Transactions of the **14th International Conference on Structural Mechanics in Reactor Technology** (SMiRT 14), Lyon, France, August 17-22, 1997

K09/3

Comparisons between effective stress and total stress methods for seismic ground analysis

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ABSTRACT : A series of seismic ground response analyses with three different methods including total and effective stress approaches is presented. The analyzed soil profile consists of a 68 m thick layered-deposit of saturated sands, silts and gravels overlaying a mud stone. Material parameters were determined based on extensive in-situ investigation and a series of laboratory tests on undisturbed soil samples. The effects of the excess pore pressures on the ground response were evaluated through a comparison between total stress analyses, effective stress analyses in which only two upper saturated sand and gravel layers were considered by the effective stress approach and an effective stress analysis in which all saturated layers above the mud stone were able to generate excess pore water pressures. In this manner, effects of the liquefaction in the sand layer as well as the pore pressure development and additional softening of the silt layers were assessed.

1. INTRODUCTION

Among seismic ground response analyses, the equivalent linear method which is based on the wave propagation theory (SHAKE), is the most frequently used due to its simplicity. However, non-linear, step-by-step integration, total stress methods provide more accurate results especially in the case when significant nonlinearity and development of large shear strains are induced by strong earthquakes. Recently, effective stress methods, which in addition account for the effects of the excess pore water pressures on the ground response, are developed and used. This paper presents a comparison between these methods by using 1-D analyses of a given site.

The analysed soil profile consists of silt, sand and gravel layers with a thickness of over 60m. Equivalent linear analyses, nonlinear total stress analyses and effective stress analyses were carried out based on the soil properties obtained from extensive in-situ investigation and laboratory tests on undisturbed soil samples from the site. Parameters of the constitutive equations used in the effective stress method (Stress - Density Model [1]) were also obtained from the laboratory tests on in-situ samples.

The results of the analyses were studied through a comparison of the computed accelerations, displacements, shear stresses, shear strains and other relevant response properties.

2. SOIL PROPERTIES

The analyzed soil profile consists of a 68 m thick layered-deposit of sands, gravels and silts, with shear wave velocities in the range between Vs = 160 and 400 m/s. Below these layers lie a mud stone with shear wave velocities exceeding 400 m/s. The water table is approximately 6 m below the ground surface. The characteristics of the soil profile throughout the depth are listed in Table 1.

Table 1 Soil profile and soil properties

G.L. (m)		Unit weight	Shear wave velocity	Shear modulus
0	Soil type	γ (t / m^3)	Vs (m / sec)	G_0 (tf / m^2)
-4	Fill	1.9	170	5600
-10	Sand 2	1.8	220	8900
-12	Gravel 2	1.8	250	11500
-24	Silt 4	1.7	160	4400
-28	Sand 1	1.9	290	16300
-38	Silt 3	1.7	160	4400
-53	Silt 2	1.7	220	8400
-58	Gravel 1	1.8	250	11500
-63	Silt1	1.8	200	7300
-68	Gravel 1	2.0	400	33000
-78	Mud stone	1.7	400	28000
	Mud stone	1.7 ~ 1.8	440 ~ 630	33000 ~ 73000

Figure 1 shows the input motion used in the numerical analyses. It is a synthetic motion which has an intensive part of approximately 25 seconds and a peak acceleration of 310 gals.

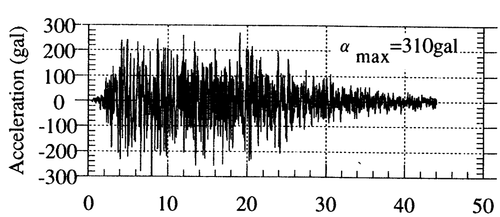


Fig.1 Input motion

3. METHODS OF ANALYSES

Three different methods of analyses, including total stress and effective stress methods, were used to evaluate the seismic response of the considered profile. The main characteristics of the employed methods are briefly described in the following.

3.1 Equivalent Linear Method (Frequency domain)

This is a widely used total stress procedure for ground response analysis (SHAKE) in which the soil is represented by a damped equivalent linear model. An equivalent shear strain (usually defined as 65 % of the maximum shear strain) represents the strain response of the soil and is used to select the corresponding shear modulus and damping according to a given strain-dependent soil properties. These shear modulus and damping are constant and assumed to be relevant at any stage of the load application. Since this is an elastic analysis with degraded shear modulus, permanent deformation can not be accounted for, i.e., upon cessation of the motion, the system returns to its initial, non-deformed, position.

