Design Procedure for the Seismic Analysis of Earth Structures

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SUMMARY A simple procedure is presented for the seismic analysis of earth structures such as dams, tailings dams and embankments. The procedure represents an improvement on the pseudostatic analyses commonly used.

1. INTRODUCTION

A procedure is presented for the seismic analysis of earth structures such as dams, tailings dams and embankments. The procedure is based upon the state of practice used in California, and although by no means a rigorous analysis, represents an improvement over the psuedostatic analysis commonly used in Australia.

An outline of the procedure is presented below.

1. Select the design seismic event.
2. Check whether any of the materials in the structure are subject to loss of strength under cyclic loading.
3. Check whether any of the materials in the structure are subject to liquefaction.
4. If materials are neither prone to significant strength loss (say greater than 20%) nor liquefaction then their probable performance may be assessed by estimating embankment deformations during the design event.
5. If materials are prone to liquefaction under the design earthquake loading then they are not suitable if there is any possibility of the materials becoming saturated.
6. For materials which exhibit significant strength loss under cyclic loading conditions it probably follows that deformations will be excessively high.

The notation used in this paper is summarised in Table I.

2. DESIGN SEISMIC EVENT

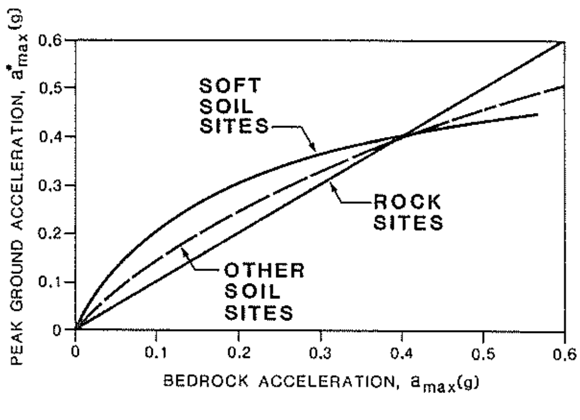
The proposed design method requires two parameters relating to the design seismic event. These parameters are:

- the maximum Richter Magnitude (ML_{max}); and
- the maximum ground acceleration at the site.

Australian design values for these parameters may be obtained from References (1) and (2). The first reference gives the estimated maximum Richter Magnitude (ML_{max}) for various zones of Australia, together with estimated data on maximum ground accelerations for an event with a 10% chance of exceedance in 50 years (equivalent to an annual probability of exceedance of one in five hundred). The second reference presents the estimated maximum ground accelerations for an event with a 10% chance of exceedance in 100 years (equivalent to an annual probability of exceedance of one in one thousand). We recommend that an annual probability of exceedance of 1:500 is used for structures with short operating or saturated lives, such as tailings dams, and an annual probability of exceedance of 1:1000 for structures with long design lives such as railway embankments and water supply dams.

The above maximum ground accelerations relate to the bedrock accelerations (a_{max}) experienced during the design seismic event. Soil deposits overlying the bedrock, in general, would tend to amplify the bedrock motion to some degree. In order to be suitably conservative in design, it is recommended that the bedrock accelerations are adjusted in accordance with the design curves shown on Figure 1 (Ref 3) to estimate the peak ground acceleration (a_{max}^*).

To assess the potential of a material to liquefy under seismic loading the average cyclic shear stress (τ_{av}) due to the seismic loading must be estimated. In addition, if laboratory test methods are used to assess liquefaction potential, an appropriate number of τ_{av} stress cycles must be estimated. These parameters may be estimated by the methods described below.



