SEISMIC RESPONSE ANALYSIS OF PILE-SUPPORTED BUILDINGS CONSIDERING MATERIAL NONLINEARITY AND PILE-SOIL SEPARATION

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SUMMARY

The numerical method for earthquake response analyses of soil-pile-building is presented. The hysteresis model considering both the material nonlinearity and the geometrical nonlinearity, which means the gap at the soil-pile interface, is proposed. The soil-pile-building system is modeled as the spring-mass model. The pile group is condensed into a single pile. In order to evaluate the soil resistance when applying the subgrade reaction to the pile, soil springs are equipped around the pile. The initial stiffness of the soil spring is calculated by the three-dimensional thin layer formulation. The soil spring provided the proposed hysteresis model is laid on both sides of the pile. To represent the material nonlinearity of the soil springs, the hyperbolic curve, Masing's rule and the Modified Ramberg-Osgood Model are used. The hyeteresis model also can consider the adhesive force between the pile and the surrounding soil properly.

The simulation analysis of the forced vibration test of a small foundation on the ground supported by a single pile is performed for the verification of the proposed method. By indicating that the results of the simulation analysis show good consistent with those of the forced vibration test, it is demonstrated that the proposed method is valuable for the nonlinear earthquake response analysis considering both the material nonlinearity and the geometrical nonlinearity by using the spring-mass model.

Furthermore, a numerical nonlinear earthquake response analysis of a pile-supported building by use of the proposed method is carried out to investigate the effects of the gap between the pile and the surrounding soil on the response of the pile and the building.

INTRODUCTION

Just five years have passed since the Hyogoken-Nanbu earthquake occurred on January 17, 1995, in Japan. It is well known that the earthquake caused extremely strong ground motions. So many buildings around Kobe area suffered serious damage and collapse from the earthquake. Among them, there were some pile-supported buildings whose piles were heavily damaged because of the soil nonlinearity due to the strong ground motions. Since then, some nonlinear earthquake response analyses of pile-supported buildings have been carried out to clarify the cause of damages of the piles by using the finite element model (FEM), the spring-mass model, and so on, in Japan.

The spring-mass model is very practical in terms of both the availability and the computation efforts in comparison with the FEM. Especially, the material nonlineartiy and the geometrical nonlinearity, which means the gap effect at the soil-pile interface, can be modeled with a simple procedure. Some analysis methods have

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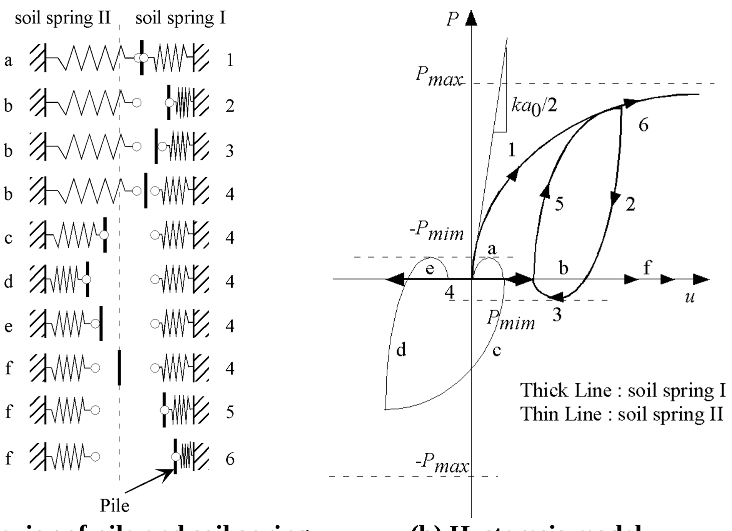
been proposed [i.e. Naggar et al. 1996, Miyamoto et al. 1998]. We have also already proposed the numerical method for a soil-pile-building system considering the material nonlinearity by use of the spring-mass model [Mori et al. 1998].