3.2 Nonlinear Total Stress Method

This is a total stress method executed incrementally in time domain, in which a nonlinear elastoplastic stress-strain relation of the soil is used. The skeleton curve is a modified hyperbolic relation [1], whereas a hardening rule that resembles the Massing rule is employed for the cyclic part of the response.

3.3 Nonlinear Effective Stress Method

In addition to the soil nonlinearity, the effects of the excess pore water pressures are considered in this method. An elastoplastic model for soil, called Stress-Density Model (S-D Model [1]), which is incorporated in a FEM code for effective stress analysis of a two-phase medium (DIANA-J*) was used in these analyses. The elastoplastic constitutive model combines two fundamental approaches for sand characterization and modelling, i.e., the state concept [2] and the modified elastoplasticity. A modified hyperbolic stress-strain relation and an energy-based stress-dilatancy relation are used as a basic relations that embody the stress-strain-dilatancy link into the model. The unique feature of the model is that the stress-strain parameters are correlated to the relative state of the sand with respect to some characteristic states of sand behaviour. Details of the constitutive model can be found elsewhere [1].

4. MATERIAL MODELLING

Material parameters were determined based on results from in-situ investigation and laboratory tests on undisturbed soil samples. In addition to the data listed in Table 1, experimental relationships of the shear moduli G and damping ratio h with the shear strain γ, G-γ and h-γ relationships respectively, were obtained for each soil layer and used in the equivalent linear analysis. These data was also used to evaluate the stress-strain relationships of the soil layers in the nonlinear total stress analysis.

A series of tests on undisturbed samples was conducted to determine the material parameters of the elastoplastic constitutive model (S-D Model) used in the effective stress analyses. This model has four group of parameters which are the elastic, state index, stress-strain and stress-dilatancy parameters. The elastic parameters were determined based on the data listed in Table 1.

A series of monotonic undrained triaxial compression tests (CU) and drained triaxial compression tests (CD) on undisturbed samples were used to evaluate the reference lines for the Sand 1 and Gravel 2 layers. Since the samples did not exhibit drop in the shear stress or quasi steady state [2], the quasi steady state line was replaced with the steady state line as a reference for determination of the state index *Is*. In other words, for practical purpose, it was assumed that the quasi steady state and the steady state line are identical. Figure 2 shows with the symbols the initial states and the steady states attained in the tests on the sand samples (Sand 2), whereas the solid lines represent the approximated reference lines. The in-situ void ratio was estimated based on the in-situ stress state and the experimentally obtained consolidation path shown with the dashed line in Fig. 2. The stress-strain parameters of the model, i.e., the initial plastic shear moduli and the peak stress ratio, were defined from the measured stress-strain curves in the drained tests (CD tests).

Finally, the dilatancy parameters of the S-D Model, which control the pore pressure development, were determined by simulations of measured cyclic strength curves or liquefaction resistance. Figure 3 shows with the symbols the cyclic strength of the sand and the gravel for 1, 2 and 5 % double amplitude shear strain, measured in cyclic undrained triaxial compression tests. The lines in these figures show the simulated liquefaction resistance with the elastoplastic model. As illustrated in Fig. 3, particular attention was given to accurately simulate the cyclic strength for different strain levels (from 1-5 %), unlike the conventional approach that consider only the 5 % double amplitude shear strain.

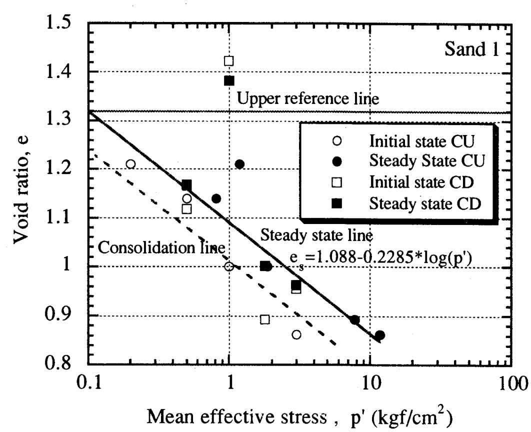


Fig.2 Determination of reference lines (Sand 1)

