

DENSE ZONE IN SUBSOIL OF EMBANKMENT AS A COUNTER MEASURE AGAINST EARTHQUAKE

Chandradhara, G. P.¹, Prasad, S. K.² and Revanasiddappa. K.³

ABSTRACT

Crest settlement, longitudinal cracking and slope failure of embankments and river dykes is a common feature of damage during earthquake. Recent studies have observed that the weakening of subsoil either due to liquefaction or due to softening of clayey soil is one of the major causes for such failures. Mitigation against such failure of embankments using densification in a specific region can be one of the convenient techniques of counter measure. This paper focuses on three aspects of failure of earthen embankments and counter measures to mitigate the damage. Initially, field investigation performed after the Bhuj earthquake of January 2001 in India in one of the earthdams is discussed in which subsoil weakening causing severe damage has been brought out. Secondly model testing using 1G shaking was performed to understand the performance of dense zone in subsoil as a mitigation measure against seismic failure of embankment – subsoil system. It was inferred that such a mitigation measure is suitable. Finally, analytical justification to the above inference is made using Finite element approach.

Keywords: Embankment, Liquefaction, Field Investigation, Shaking table, Finite Element Analysis

INTRODUCTION

Failures of embankments during earthquakes are common. Being one of the most important infra-structural facilities, any such failure to embankments can cause enormous damage to life and property and embankments can therefore be considered as life line facilities. River dykes, highway embankments, flood training works, earthdams etc. are some typical examples of embankments. Normally, earthen embankments are the most preferred structures considering the abundant availability of soils and economy of construction. Hyogo Ken Nambu earthquake in 1995 witnessed failure of a number of embankment dams. The failure ranged from minor to moderate levels of damage. Later, Bhuj earthquake of January 2001 in Gujarat, India witnessed the damage to almost each and every earthen dam with in 50 km radius of the epicenter of the earthquake. The region in the western province of Gujarat called Kutch area consisted of around 20 medium and 165 minor irrigation schemes which included earthdams. Majority of these dams suffered moderate to major levels of damage. In the past, many earthdam failures have been reported. Among these, the famous San Fernando dam breach during 1971 San Fernando earthquake occupies an important landmark.

Different types of ground and embankment failures depending on site conditions are commonly observed during earthquake. These failure mechanisms include lateral spreading, loss of bearing capacity, settlement and slope failure. Any of these mechanisms can potentially cause damage to embankments. There are a variety of ground treatment methods available for improving the properties of weakening soils in order to reduce the potential for earthquake induced failure and associated

¹ Assistant Professor, Department of Civil Engineering, S J College of Engineering, Mysore
prasad_s_k@hotmail.com

² Assistant Professor, Department of Civil Engineering, S J College of Engineering, Mysore
chandu_gpc@yahoo.com

³ Professor and Head, Department of Civil Engineering, U B D T College of Engineering, Davanagere

damage. Koga and Matsuo (1990) performed 1g shaking table tests to investigate the deformation mechanism of dykes founded on loose saturated sand deposits. They showed that embankment slumps and cracks as a result of softening of the underlying ground associated with pore water generation. Adalier et al. (1998) evaluated the effectiveness of different countermeasures based on the recorded dynamic response by conducting a series of centrifuge tests.

This paper intends to report some of the typical failures observed in Gujarat with a particular reference to Ratiya dam failure. In addition, some of the model tests performed on earthen model embankments using shaking table have been presented. The observations from the field investigation and model testing have been supported using finite element formulation. This paper also intends to propose a convenient method of retrofitting an existing vulnerable embankment by providing dense zone on either side of the embankment in the subsoil portion to the required depth.

Earthquake shaking in Bhuj during January 2001

India has suffered many major earthquakes over the years which caused huge casualties. The most glaring event was that which occurred on the Republic day of the country on 26th January 2001. A 7.7 M_w earthquake shook the western province of the country at around 8.46 a.m. local time on the black Friday. It was a time when all school children were celebrating the national day at their schools and the parents were enjoying the celebrations. The impact of earthquake was such that most of the buildings in about 50 km radius of the epicenter were reduced to rubble resulting in more than 20,000 deaths which included many children. The epicenter of the earthquake had a focal depth of 18 km and was located at 70 km east of Bhuj near Ludai along southern margin of the Banni plains at 23.40° (Latitude) North, 70.34° East (Longitude). This earthquake created an impact on the general public and the policy makers as well in addition to engineers resulting in increased research on various aspects of earthquake engineering. Unfortunately it was not possible to accurately assess the ground shaking near the epicenter due to lack of instrumentation. As strong motion acceleration records were not available to the authors near the dam sites, an attempt was made to scale peak ground acceleration (PGA) from the far field considering the fact that weak acceleration records may not depict actual motion in the near field. Though the attenuation relationships for strong motions of the order experienced around Bhuj during the Bhuj earthquake of 2001 are non linear, an initial linear approximation was made to estimate the peak ground acceleration (PGA). The nearest accelerograph recording that was available to the authors was in the passport office of Ahmedabad city, 250 km away from the epicenter. The North South acceleration component of Ahmedabad data was considered and its peak acceleration was modified using Campbell attenuation relationship (Campbell, 1981)

$$\ln PHA (g) = -4.141 + 0.868 \times M - 1.09 \ln [R + 0.0606 \times e^{0.7M}] \quad (1)$$

Here, R is the epicentral distance in km and M is the moment magnitude of earthquake on Richter scale. For an epicentral distance of 50 km, the calculated a_{max} was 0.14g and for an epicentral distance of 250 km, the calculated a_{max} was 0.03g.

