J Wood Sci (2010) 56:284–292
DOI 10.1007/s10086-009-1101-x

ORIGINAL ARTICLE

**Takafumi Nakagawa · Masamitsu Ohta
Takahiro Tsuchimoto · Naohito Kawai**

Collapsing process simulations of timber structures under dynamic loading III: numerical simulations of real-size wooden houses

Received: June 26, 2009 / Accepted: December 11, 2009 / Published online: March 23, 2010

Abstract In this study, we developed a new analysis method that enables numerical simulations of the collapse process of real-size wooden houses and evaluated the accuracy thereof by carrying out numerical simulations by shaking table tests. The distinct element method was adopted as the basic theory of our numerical analysis. This research is the first approach in which the extended distinct element method was used for Japanese timber post-and-beam construction. The size of the analysis model is a 5.5 m × 5.5 m, two-story real-size wooden house. The three analytical models were developed in terms of the strength of exterior mortar walls. The simulation results were compared with the shaking table test results. One of the collapsing processes of the numerical simulation corresponds well to the experiment results. Assessment of the possibility of collapse for real-size wooden houses was determined to be possible using our newly developed numerical analysis method.

Key words Numerical analysis · Distinct element method · Shaking table test · Post-and-beam construction

T. Nakagawa (✉)
Department of Building Materials and Components, Building Research Institute, 1 Tachihara, Tsukuba 305-0802, Japan
Tel. +81-29-864-6625; Fax +81-29-864-6772
e-mail: nakagawa@kenken.go.jp

M. Ohta
Graduate School of Agricultural and Life Sciences, The University of Tokyo, Tokyo 113-8657, Japan

T. Tsuchimoto
National Institute for Land and Infrastructure Management, Tsukuba 305-0802, Japan

N. Kawai
Department of Structural Engineering, Building Research Institute, Tsukuba 305-0802, Japan

Part of this report was presented at the 10th World Conference on Timber Engineering, Miyazaki, Japan, July 2008

Introduction

To prevent casualties during a large earthquake, preliminary safety assessment of residential houses is one of the most important measures. During the Great Hanshin–Awaji Earthquake (also referred to as the 1995 Kobe Earthquake), wooden houses without sufficient seismic capacity were heavily damaged, and studies on the seismic performance of wooden houses have been actively conducted. To investigate the safety of wooden houses during large earthquakes, it is important to assess the limit status and understand the possibility of collapse. Although the shaking table test is the most effective solution for this issue, a large cost is required, and experiments on the variety of specifications of wooden houses may be impossible.

A numerical analysis by computer simulation is an effective way to assess the seismic performance of structures as an alternative to shaking table tests, and professionals use such analysis methods for structural design in practice. In advanced structural engineering research fields, computer simulations have been developed to solve questions about unknown physical phenomena or unknown structural systems. However, it is difficult to simulate the collapsing process by the commonly used calculating software, because the collapsing behavior includes large deformation of structural elements and consideration of material nonlinearity, and geometric nonlinearity is needed for numerical calculations. Furthermore, the safety limit of wooden houses is defined by the Japanese building code and recent research,[1] but the actual collapse of conventional Japanese wooden houses occurs in a very large deformation region, from 1/10 to 1/3 rad story drift.[2,3] Consideration of very strong nonlinearity is needed to pursue the collapsing mechanism by numerical analysis. There are few analysis methods that are able to calculate this strong nonlinearity, including collapse behavior in the structural engineering field.[4]

In our previous study,[5,6] we developed a new simulation method based on the extended distinct element method (EDEM). We performed trial simulations for some timber structures, and our calculating method turned out to be useful for collapsing process analysis of wooden houses. On the other hand, the models in our previous study were simple shear walls, so investigation of analysis for real-size house models was needed.

In this article, we have improved our previous calculating program for the three-dimensional frame analyses. It is important in the collapsing process simulation to consider the P-Δ effect by structure weight and the twisting movement caused by the eccentricity of configuration of the shear walls. Our analysis automatically incorporates the P-Δ effect and twist movement. To verify our numerical analysis, we carried out collapsing process simulations for real-size wooden houses that were used in real shaking table tests.

Theory

Extended distinct element method

In this analysis, we used the extended distinct element method (EDEM) as a basic theory. A detailed explanation of EDEM was given in our previous papers.[5,6] EDEM is a noncontinuum analyzing method, so large deformation analysis of the fracture developing processes is possible. Furthermore, EDEM has advantages in terms of calculation cost, as it requires the calculation of equilibrium force equations with adjacent elements only. Through this advantage, the calculation for analysis models with several tens of thousands of degrees of freedom can be carried out relatively easily by ordinary personal computers.

Figure 1 shows the methodology of our numerical simulation. First, the earthquake ground motion is input to the bottom of the analytical model. Second, the reaction forces of each spring are calculated by solving the equilibrium force equations. Finally, the reaction forces are summed up for each element, and the acceleration, velocity, and displacement of each element of the analysis models are calculated at the same time in each step.

