DYNAMIC RESPONSE ANALYSIS OF SOIL-PILE-BUILDING INTERACTION SYSTEM IN LARGE STRAIN LEVELS OF SOILS

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SUMMARY

A series of shaking-table tests of a scaled soil-pile-building model were performed in order to study the effects of the plastic deformation of soil on dynamic characteristics of the soil-pile-building interaction system. Results showed the natural frequency and amplification factor decreased by 40% and 60%, respectively, when shear strain of soil was $4.2x10^{-2}$. Dynamic response analyses, which combined the Sway-Rocking model and an equivalent linearization method, were done. Difference in the natural frequencies by the test and by the analyses was within about 20%. For the amplification factor and the maximum acceleration, the difference became 12% and 19%. The difference in the amplification factor and the maximum acceleration was caused by overestimation in damping effects in the dynamic stiffness of the piles and beneath the foundation.

INTRODUCTION

When designing a building, it is important to evaluate earthquake performance of a building including non-linear soil-building interaction effects during an earthquake. In order to consider effects of soil non-linearity, FEM model or a mass-spring model, known as Penzien model, are efficient, but FEM model needs extremely much time to compute dynamic response of interaction models and there are complicated problems to determine soil springs or imaginary mass around piles in Penzien model. In a practical designing of a building, analytical methods should be simple so that, for example, an equivalent linearization method, like SHAKE [9], have been used frequently to evaluate a ground response. But, in the case of the non-linear soil-building interaction system, the accuracy of the method had not been tested enough.

In this study, a series of shaking table tests were done in order to evaluate the effect of plastic deformation of soils on dynamic characteristics of soil-pile-building interaction system. Dynamic response analyses, which combined Sway-Rocking model and an equivalent linearization method, of the tests were also done to evaluate the accuracy of this analytical method.

PLASTIC MATERIAL FOR 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 [1].

Figure 1 shows the soil characteristics, strain-shear modulus and strain-damping factor relationships for actual clayey soils and Plasticine, which is the plastic soil material used in this shaking table tests. The initial shear modulus, G_t (strain being 1.0 x 10^{-5}), shear modulus at each strain, G_s, and damping factors, h_g, were obtained by tri-axial compression tests in which ambient stress were kept at 1.0 kg/cm^2 and exciting frequency was 1.0 Hz. The shear modulus and damping factor of the plastic soil material, Plasticine, has strain dependency similar to those of actual clayey soils.

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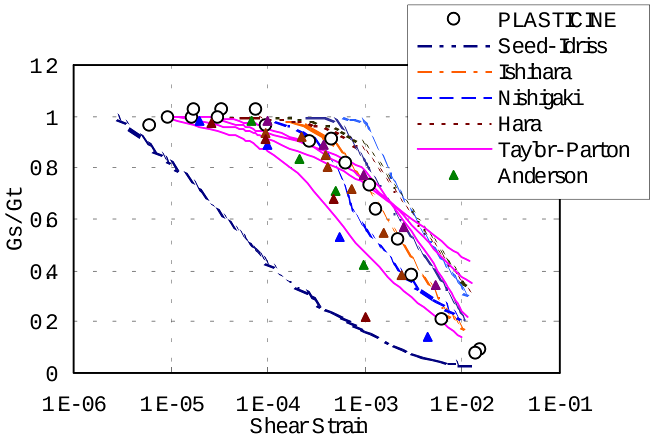
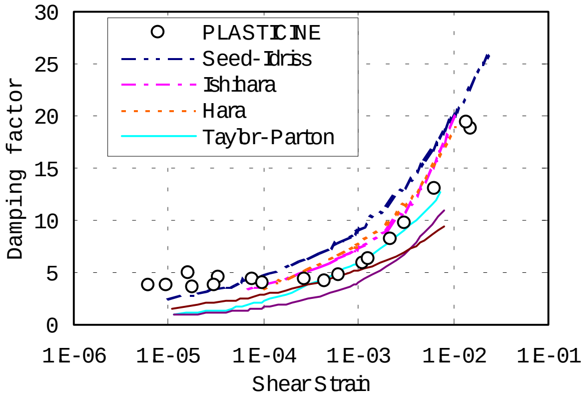


(a) Shear modulus ratios versus shear strain



(b) Damping factor versus shear strain

Figure 1: Soil characteristics

OUTLINE OF SHAKING TABLE TESTS

The similarity, which was proposed by Buckingham, was used in modeling the building and the ground soils. 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 shear modulus-strain and damping factor-strain relations similar to those of the prototype [4]. 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 wide strain levels of soil.