However, the recorded peak acceleration at 250 km from epicenter was 0.106g (A_{max}), which is about 4.66 times the calculated a_{max} . Thus, the peak acceleration near epicenter where many embankments have failed can be approximately obtained by increasing the calculated peak acceleration by the same proportion, which yields 0.493g. The corresponding ground motion is shown in Figure 1.

$$A_{max (estimated)} = \frac{a_{max @ 50 km}}{a_{max @ R=250 km}} \times A_{max @ R=250 km}$$

where, A_{max} is measured/ estimated peak acceleration

a_{max} is peak acceleration determined using Campbell attenuation relationship.

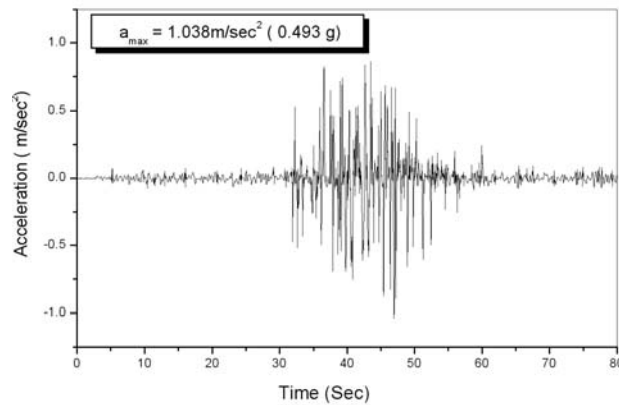


Figure 1. The North South acceleration component of Ahmedabad data modified using Campbell attenuation relationship and scaled to a peak acceleration of 0.493 g

FIELD INVESTIGATION

Kutch area is a dry tropical, flat and arid region with summer temperature as high as 48° C. Though there are not many major rivers, small streams are available in this region. As agriculture is one of the major occupation of the region, the government has provided with many small irrigation schemes that help cultivation. Hence this region encompasses around 185 earthdams of height ranging from 10 m to 25 m. All provisions are made to save precious water for both drinking purpose and cultivation. However, drought during the last couple of years made most of the reservoirs empty just before the earthquake. This act of nature appears to be the blessing in disguise as the severity of damage was many folds reduced due to the non existence of water. The first two authors along with Prof. Ikuo Towhata and Er. Tsuyoshi Honda from the University of Tokyo in Japan visited the damaged sites barely three months after the earthquake. Field investigation was carried out at four sites namely; Ratiya dam, Adhoi dam, Chang dam and Rudramata dam. The investigation included field reconnaissance survey, discussions with locals and field tests using Swedish weight sounding equipment. Swedish weight sounding is one of the field tests that works on the principle of penetration resistance of soil used to estimate the strength and deformation characteristics of soil. The biggest advantage of this technique is the ease in handling the equipment. In this paper, the results of investigations at Ratiya dam are discussed.

Ratiya dam

Ratiya dam is located near the village Ratiya about 10 km west of Bhuj. It is an earthdam, 10.6 m in height, 881 m long and constructed in 1948. The typical cross section of the dam is shown in Figure 2. Top width was maintained at 3 m and upstream and downstream slopes provided with pitching were sloped at 1 in 2 and 1 in 1.75 respectively. Inner core consisted of masonry zone, about 0.6 m top width with 6 in 1 slope. It is believed that inner core was surrounded by less pervious soil and the outer shell was made of strong and pervious mass. The catchment area was almost dry and the water in upstream was at dead storage level. Cattle and sheep were drinking water from here at the time the team visited the site. The schematic view of the dam is shown in Figure 3. At the two ends basaltic outcrop was visible and the upstream was very dry in these regions. The embankment was intact and slopes and pitching had maintained their position.

