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SEISMIC SAFETY EVALUATION OF HOKKEDO HALL IN TODAI-JI TEMPLE, WORLD HERITAGE

PART 2 SEISMIC DIAGNOSIS OF BUILDING STRUCTURE

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Abstract. *The Hokkedo-hall of Todai-ji Temple is one of the unique historical structures in Nara, Japan, characterized by combination of constructions of two periods of 8th century and 14th century. This architectural heritage of timber was not only designated as Japanese national treasure but also registered as the World Cultural Heritage. Furthermore, there stand a number of famous images of Buddha in this unique building. Although there is no record of severe structural damage due to historical earthquakes, Todai-ji Temple would be anticipated to be subjected to large earthquakes in the future. In order to ensure safety of both human lives and those valuable images of Buddha, it had been needed to conduct seismic diagnosis of this heritage structure by scientific approach.*

The present paper is composed of two contents, one is the fundamental dynamic characteristics of the structure, evaluated by the microtremore measurements. Another is the seismic diagnosis analysis of the structure against extremely strong ground motions. The natural periods and the vibration modes were evaluated by the microtremore measurements, together with the damping factor. These measurements also revealed the eccentricity of the structure from an engineering point of view.

In the present seismic diagnosis, two following methods were employed as, "equivalent linearization method" and "dynamic 3-dimentional frame model". The former method was introduced in the Guideline of Seismic Diagnosis of Cultural Properties in Japan, recommended by Japanese Government. On the other hand, the latter sophisticated method with non-linear analysis model was employed as expert assessment, and was verified by the microtremore measurements. Those analyses indicated that the structure would survive against extremely strong ground motions at the site.

1 INTRODUCTION

Among a number of ancient wooden architectural heritages that have survived in Japan for long centuries, the Hokkedo-hall (Sangatsudo) in Todaiji Temple complex, is one of the most valuable cultural heritages from both historical and architectural points of view (See Photo.1), therefore, this unique architecture was registered as World Cultural Heritage in 1998. Hokkedo-hall was originally constructed in the middle of 8th century when Nara was the capital of Japan. At that time, Hokkedo-hall was composed of two neighboring constructions, a main building "Seido" and a worship building "Raido" in Tempyo architectural style. During its long history, these buildings have been restored several times. In particular, when "Raido" was reconstructed in Kamakura architectural style in the middle of 13th century, it was connected by the roof structure. At the same time, Hokkedo-hall is indispensable not only for its unique architecture but also for dozens of images of Buddha being very precious from esthetical point of view. Most of them designated as National Treasure of Japan have been protected by Government as important cultural properties.

On the other hand, the ancient capital, Nara, is located near Kyoto and Kobe in Kansai district, shown in Fig.1. The building structure and the images of Buddha in Hokkedo-hall have been often subjected to historical large earthquakes, however, both of them survived against those earthquakes in their long history. This excellent experience indicates that both the building and the images of Buddha have inherent potentialities against earthquakes. It should be emphasized that, both for safety of images of Buddha and for human safety against earthquakes, earthquake resistant capacity of the building structure must be evaluated by scientific approach to respect its inherent anti-seismic potentialities. As introduced in Part 1 of our papers, an experimental study on seismic performance of the images of Buddha was conducted. Following that paper, the present study, Part 2, deals with seismic safety of the building itself evaluated from the analyses models together with dynamic properties obtained from microtremor measurements.

Photo 1 Hokkedo-hall, Todaiji

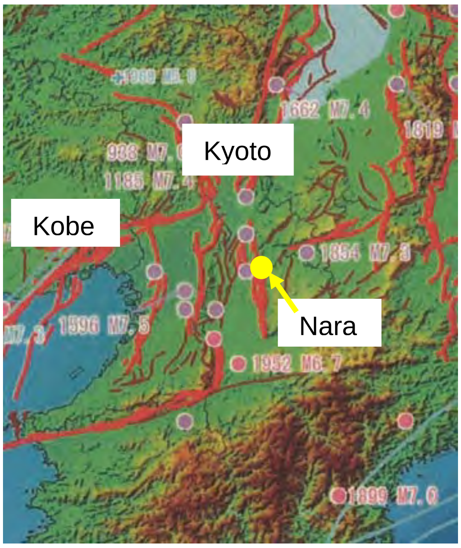

Fig.1 Seismic Activity in Kansai [1]

2. DESCRIPTION OF STRUCTURAL CHARACTERISTICS

Introduced in the previous section, Hokkedo-hall is a unique structure characterized by composing of two different period's constructions by sophisticated combining technique with a roof structure. Fig. 2 shows section and plan of the building. The building is 13m in height with a rectangle plan of 25m by 18m. There had been a total of 16 images of Buddha in ‘Seido” meaning a main hall constructed in 8th century. Another hall, “Raido” meaning worship hall was reconstructed in 1264. Both timber buildings were structurally connected and combined by the roof structure, when “Raido” was reconstructed. Shown in Photo.1, the roofing materials were tiles. From an earthquake point of view, the horizontal in-plane rigidity at the ceiling level (or the roof structure) is an important factor that must affect seismic performance and safety. Gotenjyo, a ceiling structure of which stiffness was enough to transfer the horizontal seismic force to the bearing walls, was employed in this building (See Fig.3 and Photo.2). This effect was ensured by the microtremore measurements, presented in the following section. One of the structural characteristics of ancient timber building in Japan is to utilize large diameter columns. When Hokkedo-hall was subjected to large earthquakes, its large diameter columns must have resisted against seismic force by column rocking resistance. Furthermore, the mud wall that is a major earthquake-resistant element of traditional timber building in Japan also have resisted as shear panel. Structural survey was conducted to measure such earthquake-resistant members for seismic diagnosis of the building, described in Section 4.