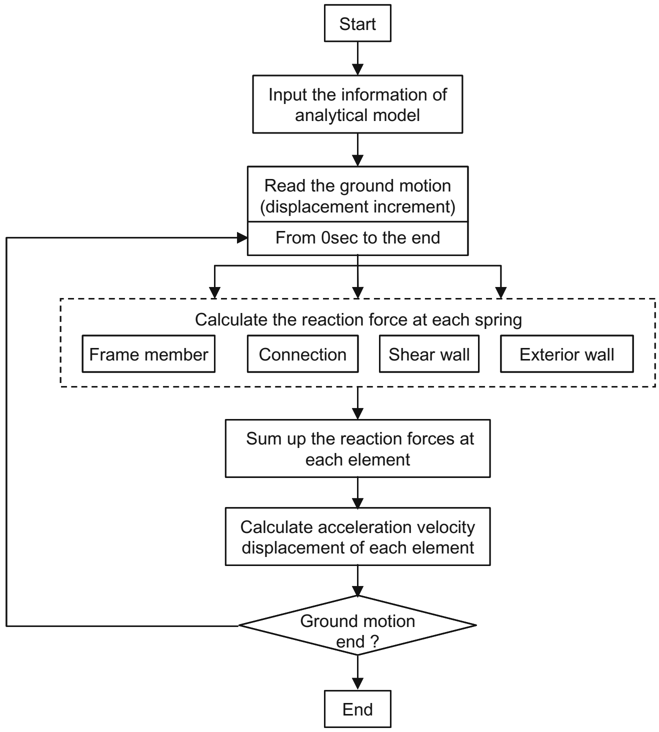


Fig. 1. Flow of the numerical analysis by the extended distinct element method

Modeling of frame members

The skeleton structure of the analysis model consists of frame members. The parameters of the frame members were defined by the characteristic values of wood used in the analysis target. In the following analytical model, the modulus of elasticity of the frame members was uniformly defined as 6.5 kN/mm^2, and the bending strength was defined as 40 N/mm^2. The maximum bending moment was defined by the modulus of section of each frame member, respectively.

Modeling of connections between frame members

As shown in Fig. 2, the frame members (columns, beams, sills, and braces) of the analysis model consist of beam elements connected in a series. The joints between frames

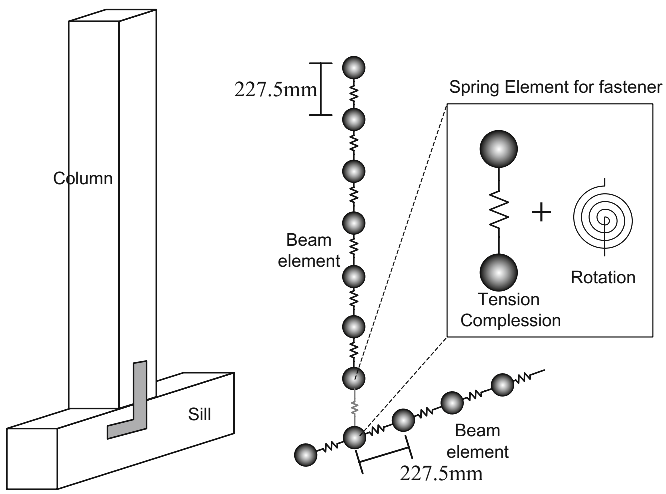


Fig. 2. Configuration of joint spring between frame members

were modeled by two kinds of springs. One is the spring that acts against the tensile and compressive forces, and the other is the spring that acts against rotational movement. The load–displacement relationships of these nonlinear springs were determined by other work[7] (Fig. 3).

Modeling of exterior walls

Exterior walls such as the mortar wall were not definitively estimated as a seismic element of the wooden houses by the Japanese building code, but the contribution of the mortar wall to the seismic performance is not negligible,[8] if the wall is adequately made. A conventional Japanese mortar wall consists of columns, beam, metal lath, and the mortar plastered over them. In this method, we modeled the mortar wall as shown in Fig. 4. The exterior mortar wall was modeled by frame elements and mortar elements. The frame elements correspond to wood lath (the sawn lumbers used for backing of mortar plastering), defined as the wood lath element here. The wood lath elements consist of beam elements connected in a series, and were connected to column elements by nail springs. The mortar elements consist of triangular solid elements and were connected to wood lath elements by staple nail springs. Figure 5 shows the stress–strain relationship of the mortar element. The mortar elements were set to disappear when the tensile stress in the triangular solid elements exceeded fracture conditions to represent the crack developing process. By detailed modeling of the exterior mortar wall, the crack expanding behavior and the size effect on the shear capacity of the mortar walls were taken into account automatically during calculation.

The staple nail springs between wood lath and frame elements were nonlinear shear springs. Their load–displacement relationships were determined by the shear tests of the walls corresponding to this target specimen.[9] Figure 6 shows the load–displacement curves of the shear tests compared with numerical simulation under the same conditions. Because the strength of the staple nails greatly affects the seismic performance of the overall exterior mortar wall, the shear strength of the wall in the analysis model was adjusted to the results of the shear tests by changing the parameters of the staple nail elements. Three different load–displacement relationships (A1–A3) were used in the numerical modeling of the exterior wall. The representative characteristic values of shear walls in each analytical model are listed in Table 1.

