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Shoei Nomura
Tokyo Institute of Technology, Tokyo, Japan

Yasuhiro Shamoto
Shimizu Co., Ltd., Tokyo, Japan

Kohji Tokimatsu
Tokyo Institute of Technology, Tokyo, Japan

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Shoei Nomura
Graduate Student, Tokyo Institute of Technology, Tokyo, Japan

Yasuhiro Shamoto
Research Engineer, Shimizu Co., Ltd., Tokyo, Japan

Kohji Tokimatsu
Associate Professor, Tokyo Institute of Technology, Tokyo, Japan

SYNOPSIS: An analytical method is presented for evaluating dynamic response of soil-pile-structure system during soil liquefaction. The method consists of a modified Penzien's model combined with an effective stress analysis for free-field soil response and a horizontal subgrade reaction model which connects free field response with pile-structure response. Shaking table tests are conducted to study the effectiveness of the proposed procedure. A model sand deposit-pile-structure system is constructed in a large container which can permit shear deformation of the soil. Soil density, pile diameter, pile rigidity, and input motion are controlling variables in the tests. Analysis is made for both liquefied and non-liquefied cases, and the results are compared with the measured values. The analytical results including the time histories of excess pore water pressures, accelerations, and displacements, and the Fourier spectra of the ground surface and the pile head, are all in good accord with the observed values, showing that the proposed analysis is effective.

INTRODUCTION

Despite the increase use of pile foundations for mitigating soil liquefaction hazards of various structures, our understanding of its behavior seems quite restricted. This is partly because there exists few case histories in which the failure of pile foundation is described in details. Although a number of shaking table tests has been conducted on model soil-pile-structure systems to understand their behavior during liquefaction, most of the results are qualitative and hence cannot directly be reflected in the practice.

Consequently, there is few dynamic response analyses which can reliably estimate the behavior of soil-pile-structure systems during soil liquefaction. Further their effectiveness has seldom been verified by comparison with behavior of prototype structures or even model structures in the laboratory.

The object of the paper is to present a relatively simple method of analysis for simulating the dynamic response of a pile-supported structure during soil liquefaction. The effectiveness of the proposed method is discussed with the results of shaking table tests conducted on a model soil-pile-structure system.

PILE-STRUCTURE RESPONSE ANALYSIS

A modified version of the analytical method proposed by Penzien et al. (1964) is used for analyzing soil-pile-structure interaction since it can easily be extended into non-linear problems. A pile-supported structure and the surrounding soil deposit are assumed to be a single-pile system and a one-dimensional soil column system, as shown in Fig. 1. The mass of each system is lumped at discrete points along a vertical axis. Each discrete mass has horizontal one degree-of-freedom and is connected with the adjacent one in the same system by a spring which has an idealized stress strain relationship of each system.

Discrete masses at the same elevation in the two system are connected with each other by a spring which holds an idealized load deflection relationship of the modeled pile.

The equation of motion for the model pile-structure system is given by:

$$M_p\ddot{x}_p + P_p\dot{x}_p + K_px_p + M_e(\ddot{x}_p-\ddot{x}_s) + C_e(\dot{x}_p-\dot{x}_s) + K_e(x_p-x_s) = -\ddot{x}_G M_p I \quad (1)$$

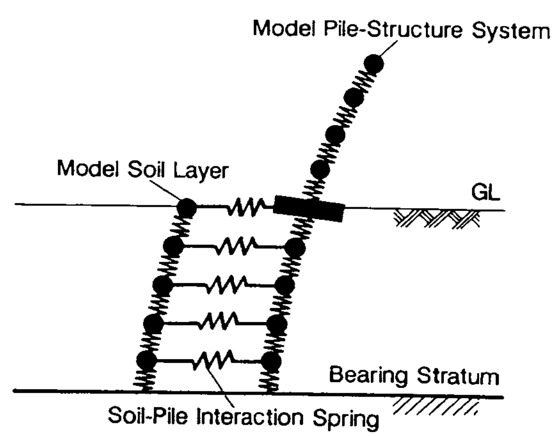


Fig. 1 Idealized model for soil-pile-structure system

in which $\mathbf{M}$ = lumped mass matrix, $\mathbf{C}$ = damping matrix, $\mathbf{K}$ = stiffness matrix, $\mathbf{I}$ = unit matrix, $\mathbf{x}$ = displacement vector relative to the base, and suffix p represents the pile-structure system, s the free-field soil system, e the soil-pile interaction, and G base input motion.

In the second line of Eq. (1), the free-field soil response and the horizontal subgrade reaction of pile are incorporated into the response of pile-structure system through $\mathbf{x}_s$ and $\mathbf{K}_e$. Both values tend to decrease with increasing pore pressure as well as increasing shear strain, which tendency must be taken into account in the analysis.

The free-field soil motion may separately be obtained by various analysis methods, and the pile-structure motion is computed based on the free-field soil motion. The solution of Eq. (1) in the time domain uses the Newmark's β method (β = 0.25).

FREE FIELD SOIL RESPONSE ANALYSIS

One-dimensional effective stress analysis method proposed by Shamoto and Shimizu (1986) is used for analyzing free field soil response during soil liquefaction. The method is similar to that proposed by Finn et al. (1977), except the following two points:

(1) The effects of soil densification during cyclic loading are neglected.
(2) The behavior of cyclic mobility is appropriately simulated.

