A Simplified Method for Dynamic Response Analysis of Soil-Pile-Building Interaction System in Large Strain Levels of Soils - Analysis for Building with Embedment and Pile

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We propose a simplified model to simulate the dynamic response of soil-pile-building interaction with embedment at large soil strain. The model consists of the Sway-Rocking model and an equivalent linearization method. Results of scalded shaking table tests were used to evaluate the accuracy of the proposed method. A part of the model ground was made of Plasticine and oil, whose stiffness and damping dependency on strain are similar to those of clayey soils. The input motion used are 1968 Hachinohe EW and 1940 El Centro NS and the peak accelerations were set to be 100, 300 600 cm/s^2 at the shaking table. Test results show the average maximum soil strain was 0.001 to 0.013 and the natural frequency and the amplification factor decreased by 58% and 41%, respectively. The transfer functions between the ground surface and the building obtained by the analysis and those obtained from the tests were very close. The difference in peak acceleration at the building obtained from the test and analyses were within 20%.

INTRODUCTION

When designing a building, it is important to evaluate its earthquake performance taking into account non-linear soil-building interaction effects. FEM models or Penzien's models are efficient to do dynamic response analyses for a soil-pile-building interaction including non-linear soil effects. However, using FEM models are quite a time consuming process due to model preparation and calculation. In Penzien's models, there are uncertainties in

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evaluating the virtual mass around the piles or ground springs. Therefore, the methods are not common in practical design processes. In the practical design of a building, analytical methods should be simple, so, for example, an equivalent linearization method like SHAKE have been used generally to calculate ground responses. However, in the case of non-linear soil-building systems, the accuracy of the method had not been tested enough. The thin layered element method was applied to analyses to evaluate the impedance of a foundation (Miura et. Al. 1995) or the response of piles by vibration tests (Kusakabe et. Al. 1994), in which the soil rigidity was reduced or there was gap between pile and ground, but the method was not applied to earthquake response analyses.

We proposed a simplified method to calculate the dynamic response of a soil-pile-building system without embedment (Tamori et. al. 2001). The method employed the Sway-Rocking model and an equivalent linearization method and simulation analyses for shaking table tests were done. From the analyses, we found the larger the peak acceleration of the input motion, the lager the phase lag between the soil strain and pile deformation, so we had to make the ratio of equivalent uniform strain divided to the maximum strain in equivalent procedure smaller when the peak acceleration was $600cm/s^2$.

In this study, we make similar analyses for shaking table tests. The difference is that this time the building has embedment and piles.

PLASTIC MATERIAL FOR THE GROUND MODEL

Plastic material for the artificial ground model used in this study was made of Plasticine and oil. Plasticine, being a mixture of calcium-carbonate and oil, has been used as a model material for plastic deformation processing of steel, since it has restoring force curves similar to high-temperature steel. Fig. 1 shows the results of tri-axial compression tests for actual clayey soils and Plasticine. The initial shear modulus, Gt (strain being 1.0×10^{-5}), shear modulus at large strain levels, Gs, and damping factors, hg, were obtained by tri-axial compression tests in which the ambient stress was kept at $1.0\ kg/cm^2$ and the excitation frequency was 1.0Hz. The shear modulus and damping factor of the plastic soil material, Plasticine, has a strain dependency similar to those of actual clayey soils.

OUTLINE OF SHAKING TABLE TESTS

The similarity proposed by Buckingham was used in modeling the building and the ground. The scale factors calculated from this formula are summarized in Table 1. This similarity is applicable to non-linear soil dynamics when the soil model material has a shear modulus-strain and a damping factor-strain relation similar to those of the prototype (Kagawa 1987). Under these conditions the ratio of shear forces in the model and the prototype were kept approximately equal to that of the damping forces for a wide soil strain range. Fig. 2 shows an outline of the building and the ground model together with the location of the measurement apparatus. Two dwelling units consists of 11-story buildings were modeled in the transverse direction. Table 2 shows the natural frequency and damping factor of the building model. The building model was made of steel weights and its columns were made of steel plates. The building foundation was made of aluminum and acryl plates. Four pillar-shaped (diameter is 38mm, length is 487mm) pile models, made of steel plates and rubber, were set at the corners of the foundation. The ground model has a block shape with a size of 2x1.46x0.6m. Stainless plates were set at both side ends in a transverse direction to the ground to prevent vertical motion of the ground.

The central part (diameter is 800mm, depth is 450mm) of the ground model was made from Plasticine and oil. The remaining portions of the model were composed of polyacrylamid and bentnite, and remained elastic throughout the tests. Table 3 shows the characteristics of the ground. Damping factors were obtained by a free torsional vibration test and shear wave velocities were obtained by P-S wave propagation tests. Earthquake records, 1968 Hachinohe EW and 1940 El Centro NS, in which the time length was corrected according to the similarity, were used for the input ground motion. Their peak accelerations were set to be 100, 300 and 600 cm/s^2 at the shaking table.