NOTE: SOFT SOILS ARE TYPICALLY NORMALLY CONSOLIDATED MATERIALS WITH UNDRAINED SHEAR STRENGTHS LESS THAN 50kPa

Figure 1 Relationship between peak ground acceleration and bedrock acceleration

The average cyclic shear stress, τ_{av} in a soil deposit during the design seismic event can be determined from the following formula by Seed (4).

$$\frac{\tau_{av}}{\sigma'} = 0.65 \frac{\sigma}{\sigma'} r_d \frac{a_{max}^*}{g} \quad (1)$$

where σ and σ' are the total and effective overburden stresses respectively, g is the acceleration due to gravity, a_{max}^* is the peak particle acceleration at the ground surface and r_d is a stress reduction factor which reduces from unity at the ground surface to about 0.9 at a depth of 10m and 0.5 at a depth of 30m.

The number of τ_{av} stress cycles needed to represent the design seismic event depends on the earthquake magnitude. Appropriate numbers of stress cycles according to Seed (4) are presented as follows:

Richter Earthquake Magnitude	Number of Significant Stress Cycles
5.5	5
6.5	10
7.0	15
7.5	25
8.0	35

3. STRENGTH LOSS, LIQUEFACTION AND CYCLIC MOBILITY UNDER CYCLIC LOADING

3.1 General

The material to be utilised in constructing the earth structure should be checked for strength loss, liquefaction and cyclic mobility when subjected to the level of cyclic loading likely to be experienced in the event of the design earthquake occurring. Generally, cohesive materials and dense granular materials do not suffer significant strength loss in such situations; on the other hand loose, saturated sands and sensitive cohesive soils can experience considerable strength loss due to cyclic loading.

Assessing materials for their susceptibility to liquefaction and cyclic mobility may be achieved using a number of simplified procedures based on field test correlations, e.g. using the standard penetration test, SPT (Ref 5), and the cone penetration test, CPT (Ref 6). Alternatively, a programme of laboratory testing may be undertaken to examine the liquefaction potential of the material.

3.2 Field Test Correlations

On the basis of correlations between field observations of soil liquefaction and N-values measured with the SPT, Seed et al. (5) developed liquefaction resistance curves for sands with different N-values and fines contents. Similar correlations were used by Robertson and Campanella (6) to develop a method for the liquefaction assessment of sand and silts based on the CPT. Both of these methods relate the SPT N-value or the CPT cone resistance value, q_c, to the likely soil liquefaction resistance. The soil liquefaction resistance is expressed as the cyclic shear stress ratio which causes liquefaction or cyclic mobility during an earthquake of a given magnitude. The soil is considered not to be susceptible to liquefaction or cyclic mobility if its liquefaction resistance exceeds the field cyclic shear stress ratio due to the design seismic event (expression (1), Section 2). If liquefaction or cyclic mobility is possible, then the maximum value of shear strain likely to be mobilised in the soil may be estimated from Figure 2.

3.3 Laboratory Test Assessment

Because insitu data will not normally be available at the design stage, a laboratory assessment of field liquefaction potential is necessary. The field liquefaction potential of the embankment material can be assessed from the results of laboratory cyclic shear testing of samples performed at voids ratio, moisture content, pore pressure and loading representative of field conditions. Appropriate testing of materials can be carried out in triaxial or simple shear apparatus with a facility to apply cyclic loading. The level of cyclic loading representative of the design seismic event can be determined using the expression for cyclic stress ratio (τ_{av} / σ') given in Section 2.

The above cyclic shear stress ratio relates to horizontal ground shaking which is the dominant form of earthquake loading. Such conditions can reasonably be simulated in a simple shear apparatus. In a triaxial shear apparatus, however, the applied cyclic shear stress ratio needs to be adjusted to account for the change from horizontal shear conditions during earthquake loading to the isotropic confining conditions of the triaxial test set-up. The adjustment in cyclic shear stress ratio for the testing of normally consolidated materials is given by the following expression:

Triaxial Cyclic Shear Stress Ratio		Earthquake or Simple Shear Cyclic Shear Stress Ratio
$\frac{q'_{cyc}}{2\sigma_c'}$	=	$1.6 \frac{\tau_{av}}{\sigma'}$ (2)

where q'_{cyc} is the applied cyclic deviator stress

and σ_c' is the initial mean effective confining pressure.