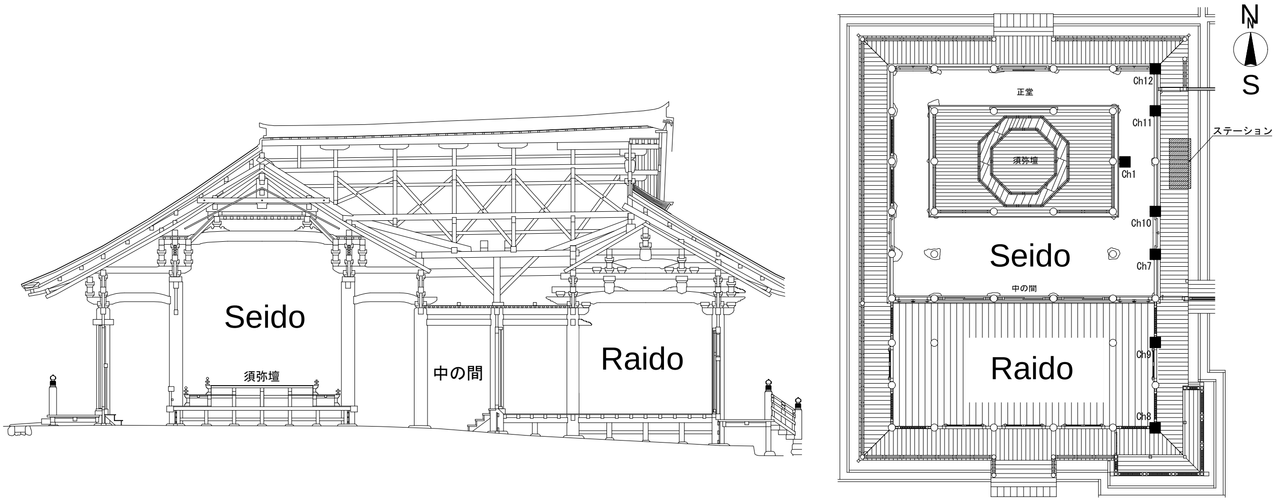

Fig.2 Elevational Section and Plan

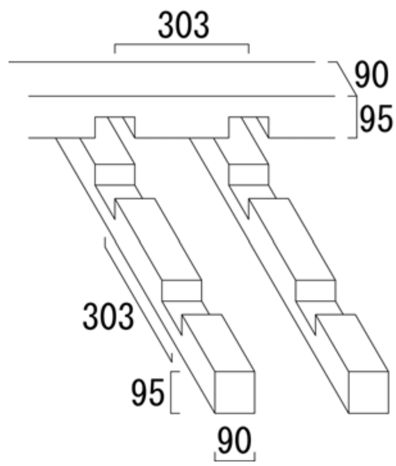

Fig.3 Rigid ceiling, Gotenjyo

Photo.2 Rigid ceiling,Gotenjyo

Photo.3 Mud wall

3. MICROTREMORE MEASUREMENTS

Mictotremore measurements can be a useful method to know fundamental dynamic properties such as natural periods, modes and damping characteristics of building structures. To investigate those dynamic performances, microtremore measurements were performed at the first stage of the present study. Utilizing SPC-51 system with a dozen of sensors of VD-15 (See Photo.4), Tokyo Sokushin Ltd., we measured microtremors of both the structure and the ground with sampling frequency of 100Hz. Fig. 4 shows the transfer function of the building from the base to the beam in EW direction. Here, the sensors were installed on the beam on the mud walls which would resist seismic horizontal force and behave as shear panels. The natural frequency of 1st translational mode in each direction was evaluated as ; 1.51 Hz and 1.72Hz, in EW and NS direction, respectively. In addition to microtremore measurements, free vibration tests were performed to evaluate the damping ratio, shown in Photo.5 and Fig.5. The damping ratio for 1st translational mode in each direction was evaluated as ; 1.9% and 2.0% in EW and NS direction, respectively. Fig. 6 describes the vibration mode for 1st translational mode in each direction. It should be noticed in Fig.6 that the structural deformation in EW direction showed irregular, i.e., the amplitude of the southern end frame in “Raido” was 5.5 times as large as that of northern end frame in “Seido”. This eccentricity was mainly

