Soil-structure interaction during earthquakes

Interaction sol-structure pendant les seismes

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SYNOPSIS A method for non-linear dynamic effective stress analysis is introduced which is applicable to the soil-structure interaction problems. Its scope is demonstrated by analysis of a drilling island consisting of a ballasted tank placed on a submerged sand berm. Full interaction including potential slip between tanker and berm is taken into account. Verification is provided by data from simulated earthquake tests on a centrifuged model.

INTRODUCTION

A major difficulty in conducting analyses of soil-structure interaction during earthquakes is the inclusion of all the factors that have a strong influence on soil behaviour. The major factors which must be considered when computing soil response are: (1) in-situ moduli, (2) non-linear variation of moduli with strain, including effects of strain hardening and softening, (3) seismically induced porewater pressures, (4) effective stress changes due to porewater pressure changes, (5) viscous and hysteretic damping, and (6) volume changes in the soil skeleton.

In an attempt to include these factors in dynamic response analysis of soils, Siddharthan and Finn (1982) developed a dynamic non-linear effective stress method of analysis and later expanded it to include soil-structure interaction. The method of analysis has been incorporated in the computer program, TARA-2 (Siddharthan and Finn, 1982). A general description of the fundamental basis of the TARA-2 analysis is given here. However, the primary objectives of the present paper are to demonstrate the coherent picture of dynamic response of soil-structure systems provided by effective stress non-linear analysis and to present preliminary results from a validation study of TARA-2 using simulated earthquake tests on centrifuged models.

METHOD OF ANALYSIS

The method of dynamic analysis incorporated in TARA-2 is an extension of the method of non-linear dynamic effective stress analysis developed by Finn et al (1976) for level ground conditions. Soil response is modelled by combining the effects of shear and normal stresses. In shear, the soil is treated exactly as in the level ground analysis; it is considered as a non-linear hysteretic material exhibiting Masing behaviour during unloading and reloading (Masing, 1926).

The stress-strain behaviour is characterised by a tangent shear modulus which depends on the current shear strain, the state of effective stress and the previous loading history. The shear model is described in detail by Finn et al (1976) and has been verified in both laboratory tests (Finn and Bhatia, 1980) and by field data (Finn et al, 1982).

The model has been extended to 2 dimensions by including response to hydrostatic effective stresses. Soil behaviour under changes in mean-normal stresses is taken to be non-linear and effective stress-dependent but essentially elastic compared to shear response.

Porewater pressures are generated during analyses using the Martin-Finn-Seed porewater pressure model (Martin et al, 1975), extended to include the effects of initial static shear stresses.

The program includes slip elements to allow for relative motion between structure and soil in both sliding and rocking modes during earthquake excitation. The elements are of the Goodman type (Goodman et al, 1968).

Both dynamic and permanent deformations, consolidation settlements, porewater pressures, accelerations and velocities are computed. The program continuously modifies soil properties for the effects of porewater pressures and dynamic strains during analysis.

VALIDATION STUDY OF TARA-2

A series of seismic tests on centrifuged models is being conducted in the Cambridge University Geotechnical Centrifuge to obtain data on the seismic response of embankments and foundation soils carrying both surface and buried structures for use in validating TARA-2. The tests are being conducted on both dry and saturated sands. Data from these tests, sponsored by the Nuclear Regulatory Commission and Army Corps of Engineers of the United

States, will be available for release at a later date. Some of these tests replicate studies reported by Lee (1983) and one of Lee's tests will be described and analysed below. More details on this test can be found in Finn et al (1984).

Model Embankment

A model embankment was constructed with Leighton Buzzard Sand passing BSS No. 120 and retained on BSS No. 200. The relative density of the model bank was D_r = 65%. The liquefaction potential of the sand (Fig. 1) was determined by cyclic simple shear tests using the University of British Columbia simple shear device.