Figure 2 shows an outline of the building and the ground model together with the location of the measurement apparatus. Two dwelling units 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 weight and its columns were made of steel plates. The building foundation was made of aluminum and acryl plates. Four cylinder-shaped(∅38mm, length is 487mm) pile models were made of steel plate attached with rubber, they were set at the corners of the foundation. The bottom of the foundation was attached to the soil ground.

Table 1: Similitude ratios

Item		Ratio(Model/Prototype)	
Soil Density	kgf/cm^3	$1/\eta$	1
Length	cm	$1/\lambda$	1/40
Acceleration	cm/s^2	1	1
Displacement	cm	$1/\lambda$	1/40
Mass	$kgf.s^2/cm$	$1/\eta\lambda^3$	$1/6.4\times10^4$
Shear Modulus	kgf/cm^2	$1/\eta\lambda$	1/40
Frequency	1/s	$\sqrt{\lambda}$	6.325
Velocity	km/s	$1/\sqrt{\lambda}$	1/6.325
Stress	kgf/s^2	$1/\eta\lambda$	1/40
Strain		1	1

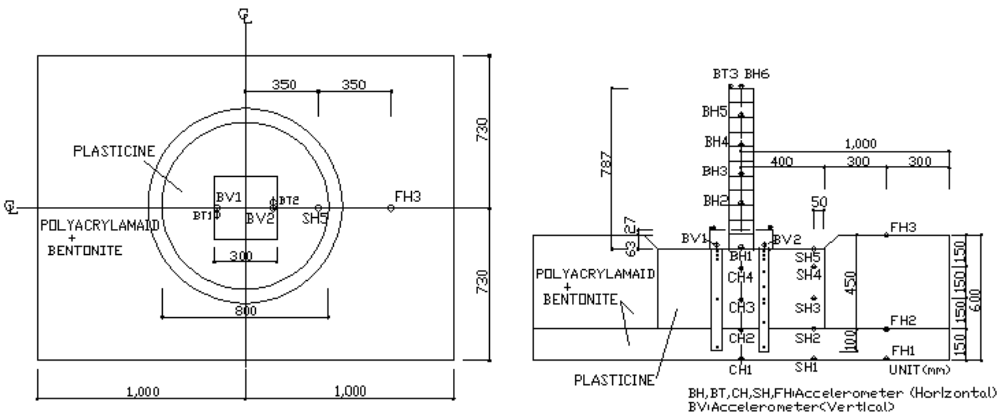


Figure 2: Test model and measurement apparatus

Table 2: Characteristics of building model

Foundation		Building		Characteristics of fixed base building	
Size (cm)	Weight (kgf)	Height (cm)	Weight (kgf)	Natural Freq. (Hz)	Damping Factor (%)
30 x 30	6.79	78.7	28.4	18.8	0.22

Table 3 Characteristics of ground model

Item	Upper layer (GL? GL-45cm)		Lower layer (GL-45? 60cm)
	Center	Edge	
Vs (m/s)	23.7	18.4	36.0
Damping factor (%)*	6.63*	5.57	6.05
Density (gf/cm^3)	1.57	1.17	1.41

* Strain level is 3.6×10^{-4}

The ground model has a block shape and its size is 2x1.46x0.6m. Stainless plates were set at both side ends in transverse direction of the ground to prevent vertical motion of the ground. The central part (φ800mm, depth is 387mm) 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 characteristics of the ground. Damping factors were obtained by a free tensional vibration test and shear wave velocity was obtained by the P-S wave propagation tests.

An earthquake record in which the time length was corrected according to the similarity was used for the input ground motion, 1968 Hachinohe EW. Maximum acceleration of the input motions was set as 100, 300 and 600 cm/s^2 on the shaking table.