TESTS RESULTS

Fig. 3 shows the first natural frequencies derived using the transfer function (BH6/SH5), and the average maximum soil shear strains. The average maximum shear strains shown in Fig. 3 and 4 were calculated as follows:

(1) Acceleration time history at SH5, SH4, SH3 and SH2 were integrated twice to obtain the displacement time histories. (2) Maximum relative displacements between those four points, SH5, SH4, SH3 and SH2, were divided by the distances between them to obtain the

maximum soil strains. (3) The average maximum soil shear strains are average of the maximum shear soil strains. Fig. 3 shows the average maximum soil shear strains were from about 0.001 to 0.013, and the first natural frequencies were from 10.5Hz to 6.1Hz. Fig. 4 shows the amplification factors, i.e., the amplitude of the transfer function (BH6/SH5) at the natural frequencies, and the average maximum soil shear strains. The amplification factors are from about 10.6 to 4.4, so, when the peak acceleration of input motions was 600 cm/s^2, the natural frequency and the amplification factors, become 58% and 42%, respectively, compared to the values obtained when the peak acceleration was 100cm/s^2.

THEORETICAL MODEL

OUTLINE

The theoretical model employed in this study was the Sway-Rocking (SR) model, and an equivalent linearization method was used for non-linear analyses. Dynamic stiffness of the pile foundation was calculated as follows:

(a) The dynamic stiffness of piles for horizontal and rocking motion proposed by Novak and Nogami (Novak and Nogami 1977) was employed. Group effects factor, which was 0.72 in this case, were derived by Iiba's method (Inoue et. al. 1988).

(b) The dynamic stiffness of piles for vertical motion was calculated by Novak's method (Novak 1977).

(c) The dynamic stiffness of the bottom of the embedment was calculated by D.G.C. (Kobori et. al. 1970) and that of the side of embedment was calculated using Yoshida's method (Yoshida 1998)

(d) The total stiffness of the basement is the sum of those of the piles and the embedment.

SOIL MODEL AND ESTIMATION OF SOIL STRAIN

Shear modulus, *Gs*, and damping factor, h_g, of the soil were modeled by the tri-axial compression tests:

$$\frac{G_s}{G_t} = \frac{1.01}{1 + 0.96(\gamma_s / 0.002072)^{1.258}} \tag{1}$$

$$h_g = 0.035 + 0.145(1 - G_s / G_t) \qquad (2)$$

where G_t is the initial shear modulus and γ_s is the shear strain of the soil.

Effects of soil non-linearity around the base were evaluated by a static FEM analysis. Fig.5 shows a FEM model, which was a 1/4 symmetry model, for the static analysis, and its size was 1.5x1.0m, and depth was 60cm. The lower layer of the ground was an elastic media and the upper layer was an elasto-plastic media, which follows the Drucker-Prager's yield function. Its cohesion was $1.96\text{x}10^3\text{N/mm}^2$ and angle for internal friction was 0 degree. Boundary conditions for the analyses are as follows: (a)The bottom and the side of the ground were fixed. (b)The base had a degree of freedom in only the horizontal direction, so its rotation was fixed. (c) Horizontal force was applied at the top of the base.

Fig. 6 shows the horizontal stiffness obtained by static FEM analysis. k_{f0} is the initial horizontal stiffness and k_{fs} are those when the horizontal displacement of the base and the ground surface is δ_f. k_{fs} was modeled as

$$\frac{k_{fs}}{k_{f0}} = \frac{1 - 0.04}{1 + (\delta_f / 0.18259)^{1.577}} + 0.04 \qquad (3)$$

Fig.7 shows the results of the same static FEM analyses for a full scale model compared to results from design equation, where $k_{fs}= k_{f0}*\delta_f^{-0.5}$. It is well-known relation that the horizontal stiffness of piles is inversely related to the square root of piles displacement. In this case, the horizontal stiffness of the base is inversely related to the 1/4 power of the base displacement.

