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Case Histories in Seismic Response Analysis

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SYNOPSIS: The reliability and utility of dynamic response analysis in geotechnical engineering is explored by a series of case histories. A detailed study of the seismic response of Mexico City sites during the 1985 earthquake shows clearly the limitations of present methods for estimating the appropriate input motions for analysis and the necessity of using a suite of representative input motions. Analyses of seismic soil-structure interaction are conducted on centrifuged models subjected to simulated earthquake loading. Finally the seismic response of a tailings dam is investigated using nonlinear dynamic effective stress analysis.

INTRODUCTION

The basic elements in the dynamic analysis of a soil-structure system are input motion, appropriate models of site and structure, constitutive relations for all materials present, and a stable, efficient, accurate, computational procedure. The specification of the input motion and the selection of an appropriate constitutive relation are the most difficult steps in the analysis.

Linear elastic analysis is appropriate for low levels of shaking in relatively firm ground. As the shaking becomes more intense, soil response becomes nonlinear. A great variety of constitutive relations are available for nonlinear response analysis ranging from equivalent linear elastic models to elastic-plastic models with both isotropic and kinematic hardening. An additional complication is the effect of seismically induced porewater pressures. If these become significant, the corresponding reduction in effective stresses will result in significant reductions in moduli and strength which must be taken into account. Therefore, for some problems, the simpler total stress methods of analysis are not adequate; effective stress methods must be used.

The most widely used methods for dynamic analysis are based on the equivalent linear model. Computer programs representative of this approach are SHAKE (Schnabel et al., 1972) for one-dimensional analysis (1-D) and FLUSH (Lysmer et al., 1975) for 2-D analysis. These programs perform total stress analyses only.

In recent years, there has been a distinct shift towards the use of nonlinear total and effective stress methods of analysis. A number of nonlinear 1-D programs are now available which give similar results for a given site (Streeter et al., 1973; Lee and Finn, 1975; Lee and Finn, 1978; Martin et al., 1978; Dikmen and Ghaboussi, 1984). A widely used program of this kind is DESRA-2 (Lee and Finn, 1978) which has been used for site response studies both onshore and offshore.

A number of programs are also available for 2-D nonlinear dynamic effective stress analysis. The simplest kind are based on nonlinear hysteretic models of soil response using hyperbolic skeleton loading curves and unloading-reloading response defined by the Masing criterion (Masing, 1926). A representative program of this type is TARA-3, the third in an evolving series of TARA programs (Finn et al., 1986). This program has been subjected to critical evaluation over the last three years using data from centrifuge model tests sponsored by the U.S. Nuclear Regulatory Commission through the European Office of the U.S. Army Corps of Engineers. Some of these tests have been described previously by Finn (1986). Some results from this study will be presented later.

2-D elastic-plastic models for dynamic effective stress analysis are generally based on Biot's equations (Biot, 1941) for coupled fluid-soil systems. However few of these have been incorporated in commercially available programs. The most widely used program of this type is DYNAFLOW (Prevost, 1981). The elastic-plastic effective stress models offer the most general descriptions of soil response. However, the properties required in some of them are difficult to measure and the programs make heavy demands on computational time. Analyses using these models have been conducted on super computers to cut the turn around time. Recent studies comparing a Japanese program of this type, DIANA-J, with the nonlinear program TARA-3 showed TARA-3 to be a minimum of 50-60 times faster (Yoshida, 1987).

EQUIVALENT LINEAR ANALYSIS OF SITE RESPONSE AT MEXICO CITY SITES

Mexico City is located in the south west corner of the Valley of Mexico on the edge of the former Lake Texcoco. During the 1985 earthquake, ground accelerations were recorded on hard sites in the foothills of the University district (UNAM) and on the soft deposits of the old lake bed. The locations of the accelerograph sites are shown in Fig. 1.

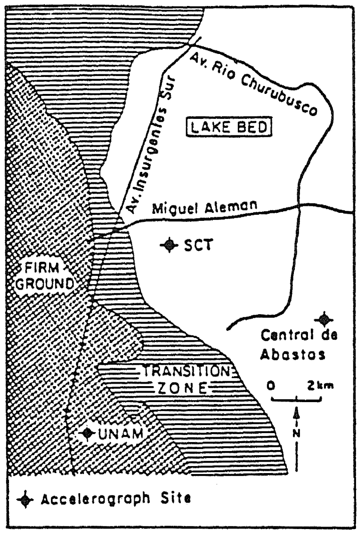

Fig. 1. Soil zones and accelerograph sites in Mexico City (after Mitchell et al., 1986).

Soil conditions in the lake zone are characterized by soft clay deposits overlying dense sands and much stiffer clays with shear wave velocities comparable to those of soft rock. Romo and Seed (1986) characterized the lake zone sites as homogeneous clay layers with the properties shown in Table 1. The CAF and CAO sites are

Table 1. Properties of Mexico City Sites for Dynamic Analysis (after Romo and Seed, 1986).

Site	Depth	Shear Wave Velocity	Unit Weight
SCT	35-40m	75-80 m/s	1.2 t/m^3
CAO	55m	65-75 m/s	1.2 t/m^3
CAF	45m	70 m/s	1.2 t/m^3

located in Central de Abastos (Fig. 1). The shear wave velocities, V_s, were determined from the natural periods of the sites obtained from the Fourier spectra of the recorded motions (Romo and Seed, 1986). The corresponding shear moduli, G, were obtained from $G = \rho V_s^2$ in which ρ = mass density of the soil. These values compared well with moduli derived from the results of resonant column and cyclic triaxial tests (Romo and Jaime, 1986). The variations of shear modulus and damping ratio of Mexico City clay as a function of shear strain (Leon et al., 1974; Romo and Jaime, 1986) are shown in Fig. 2.