RESULTS OF TESTS

Figure 3 shows first natural frequency estimated by spectral ratios of BH6/SH5 (see Figure 2). The shear strain shown in Figure 3 is maximum strain that calculated from displacement at BH1, CH3 and CH4. This strain was calculated as follows:

1) Displacement at BH1, CH3 and CH4 are calculated by integrating acceleration records at those points.
2) Maximum relative displacement between BH1 and CH4, and that between CH4 and CH3, divided by the distance of those points becomes maximum strain between BH1 and CH4, and that between CH4 and CH3.
3) Maximum strain shown in Figures 3 and 4 are average value of maximum strain between BH1 and CH4, and that between CH4 and CH3.

Figure 4 shows the amplitude of the spectral ratio at the natural frequency. The natural frequency was decreased by 40% and the amplitude of spectral ratio was decreased by 60% at most when the shear strain of soil was $4.20\text{x}10^{-2}$.

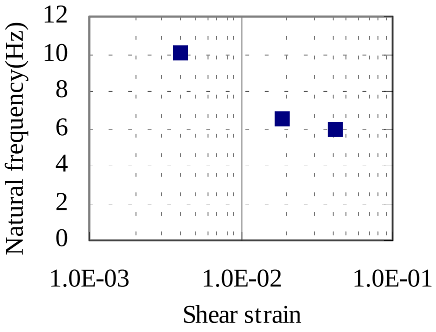


Figure 3: Natural frequency versus shear strain of soil

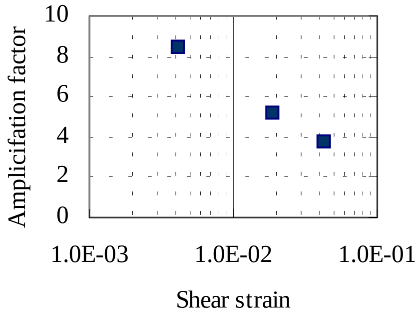


Figure 4: Amplification factors versus shear strain of soil

THEORETICAL MODEL

5.1 General of Theoretical Model

The theoretical model employed in this study is a Sway-Rocking model, and an equivalent linearization method was used for dynamic response analyses. The equivalent linearization method was employed in order to consider plastic deformation of soils

5.2 Dynamic Stiffness of Foundation

Dynamic stiffness for sway and rocking motion of the foundation were calculated as follows:

1) Dynamic stiffness of piles for horizontal and rocking motion proposed by Novak and Nogami [8] was employed.

2) Vertical stiffness of piles was calculated by a method proposed by Novak[7]. This method is known as plane strain case, so, an adjustment of the stiffness needed. In this analyses, real part of the stiffness is set to be constant and imaginary part of it is proportion to frequency, when non-dimensional frequency, $2\pi \times f \times r_0 / V_s$, where f is frequency, r_0 is radius of the pile, V_s is shear wave velocity of the upper layer and of pile, is lower than 0.3.

3) Group effects of piles were considered by a method, which derived by static analyses of piles in a ground by a FEM model [3]. In this case a coefficient of group effect became 0.76 by the method.

4) Dynamic stiffness of the bottom of the foundation was calculated by the D.G.C. [5].

5) In this study, the dynamic stiffness of the soil-pile- foundation system was calculated by the sum of the dynamic stiffness of the piles and that of the bottom of the foundation.

5.3 Strain Dependency of Shear Modulus and Damping Factor of Soil Ground

Relationships between soil strain and shear modulus, G_s, and damping factor, h_g, of the soil were determined by the tri-axial compression tests, as shown in Figure 1, according to the following equation (1) and (2) which is modified from the Hardin-Drnevich model.

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

$$h_g = 0.035 + 0.145(1 - G_s / G_t) \quad \cdots\cdot\ \cdot \tag{2}$$

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

5.4 Method to estimate equivalent strain of soil

As shown in equation (1) and (2), strain beneath the foundation, γ_s, determines the stiffness and the damping factor of soil ground. The strain of the soil assumed to be sum of that caused by wave propagates upward from bottom of ground, γ_{wave}, and that cased by relative deformation of foundation, γ_{base}. γ_{wave} was average of the maximum strain between SH3 and SH4, those between SH4 and SH5.

γ_{base} was estimated from the maximum relative displacement of the foundation, $u_{b,max}$, as follows:

Displacement of soil at depth z, $u_b(z)$, was assumed to be determined by equation (3).

$$u_b(z) = \frac{B_1}{B_1 + z/\sqrt{bc}} u_{b,\max} \tag{3}$$

Where B_1 =0.67, b and c are the width of half the foundation in the vibration and transverse direction, respectively.

