# COLLAPSE OF WOODEN HOUSES CONSIDERING INSTANTANEOUS INSTRUMENTAL SEISMIC INTENSITY

Yasuko Kuwata\* and Shiro Takada Department of Civil Engineering, Kobe University, Rokkodai 1, Nada, Kobe 657-8501, Japan

# ABSTRACT

This paper addresses the collapse process of wooden houses subject to earthquake ground motions and characteristics of its ground motion by employing DEM analysis. In order to evaluate the traditional characteristic of ground motion, Instantaneous Instrumental Seismic Intensity (hereafter, IISI), which was previously proposed as an index of seismic intensity, is used. This paper combines both of the IISI and collapse process of house and evaluates the ground motion characteristics based on the results of DEM calculation, which induces a time-sequence of collapse modes.

# **1. INTRODUCTION**

Most of fatalities during disastrous earthquakes have been caused by collapses of weak buildings. In the case of the Kobe, Japan, earthquake, collapses of wooden houses constructed with timber framing in traditional Japanese style brought about 87 percent of fatalities [1]. In order to mitigate such fatalities, investigating seismic resistance of building, constructing buildings in appropriate seismic design to withstand expected seismic force and retrofitting them occasionally in the long term are important as earthquake prevention measures in the pre-event period, whereas clarifying how such houses collapse when strong ground motion exceeds the limitation of building stiffness is another crucial issue. Many studies have focused on improving allowable stresses of structures, but little consideration has been given to the structures' behavior in the state of over limited stresses. Following the Kobe earthquake, some researches tried to make clear the collapse mechanism of wooden houses. The present study also follows the behavior of wooden house until they are completely collapsed.

Of researches on the collapse process of houses, some are assessed based on laboratory tests. Ohashi et al [2] demonstrate collapse behaviors of full-scale wooden houses on the shaking table. Suzuki et al [3] investigate full-scale timber framing with several kinds of joints. The other approaches are based on the numerical analysis. Many researches using the FEM codes have assessed the behaviors of wooden houses until the limit stresses of the

<sup>\*</sup> E-mail address of the corresponding author: kuwata@kobe-u.ac.jp

structures are exceeded [4]. In the last decade, one numerical analysis techniques called <u>D</u>iscrete <u>E</u>lement <u>M</u>ethod (DEM) has been employed by the Earthquake Engineering community. The DEM incorporates an algorithm originally proposed by Cundall [5] for analyses of rock mechanics, and is able to simulate collapse as a sequence of times when parts of a structure interact with or separate from other parts. Many researches have used this powerful tool in order to simulate disruptive states of various structures, for example concrete building, bridge pier, buried pipe and surface ground with fault lines [6], [7], and [8]. Kiyono and Furukawa [9] [10] demonstrate collapse behavior of wooden houses and assess forces of members when houses fall down and members fail and contact other members. This paper employs DEM by devising new numerical models for expressing characteristics of wooden houses behaviors. For example, for joint parts of wooden houses, two kinds of spring characteristics (screw's stiffness for tensile stress and timber's compression stiffness for compression stress) are introduced. Moreover, non-linear springs that characterize wooden shear walls are incorporated.

The principal aim of this study is to discuss the collapse process of houses subject to earthquake ground motions and its characteristics of ground motion. Murata et al [11] proposed an index to assess capability for house to collapse, called fatigue spectral intensity, which is defined as accumulated spectral intensity. The authors employ here Instantaneous Instrumental Seismic Intensity (hereafter, IISI), which was proposed as an index of seismic intensity by the authors [12] in order to express the transitional characteristics of ground motion. The IISI was originally proposed for assess the time for human evacuation, and several relationship with parameters of wave propagation was obtained based on records of observed ground motion. The time available for human evacuation was discussed solely using the IISI. This paper combines both of the IISI and collapse process of house and evaluates the ground motion characteristics based on the results of DEM calculation, which induces a time-sequence of collapse modes.