ESTIMATION OF SOIL STRAIN

We have to consider two kinds of soil strain when we want to find the response of a soil-building system to an earthquake. The first is strain due to free field motion of the ground, γ_{wave}, and the second is strain due to the relative displacement between the base and the ground, γ_f. The total strain, γ_{eq} is evaluated by

$$\gamma_{eq} = \alpha\sqrt{\gamma_{wave}^2 + \gamma_f^2} \qquad (4)$$

where α is the ratio of the equivalent uniform strain divided by the maximum strain in the equivalent linear procedure. In the analyses, γ_{wave} is a measured value, which is the average of the maximum soil shear strain shown in figures 3 and 4. γ_f was evaluated as follows: (a) A response analysis were done using the SR model with an initial stiffness obtained from a shear wave velocity and the damping factor, which corresponded to soil strain γ_{wave}. From this the maximum relative displacement, δ_f, is obtained. (b) The horizontal stiffness ratio, k_{fs}/k_{s0} is calculated by equation (3) and δ_f. (c) k_{fs}/k_{f0} is the same as the soil shear stiffness ratio, G_s/G_t, so we use k_{fs}/k_{f0} as the left term of equation (1) and we obtain γ_s. This strain is the strain due to the relative displacement between the base and the ground. (d) The equivalent soil strain, γ_{eq} , is calculated using equation (4). The reason why γ_{eq} is the root of the sum of squares of γ_{wave} and γ_f is that their maximum value doesn't occur at the same time. (e) The stiffness and damping factor of the soil were calculated using equation (1), (2) and γ_{eq}, and the response analysis with the SR model was done again. The proposed method employed the equivalent linearization method, so the processes from (b) to (e) were iterated until the response coverage.

We found in our previous study that the larger the peak acceleration of the input motion, the smaller the α in equation (4), that because the larger the peak acceleration of input motion, the larger the phase lag between γ_{wave} and γ_f. Figure 8 shows test values of γ_{wave} and γ_f obtained form the tests. The test values of γ_f were obtained from the relative displacement between the ground surface and the base using the same process to evaluate γ_f described above. The phase lag is not depends on the amplitude of the input motion, so, in this study, we use $\alpha = 0.7$ in equation (5) throughout the analyses.

ANALYSIS RESULTS

Figures 9 to 14 show transfer functions derived from the tests and the analyses. BH6, BH1 and SH5 are the horizontal motion at the top of the building, the base and the ground surface, respectively. UR is rocking motion at the top of the building in these figures. The tests and analytical values are generally very close. In the case of BH6/SH5, the transfer function between ground surface and the top of the building, when the peak acceleration of the input motion is about 100cm/s^2, the tests showed a steep single peak, and the peak of the transfer function by the analyses was a bit lower than the test value. The natural frequencies by the analyses were close to the test ones. When the peak acceleration of the input motion was about 600 cm/s^2, the transfer function, BH6/SH5, from the tests had several peaks and the transfer function obtained by the analyses was close to one of them.

Figures 15 to 17 show the time history of the acceleration at the top of the building. The test and analytical values are generally close. Figures 18 and 19 show the peak acceleration of the building obtained by the tests and the analyses. The test and analytical values are very close except the case of Hachinohe 600cm/s^2. Except Hachinohe 600cm/s^2, the ratios of the peak acceleration at the top of the building obtained by the analyses to the tests values were 0.94 to 1.06. In the case of Hachinohe 600cm/s^2, the ratio was 0.81.

Figure 20 show detail of the time history of acceleration at the top of the building. There is a pulse in the test response at 2.09sec, but analysis cannot represent it. This pulse could be seen from BH2 to BH6, but there wasn't pulse in time history at 2.09 sec of BH1 at the basement, so the pulse came from some response of the building, not from the input motion. In the case of El Centro, the kinds of pulses were not present in the time history, so the tests and the analytical values were very close.

CONCLUSIONS

We proposed a simplified method for dynamic response analyses, which consisted of the Sway-Rocking model and an equivalent linearization method. The accuracy of the method was examined using results of shaking table tests with an elasto-plastic soil-pile-building system.

Results of the tests and analyses were as follows:

(1) The average value of the maximum shear strain of the soil was found to range from 0.001 to 0.013, according to the condition, the natural frequency of the soil-pile-building system was found to range from 10.5Hz to 6.1Hz and the amplification factor ranged from 10.6 to 4.4.

(2) The ratios of the peak acceleration at the top of the building obtained by the analysis to the value obtained form the tests were 0.94 to 1.06 expect in the case of Hachinohe 600cm/s^2. In the case of Hachinohe 600cm/s^2, the ratio was 0.81.

(3) Generally, the analytical values and the tests values were close in the transfer function, the time history of the acceleration at the top of the building and the peak acceleration at the building, so the proposed method has enough accuracy for engineering purposes.