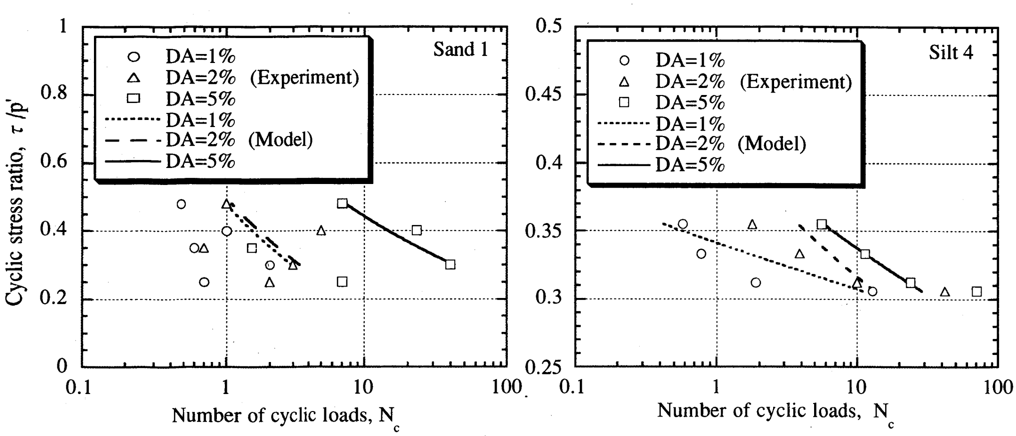


Fig.3 Experimental and simulated liquefaction strength curves for Sand1 and Silt 4

Typical effective stress paths and stress-strain curves from the element test simulations with the model are shown in Fig. 4 and Fig. 5., for sand and silt materials respectively.

As illustrated in Fig.4, the pore water pressure build-up in the sand is very fast associated with a double shear strain amplitude of about 3%. Afterwords, however, there is gradual development of the shear strain with the number of cyclic loads. This characteristic of the behaviour in the cyclic mobility is the cause for the large horizontal distance between the cyclic strength curves for 1 % (2 %) and 5 % shear strain , shown in Fig.3.

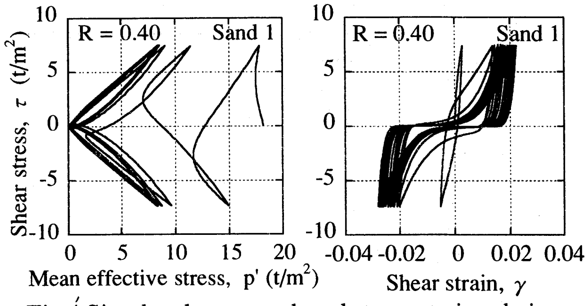


Fig.4 Simulated stress path and stress-strain relation for Sand 1

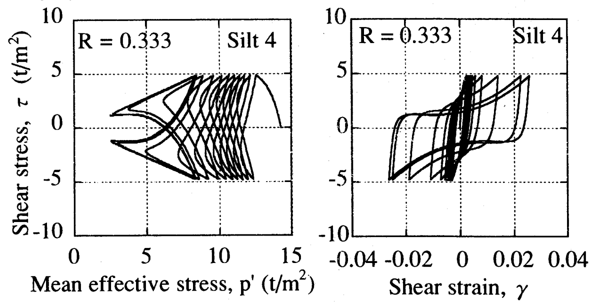


Fig.5 Simulated stress path and stress-strain relation for Silt 4

On the other hand, silt behaviour is characterized with a continuous shear strain and pore water development, with a limitted pore pressure build-up. This silt behaviour is different from the liquefaction of sand, because sudden increase in the deformation does not occour.

5. NUMERICAL ANALYSES

5.1 Numerical Model

In the nonlinear total stress and effective stress analyses a plane strain FEM model was used. The soil-column consisted of four node elements with four degrees of freedom at each node and a base damper. The alluvial layers, up to depth of 68 m, were model with a 1m thick elements, whereas the mud stone layers from 68m to abou 400 m depth were modeled with 34 elements with thicknesses between 4m and 10m. Similar stratification was used in the equivalent linear analysis conducted with SHAKE. The acceleration time history shown in Fig. 1, is the 'input' motion at depth of about 400 m.

5.2 Analysis Cases

Cases of the analyses and the corresponding methods of material modelling are listed in Table 2. Three nonlinear analyses which are a total stress analysis (Case-0), a total stress analysis with effective stress consideration of Gravel 2 and Sand 1 layers (Case-1) and an effective stress analysis (Case-2) were conducted. By comparing Case-0 and Case-1 analyses,

Table 2 Cases of analysis anf Material modelling

Cases	Material modelling
Case-0	· Non-linear total stress model for all layers
Case-1	· Non-linear effective stress model for Sand 1 and Gravel 2 · Non-linear total stress model for other layers
Case-2	· Non-linear effective stress model for Sand 1, Gravel 2 and Silt 1,2,3,4 · Non-linear total stress model for other layers
SHAKE	· Equivalent linear model