# 2. TRANSITIONAL CHARACTERISTIC OF SEISMIC INTENSITY

### 2.1 Instantaneous Instrumental Seismic Intensity

Before describing the DEM calculation, the index evaluating for the transitional characteristics of ground motion is briefly explained. When estimating earthquake-related casualties due to collapsed houses, various quantities, such as peak ground acceleration, peak ground velocity, seismic intensity and SI (Spectrum Intensity) value, have been used as measures of ground motion. Hence Okada [13] showed detailed changes in both household and human behavior focusing on the time history of ground motion. Since there was no measure without time history of ground motion so far, a new index to express transitional characteristic of ground motion has been expected. So, a new measure associated with the Instrumental Seismic Intensity (hereafter ISI) adopted by the JMA in 1996 [14] was proposed by authors [12]. Whereas before the Kobe earthquake, only the two horizontal acceleration components were considered, after it the vertical component was added to the ISI. The new measure uses a calculation method, which computes the ISI value as instant duration, and therefore is named Instantaneous Instrumental Seismic Intensity (IISI). The

IISI is calculated by four steps (Figure 1). The first is to compute the Fourier spectrum of acceleration in each component (2 horizontal and 1 vertical) and to multiply the spectrum by three kinds of filter functions as shown in equations 1 to 3, then to compute the filtered acceleration by its inverse Fourier transform.

$$F_1(f) = (1/f)^{1/2} \tag{1}$$

$$F_{2}(f) = (1 + 0.694X^{2} + 0.241X^{4} + 0.0557X^{6} + 0.009664X^{8} + 0.00134X^{10} + 0.000155X^{12})^{-1/2}$$
subject to  $X = f/f_{c}$ 
(2)

$$F_3(f) = (1 - \exp(-(f / f_0)^3))^{1/2}$$
(3)

where  $F_1(f)$  is filter corresponding to human sensitivity to shaking,  $F_2(f)$  is high cut filter,  $F_3(f)$  is low cut filter, f is frequency, and  $f_c$  and  $f_0$  are parameters of the high and low cut filters, respectively ( $f_0 = 0.5$ Hz,  $f_c = 10$ Hz).



Figure 1. Calculation method of IISI

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The filter functions have the characteristics of increasing amplitudes in the range from 0.7 to 1.0 Hz and reducing amplitudes at higher and lower frequencies, as shown in Figure 2, for the purpose of representing damage to building and human perception. The next step is to calculate the vector acceleration, v(t), by summing the three components of filtered accelerations. In the third step, the maximum amplitude,  $a_0$ , of vector acceleration that satisfies more than 0.3 second duration similarly to the ISI calculation is determined by each interval of acceleration series. The duration is defined as the sum of the time intervals between first and final peaks that exceed a threshold level of vector acceleration. In the calculation, the vector acceleration series ( $v(\tau), v(\tau + 1), ..., v(\tau + j)$ ) from the beginning time  $\tau$ , is selected based on a given time interval. A vector acceleration series is sorted from the large to small amplitudes (s(1), s(2), ..., s(j+1)), and the maximum amplitude to have proceeding time over 0.3 second is obtained based on the above condition, and defined as the amplitude of vector acceleration at time  $\tau$ . Finally, the Instantaneous Instrumental Seismic Intensity  $I_{is}(\tau)$  at time  $\tau$  is computed by a formula which is related to the Kawasumi formula [15] as follow.

$$I_{is}(\tau) = 2 \cdot \log a_0(\tau) + K_I \tag{4}$$

where,  $K_I$  is parameter, equal to 0.94.



Figure 2. Characteristics of filter functions

The IISI at another time can be calculated by shifting the time window of vector acceleration series. The most parameters and conditioning functions follow those in the ISI procedure. The IISI calculation procedure takes into account two additional parameters; the duration of the time interval and the shift to the following time window. If the calculated value  $I_{is}(t)$  is less than 0.0,  $I_{is}(t)$  is regarded as equal to 0.0. The point at which the value of  $I_{is}(t)$  exceeds 0.0 is defined as the beginning of the IISI, symbolized as  $t_{IISI=0}$ . The new

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measure IISI calculated with 0.5 second time window of acceleration records with 0.1 second shifting between them was used in the previous study. The peak IISI is known to be almost equal to the ISI. The advantage of utilizing the IISI is that transitional characteristics of three components can be treated as a simply integrated value. Peak point of the IISI time history is drawn at the same time of the envelope curves of ground motion. The weak point is to involve complicated calculations. The IISI, however, can be used incorporating past researches on human behavior during earthquakes because these studies were done using the seismic intensity given on the JMA scale.

#### 2.2 Arrival time of IISI

We have conducted to analyze the duration until reaching the limit of human behavior using the IISI. Something interesting of the IISI was obtained for the characteristics of wave propagation throughout the pervious study. When the arrival time of IISI,  $T_{is}^{m}$ , is defined the time from the beginning of the IISI to the certain value of IISI, *m*, (see Figure 3) and the time reaching IISI of  $I_{is} = m$  is symbolized as  $t_{IISI=m}$ , it can be written by:

$$T_{is}^{m} = t_{IISI=m} - t_{IISI=0} \tag{5}$$

The arrival times for IISIs of  $I_{is} = 4.0$  and 5.0 are turned out to have a relationship with parameters of earthquake magnitude and hypocenter distance based on observed ground motions. (IISI of  $I_{is} = 4.0$  almost corresponds VI on the MMI scale, and IISI of  $I_{is} = 5.0$  does IIX.)



