Apr 2nd - Apr 7th

Discussions and Replies — Session VI

Multiple Authors

DISCUSSIONS AND REPLIES SESSION VI

Seismic Response of Embankment Dams
L. Caldeira, P. Seco e Pinto and J. Bile Serra,
LNEC, Lisbon, Portugal. Paper No. 6.01
by
Hendra Jitno, Bandung Institute of Technology,
10 Ganesha St., Bandung, Indonesia 40132

The authors have presented an interesting study on the seismic behaviour of earth dams. The procedure used by the author was total stress approach, similar to that proposed by Seed et al. (1973). There are a few points which need clarification.

1. The authors did not explain whether the static analyses were carried out using an incremental procedure, or using a 'gravity turn-on' analyses. These two methods will give different pattern of dam deformation at the end of construction. Alternatively, the authors can present the deformation of the dam as a function of the dam height, instead of just maximum values listed in Table 1.
2. The total stress dynamic analyses used by the authors suffers some limitations that it tends to overestimate the effective stress of soil elements in the dam, particularly when significant pore pressure develop during earthquake shaking. Accelerations of 4.2 m/s2 (about 0.43g) and 4.61 m/sec2 (about 0.47g) used in the study are sufficiently strong to produce a non-linear behaviour of soil under earthquake shaking. This may cause some pore pressure increase in saturated granular materials. If significant pore pressures develop, the actual response of the dam will be softer than predicted. As a result, the total stress approach will overestimate the predominant frequency of the dam, but underestimate the dam deformations. Have the authors validated the approach against the actual behaviour of the dam to see how close the measured and the predicted predominant frequency or deformations of the dam ?

Reference :
Seed, H.B., Lee, K.L., Idriss, I.M. , and Makdisi, F. (1973)."Analysis of the Slides in the San Fernando Dams During the Earthquake of February 9, 1971,"

Discussion on paper titled: "Seismic Response of Embankment Dams", by L. Caldeira, P. Seco e Pinto, and J. Bilé Serra, (Paper No. 6.01)

By: Robert K. Green, Woodward-Clyde Consultants, 500 12th Street, Suite 500, Oakland, California 94607.

This paper presents state-of-the-practice seismic stability analyses of three embankment dams. The calculated displacements from the static analyses were not compared with measurements or performance of the dams during construction and first filling.

The importance of the seismic loading is pointed out by the authors. Use of ground motions corresponding to a return period of about 1,000 years is somewhat less conservative than the 3,000 to 10,000 return period recommended by USCOLD (1985), and use of more recent ground motion attenuation relationships would also be recommended for future such analyses.

The amplification of the motions at the crest of the dam show values and trends consistent with other analyses. The lower amplification at higher ground motions levels for a given dam is likely due to the non-linear effects, primarily the increased damping.

Reference: United States Committee on Large Dams, 1985, Guidelines for selecting seismic parameters for dam projects: publication prepared for National Science Foundation, Grant No. CEE-8218049, by Committee on Earthquakes, 39 p., October.

Discussion on Paper 6.02, titled: "Mine Tailing Deposition Practices, Liquefaction Potential and Stability Implications," by: Hallman and Dorey

Discussion by: Professor Laura Caldeira and Joao P. Bile Serra, Lab. Nacional de Engenharia Civil, Dept. de Geotecnia, Av. do Brasil 101, 1799 Lisboa Codex, Portugal

The authors in the first part discuss the typical mine tailing characteristics, the method of deposition and tailing dam construction techniques as well as their implications in terms of liquefaction potential. In the second part describe an interesting case history.

Some questions arise from reading the paper.

- In seismic areas, when dams are to be constructed by the upstream or centerline method, it is common to separate the coarse fraction of the tailings from the slimes by use of cyclones. The relative density of the sand sized tailings has an important influence on the potential for liquefaction. The authors have some experience with this technique and the soil parameters (e.g., relative density) achieved through its use?

- In the case history described the finite element computer program TARA-3FL was used to estimate the post-liquefaction deformation of the dam. Prediction of the soil properties of tailings requires a different approach to that for normal soils, because of the large strains involved. Were there any "in situ" or laboratory tests made to estimate the soil deformation properties?

Discussion on paper titled: "Karameh Earth Dam, A Challenging Project," by A. I. Husein et al. (Paper No. 6.06).

By: James P. Lee, Senior Consultant, Brown & Root.

Table 2 provides the design soil parameters for effective stress and total stress analyses. It is noted that cohesion values are included in the effective stress parameters. A good laboratory test should show no cohesion value for effective stress parameters (CU-bar or CD). Were these effective stress cohesion values neglected in the slope stability analyses? US Army Corps of Engineers' manual for stability of earth and rock-fill dams requires that the cohesion value be neglected in the lower normal stress portion for analyses under the sudden drawdown and the steady state loading conditions. It is thought that the apparent cohesion value may be due to the presence of negative pore pressures in the embankment soils.

Discussion on paper titled: "Gravel Liquefaction Analysis of an Embankment Dam," by D. A. Vessely et al. (Paper No. 6.07).

By: James P. Lee, Senior Consultant, Brown & Root.

This paper indicates that a bedrock (BR) motion was first determined, which is an earthquake (controlling crustal event) with a magnitude of 6.5 and an 84th percentile peak acceleration of 0.36g. The free-field (FF) motion at the ground surface, 0.41g, was then estimated utilizing the program SHAKE. The FF motion was input into the finite element (FE) program FLUSH for the dynamic analysis. It is pleasing to know that this program allows the direct input of a FF motion, instead of the motion at the base of the FE model which was required in previous practice. However, it is surprising that the response motion at the crest of the dam, 0.4g, is slightly less than the FF motion (0.41g). Furthermore, figures 2 and 3 appear to indicate that the base of the FE model is at the bedrock interface. Then why was the BR motion not directly applied in the FE analysis, instead of the FF motion which was derived from the BR motion since errors may be introduced in the process.