In this paper, our improved numerical method by using the new hysteresis model is presented. The new hysteresis model is able to consider both the material nonlinearity of the soil and the geometrical nonlinearity simultaneously. The adhesive force between the pile and the surrounding soil is also taken into account in the hysteresis model. In order to verify the accuracy of the proposed method, the results of the numerical analysis are compared with those of the forced vibration test of the small foundation on the ground supported by a single pile [Imamura, et al. 1996]. Furthermore, for an application of the proposed method, a nonlinear earthquake response analysis of a pile-supported building is carried out to investigate the effects of the gap at the soil-pile interface on the responses of the pile and the building.

MODELING OF A SOIL-PILE-BUILDING SYSTEM CONSIDERING MATERIAL AND GEOMETRICAL NONLINEARITY

We describe briefly the numerical method for the earthquake response analysis of the soil-pile-building system considering both the material nonlinearity and the geometrical nonlinearity. The soil-pile-building system is modeled as the spring-mass model, which is known as the so-called beam-on-Winkler foundation. The pile group is condensed into a single pile. The weight, the area and the moment of inertia area of the pile are summed up those of a number of piles. Lateral springs *ka*, shear springs *kb*, and lateral dashpots *Ca* (refer to Figure 4) are equipped around the pile to evaluate the soil resistance when applying the subgrade reaction to the pile. For the adequate estimation of the pile group effect, the initial stiffness values of ka_0, kb_0 and the initial damping coefficient of Ca_0 are calculated by the three-dimensional thin layer formulation. To represent the material nonlinearity of the soil springs, the hyperbolic curve and Masing's rule are used for the lateral springs and the Modified Ramberg-Osgood Model is chosen for the shear springs.

A schematic view of the proposed hysteresis model considering both the material nonlinearity and the geometrical nonlinearity is depicted in Figure 1. The figure 1(a) shows the behavior of the pile and the lateral spring. When the proposed hysteresis model is applied to the lateral springs, the lateral springs are laid on both sides of the pile to consider the gap effect at the soil-pile interface properly. The right lateral spring is called the soil spring I and the left one is called the soil spring II in this paper. Figure 1(b) shows the proposed hysteresis model. The horizontal axis indicates the displacement *u* of the pile. The vertical axis represents the subgrade reaction *P* of each soil spring. The sum of the subgrade reactions induced from a pair



(a) Behavior of pile and soil spring **(b) Hysteresis model**

Figure 1: Proposed hysteresis model considering material nonlinearity and geometrical nonlinearity.

of the lateral springs is applied to the pile. Thick lines describe the behavior of the soil spring I, and thin lines illustrate that of the soil spring II. P_{max} and P_{min}, which are parameters defining the hysteresis characteristics, are supposed to be the ultimate subgrade reaction and the adhesive force of the soil, respectively. The initial stiffness of each spring is $ka_0/2$. The earth pressure at rest is not considered. The notation of the figures and alphabets in Figures 1(a) and (b) represents the hysteresis of the behavior of the soil spring I and the soil spring II. During the subgrade reaction is the compression, the soil spring is supposed to move in accordance with the hyperbolic curve and Masing's rule. When the subgrade reaction turns out to the tension, after reaching to P_{min}, the subgrade reaction decreases as the pile departs from the soil. When the subgrade reaction becomes zero, the pile and the soil separate. After that, the subgrade reaction turns out to the compression when the pile and the soil contact again.