Figure 3 Arrival time of each IISI level

### **3. DEM MODELS FOR WOODEN HOUSES**

#### 3.1 DEM calculation procedure

The analysis method addressed here is the Discrete Element Method (DEM) in two dimensions. This study uses a DEM program that was developed by Hassani [16], called DEFA (Distinct Element Method for Fracture Analysis). As is known well, the DEM

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computes the dynamic behaviors of many discrete elements that interact through spring forces. Two types of springs, axial and tangential, are used here, thereby allowing transfer of forces and moments (i.e., torques) among neighboring elements. In a time step, forces and moments are calculated based on relative displacements and rotation of each element. Then, these forces and moments are summed for each element, and the position and rotation of each element is updated using its mass and moment of inertia based on Newton's second law of motion. This computational algorithm treats each component of a discrete element's position (displacement and rotation) as a *degree-of-freedom* (DOF) subject to external loading. These principal procedures in each time step are continued up to the termination time. The equation of motion for each DOF has the form

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = Q(t) \tag{6}$$

where, *m* is mass (or moment of inertia) of each DOF, *c* is viscous damping coefficient, *k* is spring stiffness, u(t) is relative displacement (or rotation) between elements, and Q(t) is an external force (or moment). In case of an earthquake load,  $Q(t) = -m\ddot{u}_g(t)$  for the horizontal and vertical degree of freedom in contact with the ground while the external moment are zero.

The viscous damping coefficient is determined by the critical damping ratio,  $\xi$  ( $\xi = c/c_{cr}$ ), which is equal to 0.01 (constant) in this study, and the critical damping coefficient,  $c_{cr} = 2\sqrt{mk}$ . In the case of elements with different masses  $m_i$  and  $m_j$ , the equivalent mass,  $\overline{m} = 2(m_i m_j / m_i + m_j)$ , is substituted. Apart from viscous damping, energy is dissipated during vibration cycles because the spring stiffness displays nonlinear hysteresis, which is described below for each spring type. If the relative displacement between elements exceeds a certain displacement, the spring between those elements is eliminated (cut off) and so can no longer transfer any force. This program basically computes the motions of each discrete element, so even after separating from neighbor elements, each element still moves. Moreover, if another element comes close enough to be regarded as a neighboring element, a new spring is introduced.

This program solves the equations of motion by means of the finite-difference timeintegration method. For numerical stability, the time step is less than the critical time step,  $\Delta t = 2\pi \sqrt{m/k}$ , as recommended by Hassani [16]. Early studies using this program have modeled various media, such as concrete blocks and soils, comprised of many elements. This study required development of special numerical techniques for modeling wooden houses since the simulation target is timber-framing structure and its typical failure mode, which is caused by weak shear walls and joint parts.

### 3.2 Modeling for wooden framed houses

Japanese wooden houses are generally composed of three structural elements; (1) timber framing members (timber beams and columns), (2) joint parts and (3) shear walls system (timber bracing and stiffness of wall materials). Typical joint parts connect by screws and angles or, in traditional Japanese joint parts, one timber member has a mortise (called Hozo in Japanese) into which the tenon of another columns fits. The failures of wooden houses are

dominated by failures of two structural elements, joint parts and shear walls.

Collapsed wooden houses rarely have significant failure of the framing members. Therefore, this study ignored deformation of timber members and treated each member as a rigid-body element. With respect to the interaction with other rigid bodies and circle-shaped elements, the rigid body does not have any direct connection. It involves several associated elements along the surface lines, and its associated elements contact with others, as shown in Figure 4. The forces on a rigid body are indirectly transferred from the associated elements. In a time step of computation, the spring forces on associated elements are calculated first by the relative displacements from each associated element to its neighboring elements. Then, horizontal and vertical forces and moments are summed at the centroid of each rigid body. Moving displacements and rotations of associated elements are subjected to that of their centroid. According to the updated centroid's position, associated elements subsequently move to their new positions.



Figure 4. Rigid body and associated elements

Characteristics for the joint parts that connect timber members are expressed using two spring types, axial and tangential components. In this study, the spring stiffness, k, is calculated according to the material property of elements as shown by

$$k = EA/L \tag{7}$$

where, E is Young's modulus, A is area, and L is length between associated element centroids.

