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Shear stack tests on soil-structure interaction

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ABSTRACT: Statically cyclic loading and shaking table tests were performed in order to clarify nonlinear interaction behavior between Quaternary deposits and rigid structure using scale model. Furthermore, numerical simulation analyses regarding the experimental results were carried out. As results of the tests, influences not only on the material nonlinearity of soil but also on the buried depth of rigid block as a structure model were grasped. Besides, through the comparison between the experimental results and analytical results, applicable scopes of nonlinear FEM procedure were pointed out.

1 INTRODUCTION

The basic policy in Japan requires nuclear reactor buildings to be built on rock site. However, in order to increase available locations for nuclear power plants in future, the investigation of siting technology on Quaternary deposits has been carried out as the project of the Nuclear Power Engineering Center(NUPEC). Statical cyclic loading and dynamic (Shaking Table) tests concerning to soil structure interaction problem using scale concrete block model and real soil material in large shear stack were performed. The objective of these tests was to grasp the complicated behavior on static and dynamic interaction between Quaternary deposits and rigid structure, while it is observed material nonlinear phenomena in soil at middle or large strain level.

Besides, numerical simulation analyses for respective experimental results were worked out. The objective of these analyses was to validate the analytical method which will be utilized for seismic stability evaluation of reactor buildings, and to obtain valuable data with respect to the applicable scopes of these analytical methods.

This paper describes outlines of the test procedure and representative results of the tests and numerical simulation analyses.

2 TEST METHOD

2.1 Model Ground

Profiles of the model ground and concrete block are illustrated in Fig.1. Model ground was contained in a large scale shear stack (H=2.08m, L=4.3m, W=2.85m). The material used for model ground was dry silica-sand with an uniformity coefficient Uc of 1.8. Dry silica-sand was scattered uniformly in the shear stack by using an automatic scattering apparatus. Thereafter, model ground was compacted by vibration through shaking table until settlement due to vibration of ground had not been observed. Consequently, average unit weight γ_t of the compacted model ground was 15.3kN/m^3, and the relative density D_r was 86%.

2.2 Concrete Block

The height of cubic concrete block was 1m, equivalent to one half height of soil layer. The area of its bottom was 0.64m^2, equivalent to 1/20 of the total ground surface area.

The block was composed of outside concrete and inside iron ingot. Average unit weight of the concrete block was 43.5kN/m^3, contact pressure of the bottom was 47kPa. If the geometrical scale factor was set up to 1/10 of the large scale model block constructed in Tadotsu Engineering Laboratory(NUPEC), the contact pressure of the large block was 470kPa, and it was nearly equal to that of reactor buildings.

Buried depth of the concrete block were half height of it (Case D50), 1/5 height (Case D20), and non-buried(Case D0).

2.3 Loading Method

As to the static test, increasing cyclic step loading was applied to the center of gravity of the concrete block. Horizontal step loading was applied by 50kN actuater, while an increment of the step loading was 980N, each step loading had been held for 3 minutes.

In the dynamic test, ground and concrete block model in the large shear stack was shaked sinusoidal and real earthquake waves by shaking table. As a real earthquake wave, El Centro 1940NS record was used, while time scale was adjusted to 1/6 of real recorded wave based on the similitude rule. The intensity of input motion was taken as a testing parameter, in order to comprehend the influence on material nonlinearity of soil.

2.4 Measuring Method

In the static test, the primary measurement terms were displacements and earthpressure at bottom and side of the concrete block as shown in Fig.1.

In the dynamic test, strain gage type accelerometers were installed in the model ground and on the concrete block to obtain the amplification of them as shown in Fig.2. Besides, dynamic earthpressure transducers were also installed.

