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AN ANALYTICAL STUDY ON COLLAPSING BEHAVIOR OF TIMBER STRUCTURE HOUSE SUBJECTED TO SEISMIC MOTION

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SUMMARY

To investigate the safety of the Japanese wood houses in large earthquake, it is necessary to confirm the ultimate seismic performance of the existing wood houses with insufficient specifications. In this paper, in this respect, the basic theory of the collapsing response analysis and the results of the analysis on existing Japanese conventional wood house are shown. The results are compared with the results of the shaking table tests to confirm the accuracy of the analytical theory. Next, some factors of the collapsing are considered. It is recognized that slight difference of hyteresis characteristics mainly decide to collapse or not.

INTRODUCTION

The standard of seismic design and the required specifications of buildings in Japan have been revised in every 10-20 years. The existing houses were designed according to the standard at the time, and constructed according to the specifications at the time. Consequently, the existing houses don't always have proper seismic performance suited to the newest standard. A large number of such houses collapsed in the past earthquake such as Hyogo-ken Nambu earthquake (Jan.17, 1995). To investigate the safety of the wood houses in large earthquake, it is necessary to confirm the ultimate seismic performance of the existing wood houses with insufficient specifications.

The purpose of this study is to develop the method of time history response analysis that can trace the collapsing process, as a part of the study on the estimation method of the ultimate seismic performance of the existing Japanese conventional wood houses. In this paper, firstly, based on the results of the full scale shaking table test on the Japanese conventional wood house that designed and constructed according to the old standard in Japan (Koshihara [1]), several collapsing behaviors are introduced. Secondly, the basic theory of the collapsing response analysis that has been led from the examination done as the first step of the development, and results of the response analysis, are shown. These results are compared with the test results to confirm the accuracy of the basic theory. Lastly, some factors of the collapsing are considered.

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OUTLINE OF THE SHAKING TABLE TESTS

Test models

The test models corresponded to Japanese conventional wood house that had been constructed before 30-40 years. Their shape were rectangular, 3.64m in X-direction, 5.46m in Y-direction and two stories in height as shown in Fig. 1 and Photo 1. The weight of test models is shown in Table 1. The ratio of the horizontal strength of the first story to the total weight was 0.144 in X-direction, 0.20 in Y-direction when the allowable horizontal strength of each timber bracing was 2.94kN regulated in the building standard law of Japan.

Four models were tested. They were same shape but different in their combination of input motions – No.1:only X-direction, No.2:only Y-direction, No.3:both X and Y-direction, No.4:both X,Y and Z-direction, as shown in Table 2.

Photo 1 View of the test model

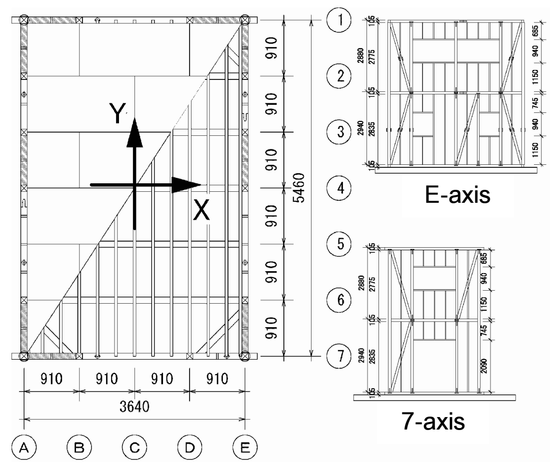


Fig. 1 Plan and elevation of the test model

Table 1 Weight of the test model

Level	Structure (kgf)	Additional Weight (kgf)	Total Weight (kgf)	Unit weight (kgf/m^2)
Roof	782	1500	2282	115
2nd floor	1138	2000	3138	160

Table 2 Direction and components of input seismic motions

Test model	Direction		
	X	Y	Z
No.1	R	—	—
No.2	—	T	—
No.3	R	T	—
No.4	R	T	U

Input seismic motion

The test models were subjected to the ground motion records obtained at JR Takatori station in Hyogo-ken Nanbu earthquake (Jan.17, 1995). NS and EW-component of the ground motion were rotated

40degree north to west to obtain R and T-component. R and T-component were input each to X and Y-direction. U(Vertical)-component was input to Z-direction being intact. The maximum value of R and T-component are shown in Table 3.

