

Analytical and Experimental Study on Structural Behavior of Traditional Wooden Frame Including *Kumimono*

Atsuo Takino

Nara Women's University, Japan

Atsushi Kunugi

JSOL Corporation, Japan

Yuji Miyamoto

Osaka University, Japan

Yasuhiko Tashiro

Nikkenn Sekkei Ltd, Japan

Yoshiyuki Suzuki

Ritsumeikan University, Japan



SUMMARY

Traditional wooden architectural structures in Japan constructed with columns, board walls, beams and *Kumimono* are generally considered to have superior deformability. However, very little study has been performed on the behavior of traditional wooden architectures in earthquakes. In this study, we attempted the application of 3D finite element analysis for analytical determination of the buckling behavior of existing *Kumimono*-bearing columns and we conducted the in-plane shear tests of frames including column, board wall, beam and *Kumimono*.

The analytical results showed generally close agreement with the trends found experimentally for the end-stage deformation even if the isotropic material is used for FEM analysis, so application of the analytical method to the *Kumimono*-bearing column is proved to be valid. In in-plane shear tests, frames including column, board wall, beam and *Kumimono* showed a stable load versus story drift angle response.

Keywords: Traditional Wooden Structure, Finite Element Analysis, Structural behavior

1. INTRODUCTION

Traditional wooden architectural structures in Japan constructed with columns, board walls, beams (including *Nuki*) and *Kumimono* (eave-supporting assemblies, which consist of bearing blocks (*Masu*), beams (*Keta*), tail rafters (*Odaruki*), and other components), are generally considered to have superior deformability and damping properties (see Fig.1, Fig.2). Very little study has been performed, however, on the behavior of traditional wooden architectural structures including *Kumimono* in earthquakes, and so elucidation of the mechanisms involved in their dynamic behavior remains incomplete. A complicating factor is the wide variation in the configurations of traditional wooden architectures, depending on the regionality and the techniques of the carpenters. General surveys and experiments representing the various cases are therefore difficult, as they must be adapted for particular cases.



Fig.1 Traditional wooden architectural structure.



Fig.2 *Kumimono*.

In the present study, we attempted the application of 3D finite element analysis for analytical determination of the buckling behavior of existing *Kumimono*-bearing columns, together with verification of the analytical method by applying it to simulations of prior buckling experiments. In addition, we conducted the in-plane shear tests of frames including column, board wall, beam and *Kumimono* in order to estimate the seismic performance.

2. ANALYSIS OF BUCKLING IN COLUMN WITH CUTOUT

2.1 Overview

To assess the validity of the analytical method, we applied it to simulations of prior experiments on columns with cutouts for *Kumimono* installation by using LS-DYNA, a general-purpose finite element analysis program. The basic test piece configurations (A and B) are described in Table 1, and the analytical models are shown schematically in Fig.3.

2.2 Analytical model

For the simulation, a rigid body constraining the translation of the column top in the x- and y-directions was modeled and interlocked with the column by coupling it to its top face (Fig.4), and a rigid body constraining the translation of the column bottom in the x-, y-, and z-directions was similarly modeled and coupled to its bottom face, to reproduce the simple support of both ends of the column used in the experiment.

Test piece A was the column model without the *Kumimono*. Test piece B was the model with the *Kumimono* installed as an integral part of the column. Separation between the components within the *Kumimono* was not considered. The slippage and the separation between the column and the *Kumimono* at their interface was included by setting the conditions of contact between them. The coefficient of friction between the column and the *Kumimono* was set as 0.49. The loading condition was assumed to result in a 15 mm forced vertical displacement of the rigid-body center of gravity (CG) at the top of the column.

The material constants of the *Zelkova serrata* (*zelkova*) wood used in the analysis are shown in Table

Table 1 Basic description of test pieces.

Test piece	Description	Notes
A	Column with semicircular cutout	Column reproduced
B	Column with semicircular cutout bearing <i>Kumimono</i>	<i>Kumimono</i> reproduced and added to column

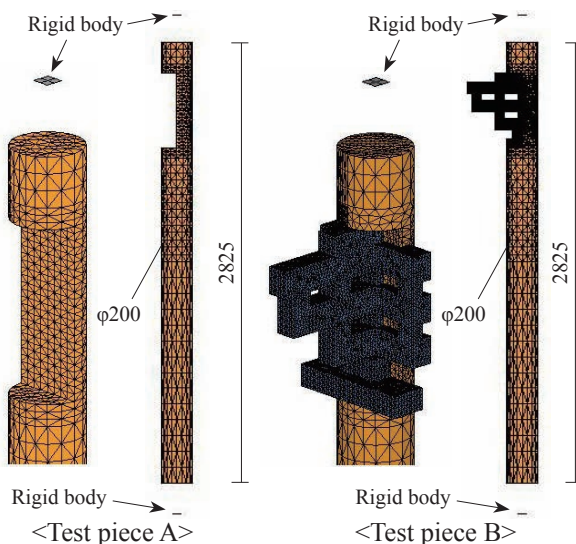


Fig.3 Analytical models.