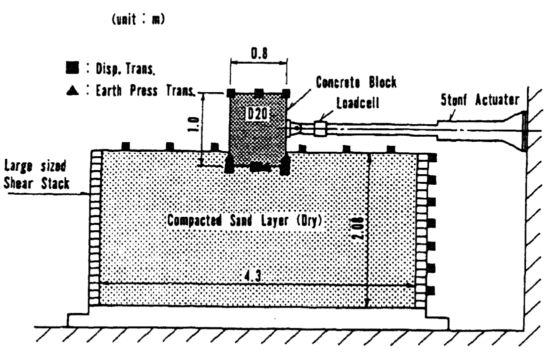

Fig.1 Profile of the static test

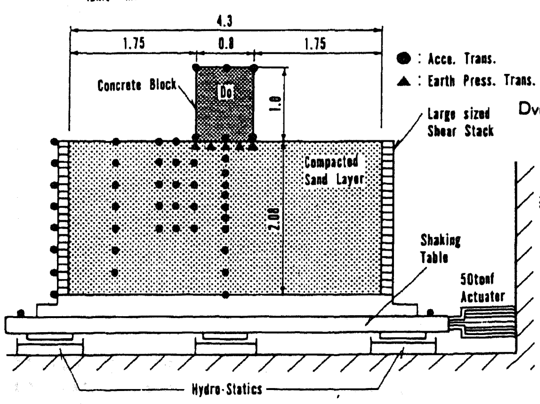

Fig.2 Profile of the dynamic test

3 TEST RESULT

3.1 Static cyclic loading Test

Relationships between horizontal load and displacements of the concrete block are shown in Fig.3. The behavior of respective measuring points presents a spindle shape, which indicates the influence of material nonlinear phenomena of soil apparently. The residual settlement of the concrete block due to cyclic loading were observed.

The horizontal and vertical displacements of the last loading cycle are shown in Fig.4. It is found that modulus of displacement of the concrete block apparently increases, and the residual displacement of that decreases in proportion to the buried depth.

Distribution of the incremental earth-pressure due to the horizontal loading is shown in Fig.5. Regarding to the bottom of the concrete block, the incremental earth-pressure becomes smaller as buried depth increased. In case of the side pressure, the same tendency was more clearly observed.

Regarding to the buried case D20 and D50, the behavior of displacement up to failure (i.e. overturning) is shown in Fig.6. The maximum loads at failure point were 21kN and 29kN for D20 and D50, respectively. Prior to the test, the overturning load had

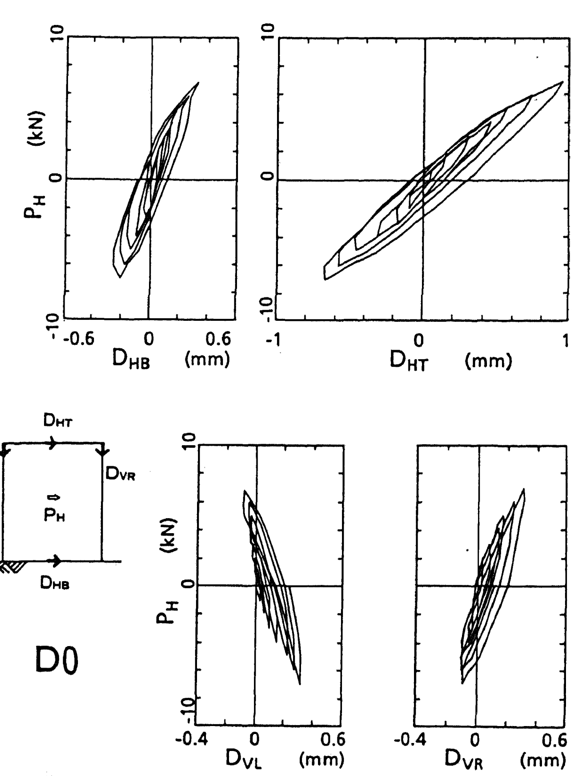

Fig.3 Load-displacement curves [D0]