Table 3 Maximum value of the input seismic motions

Component	Acceleration (cm/sec^2)	Velocity (cm/sec)	Displacement (cm)
R	741	135	50
T	624	55	18

Circumstances of collapsing

The test model No.2 subjected to T-component in Y-direction didn't collapse. Of the test models subjected to R-component in X-direction, No.1 and 3 collapsed at approximately 12-15sec. from the start of the shaking, while No.4 didn't collapse. But the residual relative story displacements of No.4 were 356.5mm(1/7.4rad) in 1st story, 314.9mm(1/9.1rad) in 2nd story, which was seemed immediate to collapse. In both of No.1 and 3, 1st and 2nd story deformed to the shape of parallelogram at the same time, and then they collapsed.

All of the bracings in X-direction of No.1, No.3 and No.4 lost the horizontal strength from getting out or buckling, during several seconds from the start of the shaking. In No.1 and No.3, bending failures of the top of columns in 2-6-axis that were probably due to the binding force of the reinforcing bolts, were observed at approximately 6-9 sec. from the start. Bending failure of the columns at the corner wasn't observed before the finish of the collapsing.

COLLAPSING RESPONSE ANALYSIS

The basic theory of the analysis

Considering large-deformation and P-Delta effect, the method of time history response analysis for 3-dimensional FEM model with truss elements had been led (Wada [2]). In this paper, this method is applied to the collapsing response analysis of the models in the shaking table test. In this method, based on an assumption that tangential stiffness is constant during infinitesimal time, dynamic force balance is described as:

$$[M]\{\ddot{D}_{n+1}\}+[C_n]\{\dot{D}_{n+1}\}+[K_n]\{\Delta D_n\}+\{F_n\}=-[M]\begin{bmatrix}1&0&0\\0&1&0\\0&0&1\\&\cdots&\end{bmatrix}\begin{Bmatrix}\ddot{a}_{X.n+1}\\ \ddot{a}_{Y.n+1}\\ \ddot{a}_{Z.n+1}+g\end{Bmatrix} \quad (1)$$

where $[M]$ is the nodal mass matrix, $[C_n]$ is the viscous damping matrix, $[K_n]$ is the tangential stiffness matrix, $\{\Delta D_n\}$ is the incremental displacement vector $(=\{D_{n+1}\}-\{D_n\})$, $\{F_n\}$ is the internal force vector, $\ddot{a}_{X.n+1}$, $\ddot{a}_{Y.n+1}$ and $\ddot{a}_{Z.n+1}$ are accelerations of ground motion in X,Y and Z-direction, g is the gravity acceleration, subscript n means that the variable has a value at the time of $t=t_n$.

$[K_n]$ is generated from the tangential stiffness matrix of elements, $[k_e]$ shown in Eq. 2.

$$[k_e]=\begin{bmatrix}[k_{e1}] & [k_{e2}]\\ [k_{e2}] & [k_{e1}]\end{bmatrix} \quad (2)$$

where

$$[k_{e1}]=\begin{bmatrix}k_p & 0 & 0\\ 0 & N_n/L_n & 0\\ 0 & 0 & N_n/L_n\end{bmatrix} \qquad [k_{e2}]=\begin{bmatrix}-k_p & 0 & 0\\ 0 & -N_n/L_n & 0\\ 0 & 0 & -N_n/L_n\end{bmatrix}$$

k_p is the predicted tangential stiffness of elements according to the hysteresis model, L_n is the length of elements, N_n is the axial force of elements. The predicted value of the displacement vector, $\{D_p\}$ at the time of $t = t_{n+1}$ is approximately given as Eq. 3 to estimate the value of k_p.

$$\{D_p\} = \{D_n\} + \{\dot{D}_n\} \cdot \Delta t + \{\ddot{D}_n\} \cdot \frac{\Delta t^2}{2} \tag{3}$$

where Δt is incremental time in the analysis which is set as 1/300 sec. in this paper. $[C_n]$ is proportioned to $[K_n]$ as shown in Eq. 4.

$$[C_n] = \frac{2h}{\omega_1} \cdot [K_n] \tag{4}$$

where h is damping factor which is set as 0.02 in this paper, ω_1 is the natural circular frequency of 1st mode based on initial stiffness of the members.