VERIFICATION OF THE PROPOSED MODEL

Outline of Forced Vibration Test of a Single Pile

A full scale forced vibration test of a small foundation supported by a single pile was carried out by Imamura et al. [1996] to investigate the nonlinear behavior of the pile foundation. Figure 2 illustrates the outline of the forced vibration test. The test pile was made of precast centrifugally compacted concrete with 15m length and 450mm diameter. The bottom of the pile was located at the depth of 14.3m. The reinforced concrete block at the pile head was constructed. The 0.45m length from the pile head was inserted into the block to make the rigid connection. A gap of 0.25m between the bottom surface of the block and the ground surface was provided to prevent the friction. The mass of the block was about 10ton. A rotating mass type shaker was placed on the block. Figure2 also shows the layout of the instruments: the accelerometer at the pile head, the velocity seismograph at the top of the block and the earth pressure cells at the ground surface designated as Ap1 and Ap2. The records of those instruments are used later for the verification of the proposed model. The soil profile in the test yard is also shown in Figure 2. The test pile was excited in the horizontal direction with a harmonic wave. Three different levels of the excitation force: a low level excitation, a middle level excitation and a high level excitation, were considered to examine the nonlinear behavior of the pile. The frequency range of each excitation level was from 1.0Hz to 20Hz.

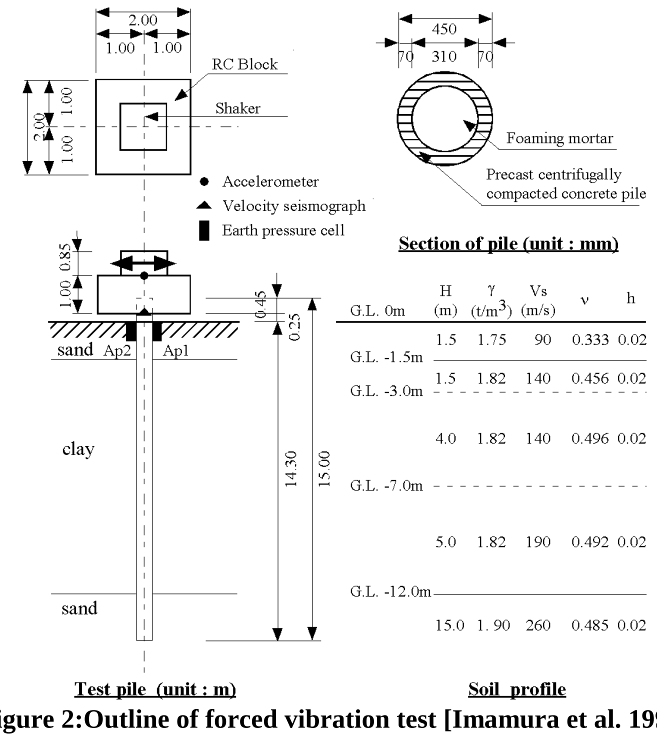
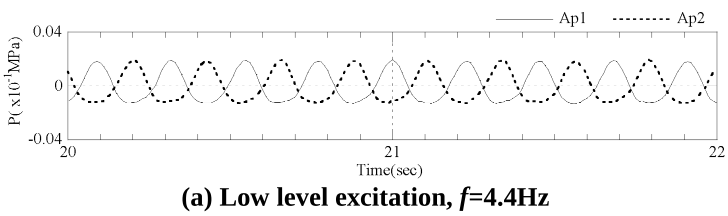
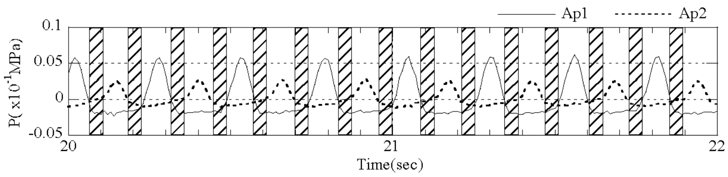


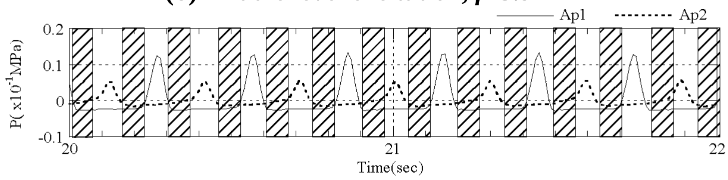
Figure 2:Outline of forced vibration test [Imamura et al. 1996].



(a) Low level excitation, *f*=4.4Hz



(b) Middle level excitation, *f*=3.9Hz



(c) High level excitation, *f*=3.4Hz

Figure 3: Soil pressure waves for each excitation level.