The response of the lake zone sites will be analyzed in two ways, (1) following the usual procedure of using a scaled acceleration record from another earthquake as a representative input motion and (2) using the rock outcrop motions recorded at UNAM (Finn et al, 1988).

Analysis Using Scaled Motion

Romo and Seed (1986) analysed the three lake bed sites using the program SHAKE (Schnabel et al., 1972). They used the Pasadena record of the 1952 Kern County earthquake (M ≈ 7.6) as a representative input motion after scaling it appropriately for peak acceleration (a_{max} = 0.035 g) and frequency to obtain strong response around a period of two seconds, the period of the SCT site. This scaling resulted in a good match between the computed acceleration response spectrum for the SCT site and the average spectrum of the two horizontal components of acceleration recorded at the site (Fig. 3). This analysis was repeated for the present study with similar results (Fig. 3).

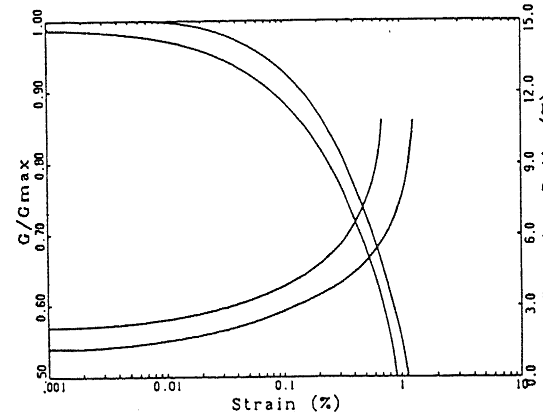

Fig. 2. Range in shear-dependent moduli and damping for Mexico City clay (after Leon et al., 1974 and Romo and Jaime, 1986).

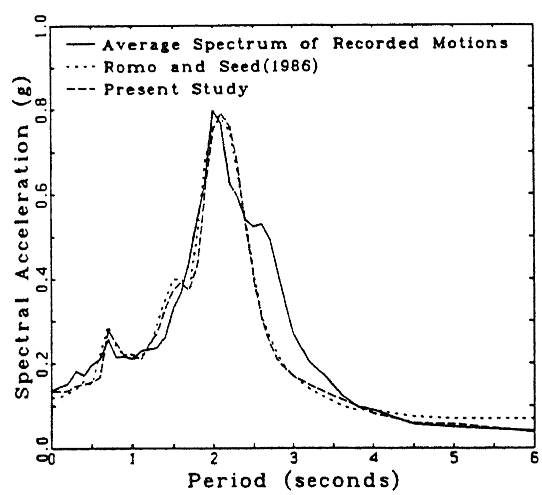

Fig. 3. Response spectra (5% damping) of computed motions and average response spectra of the recorded motions at the SCT site.

The acceleration response spectrum of the scaled Pasadena motions is shown in Fig. 4 together with the spectrum of the motions scaled for peak acceleration only. It is clear that scaling for frequency resulted in a major shift in the period of strong spectral response. Such a major scaling for frequency would probably not have been considered

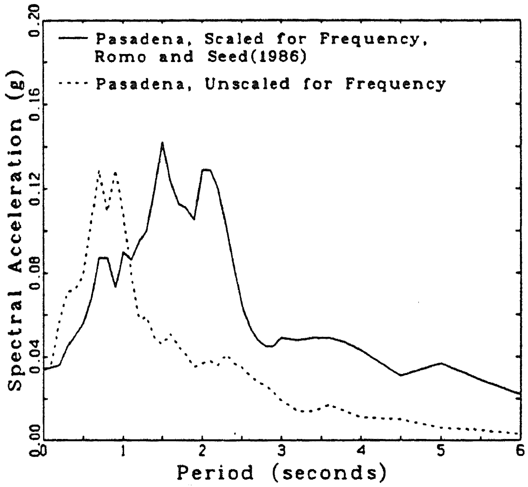

Fig. 4. Response spectra (5% damping) for Pasadena record showing effect of frequency scaling.

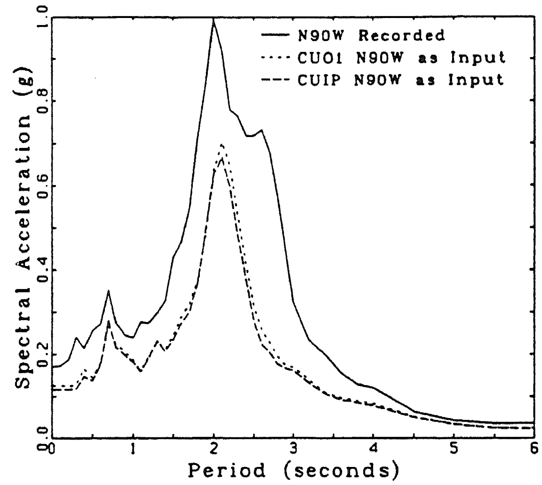

Fig. 5. Response spectra (5% damping) of recorded and computed motions in the N90W direction at the SCT site.

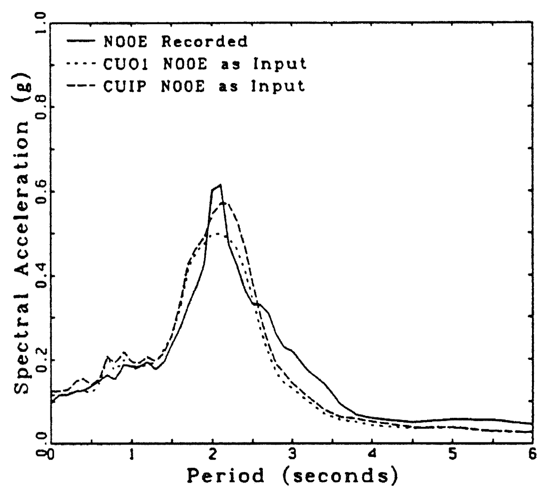

Fig. 6. Response spectra (5% damping) of recorded and computed motions in the N00E direction at the SCT site.

necessary had it not been for the availability of the 1985 ground motion records which showed that the peak response at the SCT site occurred around a period of 2 sec and that the rock motion had relatively high response at the same period. The predominant period of rock input motions would probably have been estimated at around 1.5 sec based on the relationship between predominant period and distance to the causative fault developed by Seed et al. (1969).