Photo.4 Sensors on the base beam

Photo.5 Free-Vibration test by man-power

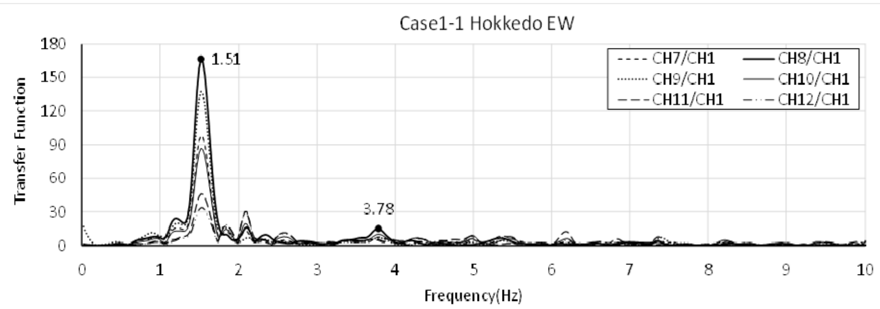

Fig.4 Transfer function from Top of seismic elements to base

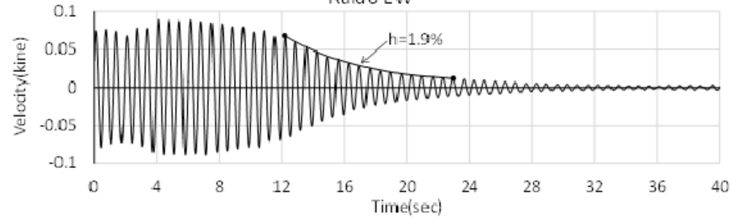

Fig.5 Wave Form by Free Vibration test

caused by the layout of the mud-walls, i.e., unbalance arrangement of shear walls, that must affect the horizontal deformation. The southern room of “Raido” was used as entrance and characterized by large opening with lack of walls, therefore, the amplitude became much larger. On the other hand, “Seido” was surrounded by the mud walls, in particular, the building had double walls in the northern structure, which caused larger stiffness with small amplitude. This dynamic behaviour should be considered in the seismic safety assessment from an earthquake engineering point of view. On the other hand, the amplitude in NS direction showed almost uniform without irregularity, described in Fig.6. No eccentricity was found in NS direction, indicating that the mud walls (shear walls) were well arranged in NS direction.

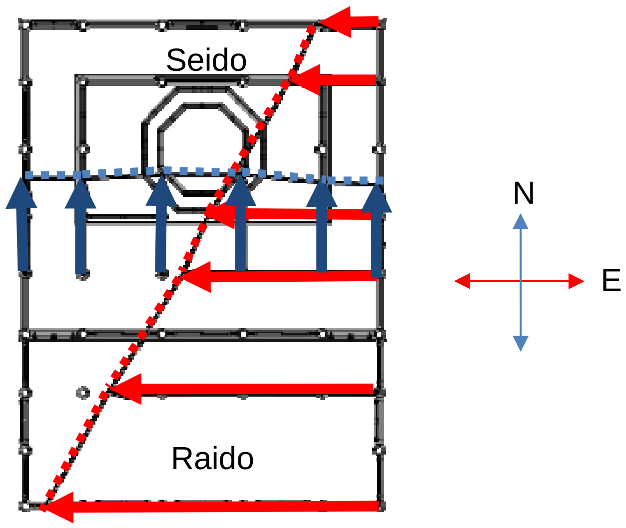

Fig.6 Vibration mode in both EW and NS direction

4. SEISMIC DIAGNOSIS

4.1 Outlines

The Kobe Earthquake of 1995 caused severe damage to many architectural heritages. After that devastating earthquake, a number of researchers and engineers in Japan have been involved in research on earthquake resistant capacity of traditional timber buildings. On the other hand, this experience promoted the Japanese Government to provide guidelines for assessment of seismic safety of important cultural properties (buildings), as Japan is an earthquake-prone country with rich cultural heritages. On the basis of those past studies, the Japanese Government issued “Guidelines for the Seismic Diagnosis of Important Cultural Properties (Building and Other Structures) in 2001 and revised it in 2012[2]. In this national guideline, the engineers’ manual introduced how to quantitatively evaluate seismic safety of heritage structures of timber by means of two methods as ; 1)Method using energy conservation law (corresponding to ultimate strength design in Japanese Building Code, JBC) and 2) Equivalent linearization method (corresponding to seismic design based on calculations of response and limit strength introduced in JBC). These methods were prescribed in the guideline as basic assessment. At the first step in the present study, the latter method using equivalent linearization was employed. Seismic load, defined by response spectra, required in Japanese building Code was applied to assess the structural safety. In addition, non-linear 3-dimensional frame analysis model was employed to evaluate seismic safety as exact as possible as expert assessment. Simulated earthquake ground motions of which target spectrum was defined by JBC were utilized for the input motions to the earthquake response analysis.