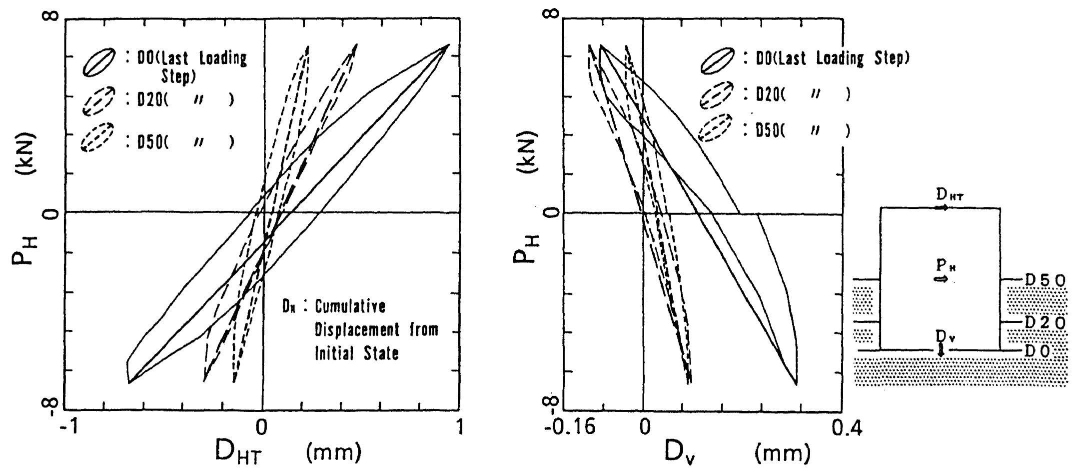

Fig.4 Load-displacement curves

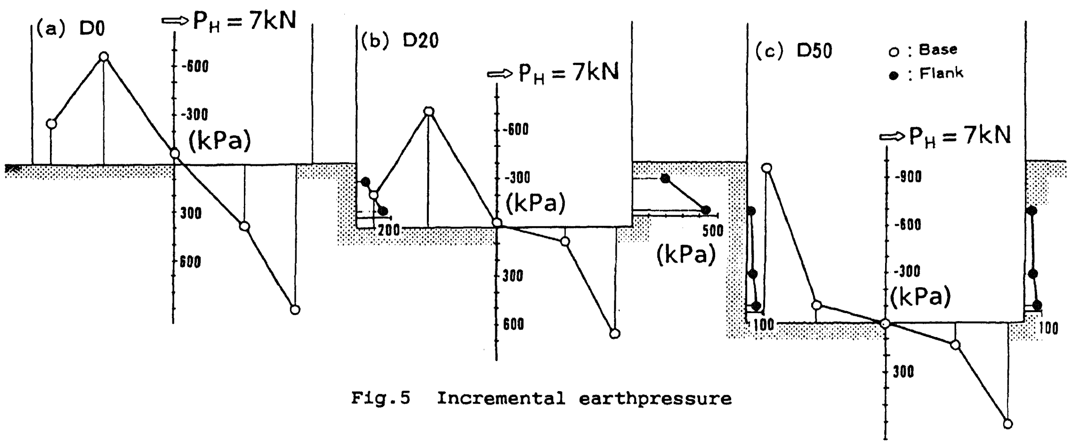

Fig.5 Incremental earthpressure

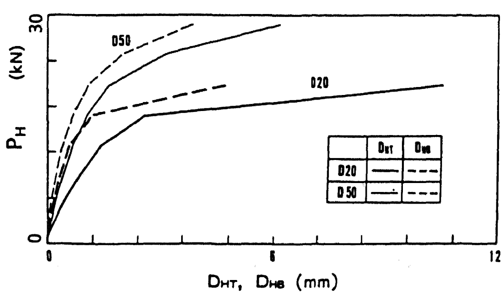

Fig.6 Overturning behavior

been predicted by using the circular arc method. The predicted overturning load was 41kN for case D20 and 58kN for case D50, when internal friction angle of 41° was substituted owing to the result of soil element test. Namely, each predicted overturning load by conventional method was about twice strength of the observed value. Consequently, the internal friction angle was adjusted to be 2/3 of measured value, then predicted failure strength of the each cases coincided with experimental result.

