# Seismic Design and Construction of a Traditional Timber-Made Five-Storied Pagoda by Applying Coupled Vibration Control

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**SUMMARY**
Traditional Timber five-storied pagodas in Japan have been structurally survived against a number of earthquakes during the long history of Japan. A traditional timber pagoda in Japan consists of a timber frame structure and a central mast. The decorative pole being upper part of the central mast is penetrating through the top roof is independent from the frame structure. Although those five-storied pagodas have well performed during the historical earthquakes, the decorative poles were damaged by the extremely strong ground motions. In order to protect the decorative pole against earthquakes, technology of coupled vibration control was introduced to structurally design a new timber five-storied pagoda. This anti-seismic designing concept was realized by connecting the central mast to the main frame structure with the viscous damper. Both two-dimensional and three-dimensional frame models were successfully employed to conduct the anti-seismic design. As results of the dynamic non-linear response analyses, the response of the decorative pole was reduced by 30% by employment of the coupled vibration control technique, demonstrating that it would be effective in protection of decorative pole during large earthquakes. After construction was completed, microtremore measurement and earthquake monitoring at the site were conducted to verify the analysis model, as well as, to examine the effect of the damper installed between the central mast and the main frame.

*Keywords: Traditional Timber Structure, Five-storied Pagoda, Coupled Vibration Control, Structural design*

## 1. INTRODUCTION

Traditional timber give-storied pagodas have survived against a number of large earthquakes during the long history of Japan. A traditional timber five-storied pagoda in Japan consists of a main frame and a central mast. It should be pointed out that the central mast is structurally independent from the main frame to prevent from damage due to the vertical deformation of the main frame from a structural point of view. Such vertical deformation is usually caused by so-called creep phenomenon of wood during long time. Upper part of a central mast is penetrating through its top roof, being called decorative pole. Although those five-storied pagodas have survived against historical earthquakes, the decorative poles on the top roof were often broken by flexural moment induced by historical both large earthquakes and

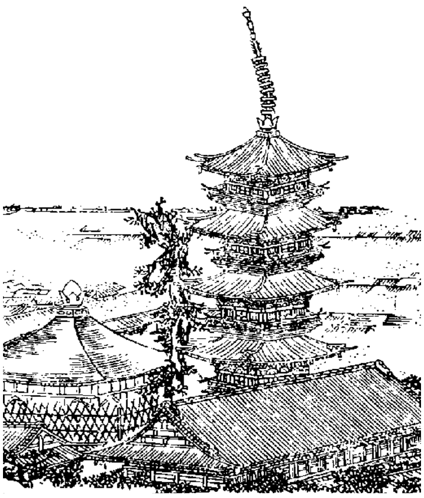


Fig.1 Damage of decorative pole of five-storied pagoda in Tokyo by Great Ansei-Edo Earthquake of 1854 (after F. Ohmori, 1921)

heavy typhoons (F. Ohmori, 1921, See Fig.1). Respecting that the central mast was originally the most important member for religious reason, we developed a new technology to ensure the seismic and wind safety of the central mast. By connecting the central mast to the main frame with the device using viscous damper, the dynamic response of the decorative pole would be reduced by coupled vibration control effect. This structural designing concept can be regarded as collaboration of traditional and modern technologies. As described above, in order to mitigate seismic damage to the decorative pole, coupled vibration control technology was successfully introduced to design, as well as, to construct a new timber five-storied pagoda in Fukuoka, Japan. The central mast was also strengthened by carbon fibre bands at the connection between the central mast and the main frame. The effectiveness of such strengthening method proposed by the authors(Ayaki, D. et al.,2010) was examined by the static loading tests of the full scale diameter model, as shown in Photo.1. Those tests demonstrated that the proposed strengthening method would improve the flexural behaviours caused by earthquakes and typhoons. In the present paper, of those new technologies proposed in this practical project, the coupled vibration control concept is focused on from an earthquake engineering point of view.