The axial spring has different kinds of material properties in compression and tension, as shown in Figure 5. In the compression side (when relative displacement, u, is shorter than initial displacement), Young's modulus for timber is used for elastic spring stiffness. When compression load exceeds the limit force,  $F_c$ , which is provided by the timber's compression stress, the spring stiffness becomes zero, called perfectly plastic. On the other hand, in the tension side, the spring stiffness,  $k_t$ , is determined by the stiffness of joint screws. Generally the joint part is fixed with screws or attached by angles, but their stiffness is much smaller than that of timber compression. In tension, this spring also has elastic, perfectly plastic

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characteristics as charted in Figure 5. The tension force never exceeds the elastic limit force,  $F_t$ . The tension force decreases (unloads) subjected to the elastic spring stiffness,  $k_t$ , until the spring force equals to zero and then there is no force until the axial relative displacement returns to zero. When loading again, the spring stiffness follows the same hysteresis route until the previous maximum displacement, and above it the same plastic force is taken. In other words, when the relative displacement is zero in the loading and unloading cycles, the spring force is zero. Furthermore, when the relative displacement is longer than a certain displacement,  $u_{tl}$ , the joint spring is cut off (eliminated). When the joint part has contact again, spring force works only in compression side.



Figure 5. Relationship between displacement and force in axial component of joint parts

The tangential spring, which represents a timber column inserted in the mortise (hole) of another beam, behaves the same way as the timber compression in Figure 5. The contacting area in this hole is considered in this spring stiffness. The spring characteristics in both compression and tension are similar to that of the axial spring in the compression side. As the timber member is modeled as rigid body, the joint part's springs are placed between associated elements.

The shear walls in wooden houses are complicated structural systems, composed of many different timber bracings, wall mortar materials, and several doors and window spaces. These components are simplified into one wall stiffness whose stiffness is modeled by two crossed bracing springs that connect between the top of one column and the bottom of the other column as shown on the right in Figure 6. In the left of Figure 6, when shear wall with stiffness,  $k_Q$ , is loaded by a force, P, the relative displacement between floors,  $\delta_Q$ , is given by:

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$$\delta_Q = P/k_Q \tag{8}$$

On the other hand, the relative displacement,  $\delta_B$ , for the bracing frame on the right of Figure 6 is similarly given by

$$\delta_B = \frac{(L_B^2 + H_B^2)^{3/2}}{2L_B^2} \cdot \frac{P}{EA_B}$$
(9)

Here, both relative displacements are same ( $\delta_Q = \delta_B$ ). Therefore, the product of modulus and area for bracing spring can be expressed by the shear wall stiffness as follow.

$$EA_B = k_Q \cdot \frac{(L_B^2 + H_B^2)^{3/2}}{2L_B^2}$$
(10)

In the computation, area of bracing is assumed as 1.0 and Young's modulus of bracing is provided by equation 10.



Figure 6. Equivalent brace stiffness of frame wall

The shear wall stiffness in this study has multi-linear characteristics controlled by the relative displacement between floors,  $\delta$ , as shown in Figure 7. In the elastic state, the spring stiffness is constant, equal to elastic spring stiffness  $K_1$ . Once the relative displacement exceeds the elastic limit,  $\delta_e$ , the spring stiffness has four kinds of spring stiffness to follow throughout the force-displacement hysteresis loop. In the plastic state, the spring stiffness  $K_1$  changes to  $K_1$ ' according to the plastic state of spring. In loading and unloading cycles, the spring stiffness changes from  $K_1$ ',  $K_2$ ,  $K_3$ , to  $K_4$ , or from  $K_1$ ',  $K_3$ , to  $K_4$ . In loading, the stiffness slope is toward the point in the hysteresis loop that represents previous maximum displacement and its force. Furthermore, when the relative displacement exceeds the plastic limit,  $\delta_p$ , the stiffness slope decreases, shifting from  $K_2$  to  $K_2$ ' (as in the figure). The hysteresis loop from  $K_1$ ',  $K_2$ ',  $K_3$ , to  $K_4$ , or from  $K_1$ ',  $K_3$ , to  $K_4$  is similarly changed. The behaviors in compression and tension are same (symmetric with respect to the origin). When

the relative displacement is 0, the spring force is 0 too. To sum up, the spring force can be expressed by:

$$\begin{cases} F = K_{1}\delta & \delta_{m} \leq \delta_{e} \\ K_{1}' \delta & \Delta\delta \cdot \operatorname{sgn}(\delta) \geq 0, \ 0 \leq |\delta| \leq \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right|, \ \left| \delta_{e}^{\operatorname{sgn}(\delta)} \right| < \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right| \\ K_{2}\delta & \Delta\delta \cdot \operatorname{sgn}(\delta) \geq 0, \ \left| \delta \right| > \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right| > \left| \delta_{e}^{\operatorname{sgn}(\delta)} \right| \\ \Delta\delta \cdot \operatorname{sgn}(\delta) \geq 0, \ \left| \delta \right| > \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right| > \left| \delta_{e}^{\operatorname{sgn}(\delta)} \right| \\ \Delta\delta \cdot \operatorname{sgn}(\delta) \geq 0, \ \left| \delta \right| > \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right| > \left| \delta_{y}^{\operatorname{sgn}(\delta)} \right| \\ \Delta\delta \cdot \operatorname{sgn}(\delta) < 0, \ \left| \delta \right| \geq \left| \delta_{m}^{\operatorname{sgn}(\delta)} - F_{m}^{\operatorname{sgn}(\delta)} \right| < \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right| < \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right| \\ \Delta\delta \cdot \operatorname{sgn}(\delta) < 0, \ 0 \leq \left| \delta \right| < \left| \delta_{m}^{\operatorname{sgn}(\delta)} - F_{m}^{\operatorname{sgn}(\delta)} \right| / K_{3} \right|, \ \left| \delta_{e}^{\operatorname{sgn}(\delta)} \right| < \left| \delta_{m}^{\operatorname{sgn}(\delta)} \right| \end{cases}$$

where, *F* is spring resistance force,  $\delta$  is relative displacement between floors, *K* is shear wall stiffness,  $\delta_m$  is maximum relative displacement between floors during the proceeding response, and  $\operatorname{sgn}(\delta)$  is signature for the relative displacement. When  $\delta \ge 0$ ,  $\operatorname{sgn}(\delta)=1$ . Otherwise,  $\operatorname{sgn}(\delta)=-1$ .



Figure 7. Displacement and force hysteresis loop

# 4. APPLICATION

#### 4.1 Wooden houses models

The wooden houses investigated here are modeled as two-story timber-framed houses as shown in Figure 8. The construction designs of Japanese wooden houses have been changed according to the construction regulations [17]. Before 1970s, common construction was extremely heavy mud-plastered roofs and shear walls. After that, building weight has been lighter thanks to light china roofs and mortared walls. In the construction community, many efforts to improve shear wall stiffness have been made so far. These models also consider following background. The wooden houses collapsed during the Kobe earthquake are not only old mud-walls houses. Due to the small space to live in the urban area, houses are small and built quite close to the next houses. Therefore, they do not have shear walls enough to meet seismic forces in order to have exits and windows. Even young-built houses have possibilities to be damaged more. Considering these changes of wooden houses, three kinds of wooden houses models are investigated as basic model; model 1 is an older heavy-roof house construction (mud-plastered roofs and walls), and models 2 and 3 are lighter houses with different shear wall stiffness. The models 2 and 3 involve such considerations of houses in urban area.



Figure 8. Wooden house model

Table 1 lists the parameters of springs. Except for the elastic stiffness for shear walls, the same parameters of shear wall spring are used. The Young's modulus of timber members is  $88.2 \times 10^2 (\text{kN/cm}^2)$ . It is assumed that five timber columns support floor and roof weights in the 7.4 m of depth. The relative displacement between floors for elastic limitation is 2.0 cm, which is corresponds to around 1/120 radian of story deformation angle, and 8 cm of the yield displacement is around 1/30 radian of story deformation angle. These parameters are obtained from the laboratory test data [18]. Table 2 lists weight and elastic shear wall stiffness for the three models. The masses are estimated from the weight per floor and wall areas. The weight of model 1 is assumed to be that of mud-roof and wall's house, while the weight of models 2 and 3 is that of houses designed after 1970s. According to the responses

induced by small horizontal loads, their natural periods are 0.53 second for model 1, 0.35 second for model 2, and 0.49 second for model 3. Existing houses with light roofs generally have 0.2 to 0.4 seconds of natural period and model 2 has its period in that range. Model 3 was designed to have longer natural period than existing light-roof wooden buildings but close to model 1 in order to increase its chances of collapse in these simulations.