Table 2 Material constant.

		<i>Zelkova</i>	
		Column	<i>Kumimono</i>
Element		Isotropic elasto-plasticity	
Density (ton/mm ³)		0.7×10 ⁻⁹	
Young's modulus (N/mm ²)		10500	1575
Poisson's ratio		0.4	
Yield strength (N/mm ²)	case1	21	4.2
	case2	40	8
Post-yield plastic hardening modulus (N/mm ²)		210	31.5

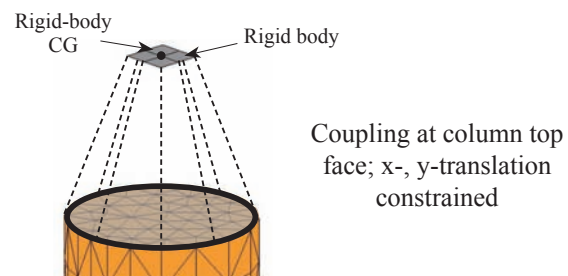


Fig.4 Rigid-body coupling.

2. In the model, *zelkova* was assumed to be an isotropic elasto-plastic material. The density, Young's modulus, and Poisson's ratio were taken from the Wood Industry Handbook. In Case 1, the yield strength was the standard value given in the Standard for Structural Design of Timber Structures by Architectural Institute of Japan. In Cases 2 and 3, the analysis was performed for test piece B with the yield strength value as a parameter. The values used for the column were in the fiber direction (axial), and those used for the *Kumimono* were normal to the fiber direction (transverse).

2.3 Result

Figures 5 and 6 show the relation between the vertical displacement of the rigid-body CG and the load, and between the horizontal displacement (see Fig.7) of the column in the *Kumimono* region and the load, respectively. In the experiments, the strength of the test pieces decreased sharply after buckling due to tensile fracturing, as shown in Fig.8. In the analysis, however, the post-yield plastic deformation proceeded largely without tensile fracturing because the tensile side was modeled as a completely elasto-plastic material. The post-yield behavior shown in the analysis diverged substantially from that found in the experiments and was therefore deemed unreliable and excluded from consideration. The rigidity values obtained in the analysis were generally higher than those found in the experiments, both with and without the *Kumimono* in place. In the experiments, moreover, the horizontal

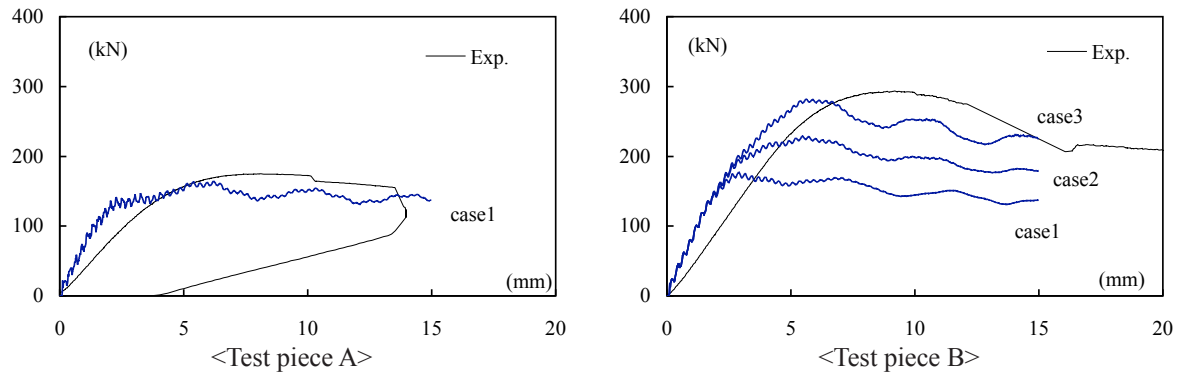


Fig.5 Load-vertical displacement.