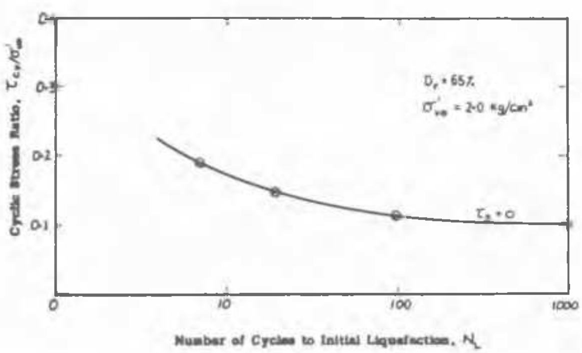

FIG. 1 Liquefaction resistance curve of medium dense Leighton-Buzzard sand.

De-aired silicon oil was used as a pore fluid in order to model the drainage conditions in the prototype during the earthquake. In a 1/N linear scale model, excess porewater pressures dissipate 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) showed that the stress-strain behaviour of fine sand was not changed when silicon oil was substituted for water as a pore fluid.

A plane strain model of a submerged embankment is shown in Fig. 2. The model is 90 mm high with side slopes 3:1 and a crest width of 200 mm. The centrifuge acceleration used in the preliminary test series was 40 g. The model, therefore, corresponds to a prototype 3.6 m high with a crest width of 8 m. Structural loading on the bank was simulated by using mild steel plates of various thicknesses. The average contact pressures of the steel plate on the bank for the tests, described later, were 15 kPa (Test 1) and 31 kPa (Test 2). The model was instrumented by 6 DJB A23 piezoelectric accelerometers (ACC), 6 Druck PDCR 81 porewater pressure transducers (PPT) and 2 displacement transducers (LVDT). The locations of these instruments are shown in Fig. 2.

Typical Test Data

Signals from the model were recorded on a 14 track RACAL tape recorder. These analogue signals were processed and digitised using the software package, FLY-14, developed by Dean (1984). The raw digitised data were smoothed once using a three point average scheme taking 1/2 of the value at the current point, 1/4 of the value at the previous point and 1/4 of the value at the next point. This was necessary to filter out very high frequency electrical noise which contained negligible energy.

A sample of typical test data is shown in Fig. 3. It should be noted that there are wide variations in the scales of the various records and the apparently quite different forms of some of the records are due primarily to differences in scale. All scales are model scales. The accelerations are expressed as % of the centrifuge acceleration. Porewater pressures are those actually measured. Because of limitations on available channels, the LVDT's were read only at the beginning and end of the tests to give the total displacements.

Equivalent prototype times are given by multiplying measured times by 40, the linear scale

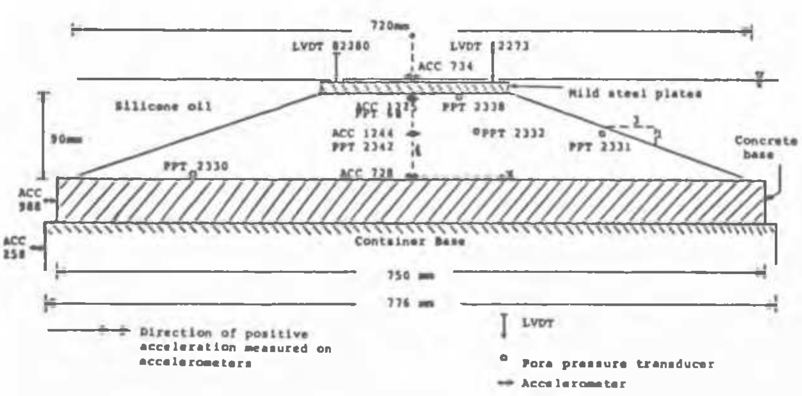

FIG. 2 Submerged embankment showing transducer locations.