Discussion on paper titled: "Gravel Liquefaction Analysis of an Embankment Dam", by D. Andrew Vessely and Nan Deng, Paper No. 6.07.

By: V.S.Pillai, Geotechnical Department, B.C.Hydro, Burnaby, B.C., Canada,V3N 4X8

The Authors have presented a case study where they have carried out a site investigation and performed liquefaction analysis using Seed's approach which is considered highly empirical. The dam embankment is an important facility for the city of Portland. They have carried out a triggering (liquefaction) analysis and proceeded to carry out post-earthquake slope stability analysis using parameters and generic correlations which could mislead the real performance of the dam. The authors claim that the work is comprehensive, while it lacks the detail in the application and the choice of right parameters, the analyses and presenting the results.

Liquefaction is a complex phenomenon and is more governed by site-specific factors such as the density, confining stress, initial static bias and the loading mode (compression, extension or simple shear). The filter zone which is considered liquefiable appears to consist of sands and gravels (D_{50} = 0.25"). The authors indicate that a review of construction records did not give any indication as to the relative density of the filter material and had to proceed with a 4-drill hole investigation at two locations to obtain Becker blow counts. These were compared with highly empirical correlations to obtain an " average" $(N_1)_{60}$ value of 27. It appears that the filter zone is of considerable thickness (~ 12 ft ?) and occupies a considerable area under the downstream shell below the tailwater. Does this $(N_1)_{60}$ value represents entire filter zone? The low end $(N_1)_{60}$ values or its scatter were not provided in the paper. Secondly, the Harder and Seed (1986) correlation to convert the Becker blows to SPT $(N_1)_{60}$ values are subjective (Sy.A and Campanella, R.G. 1993) and can be misleading.

With respect to potential for liquefaction, the most critical portion of this dam is the zone of sand and gravel filter under the shallow depth near the downstream toe. In this portion, the filter is subjected to very low confining stresses, extension loading mode and of course large static bias values (~0.2). Under extension loading mode, the sand and gravel filter may exhibit contractive behaviour which could lead to significant loss in liquefaction resistance particularly with large initial static bias (Pillai and Stewart, 1994). (Initial static bias, on the other hand would increase liquefaction resistance under dilative condition (compression loading) which may be the case under the main body of the dam.). Again in the downstream portion, for the given design earthquake, the estimate of the induced cyclic stress ratio (CSR) appears to be very significant (CSR > 0.3). Because of the extension loading condition and the large initial static bias in the filter in the downstream portion, it is possible that the cyclic resistance ratio (CRR) [cyclic strength] of the filter to drop significantly and may not be adequate enough (CRR < 0.3) to prevent the material from liquefying during the design earthquake?. These aspects were not assessed.

In conclusion, the Discusser believes that the paper lacks detail, the investigation and the analyses do not reflect the state-of-the-art knowledge in the cyclic behaviour of granular materials and their variability in properties during different loading modes. Therefore the conclusions drawn may be misleading.

References:

1. Pillai, V.S. and Stewart, R.A. (1994) "Evaluation of Liquefaction Potential of Foundation Soils at Duncan Dam", Canadian Geotechnical Journal, Vol. 31, December, 1994.

2. Sy, A and Campanella, R.G., (1993) "BPT-SPT Correlations with Consideration of Casing Friction", 46th Canadian Geotechnical Conference, Saskatoon, Saskatchewan, 401-411.

Discussion on Paper 6.08, titled:
"Elasto-plastic Seismic Response Analysis of Earth Dams," by: Yiagos, A.N.

Discussion by: Professor Laura Caldeira and Joao P. Bile Serra, Lab. Nacional de Engenharia Civil, Dept. de Geotecnia, Av. do Brasil 101, 1799 Lisboa Codex, Portugal

The numerical method presented in this paper is very powerful and simple enough to be considered a powerful tool for dam designers in preliminary studies. It has all the major aspects of dynamic soil behaviour, including the two-phase porous medium formulation, with the possibility of pore pressure build up evaluation, and the hysteretic constitutive model taking into account the damping of the solid skeleton.

Nevertheless, I would like to raise a few questions:

- If one wants to apply the model to a zoned dam, what are the equivalent properties that should be used to simulate the presence of the core cohesive material and the granular shell material at the same layer?

- In order to properly compare the computed and the observed accelerations it is very useful to use Density power spectral functions or Fourier Transform functions. Could the author present this type of graphics for both horizontal and vertical direction in the Santa Felicia and Long Valley Dams?

- At the Long Valley Dam, the author said that a cohesionless material was assumed at the three top stress points. What are the properties of this type of material? The same as the material described only with the zero cohesion value?

- It would be interesting to know about the possibility of introducing in the program boundary conditions with a more complex definition.

Report No EERC 73-2, University of California, Berkeley, Calif., NTIS No PB 223 402, 150 pp.