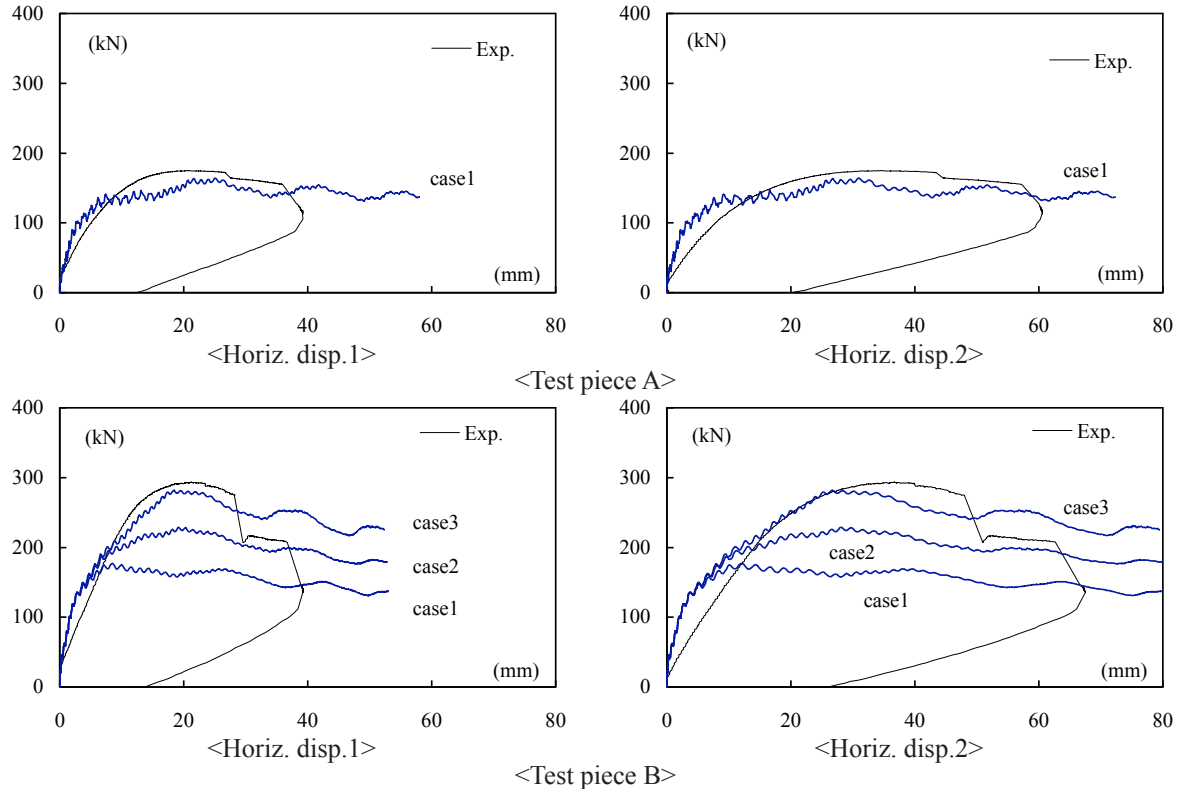


Fig.6 Load-horizontal displacement.

displacement was found to differ somewhat from the measurement position, but this tendency was not clearly observed in the analysis.

Overall, the analytical results showed generally close agreement with the trends found experimentally for the end-stage deformation. (see Fig.9.) In regard to column strength, in Case 1 the analysis showed substantially lower yield strength than the experiments presumably because of the use of the standard values for yield strength taken from the Standard for Structural Design of Timber Structures. In Case 3, in contrast, in which the assumed values for the yield strength were close to the actual compressive strength, the analytical results closely matched the experimental values.

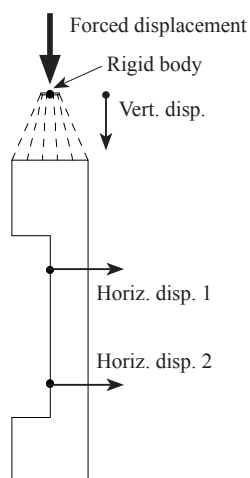


Fig.7 Analytical conditions.

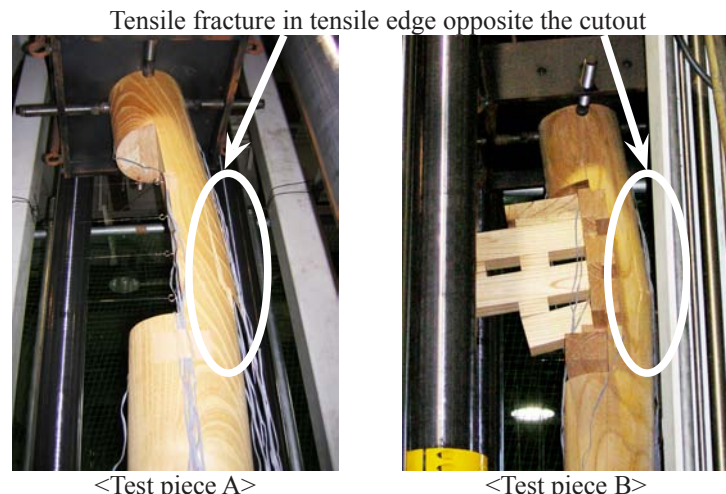


Fig.8 Fracture configuration.