4.2 Safety Evaluation by Equivalent Linearization Method (Basic Assessment)

The timber structure was idealized by a single degree of freedom model on the assumption that horizontal in-plane rigidity of the roof structure including the ceiling frame was stiff enough. The weight of the model was estimated from the structural survey and the literature[2].The horizontal load-deformation relation was evaluated by sum of resistance by anti-seismic elements. Hence, the anti-seismic elements of this traditional timber structure were semi-rigidity joints of penetrating beams, mud walls, hanging walls, and column rocking resistance. The architectural site survey of Hokkedo hall gave us both the dimension of the structural members and the structural condition of the joints, which enabled us to calculate the load-deformation relation of each anti-seismic element listed above. Fundamental load-deformation relations of various anti-seismic elements used for seismic design and seismic diagnosis of the traditional timber buildings have been available in Japan[2,3]. Utilizing these mechanical models of the anti-seismic elements, non-linear horizontal load-deformation relation of the timber structure in each direction (EW, NS) was finally modelled. Fig. 7 shows the load-deformation relation model in EW direction. Shown in Fig.6, irregular deformation due to eccentricity was found in EW direction, therefore, load-displacement relation of the total was reduced as described in Fig.7.

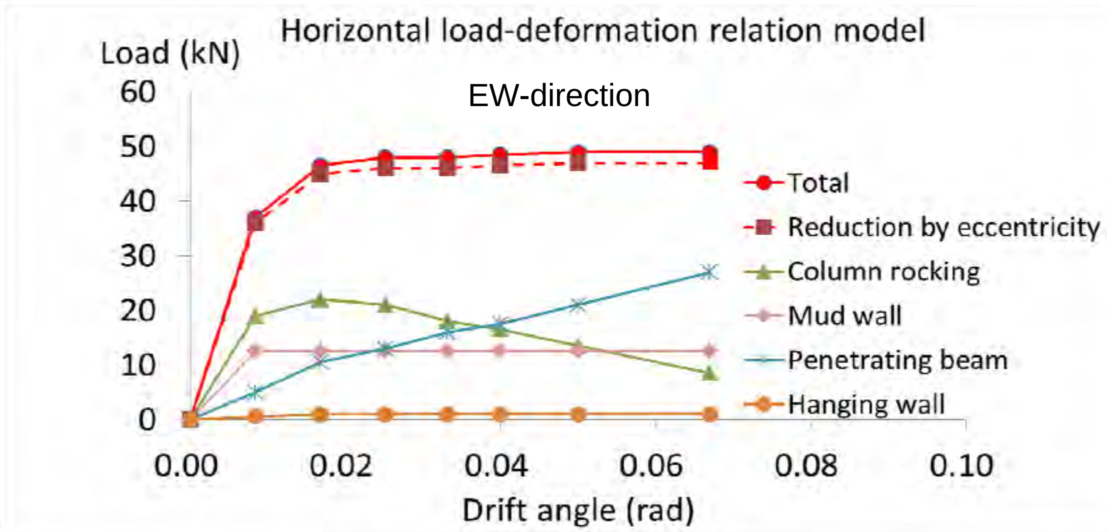

Fig.7 Load-deformation relation model in EW direction

As the seismic load for the present diagnosis, the acceleration response spectrum to assess safety limit, prescribed in Japanese Building Code, was referred, shown in Fig.8. The response spectrum of “major earthquake”, i.e. the extremely strong ground motion, was utilized for seismic safety evaluation. The seismic load given by the response spectra, shown in Fig.8, is corresponding to earthquake motions at the engineering bedrock of which shear velocity is larger than 400m/s. Hence, if surface layers exist on the engineering bedrock, soil response of surface layers should be taken into account. However, Hokkedo-hall was constructed on rocky stiff soils at the mid-slope of the mountain. Therefore, no amplification due to soil response was considered in this seismic diagnosis. On the basis of the load-deformation model shown in Fig.7 and the required spectra shown in Fig. 8, seismic performance in Equivalent Linearization Method can be described in Fig.9. From this figure, story drift angle at the performance point in EW direction was calculated to be 1/40 for the major earthquake. In Japan, story drift of 1/30 has been recommended at safety limit for seismic evaluation of timber heritages designated as important cultural properties[2]. This calculation therefore indicated that Hokkedo-hall would be structurally safe against the major earthquakes defined in Japanese Building Code. Furthermore, it verified the historical record that the building have survived against large earthquakes during its long centuries.