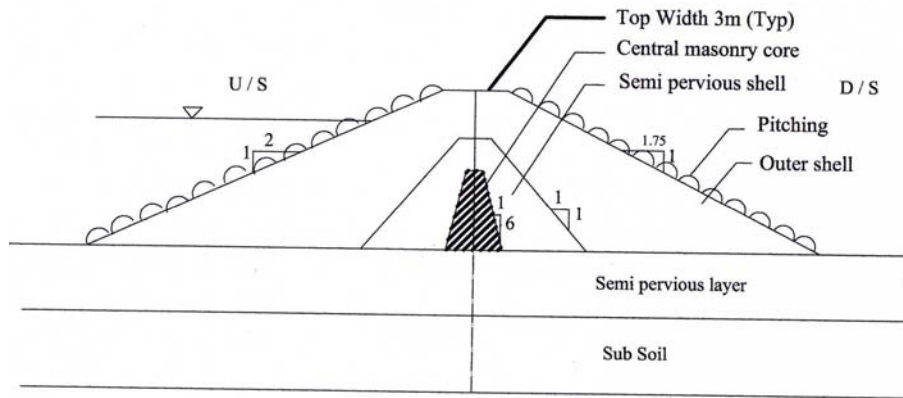


Figure 2. Typical cross section of Ratiya dam

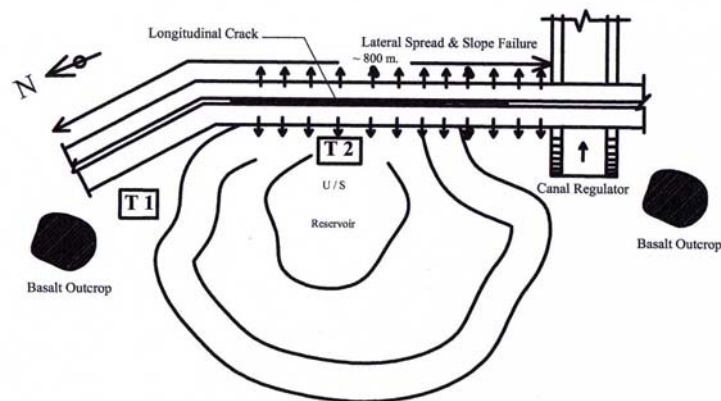


Figure 3. Schematic view of Ratiya dam site

However, in the middle portion, for a distance of about 400 m, longitudinal cracks all along the length were visible. These cracks extended vertically for a depth of about 2 m, almost extending up to the masonry core. The slopes had suffered lateral movement and it appeared that the slip was more pronounced on the upstream. The stones on the pitching were totally disturbed. The embankment showed subsidence of the order of 1 to 2 m. The sluice and canal regulator made out of masonry had suffered serious damage.



Figure 4. Longitudinal Crack along the length of Ratiya dam



Figure 5. Upstream slope failure at canal regulator of Ratiya dam

Figure 4 shows the longitudinal crack of width of around 150 mm observed at the crest of Ratiya dam. Figure 5 presents the damage suffered on the upstream at the canal regulator. The disturbed pitching stones indicate slope failure and lateral movement. Swedish weight soundings were performed at two locations T1 and T2 both near the upstream toe of the dam. T1 was in the undamaged portion free

from surface water and T2 was in the severely damaged central reservoir portion where water at dead storage level was available.

Figure 6 shows the Swedish weight soundings taken at the intact (T1) and damaged (T2) sites near the upstream toe of Ratiya dam. The graph consists of two parts. First part is the plot between relative weight (W_{sw}) along horizontal axis and depth of sounding along the vertical axis. Relative weight ranged from 0 to 1. When the soundings were taken with complete weight of 1 kN attached, relative weight was 1. The second part is the graph between number of half rotations for 1 m penetration of Swedish device (N_{sw}) and depth of penetration. It can be seen that T1 was performed only for a depth of around 1.2 m after which the penetration rod broke. It was difficult to further penetrate suggesting strong resistance. However, T2 penetrated immediately after full weights were placed for a depth of 2.2 m without any rotation being provided. After about 4.4 m of penetration the sounding was stopped as relatively strong frictional soil was encountered.

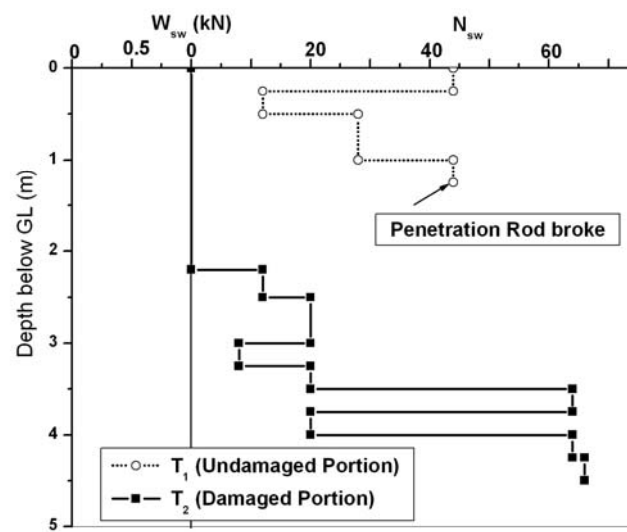


Figure 6. Swedish weight soundings at Ratiya dam site

Also, the horizontal acceleration is commonly used to describe ground motion because of its natural relationships to inertia forces. The peak horizontal acceleration near the damaged dams was estimated to be around 0.5 g. Since the strong motion part of earthquake was for at least 25 seconds and the thickness of weak subsoil was sufficiently large, the shear strain experienced by ground was sufficiently large. Laboratory tests have shown that soil stiffness is influenced by cyclic strain amplitude, void ratio, effective stress and number of loading cycles. The shear modulus of ground will be high at low strain amplitudes, but decreases as the strain amplitude increases. Also it is well established that the cyclic shear stress decreases as the number of cycles of loading increases (Okamoto, 1984).

The investigation revealed that the subsoil in the damaged regions had been weak during the earthquake. This subsoil was strong enough to support the static forces including the hydrostatic forces on the reservoir side. However, strong shaking during the earthquake was responsible in creating a very large strain level and degradation in both strength and stiffness of the material of the subsoil. As the subsoil below the Ratiya dam was inferred to be clayey in nature, cyclic loading must have created high amplification resulting in increased strain level. Inertial forces and perhaps the increase in the excess pore water pressure must have resulted in higher degradation.