The method can take into account the degradation of the stress strain relationship of soil due to pore water pressure generation with the following models:

Pore Pressure Model

The rate of generation of pore pressures in a soil deposit due to cyclic loading is estimated by extending the following empirical equation proposed by Seed et al. (1976).

$$\frac{u_g}{\sigma_o'} = \frac{2}{\pi}\arcsin(R_N^{1/2af}) \qquad (2)$$

in which u_g = excess pore water pressure, σ_o'= initial effective stress, af = a function of the soil properties and stress conditions, and r_N = cycle ratio defined by the ratio between the number of applied stress cycles N to the accumulative number of cycles required to cause initial liquefaction N_L.

Unlike the laboratory test conditions in which a constant amplitude of cyclic loading is used, the loading conditions in the field during earthquake are random in nature, which requires the evaluation of r_N in Eq. (3) by the following equations:

$$r_N = \Sigma A[(\frac{1}{N_i}) - (\frac{1}{N_{i-1}})] \qquad (3)$$

$$N_i = 2C_3(\frac{|\Delta\tau|}{2\sigma_o'C_1})^{1/C_2} \qquad (4)$$

in which A=1, $C_1 = (1/20)^{C_2} R_{20}$, C_2 = -0.25, = 1.0 and R_{20} = shear stress ratio causing initial liquefaction at 20 cycles. Eq. (4) c be derived assuming that the relationship between the stress ratio and the number of cycl causing liquefaction is linear on a log-log plot. $\Delta\tau$ in Eq. (4) is defined by an increment shear stress from the latest turning point of stress strain relation of soil in cyclic load ing.

It is known that dense sands tend to dilate subjected to large amplitudes of shearing. is called cyclic mobility behavior and charac terized by a pore pressure drop during shear when the sand is at or near failure. Such a change in pore pressure cannot be estimated Eq. (3).

The rate of pore water pressure change once after the stress strain relation of soil hit the phase transformation line may approximat be given by:

For $|\tau/\sigma'| = M_f$

$$\Delta u_g = -|\Delta\tau|/M_f \qquad (5)$$

For $M_o < |\tau/\sigma'| < M_f$

$$\Delta u_g = -\theta_{max}\frac{|\tau/\sigma'| - M_o}{M_f - M_o}|\Delta\tau| \qquad (6)$$

in which σ' = current effective stress, M_f = tangent of failure line, M_o = tangent of pha transformation line, and θ_{max} = a function t governs the positive dilatancy characteristi of sand and is assigned a value between 0 an 2.5 depending on such factor as soil density

For $|\tau/\sigma'| < M_o$, the rate of pore pressure change may be estimated by Eq. (3) with A-va which is greater than 1 and increases with application of shear stress (Shamoto and Shimizu, 1986).

Stress Strain Relationship

The stress strain relationship of soil duri liquefaction is defined by the Ramberg-Osgo model. Its skeleton curve is defined by:

$$\tau = \frac{G_o'\gamma}{1 + a|\tau/\tau_r'|^b} \qquad (7)$$

in which a=1, b = a function controlling th degradation of shear modulus with shear str G_o' = initial shear modulus under current e tive stress, τ_r' = reference shear stress u current effective stress. These values may defined by:

$$b = (2 + \pi h_{max})/(2 - \pi h_{max}) \qquad (8)$$

$$G_o' = G_o(\sigma'/\sigma_o')^{0.5} \qquad (9)$$

$$\tau_r' = \tau_r(\sigma'/\sigma_o') \qquad (10$$

in which h_{max} = maximum damping ratio of soil, G_o = initial shear modulus under initial effective stress, τ_r = reference shear stress under initial effective stress (= $G_o\gamma_r$), and γ_r = reference shear strain. The hysteresis loop is determined by the Masing's law.

HORIZONTAL SUBGRADE REACTION OF PILE

The lateral load-deflection relationships of piles, i. e., the relationship between horizontal subgrade reaction of pile, P, and relative displacement between soil and pile, y, has been extensively studied by many investigators with various techniques. For example, the study by Kagawa and Kraft (1980) gives a sophisticated load-deflection relationship which includes non-linear behavior of soils. However, their study cannot directly be used for soils involving liquefaction conditions.

The load-deflection relationship used in this study is also modeled by the Ramberg-Osgood's model in which the effects of decreasing effective stress as well as increasing strain level during liquefaction can be taken into account.

The skeleton curve of the load-deflection relationship is defined by:

$$P = \frac{K_{ho}'y}{1 + a\left|P/P_r'\right|^b} \tag{11}$$

in which a and b are constants and assigned the same values as used in Eq. (7), and K_{ho}' and P_r' are effective coefficient of horizontal subgrade reaction of pile and effective reference subgrade reaction under current effective stress as defined by:

$$K_{ho}' = K_{ho}\,(\sigma'/\sigma_o')^{0.5} \tag{12}$$

$$P_r' = P_r\,(\sigma'/\sigma_o') \tag{13}$$

in which K_{ho} = coefficient of horizontal subgrade reaction under initial stress, and P_r = reference subgrade reaction under initial stress. The hysteresis curve is again defined by the Masing's rule.