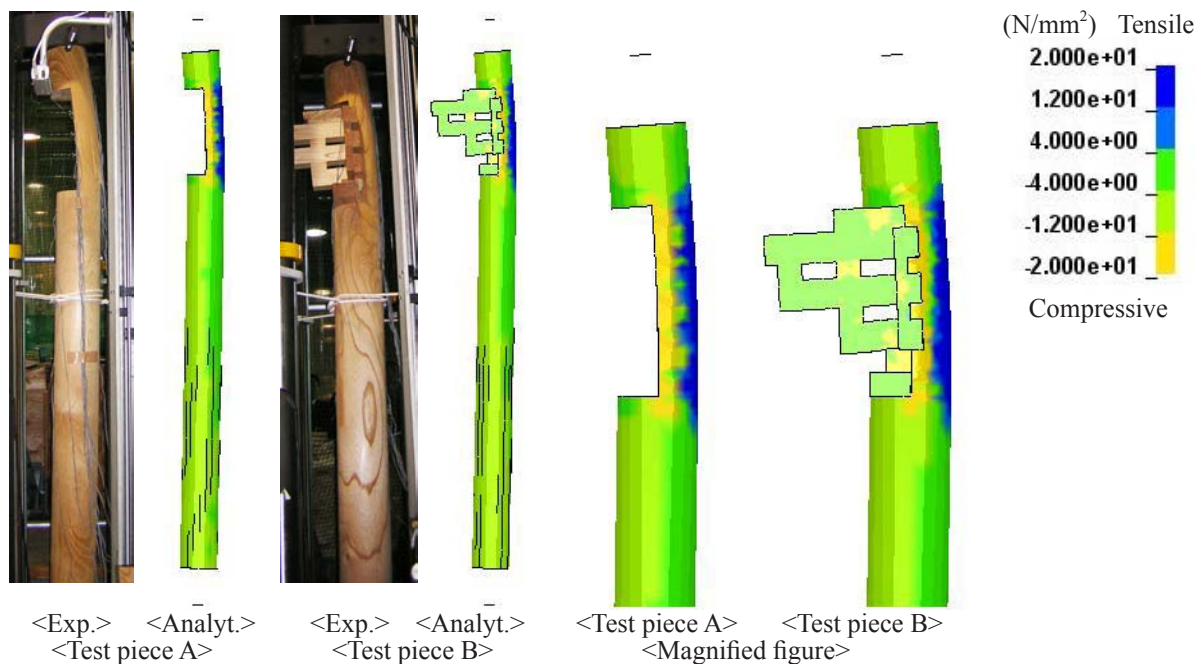


Fig.9 Nature of the deformation.

3. ANALYSIS OF KUMIMONO-BEARING COLUMN

3.1 Overview

The analysis was performed for a column bearing the *Kumimono*, including reproduction of the *Masu* (bearing blocks), *Doiketa* (main beam), *Doiban* (main-beam footplate), *Dabo* (dowels), *Keta* (outer beams), *Odaruiki* (tail rafters), and *Ebikoryo* (rainbow tie-beam). Figures 10 and 11 show an elevation view of the column installation and Fig.12 shows the analysis model. The *Kumimono* is shown in detail in Figures 13 and 14. Each element of the bearing blocks and outer beams in the *Kumimono* was

individually modeled, their contact conditions were assigned, and the slippage and the separation were considered for all component material interfaces. The column was assumed to stand on a rigid body above a foundation stone, and consideration was given to the column contact with the rigid body. The coefficient of friction was assumed to be 0.5 between the column base and the rigid body and 0.49 between the bearing blocks, beams, and other components.

The column loading conditions are shown in Fig.15. The roof load first assigned as the vertical load on the column was the load exerted by the *Ebikoryo* at its coupling with the column (see Fig.16), and the load then assigned was the horizontal load on the region of the coupling. In the analysis model, it was assumed that the roof load was transmitted to the column via the *Doiketa* mounted on the *Doiban* in the top tier of the model, and rigid bodies 1 and 2 were coupled to the *Doiban* at its points of contact with the *Doiketa*. The load of the upper structure was thus exerted simultaneously by the two rigid-body centers of gravity, and was set at 400 kN at rigid body 1 and 100 kN at rigid body 2. The horizontal load on the column was assumed to be 100 kN, exerted at the CG of rigid body 3 coupled to the side of the column.

Table 3 shows the material constants used in the analysis. The column and the *Kumimono* were assumed to be composed of *zelkova*, which in the material model was assumed to be an isotropic elasto-plastic material. The dowels were *apitong* wood, which is higher in rigidity than *zelkova* and assumed to be isotropic elastic bodies in the material model. The density, Young's modulus, and Poisson's ratio of the materials were taken from the Wood Industry Handbook. The yield strength of *zelkova*, given in Case 3 described in Section 2.2, was close to the actual strength. The values used for the column were the axial values relative to the fiber direction, and those used for the *Kumimono* were the transverse values.