3.2 Shaking table test

Resonant curves at top of the concrete block are drawn in Fig.7. Except input intensity of 0.8m/sec^2, two resonant peaks are obviously observed. Due to material non-linearity of soils, the maximum amplitude and resonant frequency decrease with intensity of the input motion. In case of 0.8m/sec^2 input motion, the shape of resonant curve about 1st peak is rather flat due to not only material nonlinearity but also geometrical nonlinear behavior such as slide and separation.

Respective degree of resonant frequencies increase according to buried depth increased. However, the maximum amplitude was not in proportion to the buried depth, and the amplitude at case D20 shows the largest among the present test cases. If the inherent resonant frequency of the ground closes to the rocking resonant frequency of the concrete block in case D20, it seems that both resonant frequency stimulates each other, and it makes the amplitude of case D20 larger.

Fig.8 shows the 1st and 2nd resonant modes. The 1st mode of case D0 and 2nd mode of the buried cases are estimated as a rocking mode of concrete block. Conversely, The 2nd mode of case D0 and 1st mode of the buried case was estimated as a coupling mode of ground and concrete block. Thus, it could be said that the characteristic of vibration mode is affected by buried depth.

The amplitude of dynamic earthpressure was similar to the distribution of incremental earthpressure in the static test as shown in Fig.5.