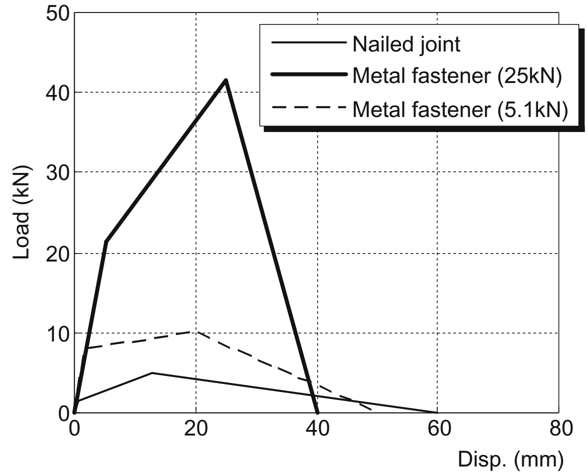


Fig. 3. Load–displacement relationships of joint springs used in the analytical model. The value of the metal fasteners means the short-term allowable tensile load regulated by Ministry of Construction notification no. 1460 in 2000

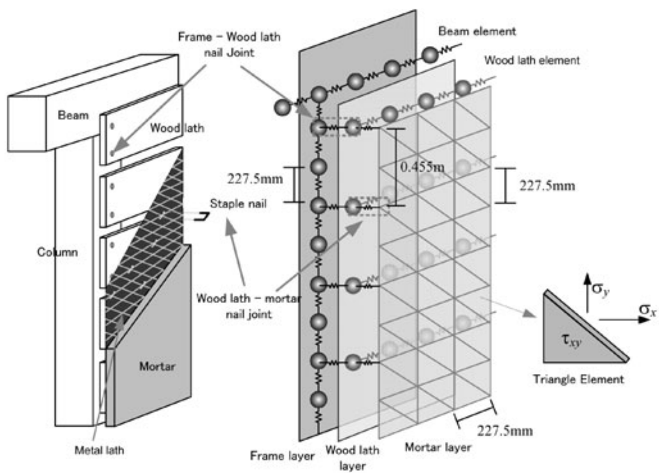


Fig. 4. Configuration of mortar wall elements of the analytical models

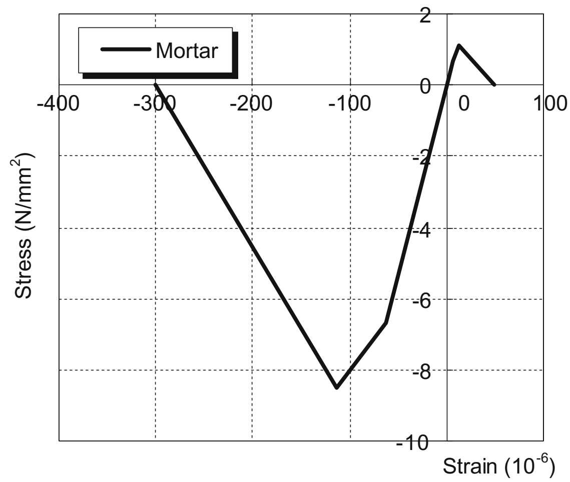


Fig. 5. Stress–strain relationships of mortar elements of the analytical models

Table 1. Characteristic values of representative aseismic elements

Aseismic element	Initial stiffness (kN/mm)	Maximum resistance (kN)	Displacement at maximum resistance load (mm)
Exterior mortar wall (A1)	0.50	12.6	52.5
Exterior mortar wall (A2)	1.00	18.4	45.0
Exterior mortar wall (A3)	1.30	26.3	47.5
Mud wall[a]	0.27	10.6	53.0

Dimension of all walls: 1820 mm wide and 2730 mm high
[a] Mud wall thickness is 70 mm