Note that uniform scaling of a record for frequency does more than simply shift the period of peak response to the desired frequency. It also enlarges the frequency range in which strong response may be encountered. Therefore input motions with predominant periods greatly different from the required predominant period should not be uniformly scaled unless broad band strong response is desired.

Analysis Using Rock Outcrop Motions

Accelerations were recorded at two hard sites at UNAM. The motions, designated CU01 and CUIP, were recorded on the first floor of a three storey building and in the free field respectively. The seismic response of the SCT site was analyzed using the N90W and N00E components of these motions as input motions to the SHAKE program. The peak horizontal accelerations of these components are as follows: CU01-N90W = 0.034 g, CU01-N00E = 0.028 g, CUIP-N90W = 0.035 g and CUIP-N00E = 0.032 g. Detailed discussions of the characteristics of the recorded ground motions are given by Anderson et al., 1986a, 1986b.

The acceleration response spectra for the computed ground motions are compared with the corresponding spectra of the recorded motions in Figs. 5 and 6. In the N90W direction, both spectra of the computed motions underestimate the spectrum of the recorded motions both in terms of peak spectral acceleration and the range of strong response (Fig. 5). The agreement in the case of the N00E components is much better (Fig. 6). Clearly the motions at the SCT site must have a much greater directional bias than the rock motions recorded at UNAM. This is evident from the acceleration plots in Figs. 7a and 7b, which show the total acceleration paths for the CUIP motions at the UNAM site and the SCT site respectively. Note that the acceleration paths for the SCT site lie in an elongated band inclined significantly more to the N90W direction than to the N00E direction.

A close match between the computed and measured spectra for the N90W direction is obtained if the CUIP input motions are scaled to a peak acceleration of 0.095 g from 0.035 g as shown in Fig. 8. Note that the peak spectral accelerations are approximately equal and the range of peak response is now considerably wider than that obtained using the unscaled motions as input (Fig. 5). Even the shoulder in the recorded response spectrum to

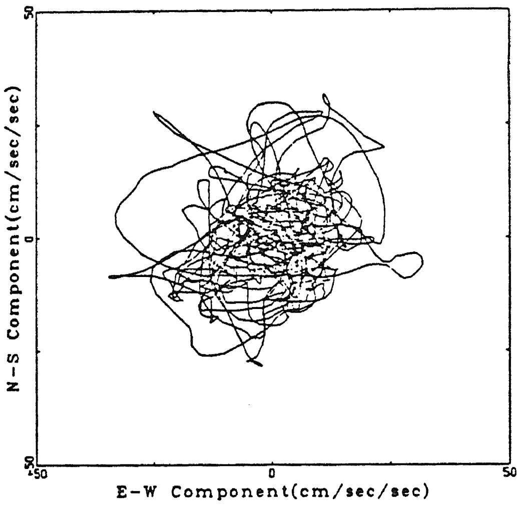

Fig. 7a. Recorded accelerations at the CUIP site.

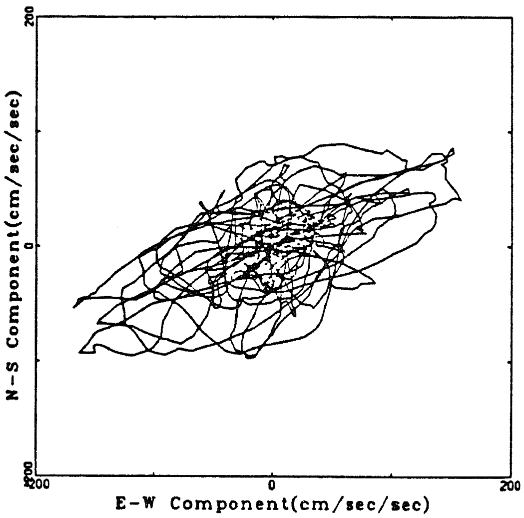

Fig. 7b. Recorded accelerations at the SCT site.

the right of the peak response is now reproduced. When the N90W component of the CU01 motions are scaled also to 0.095 g, the computed response is slightly greater than the recorded response but the correct shape is reproduced. By further refinement in the scaling a closer match could be obtained in the region of peak response. But the lesson is already clear that scaling of the rock outcrop motions by a factor of about 2.5 is necessary to get a good match between recorded and computed spectra in the N90W direction in the region of peak response. Note that the scaling to match peak response has resulted in higher computed responses around a period of 1 sec. It is not possible to get simultaneously a good match in both these regions of the response spectrum. A reasonable match may be obtained in the N00E direction with little or no scaling (Fig. 6).

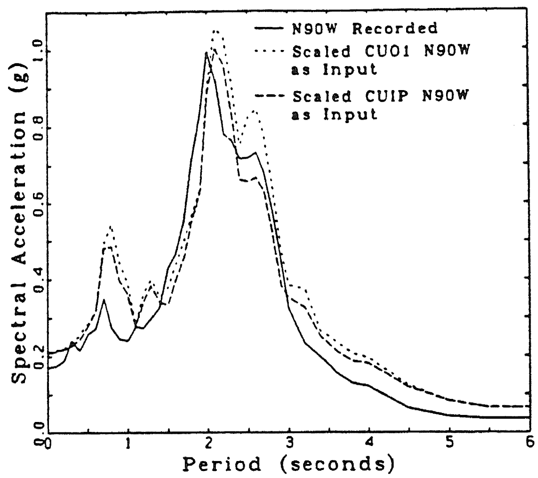

Fig. 8. Response spectra (5% damping) of recorded and computed motions in the N90W direction at the SCT site, scaled outcrop motions as input.