Joint part	stiffness <i>k</i>	Shear wall	stiffness K
$k_{nc}$ (kN/cm)	3.18x10 <sup>4</sup>	$K_2/K_1$	1/3
$k_{nt}$ (kN/cm)	$3.52 \times 10^2$	$K_{3}/K_{1}$	3
$k_s$ (kN/cm)	1.59x10 <sup>4</sup>	$K_{4}/K_{1}$	0
$f_e^{nc}$ (kN/cm <sup>2</sup> )	0.29	$K'_2/K_1$	-2
$f_e^{nt}$ (kN/cm <sup>2</sup> )	0.082	$\delta_e(cm)$	2.0
$f_e^{sc}$ (kN/cm <sup>2</sup> )	0.29	$\delta_p(cm)$	8.0
		$\delta_u(cm)$	10.0

Table 1: Parameters of springs for house model

	model 1	model 2	model 3
Weights to be estimated			
Roof weight per area (kN/m <sup>2</sup> )	2.16	0.69	0.69
2 <sup>nd</sup> floor weight per area (kN/m <sup>2</sup> )	1.18	1.18	1.18
Outside wall weight per wall area (kN/m <sup>2</sup> )	1.76	0.88	0.88
Inside wall weight per wall area (kN/m <sup>2</sup> )	0.78	0.39	0.39
Shear wall stiffness			
$K_1$ (kN/cm)	39.2	39.2	19.6

Table 2: Weights and shear wall stiffness for three models

(Note: each floor has 54.76 m<sup>2</sup> (7.40m x 7.40m) of area. 50 percent of wall are present.

# 4.2 Input ground motions

With respect to the input ground motion, 9 ground motions are used as shown in Figure 9.

They were observed in the 1995 Kobe earthquake ((a), (b), (c), (d) and (e) in Figure 9) [19], the 1999 Western Tottori earthquake ((g), (h) and (i)) [20], and the 2001 Geiyo earthquake ((f)) [20]. All the earthquakes occurred around Japan. Among them, the 3 ground motions (a), (f) and (g) have peak ground acceleration near 0.8 g, but their predominant periods differ. The ground motion of JMA Kobe (a) is commonly known as high PGA with relatively long predominant period. Its vertical component is also high compared to the others. Its ISI is of  $I_s = 6.4$ . The ground motion recorded at Sakaiminato (g) changes its predominant period subsequently as if it seems to be caused by liquefaction. For the Yuki ground motion (f), Sakai and Fujii [21] reported that damage to general buildings observed at the Yuki site was slight in spite of strong peak ground acceleration of 827gal and the ISI of  $I_s = 5.7$ , because of shorter than 0.5 second predominant period of acceleration. They conclude that actual damage corresponds to the amplitude at 1.0 second in acceleration spectra. For the simulation, one of the horizontal components with higher PGA value and the vertical component are used in the simulation.





# 4.3 Results

As the results of three wooden house models given by the ground motions, the ground motion of JMA Kobe showed significant behaviors of house vibrations. Among cases given by 3 ground motions with high PGA ((a), (f) and (g) in Figure 9), house models 1 and 3 given by the ground motion of JMA Kobe demonstrated the collapse behaviors such that the second floor falls on the ground. Other cases are seen to reach the plastic state of the bracing spring, but still to stand.



(1) Response acceleration (Kobe-model 1)



(2) Relative displacement (Kobe-model 1) (a) Kobe-model 1



(1) Response acceleration (Kobe-model 3)





(1) Response acceleration (Kobe-model 2)



(2) Relative displacement (Kobe-model 2) (b) Kobe-model 2



(1) Response acceleration (Sakai-model 3)



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(2) Relative displacement (Kobe-model 3)	(2
(c) Kobe-model 3	

(2) Relative displacement (Sakai-model 3)(d) Sakaiminato-model3

Figure 10. Response acceleration and relative displacement

Figure 10 shows the response for several cases for examples. For each case, the upper figure shows response acceleration at the second floor (rigid body element) and the lower figure is the horizontal relative displacement at first floor. With respect to the ground motion of JMA Kobe, all three models exceed the elastic limit of the bracing spring (square point plotted in figures) at almost same time. This is 7.7 second of time when 600 gal of ground motion is input. After that, the response frequencies of collapsed models (models 1 and 3) become lower than that of standing model (model 2), and then go to collapse after the share wall spring eliminated (triangle point in Figure 10. Model 2 shows 1,000 gals of response acceleration at several times, but the relative displacement is less than 5 cm for both sides.

Other ground motions that did not induce collapse have higher frequency compared to JMA Kobe. The shear walls in these cases exceeded their elastic limits when the input ground motion is peak. However, as the stiffness of the structure decreases, the higher predominant frequency of ground motion does not match lower frequency of structure and, as a result, it does not collapse. As far as the ground motion of Sakaiminato, the predominant frequency itself shifts from higher to lower after 13 seconds in the figures. The houses respond well with the low frequency of ground motion, while the amplitude of ground motion is smaller. Therefore, such ground motion could not make the house collapse. The ground motion of Yuki has much higher predominant frequency over its duration. At the time of peak ground motion the shear wall stiffness changes to elastic. The response acceleration marks around 1,000 gals in model 2, but the relative displacement is less than 3 cm.