Photo.1 Static flexural test of the central mast model strengthened by carbon fiber bands

## 2. OUTLINES OF BUILDING

A new five-storied timber pagoda of Tocho-ji Temple, in Fukuoka, was structurally designed to verify seismic and wind safety. The construction work of this Japanese traditional architecture was completed in May 2011. Fig.2 and Photo.2 show its elevational section and the overview of the pagoda, respectively. The outline of the present pagoda is as followings. The height of the pagoda including the decorative pole is 25.9m, as well as, the dimension of the square at the first story is 4.33m.Such dimension indicates that this pagoda is categorized into small ones. The wood material

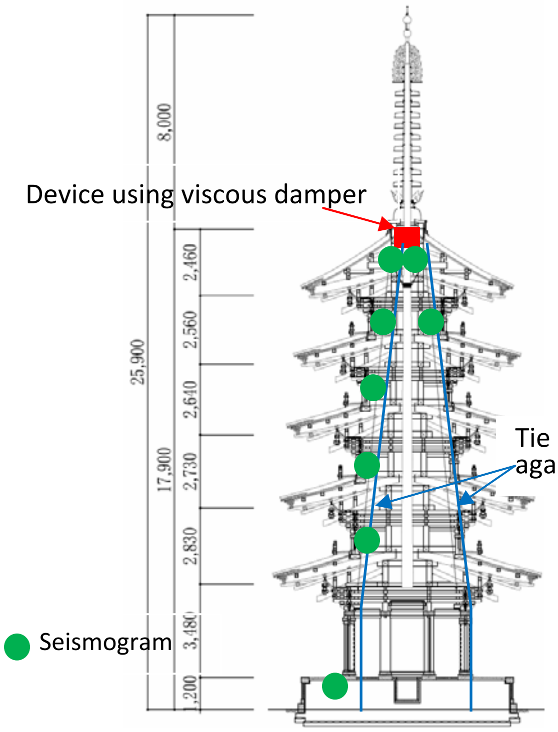


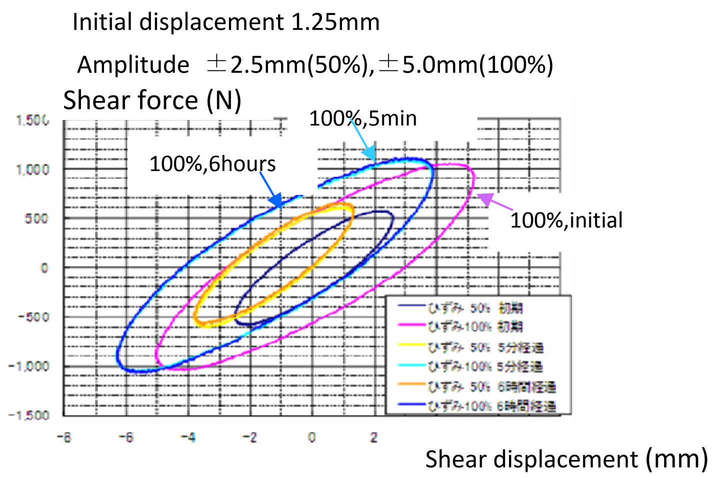

Fig.2 Elevational section of the pagoda with device, wind strengthening and seismograms

Photo.2 Overview of the Five-Storied Pagoda of Tocho-ji Temple after completion

used was Japanese cypress tree. Tiles were used for roofing. A total weight of the super-structure was estimated to be 1,300kN. The massive base of rein-forced concrete, was supported by pile foundation. Horizontal resistance needed for seismic and wind safety is mainly produced by the rigid frame structure characterized by semi-rigidity at joints between penetrating beams and columns (T. Hanazato et al. 2004). Such semi-rigidity performance at the joints would ensure the deformability of the structure. The wooden shear walls are also installed as important structural elements to resist horizontal loads. A number of bracket complexes that are distributed sufficiently at each layer contribute to energy absorption during the dynamic response at earthquakes and typhoons. Regarding wind safety, a stainless steel tie-bar was installed against uplift of the column at each corner since such high-rise timber structure is severely affected by strong wind(See Fig.2). This wind resistant technique has been employed in recent years to construct new timber pagodas in Japan (T. Hanazato et al.,2004).