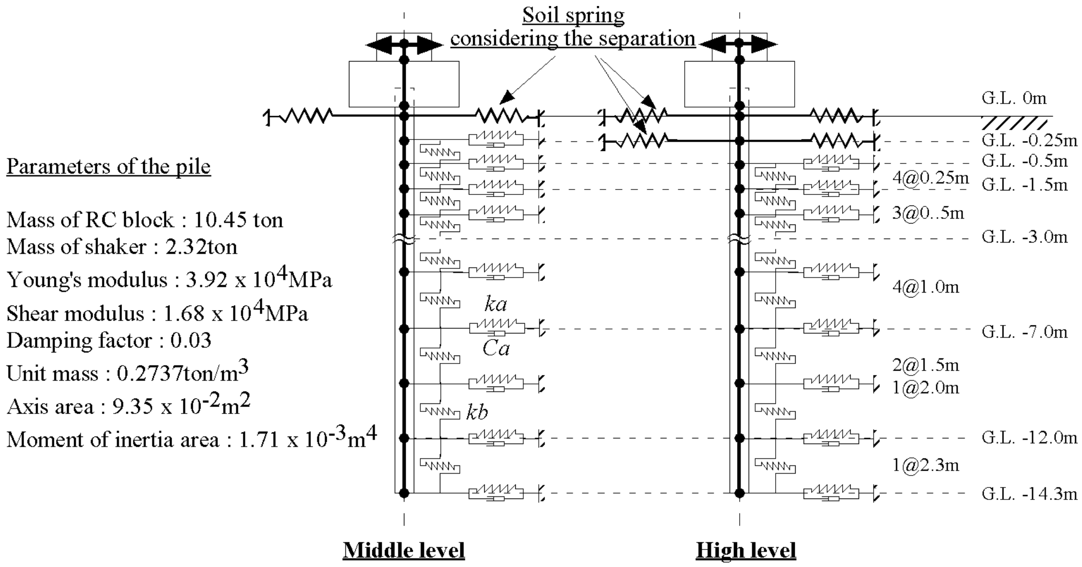


Figure 4: Analysis model for verification.

Figure 3 shows the soil pressure waves at Ap1 and Ap2 (shown in Figure 2) for each excitation level. The duration time of these presented records is 2 seconds during the stationary state at the resonance frequencies of the system, which are *f*=4.4Hz, *f*=3.9Hz and *f*=3.4Hz for the low, the middle and the high level excitation, respectively. In all records, the negative pressures exist, since the soil pressures were calibrated under the earth pressure at rest. In Figure 3(a) of the low level excitation, it is supposed that the gap at the soil-pile interface dose not occur, because the amplitude of the negative pressure is almost equal to that of the positive soil pressure. On the other hand, in Figures 3(b) and 3(c) of the middle level excitation and the high level excitation, one can see that the amplitude of the negative pressure is much smaller that that of the positive pressure and that the waveforms are nearly flat during the negative pressure is producing. These phenomena indicate that the pile and the surrounding soil separate. Furthermore, as being indicated by shading in Figures 3(b) and 3(c), the waveforms at Ap1 and Ap2 are flat simultaneously at some time zones. It can be estimated that the boundary at the soil and the pile was completely separated. It is also reported that the gap was clearly observed after the forced vibration test. The linear elastic simulation analysis in the frequency domain has already been performed and then it has been estimated that the depth of the gap is 0.25m and 0.4m for the middle level excitation and the high level excitation, respectively.