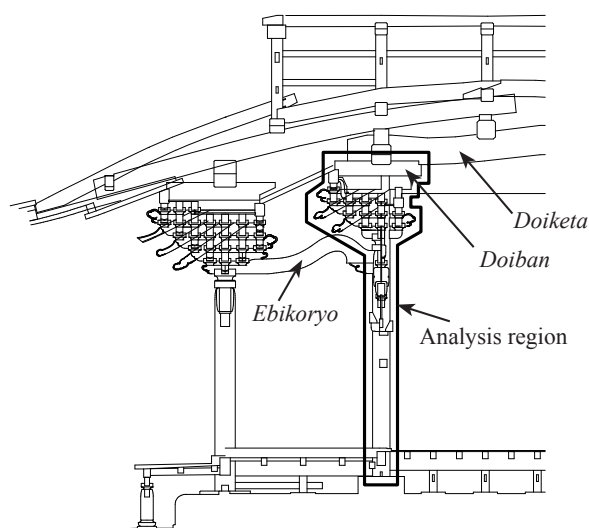


Fig.10 Column structure. (Elevation).



Fig.11 Column structure.

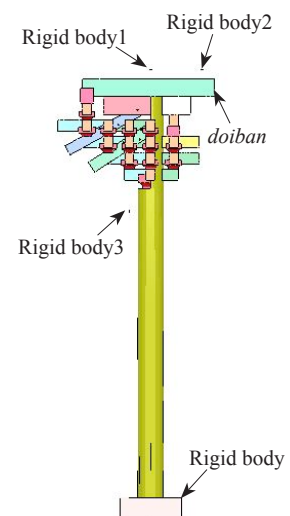


Fig.12 Analysis model.

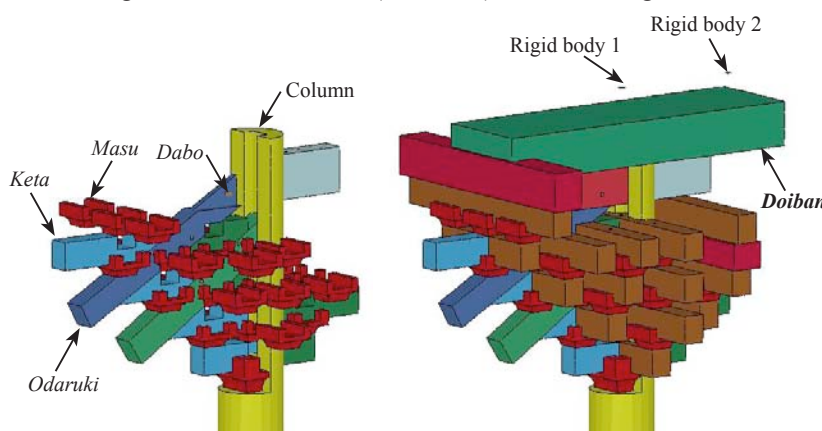


Fig.13 Kumimono details.

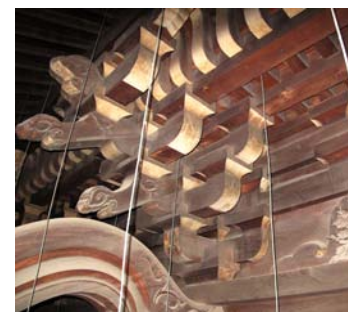


Fig.14 Photo of Kumimono.

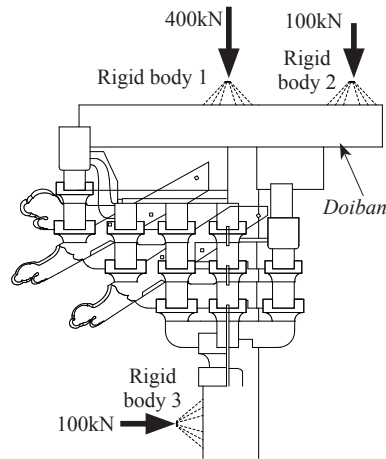


Fig.15 Loading conditions.



Fig.16 Ebikoryo.

Table 3. Material constants.