## 3. DEVICE DEVELOPED FOR PRESENT PROJECT

To protect the central mast from the damage caused by earthquakes and typhoons, a new technology of coupled vibration control system was introduced and developed in the present study. By connecting the central mast to the main frame with the viscous damper, it was expected that the response of the upper part of the central mast, the decorative pole, would be reduced. The device was planed to install at the upper part of the central mast, shown in Fig.2. Photo.3 shows the device using viscous damper, developed in the present practical study. For such traditional timber structures, initial displacement is often caused during construction works of wood, although it is only within a few millimeter. Cyclic loading test was conducted to check the performance of the device when the initial displacement was given. Fig. 3 shows the hysteresis curves obtained by the cyclic test, indicating the initial displacement would not significantly affect its performance as damper. The parameter of the viscous damper was determined on the basis of the dynamic response analysis described in the next section. Fig.4 and Photo.4 show the plan of installation of the device and the practice of the installation, respectively.



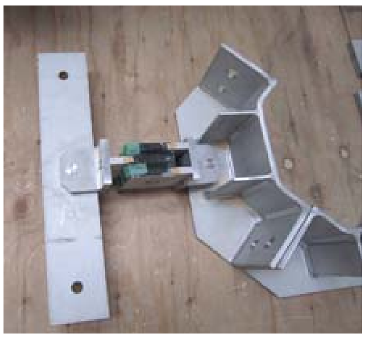



Photo.3 Device for installation

Fig.3 Hysteresis loops by cyclic test of the device under initial displacement

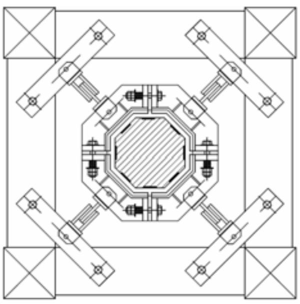


Fig.4 Plan of installation of device to the wooden frame

Photo.4 installation of device

## 4. SEISMIC STRUCTURAL ANALYSIS

### 4.1 ANALYSIS MODELS

*4.1.1 General*

In the present practical study, non-linear dynamic response analyses utilizing 2 numerical models were conducted. One was 2-dimensional frame model that was already introduced in the past study (T. Hanazato et al.,2004), and the other was 3-dimensional model that was developed in the present project. By employing 3-dimensional model, torsional mode was taken into account. As described in the previous section, the mechanical condition of the joints of the frame is a key factor affecting the seismic resistance. In the present analyses, semi-rigidity model was introduced to express the relationship between flexural moment and rotational angle at the joints of the frame (T. Hanazato et al.,2004). As well as, .rigid model at the joints was also assumed to simulate the microtremor records. As the material property, stiffness of wood was evaluated from the laboratory tests ; the flexural test of the beam and the elastic wave test.