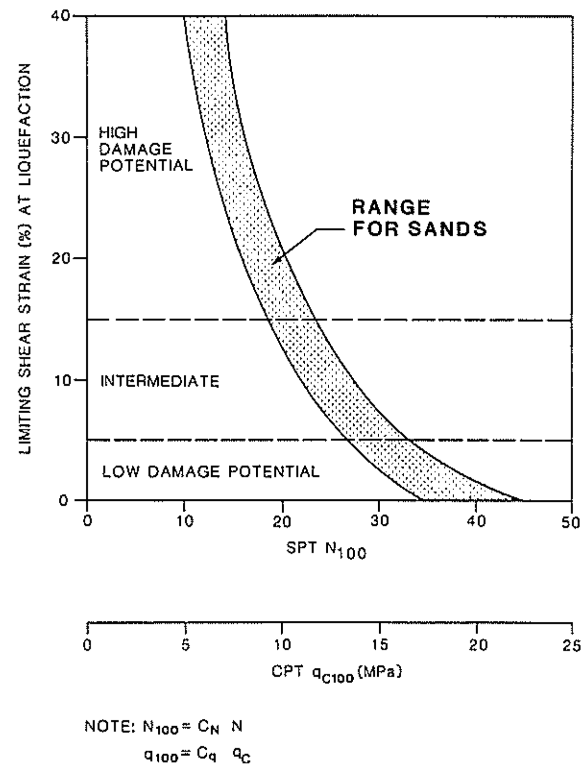


Figure 2 Limiting shear strain at liquefaction

4. ESTIMATING EARTHQUAKE-INDUCED DEFORMATIONS IN EARTH STRUCTURES

If the materials in an earth structure are neither prone to significant strength loss (say greater than 20%) nor liquefaction then their probable performance during the design seismic event may be checked by estimating earthquake-induced deformations in the structure. Makdisi and Seed (7, 8) proposed a simplified procedure for estimating such deformations in dams and embankments constructed of cohesive materials. This procedure is summarised on Figures 3, 4 and 5.

For dams and embankments constructed of granular materials, earthquake induced deformations may be estimated using a relationship proposed by Bureau et al (10) between the earthquake severity index (ESI), the embankment material friction angle and the crest settlement. This relationship is presented on Figure 6.

5. ACCEPTANCE CRITERIA

The methods described above result in an estimate of the measure of performance of a structure under a particular earthquake loading. When assessing the acceptance of the estimated deformation of a structure a number of factors must be taken into consideration, these factors include:

- o the type of structure, e.g. railway embankment, water retaining dam, tailings dam;
- o the consequence of failure, e.g. downstream effects of spills, potential for loss of life, cost of repair; and
- o the probability of the design event occurring during the life of the structure.

Because of these factors there is no simple set of design criteria that may be followed; rather a judgment must be made which will take the above factors into account.

Two of the most important factors to be judged in determining acceptable displacements are:

- o the amount of displacement necessary for a breach to occur in the dam (or in the case of a railway embankment sufficient track displacement for a derailment to occur); and
- o the amount of displacement available for the embankment material to remain in its elastic range of stress-strain behaviour.

The former of these factors must be dealt with on a case by case basis. The second factor may be assessed by the examination of the stress-strain relationships of the embankment materials. Typically these data would be available from triaxial testing conducted at stress levels appropriate to the embankment configuration. Because of the non-linear stress-strain relationship of many materials the adopted acceptable strain should, in order to preserve a degree of conservatism, preferably be less than the strain to 50% of the peak strength.

