



ANALYSIS OF EARTHQUAKE RESISTANCE OF FIVE-STORIED TIMBER PAGODA

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SUMMARY

Recent research developing in the area of traditional timber buildings has enabled us to evaluate the earthquake resistant capacity of a five-storied timber pagoda. The scope of the present paper is to describe the analysis methods developed for seismic design of a five-storied pagoda that is a unique traditional architecture in Japan, as well as, to verify the analysis model by comparing with the microtremor measurement. The timber pagoda in the present study was originally designed by a modern carpenter today, being strengthened by us against earthquake and wind loads. We employed two types of models to conduct the structural design analysis. One is a 2-dimensional non-linear frame model utilized for step-by-step static analysis to calculate the load-displacement relation, where the ultimate resistance of the frame was described. As a key mechanism of earthquake resistance, a semi-rigidity model was introduced at the joint between a penetrating beam and a column in this frame model. Another model is a lumped masses one of which spring model was derived from the static analysis using the frame model. The numerical results demonstrated that the structure would be safe against the anticipated ground motion for return period of 800 years. Finally, the natural periods and modes obtained from the microtremor measurement were compared with the analysis model. The comparison showed significant difference, suggesting the actual behavior of the timber pagoda at high level of response amplitude should be recorded during large earthquakes or heavy typhoons to verify the analysis models employed for structural design.

INTRODUCTION

Five-storied timber pagodas constructed in Japan have been subjected to a number of destructive or damaging earthquakes during their history and they have survived due to the excellent seismic performance or by chance owing to the characteristics of the earthquake motions that hit them. Question why the traditional timber pagodas withstood safely the strong motions has been discussed in Japan since the beginning of 20th century from a structural point of view. The seismic characteristics discussed in the past studies were exemplified as: 1) flexible structure that means a structural idea to make the structure flexible to reduce seismic forces, 2) comparatively large damping at a number of joints with high non-

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linearity, and 3) energy-bypass effect caused by a collision between a central column and the frame. However, owing to its inherent structural complex, there was no traditional timber pagoda designed on the basis of structural safety evaluation through structural analyses. In the history, traditional timber pagodas were designed and constructed only by carpenters from their experience with their technical knowledge of the existing structures. On the other hand, the recent studies developing in the area of traditional timber structures has enabled us to evaluate the earthquake resistant capacity of a five-storied timber pagoda. The main scope of the present paper is to describe the analysis methods developed for structural design of a five-storied timber pagoda. Furthermore, the present paper deals with the dynamic characteristics obtained from the microtremor measurements after completing the building that was structurally designed through the developed approach. It was the first case in Japan that a five-storied timber pagoda was approved and constructed on the basis of engineering evaluations. The timber pagoda introduced in this paper, Eimyoin Goju-no-to, constructed in Himi-City, Toyama Prefecture in Japan in 2002.

STRUCTURAL DESIGN AND ANALYSIS MOEL

Structural Design

A newly built five-storied timber pagoda, Eimyoin Goju-no-to, shown in Figs. 1 through 3, was designed originally by a carpenter today following the type of the Edo era (during 17th and 18th century). The overall height is 15.2m with a decorative pole, being smaller one among the existing pagodas in Japan.