*4.1.1 2-dimensional frame model*

As described in Fig. 4, the main frame was composed of the exterior square formed by “Gawa-bashira” (outside columns, b-b section) and of the interior square formed by “Shiten-bashira” (inside columns, a-a section). Since the horizontal in-plane rigidity of the frame at each floor was evaluated to be sufficiently stiff owing to the Japanese traditional joint, the 2-dimensional analysis model was able to be employed by assumption that the inner structural face was connected with rigid member to the outer structural face at each floor level. At the joint of a penetrating beam “Nuki” and a column, the semi-rigidity model was employed, shown in Fig.5, to express the non-linear behaviours due to embedding effect of the column to the beam. This figure also shows the bi-liner slip-type hysteresis model. Such elasto-plastic model of the semi-rigidity at the joints of the frame must provide one of the main anti-seismic properties of Japanese traditional timber buildings. In general, wood has inherently high anisotropy in the mechanical properties, therefore, embedding effect is caused by such anisotropic properties. Another anti-seismic element was a timber shear wall. Fig. 6 shows horizontal load–displacement characteristics presented in the textbook of structural design for traditional timber houses in Japan (Japan Housing and Wood technology Center 2008), describing the analysis model. In the present analysis, elasto-plastic relationship was idealized on the basis of this experimental result. The semi-theoretical equation of the mechanical model to take account of embedding effect was used to evaluate the load-displacement of the shear walls called “Otoshi-komi-itakabe” . This type of traditional timber wall was constructed by inserting the wooden board elements to the surrounding frame. The bracket complex “Masugumi”, the most specific element to the traditional timber structure, was idealized by an equivalent beam element. For modelling of the beam element, it was assumed that rotational deformation was caused by embedding effect at the upper and the lower boundaries of the element. This embedding effect at the bracket complex means that the wooden brocks “Masu” were embedding into the horizontal beams. Furthermore, it was also assumed for the modelling that the wooden block behaved as an elastic body.

Finally, the damper was modelled by using both damping element and shear spring. The parameters of the damping effect were evaluated at the natural frequency of $1^{st}$ mode, 0.8Hz. Fig.8 shows the relationship between damping force and the velocity at frequency of 0.8Hz.

*4.1.2 3-dimensuional frame model*

Figs. 9 and 10 shows 3-dimensional frame model and distribution of mass at the roof frame, respectively. Non-linear mechanical model of the joints, the shear walls, and the bracket complex are evaluated by the same procedures as those introduced in 2-dimensional frame model. Furthermore, the parameters of damping element to express the damper to connect the central column and the frame were the same ones as those of 2-dimensional frame model.