It was therefore inferred that subsoil weakening was the major cause of failure of the earthdam. Similar situations were seen in most of the other dams in the region. All these dams were built immediately after the Indian independence in 1947. During that period, even geotechnical engineering was in infant stage in India and there was no design requirement from earthquake point of view. Hence, not much emphasis was given to strengthen the subsoil from earthquake shaking. Most of the dams being situated on weak subsoil need strengthening from earthquake effects. Since it is very difficult to strengthen the entire subsoil, focus can be given to strengthen only a small zone. This will also enable retrofitting the existing structure.

MODEL TESTS

Model testing in earthquake geotechnical engineering is vital because of the complexities and uncertainties involved in this field. Such tests help in understanding the performance of structures during earthquake and provide important information for possible mitigation measure. In model testing, usually the boundary conditions, material type and geometry are reproduced in a small-scale model to understand full scale prototype behavior. Model tests help in understanding the performance of a prototype and to study the effects of different parameters and more importantly one can directly observe the behavior and the process leading to failure of the model at a real time. Model test on the performance of embankments during shaking have already been presented elsewhere (Chandradhara et.al., 2002) and it was inferred that subsoil weakening was the most vulnerable aspect of embankment failure. Weak subsoil resulted in the development of longitudinal cracks at the initial stage and continued slump of embankment in addition to slope failure. A couple of cycles of shaking developed the complete collapse of embankment particularly when the subsoil was weak.

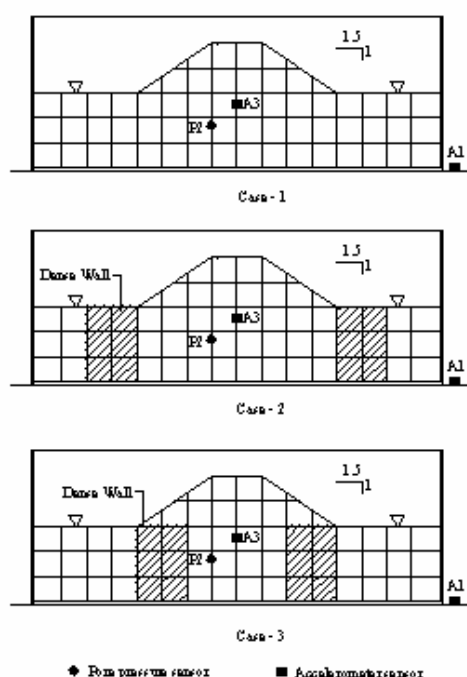


Figure 7. Schematic figures of Model Test setup

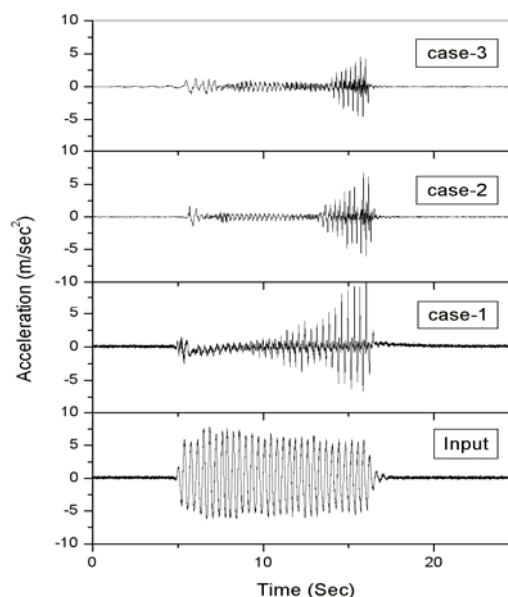


Figure 8. Acceleration time histories of Input (A1) and response (A3) in all the cases

In the present work, model tests were carried out using manual shaking table and acrylic container of size 500 mm X 1650 mm in plan and 600 mm in depth. The model test was performed using sub-angular, fine, uniformly graded locally available sand (Chndradhara et al., 2002). The model consisted of 200 mm high embankment having 200 mm top width and 1 in 1.5 side slope and resting on 300 mm

thick subsoil. The subsoil was completely saturated to represent most vulnerable condition. The shaking table was subjected to horizontal unidirectional shaking of acceleration of around 0.5g with a frequency of about 2 Hz. The instrumentation included pore water pressure sensors and accelerometers that were placed in the model ground at salient locations. The colored sand was used to form grids which helped to understand the deformation experienced at different locations in the embankment and subsoil during shaking.

In the present paper, the responses of one accelerometer (A3) and one pore water pressure sensor (P2) are explained. The emphasis of this paper is to compare the performance of benchmark embankment (case-1) with those having dense zone as mitigation measure against seismic failure. For this purpose, two types of dense zone formations in subsoil were attempted, namely, dense zone in the subsoil on the outside of the toe (case-2) and dense zone in subsoil on the inside of the toe (case-3) as shown in Figure 7. Dense zone included 200 mm thick zone throughout the height of subsoil. The densification was made by tamping. Dense zone had an average relative density of around 70 % and loose region had an average relative density of around 15 %.