Analysis Model for Verification

The simulation analysis for the forced vibration test is performed in order to verify the accuracy of the presented method. An analysis model for each excitation level and employed parameters are shown in Figure 4. The pile is modeled as the beam element and is divided as shown in this figure so as the depth of the gap to correspond to that of the vibration test for each excitation level suitably. According to the results of the linear elastic simulation analysis, the proposed hysteresis model is applied to the lateral spring equipped at G.L.0.0m, G.L.-0.25m for the middle level excitation and G.L.0.0m, G.L.-0.25m and G.L.-0.5m for the high level excitation. The shear springs are not provided in those regions. The adhesive force is not considered because the soil at the ground surface is sand and the depth of the gap is not so thick. The ultimate subgrade reaction, which is defined as P_{max}, is evaluated by referring Broms [i.e. Broms 1964a]. The excitation frequencies are assumed to be the resonance frequencies of the system, which are 3.9Hz for the medium excitation and 3.4Hz for the large excitation. The duration time is taken enough for the response of the system to become the steady state.

Comparison of the numerical analysis and the forced vibration test

Figures 5 and 6 show comparison of the numerical analysis and the forced vibration test for the middle level excitation and the high level excitation. The acceleration waves at the pile head, the velocity waves at the top of the block and the soil pressure waves at Ap1 are illustrated in these figures. Thick lines indicate results of the numerical analysis and thin lines show those of the forced vibration test. The earth pressure at rest, which is not included in the proposed hysteresis model, is added to the soil pressure wave at Ap1. In order to correspond the dimension of the numerical analysis with that of the experiment, the subgarde reaction of the lateral spring is divided by the area, which is calculated by the multiplication of the diameter of the pile and the dominant depth of the lateral spring. All results are fairly good coincidence. It is found from these results that the proposed method is valuable for the nonlinear response analysis considering the material nonlinearity and the geometrical nonlinearity.

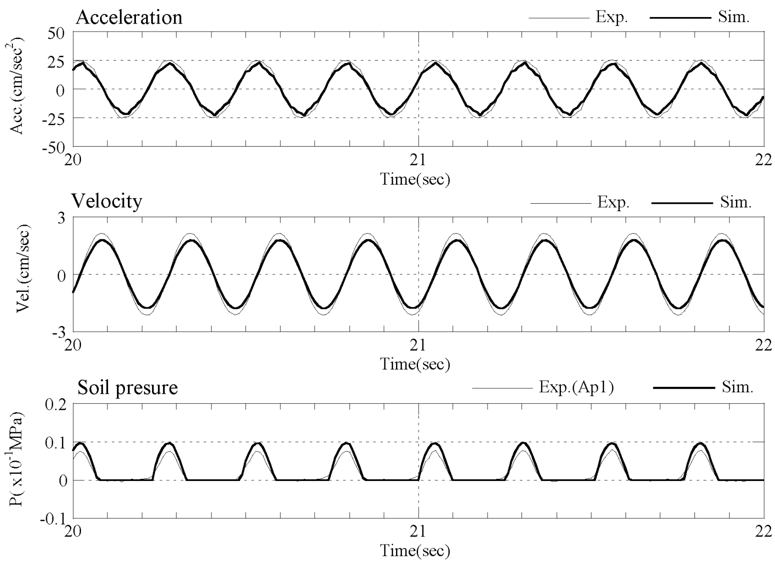


Figure 5: Comparison of forced vibration test and numerical analysis (Middle level excitation, *f*=3.9Hz).

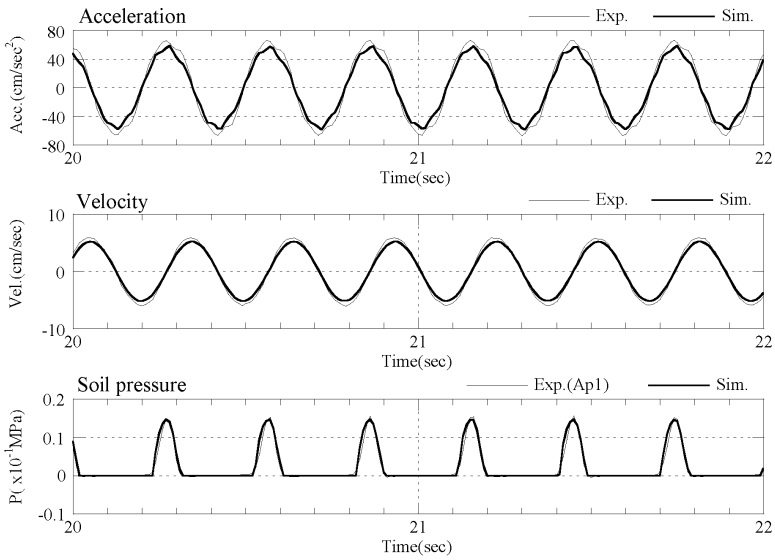


Figure 6: Comparison of forced vibration test and numerical analysis (High level excitation, *f*=3.4Hz).