Soil	Case-0	Case-1	Case-2
Fill	TS	TS	TS
Sand 2	TS	TS	TS
Gravel 2	TS	ES (pwp)	ES (pwp)
Silt 4	TS	TS	ES (pwp)
Sand 1	TS	ES (pwp)	ES (pwp)
Silt 3	TS	TS	ES (pwp)
Silt 2	TS	TS	ES (pwp)
Gravel 1	TS	TS	TS
Silt 1	TS	TS	ES (pwp)
Gravel 1	TS	TS	TS
Mud stone	TS	TS	TS

TS: Total stress model
ES (pwp): Effective stress model
(Pore water pressure is considered)

it is intended to assess the effects of possible liquefaction in Gravel 2 and Sand 1 on the ground response. In a similar vein, the results from Case-1 and Case-2 evaluate the effects of the softening in the silt layers due to pore pressure build-up, on the site response. Finally, SHAKE analysis was used to compare the results from the above analyses with the conventional total stress method for seismic response analysis.

5.3 Results and Discussion

The results of the analyses are firstly discussed through a comparison of the maximum

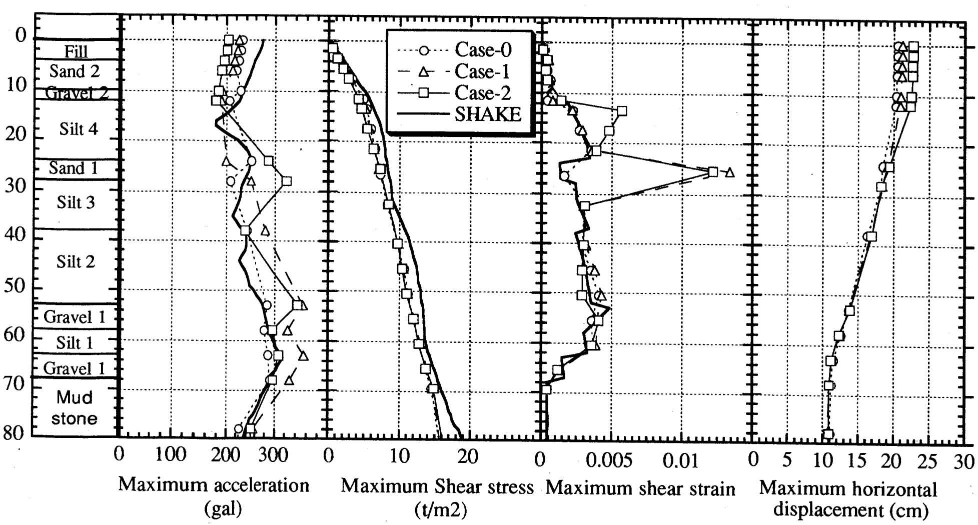


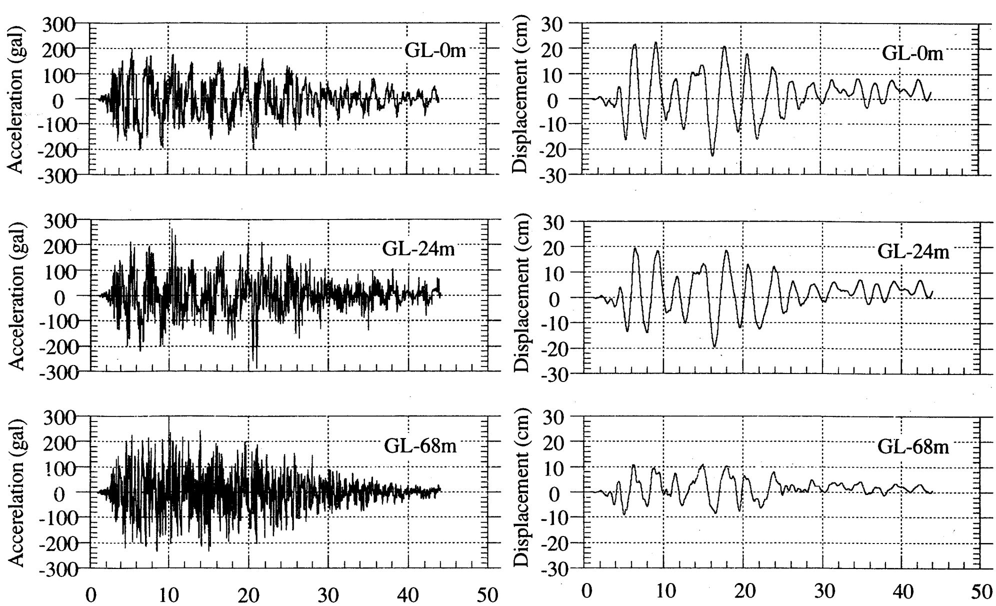
Fig.6 Maximum responses throughout the depth of the alluvial layers

responses, and subsequently some characteristics of the response obtained in the effective stress analysis (Case-2) are presented.