SHAKING TABLE TEST

Apparatus and Model Soil-Pile-Structure System

Shaking table tests were performed to study the effectiveness of the proposed analysis. As shown in Fig. 2, a model soil-pile-structure system was constructed in a large shear container 4 m long, 2 m wide, and 2 m deep. Consisting of a stack of twenty five 8 cm-thick steel frames, it can allow shear deformation of the inside soil. A rubber membrane attached inside the container makes the system waterproof. A layer of coarse sand was glued on the bottom surface of the container to prevent the model ground from slippage.

The sand used is a clean silica sand, of which physical properties are listed in Table 1.

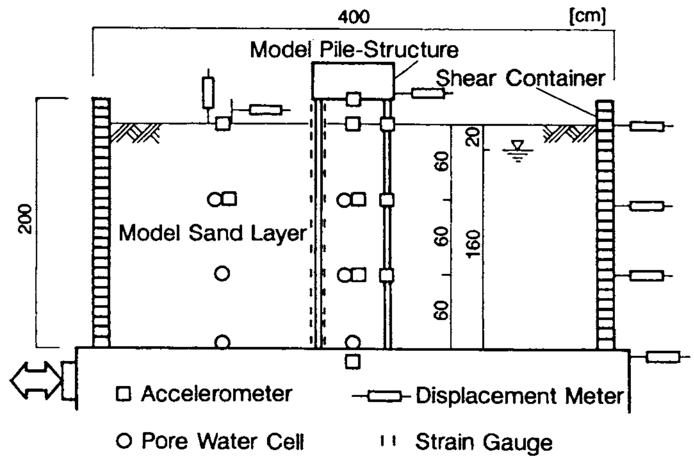


Fig. 2 Shaking table test apparatus and test arrangements

Table 1 Physical properties of test sand

Item	Property
Specific Gravity of Soil Particle	G_s = 2.651
50 % Diameter of Soil Particle	D_{50} = 0.28 [mm]
10 % Diameter of Soil Particle	D_{10} = 0.17 [mm]
Uniformity Coefficient	U_c = 1.71
Maximum Dry Density	ρ_{max} = 1.619 [g/cm^3]
Minimum Dry Density	ρ_{min} = 1.296 [g/cm^3]

Table 2 Similitude requirements

Item	Similitude (model/prototype)	Similitude Ratio
Length	L_m/L_p	$1/\lambda$
Mass Density	ρ_m/ρ_p	1
Time	t_m/t_p	$1/\lambda^{1/2}$
Acceleration	$(L_m/t_m^2)/(L_p/t_p^2)$	1
Strain	γ_m/γ_p	1
Flexural Rigidity	$(\rho_m L_m^5)/(\rho_p L_p^5)$	$1/\lambda^5$
Internal Friction Angle	ϕ_m/ϕ_p	1

The model piles were made to simulate the conditions in which the pile is fixed at its head to the structure and penetrates adequately into stiff non-liquefied layer overlain by a liquefiable layer. The model structure was idealized by a mass 270 kgf weight. The similitude requirments used are listed in Table 2.

Test Procedures

The model pile-structure system was constructed in the container before the placement of sand. Static horizontal loading tests and free vibration tests were made on the pile-structure system in the air and the water without soil layer.

The soil layer was then constructed by placing the sand under water. Static horizontal loading tests were again made on pile-structure system to determine the initial load deflection curve of the piles. The shear wave velocity and the cone penetration resistance of the model deposit were also measured.

Shaking table liquefaction tests were then conducted. In the test, soil density, pile diameter, pile rigidity, thickness of the liquefied layer and input motion were controlling variables. Table 3 summarizes the list of the tests. All the input motions were similar to Taft 1952 NS or El Centro 1940 NS. The time interval of these motions was scaled down to satisfy the similitude requirement. The two-layer model in the table was intended to study the effects of a non-liquefied layer overlying a liquefied layer on the pile-structure response. The thickness of the non-liquefied layer was 60 cm and the water table was set at 60 cm below the ground surface. For other tests, the water table was set at 20 cm below the ground surface.

In a typical sequence of shaking table liquefaction tests, maximum amplitudes of acceleration of input motions were applied in stages by starting with 25 gal and then increasing it by a factor of about 2, until the soil layer completely liquefied. During the test, pore pressures, accelerations, and displacements of soil-pile-structure system, and bending moment of pile were monitored and recorded.

Table 3 List of liquefaction tests conducted

Input Motion	Flexural Rigidity of Pile [kgf·cm^2]	Diameter of Pile [cm]	Natural Freq. of Pile-Str. [Hz]	Pile Identification	Number of Sand Layers
Taft	2.48×10^7	4.86	2.7	EI-pile	1
El Centro	2.48×10^7	4.86	2.7	EI-pile	1
Taft	6.44×10^7	5.00	4.2	2.5EI-pile	1
Taft	1.30×10^7	5.00	1.8	0.5EI-pile	1
Taft	2.53×10^7	7.62	2.6	1.5D-pile	1
Taft	2.48×10^7	4.86	2.7	EI-pile	2