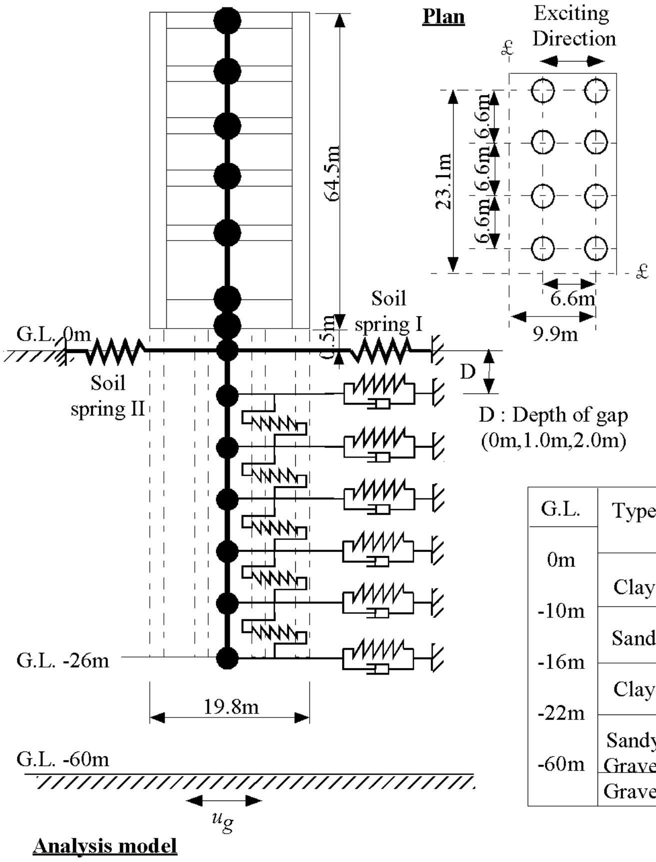
A NONLINEAR EARTHQUAKE RESPONSE ANALYSIS OF A PILE- SUPPORTED BUILDING BY THE PROPOSED METHOD

Analysis Model of a Pile-Supported Building

We present a nonlinear earthquake response analysis of a pile-supported building considering the material and geometrical nonlinearity by using the proposed method to investigate the effect of the gap at the soil-pile interface on responses of the pile and the building. The analysis model and the used parameters are shown in Figure 7. The building is the reinforced concrete building and 64.5m height. The material nonlinearity is included in the relationship between the shear force and the relative story displacement of the building. The gap between the building and the ground surface is assumed to be 0.5m to neglect the soil-building interaction at the ground surface. The number of piles is 32 (4 x 8). The pile is steel and supposed to be elastic. The bottom of the pile is located at G.L.-26m. The rotations of the pile head are set to be fixed. The depth of the gap D is assumed to be 0.0m and 1.0m and 2.0m. The soil springs provided with the proposed hysteresis model are equipped at intervals of 1.0m in accordance with the assumed D. The adhesive force of the soil is not considered. The soil condition is also shown in this figure. As the input motion, we use the artificial wave as shown in Figure 8. The maximum acceleration of the input motion is set to be 400cm/sec^2 at the level of G.L.-60m.