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Table 1 Similitude ratios

Item		Ratio (Model/Prototype)	
Soil Density	kg/m³	1 / η	1
Length	m	1 / λ	1/40
Acceleration	m/s²	1	1
Displacement	m	1 / λ	1/40
Mass	kg	1 / η λ ³	1/6.4×10⁴
Shear Modulus	N/m²	1 / η λ	1/40
Frequency	1/s	√ λ	6.325
Velocity	m/s	1 / √ λ	1/6.325
Stress	N/m²	1 / η λ	1/40
Strain		1	1

Table 2 Characteristics of tests model

Foundation		Building		Characteristics of Fixed Base Building	
Size (m)	Mas (kg)	Height (m)	Mss (kg)	Natural Freq. (Hz)	Damping Factor (%)
0.3 × 0.3	6.79	0.787	28.4	18.8	0.22

Table 3 Characteristics of the ground

Item	Upper Layer (GL ~ GL-0.45m)		Lower Layer (GL-0.45 ~ 0.6m)
	Center	Edge	
Vs(m/s) Damping Factor (%) Density (kg/m³)	23.7 6.63* 1.57x10-3	18.4 5.57 1.17x10-3	36.0 6.05 1.41

*Strain level is 3.6×10-4

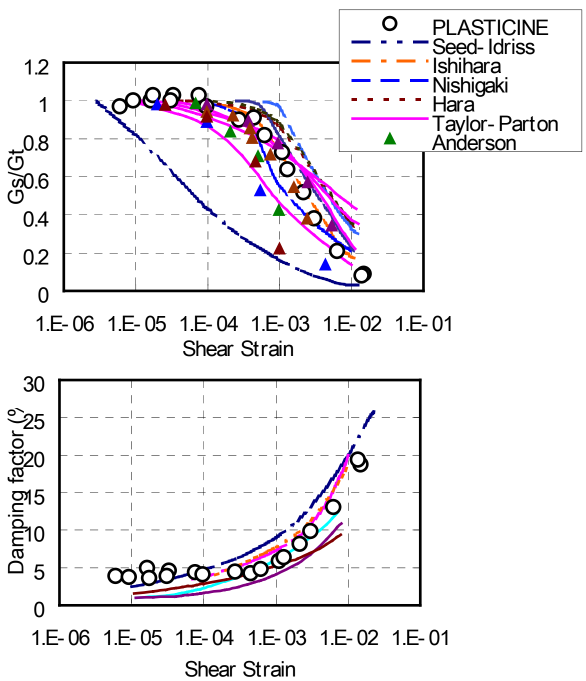