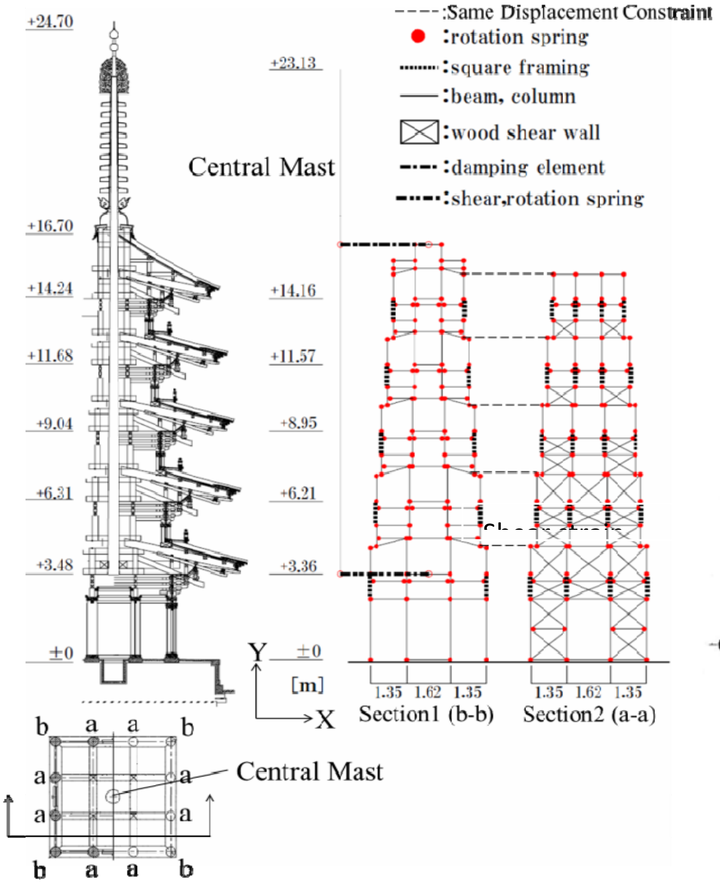



Fig.5 2-dimentional analysis model

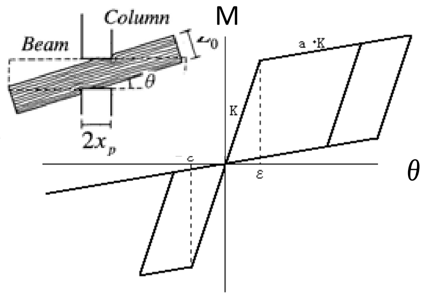
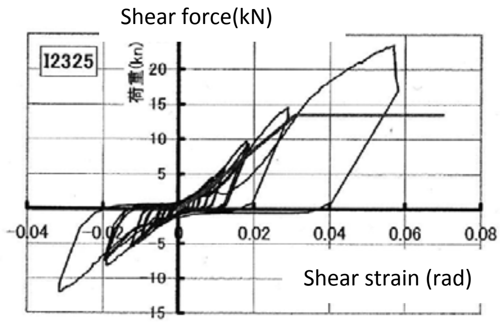



Fig.6 semi-rigidity model of joint of penetrating beam and column




after Japan Housing and Wood Center(2008)

Fig.7 Experimental relationship of shear force and shear strain angle compared with analysis

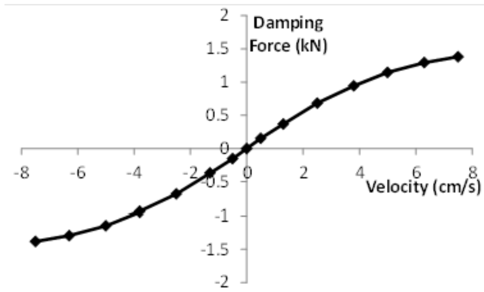



Fig.8 Performance of damper at calculated natural frequency of the structure, 0.8Hz