Results and Discussions

Figure 9 shows the response waves of the subgrade reaction of the soil springs provided with the proposed hysteresis model in the case of D=2.0m. Thick line presents the response of the soil spring I and thin line shows the response of the soil spring II (shown in Figure 7). In Figure 9(a), it is found that the subgrade reaction of both soil springs are around zero simultaneously at the narrow ranges of duration time near about 30sec., 40sec., 50sec. and 55sec. This result indicates the pile and the surrounding soil separate completely at G.L.0.0m. On the other hand, the gap is not found explicitly at G.L.-1.0m from Figure 9(b). Figure 10 shows the comparison of the subgrade reaction of the soil springs at G.L.0.0m for D =1.0m (solid lines) and D=2.0m (broken lines). There is less difference between them. The reason is assumed that the gap does not occur at G.L.-1.0m as shown in Figure 10(b). Figure 11 shows the distribution of the maximum acceleration and the maximum shear force of the building and that of the maximum bending moment of the pile for each D. The response values of the building (the acceleration and the shear force) are not affected by the depth of D. The bending moment at the pile head becomes slightly larger as D is deeper. These facts suggest that the gap does not have a great influence on the response of the pile-supported building.



Building (Reinforced Concrete)
Total mass : 26,200 ton
Area : 22m x 50m
Natural period : 1.3sec
Damping factor : 0.03

Pile (Steel)
Number of piles : 32
Axis area : 2.26m^2
Moment of inertia area : 0.39m^4
Young's modulus : 2.1x10^{5}MPa
Unit mass : 7.845ton/m^3
Strength of steel : 3.3x10^{2}MPa
Damping factor : 0.01

G.L.	Type	Thick-ness	γ (t/m^3)	Vs (m/sec)	ν	h (%)
0m ~ -10m	Clay	10m	1.5	150	0.48	0.02
-10m ~ -16m	Sand	6m	1.8	100	0.45	0.02
-16m ~ -22m	Clay	6m	1.5	220	0.48	0.02
-22m ~ -60m	Sandy Gravel	6m	2.0	300	0.45	0.02
	Gravel	38m	2.0	500	0.40	0.02

Soil condition

Figure 7: Analysis model and employed parameters.

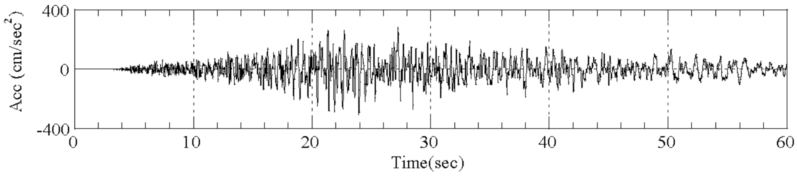
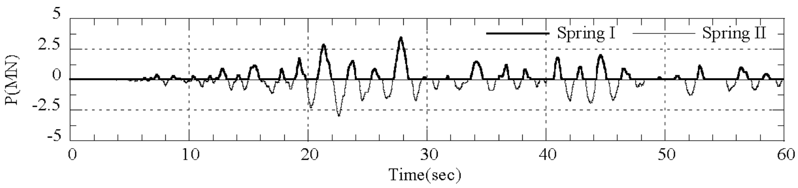
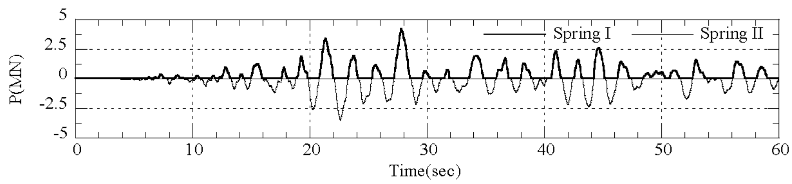


Figure 8: Time history of artificial wave for input motion.



(a) G.L.0.0m



(b) G.L.-1.0m

Figure 9: Subgrade reaction at each soil spring provided with proposed hysteresis model (*D*=2.0m).