Evaluation of Earthquake-Induced Slope Displacements
S. Salah-Mars, R.K. Green, H. Kanakari, P.J. Boddie, L.H. Mejia, and K.D. Weaver. Paper No. 6.10.
by
Hendra Jitno, Bandung Institute of Technology,
10 Ganesha St., Bandung, Indonesia 40132

The authors evaluated earthquake-induced deformations of a slope using three different methods : two-dimensional non-linear finite element (FE) dynamic analyses, simplified Newmark and simplified Makdisi-Seed methods. The slope considered has a distinct failure surface and the soil at this surface does not seem to generate significant pore pressure during the earthquake. The authors calibrated the FE dynamic analyses against the recorded displacements of the slope due to the Mount Lewis earthquake. Based on the calibrated soil properties, the authors predicted the displacements of the slope due to the Morgan Hill and Loma Prieta earthquake which gave very good agreement with the measured values. Finally, the authors predicted the slope displacements due to design earthquake and the results were compared with those predicted using Newmark and Makdisi-Seed methods. The authors concluded that the results obtained using the FE approach gave the most reasonable estimate of the slope displacements. While the proposed methodology is very realistic, there are questions that need some answers :

1. Did the authors also calibrate the Newmark and Makdisi-Seed methods against observed displacements to obtain soil properties that give the best agreement with observation? Since the 'full' Newmark and Makdisi-Seed methods also use acceleration time histories to predict the slope displacements, would it be more fair if the authors also treat the other two methods in the same way as the FE approach? If the three methods are treated similarly, it will be interesting to see which one of these methods that gives the best results.

2. The more sophisticated FE dynamic analyses predicted the slope movement of 1-2 ft in comparison to 1-3 ft predicted by simplified Newmark method. These two methods gave essentially the same range of slope displacements. However, considering the simplicity of Newmark method as oppose to the more complicated and expensive FE method, would it be more economical to use Newmark method for this case? To the discusser, the comparative study presented in this paper only supports the view that the Newmark method is the best method for predicting this type of slope movement (i..e. that exhibits a distinct failure surface and involving soils that do not develop significant pore pressure rise due to earthquake shaking). Newmark's method is relatively simple and is capable of giving slope displacements that agree well with those predicted by the more sophisticated FE dynamic analyses.

Discussion on Paper 6.12, titled:
"Model Parametric Studies of the Earthquake Response of Embankment Dams," by: Law and Ko

Discussion by: Professor Laura Caldeira and Joao P. Bile Serra, Lab. Nacional de Engenharia Civil, Dept. de Geotecnia, Av. do Brasil 101, 1799 Lisboa Codex, Portugal

I would like to congratulate the authors for an excellent picture on the seismic behaviour of embankment dams. The analyzed variables are indeed key variables of the subject. The set of results constitutes by its own a reference for future work.

- There seems to be an apparent incoherency in the set of results. The densier (stiffer) material suffered a stronger amplification than the looser one. This clearly shows that the frequency distribution of the motion is more important on the higher frequency range. This merely reflects how close to the dam the epicenter of the recorded motion was. On the other hand the motion amplitude increasing has led to a larger amplification. This suggests that the amplification function amplitude has been enlarged on the higher frequencies. How does this match with the larger strain amplitude observed? How do the authors explain this situation? Is it possible that this has been caused by the uniform scaling of the motion in the frequency domain rather than a selective one on the higher frequencies?

- Which boundary conditions were imposed to the model? How important do the authors find the existence of non-reflecting boundary conditions during dynamic tests on centrifuge type of devices?

- Are there any plans to carry on with these tests with horizontal and vertical motions being imposed simultaneously?

Discussion on paper titled: "River Dike Failure by Earthquakes in 1993", by M. Kaneko et al., Paper No. 6.13.

By: V.S.Pillai, Geotechnical Department, B.C. Hydro, Burnaby, B.C., Canada, V3N 4X8.

The authors have documented failures of river dikes due to two major earthquakes that occurred in Hokkaido, Japan in 1993. The earthquakes were of a magnitude of M7.8 and were called the Kushiro-Oki Earthquake (15 January, 1993) and the Hokkaido Nansei-Oki Earthquake (12 July, 1993). The authors conclude that the two earthquakes have exhibited different patterns of liquefaction.

The paper concerns a very long dyke system along a river. The dykes were about 5 m high. Some dykes were built on loose sand and others on peat beds. The modes of deformation were illustrated well, however details are lacking particularly in the analytical studies. It appears that the authors have introduced some unconventional terminologies in the analysis. The authors have not provided any definition of the Liquefaction Resistance Factor (FL). The concept of the stress relaxation and its relationship to liquefaction of sand need to be clarified in terms of conventional soil mechanics. Despite the lack of detail, the Discusser believes that the paper provides some valuable information on liquefaction of river bank materials and failures of small dykes.

Discussion on paper titled: "River Dike Failure in Japan by Earthquakes in 1993", by M. Kaneko, J. Nihikawa, Y. Sasaki, M. Nagase, and K. Mamiya, (Paper No. 6.13)

By: Robert K. Green, Woodward-Clyde Consultants, 500 12th Street, Suite 500, Oakland, California 94607.

This paper emphasizes the importance of evaluating river levees for seismic stability by presenting some failures during large earthquakes. The damage to the levees was attributed to liquefaction, which in some cases was exacerbated by settlement of underlying peat layers.

The analytical studies of the liquefaction show that liquefaction would have been expected for the loose saturated sands. The analyses of the effects of the settlement on the state of stress in the embankments included parametric studies on the effects of thickness of peat, thickness of the dike, slope, and tensile strength of the dike. The authors adopted linear elastic properties for both compression and tension, which doesn't seem reasonable.

The authors conclude that materials just above the water table can become saturated and move into lower confining states if settlement occurs, and thus become vulnerable to liquefaction. They conclude that stress relaxation is a major factor increasing the vulnerability to liquefaction, and can be reduced by flatter slopes.

Dikes along rivers are often constructed over rather poor foundation soils that include loose sand and peat, both of which can increase the likelihood of failure during earthquakes. The seismic evaluation of levees including assessment of both native and fill materials is necessary to protect against such failures.

Discussion on Paper 6.14, titled:
"Backanalysis of Deformations for Case Histories Involving Flow-type Failures," by: Sully, Fernandez and Zalzman

Discussion by: Professor Laura Caldeira and Joao P. Bile Serra, Lab. Nacional de Engenharia Civil, Dept. de Geotecnia, Av. do Brasil 101, 1799 Lisboa Codex, Portugal

- Do the authors have any additional data on the acceleration level induced during the seismic survey?