The analytical model deforms slightly in each step. Eq. 1 based on the deformation at the time of $t = t_n$ is solved to obtain the response $\{\ddot{D}_{n+1}\}$, $\{\dot{D}_{n+1}\}$ and $\{D_{n+1}\}$ which is at the time of $t = t_{n+1}$. Therefore, the unbalances forces exist when these response are substituted for Eq. 1 based on the deformation at the time of $t = t_{n+1}$. These unbalance forces are included in $\{F_{n+1}\}$ which acts as the counter force to dissolve the unbalance forces in next step.

FEM model for the analysis

The 3-dimensional FEM model that corresponds to the models of the shaking table test was set as shown in Fig. 2. Total mass of the floor level, m_1 and the roof level, m_2 were set each as 3.138tons, 2.282tons according to Table 1. Total mass was divided to nodal masses so that the ratio of nodal masses in 1, 4 and 7-axis was 1:2:1.

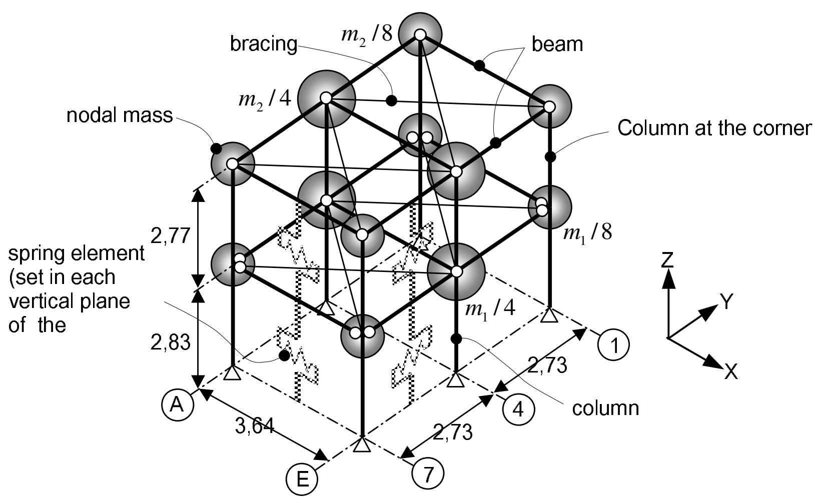
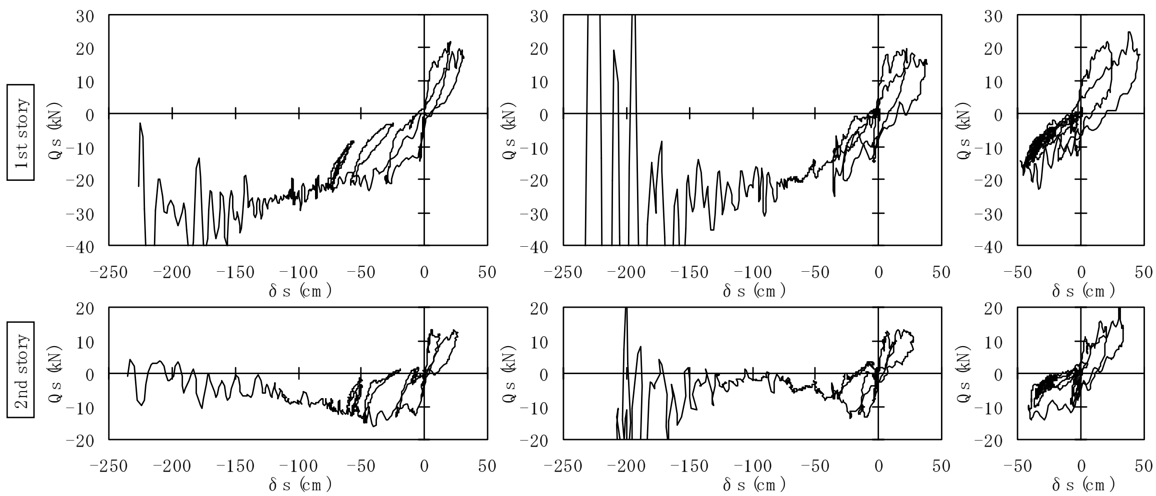


Fig. 2 FEM model

The elements that correspond to the columns at the corner were rigid-jointed at the floor level, pin-jointed at the other level. All of the other elements were pin-jointed. The bending stiffness of the columns at the corner was substituted to the relation of translational stiffness among the nodes at each level. The linear stiffness of columns and beams were led from the standard value of the elastic modulus (950kN/cm^2). The horizontal planes of floor and roof were substituted to the bracings so that the in-plane stiffness in each level was 470kN/rad. The horizontal force-deformation relationships of vertical planes of structure were substituted to non-linear spring elements in each vertical plane of the structure, as shown in Fig. 2. They had stiffness only in horizontal direction. Their hysteresis models were set as follows.