Table 4(a) Physical and mechanical properties used in analysis for Test A with 0.5EI-pile

h [cm]	ρ [g/cm^3]	D_r [%]	V_s [m/s]	γ_r $\times10^{-4}$	R_{20}	h_{max} [%]	K_{ho} [kgf/cm^3]	P_r [g/cm^2]
30	1.90	50	70	0.80	0.135	30	4.6	36
30	1.90	50	80	1.15	0.135	30	6.5	71
30	1.90	50	90	1.55	0.135	30	7.5	94
30	1.95	50	95	1.95	0.135	30	8.7	126
30	1.95	50	100	2.35	0.135	30	9.7	157
30	1.95	50	105	2.75	0.135	30	10.7	189

Table 4(b) Physical and mechanical properties used in analysis for Test B with 2.5EI-pile

h [cm]	ρ [g/cm^3]	D_r [%]	V_s [m/s]	γ_r $\times10^{-4}$	R_{20}	h_{max} [%]	K_{ho} [kgf/cm^3]	P_r [g/cm^2]
30	1.90	50	75	0.90	0.135	30	5.2	39
30	1.90	50	80	1.10	0.135	30	7.3	78
30	1.90	50	90	1.55	0.135	30	9.0	116
30	1.95	50	95	1.95	0.135	30	10.3	155
30	1.95	50	100	2.35	0.135	30	11.6	194
30	1.95	50	105	2.75	0.135	30	12.7	233

Table 4(c) Physical and mechanical properties used in analysis for Test C with 2.5EI-pile

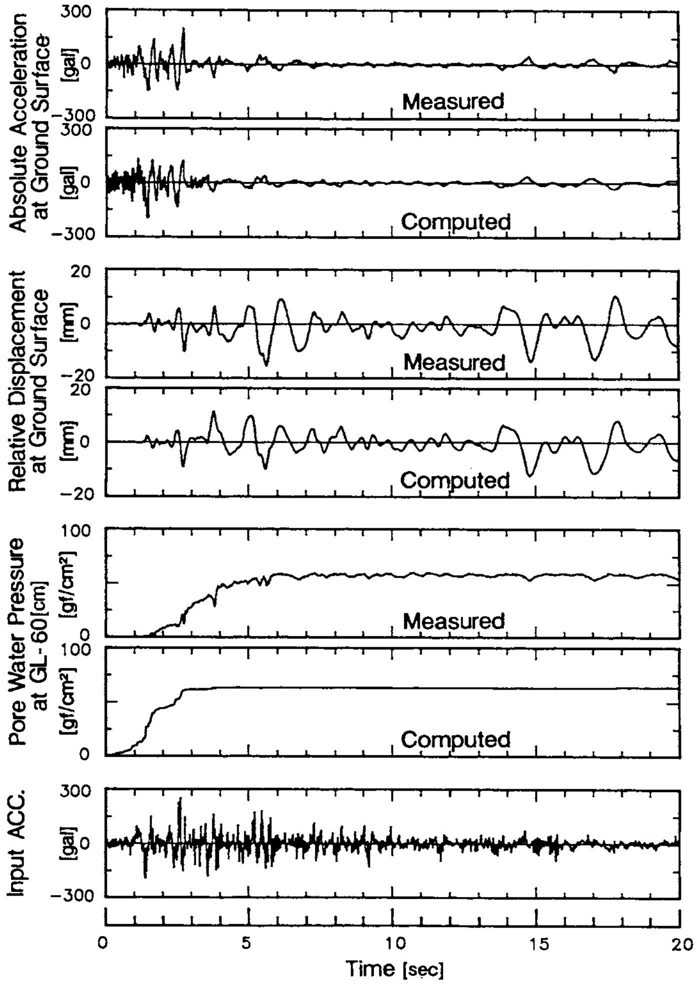
h [cm]	ρ [g/cm^3]	D_r [%]	V_s [m/s]	γ_r $\times10^{-4}$	R_{20}	h_{max} [%]	K_{ho} [kgf/cm^3]	P_r [g/cm^2]
30	1.95	50	60	0.90	0.135	30	5.2	39
30	1.95	50	65	1.10	0.135	30	7.3	78
30	1.95	50	70	1.40	0.135	30	9.0	116
30	1.95	50	75	1.70	0.135	30	10.3	155
30	1.95	50	80	2.00	0.135	30	11.6	194
30	1.95	50	85	2.30	0.135	30	12.7	233

COMPARISON OF MEASURED AND COMPUTED VALUES

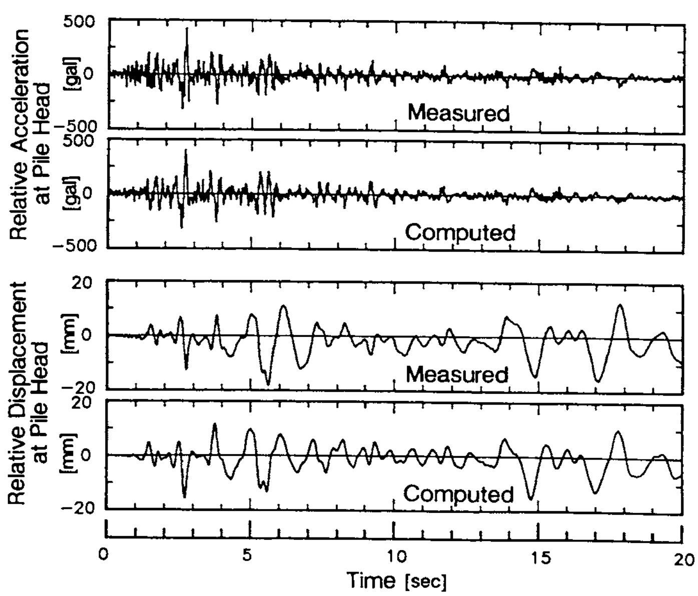
Time histories of measured accelerations, displacements, and pore water pressures are compared with those computed by the proposed analysis for three tests including two liquefied cases, Tests A and B, in Figs. 3 and 4 and one non-liquefied case, Test C, in Fig 5. Figs. 3(a) to 5(a) correspond to the data in or on the ground, while Figs. 3(b) to 5(b) to the data at the pile head. The piles in Test A have about 20% flexural rigidity of those in Tests B and C.