Figure 8 presents the typical input (A1) and responses in three different cases through time histories of acceleration recorded from accelerometer placed 50 mm in to the subsoil (A3). It can be seen that subsoil showed softening behavior as sharp peaks are not visible in the time history. It may be observed that the magnitude of acceleration was extremely small in the initial period compared to the input acceleration in all the three cases. However, after some shaking, the acceleration level gradually increased suggesting the gain in stiffness. This behavior was more pronounced in case-1 compared to those of other two. It can be inferred that the subsoil immediately softened due to shaking and gain in stiffness was due to the settlement and densification. Excess pore water pressure variation with time in the subsoil (P2) is presented in Figure 9. The interesting point is that the level of excess pore water pressure was equal to that of initial effective stress in case-1, while it was only 90% and 80% respectively in case-2 and case-3. It can therefore be inferred that the subsoil experienced complete liquefaction in case-1 and lesser degree of liquefaction in cases 2 and 3. Subsoil had absolutely no bearing capacity in case-1 and relatively higher bearing capacity in the other two cases. This resulted in the embankment experiencing settlement and the material of embankment punching into the subsoil in case-1. In case-2 and case-3, embankment experienced reasonable resistance to this action.

The dense zones in case-2 and case-3 were very helpful in reducing total spreading. They provided stability for the embankment to retain its position. It was inferred that the dense zone below the embankment acted as supporting columns, where as dense zone on the outer face acted as an earth pressure zone. The tested counter measures were found to reduce embankment settlement and lateral deformations.

It was however feared that the performance of dense zone was to some extent reduced because of

- excess pore water from soft region entering dense zone and reducing the effective stress in dense zone.
- the loss of stiffness of neighboring soft region reducing the confining effect there by causing lateral instability of dense zone.

In order to avoid the lateral instability, the dense zone portion was confined by wrapping geo-textile around. The performance of the embankment in the enclosed dense zone was further improved. The deformation pattern of the model embankment with and without mitigation measure is illustrated in Figure 10 at three seconds after the shaking started. It is clear from the figure that the embankment slumped considerably when there was no mitigation measure. When the dense zone was provided, there was sufficient delay in the settlement of embankment.

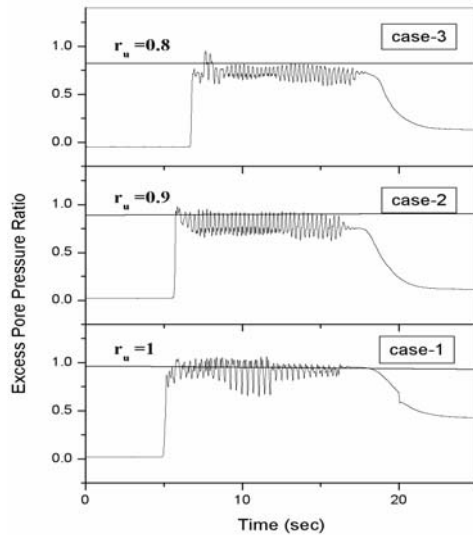


Figure 9. Time histories of Excess pore water pressure (P2) in all the cases

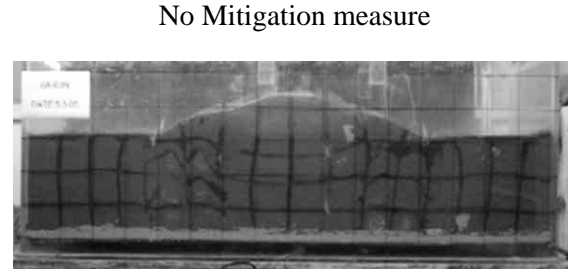


Figure 10. Deformation pattern of Model Embankment with and without mitigation measures at three second after shaking started.

FINITE ELEMENT ANALYSIS OF EMBANKMENT SUBSOIL SYSTEM

Finite element method has been well established as a versatile numerical tool to handle complex geotechnical problems. Attempt has been made to model an embankment and subsoil system and to show the usefulness of providing dense zone in the subsoil. In the present study, the embankment models were analyzed using GEO-SLOPE (1992), a Finite element software which can be used for both dynamic and static analysis of earth structures.

In order to apply the finite element concepts to a real geotechnical problem, certain assumptions and idealizations must be made. In particular, it is necessary to specify soil behavior in the form of a mathematical constitutive relationship. Due to the special geometric characteristics of an embankment, plane strain idealization has been made for the finite element modeling. One of the important requirements in finite element modeling is to progress from simple to complex. When all the possible complexities are included at the start of an analysis, it often becomes difficult to interpret the results, particularly when the results are unrealistic. Moving from the simple to the complex solutions makes it easier to pinpoint the reasons for unrealistic results. It is therefore a good practice to first define a simplified version of the problem and then add complexity in stages. Thus a good method of moving from a simple analysis to a complex analysis is to start with a linear-elastic model. These results provide a reference with which more complex nonlinear results are compared. Also, it is preferred over complete non linear analysis because of the following reasons.