With respect to collapse cases (house models 1 and 3 by the Kobe ground motion, houses collapsed towards the side on which bracing spring becomes plastic first. When the relative displacement increases again and exceeds the spring tension limit at the same side, the house collapses. In that time, the input ground motion is peak at 818 gal, and the response acceleration shows high value and high frequency state. This behavior is thought to be induced by the unsteady state of structure system in a short time. After cutting off of the bracing spring, the rigid body element of the second floor is receiving forces only through the springs of joint parts. Then, when the horizontal relative displacement is over 80 cm, the gravity load is predominant rather than the ground motion and the ground motion is not transferred to above elements. The time to start falling down is the time when the horizontal relative displacement reaches 80 cm and the second story is free from the external force besides gravity load. Figure 11 shows the height of rigid body element representing the second floor. As model 1 reaches 80 cm of relative displacement sooner than model 3 does, it fell down on the ground 2.5 seconds earlier than model 3. Figure 12 shows the collapse process of model 3. The second story does not show collapse because the simulations were stopped before the second story hits the ground.

To conclude for house responses, heavy-roof houses with weak shear walls collapsed. The most damaging ground motion is JMA Kobe, which has relatively low predominant frequency. After inducing plastic response of the cross bracing prior to PGA, the ground motion makes the house collapse at the PGA. In order to cause collapse, firstly the large amplitude of ground motion is necessary for the shear wall stiffness to be well above elastic. Secondly, even though the natural frequency of houses becomes lower in the plastic state, the lower frequency and high-amplitude of ground motion is necessary to match the lower frequency of structure.



Figure 11. Collapse process of second floor (Kobe- models 1 and 3)



Figure 12. Collapse process of house case (Kobe model 3)

# **5. COLLAPSE PROCESS OF HOUSES AND IISI**

### 5. 1 IISI arrival time and elastic limits of houses

This section focuses on the time-domain characteristics of ground motions that induce the collapse process of houses. 9 ground motions are used; three were used above and other six have been selected to increase chances of collapse process (Figure 9). Regarding that model 2 as the basic model, five cases of house models (models 2 to 6) are investigated by changing shear wall stiffness. The natural periods of house model ranges from 0.23 to 0.63 periods. Figure 13 shows the IISI time histories for nine strong ground motions. A "+ "- shaped points in the figures indicates a time when the house exceeds the elastic limit of its

shear wall springs for the first floor, while an "x "-shaped point is mean the time when a house exceeds the plastic limit, resulting in collapse due to elimination of the shear wall stiffness. Among the total of 45 cases (5 house models by 9 ground motions), houses with short natural periods stayed elastic throughout the ground motions, while houses with long natural periods collapsed due to strong ground motions, as in the case of the Kobe earthquake (JMA Kobe (a), Takatori (b), and Motoyama (c)).





Figure 13. IISI time histories and structural limitations

Something common among these cases is that the elastic limit occurs roughly around when the IISI of input ground motion exceeds the level of  $I_{is} = 5.0$ , especially in cases of long period houses. The short natural period houses often exceeded their elastic limit 2 or 3 second after the former houses. Moreover, the plastic limits were exceeded around 2 seconds after the elastic limits. After that time, the IISI time history remains at high seismic intensity, above  $I_{is} = 5.0$ . Such response during high seismic intensity affects the houses greatly.



Figure 14. Arrival time of  $I_{is} = 5.0$  and time to elastic limit

Figure 14 shows the relationship between the arrival time of  $I_{is} = 5.0$  for input ground motion and the time from the beginning of IISI to exceeding the elastic limit of shear wall stiffness. They have a strong correlation, so it can be said that the time exceeding the elastic

limits of houses explain with the arrival time of  $I_{is} = 5.0$ . In particular, the houses with more than 0.35 second of natural period become plastic within 1 second after the arrival time of  $I_{is} = 5.0$ . Furthermore, collapsed houses are observed in the cases of ground motions for which the arrival time of  $I_{is} = 5.0$  is shorter than non-collapsed houses (the differences from the point in Figure 14 mean the time from elastic limit to plastic limit). More specifically, there are increased chances of collapse when the houses with more than 0.35 second of natural period are provoked by ground motion with short arrival time of  $I_{is} = 5.0$ . As the relationship between the hypocentral distance and the arrival time of  $I_{is} = 5.0$  was already known in previous study [12], the short arrival time produced by earthquakes with nearby hypocenter is a significant factor to lead to the collapse process.