The physical and mechanical properties used for each analysis are summarized in Tables 4(a), 4(b), and 4(c) in which h = thickness of layer, ρ = mass density, D_r = relative density of soil, V_s = shear wave velocity. The values of γ_r and R_{20} were estimated from results of a dynamic triaxial test on the test sand. The values of K_{ho} and P_r are determined from the horizontal loading tests conducted before shaking table tests.

The following significant features can be pointed out from Figs. 3(a) and 4(a) concerning the time histories of the measured values of the ground. The liquefaction occurred after about 4 to 6 seconds from the start of shaking in both tests. As the pore pressures approach to the maximum values, the accelerations at the ground surface decrease considerably, whereas the displacements of the ground surface increase. The later is due to the degradation of the soil modulus with increasing effective stress and the former to the effect of the degradation of soil modulus on the ground response. The analytical results shown in the figures appear to simulate the above features well.

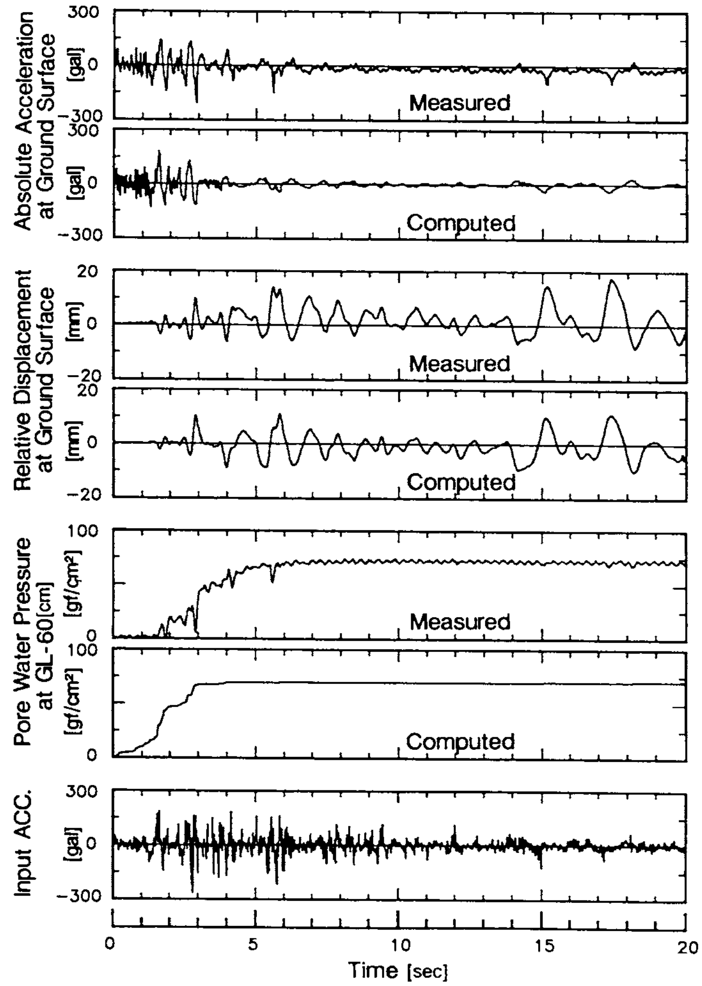


(a) Soil deposit

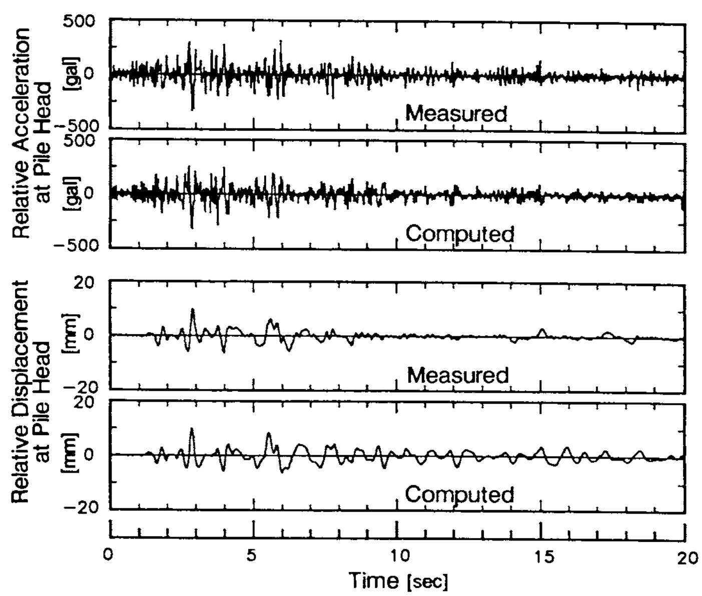


(b) Pile head

Fig. 3 Comparison of measured and computed time histories for Test A with 0.5EI-pile