Figure 1 Tri-axial test results of model soil

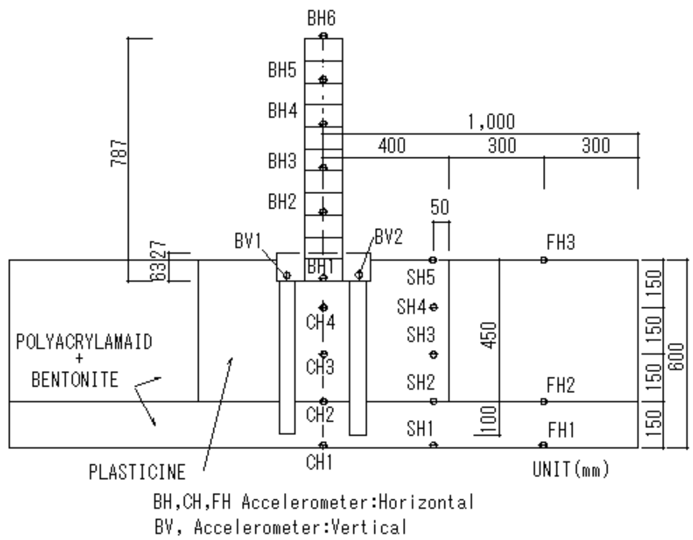


Figure 2 Section of tests model

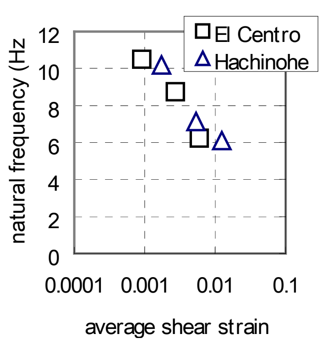


Figure 3 Natural frequency and shear strain of soil

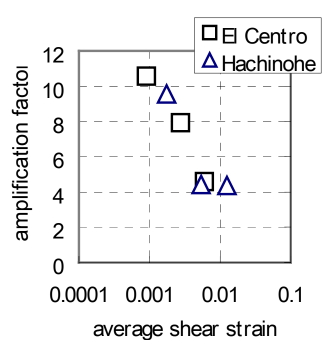
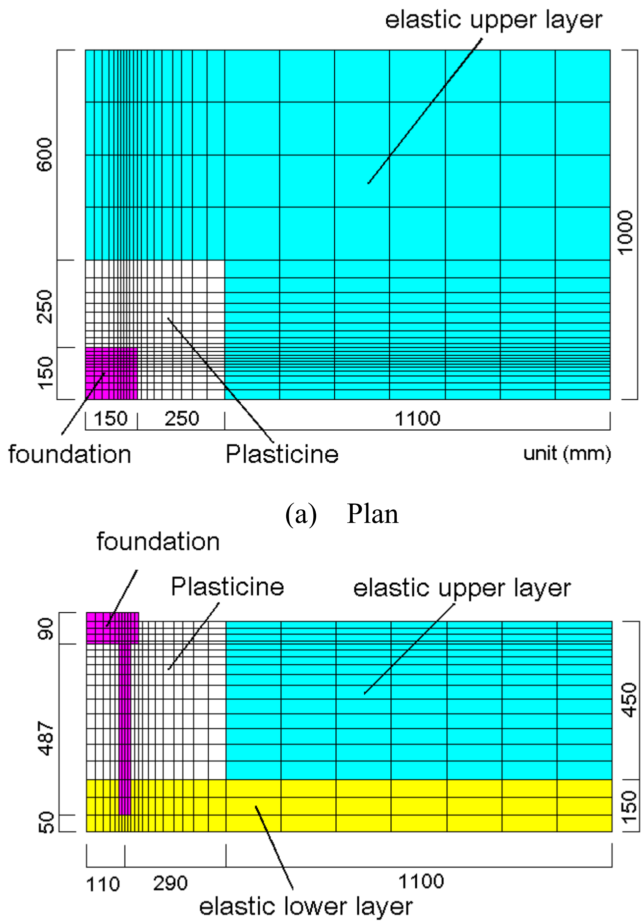


Figure 4 Amplification factor and shear strain of soil



(a) Plan

(b) Section

Figure 5 FEM model for static analysis

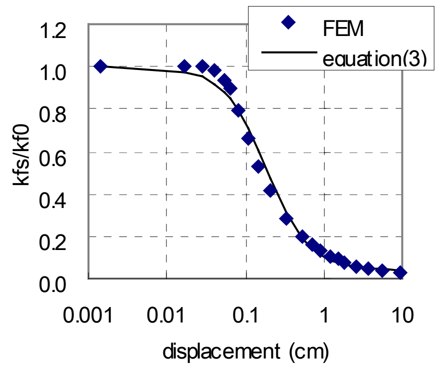


Figure 6 Lateral displacement and horizontal stiffness by static FEM analysis

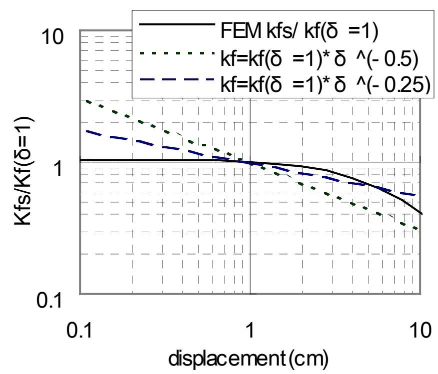


Figure 7 Lateral displacement and horizontal stiffness by static FEM analysis in full scale model

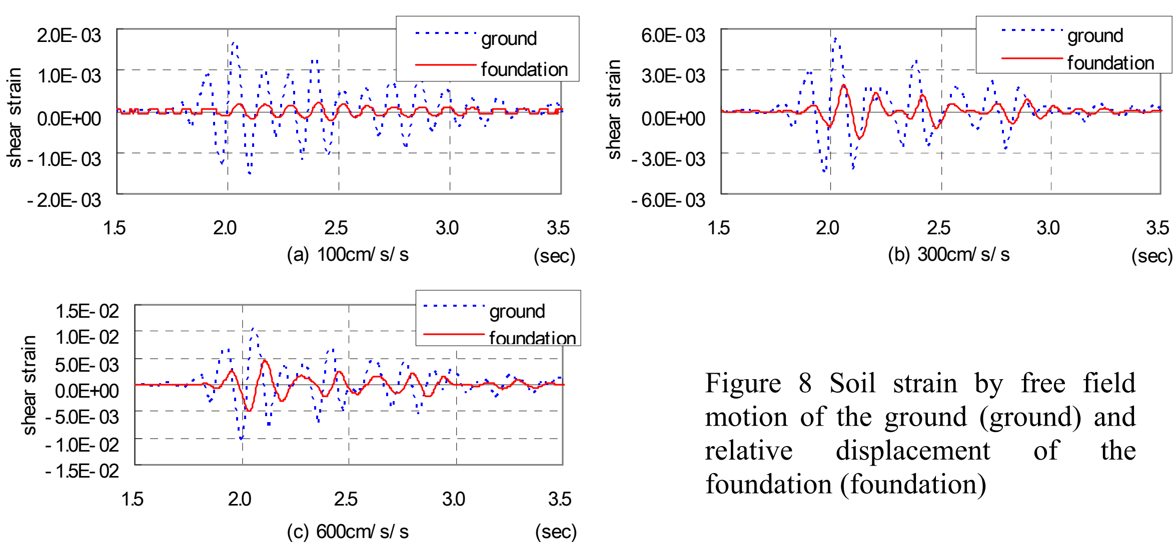


Figure 8 Soil strain by free field motion of the ground (ground) and relative displacement of the foundation (foundation)