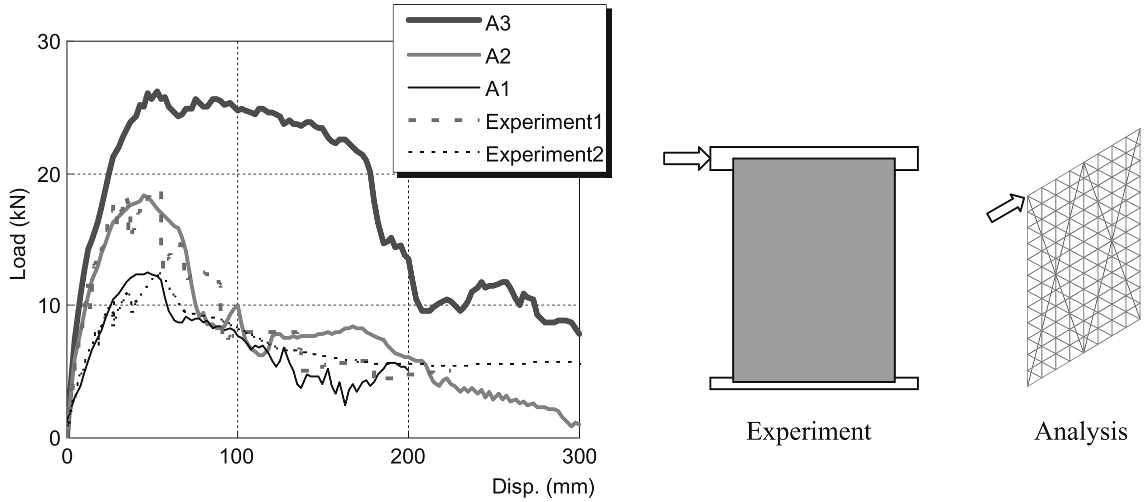
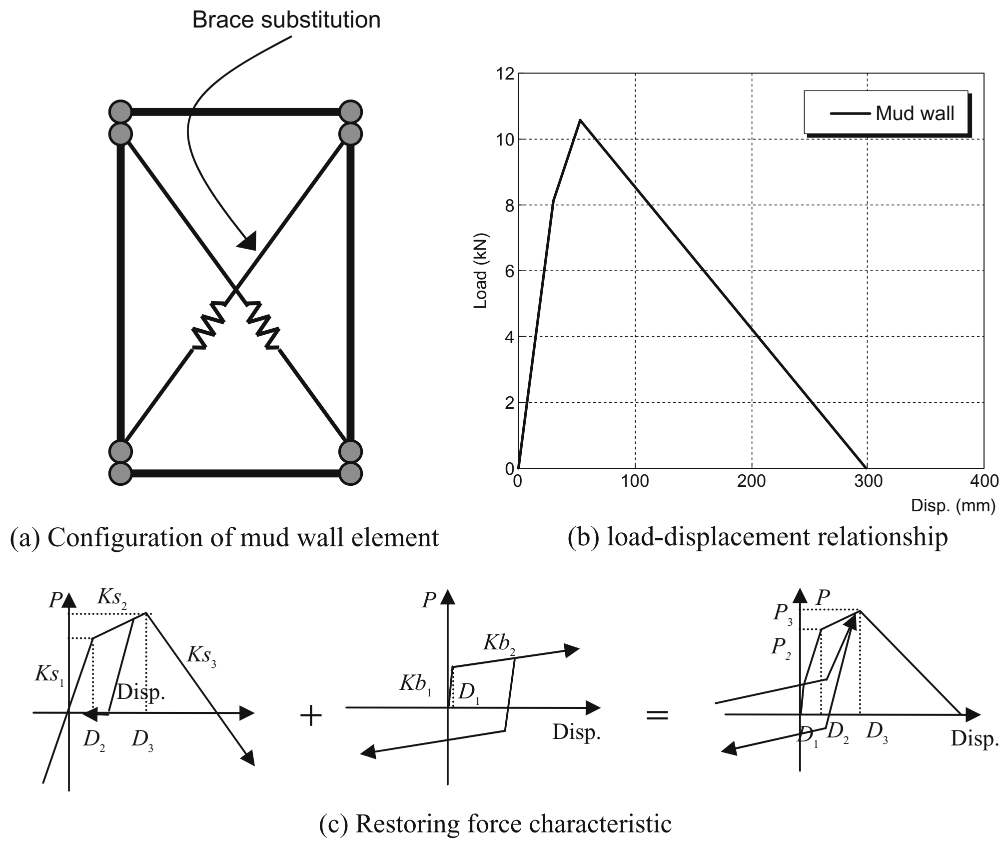


Fig. 6. Load–displacement relationships of the mortar wall elements of the three analytical models. *A1*, *A2*, *A3,* exterior mortar walls defined in Table 2

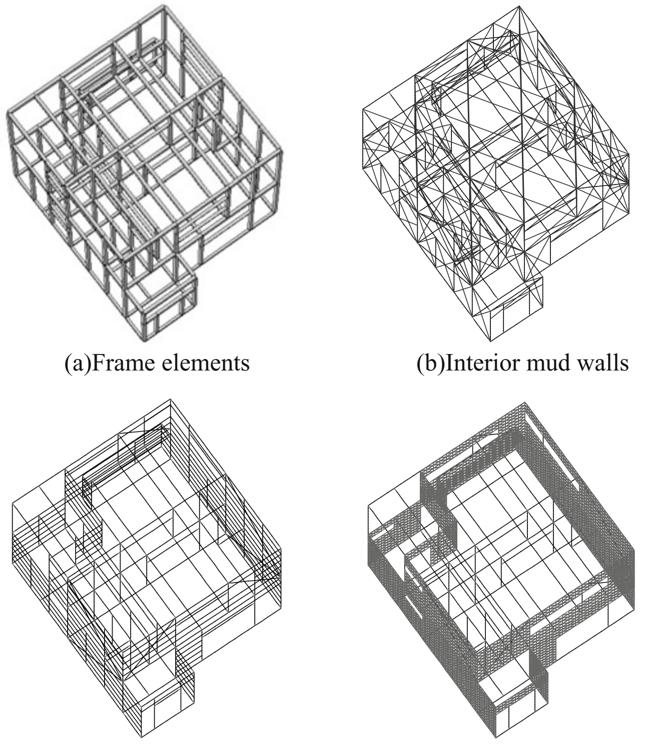
Fig. 7. Configuration and the load–displacement relationships of the mud wall element. The load–displacement relationship indicates the skeleton curves of the 1820-m-long mud wall



Modeling of mud walls and floors

Figure 7 shows the configuration and the load–displacement relationships of the mud wall. The mud walls were replaced by the two diagonal springs (brace substitution) and were modeled as truss elements. The load–displacement relationship of the truss elements of the mud walls acting as the seismic elements were defined empirically by comparison between the shear wall experiments[9] and the calculation of the equivalent analytical models. Figure 7c shows the restoring force characteristic of the truss elements. The restoring force characteristic model was defined by the bilinear and slip curves.