- The displacement profiles observed in the field correspond to large deformation patterns. How reliable do the authors find a small deformation model, such as the hyperbolic-type, to back-analysis such patterns?

- Remedial measures design is said in the introduction to be the major goal of the study. What are the remedial measures suggested by the conclusions?

Discussion on paper titled: "Seismic Stability Analysis of a High Earth and Rockfill Dam", by N. Deng, F. Ostadan, I. Arango, and J. Marrone, (Paper No. 6.18)

By: Robert K. Green, Woodward-Clyde Consultants, 500 12th Street, Suite 500, Oakland, California 94607.

The authors are commended for an excellent summary of the state-of-the-practice procedures and results of a seismic stability analysis for an embankment dam. It is interesting to note that the procedures for the original 1969 seismic stability analysis followed most of the same procedures used for the current analysis. The much larger ground motion for the new analyses (peak acceleration of 0.5g instead of 0.08g) being the most striking difference. The predicted excellent performance of the dam to so much larger ground motions is a credit to the designers, and an example of the conservatism inherent in many well designed dams.

The comparison of the predicted deformations with measured crest movements of rockfill dams is a good way of evaluating the reasonableness of the prediction. None of the rockfill dams in that comparison have been subjected to ground motions with peak accelerations as large as the safety evaluation earthquake for this project. A similar comparison of the predicted response of the dam with the response of other dams would likely show similar levels of amplification (Harder, 1991).

Reference: Harder, L.F., Jr., 1991, Performance of earth dams during the Loma Prieta earthquake: Proceedings, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. 2, Paper No. LP05, pp. 1613-1629, March 11-15.

Discussion on Paper 6.18, titled:
"Seismic Stability Analysis of a High Earth and Rockfill Dam," by: Deng, Ostadan, Arango and Marrone

Discussion by: Professor Laura Caldeira and Joao P. Bile Serra, Lab. Nacional de Engenharia Civil, Dept. de Geotecnia, Av. do Brasil 101, 1799 Lisboa Codex, Portugal

The authors have made a careful re-evaluation of the safety of a very high dam built in 1960's. This is a key aspect of the geotechnical engineering practice and its importance grows every year as dams aging proceeds.

I would like to draw up a few questions in order to have a clearer picture of the work carried out:

- A large value of K_z was considered in the equation $1000K_b(\sigma'_m)^{1/2}$. It might happen that this value has been a key factor on the dynamic response obtained. What considerations led to the selection of this value?

- Is it likely that there is no pore pressure build-up in the lower third of the core during earthquake occurrence? Being this zone subjected to high mean effective stress do the authors expect dilatant behaviour to occur?

- Have the authors accounted the change in mechanical properties of the materials due to aging?

- Which G-γ curves were used to model the shell materials?

- How did the dam behave during the severe earthquakes that occurred in California since its construction?

- It would be interesting to analyze the dam under the action of the original time history in order to compare the safety factors that were previously considered with those achieved with today's concepts. What do the authors think about this subject?

Discussion on paper titled: "Behaviour and Damage of Dams Under the 1993 Big Earthquakes in Japan", by Iwashita, T and Nakamura, A, Paper No. 6.23.

By: V.S.Pillai, Geotechnical Department, B.C.Hydro, Burnaby, B.C., Canada, V3N 4X8.

For the two major earthquakes that occurred in Hokkaido, Japan in 1993, the authors have analyzed the maximum rock acceleration at the damsite and compared with the observed accelerations. The authors also documented damages to a few dams and concluded that recent dams that were constructed on rock foundations using modern design and construction technologies were more resistant to earthquakes as compared to old earthfill dams on loose riverbed.

The paper provides some glimpse of the current Specifications of Highways and Bridges of Japan (1990) to calculate a safety factor against liquefaction. This gives an indication of the Japanese approach for determining zones that have potential for liquefaction triggering. The formula appears to be somewhat empirical and conservative considering a large fine content (FC) of 40% as the separating factor of two levels of resistance. Normally this order of fines in sand material would inhibit any strain softening or liquefaction. However, for the Niwa-Ikumie Dam the authors have shown that the embankment soils (silty sand) at depths from 2m to 9m liquefied and this has caused some settlement in the upstream slope. It appears that the liquefaction was mainly dictated by the very low N-values. The authors could have enhanced the quality of the paper by providing more detail on the observations and investigations such as grain size distribution of the embankment and foundation soils and profiles of N-values.

Discussion on paper titled: "Behavior and Damage of Dams Under the 1993 Big Earthquakes in Japan", by T. Iwashita, A. Nakamura, and N. Yasuda, (Paper No. 6.23)

By: Robert K. Green, Woodward-Clyde Consultants, 500 12th Street, Suite 500, Oakland, California 94607.

The performance of dams during earthquakes is a true verification of design and analytical procedures, and the publication of the performance, both good and bad, is applauded.

The much slower attenuation of peak acceleration with distance from the 1993 Kushiro Oki earthquake is likely attributable to the very deep focal depth of the earthquake. This shows a limitation of attenuation relationships based on epicentral distance without accounting for the focal depth.

The damage to the concrete facing blocks of Mombetsu Dam due to settlement shows the problem of having rigid elements over more deformable soils. The two low permeability zones in the Niwa-Ikumine Dam which confined the loose sandy zone that liquefied during the earthquake would be expected to have larger deformations and lower post-earthquake factors of safety than a single central core because the generated pore pressures would dissipate more slowly. The good performance of the more modern dams and the poorer performance of older more poorly constructed dams shows the importance of performing seismic safety evaluations of existing dams.