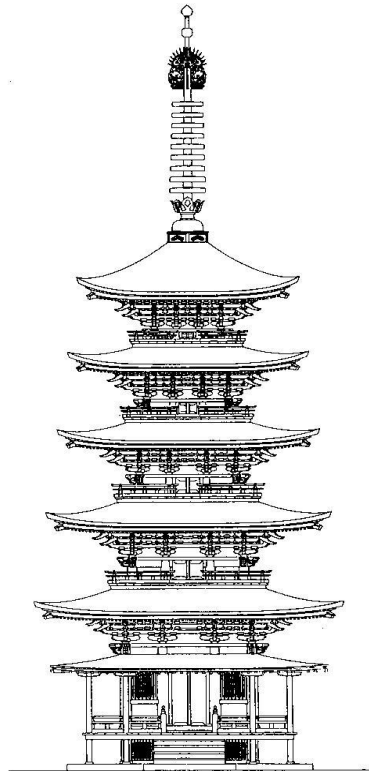


Fig.1 Elevation

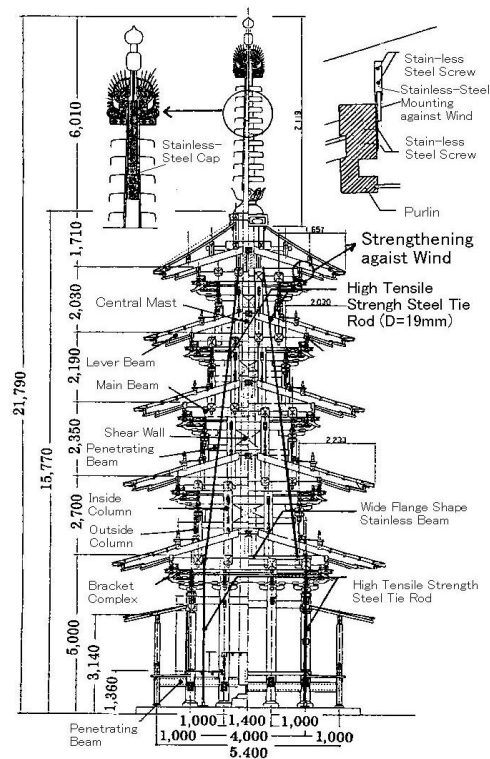


Fig.2 Sectional elevation with strengthening

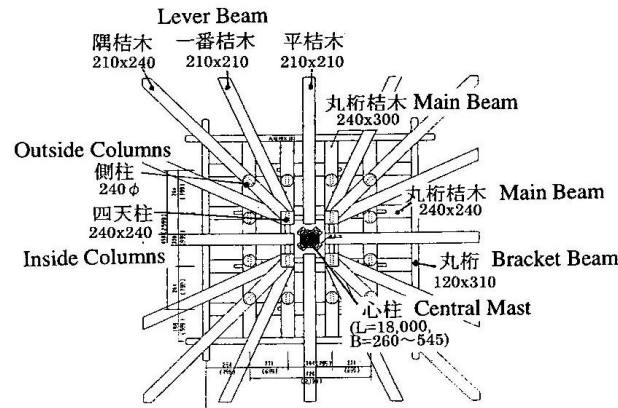


Fig.3 Sectional plan at 3rd story

Since the preliminary calculation revealed us its lack of horizontal resistance against earthquake and wind loads, strengthening of the structure was designed by us, shown in Fig.2. The structural strengthening was designed by; 1)Adding the penetrating beams, 2)Installing the timber shear walls, and 3)Strengthening the decorate pole of bronze. Furthermore, wind-resistance design was conducted by; 4)Rodding high tensile strength steel tie from the base to the uppermost story, and 5)Mounting the stainless-steel members at the eaves.

Static Analysis Model

Fig.4 describes the two-dimensional frame model employed in the structural design analysis against earthquake and wind. The structure is composed of the exterior square formed by “Gawa-bashira” (outside columns, b-section in this figure) and the interior square formed by “Shiten-bashira” (inside columns, a-section). The central must “Shin-bashira” is suspended from the level of the 4th and 5th story(See Fig.2). The structural members were mainly of Japanese Hinoki cypress and Hiba arborvitae trees. The roof was sheathed with bronze plate of thickness of 4mm. Total weight of the monument was estimated to be 470kN. Since the horizontal in-plane rigidity of the frame at each floor was evaluated to be sufficiently stiff from the structural analysis, the timber structure model composed of the outer and inner structural faces(Fig. 4). At the joint of the penetrating beam “Nuki” and the column, the semi-rigidity model [1] was introduced, characterized by the embedding behavior of the column into the beam, shown in Fig.5. In general, such elasto-plastic model of the semi-rigidity at the joint of the frame must provide one of the main anti-seismic properties of the traditional timber buildings. The bracket complex “Masugumi”, the most specific element to the traditional timber structure, was idealized by a series of translational springs illustrated in Fig.6. For modeling of the translational spring, the wooden dowel was embedded into the wooden block, and at the same time, both the dowel and the block were deformed as an elastic shear body, presented in Fig.7. The ultimate horizontal resistance of a bracket complex was evaluated from shear strength of dowells. The wooden shear wall installed for strengthening was also modeled by a shear spring characterized by bi-linear elasto-plastic manner on the assumption the wooden panel was embedded into its surrounding column and beam, shown in Fig.8.