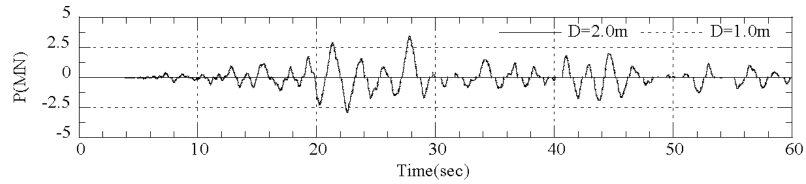
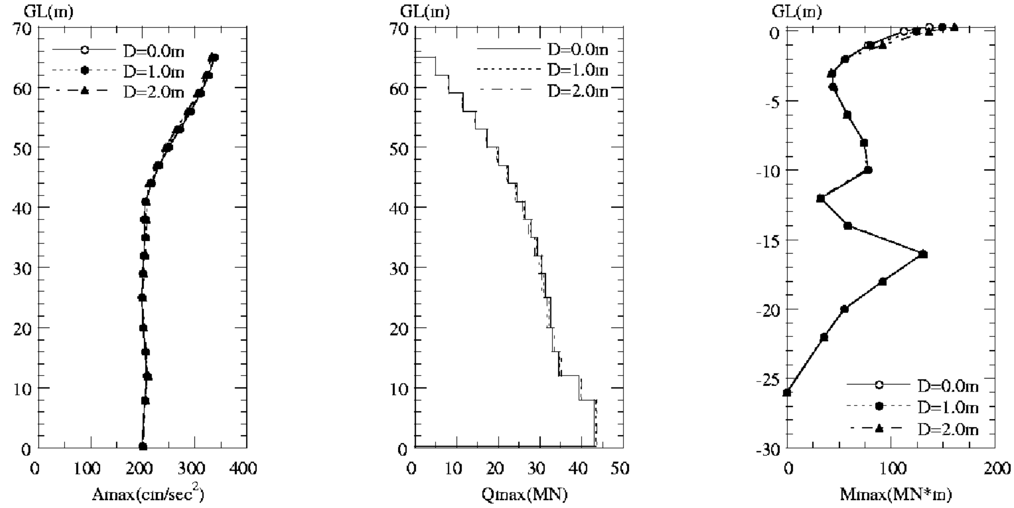


Figure 10: Comparison of subgrade reaction at G.L.0.0m for each *D*.



(a) Acceleration of building **(b) Shear force of building** **(c) Bending moment of pile**

Figure 11: Results of numerical analyses.

CONCLUSIONS

Conclusions are as follows:

1. The numerical method for the earthquake response analysis of the soil-pile-building system was presented, in which the hysteresis model of soil springs considering the material and the geometrical nonlinearity simultaneously was proposed.
2. The accuracy of this proposed method was verified through the simulation analysis of the forced vibration test of a small foundation supported by a single pile.
3. Through the nonlinear earthquake response analysis by using the proposed method, it is concluded that the gap at the soil-pile interface does not have a great influence on the response of the pile-supported building.

REFERENCES

A. Imamura, K. Hijikata, Y. Tomii, S. Nakai and M. Hasegawa (1996), "An Experimental Study on Nonlinear Pile-Soil Interaction Based on Forced Vibration Tests of A Single Pile and A Pile Group ", *Proceedings of the eleventh World Conference on Earthquake Engineering*, Paper No.563.

Broms, B.B. (1964a), "Lateral Resistance of Piles in CohesiveSoils", *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol.90, No.SM3, pp27-63.

Masafumi Mori and Masayuki Hasegawa (1998), "Nonlinear Analysis Method for Earthquake Response of Pile-supported Buildings ", *The 10th Japan Earthquake Engineering Symposium*, Vol.2, pp1711-1716 (in Japanese).

M. H. El Naggar and M. Novak (1996), "Nonlinear analysis for dynamic lateral pile response", *Soil Dynamics and Earthquake Engineering*, Vol.15, pp233-244.

Yuji Miyamoto and Yuji Sako, et al (1995), "Earthquake Response of Pile Foundation in nonlinear Liquefiable Soil Deposit", *J. Struct. Constr. Eng.*, No.471, pp.41-50 (in Japanese) .