The story shear force, Q_s - relative story displacement, δ_s relationships of the model No.1, 3 and 4 were obtained in the shaking table test as shown in Fig. 3. The value of Q_s was subtracted horizontal force of P-Delta effect and the columns at the corner from the product of measured acceleration and total mass of each level. The strength of 1st story didn't go down when the relative story deformation got generally to 1.5-1.8m as shown in Fig. 3. It was probably due to share in story shear force of the frameworks. The hysteresis model shown in Fig. 4 (Magara [3]) was adopted as the model that corresponds to these Q_s - δ_s relationships. This model had been made referring Ohashi model (Ohashi [4]). The parameters in the hysteresis models were set for each test models. The ratio of strength in axis A and E was set 1:1, in axis 1, 4 and 7 was set 1:2:1.

The input seismic motions were the acceleration measured on the shaking table in the tests.



(a) No.1 (b) No.3 (c) No.4

Fig. 3 Story shear force-relative story displacement relationships in the shaking table tests (X-direction, excepting the force of P-Delta effect)

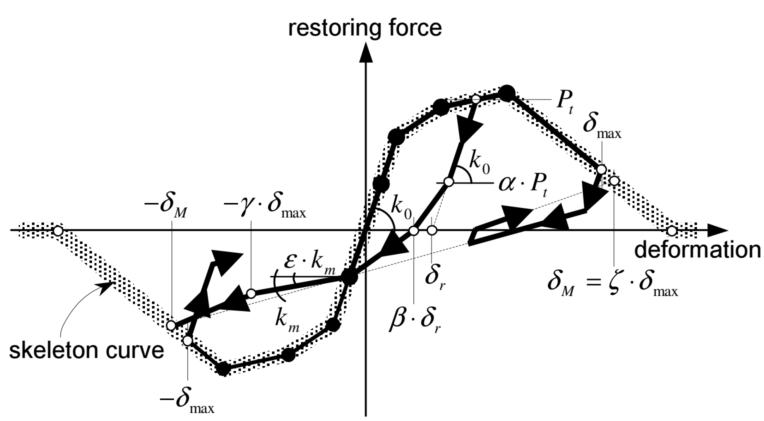


Fig. 4 The hysteresis model of the stories

Results of the analysis and its accuracy

The test model No.1 and 3 that collapsed in the test also collapsed in the analysis, No.4 that didn't collapse in the test also didn't collapsed in the analysis. The analytical Q_s - δ_s relationships in X-direction of the specimen No.1, 3 and 4 are shown in Fig. 5. The shape of Q_s - δ_s curves are generally agree with the experimental curves shown in Fig. 3. The analytical time histories of the relative story displacements in X-direction are shown in Fig. 6 comparing with the experimental values. The analytical values well agree with the experimental value, with respect to the time of collapse and the process of deformation to collapse. Fig. 7 shows the analytical deformation of the model No.1 at the specific time, comparing with experimental deformation. Both of them are similar each other.