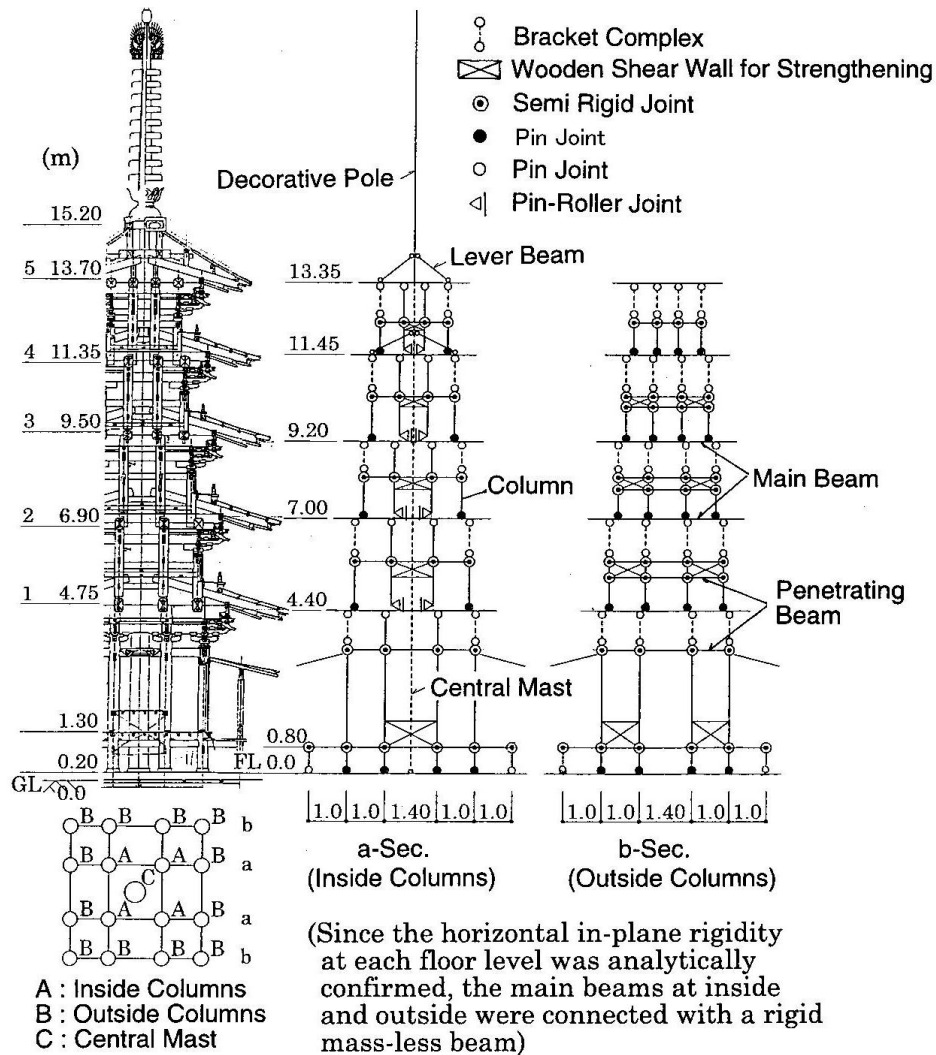


Fig.4 2-dimensional frame model provided for static analysis

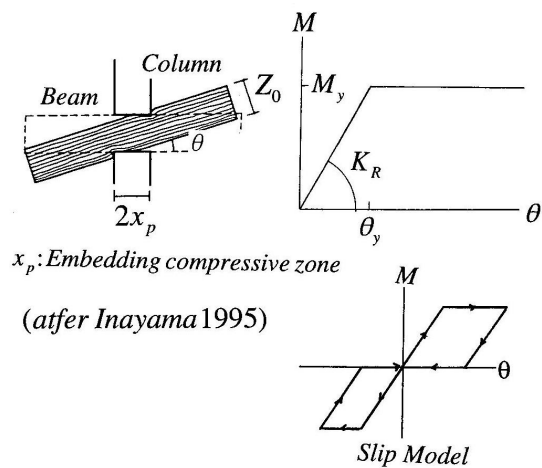


Fig.5 Semi-rigidity model at joint of penetrating beam and column

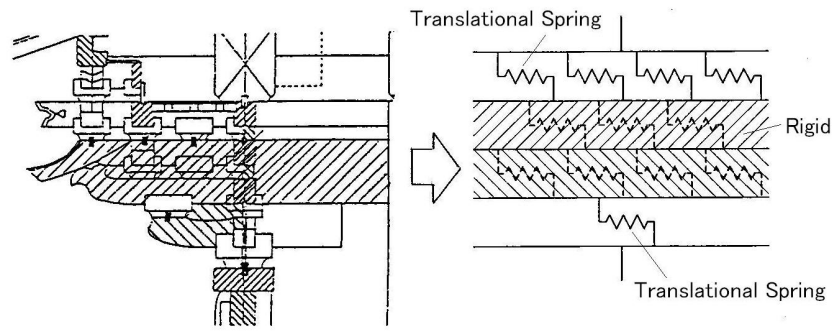


Fig.6 Formation of spring model of bracket complex

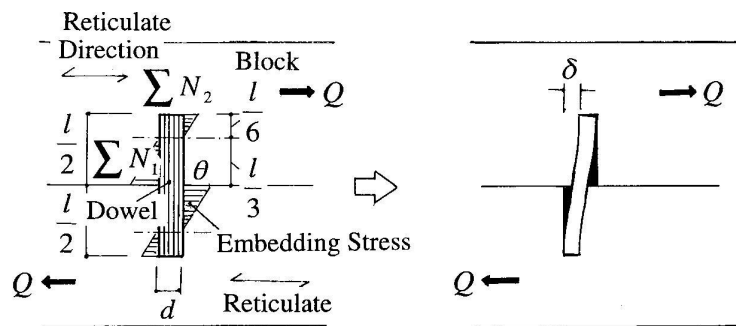


Fig.7 Mechanical model of horizontal resistance by block and dowel

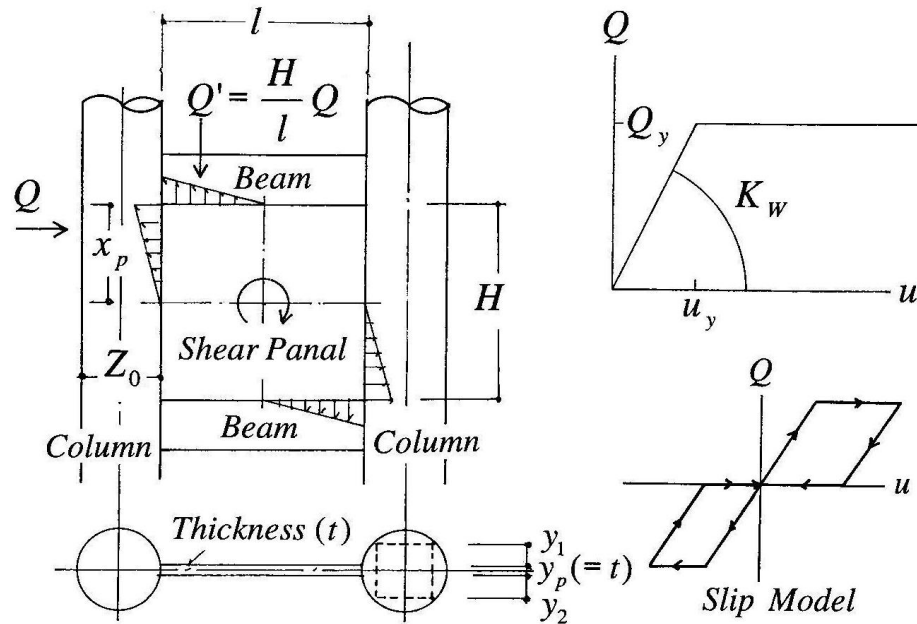


Fig.8 Mechanical model of wooden shear wall

Eigenvalue Analysis

Eigenvalue analysis of the frame model shown in Fig.4 resulted in the natural period of 1.1s and 0.78s for 1st and 2nd modes, respectively. The calculated natural period suggested that the structure had the same seismic property expressed by “Flexible structural design” as the existing historical pagodas. Hence, it should be recognized that the natural period was calculated at the story drift angle of 1/200, defined as the ratio of the story drift to the floor height. Note that natural period of a traditional timber building depends on amplitude of response. Since the fundamental period of the five-storied timber pagoda in Horyuji-temple, the oldest extant timber building in Japan, was reported to be 1.1s observed in the microtremor records at much lower level of amplitude, and since the natural period was estimated to be 0.78s from the empirical correlation equation between the natural period and the height, $T=0.05H$ (Yamabe and Kanai 1988[2]), the calculated period could be adopted from an earthquake engineering point of view.

Lumped masses Model

In order to conduct the seismic response analysis, a multi-degree of freedom model equivalent to the 2-dimensional frame one shown in Fig.4 was successfully employed. Fig.9 describes the lumped masses model utilized in the present study. The elasto-plastic (perfectly elasto-plastic bi-linear model) stiffness of the shear spring of this MDOF system was obtained from the static step-by step analysis using the 2-dimensional frame model, as illustrated in Fig.10. In the present model using the computer code ABAQUS, energy-bypass effect caused by a collision between a central column and the frame was simulated by using GAP-element in the dynamic response analysis to severe earthquakes.