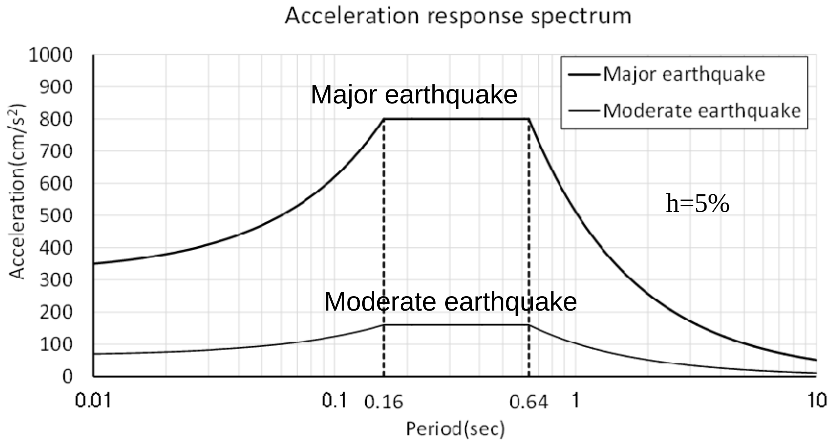

Fig.8 Acceleration response spectra defined in Japanese Building Code

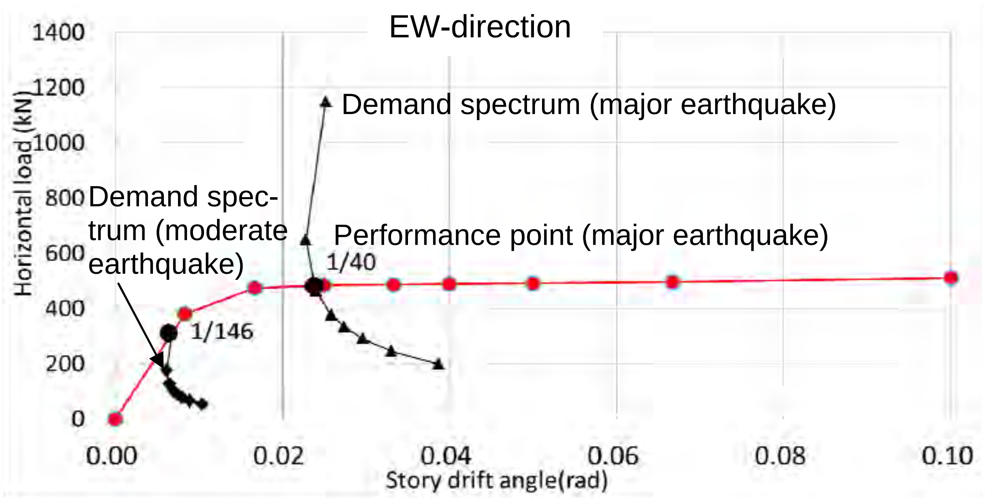

Fig.9 Earthquake performance evaluation in EW direction

4.3 Non-linear 3-Dimensional Frame Model (Expert Assessment)

In the previous section, earthquake resistant capacity of the structure was roughly estimated by the seismic diagnosis employing Equivalent Linearization Method, where a simplified single degree of freedom model was assumed. Although horizontal in-plane rigidity of the roof structure with the ceiling structure of lattice frame seemed stiff enough, it might be needed to evaluate its deformability in order to assess the seismic safety as exactly as possible. Furthermore, actual behaviours of high non-linearity at safety limit were replaced by equivalent linear model. In consideration of limit of such equivalent linearization method, non-linear 3-dimensional frame model was also employed to assess the seismic safety in detail. This sophisticated 3-D model is categorized into exert assessment introduced in the guideline for the seismic diagnosis of heritage structures [2]. In the present analysis, mechanical properties of the wood, Japanese cypress trees and zelkova trees, were given by AIJ standard [4].

Fig. 10 shows the 3-dimantional analysis model with a total of 2773 joints and 2227 members. Mud walls and hanging walls were idealized by elastic shell elements. Figs. 11 and 12 describe non-linear mechanical models of a joint of a penetrating beam and column and column-rocking resistance, respectively. Fig. 13 describes load-deformation relation of the mud-wall proposed in the past studies.

On the basis of those studies, the shear modulus G of the mud walls of the present analysis model was estimated to be 0.01kN/mm^2. Eigenvalue analysis was conducted to evaluate the natural frequencies and modes, as natural frequencies and modes were evaluated from the microtremore measurements, presented in Section 3. Hence, it should be considered that the joint would not behave, during micto-tremore, as semi-rigid shown in Fig.11, but would behave as rigid joint at the joint between a penetration beam and a column, because strain level might be too small to develop rotation at the joint. At the same time, it was considered that the mechanical condition of the joints of the roof frame structure, rigid or semi-rigid, were unknown. Taking into account these considerations, eigenvalue analysis was conducted for three models by paying attention to the joint of a penetrating beam and a column as ;A) rigid model, B)semi-rigid joint model with partially rigid joints , and C) semi-rigid joint model with partially hinge joints. Hence, “partially rigid joints” and “partially semi-rigid joints” were corresponding to the assumed joint mechanical condition of the roof frame structure. Fig.14 shows how to model the bracket complex. Rotation was produced at the base of a brock by embedding effect as shown in Fig.11. As a result of the eigenvalue analysis, Table 1 compares the measured natural frequencies with analysis models. Compared in Table 1, the analysis model of the rigid joint “Model A” showed better agreement with the measurements. Furthermore, it would not affect the natural frequency whether the mechanical condition of the joints of the roof structure was rigid (Model B) or semi-rigid (Model C). This comparison indicates that no rotation would be caused between a penetrating beam and a column during microtremore, therefore, would behave because a rigid joint, as the strain level would be so small. Fig. 15 shows the vibration modes of which natural frequencies are listed in Table 1.