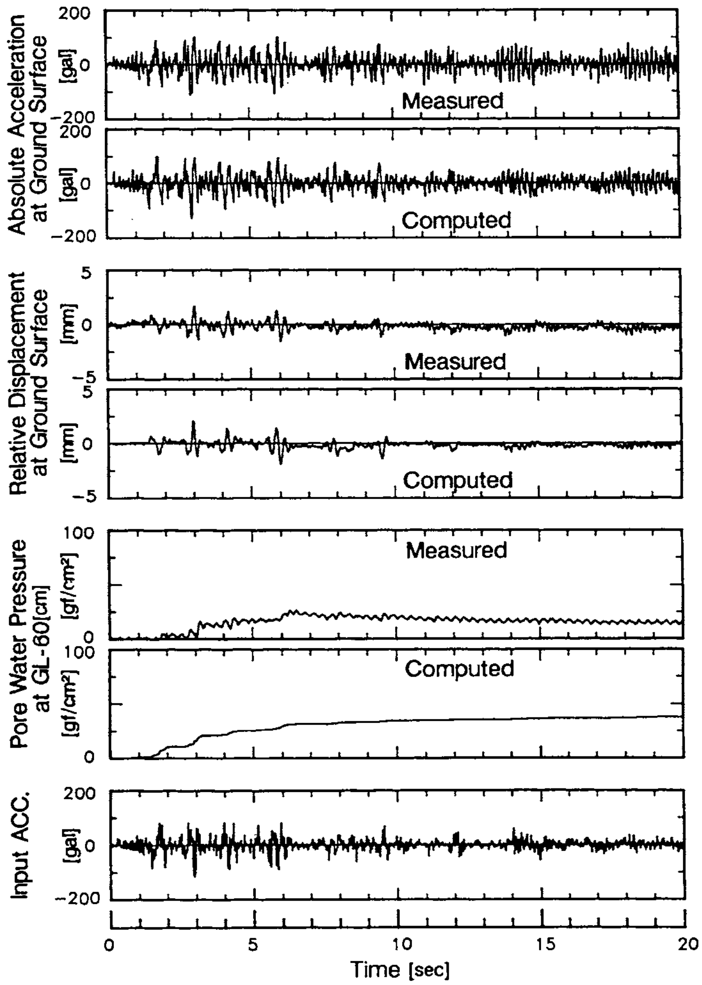


(a) Soil deposit

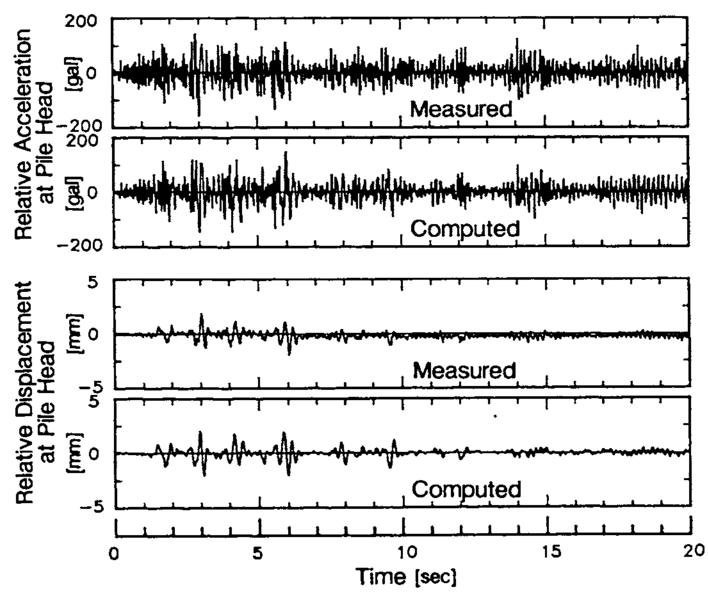


(b) Pile head

Fig. 4 Comparison of measured and computed time histories for Test B with 2.5EI-pile



(a) Soil deposit



(b) Pile head

Fig. 5 Comparison of measured and computed time histories for Test C with 2.5EI-pile

Concerning the measured pile response, the displacement of the pile with low flexural rigidity shown in Fig. 3(b) is very similar to and thus considered to be controlled by that of the ground surface throughout the test. In contrast, the displacement of the pile with high flexural rigidity shown in Fig. 4(b) is similar to the ground displacement only before liquefaction and becomes independent after liquefaction. The computed results appear to simulate the significant difference in pile response caused by the difference in pile rigidity.

The maximum excess pore pressure ratio induced in the test shown in Fig. 5 is 0.35. Since the reduction in soil modulus is relatively small compared with the liquefied case, the displacement of the soil is about one order of magnitude smaller than that for the liquefied cases. Under such a condition, the displacement of the pile is significantly affected by the ground displacement. These characteristics of the measured records are again simulated well by the proposed analysis.

The good agreements in the time histories of accelerations, displacements, and pore pressures both of the ground and at the pile head for the liquefied and non-liquefied cases indicate that the proposed method has a significant potential of evaluating the dynamic soil-pile-structure response during soil liquefaction.