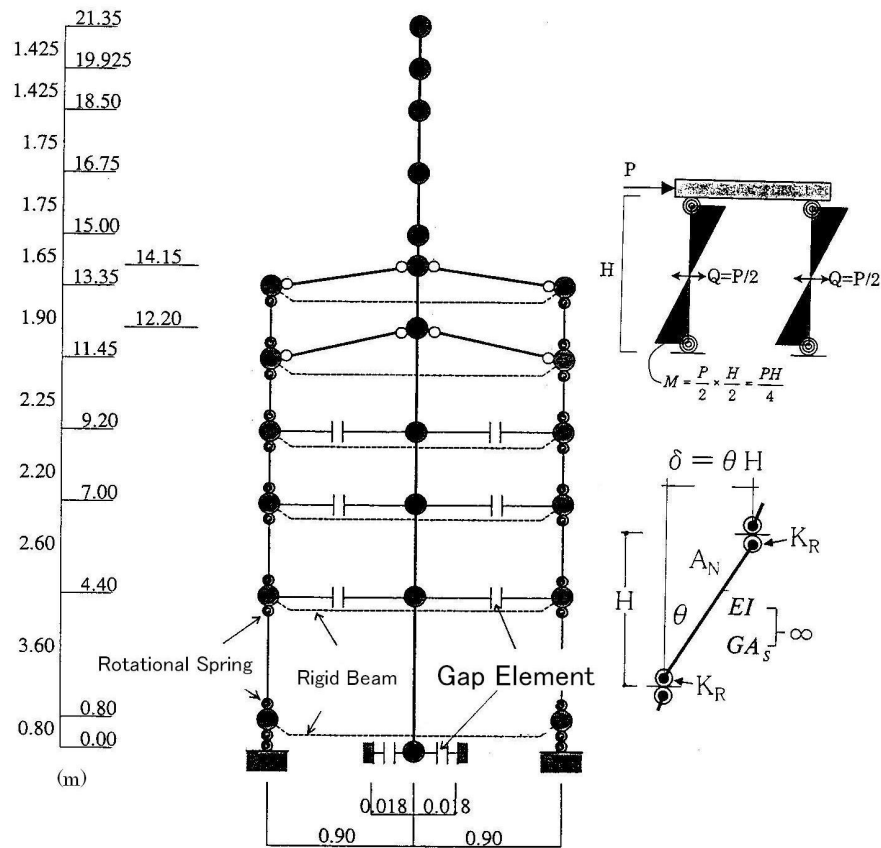


Fig.10 Lumped masses model employed for dynamic response analysis

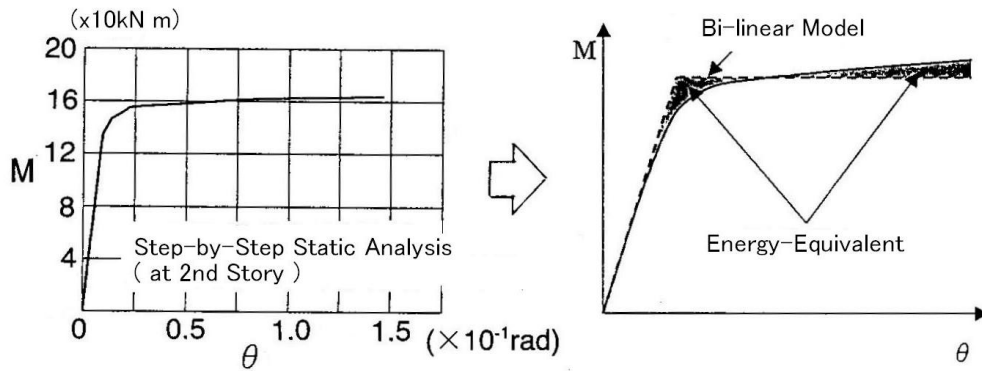


Fig.11 Bi-linear spring representation for seismic response analysis using MDOF model

Input Ground Motion

The input ground motion level required to examine the earthquake resistant capacity was probabilistically evaluated from the seismic hazard analysis utilizing the database of the earthquake records with the magnitude greater than 5.0 that occurred during the period between 1600 and 1995 in the central region of Japan. Since the soil condition has a significant effect on the amplification of the earthquake motions, a one-dimensional model of shear wave propagation was employed through the equivalent-linear technique. This approach to define the input motion level at the ground surface provided level of $25.0 \times 10^{-2} \text{ (m/s)}$ that was corresponding to the level of return period of 800 years. The soil response analysis showed that the amplification ratio of the velocity from the base rock to the surface was approximately 1.0-1.5 with the predominant period of 0.25s. As the input ground motions for the seismic response analysis of the structure, the earthquake records of ELCENTRONS40, TAFT52EW and HACHINOHE68NS were utilized after normalizing at the velocity amplitude of $25.0 \times 10^{-2} \text{ (m/s)}$.

ANALYSIS RESULTS

Static step-by-step analysis

As results of the non-linear static analysis utilizing the 2-dimensional frame model shown in Fig.4, Fig. 12 presents the relationship between the base shear and the displacement at the main beam of 5th story's roof. It can be noticed that the load-displacement relation depends on either earthquake or wind load distributed along the height of the structure. Furthermore, Fig. 13 demonstrates the effectiveness of the strengthening of the structure, where the ultimate capacity ensured after strengthening was approximately as twice as that of the original structure designed by the carpenter. The significant increase in the horizontal resistance of the frame was attained by both addition of the penetrating beams and installation of the shear walls.