Discussion on paper titled: "Some Aspects of Liquefaction Assessment of Duncan Dam:, by Pillai, V.S., Plewes, H.D. and Stewart, R.A. #6.25

By: David Hallman, Project Manager, Steffen Robertson and Kirsten (U.S.), Inc.

The authors are to be congratulated for their contribution to the conference. Their paper presents the results of what was obviously an extensive study and represents an excellent example of a phased investigation of liquefaction potential and seismic stability for an earth dam. The results from each of the two phases of the study are compared and illustrate an apparently large discrepancy between the methods of analysis adopted in each phases. The following comments are offered as cautionary remarks to those contemplating or evaluating similar studies, to encourage the authors to provide additional information on their interpretation of the studies, and to stimulate further research into liquefaction phenomena.

Having established that a soil is susceptible to liquefaction, one must then estimate the post-liquefaction undrained residual strength (S_{ur}) in order to assess the stability of a slope or dam. The authors devote a considerable portion of their paper to this aspect of the studies. From the results of the laboratory testing on undisturbed soil obtained from frozen soil samples, the authors conclude that the ratio of S_{ur} to the initial effective vertical stress (δ'_{vo}) is constant and equals 0.21. Yet their Figure 20, which presents the lab data, indicates that S_{ur}/δ'_{vo} ranges from approximately 0.13 to 0.32. However, Figure 22 which presents the lab data in a slightly different format, appears to substantiate the authors' S_{ur}/δ'_{vo} ratio. It is unclear as to what the difference in these two figures actually is.

Evaluation of the Duncan Dam data by Seed's empirical method resulted in residual strengths for the liquefied sand which are considerably lower than the laboratory based strengths. The authors attribute this discrepancy to the large confining stresses present in the layers of concern and place much greater confidence on the laboratory data. However, neglecting the empirical data obtained from back analysis of the actual performance of soil deposits during earthquake induced liquefaction case histories may not always be appropriate.

A failure path through a soil mass will preferentially pass through zones or layers of weakness and avoid or limit the extent of the path through stronger material. As all soil masses are variable in nature to some extent, this effect should be accounted for in any analysis procedure. Seed's empirical procedure for estimating S_{ur} from actual case histories inherently incorporates this effect and should result in a lower S_{ur} for an average soil property, such as void ratio or $(N_1)_{60}$ value, than an S_{ur} determined through laboratory testing on an average sample. This effect may explain Seed's (1987) observation that laboratory procedures for estimating S_{ur} often result in values which are significantly higher than those obtained through back analysis of actual case histories. Although Figures 6, 7, 8 and 12 illustrate that there are void ratio variations in the foundation at each depth, an average trend was apparently utilized to represent the foundation conditions with respect to the lab data interpretations. It is unclear which method of void ratio measurement was used for estimating the "true" average insitu void ratio and ultimately formed the basis for the S_{ur}/δ'_{vo} relationship presented as equation 3.

Ishihara (1994) states that the "steady state" line for a clean sand has a flat slope and that small changes in void ratio can have a large effect and make it difficult to accurately estimate S_{ur}. Marcuson, Hynes and Franklin (1990) report that void ratio changes of 0.05 are sufficient to change the S_{ur} by a factor of 3 or more. Thus the difficulty in estimating insitu void ratios and the variations inherent in any deposit may explain the large difference in S_{ur} estimated from the two methods of analysis presented by the authors. As actual field performance data is the only method by which geotechnical engineering evaluations can be accurately assessed, the empirical procedure advocated by Seed for determining S_{ur} should not be abandoned.

These issues illustrate that considerable engineering judgement is required in any residual strength estimation to fully appreciate the limitations of each of the methods of analysis. The comprehensive Duncan Dam studies presented by the authors contained significantly more extensive evaluations and analyses than could be summarized in a 14 page paper and would probably provide clarification to the concerns raised herein.

Due to the complexities and uncertainties involved, one should probably consider a range of residual strengths and conduct parametric studies to assess the sensitivity to changes in the residual strength. Ultimately, probablistic analyses incorporating the possible range of material properties (void ratios, N-values, etc.) and a range of residual strengths for each soil state may become the norm. Much additional research and investigation of post-liquefaction residual strengths is clearly needed.

References Cited

Ishihara, K. 1994; "Evaluation of Residual Strength of Sandy Soils". Thirteenth Int. Conference on Soil Mechanics and Foundation Engineering, New Delhi, India, January 5-10, Vol. 5, pp. 175-181.

Marcuson, W.F. III, Hynes, M.E. and Franklin, A.G., 1990; "Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments". Earthquake Spectra, Vol. 6, No. 3, pp. 529-572.

Seed, H.B., 1987; "Design Problems in Soil Liquefaction". ASCE Journal of Geotech. Eng. Vol. 113, No. 8. pp. 827-845.

Paper No. 6.02
Reply by David S. Hallman

We have considerable experience with the use cyclones to separate the coarse tailings fraction from the slimes and the subsequent use of the coarse sand fraction for dam construction. This practice is common with either upstream, centerline or downstream methods of construction and is typically adopted for construction material considerations. That is, the coarse tailings fraction represents a large supply of readily available material and can be much more cost effective to utilize for tailings dam construction than native soil or mine waste rock. This practice appears to be used equally in seismic and non-seismic areas.

In terms of the typical properties for cycloned tailings, the method of placement following cycloning controls the relative density of the material. If the coarse tailings fraction is deposited hydraulically following cycloning, a relative density on the order of 30 to 50 percent would be typical. On the other hand, the coarse fraction can be mechanically compacted to a relative density as high as 90 to 100 percent. Typically, the cycloned tailings sand is mechanically spread and lightly compacted to a relative density on the order of 60 to 70 percent.