	<i>Zelkova</i>		<i>Apitong</i>
	Column	<i>Kumimono</i>	<i>Dabo</i>
Material model	Isotropic elasto-plasticity		Isotropic elastic
Density (ton/mm ³)	0.7x10 ⁻⁹		0.64x10 ⁻⁹
Young's modulus (N/mm ²)	10500	1575	20000
Poisson's ratio	0.4		0.4
Yield strength (N/mm ²)	40	8	-
Post-yield plastic hardening modulus (N/mm ²)	210	31.5	-

3.2 Results

Fig.17 shows the deformation diagrams of the column after vertical loading and then after horizontal loading, together with magnification of the *Kumimono* region at each time. Fig.18 shows the effect of the vertical loading on the vertical displacement of rigid body 1 and on the horizontal displacement of the rigid body 3 position. In each case, the displacement was assumed to occur at the rigid-body CG. The stress in the vertically loaded column in the region of the cutout is lower than the stress found in the buckling analysis of Section 2, and the deformation diagram also confirms the absence of column buckling. It might be expected that, in the central compressive state, deformation in which the cutout

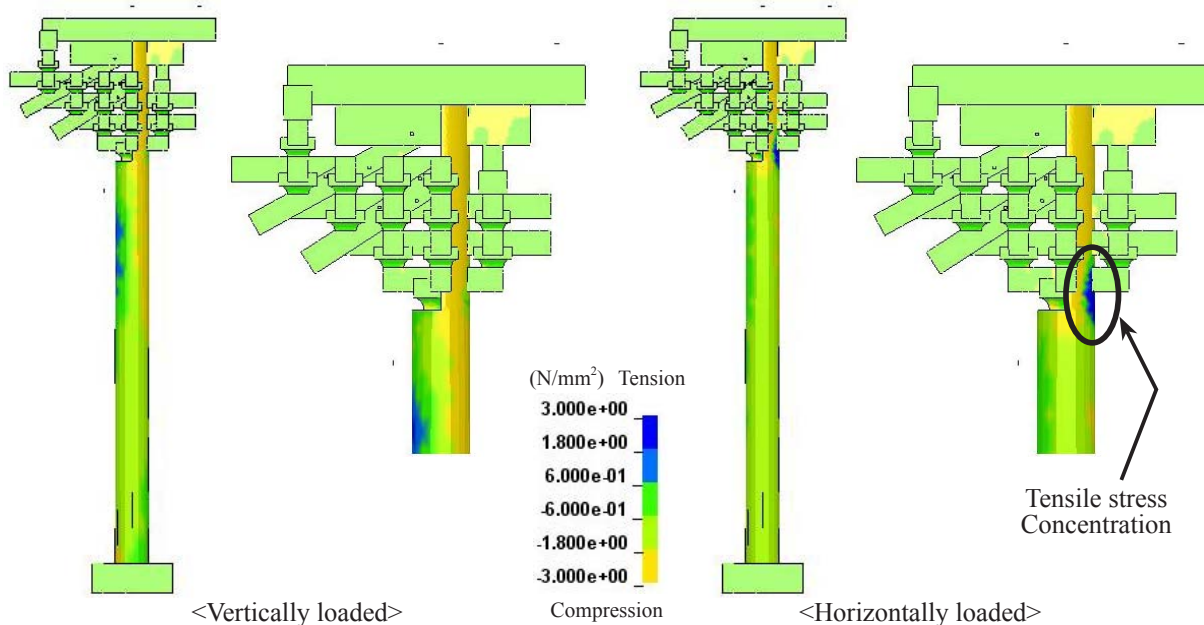


Fig.17 Deformation diagrams.

becomes the compressive region would occur in the same way as that found in Section 2, but in the present analysis the column was found to deform in the opposite direction, presumably because the vertical load acted not only on the center but also on the eccentric positions. It is also possible that the *Kumimono*-bearing column was pushed back by a slight force acting on it from the *Ebikoryo* and from the *Kumimono*-bearing column to which the *Ebikoryo* was coupled, as shown in Fig.10. However, it was also confirmed that when a horizontal force was applied from the direction of the *Ebikoryo*, even if the load was small, column bending deformation predominated and buckling-type deformation rapidly proceeded. As the deformation proceeded, the occurrence of tensile stress concentration in the column cutout region was observed.