There are a number of possible explanations for the discrepancies between recorded and computed spectra when the rock outcrop motions are used as input. If it is assumed that the shear beam model adequately represents the dynamic response of the sites then it must be assumed that the rock outcrop motions are modified as they pass into the hard sand layer, acquiring a directional bias and a redistribution of peaks. But it seems unlikely that this is the sole explanation. It is probable that surface waves are generated in the stratum above the sand layer with periods dependent on the local thickness and stiffness of the layer. Information on these waves would not be included in the rock outcrop motions, resulting in significant long period differences between computed and recorded responses. An interesting example of this is provided by data from the Oji site in Japan (Ohta et al., 1977).

2-D NONLINEAR DYNAMIC EFFECTIVE STRESS ANALYSIS

2-D dynamic analyses are usually conducted using equivalent linear finite element analyses in the frequency domain. There has been little verification of these methods because of a lack of adequate field data.

There are certain important phenomena in soil-structure interaction outside the scope of conventional frequency domain analysis. Typical examples are uplift during rocking, permanent deformations, the effects of seismically induced porewater pressures, hysteretic behaviour and stick-slip behaviour at interfaces between structure and foundation soils.

The program TARA-3 (Finn et al., 1986) was developed to cope with such problems. The capability of the program will be demonstrated by using it to analyze one of the NRC centrifuge tests which models the response of a heavy two-dimensional structure embedded in a saturated sand foundation to seismic excitation.

ANALYSIS BY TARA-3

In TARA-3, response in shear is assumed to be nonlinear and hysteretic with unloading and reloading stress-strain paths defined by the Masing criterion (Masing, 1926). The response of the soil to uniform all round pressure is assumed to be nonlinearly elastic and dependent on the mean normal effective stress. Porewater pressures during shaking are computed using the Martin-Finn-Seed porewater pressure model (Martin et al., 1975) modified to take into account the effects of initial static shear stress. Moduli and strength are continuously modified during analysis to reflect changes in the effective stress regime. A detailed description of the constitutive relations in TARA-3 is given by Finn (1985).

For analysis involving soil-structure interaction it may be important to model slippage between the structure and soil. Slip may occur during very strong shaking or even under moderate shaking if high porewater pressures are developed under the structure. TARA-3 contains slip elements of the Goodman (Goodman et al., 1968) type to allow for relative movement between soil and structure in both sliding and rocking modes during earthquake excitation.

MODEL STRUCTURE EMBEDDED IN SATURATED SAND

A schematic view of the model structure is shown in Fig. 9. It is made from a solid piece of aluminum alloy and has dimensions 150 mm wide by 108 mm high in the plane of shaking.

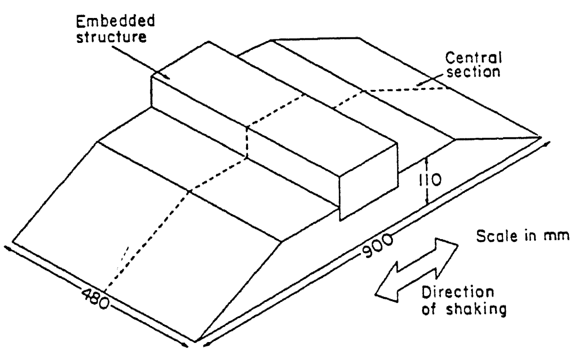

Fig. 9. Centrifugal model of embedded structure.

The length perpendicular to the plane of shaking is 470 mm and spans the width of the model container. The structure is embedded a depth of 25 mm in the sand foundation. Sand was glued to the base of the structure to prevent slip between structure and sand.

The foundation was constructed of Leighton Buzzard Sand passing British Standard Sieve (BSS) No. 52 and retained on BSS No. 100. The mean grain size is therefore 0.225 mm. The sand was placed as uniformly as possible to a nominal relative density D_r = 52%.

During the test the model experienced a nominal centrifugal acceleration of 80 g. The model therefore simulated a structure approximately 8.6 m high by 12 m wide embedded 2 m in the foundation sand.

De-aired silicon oil with a viscosity of 80 centistokes was used as a pore fluid in order to model the drainage conditions in the prototype during the earthquake. If the linear scale factor between model and prototype is N, then excess porewater pressures dissipate approximately N^2 times faster in the model than in the prototype if the same fluid is used in both. The rate of loading by seismic excitation will be only N times faster. Therefore, to model prototype drainage conditions during the earthquake, a pore fluid with a viscosity N times the prototype viscosity must be used. This viscosity was achieved by an appropriate blending of commercial silicon oils. Tests by Eyton (1982) have shown that the stress-strain behaviour of fine sand is not changed when silicon oil is substituted for water as a pore fluid. In the gravitational field of 80 g, the structure underwent consolidation settlement which led to a significant increase in density under the structure compared to that in the free field. This change in density was taken into account in the analysis.

The locations of the accelerometers (ACC) and pressure transducers (PPT) are shown in Fig. 10. Analyses of previous centrifuge tests

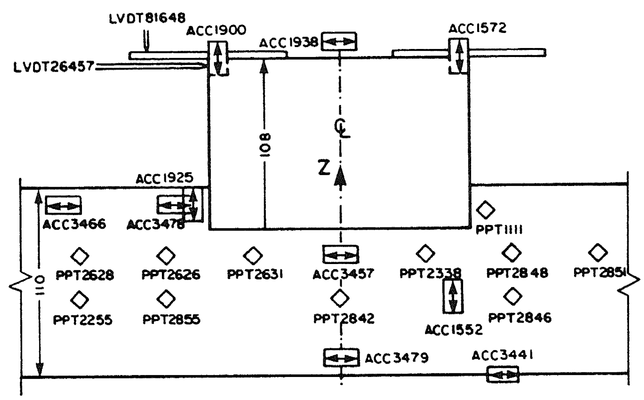

Fig. 10. Instrumentation of centrifuged model.

indicated that TARA-3 was capable of modelling acceleration response satisfactorily. Therefore, in the present test, more instrumentation was devoted to obtaining a good data base for checking the ability of TARA-3 to predict residual porewater pressures.