- Uncertainty in the in-situ soil behavior, especially under dynamic conditions.
- Incomplete information of seismic load.
- Use of simplified mathematical models.
- Error involved in numerical modeling due to simplification.

In the present analysis, linear-elastic analysis using GEO-SLOPE was carried out in order to understand the dynamic behavior of embankment. Also, the objective was only to identify the regions of stress concentration and locations of excessive displacements.

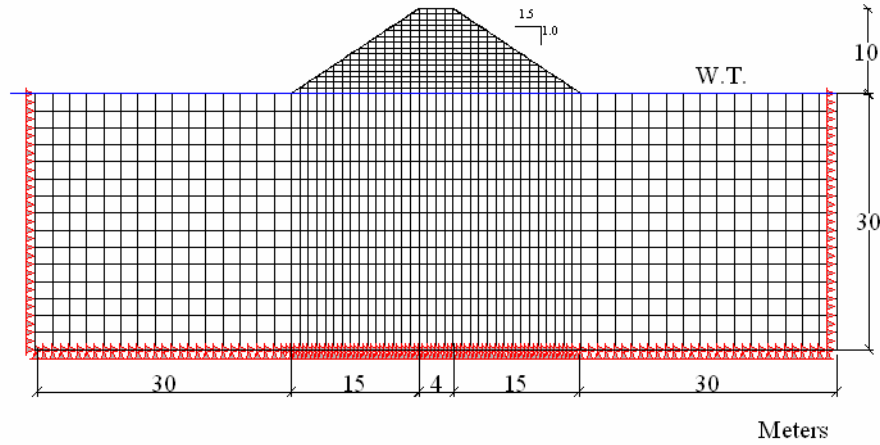


Figure 11. Finite Element Mesh and Dimensions of Embankment and Subsoil

The geometry of a typical embankment and subsoil considered for the finite element analysis and its discretization are shown in Figure 11. In order to represent the infinite boundary conditions, parametric study was made earlier to determine the finite depth and length of free field. The crest width of the embankment model was 4 m and height was 10 m with a side slope of 1 in 1.5. The subsoil was 30 m thick below which the presence of hard rock was idealized and free fields on both sides were maintained at 30 m each so as to depict the behavior of infinite boundary. The dense zone which extends for the full depth of subsoil was 4 m thick ($0.4 H$, where H is the height of embankment) on either side of the embankment. The aspect ratio of elements was maintained between 1 and 2 to achieve the best results. Quadrilateral and triangular elements with secondary nodes were chosen for the best performance. The optimum number of 1260 elements and 3909 nodes were chosen after convergence study.

In 2-D dynamic analysis under earthquake loading, it was idealized that the elastic continuum on top is subjected to base acceleration of given ground motion. Since the analysis was carried out considering one directional motion, the boundaries were free to move in the horizontal direction and restrained to move in the vertical direction.

The same three model configurations, one without any mitigation measure (benchmark model, Case-1) and the other two with mitigation measures were considered. In Case-2, dense zone was provided outside the toe of the embankment, while in Case-3 dense zone was provided inside the toe of the embankment. In all the cases, the water table was specified at the top of subsoil.

Table 1. Material properties used for the finite element analysis

| S.No | Property | Embankment and Subsoil | Dense wall |
|------|-----------------------|------------------------|-----------------------|
| 1 | Initial shear modulus | 7.7×10^3 kPa | 8.7×10^6 kPa |
| 2 | Unit weight | 14 kN/m ³ | 21 kN/m ³ |
| 3 | Poisson's ratio | 0.3 | 0.15 |
| 4 | Damping ratio | 10 % | 10 % |

The material properties used for the finite element analysis are shown in Table 1. The modified North South acceleration component of Ahmedabad data shown in Figure 1 was used as input in the analysis.

Results of Finite Element Analysis

In the present investigation, the objective was to identify the regions of stress concentration, locations of excessive displacements and to evaluate the effectiveness of mitigation measures. Thus the analytical results are presented with the purpose of clarifying effects of dense zone in the subsoil as a mitigation measure.