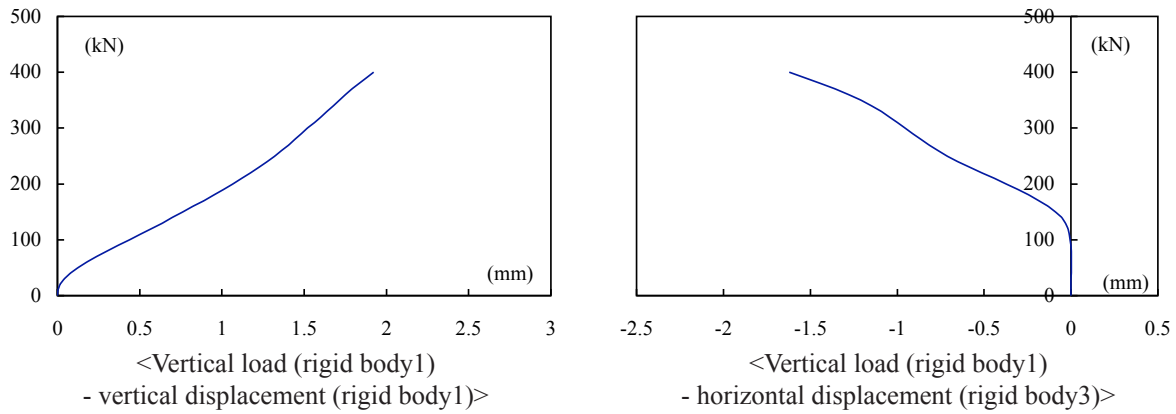


Fig.18 Relation between vertical load and displacement.

4. IN-PLANE SHEAR TEST OF FRAMES

4.1 Specimen

Fig.19 shows the test specimen. A total of two frame specimens of one-third scale were tested. Test specimens were designed based on a field survey of the existing temple. Both specimens have a column with 180mm circular section and the span between columns is 1180mm. The specimens consist of columns, beams (*Nuki*, *Koryo*, *Daiwa* and *Hijiki*), *Kumimono* and board walls. Compared with test specimen A, test specimen B is reinforced by *Nageshi*, which is newly-created beam. Almost all members are made by *zelkoba* without the top beam made by *Douglas fir*.

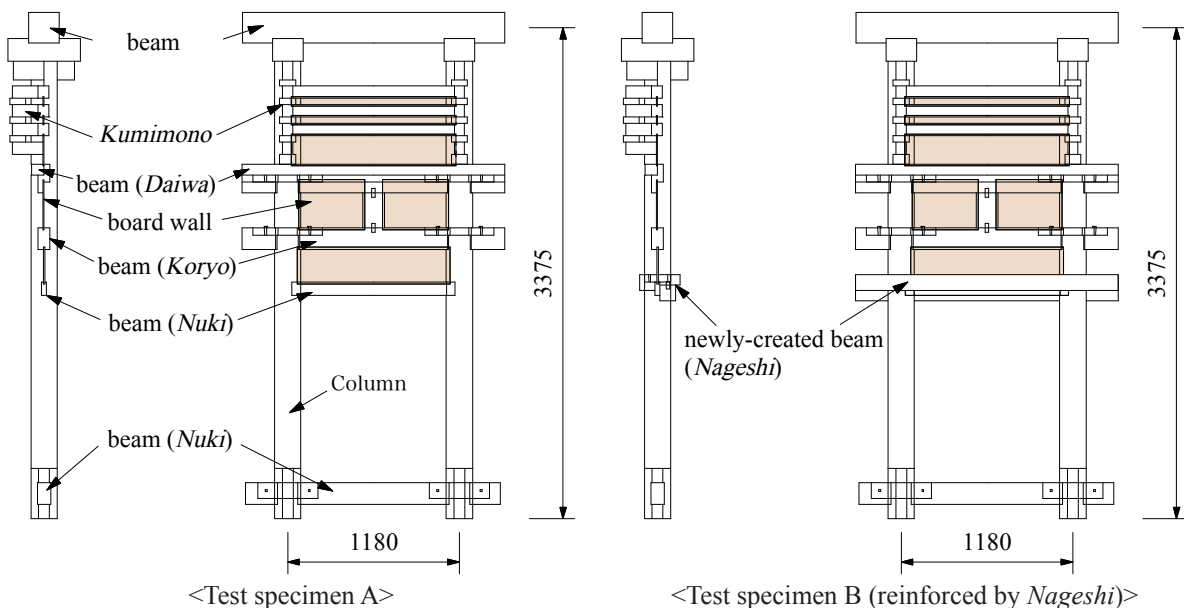


Fig.19 Test specimen of in-plane shear test of frames.

4.2 Test setup

Fig.20 shows the test setup. The specimens were horizontally constrained at the bottom of the columns, however the bottom of the columns can be uplifted and rotated. The specimens were loaded lateral cyclic shear forces by a horizontal hydraulic jack while a constant axial load was applied by a vertical hydraulic jack. The magnitude of applied axial load was 100kN. The incremental loading cycles were controlled by story drift angle, R , defined as the ratio of horizontal displacement to specimen's height. The lateral load sequence consisted of three cycles to each story drift angle, R of 1/450, 1/300, 1/200, 1/150, 1/100, 1/75, 1/50, 1/30, 1/15 radians followed by a half cycle to R of 1/10 radians.