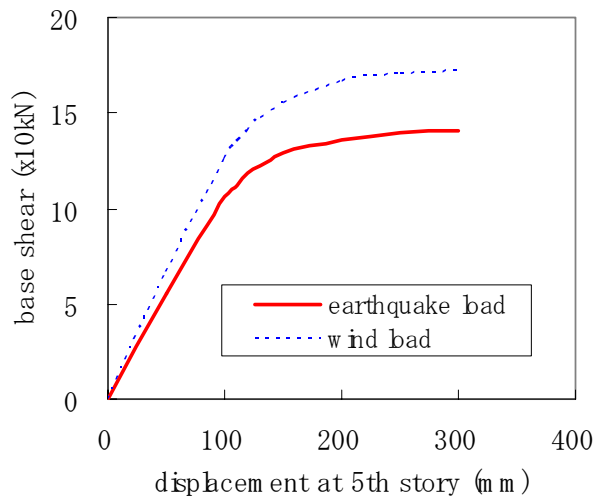


Fig.12 Load-displacement relationship

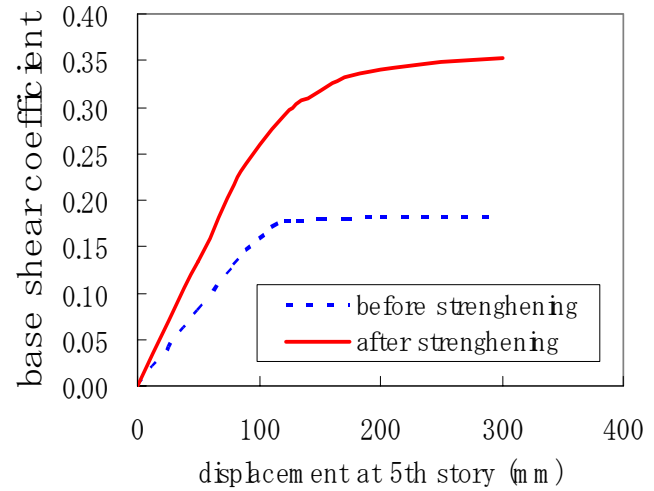


Fig.13 Effect of strengthening on horizontal resistance

Earthquake Response Analysis

Fig.14 presents the story drift angle obtained from the earthquake response analysis utilizing the MDOF model shown in Fig.9. In our structural design analysis, damping of 5% was assumed in consideration of the energy absorption at a number of joints. Although the story drift angle at 2nd and 3rd stories showed larger, the calculated story drift did not exceed the deformation limit of 1/30 corresponding to structural failure of traditional timber structures, indicating that the present five-storied pagoda would be safe against the earthquake ground motions anticipated for the return period of 800 years. Hence, the criterion angle of 1/30 was conservatively drawn from static loading tests of traditional timber frames. The present analysis also demonstrated that the decorative pole at the top acted as a mass damper during severe earthquakes. Shown in Fig.15, the peak drift angle that was calculated at 2nd story was reduced by approximately 30% due to mass damper effect. The analysis result also suggested no occurrence of collision, causing energy-bypass effect, between the central mast and the beams during the anticipated earthquake ground motions.

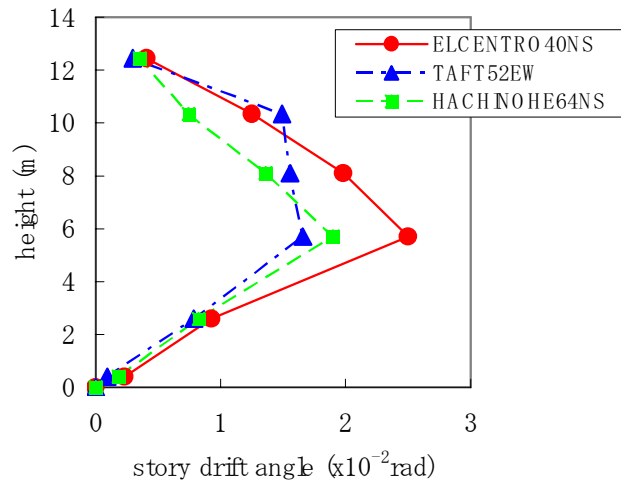


Fig.14 Calculated peak story drift angle

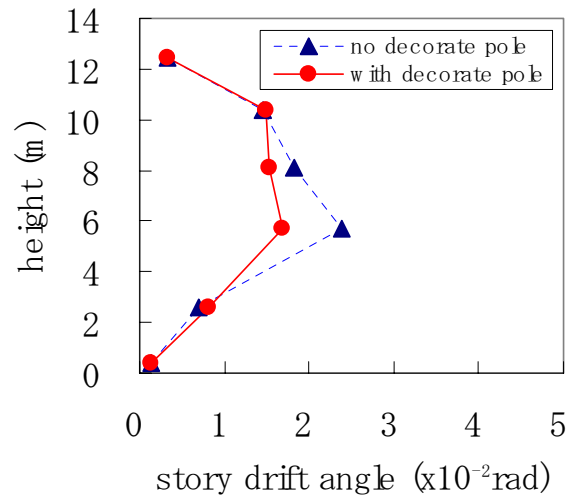


Fig.15 Effect of decorate pole on reducing response (ELCENTRO40NS)

