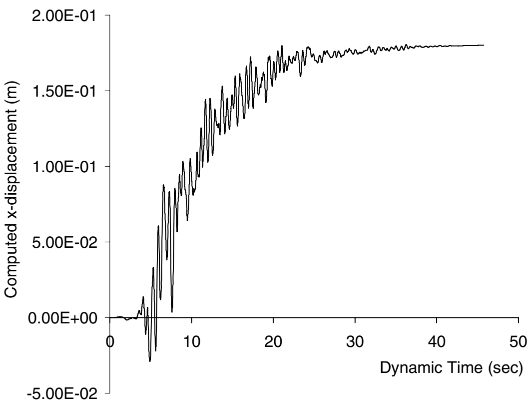

Figure 7a Relative z- displacement – time history record of the crest of the dam section calculated for case (a) study [maximum displacement = 18 cm]

In the second case (b), the old Chang dam section is analyzed with the consideration of liquefiable layer beneath the foundation. The material behavior is assumed as Finn pore pressure generation model in the liquefiable layer with a SPT value of 3.0. The liquefiable layer was present on both the sides (u/s and d/s) of the foundations as indicated by Singh et al. (2005). Fig. 8 shows the pore pressure generation-time history record at different location within the liquefiable layer beneath the foundation. Comparing Fig. 5 and Fig. 8, it can be seen that the Finn and Byrne pore pressure generation model captures the building up of the excess pore pressures in the zones during the earthquake loading, which is not well captured in Mohr-Coulomb material behavior.

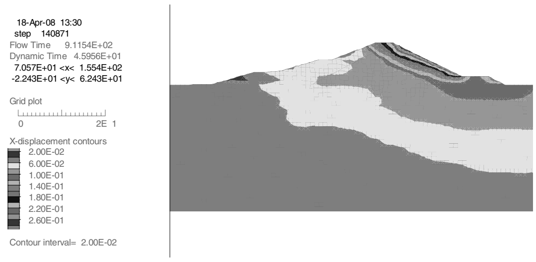

Figure 7b x-displacement contours indicating major movement within the slope of the dam section on the downstream side

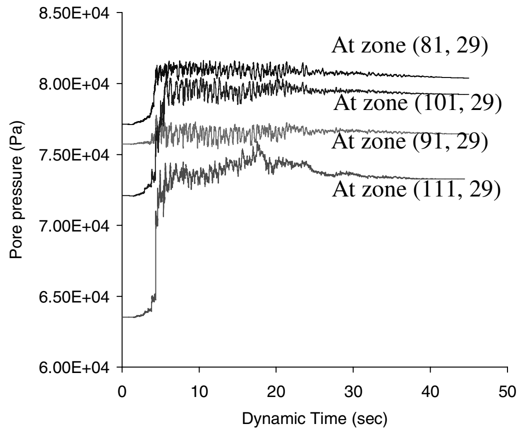

Figure 8 Pore pressure generation –time history record with in the liquefiable layer beneath the foundation (Finn –Byrne model)

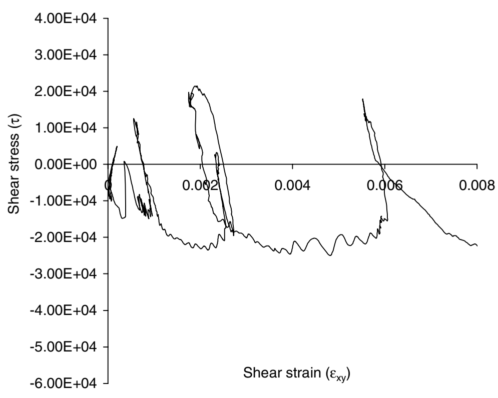

Figure 9 Shear stress - shear strain time history record in a typical soil zone (110, 29)

Fig. 9 shows the shear stress (τ_{xy})-shear strain (ε_{xy}) time history record in a typical soil zones within the liquefiable layer. It can be seen that there is gradual softening of the shear stress (τ_{xy}) -shear strain (ε_{xy}) loops with the passage of time indicating large accumulation of shear strains (maximum strains are not shown in the figures) in the liquefiable layer that are responsible for causing large scale deformations within the body of the dam section.