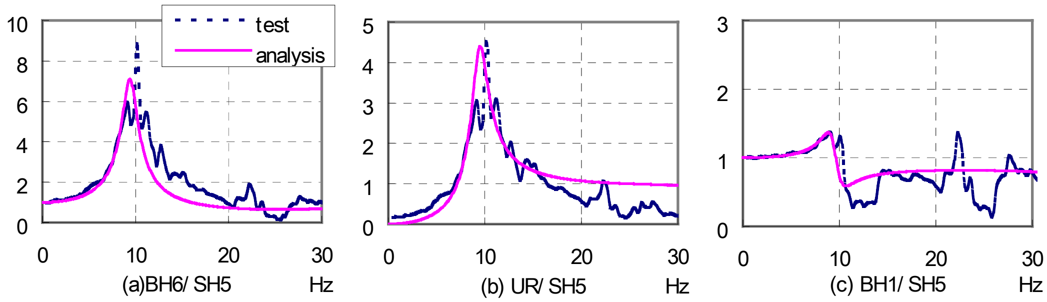


Figure 9 Transfer function (Hachinohe 100cm/s^2)

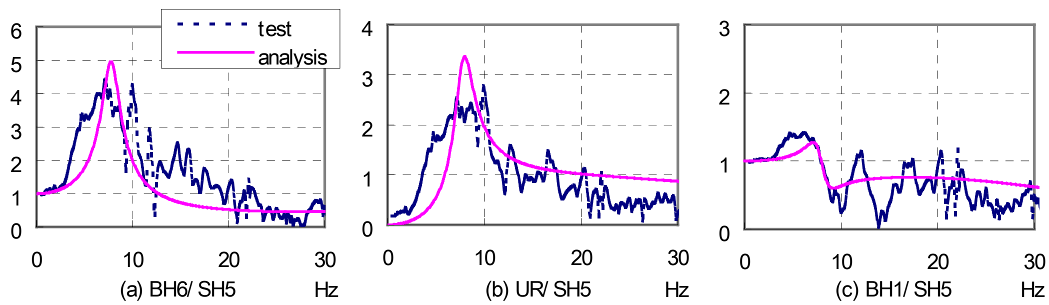


Figure 10 Transfer function (Hachinohe 300cm/s^2)

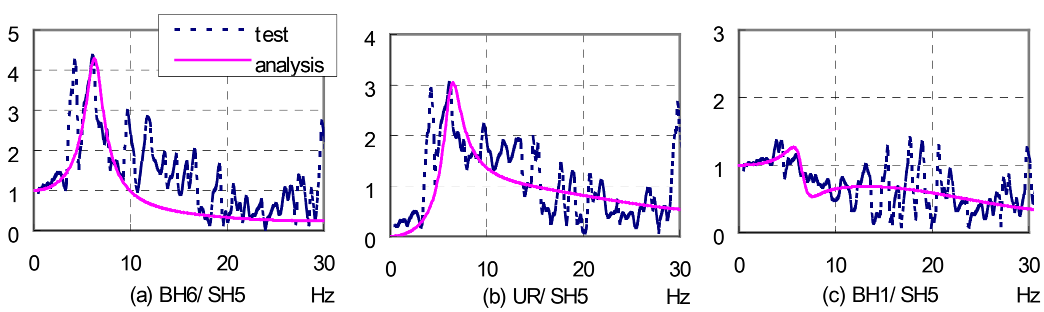


Figure 11 Transfer function (Hachinohe 600cm/s^2)

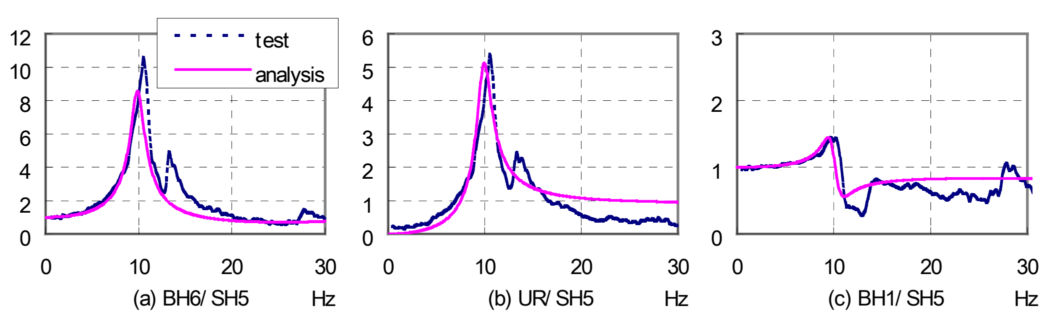


Figure 12 Transfer function (El Centro 100cm/s^2)