MICROTREMOR MEASUREMENTS

Measurements

After completing the five-storied timber pagoda(See Photo.1), Eimyou-in Gojyu-no-to, the microtremor measurements and the free vibration tests were conducted to investigate the fundamental dynamic characteristics of this newly-built structure, as well as, to verify the structural analysis model. As a result of microtremor measurements, Fig.16 presents the transfer function from the base to the main beam level of 5th story' roof. Table 1 summarizes the natural period and damping factor, evaluated from both the microtremor records and the free vibration tests. Shown in Table 1, although the displacement amplitude differed in level, there showed almost the same natural period and damping factor between the microtremor and the free vibration. The natural frequencies were 1.8Hz and 5.5Hz for the 1st and 2nd translational modes, respectively. There also appeared the peak of the transfer function at the frequency of 3.0Hz corresponding to the torsional mode(See Fig.16). It can be noticed that the observed damping factor of 1.6-1.7% was smaller than those reported in the past experimental studies of the historical timber pagodas. Such small damping came from little loosening at the joints of the newly built structure owing to effort done by the carpenter. As a result of the free vibration tests, Table 2 provides the component of the displacement at the main beam level of 5th story's roof, caused by 1)sway and rocking motion of the base, 2)shearing deformation of the frame, and 3)bending deformation of the frame. As well as the past experimental studies, the bending deformation predominated in the dynamic behaviors observed in both the microtremor and the free vibration.



Photo.1 Eimyou-in Gojyu-no-to

Table 1 Natural period and damping evaluated from measurement

Method	Direction	displacement amp. * ($\times 10^{-3}$ mm)	1 st mode		2 nd mode
			Natural Period(s)	Damping Factor(%)	Natural Period(s)
Microtremor	EW	10-30	0.54	1.6	0.18
	NS	3-4	0.56	1.6	0.18
Free-Vibration	EW	340	0.54	1.7	0.18
	NS	350	0.55	1.8	0.18

* Displacement amplitude measured at main beam of roof of 5th story

Table 2 Displacement ratio at level of main beam of roof of 5th story

Direction	Base		Frame	
	Sway	Rocking	Shearing Deformation	Bending Deformation
EW	1	5	24	70
NS	1	6	26	67

(%)

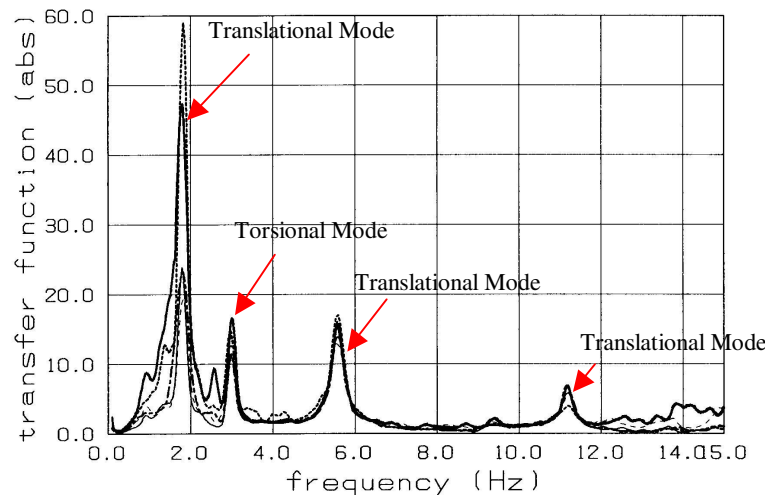


Fig.16 Transfer function from base to main beam at 5th story's roof (Horizontal NS-component)

Comparison of Measurements with Analysis

Comparison of analysis with microtremor measurements shows a significant difference in the natural period. The natural period of the 1st translational mode obtained from the measurement, 0.54s, was half as short as that of the analysis model, 1.1s. Furthermore, there found a considerable difference in the modal shape, illustrated in Fig.17. While the modal shape obtained from the measurement showed predominantly bending deformation of the frame, the analysis model presented in Fig.4 provided large amplitude of shearing deformation composed of the rotational displacements at semi-rigidity joints of penetrating beams and columns and of shear deformation of the frame. In spite of insufficient correlation between the

microtremor measurements and the analysis, the present measurements showed the same tendency of dynamic characteristics as the past experimental study[3] of a newly built five-storied timber pagoda by using the microtremor records.