As may be seen in Fig. 10, the porewater pressure transducers are duplicated at corresponding locations on both sides of the centre line of the model except for PPT 2255 and PPT 1111. The purpose of this duplication was to remove any uncertainty as to whether a difference between computed and measured porewater pressures might be due simply to local inhomogeneity in density.

The porewater pressure data from all transducers are shown in Fig. 11. These records show the sum of the transient and residual porewater pressures. The peak residual pressure may be observed when the excitation has ceased at about 95 milliseconds. The pressures recorded at corresponding points on opposite sides of the centre line such as PPT 2631 and PPT 2338 are generally quite similar although there are obviously minor differences in the levels of both total and residual porewater pressures. Therefore it can be assumed that the sand foundation is remarkably symmetrical in its properties about the centre line of the model. Hence measured and computed porewater pressures are compared only for locations on one side of the centre line of the model only, the right hand side.

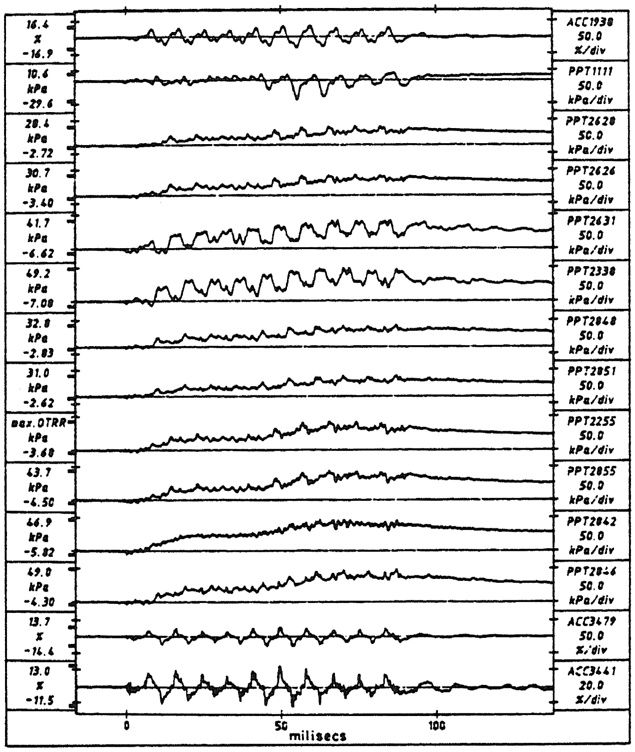

Fig. 11. Complete porewater pressure data from centrifuge test.

COMPUTED AND MEASURED ACCELERATION RESPONSES

The soil-structure interaction model was converted to prototype scale before analysis using TARA-3 and all data are quoted at prototype scale. Soil properties were consistent with relative density.

The computed and measured horizontal accelerations at the top of the structure at the location of ACC 1938 are shown in Fig. 12. They are very similar in frequency content, each corresponding to the frequency of the input motion given by ACC 3441 (Fig. 11). The peak accelerations agree fairly closely.

The vertical accelerations due to rocking as recorded by ACC 1900 and those computed by TARA-3 are shown in Fig. 13. Again, the computed accelerations closely match the recorded accelerations in both peak values and frequency content. Note that the frequency content of the vertical accelerations is much higher than that of either the horizontal acceleration at the same level in the structure

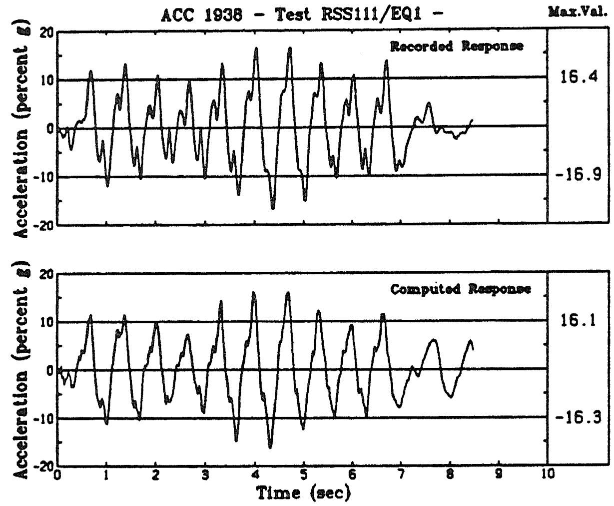

Fig. 12. Recorded and computed horizontal accelerations at ACC 1938.

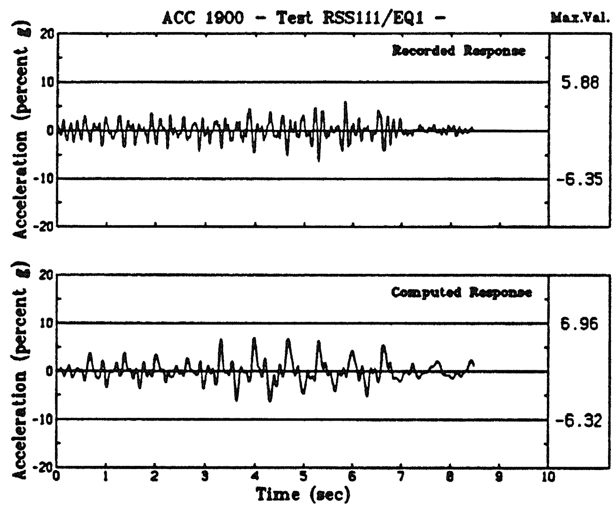

Fig. 13. Recorded and computed vertical accelerations at ACC 1900.

or that of the input motion. This occurs because the foundation soils are much stiffer under the normal compressive stresses due to rocking than under the shear stresses induced by the horizontal accelerations.