As a guide to the probable displacements which may take place without disastrous effects, we have reviewed published data concerning the effects of earthquakes on earth dams primarily in Chile, Mexico and California. The data considered apply to earthquakes of magnitude 5.9 or greater on the Richter scale and would therefore be considered large by Australian conditions. The data concerned a total of 14 dams affected by 24 earthquake events (7) (10) (11) (12) (13) (14) (15). Five of the dams

1. CALCULATE FIRST NATURAL PERIOD OF EMBANKMENT, T_0

   T_0 = Function of Embankment Height and Material Type (see Figure 4 for typical values of T_0)

2. DETERMINE PEAK GROUND ACCELERATION, a^*_{max} FOR EARTHQUAKE OF MAGNITUDE, M

   Refer to Section 2 for design values of a^*_{max}.

3. FOR A GIVEN CIRCULAR SLIP SURFACE WITHIN THE EMBANKMENT, CALCULATE YIELD ACCELERATION, k_y

   k_y = Lateral g force to give a factor of safety of unity in a pseudostatic slope stability analysis of the embankment assuming soil undrained shear strength reduced by 20%.

4. CALCULATE MAXIMUM AVERAGE ACCELERATION, k AT BASE OF CIRCULAR SLIP SURFACE (see Figure 5a)

5. CALCULATE EARTHQUAKE-INDUCED LATERAL MOVEMENT, U (see Figure 5b)

   U = Function of T_o, k_y, k and earthquake magnitude M

6. REPEAT STEPS 3 TO 5 FOR DIFFERENT CIRCULAR SLIP SURFACES WITHIN THE EMBANKMENT. QUOTE MAXIMUM U VALUE.

Figure 3 Procedure for estimating earthquake induced deformations in cohesive embankments