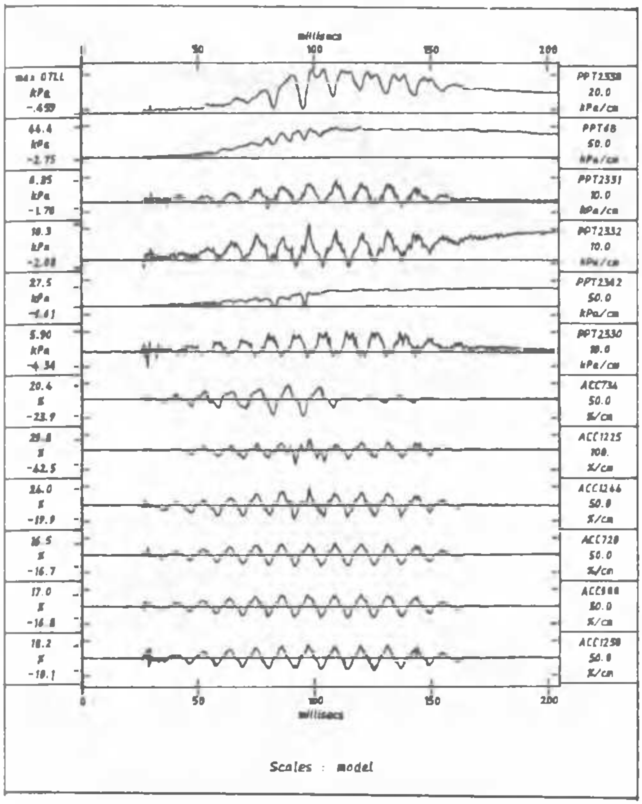

FIG. 3 Typical test data from centrifuge test.

factor. The accelerations expressed as % of model gravity and porewater pressures are the same in model and prototype. For transducer PPT 2338, the maximum positive value is replaced by OTLL in Fig. 3. This means that part of the transducer trace is outside the guaranteed linear range of the tape recorder. Values read from traces marked OTLL must be viewed with some caution.

Accelerations are amplified up through the bank from ACC 728 to ACC 1225. The acceleration record of the steel plate ACC 734 has quite a different character and shows that only very small accelerations were transmitted from the bank after about 100 ms of shaking. Beyond this point, the motions of the plate, as depicted by ACC 734, appear to be largely uncoupled from the motions of the bank as given by ACC 1225.

Porewater pressure transducers PPT 2338 and PPT 68 record porewater pressures near the surface of the berm under the plate. The pressures are at a level synonymous with liquefaction after t = 100 ms. Therefore, there is little effective normal stress between plate and berm and consequently little shearing resistance. Hence, shear waves cannot be propagated from berm to plate with the result that after t = 100 ms only very low accelerations are recorded by ACC 734.

PPT 2338 is nearer the edge of the embankment than PPT 68 and consequently shows higher rates of porewater pressure dissipation than PPT 68.

There is little residual porewater pressure accumulated at the location of PPT 2331. This transducer is located near the edge of the embankment and easy drainage prevents the build-up of residual porewater pressures. Little drainage takes place from PPT 2342 during excitation because of the lengths of any drainage paths. The internal redistribution of porewater pressures after the earthquake is clearly seen in the record from PPT 2332. After the earthquake, the pressure increases at this location are due to migration from surrounding areas of high porewater pressure.

The oscillations in the porewater pressures are due to contemporaneous changes in total mean-normal stresses which arise from the dynamic stresses and dilations at larger strains. These oscillations are superimposed on the steadily accumulating residual porewater pressures which control stability, liquefaction potential and consolidation settlements. TARA-2 computes residual porewater pressures only and ignores the superimposed oscillations.

Analysis of Data

The model bank shown in Fig. 2 was analysed by TARA-2 using soil properties consistent with a relative density of D_r = 65%. The accelerations recorded by ACC 988 were used as input motions. The results of the calculations and corresponding test data are given in later figures at prototype scale.

Data are presented for 2 tests conducted by Lee (1983). In Test No. 1, the peak acceleration of the input motion was 0.11 g. Liquefaction did not occur and residual porewater pressures are relatively low. Motions between bank and plate remained strongly coupled throughout the excitation. In Test No. 2, data from which was presented in Fig. 3, the peak acceleration of the input motion was 0.17 g. High porewater pressures occurred and eventually motion between structure and bank became uncoupled. During analysis of this case, the slip elements in TARA-2 were activated. The importance of accounting for such slip is demonstrated by analysing the island with and without slip elements in this case.