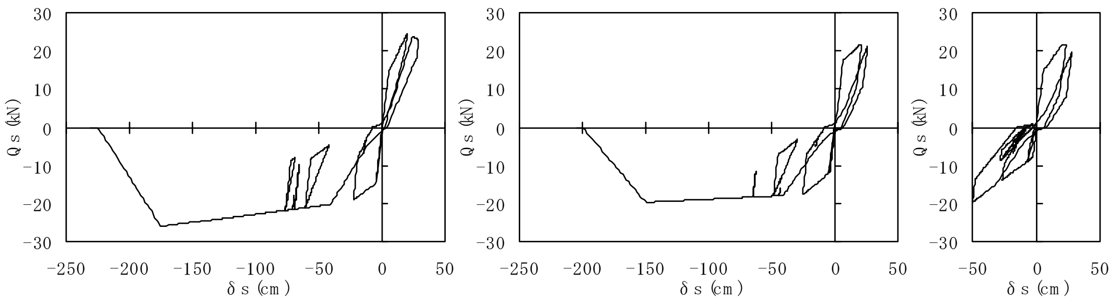
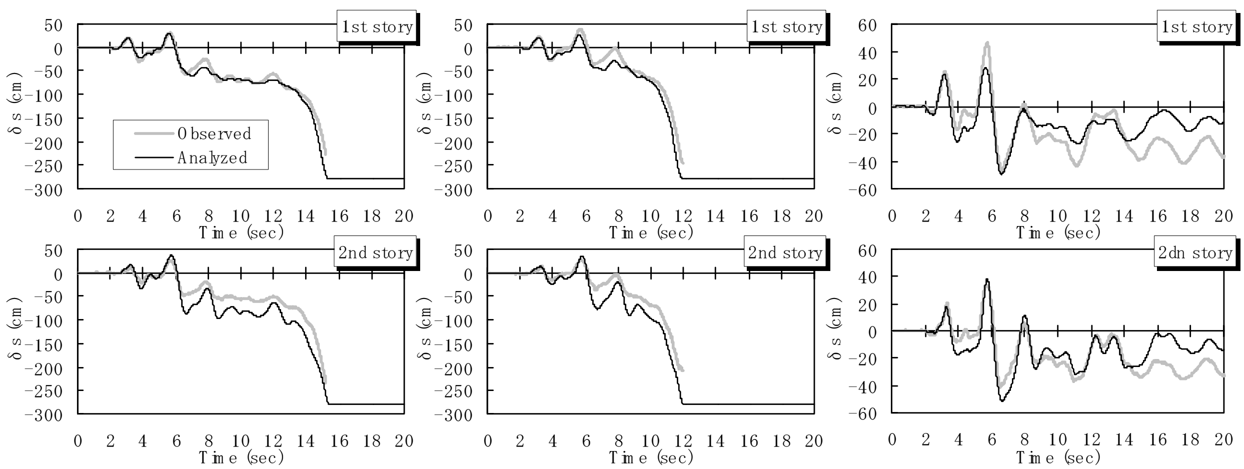
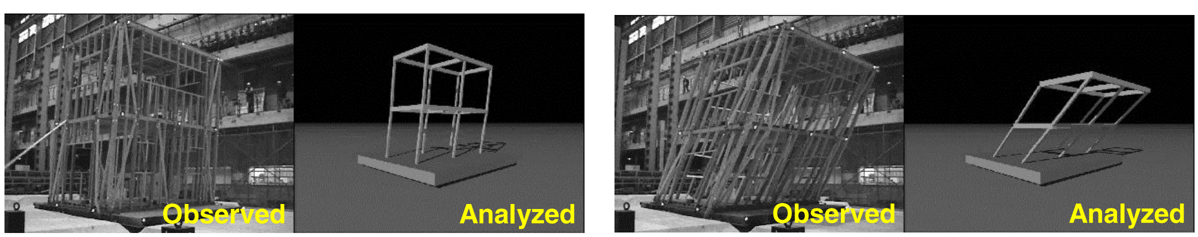


Fig. 5 Analyzed story shear force-relative story displacement relationships (X-direction, 1st story)



(a) No.1 (b) No.2 (c) No.3

Fig. 6 Time history of relative story displacement in X-direction



(a) 8.01sec. (b) 14.24sec.

Fig. 7 Deformation of the model No.1

ESTIMATION ON THE FACTOR OF COLLAPSING

Effect of the combination of input seismic motion

As above mentioned, the test model No.1 and 3 collapsed while No.4 didn’t collapse in both of the shaking table test and the analyses. Differences among these test models were only hysteresis characteristics and the combination of directions of input seismic motions.

First, to examine the effect of the combination of input seismic motions, each model was analytically subjected to all combinations of input direction, i.e. 1-direction (X), 2-drection (X, Y) and 3-drection (X, Y and Z). As the results of the analyses, the time histories of relative story displacement in X-direction are shown in Fig. 8. The model No.1 and 3 collapsed in all combination of input directions. The more the input direction, the earlier the model collapsed. Especially, obvious difference in the collapsing time is observed between 1-direction and 2-direction. The collapsing time of the model No.1 and 3 on 2-direction are earlier each approximately 4sec. and 8sec. than the collapsing time on 1-direction. In the model No.4, the relative story displacement tended to be biased on multi-direction input, however, the model didn't collapse in any combination of input directions. Therefore, in these models, the combination of input seismic motions wasn't critical to collapse, although the increase of input directions made collapsing time earlier.