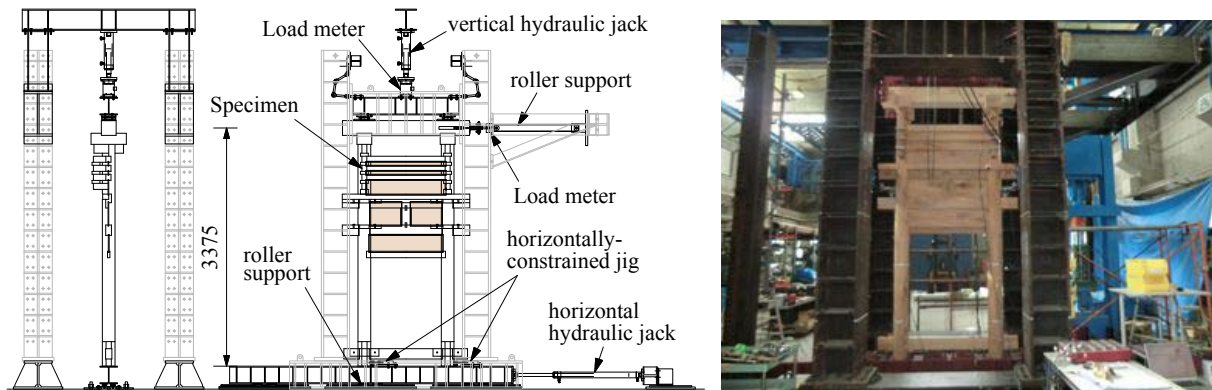


Fig.20 Test setup

4.3 Results

The relation between horizontal load and story drift angle and ultimate deformation state are shown in Fig.21 and Fig.22, respectively. As shown in Fig.21, both specimens showed a stable load versus story drift angle response. The maximum load of specimen B (reinforced type) is about two times greater than that of specimen A (normal type), so the method to reinforce by the newly-created beam (*Nageshi*) is proved to be effective.

For both specimens, the stiffness is gradually decreasing from R of 1/150radian, because partial compressive deformation perpendicular to the grain were occurred at the end of the beams and board walls (see Fig.23). In addition, the rotation of the bottom of the columns was observed at R of 1/100radian (see Fig.24). However, fatal damage and crack could not be found until R of 1/15radian. Finally, board wall was broken due to out-of-plane bending (see Fig.25), and then the horizontal load was considerably reduced.

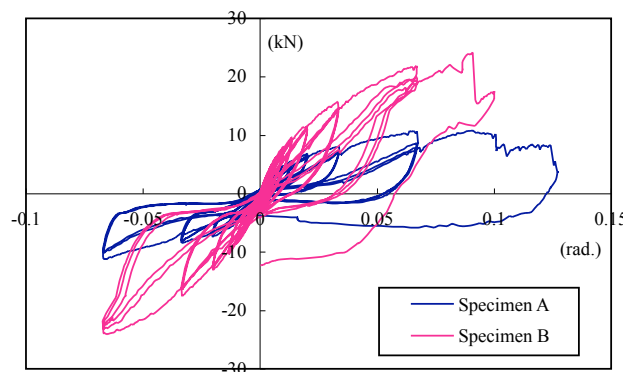


Fig.21 The relation between horizontal load and story drift angle



Fig.22 Ultimate deformation state.



Fig.23 Partial compressive deformation.



Fig.24 Rotation of the bottom of the column.



Fig.25 Crack of board wall.

5. CONCLUSIONS

In this study, we performed a 3D finite element analysis of a column bearing an integral *Kumimono* assembly and the in-plane shear tests of frames. The findings were as follows.

1. When the analytical method was applied to the buckling in a column with a cutout region, it initially led to column rigidity values substantially higher than those found experimentally, but yielded generally accurate values when the yield strength was assumed to be close to that of the actual material strength.
2. Application of the analytical method to the *Kumimono*-bearing column confirmed the absence of column buckling with only a vertical load.
3. The analysis also confirmed that if even a small horizontal load is applied to the column by a *Ebikoryo*, bending deformation rapidly progresses and the stress concentration rises on the tensile side of the column.
4. In in-plane shear tests, frames including column, board wall, beam and *Kumimono* showed a stable load versus story drift angle response.
5. The method to reinforce by the newly-created beam (*Nageshi*) is proved to be effective because the maximum load of reinforced specimen is 2 times greater than that of the normal specimen.

ACKNOWLEDGEMENTS

This study was performed at the direction of the fiscal 2010 Higashi Honganji Seismic Research Committee, and was also performed by the Kajima Foundation's Research Grant. We are deeply grateful to JSOL Corporation for their advice and assistance relating to the finite element analysis.

REFERENCES

- Wood Industry Handbook, Maruzen Co., Ltd., 1982.6
 Standard for Structural Design of Timber Structures, Architectural Institute of Japan, 2006.12