Test No. 1

Recorded and computed accelerations for ACC 1244 are shown in Figs. 4(a) and 4(b) and show satisfactory agreement. Note that there is high frequency noise in the recorded accelerations. This noise is considered to come from the walls and top of the model container and not to be propagated as shear waves from the base. Comparisons between computed and recorded peak accelerations are given in Table 1 and appear to be quite good.

In Test No. 1, residual porewater pressures are quite low and will not be examined here.

Test No. 2

High porewater pressures are the outstanding characteristics of Test No. 2 and attention will be confined to these. PPT 2342 is located in the middle of the bank. It is far away enough from the structure to suggest that its response may not be greatly affected by slip

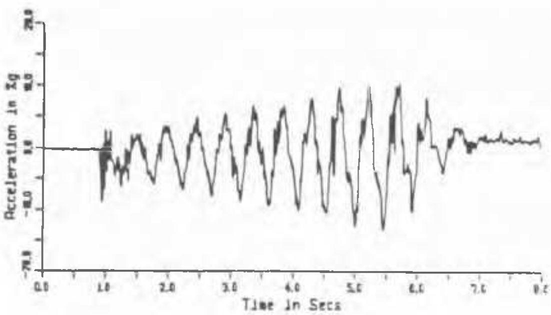

FIG. 4(a) Recorded acceleration of ACC 1244 in Test 1.

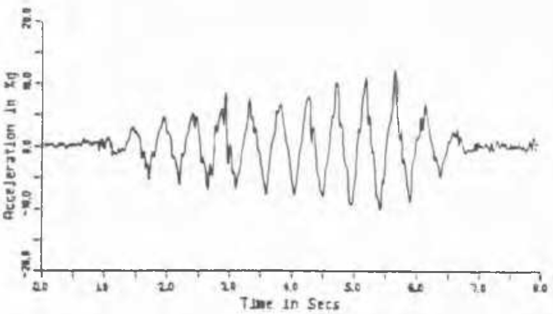

FIG. 4(b) Computed acceleration of ACC 1244 in Test 1 (with and without slip elements).

TABLE 1

Recorded and Computed Maximum Accelerations

Instrument Location	Maximum Acceleration, % g		
	Recorded	Computed by TARA-2	
		Without Slip Elements	With Slip Elements
A1244	13.3	11.6	11.6
A1225	15.9	12.5	12.5
A734	13.9	12.7	12.7

between structure and berm. This is confirmed by dynamic analysis which showed similar computed porewater pressures whether slip elements were used or not (Fig. 5). The computed pressures compare very well with those recorded.

PPT 668 is located near the top of the bank under the structure. At this location, the effects of any decoupling between the motions of the structure and the bank would be greatest. This is clearly shown by the results of dynamic analysis shown in Fig. 6. Analysis including slip elements predicts porewater pressures very close to those recorded. If slip is not allowed during analysis, only very low residual porewater pressures are predicted.

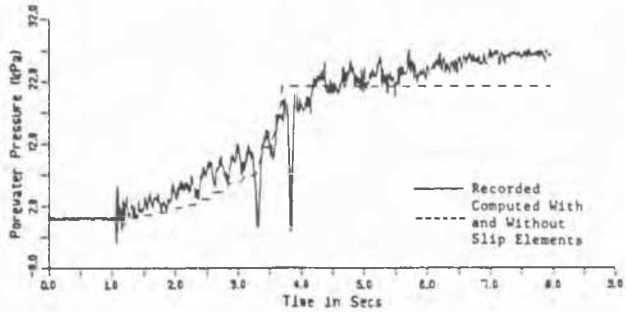

FIG. 5 Recorded and computed porewater pressure of PPT 2342 in Test 2.