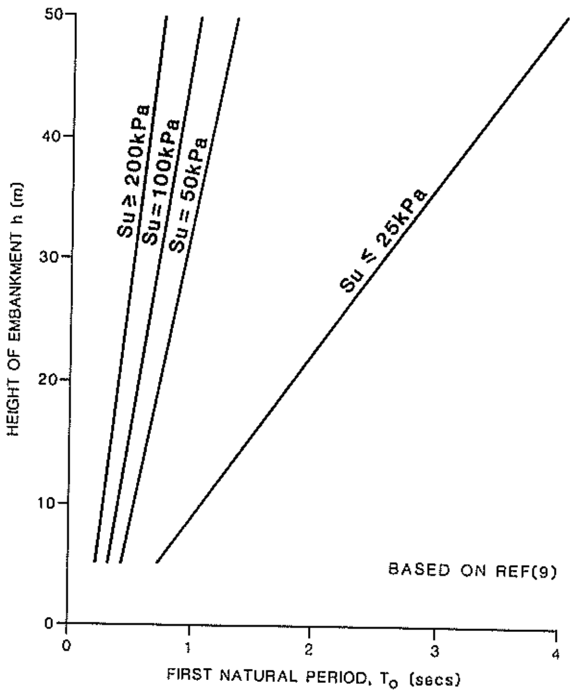


Figure 4 Values of first natural period of embankment

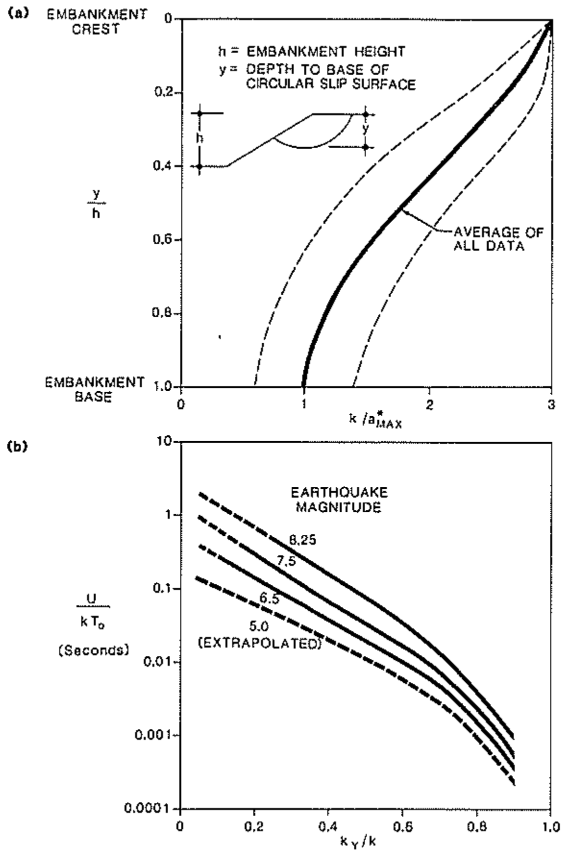
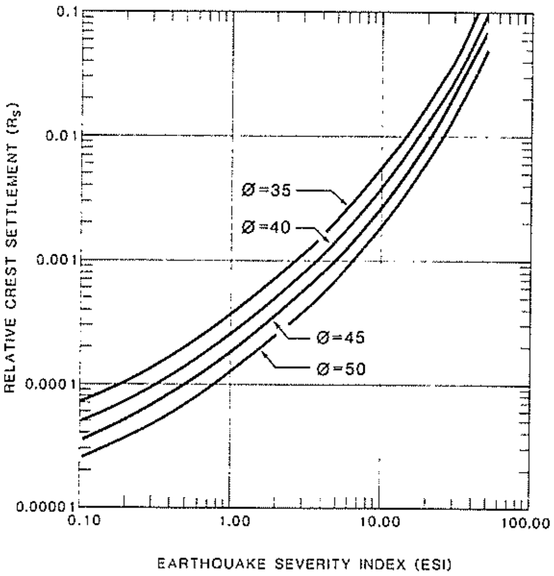


Figure 5 Earthquake induced deformations in cohesive embankments



where $ESI = \frac{a^*_{max}}{g} (ML_{max} - 4.5)^3$

and Crest settlement, $s = R_s h$

Figure 6 Procedure for estimating earthquake induced deformations in granular embankments

failed as a result of the seismic loading and experienced large vertical settlements of greater than 8% of their embankment height. The other 9 dams did not fail and experienced serviceable settlements of between 0.01% and 1.5% of the embankment height, with a median value of 0.24%. Perusal of the data and a review of the data source references suggest that embankments can suffer strains in the order of 1.5% without undue distress to their overall integrity. For dams with clay liners and seals we suggest that the tolerable estimated displacement should be limited to 10% of the thickness of the liner/seal, and for dams with plastic membrane liners we suggest that the tolerable estimated displacement should be limited to about 50mm.

This may be summarised:

Type of Structure	Suggested Tolerable Estimated Displacement *
Homogeneous Embankment	1.5% of overall height
Dam with Clay Liner	1.5% of overall height or 10% of liner thickness
Dam with Plastic Membrane	50mm

* Tolerable displacements may be limited by other factors such as consequential damage (e.g. breaches, derailments), or the strain level to limit deformation in the elastic range.

Based on our review of the data cited above and other work described by Seed et al. (16), it is considered that acceptance of the above criteria represents a low probability of seismic failure for the structure. A low failure probability, however, may not equate to an acceptable failure risk, and hence it is important to assess the failure probability in relation to the consequence of failure of the structure.

This may be carried out by the method proposed by Vick et al. (17) summarised on Figure 7 and the relative risk shown on Figure 8. The probability calculations on Figure 7 permit the lifetime probability of seismic failure to be evaluated. Figure 8 gives an indication of the observed risk of failure for various earth structures, and therefore provides some guideline as to what constitutes an acceptable failure risk. By way of example, the probability that an earth structure will fail under the design ground motion for an estimated displacement of 1.5% of the structure height may be less than 5% say, i.e. $p(f/a)=0.05$. Let there be a 10% chance of occurrence or exceedance of this ground motion over the operational life of the dam, i.e. $p(a_i)=0.10$ (refer to Section 2). On this basis, the lifetime probability of failure, $p(f)$ is 0.005, which is within the range of acceptable values for dams presented on Figure 8.