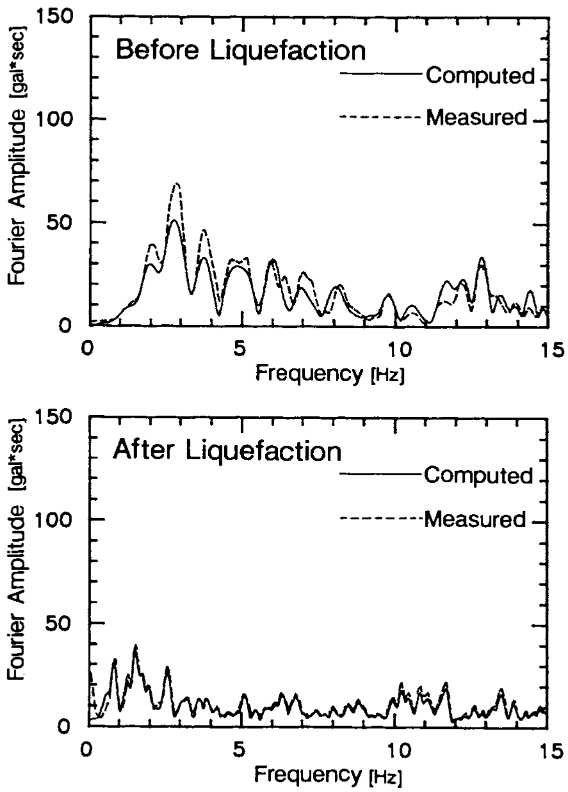
The above analyses of shaking table test results also indicate the following:

(1) Regardless of pile rigidity, the dynamic response of the pile-structure system before liquefaction is significantly influenced by the response of soil.

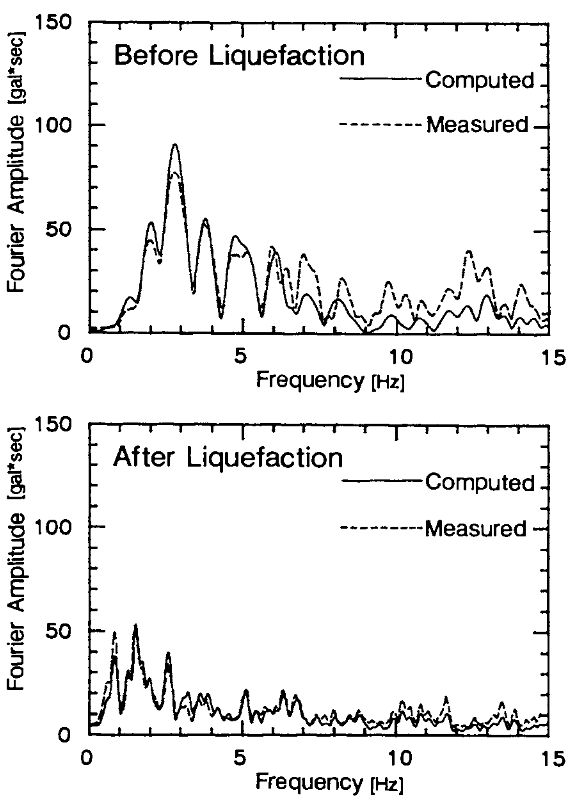
(2) Unless the pile is rigid enough, the dynamic response of the pile-structure system after liquefaction is also influenced by the response of soil.

Both the measured and computed time histories of acceleration of the ground surface and the pile head before and after liquefaction for Tests A and B are converted into the frequency domain, and their Fourier spectra are compared in Figs. 6 and 7. Figs. 6(a) and 7(a) correspond to the ground surface and Figs. 6(b) and 7(b) to the pile head. The computed spectra show good agreements with the measured ones irrespective of the degree of liquefaction and the rigidity of pile, indicating that the propose analysis can simulate dynamic response characteristics of the soil-pile-structure system in the frequency domain.

Fig. 8 shows the relationship between measured and computed maximum accelerations for all the tests conducted. Solid symbols correspond to liquefied cases, open symbols to non-liquefied cases. All the data fall on or near the line having a slope of 1:1, indicating that the computed and measured values are in good agreement with each other. This suggests that the proposed procedure is effective irrespective of such factors as the extent of soil liquefaction, the soil density, and the flexural rigidity of piles.