The floors were also modeled by the truss elements, and the truss elements of the floor were assumed to be rigid.



(a)Frame elements (b)Interior mud walls

(c)Wood laths (d)Exterior mortar walls

Fig. 8. Configuration of the analytical models

Numerical analysis

Analysis model

The analysis models used here are depicted in Fig. 8. The size of the analysis model is 5.5 m × 5.5 m. This model is a two-story, real-size wooden house. It has three living rooms and one dining and kitchen room. The total number of nodes for analysis models is 4604, and the number of degrees of freedom is 27624. The plan of this analysis model was based on the specimen used in the shaking table test. Figure 9 shows the floor plan and the elevation plan of the tested specimen and analytical model. This specimen is a conventional post-and-beam wooden two-story Japanese house that was built in 1974 in a Japanese city and transported to be used as the shaking table test specimen. The shaking table test for this house was conducted at E-Defence (a full-scale shaking table in Miki City, Hyogo Prefecture, Japan) under the project "A Special Research Project for Earthquake Disaster Mitigation in Urban Areas" in December 2005.[10] The grade defined by the guideline for seismic diagnosis and retrofitting in Japan[11] is 0.5. This value means the rate to the demand level at the Japanese building standard, so the risk of collapse is identified as "high" according to the guidelines.

The parameters of the connections were defined according to the specifications of the target specimen, and the corresponding load–displacement relationships in Fig. 3 were selected for each connection, respectively. The cross section of frame elements and the joint specification of the analytical model are shown in Table 2. The mass of the analysis model was equalized with the actual weight of the target specimen measured by the load cell of the crane before the shaking table test. The weight of the first floor was 119.1 kN, and the weight of the second floor was 91.4 kN. The weight was distributed to the element of each floor in the analysis model.

Table 2. Basic information of analytical model

Cross section	
Column	100 × 100 mm
Sill	85 × 85 mm
Beam	150 × 150 mm
Joints	
Column-sill	Nailed joint (N75 type nail)
Beam-beam	Metal fastener (5.1kN)
Staple nail	1210F type nail

The type of nailed joint corresponds to JIS A 5508 (1992)
The type of staple nail corresponds to JIS A 5556 (1993)

All lateral elements against the horizontal forces in the shaking table specimen were modeled in the analytical model. The roof structure was not modeled but was accounted for as seismic weight. The damping coefficients were defined to be 5.0%. The method of determination for damping coefficients of large deformation analysis such as our model requires ample studies, but we temporarily defined it by the experimental results at the vibration test[12] of the target specimen. We used the instantaneous stiffness proportional damping. The damping coefficients were assumed to be zero, when the instantaneous stiffness became negative as a matter of analytical convenience.

Ground motion

Two earthquake waves were used as the input ground motion in the numerical simulations. One is the wave recorded at East Japan Railway Company's Takatori station in the 1995 Kobe Earthquake and is called "JR Takatori." The seismic intensity of JR Takatori is a 7 on the Japanese scale. The other wave is an artificial earthquake wave,[13] called "BCJ level 2." The latter wave is used for structural design in terms of the Japanese building code. JR Takatori was input to the frames at the bottom of the first floor by changing the X–Y–Z displacement thereof. BCJ level 2 was input on the *x*-axis of the numerical element. The earthquake ground motion was applied as the disturbance input of the displacement.

Simulation results

Figure 10 shows the appearance of the analysis model after BCJ level 2 input. The thick mesh line shows the surviving mortar elements of three different performance mortal walls. The three analytical models did not collapse, but many cracks occurred in the mortar, and almost all