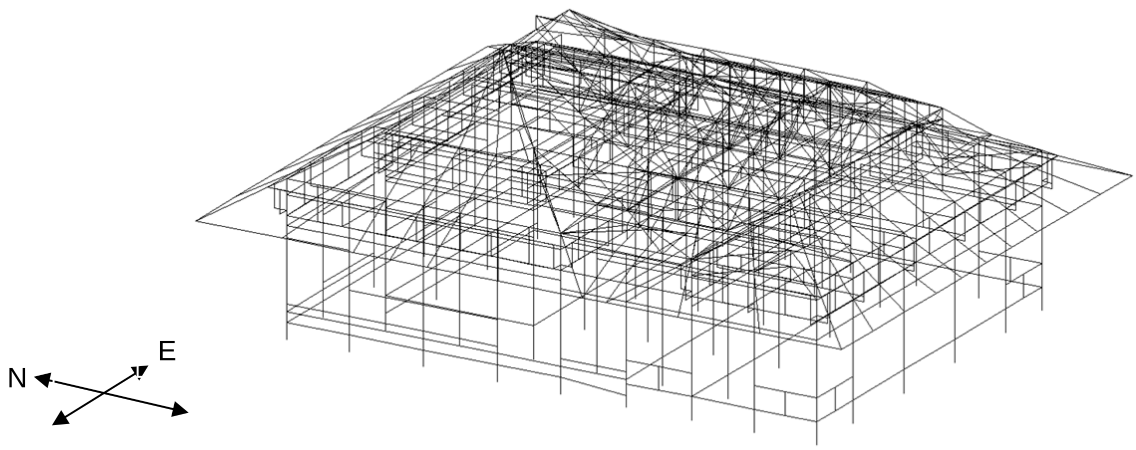

Fig.10 3-dimensional frame model

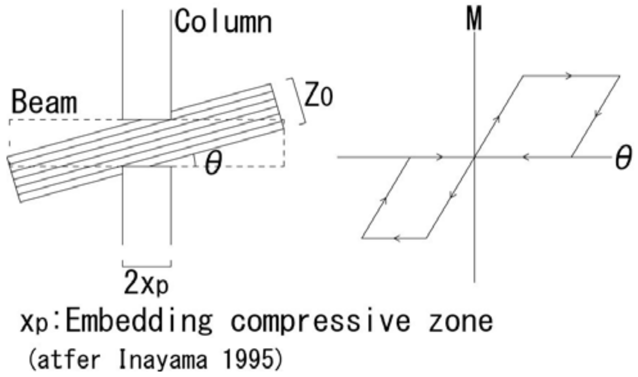

Fig.11 Model of joint of penetrating beam and column

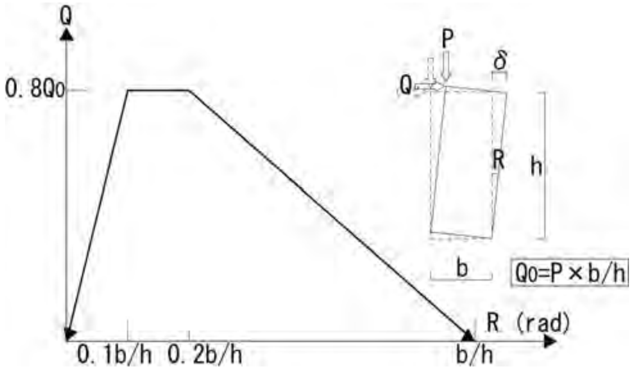

Fig.12 Model of column rocking resistance

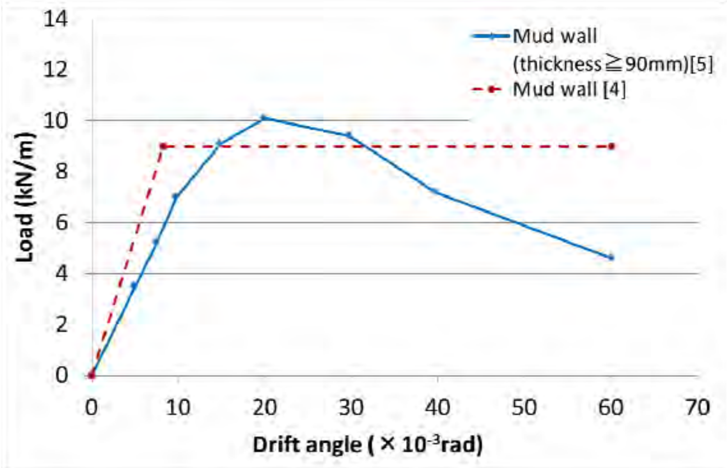

Fig.13 Load-story drift relation model of mud walls

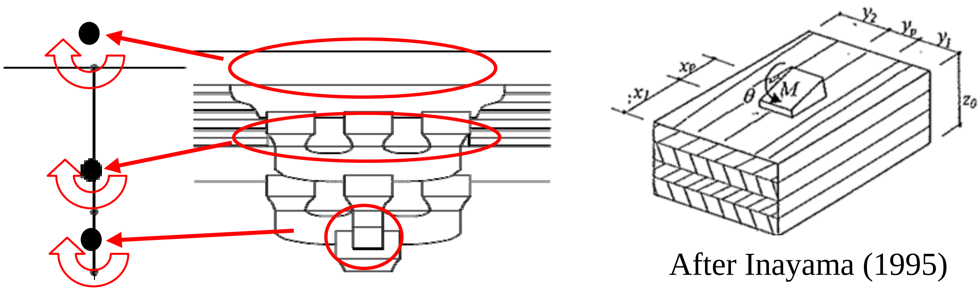

Fig.14 Model of bracket complex by using rotational spring

Table 1 Natural frequencies

		1st mode	2nd mode	3rd mode
Microtremore measurement		1.51(Hz) EW-direction translational	1.73(Hz) NS-direction translational	2.10(Hz) Torsional
Analysis model	Model A	1.71(Hz) EW-direction translational	1.93(Hz) NS-direction translational	2.02(Hz) torsional
	Model B	1.02(Hz) EW-direction translational	1.16(Hz) NS-direction translational	1.24(Hz) torsional