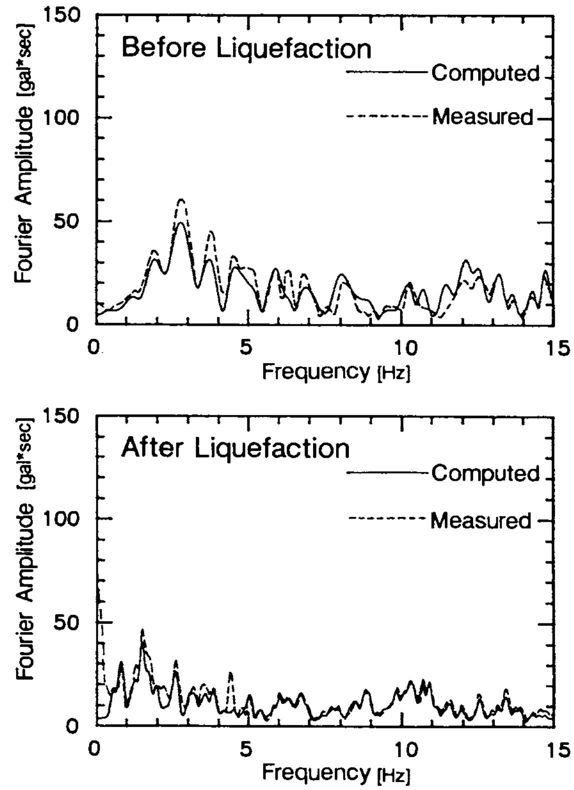


(a) Ground surface

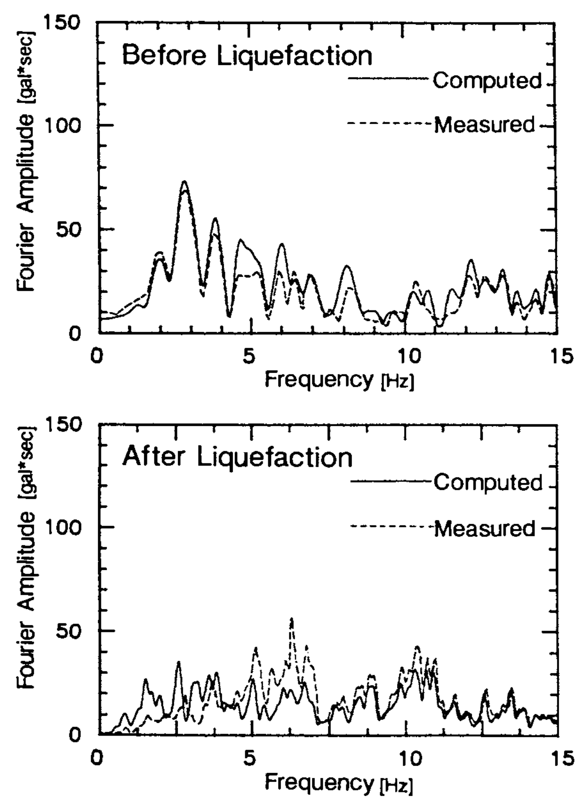


(b) Pile head

Fig. 6 Comparison of measured and computed Fourier spectra for Test A with 0.5EI-pile



(a) Ground surface



(b) Pile head

Fig. 7 Comparison of measured and computed Fourier spectra for Test B with 2.5EI-pile

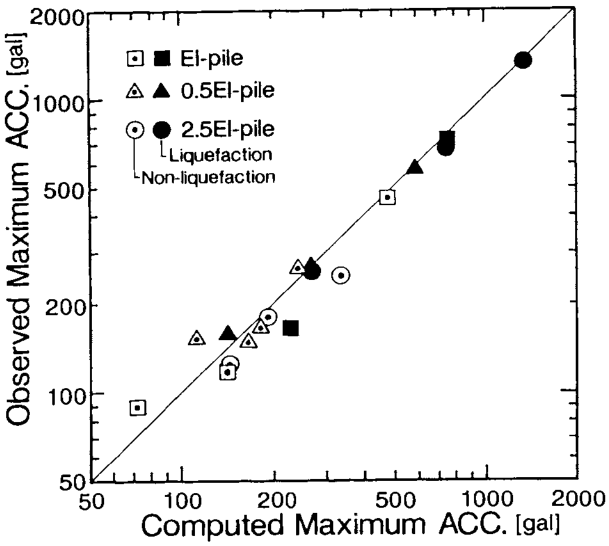


Fig. 8 Relationship between observed and calculated maximum acceleration for all tests conducted

CONCLUSION

The analytical method is presented for evaluating dynamic response of soil-pile-structure system during soil liquefaction. The method consists of the modified Penzien's model combined with the effective stress analysis for free-field soil response and the horizontal subgrade reaction model which connects free field response with pile-structure response.

To validate the effectiveness of the proposed method, large shaking table tests were conducted on model soil-pile-structure systems, and the results were compared with the values computed by the proposed analysis for liquefied and non-liquefied cases. The analytical results including the time histories of accelerations and displacements, and the response spectra of the ground surface and the pile head, were all in good accord with the observed values, showing that the proposed analysis is effective.

REFERENCE

Finn, W. L., Lee, K. W., and Martin, G. R. (1977) "Effective Stress Model for Liquefaction," Proc., Geotechnical Engineering Division, ASCE, Vol. 103, No. GT6, pp. 517-533.

Kagawa, K. and Kraft, L. M. (1980) "Lateral load-deflection relationships of piles subjected to dynamic loadings," Soils and Foundations, Vol. 20, No. 4, pp. 19-36.

Penzien, J., Scheffey, C. F., and Parmelee, R. A. (1964) "Seismic analysis of bridges on long piles," Proc., Engineering Mechanics Division, ASCE, Vol.90, No. EM3, pp. 223-254.

Seed, H. B., Martin, P. P., and Lysmer, J. (1976) "Pore-water pressure changes during soil liquefaction," Proc., Geotechnical Engineering Division, ASCE, Vol.102, No. GT4, pp. 323-346.

Shamoto, Y. and Shimizu, K. (1986) "The applicability of a one-dimensional liquefaction analysis with cyclic mobility to loose and dense sand layer, "Proceedings of The 7th Japan Earthquake Engineering Symposium, pp. 685-690, (in Japanese).