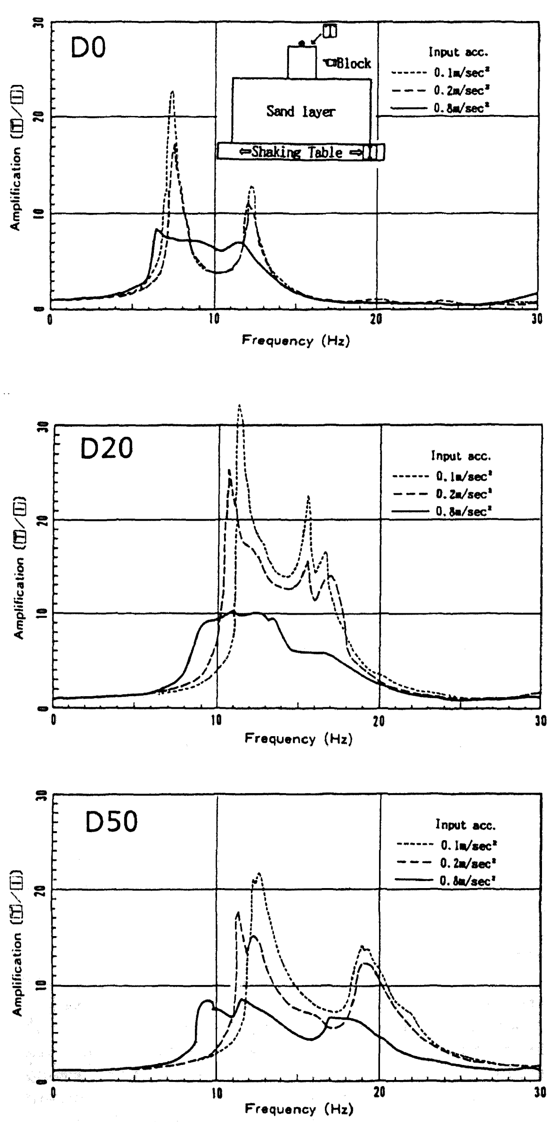

Fig.7 Resonant curves

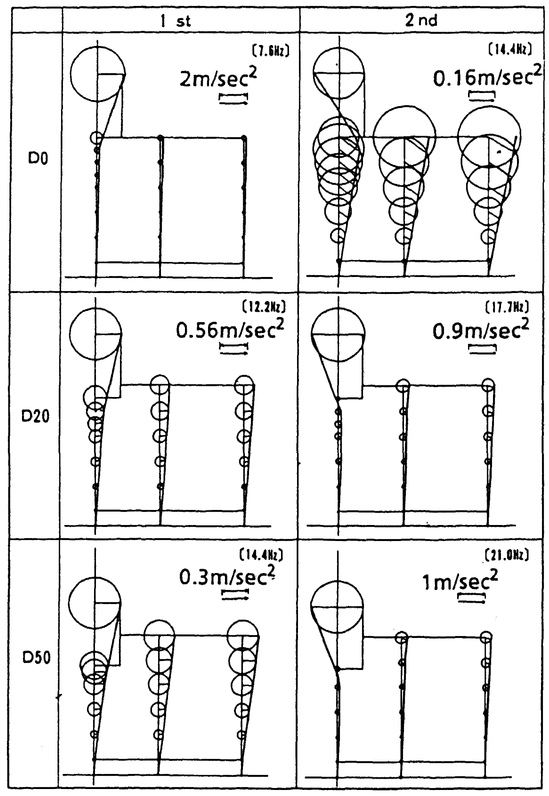

Fig.8 Resonant modes

4 NUMERICAL SIMULATION ANALYSIS

4.1 Nonlinear static analysis

2-dimensional nonlinear FEM analysis was applied for the static test. Duncan-Chang Model was used for a nonlinear constitutive law of ground as given in Equation-(1).

$$E_t = \left\{ 1 - \frac{R_f/(1-\sin\phi)(\sigma_1-\sigma_3)}{2\,C(\cos\phi)} \right\}^2 K\,P\,(\sigma_3/P)^n \quad (1)$$

where, σ_1, σ_3 :Major & minor principal stresses
P :Atomosphic pressure (MPa)
C, ϕ : strength parameters (ϕ = 41° ,C=0)
K, R_f, n:D-C Model parameters
(K =26.5MPa, R_f=0.85~0.95, n=0.6)

Strength parameters of C, ϕ and inherent parameter of K were principally estimated from the results of soil element test.

The analytical relationship between load and displacement of case D0 is shown in Fig.9 togather with the experimental result. When parameter R_f was taken between 0.85~0.9, the analytical result coincided well with that of the experiment. It is found that secant modulus of K has a large influence on the analytical result, however from the comparison as shown in Fig.9, it could be estimated by conventional triaxial test of sampled soil specimen.

Fig.10 draws a result of the comparison up to the overturning state of the concrete block. The overturning load could not be determined by FEM method, however Fig.10 shows good agreement between the analytical result and that of experiment up to about 60% of the overturning load.