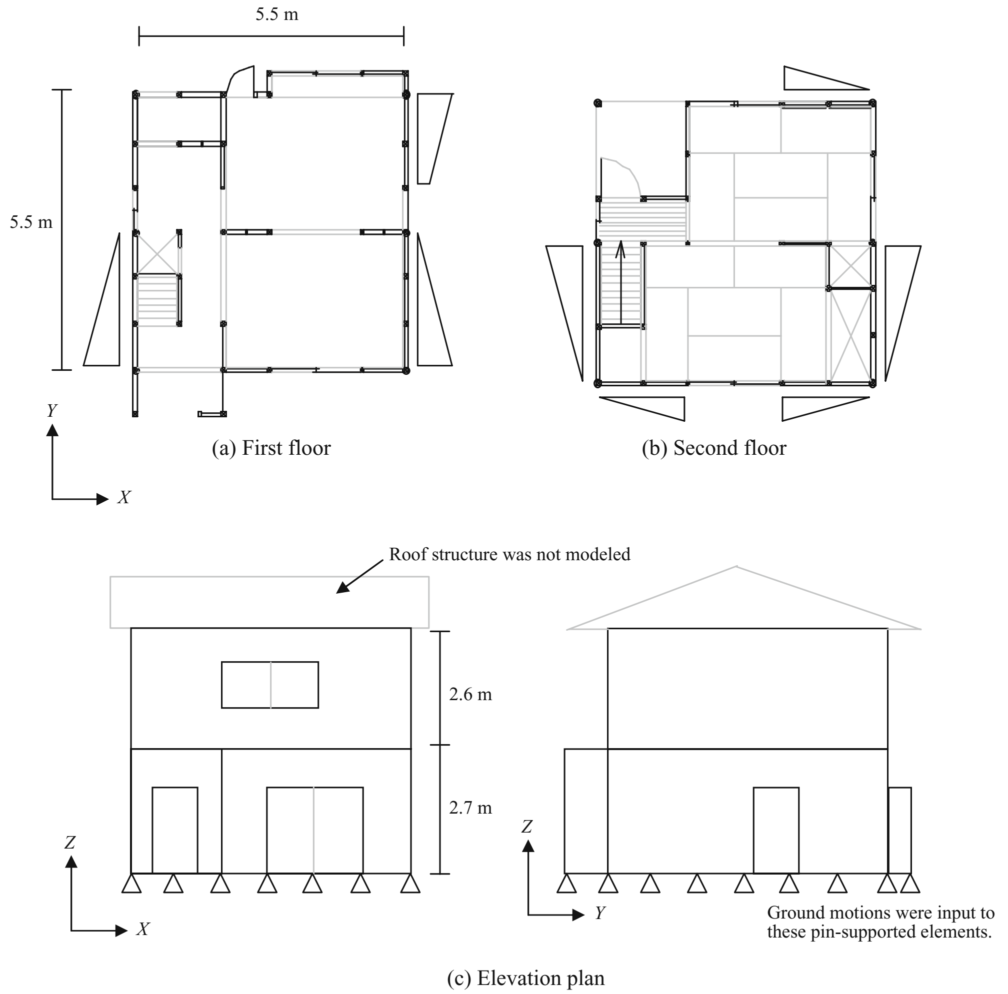


Fig. 9. Floor plan and elevation plan of the analytical model. Triangular marks indicate bracing shear walls

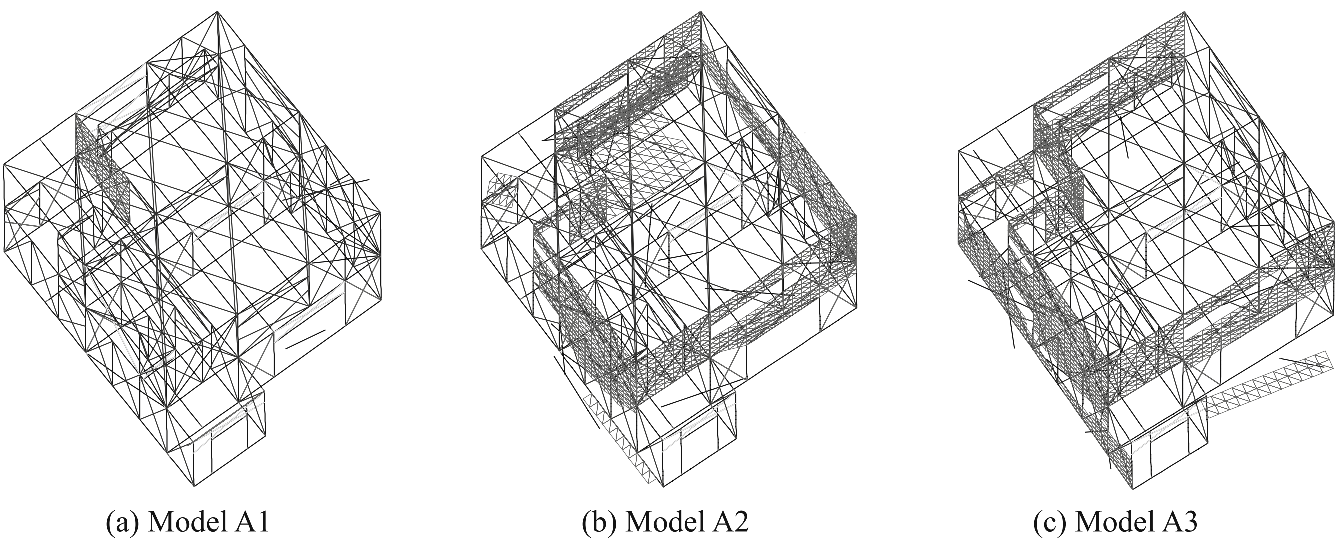


Fig. 10. Appearance of the damage of the analytical models after BCJ level 2 input. *Thick lines* show surviving mortar walls