	Model C	1.01(Hz) EW-direction translational	1.15(Hz) NS-direction translational	1.24(Hz) torsional

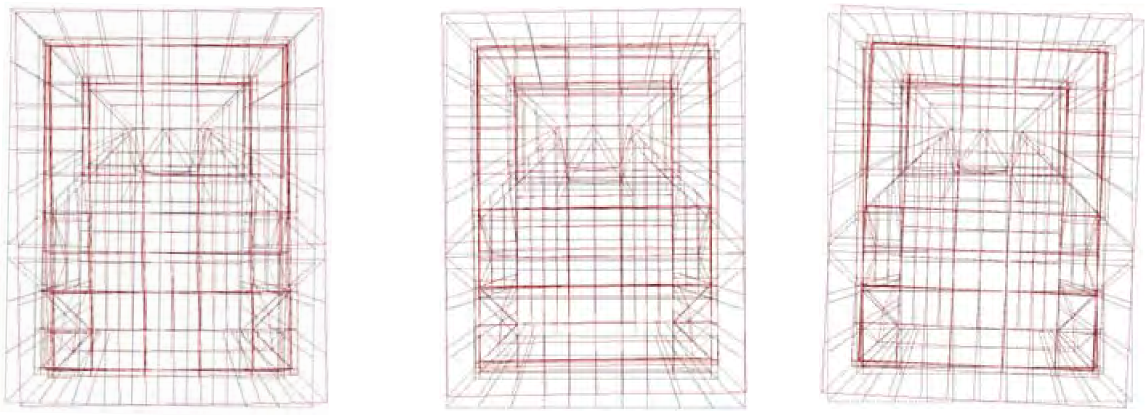

Fig.15 Vibration modes (Model A)

Non-linear earthquake response analysis of the 3-dimensional frame model was conducted. The present paper describes the analysis results for the input ground motions as ; 1) the simulated ground motion of which response spectrum agreed with that designated on the engineering bedrock in Japan Building Code. 2), and 2) the acceleration data recorded during Kobe earthquake of 1995 (NS component recorded at JMA Kobe. Time histories and response spectra of those input motions are shown in Figs 16 and 17. PGA of the input ground motions for the former and the latter ones are 0.34G and 0.82G, respectively. Those ground motions were input to the analysis model in NS direction, and in EW direction, respectively.