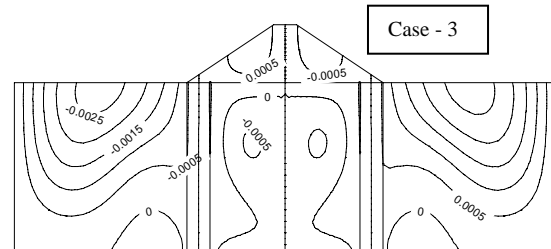
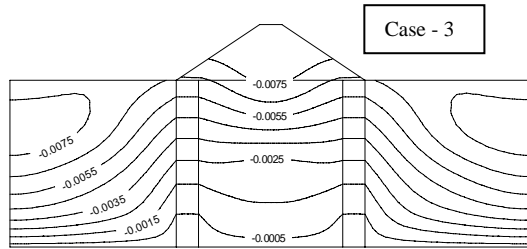
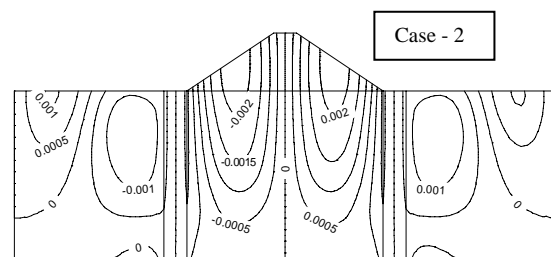
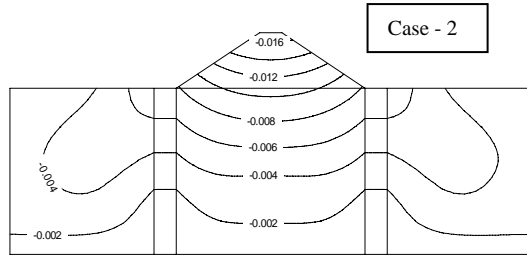
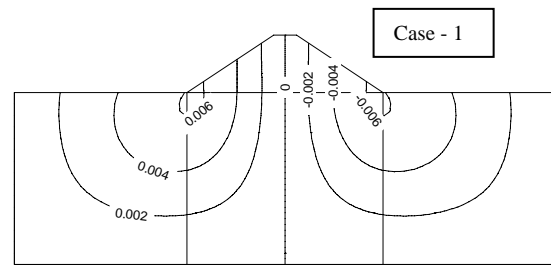
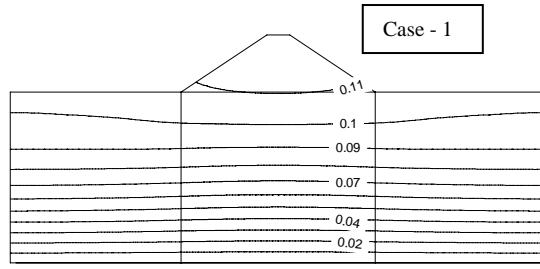


Figure 12. Contours of Horizontal Displacement at 46.96 sec. in all cases

Figure 13. Contours of Vertical Displacement at 46.96 sec. in all cases

Since the peak input acceleration occurred at 46.96 sec., the contours of displacements and stresses have been plotted at this time. Figure 12 and Figure 13 illustrate a set of typical results from finite element analysis to compare the earthquake induced deformations with and without mitigation measures. From the contours of horizontal displacement, it is seen that the displacement in Case-1 at the base of the embankment is 15 times more than that of displacement in Case-2 and 9 times more than that of displacement in Case-3. In the subsoil portion, the horizontal displacement reduced by 10 times, when mitigation measures were provided. Vertical displacement is maximum in Case-1 near the toe, indicating the necessity of the dense wall near the toe of the embankment. Further, in the other cases, the contour values show pockets of zero displacement at a number of regions. This substantiates the necessity of dense wall in the subsoil as a mitigation measure.

Figure 14 shows the shear stress contours at the same time. Though the magnitude of shear stresses increase near the vicinity of dense zone in mitigation models, the dense wall substantially reduces the shear stress in the subsoil away from the embankment and the subsoil is almost stress free in case-2. In Case-1, the shear stresses are more at the subsoil and in the region away from the embankment. Thus by providing dense zone near the toe, the shear stresses in the subsoil, especially below the embankment are reduced. However, shear stresses in the body of embankment are slightly increased, especially at the edges. From the shaking table tests it was found that the subsoil away from the embankment liquefied first compared to the subsoil under the embankment when there was no mitigation measure (Chandradhara et al., 2002). Thus, for the given cyclic resistance of the soil, if the cyclic shear stress required to initiate liquefaction reduces, the consequence of liquefaction can be reduced. This phenomenon clearly suggests that the dense zone near the toe of the embankment reduces the possibility of liquefaction and acts as a suitable mitigation measure.