This formula was proposed by Kobori et. al. [6]. By averaging the strain from z = 0 to H, considering energy caused by the displacement, $u_b(z)$, the equivalent maximum strain of the soil caused by displacement of the foundation, γ_{base}, becomes,

$$\gamma_{base} = \sqrt{\int_0^H \frac{1}{H}\left(\frac{du_b(z)}{dz}\right)^2 dz} \tag{4}$$

Where $H = b$.

The maximum shear strain of the soil beneath the foundation, γ_{soil}, is estimated by equation (5) and equivalent strain of soil beneath the foundation, γ_{eq}, is estimated by equation (6).

$$\gamma_{soil} = \sqrt{\gamma_{base}{}^2 + \gamma_{wave}{}^2} \tag{5}$$

$$\gamma_{eq} = 0.7\gamma_{soil} \tag{6}$$

The maximum strain caused by wave propagation, γ_{wave} , and relative deformation of the foundation to the ground, γ_{base}, did not occurred at the same time, so, the maximum strain beneath the foundation, γ_{soil} , was calculated by square root of sum of squares of γ_{wave} and γ_{base} as equation (5). Ratio of the equivalent strain divided by maximum strain was set to 0.7 in this case. This constant, 0.7, was employed to consider non-stationary of the amplitude of the soil strain.

Figure 5 shows maximum soil strain obtained by the tests. In this figure, γ_{wave} is observed value and γ_{base} was calculated by equation (3) and (4) by using observed value of $u_{b,max}$. γ_{obs} is observed value and it is average value of the maximum strain between BH1 and CH4 and that between CH4 and CH3 (see Figure 2). RSS and ABS is square root of sum of squares of γ_{wave} and γ_{base}, and sum of their absolute value, respectively. RSS, which corresponds to γ_{soil} in equation (5). If equation (5) is right, RSS must be same as γ_{obs} in Fig. 5. As shown in Figure 5, RSS and γ_{obs} have similar value, so that estimation of soil strain by equation (5) has enough accuracy.

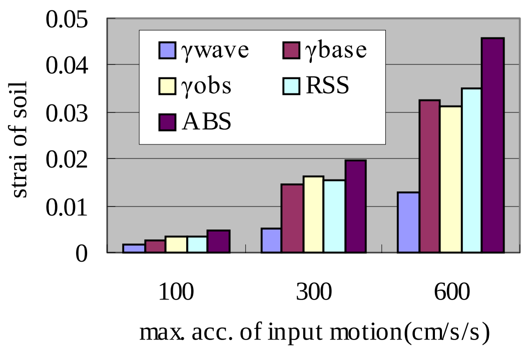


Figure 5: Maximum strain of soil observed by tests

RESULTS OF ANALYSES

Figure 6(a) shows the first natural frequencies detected form spectral ratio of BH6/SH5 (see Figure 2) and 6(b) shows amplification factors, which are the amplitude of the spectral ratio at the first natural frequency. Figure

6(c) shows maximum acceleration and 6(d) shows maximum shear strain of the soil beneath the foundation. As shown in Figs. 6(a)&(d), difference in the natural frequencies by the test and by the analyses were within about 20%. For the amplification factor and the maximum acceleration, the difference became 12% and 19%, respectively. The maximum shear strain beneath the foundation was underestimated by the analyses. This fact was caused by underestimation of swaying motion by the analyses.

Figs. 7, 8 and 9 shows spectral ratios derived by the tests and analyses. In these figures, UR is rocking motion at the top of the building and UH is relative deformation of the building between the top and the base of the building. When the maximum input acceleration was 100 cm/s^2, the natural frequency and the amplification factor at the top of the building was properly estimated by the analysis, but the amplification factor for rocking motion was overestimated and that for swaying motion was underestimated. When the maximum input acceleration is 300 cm/s^2 the natural frequency by the analysis is 1.2 times larger than that by the test. In this case, the amplification factor for rocking motion was overestimated and that for swaying motion was underestimated. In the case of 600 cm/s^2, the natural frequency estimated properly by the analysis and the amplification factor rocking motion underestimated, respectively. As written above, the relationships between results by the analysis and the tests were different by amplitude of input motion. When the maximum acceleration of input motion was 100 and 300 cm/s^2, the spectral ratio of swaying motion shows damping effects of dynamic stiffness of piles and the beneath the foundation overestimated by he analyses. The vertical stiffness of pile is calculated by plane strain case, so this assumption brought over estimation of damping effects of the soil ground.