COMPUTED AND MEASURED POREWATER PRESSURES

The porewater pressures in the free field recorded by PPT 2851 are shown in Fig. 14. In this case the changes in the mean normal stresses are not large and the fluctuations of the total porewater pressure about the residual value are relatively small. The peak residual porewater pressure, in the absence of drainage, is given directly by the pressure recorded after the earthquake excitation has ceased. In

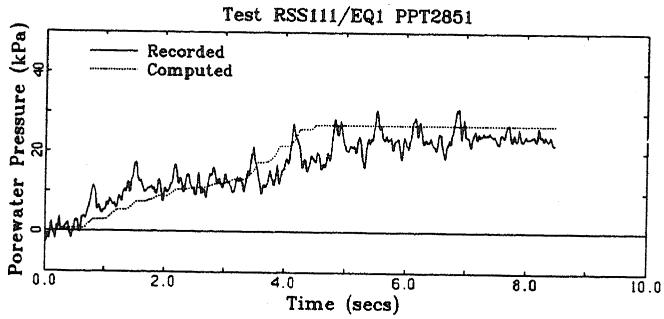

Fig. 14. Recorded and computed porewater pressures at PPT 2851.

the present test, significant shaking ceased after 7 seconds. A fairly reliable estimate of the peak residual pressure is given by the record between 7 and 7.5 seconds. The recorded value is slightly less than the value computed by TARA-3 but the overall agreement between measured and computed pressures is quite good.

As the structure is approached, the recorded porewater pressures show the increasing influence of soil-structure interaction. The pressures recorded by PPT 2846 adjacent to the structure (Fig. 15) show somewhat larger oscil-

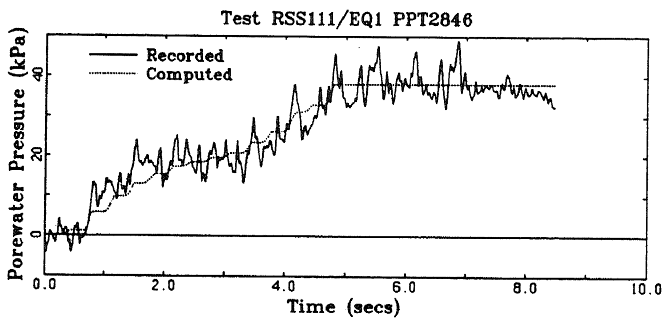

Fig. 15. Recorded and computed porewater pressures at PPT 2846.

lations than those recorded in the free field. This location is close enough to the structure to be affected by the cyclic normal stresses caused by rocking. The recorded peak value of the residual porewater pressure is given by the relatively flat portion of the record between 7 and 7.5 seconds. The computed and recorded values agree very closely.

Transducer PPT 2338 is located directly under the structure near the edge and was subjected to large cycles of normal stress due to rocking of the structure. These fluctuations in stress resulted in similar fluctuations in mean normal stress and hence in porewater pressure. This is clearly evident in the porewater pressure record shown in Fig. 16. The

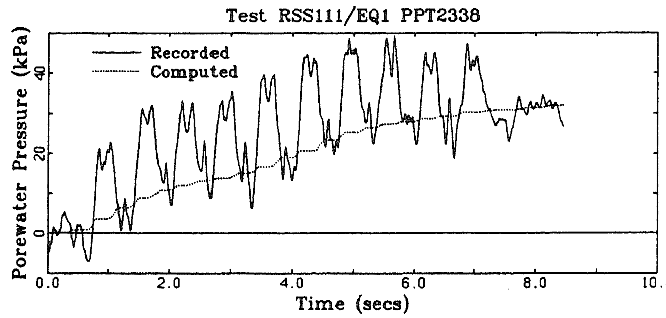

Fig. 16. Recorded and computed porewater pressures at PPT 2338.

higher frequency peaks superimposed on the larger oscillations are due to dilations caused by shear strains. The peak residual porewater pressure which controls stability is observed between 7 and 7.5 seconds just after the strong shaking has ceased and before significant drainage has time to occur. The computed and measured residual porewater pressures agree very closely.

Contours of computed porewater pressures are shown in Fig. 17. They indicate very symmetrical distribution of residual porewater pressure. Recorded values are also shown in this figure.

Stress-Strain Response

It is of interest to contrast the stress-strain response of the sand under the structure with that of the sand in the free field. The stress-strain response at the location of porewater pressure transducer PPT 2338 is shown in Fig. 18. Hysteretic behaviour is evident but the response for the most part is not strongly nonlinear. This is not surprising as the initial effective stresses under the structure were high and the porewater pressures reached a level of only about 20% of the initial effective vertical stress.

The response in the free field at the location of PPT 2851 (Fig. 19) is strongly nonlinear with large hysteresis loops indicating considerable softening due to high

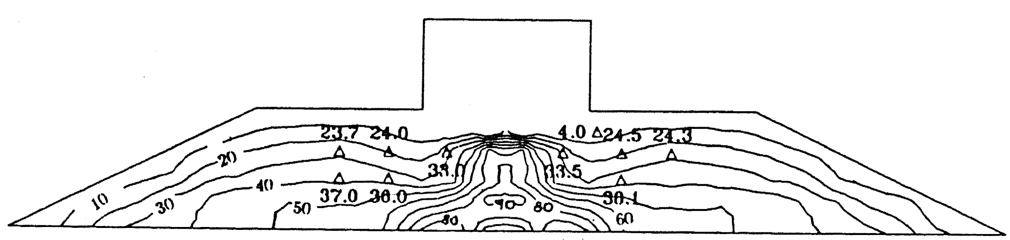

Fig. 17. Contours of computed porewater pressures.