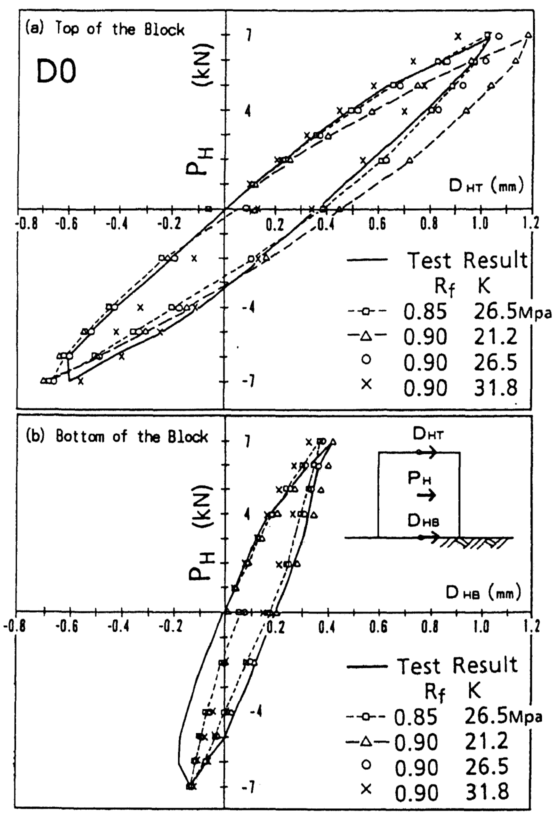

Fig.9 Analytical load-displacement curves

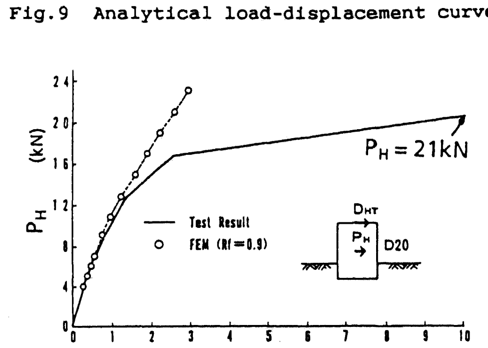

Fig.10 Analytical overturning behavior

4.2 Earthquake response analysis

Earthquake response analysis for the dynamic test was curried out by means of the Equivalent Linear Method (ELM) in frequency domain. The dynamic properties of soils such as compatible shear modulus and damping factor corresponded to shear strain level was determined by using Hardin-Drnevich model(H-D model) as given in Equation-(2). Parameters of H-D model were determined by Dynamic Back Analysis, using the results of resonant tests with respect to the ground model alone.

$$G = 20\sigma_m^{0.6} / (1 + \gamma / 3.3\times10^{-4}) \qquad (2a)$$

$$h = 19.6 / (3.3\times10^{-4} / \gamma + 1) \qquad (2b)$$

where, G :Compatible Shear Modulus(MPa) corresponding to Strain of γ, h :Critical damping ratio(%), σ_m :Average confining pressure (MPa), γ :Effective shear strain

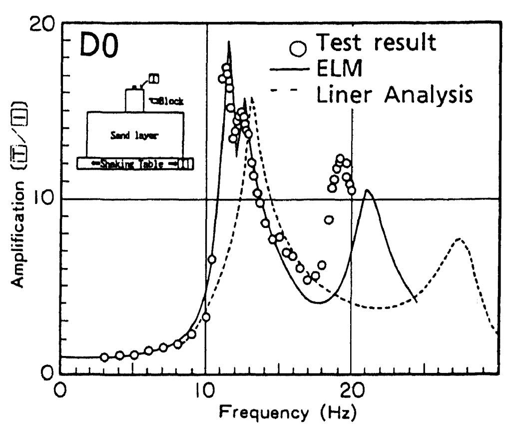

Fig.11 Comparison of resonant curves

Fig.11 presents the resonant curves at top of the concrete block of buried case D50. Regarding to the 1st resonant frequency, the analytical results of ELM agree with the test results, whereas the linear analysis dose not express the test results. In regard to the 2nd resonant peak, the coincidence between ELM and test result declined comparing with that of the 1st resonant peak.

Fig.12 shows response acceleration waves at top of the concrete block in the case D50. At various input intensities, respective analytical results agree approximately with the test results. Fig.13 shows the response spectrum in the case D20, and it indicates the same tendency in frequency domain.

Fig.14 illustrates the distribution of the maximum dynamic shear strain in ground. In the case D50 at 3.2m/sec input motion, the maximum shear strain attained up to 3.7×10^{-3}. In this case, the region in which exceeded shear strain of 10^{-3} occupied approximately more than 70% of the total ground region. From the other point of view, the reduction ratio of shear modulus against the initial state was about 80%, in the half of ground region. Owing to the good agreement between