6. CONCLUSIONS

A procedure is presented for the seismic analysis of earth structures such as dams, tailings dams and embankments. The procedure is an improvement over a simple pseudostatic analysis and gives due consideration to the design seismic event, the design ground acceleration, the potential for liquefaction or cyclic mobility of the soil, and a performance assessment of the structure on the basis of acceptable earthquake-induced deformations.

1. CALCULATE PROBABILITY OF GROUND MOTION OCCURRENCE OR EXCEEDANCE DURING DESIGN LIFE

   $p(a_i) = 1 - (1 - p(a_o))^i$

   where:

   $p(a_i)$ = probability of occurrence or exceedance of design ground motion in i years

   $p(a_o)$ = annual probability of occurrence or exceedance of design ground motion

   i = life of impoundment (operational or saturated life)

2. CALCULATE LIFETIME PROBABILITY OF SEISMIC FAILURE

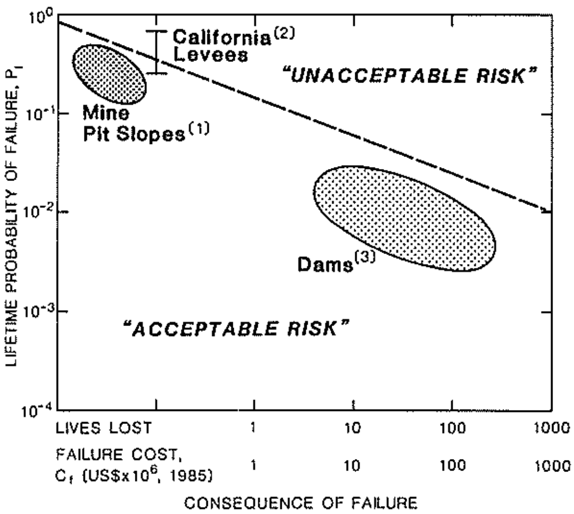
   $p(f) = p(a_i)\, p(f/a)$

   where:

   $p(f)$ = lifetime probability of seismic failure

   $p(f/a)$ = probability that structure will fail under design motion

Figure 7 Estimation of probability of seismic failure



(1) BASED ON 10 YEAR LIFE
(2) BASED ON 40 YEAR LIFE
(3) BASED ON 100 YEAR LIFE

Figure 8 "Accepted" risk - dams and mining-related projects

TABLE I

NOTATION

Symbol	Units	Definition
a		Bedrock acceleration at epicentre
a_{max}		Bedrock acceleration at site
a_{max}^*		Acceleration at natural ground surface
C_N		Depth correction factor for N value
C_q		Depth correction factor for cone resistance
ESI		Earthquake severity index
g	(m/s^2)	Acceleration due to gravity
h	(m)	Embankment height
i	(yrs)	Life of impoundment (operational or saturated life)
k		Acceleration within embankment at a given level
k_y		Yield acceleration of embankment at base of failure surface
M		Earthquake magnitude - Richter scale
ML_{max}		Maximum Richter Magnitude
N		SPT 'N' value
N_{100}		Factored N value
$p(a_i)$		Probability of occurrence or exceedance of design ground motion in i years
$p(a_o)$		Annual probability of occurrence or exceedance of design ground motion
$p(f)$		Probability of seismic failure
$p(f/a)$		Probability that structure will fail under design ground motion
q_c	(MPa)	Cone resistance
q_{c100}		Factored cone resistance
q'_{cyc}	(kPa)	Applied cyclic deviator stress
r_d		Stress reduction factor
R_s		Relative crest settlement
s	(m)	Crest settlement
T_o	(sec)	First natural period of embankment
U	(m)	Earthquake induced lateral movement
y	(m)	Depth to base of circular slip surface
$\tau_{av}liq$	(kPa)	Cyclic shear stress to cause liquefaction
τ_{av}	(kPa)	Average cyclic shear stress
σ	(kPa)	Total overburden stress
σ'_c	(kPa)	Initial mean effective confirming pressure
σ'	(kPa)	Effective overburden stress
σ'_{vo}	(kPa)	Effective overburden stress
ϕ	(degrees)	Friction angle of embankment material

Note: Accelerations expressed as a fraction of g.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

1. Gaull, B.A., Michael-Leiba, M.O. and Rynn, J.M.W. "Probabilistic Earthquake Risk Maps of Australia". Australian Journal of Earth Sciences, Vol. 37, No. 2, June 1990, pp.169-187.