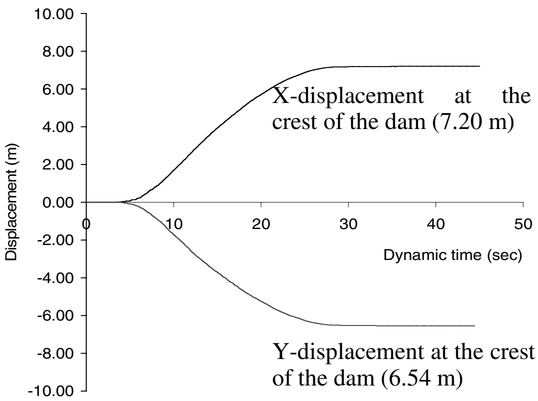

Figure 10 Displacement time history computed at the crest of the dam section

Fig.10 shows the relative x-displacement and y-displacement time history record of the crest of the dam section, respectively indicating large movements in the horizontal (7.2m) and vertical directions (6.54m). It can be seen that the computed displacements are fortuitously quite comparable with the field observations that revealed the movement of the crest of the Chang dam in the range of 6.0m – 12.0m.

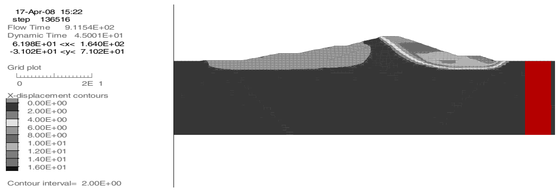

Figure 11. x-displacement contours within the body of the dam section indicating a base type failure due to presence of liquefiable layer

Fig. 11 shows the x-displacement contours within the body of the Chang dam section that indicates the possibility of base failure mechanism due to presence of liquefiable layer beneath the foundation, the feature which was also observed in the field reconnaissance and in the previous study performed by Sivakumar Babu et al. (2007) pseudostatically using limit equilibrium analysis. Hence, from the back analysis of the Chang dam section, it can be stated that the presence of liquefiable layer beneath the foundation soil was responsible for the large deformations within the body of the dam section. It also triggered a base type of failure mechanism, a feature which was also observed in the field.

With these results of the back analysis of the dam sections; it can be stated that although the extent of damages of the dam sections observed in the field investigations are not exactly computed through the numerical analysis, yet the underlying failure mechanism and patterns are well captured. The reasons for the anomaly in the observed performance and numerical calculations can be attributed to several factors involved. (i) The first and foremost is the lack of sufficient geotechnical data and extensive laboratory testing investigation reports for the embankment materials before of the occurrence of earthquake. The representative soil properties considered for the analysis investigators were only able to capture the mechanism of failure. To some extent the computed deformations for the old Chang dam section was in good agreement with the observations in the field. Other factors includes (ii) heterogeneity in the material deposit within different zones of the dam section, (iii) post liquefaction behavior, (iv) spatial variability modeling of the soil properties and (v) spatial variation of the input ground motion, etc. Inspite of all these limitations involved in the dynamic numerical analysis of the dam section, it can be concluded with a certainty that the presence of liquefiable layer beneath the foundation was the major cause of failure or reason for the significant damages within the body of the dam sections. In the absence of liquefiable layer, the dams would have been survived requiring minor repair works. It is also observed from the results of the numerical analysis that the presence of liquefiable alluvium triggered the base type of failure mechanism, a feature that is also observed in the field observations.

4 CONCLUSION

The present work discuss the results of the fully coupled dynamic numerical analysis of the dam sections using Mohr-Coulomb material model and Finn pore pressure generation model. The comparison of results indicated that the Mohr – coulomb material model does not capture the essential feature of dynamic loading i.e. pore pressure generation and also the computed displacement are relatively less than those obtained using Finn pore pressure generation model. Further, it is established that the presence of liquefiable layer beneath the foundation not only caused major damages to the dam sections, it also modified the failure pattern (a base type of failure mechanism instead of a slope failure). The results of the back analysis indicate that the underlying mechanism and failure pattern is well captured and consistent with the observed field performances of the dam sections during the earthquake.

It should be understood that with advancement in understanding the soil behavior under dynamic loading conditions and development of geotechnical analysis approach in the numerical tools, the prediction of seismic response of an earthen dam under earthquake loading conditions is considered to be a better tool than any other methods of analysis like limit equilibrium based pseudo-static approach or deformation based analytical solutions. Inspite of these advancements, it is required that any numerical tool should be used with caution and the results obtained from the numerical analysis requires lot of judgment and understanding before making any definite conclusions. It is because of the fact that (i) in any numerical analysis, actual field conditions can not be reproduced exactly in the numerical modeling process. There is always some approximation and assumptions involved in the process of numerical modeling and analysis; (ii) for the two dimensional analysis, the input motion is given in the x-direction only which is far from the real 3-dimendional actual earthquake motion; (iii) the average values of soil parameters, defining its strength and stiffness characteristics, are used as input parameters for the numerical analysis but in actual field conditions these parameters vary in space.

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