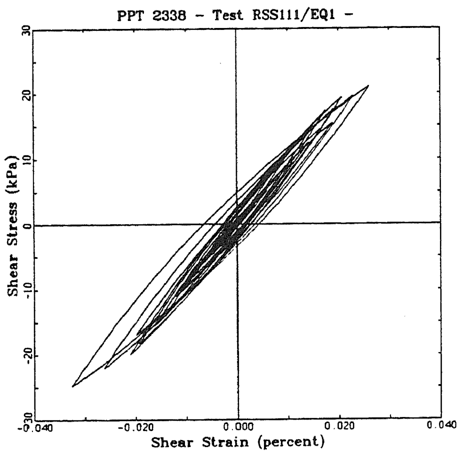

Fig. 18. Stress strain response under the structure.

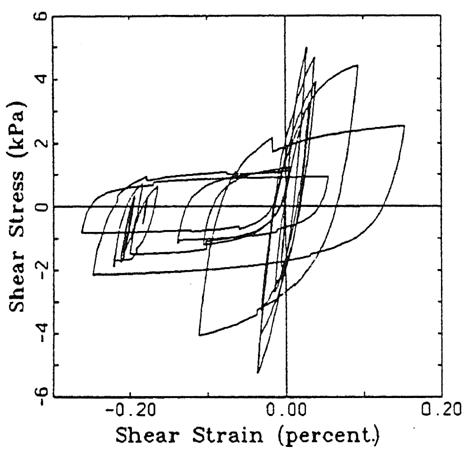

Fig. 19. Stress strain response in the free field.

porewater pressures and shear strain. At this location the porewater pressures reached about 80% of the initial effective vertical pressure.

ANALYSIS OF DAMS

Since the development of TARA-3 in 1986, it has been used to estimate the seismic response of a number of dams. In particular, it has been used to determine the peak dynamic displacements and the post-earthquake permanent deformations. Typical results for the proposed Lukwi tailings dam in Papua New Guinea will be presented to show the kind of data that is provided by a true nonlinear effective stress method of analysis (Finn et al., 1987,1988).

Lukwi Tailings Dam

The finite element representation of the Lukwi tailings dams is shown in Fig. 20. The sloping line in the foundation is a plane between two foundation materials. Upstream to the left is a limestone with shear modulus $G = 6.4 \times 10^6$ kPa and a shear strength defined by $c' = 700$ kPa and $\phi' = 45°$. The material to the right is a siltstone with a low shearing resistance given by $c' = 0$ and $\phi' = 12°$. The shear modulus is approximately $G = 2.7 \times 10^6$ kPa. The difference in strength between the foundation soils is reflected in the dam construction. The upstream slope on the limestone is steep whereas the downstream slope on the weaker foundation is much flatter and has a large berm to ensure stability.

The dam was subjected to strong shaking with a peak acceleration of 0.33 g (Fig. 21). The

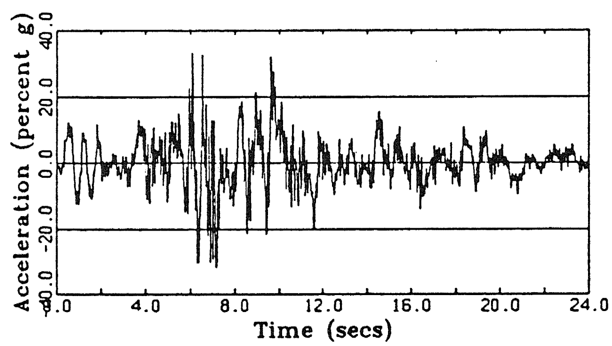

Fig. 21. Input motion of analysis of Lukwi tailings dam.

response of the limestone foundation is almost elastic as shown in Fig. 22 by the shear stress-shear strain response for a typical element.

The response of the siltstone foundation is strongly nonlinear. The deformations increase progressively in the direction of the initial static shear stresses as shown in Fig. 23. Since the analysis starts from the initial

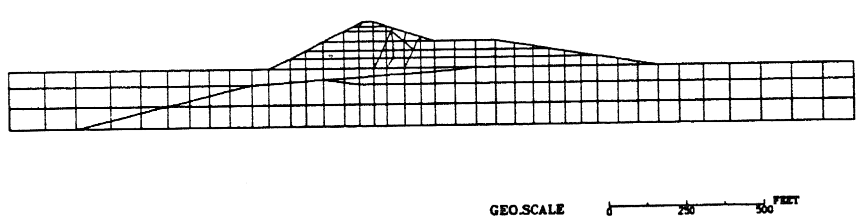

Fig. 20. Finite element idealization of Luwki tailings dam.

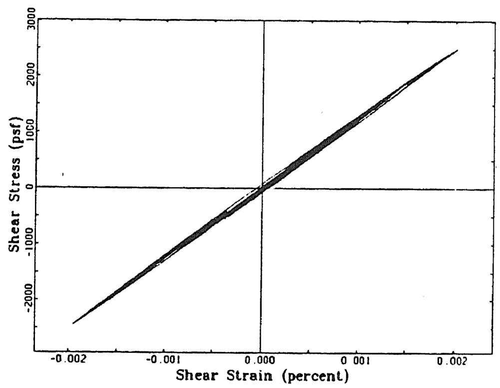

Fig. 22. Shear stress-shear strain response of limestone foundation.