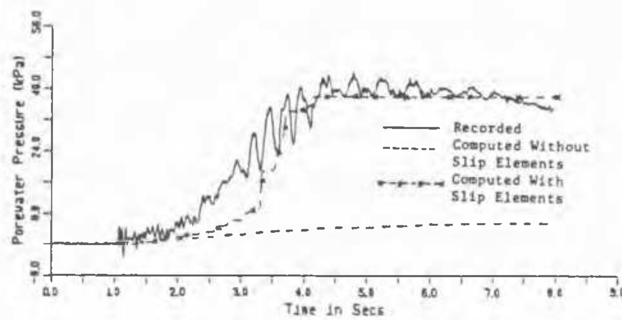

FIG. 6 Recorded and computed porewater pressure of PPT 68 in Test 2.

A complex part of the analysis is the proper transmission of accelerations from the embankment to the surface structure across the slip elements. In particular, the model should be capable of reproducing the rapid decay in acceleration after 4 sec of shaking due to the loss in shearing resistance because of very high porewater pressures. The recorded and computed accelerations are shown in Figs. 7(a) and 7(b). The decay in acceleration after 4 sec is modelled satisfactorily but three peaks in the recorded record are significantly higher than those computed. These higher accelerations are not surprising. In the course of the test, the surface plate underwent a slight embedment which was not modelled in the analysis. Such an embedment would provide for somewhat larger decelerations in each direction than the shearing resistance, alone, under the plate would do. The program is now being extended to include the analysis of embedded structures.

These preliminary results indicate that TARA-2 may prove useful in the dynamic effective stress analysis of soils and soil-structure systems.

CONCLUSION

Dynamic non-linear effective stress analysis has the capability to develop a coherent picture of dynamic response of soil structures including soil-structure interaction effects. It can compute directly permanent deformations and seismically induced porewater pressures as well as taking into account continuously during

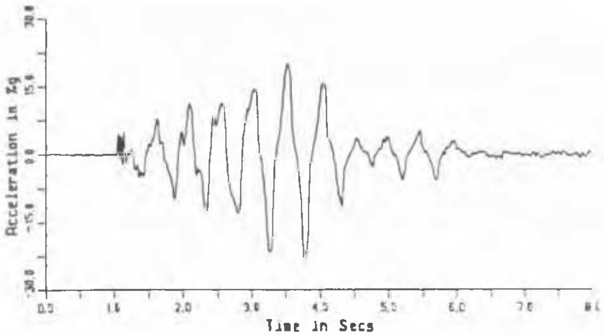

FIG. 7(a) Recorded acceleration of ACC 734 in Test 2.

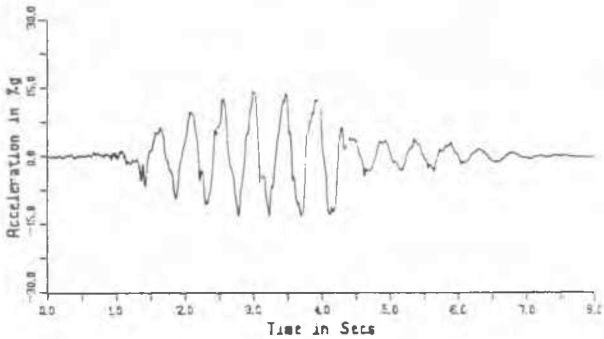

FIG. 7(b) Computed acceleration of ACC 734 in Test 2 (with slip elements).

analysis the effects of porewater pressures on soil properties. Particularly important is the ability to take relative motions between soil and structure into account. Such motions have been shown to exert a profound effect on the development of porewater pressure and shear strains.

Preliminary data from simulated earthquake tests on centrifuged models indicate that the method of analysis incorporated in TARA-2 may prove useful in dynamic effective stress analysis. Comparisons between predicted and measured accelerations and porewater pressures so far are quite good. More detailed verification studies using a variety of centrifuge models are now underway and are expected to be completed in 1985.

ACKNOWLEDGEMENTS

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