The TARA-3FL finite element analyses mentioned in the case history were actually performed under the supervision of Prof. Finn at UBC, however, I do not believe that any specialized testing was conducted to define the soil deformation properties. To the best of my knowledge, the soil properties were based on typical soil tests such as gradation, Atterberg limits, SPT, CPT, consolidation and triaxial shear and comparison with similar materials.

Paper No. 6.06
Reply by: Husein (Malkawi) et al.

The writers appreciates the discussion by Mr. James P. Lee. We fully agree with him and in regard to his question about the cohesion values. Yes, in the analyses of slope stability the cohesion values were neglected.

Paper No. 6.08
Reply by Alexandros N. Yiagos
Geotechniki Ltd., Athens, Greece

• The model is not applicable to zoned dams and the author is not aware of a meaningful way of simulating the presence of the core cohesive material and the granular shell material at the same layer.

• The Fourier Amplitude Spectra of the computed and recorded absolute acceleration responses in the horizontal and vertical directions at the crests of the Santa Felicia and Long Valley Dams are given in the following along with the respective computed and recorded absolute accelerations.

• At the Long Valley Dam application, a cohesionless material is assumed at the three top stress points. The properties of this type of material are the same as the material described with the only exception of a zero cohesion value.

• The applied boundary conditions are those of the employed basis functions. Therefore it is possible to introduce more complicated boundary conditions by changing the set of basis functions appropriately. A possible extension of the program would be to take into account the longitudinal deformations of the dam by introducing a suitable set of basis functions.

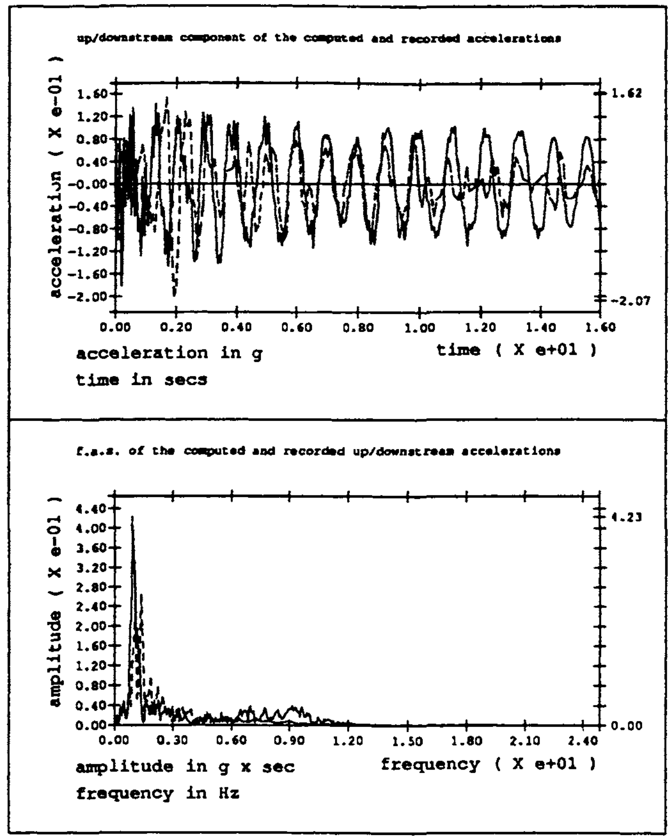

Fig. 1. Computed and Recorded Absolute Acceleration Responses of the Solid Phase in the Up/Downstream Direction at the Crest of the Santa Felicia Dam and the respective Fourier Amplitude Spectra (the computed response is plotted with a solid line)

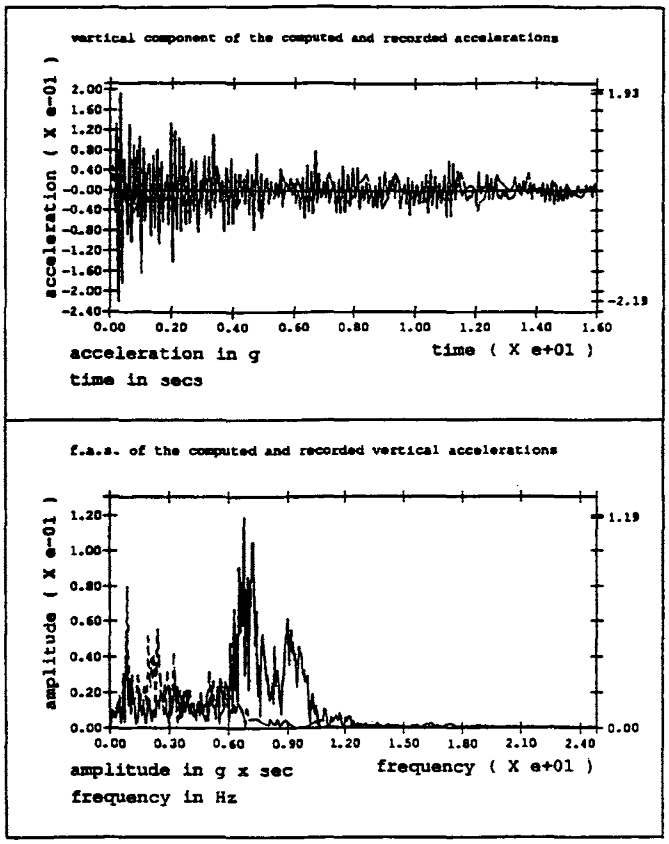

Fig. 2. Computed and Recorded Absolute Acceleration Responses of the Solid Phase in the Vertical Direction at the Crest of the Santa Felicia Dam and the respective Fourier Amplitude Spectra (the computed response is plotted with a solid line)