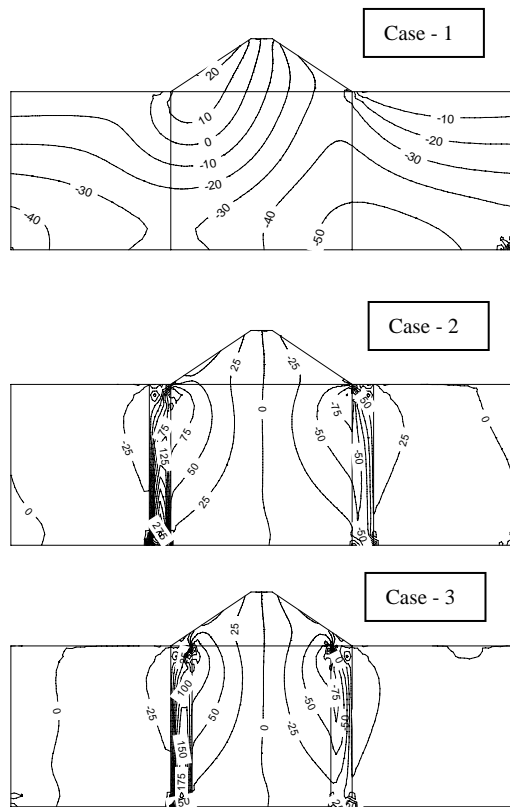


Figure 14. Contours of Shear stress at 46.96 sec. in all the cases

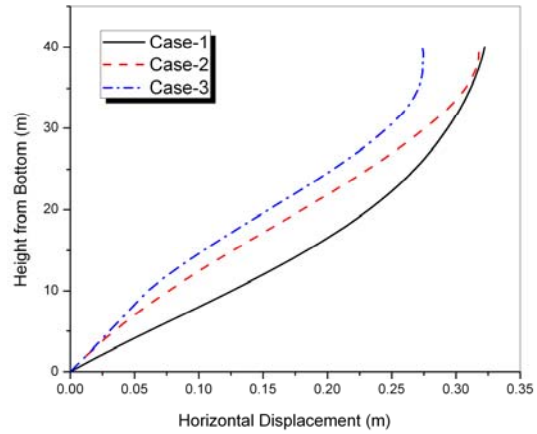


Figure 15. Peak Horizontal displacement along the center line of the Embankment

The variations of peak horizontal displacement along the vertical center line of the embankment and subsoil in all the cases are shown in Figure 15. The peak displacement increases with movement towards the top of the embankment in all the cases. However, when dense wall was placed inside the toe of the embankment (Case-3), the peak displacement was less throughout the height when compared to the peak displacement in other cases. The variation in horizontal displacement from Case-1 to Case-3 is considerable in the subsoil portion compared to that in embankment portion. This indicated that the provision of dense zone in subsoil enormously reduces the horizontal displacement in subsoil which in turn reduces the strain levels. Thus the possibility of settlement of embankment due to increase in horizontal displacement in the subsoil can be controlled by providing dense wall in the subsoil.

CONCLUDING REMARKS

Weak subsoil is the most common cause for damage to earthen embankments in many instances. Field investigation and laboratory model tests substantiated this fact. The estimated horizontal peak acceleration at the locations of failed embankment 50 km from the epicenter of Bhuj earthquake of 2001 was found to be 0.493g. The severity of the damage in model test was assessed by the rate of deformation, pore pressure and acceleration recordings, amount of collapse and the final deformed shape of the embankment. The present paper has focused on establishing the mechanism of failure of embankments due to subsoil weakening and the concept of mitigation measure by the provision of dense zone. Finite element analysis showed that both horizontal and vertical displacements in embankment and subsoil reduced when the mitigation measure were adopted. Dense zone on the inside of the toe showed a clear advantage of improving the existing embankment structure compared

to that on the outside. This fact was substantiated both from model testing and finite element idealization.

ACKNOWLEDGEMENTS

The authors wish to acknowledge with thanks Prof. Towhata and Er. Honda of University of Tokyo, Japan for all the technical support during the field visit in Bhuj, India. The financial assistance was provided by the Japanese Society of Soil Mechanics and Foundation Engineering.

REFERENCES

- Adalier, K., Elgamal, A. W., and Martin, G. R., "Foundation Liquefaction Countermeasures for Earth Embankments", *Journal of Geotechnical and Geo-environmental Engineering*, Vol:124, No. 6, pp. 500-517, 1998.
- Campbell, K. W., "Near source attenuation of peak horizontal acceleration", *Bulletin of the Seismological Society of America*, No. 71, pp 2039-2070, 1981
- Chandradhara, G. P., Prasad, S. K., and Nanjundaswamy, P., "Seismic response of model embankments from shaking table tests", *Proc. of 12th symposium on Earthquake Engineering at I.I.T., Roorkee*, Vol 1., Phoenix Publishing House Pvt. Ltd., New Delhi, 2002.
- Earthquake Spectra, "2001 Bhuj, India earthquake Reconnaissance report", *EERI Publication No 2002-01*, 2002
- Geo-Slope, Users manual, Geo-Slope Office, Calgary, Alberta, Canada, 1992.
- Kramer, S. L., "*Geotechnical Earthquake Engineering*", Pearson Education Pvt. Ltd., Singapore, 1996.
- Koga, Y. and Matsuo, O., "Shaking table tests of embankment resting on liquefiable sandy ground", *Soils and Foundations*, Vol. 30, No.4., pp 162-174, 1990.
- Okamoto, S., "*Introduction to Earthquake Engineering*", University of Tokyo press, 1984.
- Prasad, S. K., "Evaluation of deformation characteristics of 1-G model ground during shaking in a laminar box", *Ph.D. thesis submitted to University of Tokyo*, Japan, 1996.
- Prasad, S. K., Chandradhara, G. P. and Vijayendra. K. V., "Seismic Ground Response at Ahmedabad during the Gujarat Earthquake of 2001", *Indian Geotechnical Conference*, Indore, India, 2001
- Prasad, S. K., Towhata, I., Chandradhara, G. P. and Honda, T., "Field investigation of earthen dams failure during the Gujarat earthquake", *Proc. of 12th symposium on Earthquake Engineering at I.I.T., Roorkee*, Vol 1, Phoenix Publishing House Pvt. Ltd., New Delhi, 2002.
- Prakasha, H. N., "Dynamic response of embankments", *M.Tech. Dissertation submitted to Visvesvaraya Technological University*, Belgaum, Karnataka, India, 2005.
- Towhata, I., Prasad, S. K., Honda, T and Chandradhara, G. P., "Geotechnical Reconnaissance study on damage caused by 2001 Gujarat earthquake, India", *Soils and Foundations*, Vol. 42, No. 4, pp 77-88, 2002.