2. DR91094:S-1991. "Minimum Design Loads on Structures, Part 4:Earthquake Loads", to be AS1170.4, Standards Association of Australia, 1991.

3. Idriss, I.M. "Response of Soft Soil Sites During Earthquakes". Proceedings H.B. Seed Memorial Symposium, UC Berkeley Calif., Vol. 2, 1991, pp.273-289.

4. Seed, H.B. "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes", Journal of Geotech. Engng Div., ASCE, Vol.105, No.GT2, 1979, pp.201-255.

5. Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M. "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations", Journal of Geotech. Engng Div., ASCE, Vol.111, No.GT12, 1985, pp.1425-1445.

6. Robertson, P.K. and Campanella R.G. "Liquefaction Potential of Sands Using the CPT", Journal of Geotech. Engng Div., ASCE, Vol.111, No.GT3, 1985, pp.384-403.

7. Makdisi, F.I. and Seed, H.B. "A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments", Earthquake Engineering Research Centre, College of Engineering, UC Berkeley Calif., USA, Report No.UCB/EERC-77/19, August 1977.

8. Makdisi, F.I. and Seed, H.B. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations", Journal of Geotech. Engng Div., ASCE, Vol.104, No.GT7, 1978, pp.849-867.

9. AS2121-1979. "The Design of Earthquake-Resistant Buildings, SAA Earthquake Code", Standards Association of Australia, 1979.

10. Bureau, G., Volpe, R.L., Roth, W.A. and Udaka, T. "Seismic Analysis of Concrete Face Rockfill Dams", closure, Journal of Geotech. Engng Div., ASCE, Vol. 113, No. 10, October 1987, pp.1255-1264.

11. Bureau, G., Bubbitt, D.H., Biachoff, J.A., Volpe, R.L. and Tepel, R.E. "Effects on Dams of the Loma Prieta Earthquake of 17 October 1989", USCOLD Newsletter Issue No. 90, November 1989.

12. Seed, Dickenson, Riemer, Bray, Sito, Mitchell, Idriss, Kayen, Knopp, Harden and Power. "Preliminary Report on the Principal Geotechnical Aspects of the October 17, 1989 Loma Prieta Earthquake". Earthquake Engineering Research Centre, UC Berkeley Calif., USA. Report No. UCB/EERC-90/05, April 1990.

13. De Alba, P.A., Seed, H.B., Retamal, E. and Seed, R.B. "Analysis of Dam Failures in 1985 Chilean Earthquake", Journal of Geotech. Engng Div., ASCE, Vol.114, No.GT12, December 1988.

14. Castro, G., Poulos, S.J. and Leathers, F.D. "Re-examination of Slide of Lower San Fernando Dam". Journal of Geotech. Engng Div., ASCE, Vol.111, No.9, September 1985.

15. Renner, P., Salazar, A. and Wellmann, P. "Tailings Disposal in Seismic Zones". Tailings Disposal Today, Volume 2, Proceedings of the Second International Tailings Symposium, Denver, USA, May 1978.

16. Seed, H.B., Makdisi, F.I. and de Alba, P. "Performance of Earth Dams during Earthquakes". Journal of Geotech. Engng Div., ASCE, Vol. 104, No. GT7, July 1978, pp.967-994.

17. Vick, S.G., Atkinson, G.M. and Wilmot, C.I. "Risk Analysis for Seismic Design of Tailings Dams". Journal of Geotech. Engng Div., ASCE, Vol. 111, No. 7, July 1985, pp.916-933.