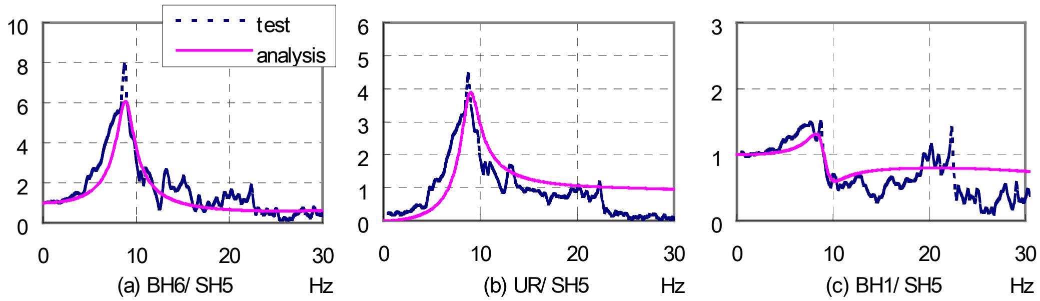


Figure 13 Transfer function (El Centro 300cm/s^2)

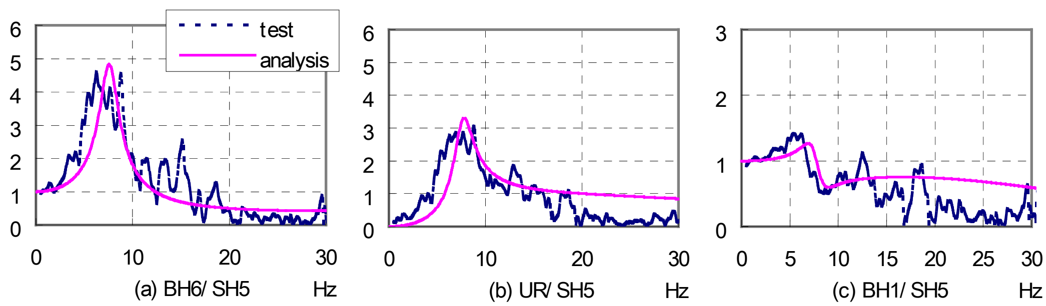


Figure 14 Transfer function (El Centro 600cm/s^2)

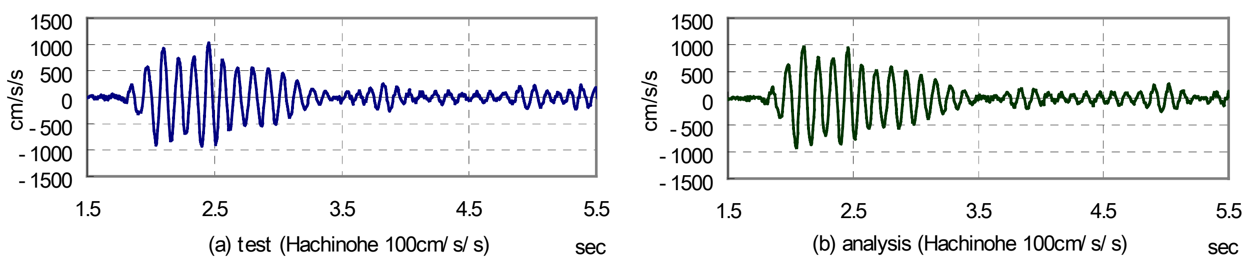


Figure 15 Acceleration time history at the top of the building
(Hachinohe 100 cm/s/s)

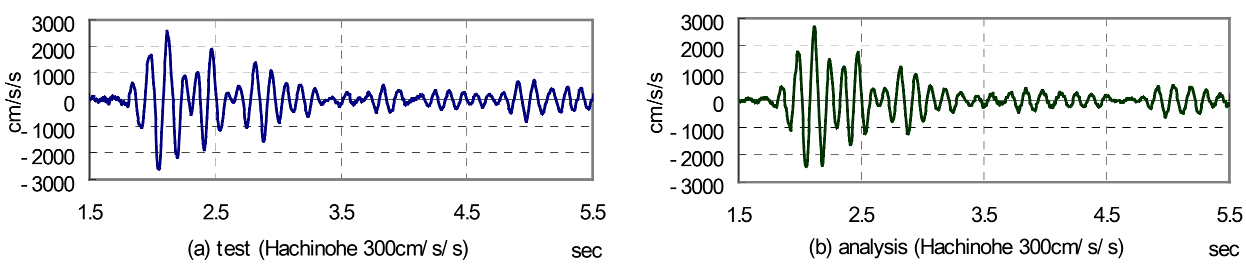


Figure 16 Acceleration time history at the top of the building
(Hachinohe 300 cm/s/s)