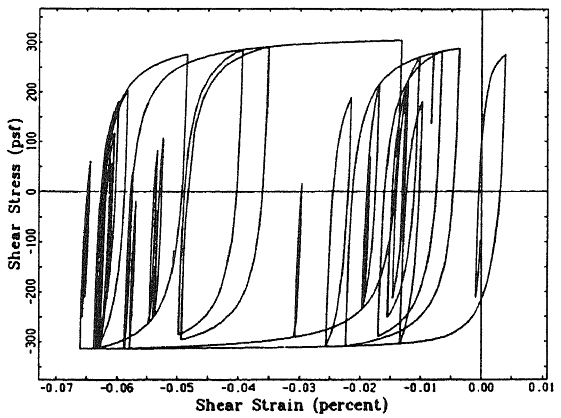

Fig. 23. Shear stress-shear strain response of siltstone foundation.

post-construction stress-strain condition subsequent large dynamic stress impulses move the response close to the highly nonlinear part of the stress-strain curve. It may be noted that the hysteretic stress-strain loops all reach the very flat part of the stress-strain curve, thereby ensuring successively large plastic deformations.

An element in the berm also shows strong nonlinear response with considerable hysteretic damping (Fig. 24).

The acceleration time history of a point near the crest in the steeper upstream slope is shown in Fig. 25. The displacement time history of the point is shown in Fig. 26. Note that the permanent deformation is of the order of 25 cm. Most of this was generated by a large permanent slip which occurred about 8 secs after the start of shaking.

The deformed shape of the central portion of the dam is shown to a larger scale in Fig. 27.

CONCLUSIONS

The selection of representative motions for use as input in seismic response analyses requires considerable skill and a deep understanding of the role of system characteristics

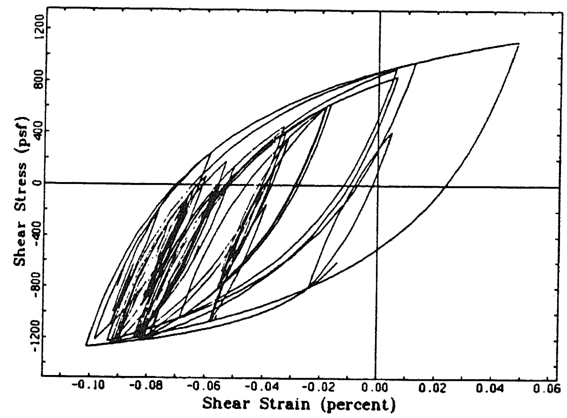

Fig. 24. Shear stress-shear strain response in the berm.

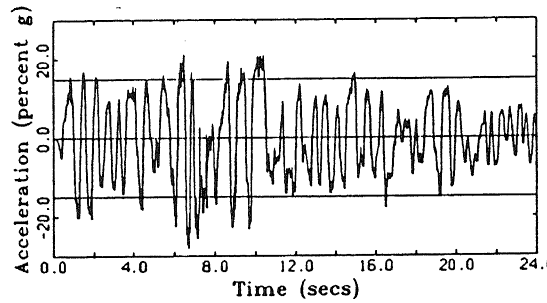

Fig. 25. Computed accelerations of a point near the crest.

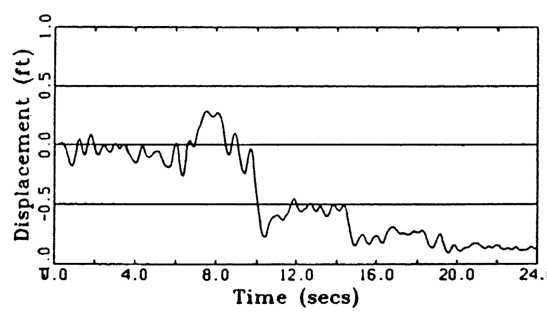

Fig. 26. Displacement history of a point near the crest.

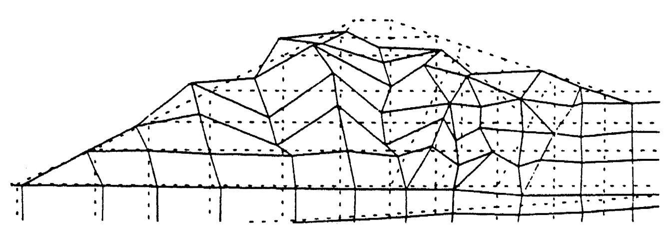

Fig. 27. Deformed shape of the dam after the earthquake to enlarged scale.

in defining seismic response. The practice of selecting just a few candidate motions, which is unfortunately fairly common, may be dangerously unconservative.

The very common practice of modelling the incoming seismic waves as horizontal shear waves propagating vertically is inadequate wherever significant surface waves are present. It may also be inadequate close to the epicentre.

Dynamic analysis provides a constant ordered approach to estimating the characteristics of site specific motions. It allows parametric studies to be conducted which are powerful guides to judgement. Estimating "exact" seismic response parameters is impossible; defining safe but economical design parameters with the help of an adequate supply of representative input motions for dynamic response analysis is feasible and practical.

Phenomenological aspects of soil-structure interaction are clearly demonstrated in centrifuge tests such as high frequency rocking response, the effects of rocking on porewater pressure patterns and the distortion of free-field motions and porewater pressures by the presence of a structure.

The comparison between measured and computed responses for the centrifuge model of a structure embedded in a saturated sand foundation demonstrates the wide ranging capability of TARA-3 for performing complex effective stress soil-structure interaction analysis with acceptable accuracy for engineering purposes. Seismically induced residual porewater pressures are satisfactorily predicted even when there are significant effects of soil-structure interaction. Computed accelerations agree in magnitude, frequency content and distribution of peaks with those recorded. In particular, the program was able to model the high frequency rocking vibrations of the model structures. This is an especially difficult test of the ability of the program to model soil-structure interaction effects.

The program TARA-3 can compute directly the permanent deformations of earth dams under seismic loading. The utility of TARA-3 in practice was demonstrated by the analysis of the Lukwi tailings dam. Computed stress-strain responses show the widely different responses of the different foundation materials to the design earthquake. The computed final deformed shape of the dam itself reflected clearly the influence of dam geometry and the different foundation materials.

The nonlinear effective stress analysis provides a very clear overall picture of the response of the dam to the design earthquake as well as providing the designer with all the details necessary in zones of potential concern.

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