With regard to the human behavior, the responding states of buildings passed the threshold of human evacuation (defined as  $I_{is} = 5.0$  in the previous study [12]) only when the ground motion with short arrival time of IISI and the house with long natural period meet. However, when building response does not reach the plastic state, the IISI decreases soon and people can have a chance to escape again. Whether the time available for evacuation becomes longer depends on the ground motion and building setting that induces the plastic limits.

### 5.2 Running spectra and decreasing process of structures' stiffness

Though the elastic limit of shear wall stiffness is clearly related to the IISI, the process from elastic limit to plastic limit cannot be explained simply in terms of IISI. Thus, its process is considered from the view of time-transiting period of buildings. As a ground motion at a site has frequency characteristics, but speaking more precisely, the frequency characteristics are not the same throughout the duration of ground motion, but rather change with time. Likewise, the natural period of a house increases once the plastic state ensues. The collapse process appears when the predominant period of ground motions and the natural period of the house are closely matched for a significant length of time during the earthquake.

The strategy of analyzing these processes is to evaluate the frequency characteristics of the ground motion and house throughout the ground shaking. So, the running spectra of ground motions are adopted. This study considers the Fourier spectra of horizontal acceleration for 2 seconds duration at each second. Because of 2 second duration, it is difficult to pursue the low frequency characteristics. But, it is thought that the range of period satisfies the target period of houses. Figure 15 shows the running spectra of input ground motion. Three lines in the charts show the period of houses for three models: model 2 (natural period 0.35 second), model 4 (natural period 0.23 second), and model 6 (natural period 0.63 second). The houses' periods in the plastic region were evaluated from the hysteresis loop of the first floor's shear wall (the equivalent natural period is estimated based on the slope of hysteresis loop).

From these charts, the houses with short natural periods have low chances to enter plastic region, because most of the ground motions do not have large amplitudes at the short period. Houses having natural period between 0.1 and 0.2 second did not collapse even during the Kobe earthquake. When a house with longer period passes through high-amplitude ground motion, the house's period increases and then collapse occurs. When a house's period is away from the predominant period of ground motion, collapse does not occur even though

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plastic response occurs (for example, Figure 15 (h)). The ground motions inducing collapses (JMA Kobe, Takatori, and Motoyama) have high amplitudes in large regions from 0.2 to 1.0 second. Generally, the waves in short periods are predominant when the hypocenter is nearby, whereas the waves in the long periods are predominant when the hypocenter is far away. However the three ground motions in the Kobe earthquake are predominant in long period region even though they were close to the hypocenter. Such ground motion may be special due to inland earthquake. It is said that during the Kobe earthquake several existing faults subsequently moved following the movement of active fault at earthquake source. Such earthquake mechanism is thought likely to produce so significant, long periods of ground motion.





(i) Yonago

Figure 15. Running spectra and transiting period of houses

# 6. CONCLUDING REMARKS

By using the DEM code, the collapse process of wooden houses is simulated and characteristics of ground motion are discussed in terms of IISI.

- In modeling of wooden houses, following special techniques for rigid body, joint part of timber framing, and shear wall stiffness, were developed. Their efforts produced the collapse process of houses in the simulation. Not only previously investigated code but also the DEM is a beneficial tool to analyze the failure mode of wooden houses.
- It is confirmed that the time exceeding the elastic limits of houses have a strong correlation with the arrival time of  $I_{is} = 5.0$ . In particular, the houses with more than 0.35 second of natural period become plastic within 1 second after the arrival time of  $I_{is} = 5.0$ . The ground motion with short arrival time of IISI is significant ground motion. Referring to the relationship between the hypocentral distance and the arrival time of  $I_{is} = 5.0$ , the significant ground motion is produced by an earthquake with nearby earthquake source.
- When a house's period is away from the predominant period of ground motion, collapse does not occur even though plastic response occurs. The ground motions inducing

collapses have high amplitudes in large regions from 0.2 second to 1.0 second.

- The responding states of buildings passed the threshold of human evacuation only when the ground motion with short arrival time of IISI and building with long natural period meet. However, when building response does not reach the plastic state, the IISI decreases soon and people can have a chance to escape again. Whether the time available for evacuation becomes longer depends on the ground motion and building setting that induces the plastic limits.

Furthermore, the following limitations are indicated:

- The wooden houses investigated here were modeled with size and weight that are popular in urban area, but complicated shear walls and joint parts are the same in all models, and especially many weaker models were used. Further computations are necessary in order to achieve comprehensive results. However, considering shorter natural period of most other houses as well as running spectra of ground motions, such houses are strong enough to resist.

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