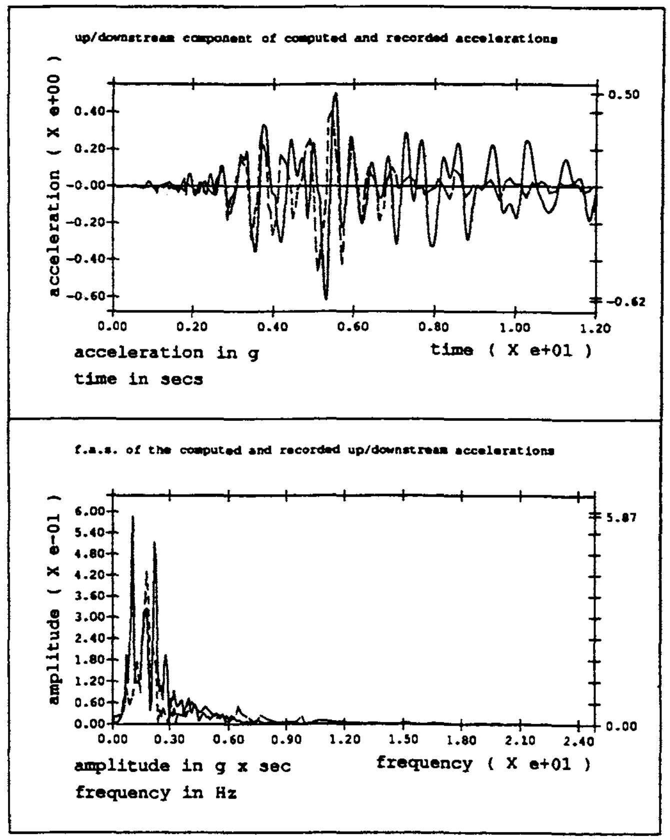

Fig. 3. Computed and Recorded Absolute Acceleration Responses of the Solid Phase in the Up/Downstream Direction at the Crest of the Long Valley Dam and the respective Fourier Amplitude Spectra (the computed response is plotted with a solid line)

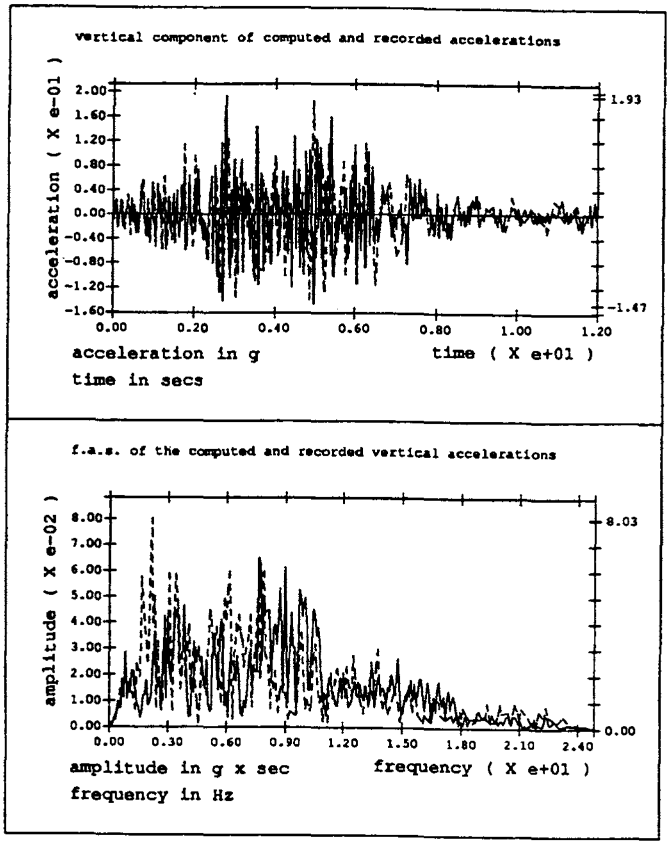

Fig. 4. Computed and Recorded Absolute Acceleration Responses of the Solid Phase in the Vertical Direction at the Crest of the Long Valley Dam and the respective Fourier Amplitude Spectra (the computed response is plotted with a solid line)

Paper No. 6.10

Reply by Dr. Said Salah-Mars, Senior Project Engineer
Woodward-Clyde Consultants, Oakland, California

The reviewer questioned the use of the F.E. approach for estimating slope formations and suggested that the Newmark method would have been enough.

I would like to inform the reviewer that within the project scope we were also interested by the slide mass strain-induced deformations. Furthermore, the model was also to characterize a smaller slide riding a larger slide. We had to limit the text of the paper extensively to satisfy the required format. Many interesting aspects of the study were not included in the paper.

Paper No. 6.12
Reply by Hubert K. Law and Hon-Yim Ko

We would like to appreciate for raising pertinent questions particularly regarding the behavior of amplification.

- We tried to examine the transfer functions at least between the motions recorded at the crest and the base. There seemed selective amplification at a certain frequency that could be the fundamental natural frequency of the structure. The natural frequency of the dense material was somewhat higher than the loose one. Nevertheless, the larger transfer function at the fundamental frequency in the dense soil, due to smaller damping, could lead to increase in the overall RMS acceleration. This is analogy to amplification for a single-degree-of-freedom system. If the material is linear elastic, the amplification factors would not be dependent on the level of earthquake excitation. However, if there is a decrease in shear modulus for large amplitude loading, large deformations are anticipated. In these centrifuge model tests, no strain measurement was made to verify the large strain behavior.