Figure 6 shows the distribution of the maximum accelerations, shear stresses, shear strains and horizontal displacements throughout the depth of the alluvial layers. The distribution of the maximum shear strains clearly illustrate the effects of the pore pressure build-up in the Sand 1 and Silt 4 layers that resulted in significant increase in the shear strain up to 1.2 % and 0.55 % respectively. On the other hand, there is a relatively small difference in the strain responses of the other layers. Thus, the increase in the horizontal displacements for Case-1 and Case-2 are due to the liquefaction in the Sand 1 layer and softening of the Silt 4 due to the pore pressure development. The relatively small increase in the horizontal displacement is due to the small thickens of the liquefied sand layer (4m thick) and the previously discussed characteristics of the response in cyclic mobility.

In general, there is a very good agreement between the results of the total stress analysis (Case 0) and SHAKE, except for some expected differences in the acceleration response. The larger maximum shear stresses in SHAKE analysis are pure consequence of the overestimation of the stiffness in this method regarding the maximum response, since 65 % of the maximum shear strain is used as a reference for the evaluation of the shear modulus.

In order to more clearly illustrate the characteristics of the response obtained in the effective stress analysis (Case-2), Figs. 7-10 show typical results from this analysis including effective stress paths, stress strain curves and excess pore pressure time histories for the



Figs.7 Time histories of horizontal acceleration and displacements for Case-2 at GL-0m, 24m, 68m.

liquefied sand layer (Sand 1) and Silt 4 layer. It is apparent that the characteristics of these materials touched upon in the material modelling are strongly reflected in the response.

6. CONCLUSIONS

A series of seismic response analysis including conventional total stress analysis as well as effective stress analysis are used to assess the effects of pore pressure development and the associated softening on the ground response of a given site. A good agreement is obtained between the results of the total stress methods. The effects of the pore pressure increase and eventual liquefaction are clearly reflected on the shear strain response and consequently on the horizontal displacements. It should be emphasized that quantitative evaluation of the effects of the pore pressure development and liquefaction can only be done if both the liquefaction resistance and the associated strain development are precisely modeled, for each layer that is expected to be affected by the excess pore pressures.

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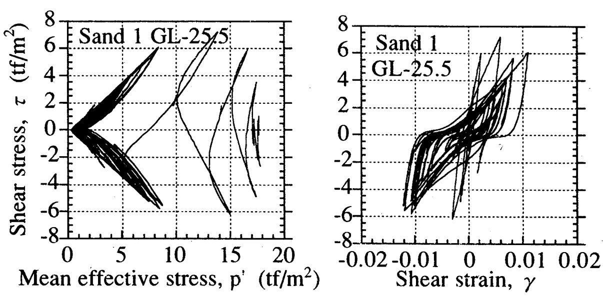

Fig.8 Stress path and stress-strain relation for Sand 1 (Case-2)

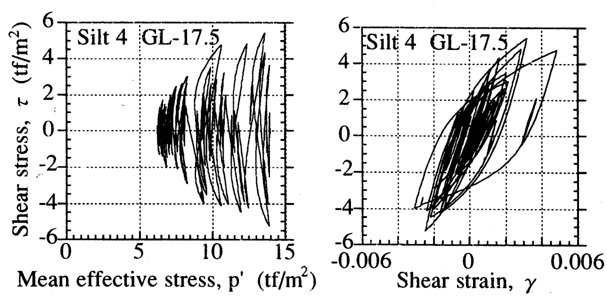

Fig.9 Stress path and stress-strain relation for Silt 4 (Case-2)

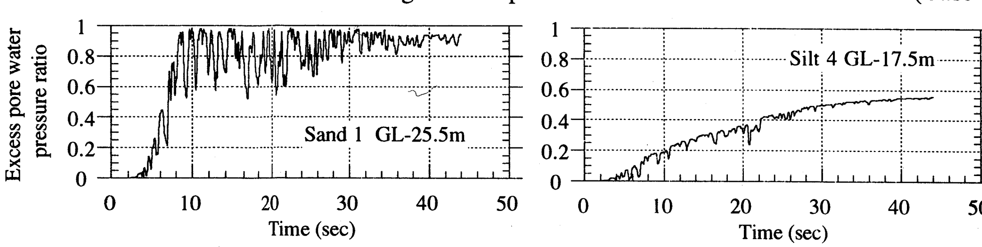

Fig.10 Excess pore water pressure ratio time histories for Sand 1 and Silt 4