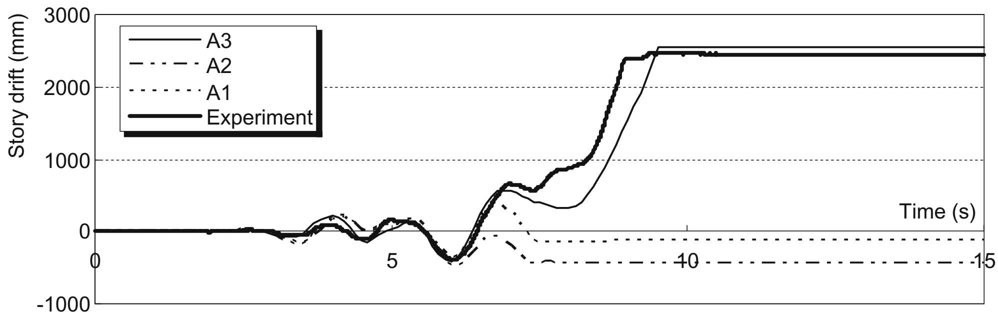


Fig. 11. First-story drift of the three analytical models and experimental results. *A1*, *A2*, and *A3* indicate the model houses equipped with the mortar walls indexed in Table 2

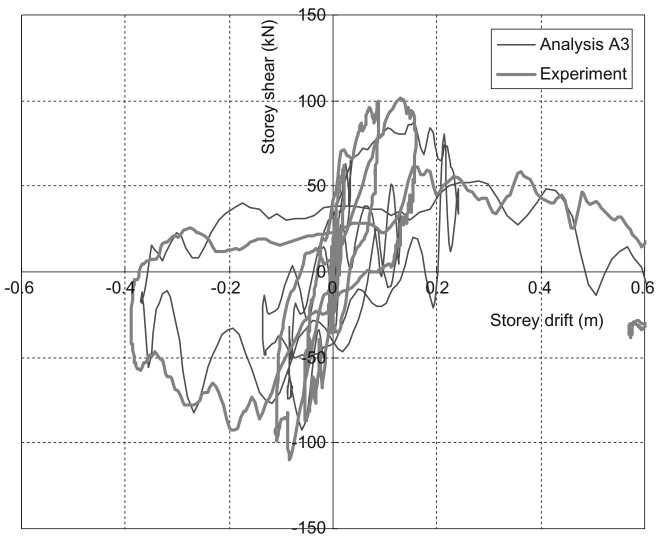


Fig. 12. Relationship between storey shear and first-story drift of analytical models and experiment

the mortar elements fell down in model A1. The shear deformation of the most internal mud wall exceeded 1/50 rad. The restoring force of the model greatly decreased compared to the virgin condition, so this result suggests the occurrence of more heavy damage by the next aftershock. Because the seismic capacity of this model is about the half the level demanded by the Japanese building code, the simulation damage results are appropriate.

The shaking table specimen and all the analysis models collapsed in the JR Takatori input. Figure 11 shows a time series of the first-story drift, defined as movement of the second-floor gravity center, of the analytical model houses with three different performance mortar walls (A1, A2, and A3) and the experimental result of a real house by JR Takatori input. The direction of the fracture differs among analysis models. The collapsing process of the analytical results of model A3 corresponds well to the experimental result. The collapsing process of model A1 and model A2 is similar to the experimental results in the small deformation region (story drift, <500 mm), but the directions of collapse differed.

Figure 12 shows the load–displacement relationships of the experiment and analysis (A3). The maximum story shear of the experiment was larger than that of the analysis. The total shape of the load–displacement relationship of the analysis was similar to the experimental result in the collapsing process.

By these numerical results, the load–displacement relationships between the mortar walls were considered to be similar to those of model A3. It is important to estimate the restoring force of the exterior mortar wall, which is neglected in the building code as shear wall, to predict the detailed fracturing pattern via the difference of the fracture pattern among the analytical models.

Figure 13 shows the results of the numerical simulation of model A3. The cracks occurred at the mortar wall at stage 1, and the mortar walls of the first story started to fall down at stage 2. At stage 3, story drift exceeded 1/3 rad, and the descent of the second-floor level became larger by the P-Δ effect. Finally, the first story collapsed, and the second story fell down at steps 4–5. Figure 14 shows a video still of the shaking table test. Similar fracturing patterns were observed in the shaking table test.

We also investigated the detailed fracture process of mortar wall in the JR Takatori input. Figure 15 shows an example of the stress distribution of the analysis model. Crack-developing behavior is observed around the window openings in this model.

Conclusion

Collapsing process simulations for real-size wooden houses were carried out using our newly developed calculation program based on the extended distinct element method. The collapsing process results from one of the analyses correspond well to the experimental results. We determined that the collapsing process simulation, including the fracturing behavior of the exterior wall, was possible by our analysis method.

Acknowledgments This work was supported in part by a Grant-in-Aid for Scientific Research from the Japan Society for the Promotion of Science, Japan (No.B19780140 and No.B16380114). The shaking table tests in this research are part of the research project "A Special Project for Earthquake Disaster Mitigation in Urban Areas," which is financially supported by the Ministry of Education, Culture, Sports, Science and Technology. The authors are grateful to the members of the research committee for wooden structures for their great efforts and valuable discussions about conducting the experiments.