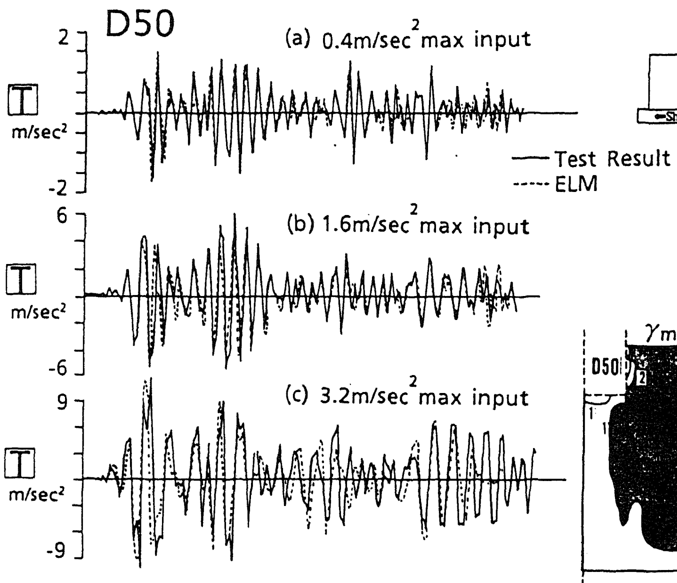

Fig.12 Responded accelerogram

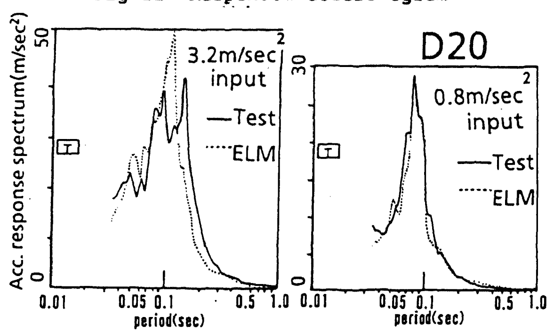

Fig.13 Acceleration response spectrum

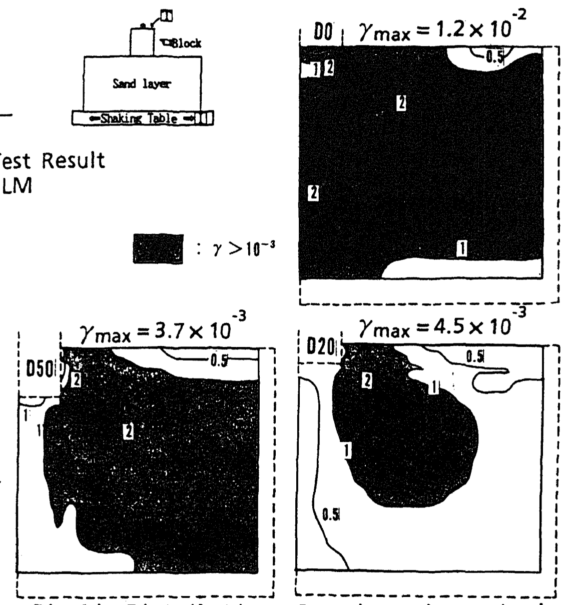

Fig.14 Distribution of maximum shear strain

the present method and test, these values indicate ranges of the applicability of the ELM as far as this kind of interaction problems.

5. CONCLUSION

Based on the results of the test and simulation analysis, the following principal conclusions were obtained.

1. The modulus of displacement against holizontal loading increase apparently with the buried depth. Contrary, the amount of residual settlement decreases with that.
2. With increasing intensity of the input motion, the resonant frequencies and maximum amplitudes obviously decrease due to the material nonlinearity of soils.
3. With buried depth, characteristics of resonant modes clearly change from rocking mode to coupling mode with ground and concrete block.
4. The analytical results which were calculated by using Duncan-Chang model, agree well with the test results up to 60% of the overturning load. Parameters in the constitutive model of the present method were estimated by conventional soil specimen tests, whereas secant modulus — K especially affects the analytical result.
5. While material nonlinearity of soil affects soil structure interaction behavior, the present method can simulate as far as this kind of interaction.
6. The maximum shear strain in the buried case attained up to 3.7×10^{-3}. It seems that shear strain of 3.7×10^{-3} was one of the range of the applicability for the mentioned subject.

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