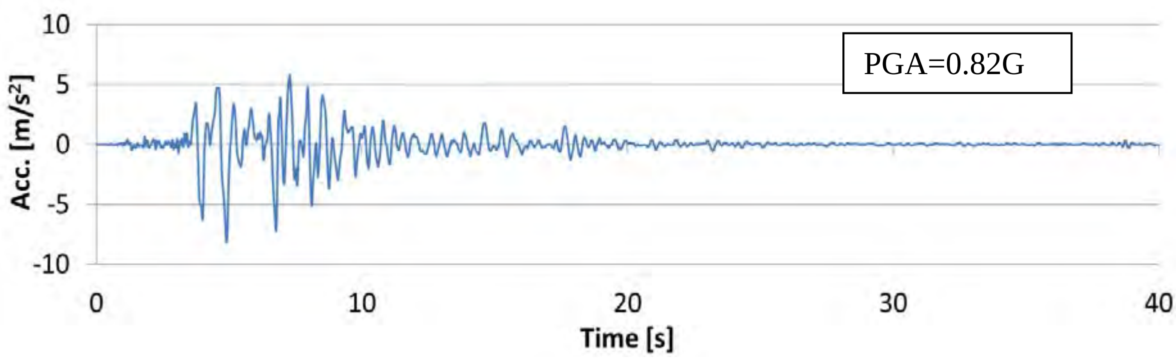

Fig.16 Wave form of input ground motion (JMA Kobe NS,

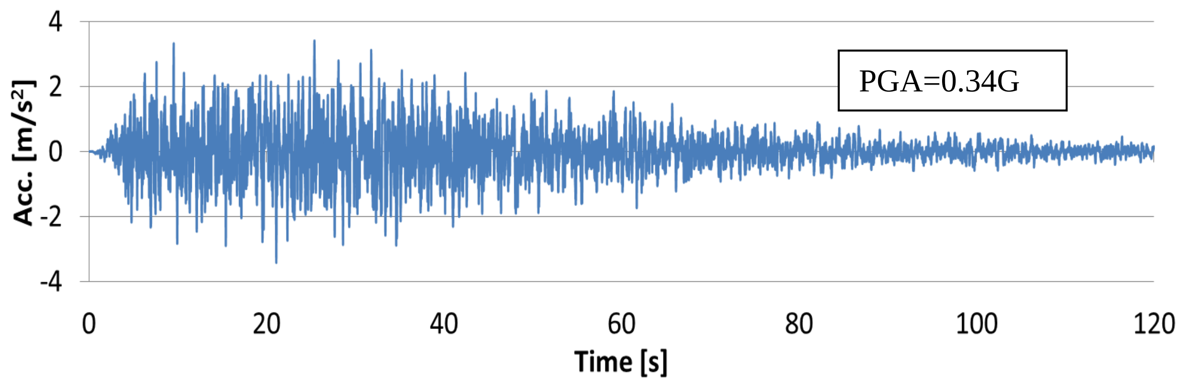

Fig.17 Wave form of input ground motion (Simulated ground motion of which response spectra is compatible with that of Japan Building Code)

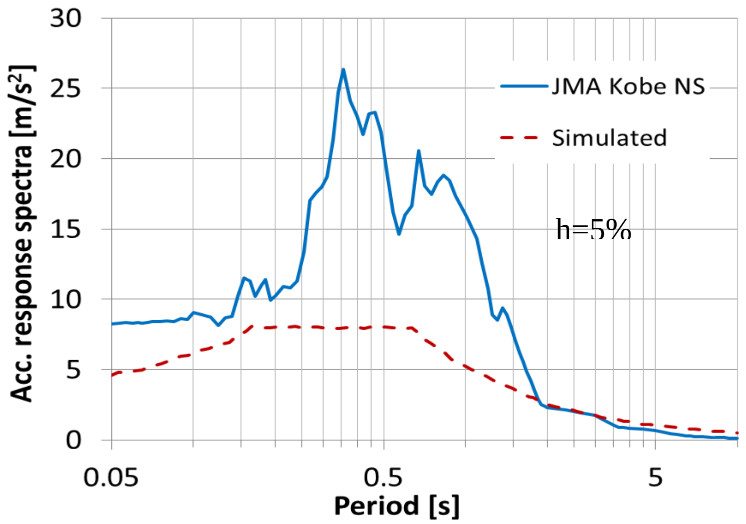

Fig.18 Acceleration response spectra of input ground motions

Tables 2 and 3 summarize the maximum story drift angle calculated for each direction of analysis case. A well as the microtremore analysis, seismic response was not affected by the mechanical condition of the joints in the roof frame, i.e., hinge (Model B) or semi-rigid (Model A). As introduced in Section 4.2, the recommended safety limit of traditional timber buildings in Japan was 1/30 in story drift angle[2]. Although the maximum story drift reached the safety limit for the input case of JMA Kobe NS in EW direction, seismic response displacements were less than safety limit for the other analysis cases. The present analysis indicates that the structure of Hokkedo hall, where a number of images of Buddha stand , has considerable inherent potentialities against large earthquakes, being consistent with the historical record that the structure have survived against them.

Table 2 Maximum story drift angle in EW direction

Model	Input motion	Peak acc. (G)	Peak vel. (cm/s)	Direction of Model	Story drift angle (rad)
Model B	Simulated motion	0.34	34	EW	1/76
	JMA Kobe NS	0.82	81	EW	1/14
Model C	Simulated motion	0.34	34	EW	1/75
	JMA Kobe NS	0.82	81	EW	1/14

Table 3 Maximum story drift angle in NS direction

Model	Input motion	Peak acc. (G)	Peak vel. (cm/s)	Direction of Model	Story drift angle (rad)
Model B	Simulated motion	0.34	34	NS	1/105
	JMA Kobe NS	0.82	81	NS	1/16
Model C	Simulated motion	0.34	34	NS	1/104
	JMA Kobe NS	0.82	81	NS	1/16

CONCLING REMARKS

In order to ensure the safety of images of Buddha in Hokkedo-hall, Totaiji Temple, which has survived against large earthquakes, not only the safety of the images of Buddha themselves but also earthquake resistance of the building should be assessed. The present paper introduced this irreplaceable unique structure characterized by combination of two constructions by the traditional roof structure. Microtremore measurements evaluated the fundamental dynamic characteristics of this timber monument. Furthermore, the torsional mode in dynamic phase was found by the measurements in EW direction, caused by eccentricity due to arrangement of the anti-seismic elements. However, seismic diagnosis by both employing simplified equivalent linearization method and non-linear 3-dimensional frame models demonstrated the inherent earthquake resistant capacity of the timber building where a number of images of Buddha were standing, which would consistent with the historical record that the structure have survived against large earthquakes. Further study would be expected to evaluate more exactly the material's mechanical properties and to assess anticipated ground motions in more detail in the future at the site.

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