Figure 10 shows maximum acceleration distribution at the building. As shown in this figure, shape of acceleration distribution had good agreement in the test and analyses.

CONCLUSION

This study involved performing shaking table tests on elasto-plastic soil material to investigate the soil-pile-building interaction system in large strain levels of soils. Dynamic analyses of the test, which incorporated Novak and Kobori's methods and an equivalent linearization method, were used to determine dynamic stiffness of foundation and piles.

Results of the analyses were as follows:

(1) The maximum strain of soil beneath the foundation can be estimated by square root of sum of squares of maximum strain by wave that propagate up word and that by relative deformation of the foundation and soil ground.

(2) Ratio of equivalent strain divided by maximum strain was set to be 0.7 in this case. Difference in the natural frequencies by the test and by the analyses was within about 20%. For the amplification factor and the maximum acceleration, the difference became 12% and 19%. The difference in the amplification factor and the maximum acceleration was caused by overestimation in damping effects in the dynamic stiffness of the piles and beneath the foundation.

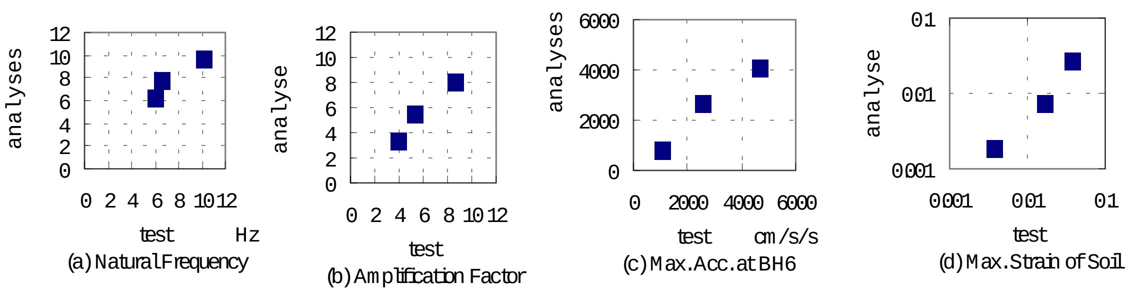


Figure 6:Comprison results of tests and analyses

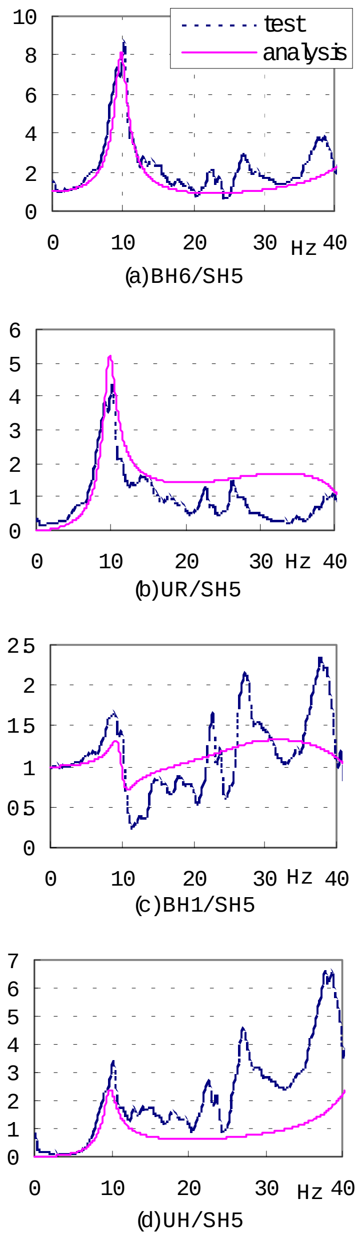


Figure 7: Spectral Ratio $(100cm/s^2)$

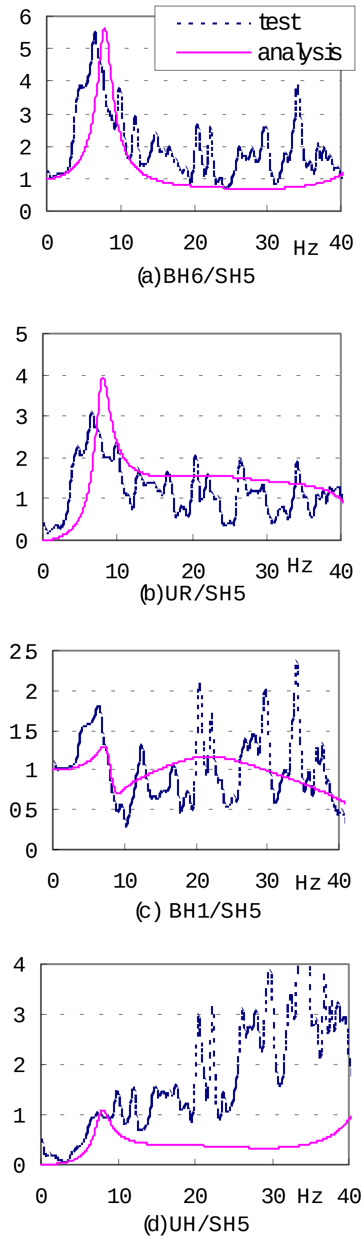


Figure 8: Spectral Ratio

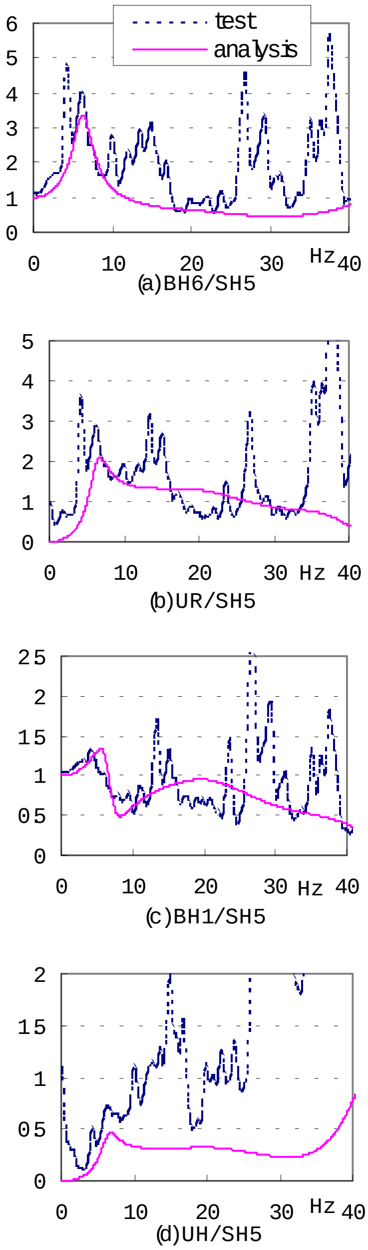


Figure 9: Spectral Ratio

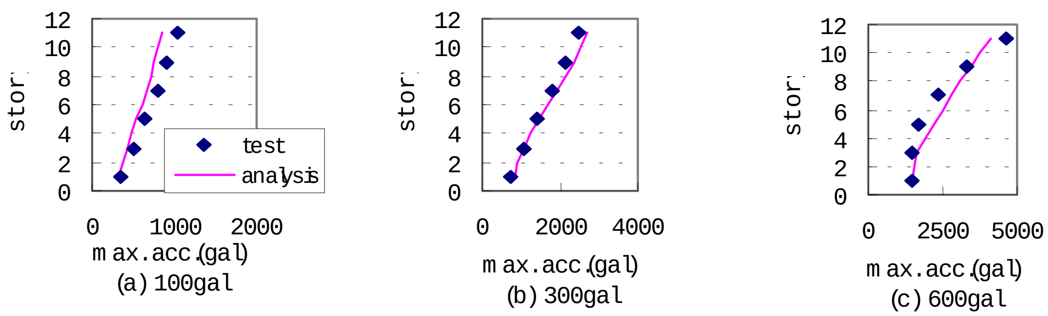


Figure 10: Maximum acceleration distribution at building

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