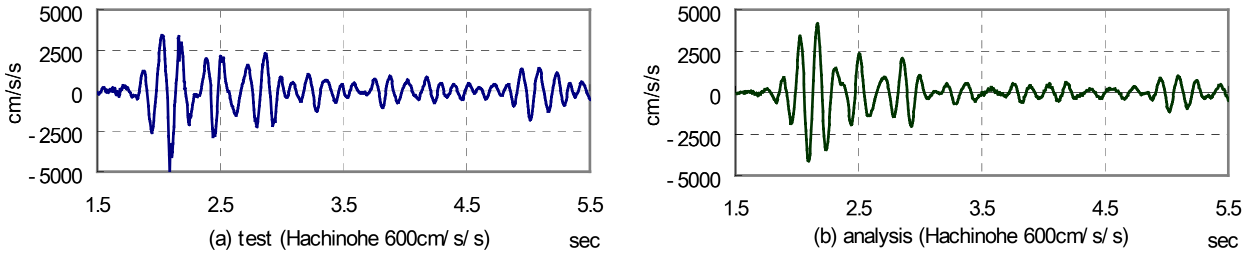


Figure 17 Acceleration time history at the top of the building (Hachinohe 600 cm/s/s)

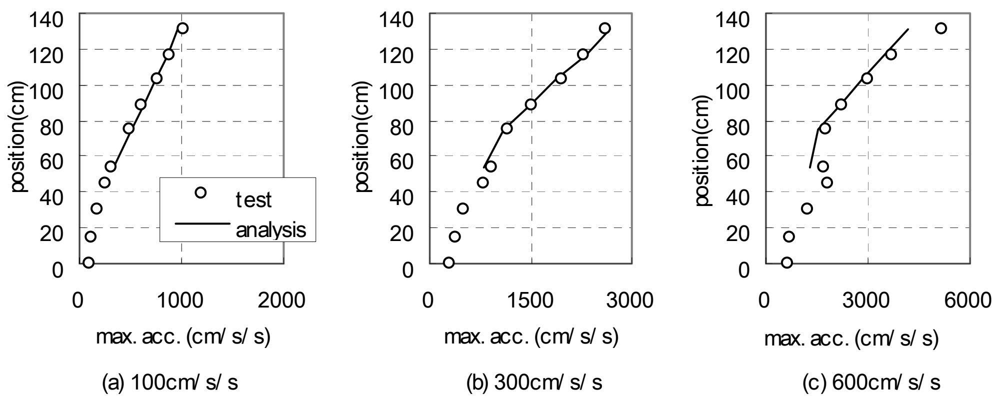


Figure 18 Peak acceleration of the building (Hachinohe)

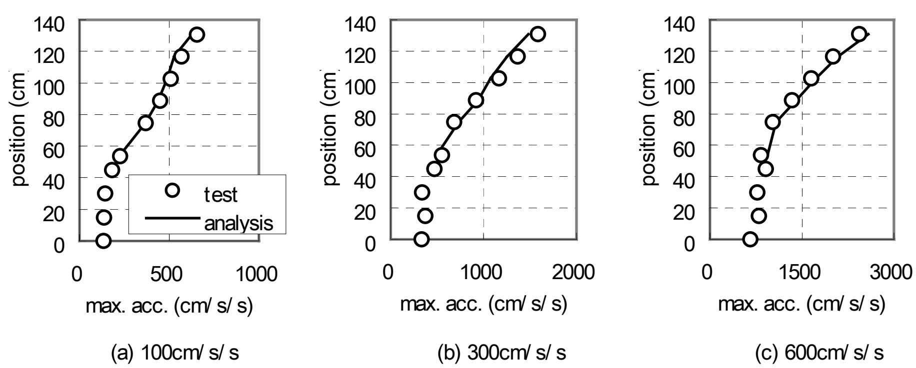


Figure 19 Peak acceleration at the building (El Centro)

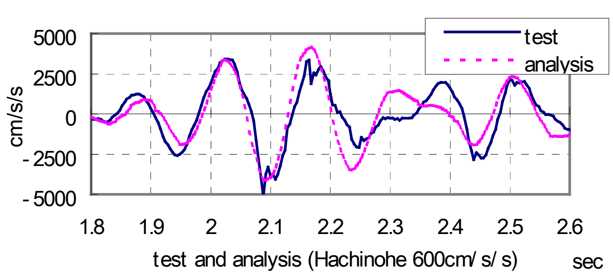


Figure 20 Acceleration time histories at the top of the building in detail (Hachinohe 600 cm/s/s)