- The model embankment dam was contained in a rigid box. In most cases where the foundation was not included in the model, the effects from the container's ends were eliminated, because the shoulder of the dam did not touch the end boundaries of the container. In other cases, the foundation was in contact with the container ends; but no special treatment was made to the container to prevent wave reflection. It is not clear if the problem is serious here. However, we realized another problem due to the side boundaries of the container. Ideally, these boundaries should provide free friction and minimal deflection to maintain a plain strain condition. Since the upstream reservoir was to hold water, we abandoned the idea of lubricating the container's sides to avoid leakage of water along the dam-container interface. This might have violated the plain strain condition.

- A true two-dimensional shaking table is not currently available in a centrifuge environment. However, most shake tables, which are designed to deliver horizontal motions only, produce uncontrollable vertical motions. The model dams in this test program indeed experienced both horizontal and vertical motions together. The vertical accelerations were recorded only at the base and crest; but, they are not reported in the paper.

Reply to discussion by Dr. V.S. Pillai on paper titled: "Behavior and Damage of Dams Under the 1993 Big Earthquakes in Japan", by T. Iwashita, A. Nakamura and N. Yasuda, (Paper No. 6.23.)

We wish to thank Dr. Pillai and Dr. Green for their comments.

We apologize for in sufficiency of detail on the investigations for Niwa-Ikumine Dam. We supply more detail information on the drilling inspection and soil tests for Niwa-Ikumine Dam. Figure 1 shows the distribution of N-value by the drilling inspection at the dam body of upstream side. Figure 2 shows the distribution of grain size of embankment soil and deposit on riverbed. Figure 3 shows the result of compaction tests of embankment soil consisted in silty sand.

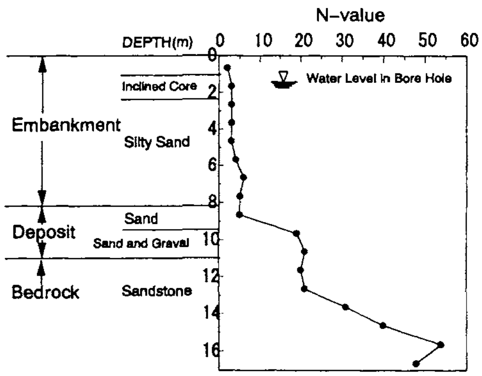

Fig.1 N-value at dam body of upstream side (Boring No. B-2)

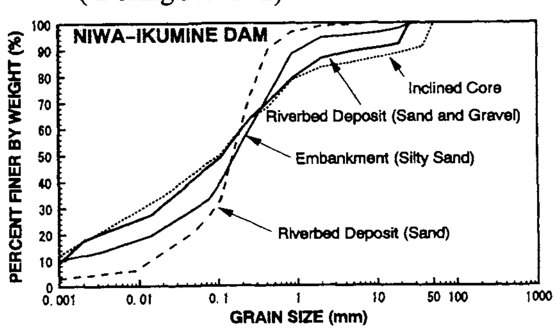

Fig.2 Distribution of grain size of embankment soil and deposit on riverbed

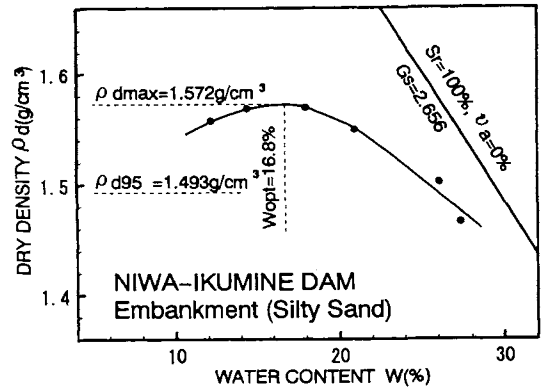

Fig.3 Compaction test of embankment soil

Discussion on themes titled:"Stability of Slopes and Earth Dams Under Earthquakes".

By: Shevtcenko Andrey, Department of Engineering Geology, Mining Institute of St.Petersburg, Russia.

In the seismic areas the large slips ofen are being called by earthquakes wich dangerous for life of peoles, be at the bottom of material and ecological damage.

For evaluatin of slope stability under earthquakes it is necessary to have datas about rock mass properties and earthquake intensivity.

In general case evaluatoin of slope stability include two independent stages. On the first stage are being conducted mathematical description of shearing and restranning powers, colculating along any shear surface in the slope, on the second stage characterize searching method of shear surface along which coefficient of stability is minimum.

Such approach require use of computers and allow obtain reliable results unda complicated geological conditions.

For the seismic areas value of shearing strengths is being calculated with take into consideration seismic acceleration which appear in rocks by an earthquake. In the each point the vector of seismic acceleration is being dirested along touching of the shear surface. Sum total seismic power included into sum of searing strangths and are depended with position of the shear surface in the massif.

Quantity of the seismic acceleretion (a_s) is being choosed in conformity the seismic area point (I) and amounts a_s - 0.5 - 2.5 (m/cek^2) under I - 9 -10 (point) (MSK-64). The seismic power is being colculated for each indiwidual element heving the definite weight (P_i), sum of total seismic pover $S - \sum a_s \times P_i/10$.

The restraining power is being calculated by characteristics of shear resistance obtained by the procedure of accelerated tests as the seismic power is transitory. However, earthquakes very increase shear strainin in the rocks.

The study of strain and shearing strength in the flaky environment we condukt by the many-boxes shearing device. This device are being loated sampels modeling real geologic section, shear test are being conduct by normal stress corresponding by colculated scheme and different for each sampl. Shear stress is passed to all the samples simultaneously. These tests allow determine dependence between shear stress and shear daformation.

The dengerous shear surface is being searched with the calculation for each surface the seismic power which is depended with position of the shear surface in the rock massif.

Calculations and observations have been executed in seismic areas of Uzbekistan and Far East of Russia had showed that position of the shear surface under earthquakes is benig removed at large depth into rock mass. Volume of the real landslides is immortalizing.