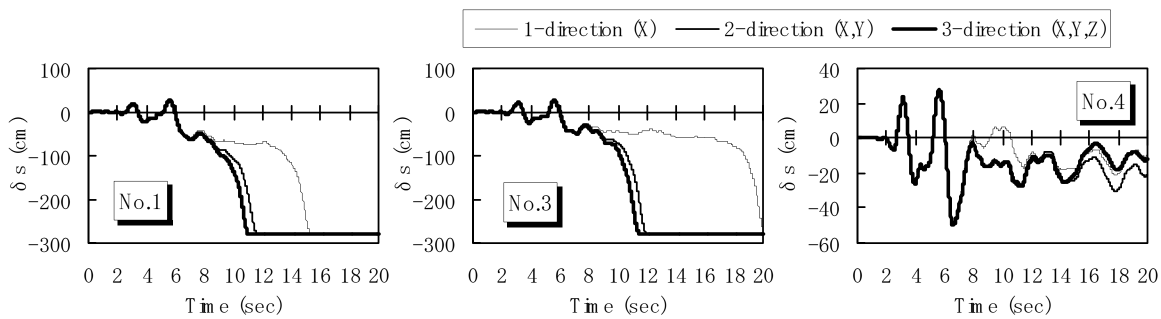


Fig. 8 Difference of the relative story displacement response of 1st story (X-direction) due to the combination of input seismic motions

Effect of the hysteresis characteristics

"The Calculation of Response and Limit Strength" (CRLS) is ruled as a method of the seismic design in the building standard low of Japan. CRLS is based on the equivalent linearization method where dynamic response of multi-story building is converted into the response of the equivalent single degree of freedom system (ESDOF) as shown in Eqs. 5a and 5b (Kuramoto [5]).

$$\Delta_1(t) = \frac{\sum_{i=1}^{N} m_i \cdot {}_1\beta \cdot {}_1u_i \cdot \delta_i(t)}{\sum_{i=1}^{N} m_i \cdot {}_1\beta \cdot {}_1u_i} \qquad (5a) \qquad\qquad A_1(t) = \frac{\sum_{i=1}^{N} P_i(t) \cdot {}_1\beta \cdot {}_1u_i \cdot \Delta_1(t)}{\sum_{i=1}^{N} m_i \cdot {}_1\beta \cdot {}_1u_i \cdot \Delta_1(t)} \qquad (5b)$$

where:

$\Delta_1(t)$ = response displacement of ESDOF at the time of t

$A_1(t)$ = response acceleration of ESDOF at the time of t

m_i = mass of i-story of multi-story building

${}_1\beta \cdot {}_1u_i$ = participation function on 1st mode of i-story

$\delta_i(t)$ = response displacement of i-story of multi-story building at the time of t

$P_i(t)$ = external force of i-story of multi-story building at the time of t

N = Number of stories

The relationship of $A_1(t)$ and $\Delta_1(t)$ is namely Capacity Spectrum (CS) which means the strength of the building. The relationship of the response acceleration spectrum, S_a and the response displacement spectrum, S_d is namely Demand Spectrum (DS) which means the intensity of seismic motion. The maximum response is estimated from the intersection of CS and DS.

Using this method, the response in X-direction of the model No.1, 3 and 4 were converted into the response of ESDOF to obtain CS. The participation function, ${}_1\beta \cdot {}_1u_i$ of 1st and 2nd stories were set as 1:2, referring the deformation at the turning point of displacement of Q_s - δ_s curves shown in Fig. 3. CS from the experimental response and the analytical response, including the horizontal force of P-Delta effect, is shown in Fig. 9 with DS of the input seismic motion in X-direction when damping factor, h=0.05 and 0.10. CS from the analytical response generally agree with CS from the experimental response. CS have obvious difference between collapsed model No.1, 3 and not collapsed model No.4, while Q_s - δ_s curves shown in Fig. 3 and Fig. 5 don't have obvious difference among each model in the region of ± 50 cm. CS of No.1, 3 intersected DS with negative tangential inclination, thereafter they collapsed. CS of No.4 reached the maximum displacement while tangential inclination was almost 0, thereafter turned displacement near the intersection with DS. The response accelerations, $A_1(t)$ of CS at the intersection with DS don't have correlation with the collapsing. In these models, it is recognized that the degree of negative tangential inclination of CS near the intersection with DS is critical to collapse.