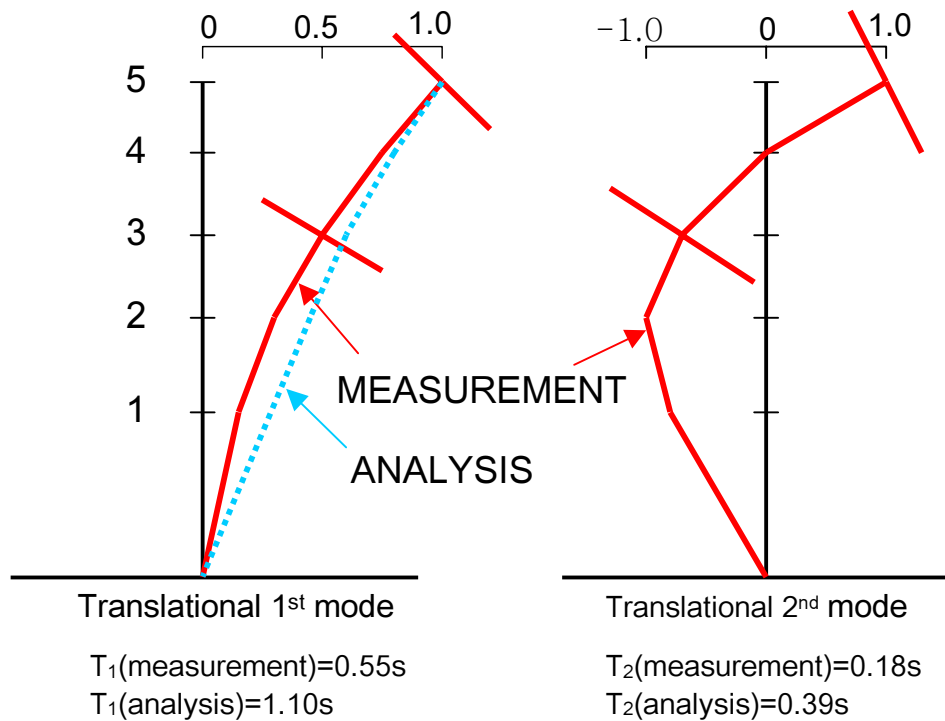


Fig.17 Modal shapes obtained from microtremor records as compared with analysis

From an engineering point of view, it is important to discuss reasoning why there showed such significant difference in the fundamental dynamic characteristics. This might come from the following reasons. 1)The actual stiffness of the structure at low-level of displacement amplitude was affected by non-structural members. 2)The story drift angle observed in the measurements was approximately 1/20,000 even in the free vibration tests, being much smaller than that of the analysis model. The stiffness of the rotational spring at semi-rigidity joints was evaluated at the rotational angle of about 1/200. 3)Since the analysis model was developed for the structural design to ensure safety of the structure, some of the mechanical contributions to the structural performance were not modeled to conduct safety side evaluation. In the present study, restitutive force produced by rocking of columns was not accounted because the proportion of diameter to height of the column was considered to be insufficient. The lower edges of the outside and inside columns were molded by pin and pin-roller expression, respectively, in the analysis model shown in Fig.4. However, rotational resistance was actually produced at those joints because the joints were improved by the carpenter to make the structure stronger.

It seems that actual behavior of structure of a five-storied timber pagoda would perform with high-nonlinearity during severe earthquakes or heavy typhoons. Nevertheless, the actual behaviors at high level of deformation have been unknown because of no records. The main purpose of the analysis methods introduced in this paper is to evaluate safety limit of traditional timber pagodas against severe earthquakes

and typhoons. Although the analysis methods presented in this paper yielded reasonable numerical results for the structural design basis, verification of the methods should be expected by comparing the analysis with records during large earthquake or heavy typhoon. There would need the further study to record the actual behaviors at high level of dynamic response. The authors, therefore, have initiated the earthquake observation at the traditional five-storied timber pagoda of Tsu-kannon temple in Tsu City, Mie Prefecture in Japan.

CONCLUDING REMARKS

The present paper deals with the structural analysis models employed for designing a traditional timber pagoda, based on achievements of the recent structural studies in the area of traditional timber buildings in Japan. As a key mechanism of resistance against horizontal load, the embedding behavior at joints of beams and columns was modeled from the semi-rigidity representation proposed by Inayama(1995). The embedding behaviors were also considered to provide the models of bracket complexes and shear walls. By employing the proposed models, we conducted the structural design analyses of a newly built five-storied pagoda. Static analysis utilizing a 2-dimensional frame model provided us the ultimate horizontal resistance, the load-displacement relation at each story and the effectiveness of structural strengthening. Furthermore, earthquake response analysis employing a lumped masses model demonstrated its seismic safety against the earthquake ground motions that were probabilistically evaluated from the seismic hazard analysis. Microtremore measurements and free vibration tests of the completed structure were successfully carried out to investigate the fundamental dynamic characteristics and to verify the analysis model. The observed dynamic characteristics showed the same tendency as those of the experimental past study of a newly built traditional timber pagoda using the microtremor records. However, comparison of the analysis model with the measurements showed significant difference in the natural periods and modes. Discussion of the reasoning why such difference arose suggested the necessity for recording actual behaviors of traditional timber pagodas during large earthquakes or heavy typhoons because the analysis model should be adopted for structural safety evaluation at high level of dynamic response.

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