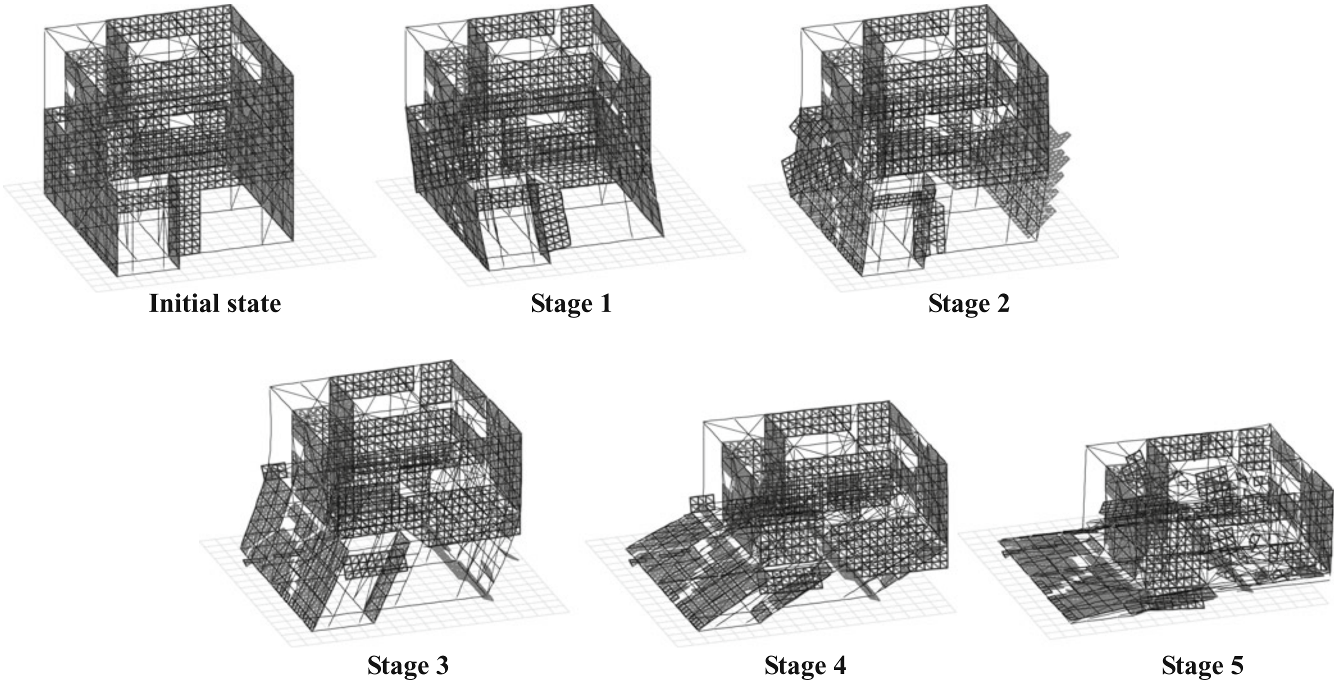


Fig. 13. Simulation results of analytical model A3 during JR Takatori excitation. *Thick lines* show mortar walls.

Fig. 14. Collapsing process of specimen in the shaking table test (JR Takatori)

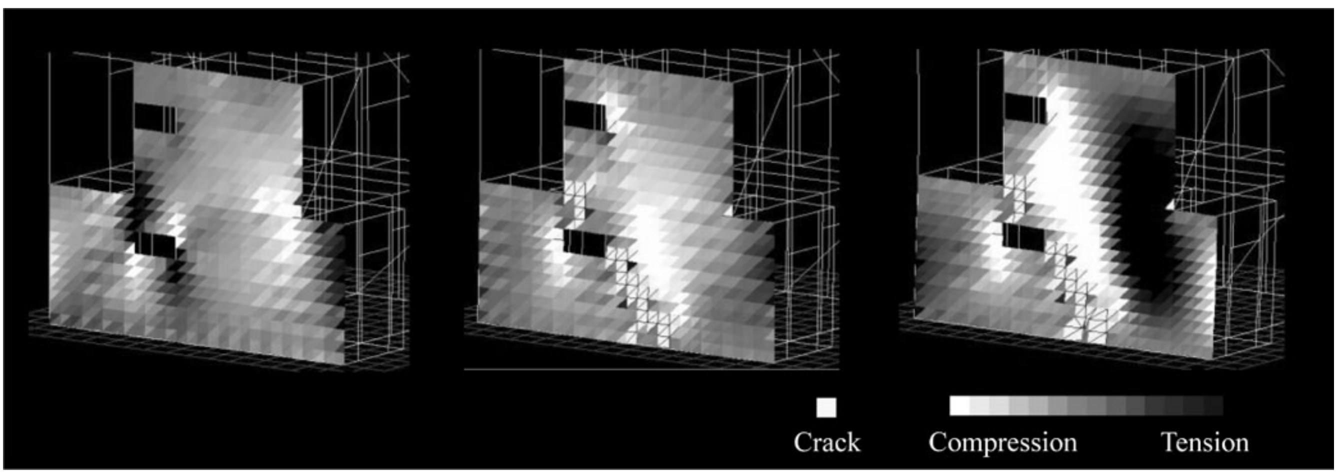


Fig. 15. Change of stress distribution of the mortar wall model during calculating process under the JR Takatori excitation

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