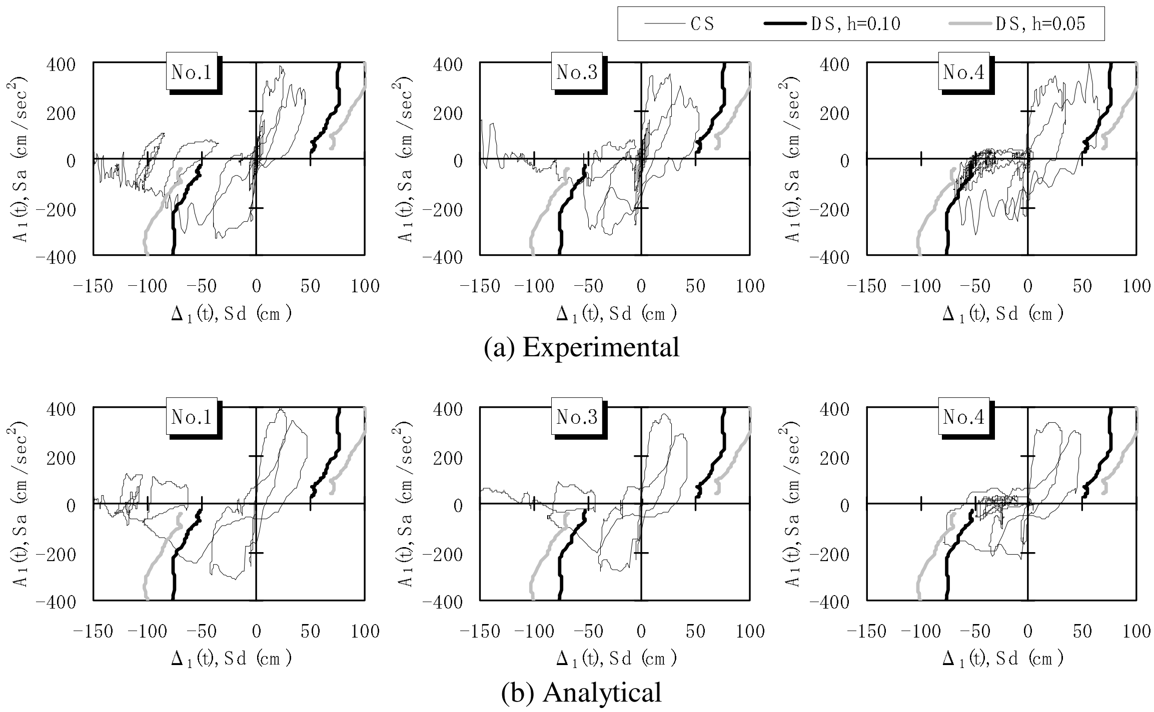


Fig. 9 CS of the equivalent single degree of freedom system and DS of the input seismic motions (X-direction)

CONCLUSION

Based on the past theory on the response analysis considering large-deformation and P-Delta effect, the basic theory of the collapsing response analysis for Japanese conventional wood house was led. To confirm the accuracy of the basic theory, the results of the analyses were compared with the results of the shaking table tests which were carried out with this study. As results, it was recognized that the analytical results generally agreed with the experimental results with respect to whether the models collapsed or not, the collapsing time, and the process of deformation to collapse.

In the shaking table tests, although all model had same shape and same specification, some models collapsed while another model didn't collapse. As the factors which decide to collapse or not, the combination of direction of input seismic motion and the eventual difference of the hysteresis characteristics were estimated.

To confirm the effect of the combination of input direction, each model was analytically subjected to all combinations of input direction. As result, the increase of input directions made the collapsing time early, however, in any combination of input directions, the models which collapsed in the test also collapsed, and the model which didn't collapse in the test also didn't collapse. The combination of input directions didn't decide to collapse.

Next, to confirm the effect of the difference of the hysteresis characteristics, based on an equivalent linearization method, the responses of the models were converted to Capacity Spectra (CS) of the equivalent single degree of freedom systems, and Demand Spectra (DS) of the input seismic motions were obtained. As result, it was recognized that the degree of negative tangential inclination of CS near the intersection with DS was critical to collapse.

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