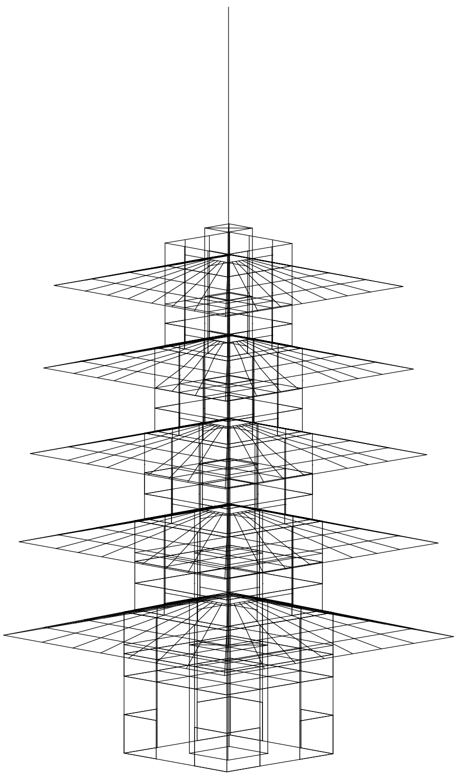


Fig.9 3-dimensional frame model

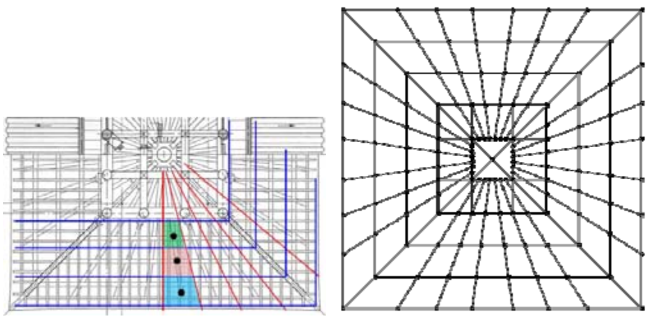


Fig.10 distribution of mass at roof frame

### 4.2 EINGENVALUE ANALYSES

For such analysis models, it is essential to perform the eigenvalue analysis from an earthquake engineering point of view. In particular, since the microtremor measurements were carried out to study the fundamental dynamic characteristics of the structure, it was worth while comparing the analysis model with the measurements, To compare with the microtremor measurements described in Section 4, eigenvalue analysis was conducted under the condition that the joints were rigid without embedment effect causing semi-rigid joint. This assumption was made in consideration that deformation during microtremor was so small that embedding phenomenon might not occur. Table 1 and Table 2 show the natural frequencies evaluated from 2-dimensional and 3-dimensional analysis models, respectively.

Table 3 also shows the natural frequencies of 3-dimensional model of which joints are semi-rigid as the embedding effect affects the dynamic behaviours. Note that the elastic modulus to be used for semi-rigid joint was evaluated at the deformation level (rotational angle) of 1/200. These results of eigenvalue analysis are discussed in Section 4.

Table 1 Natural frequencies of 2-dimensional model of which joints are rigid

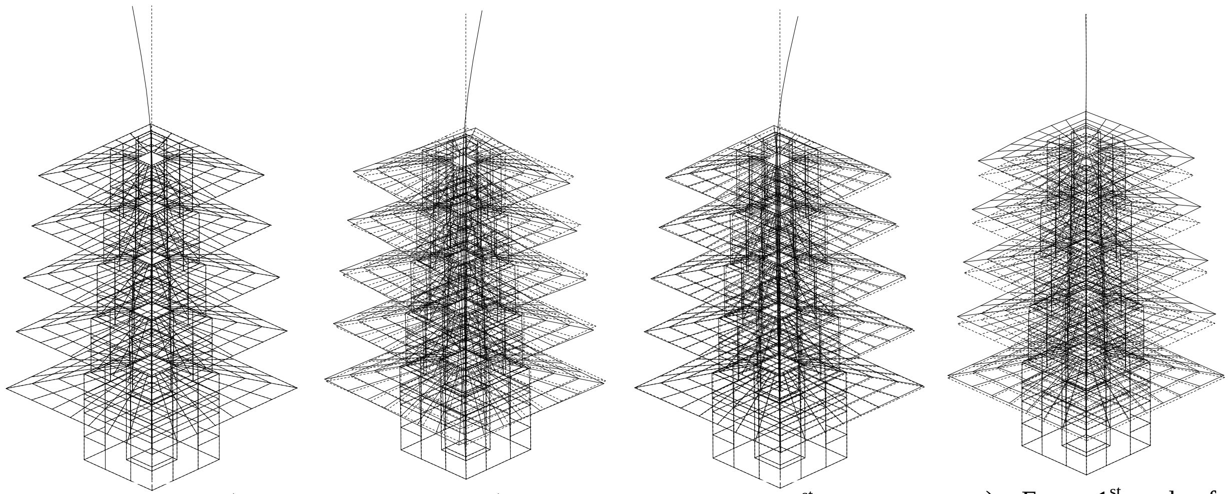
| Natural frequency (Hz) | Effective mass/Total mass | Mode |
|---|---|---|
| 0.89 | 0.02 | Central mast |
| 1.40 | 0.90 | Translational mode of frame 1st<br>Central mast 2nd |
| 3.77 | 0.02 | Centaral mast 3rd |
| 3.81 | 0.06 | Translational mode of frame 2nd |
| 5.83 | 0.00 | Central mast 4th |
| 7.06 | 0.01 | Translational mode of frame 3rd |

Table 2 Natural frequencies of 3-dimensional model of which joints are rigid

| Natural frequency (Hz) | Effective mass/Total mass | Mode |
|---|---|---|
| 0.93 | 0.04 | Central mast 1st |
| 0.93 | 0.04 | Central mast 1st |
| 1.11 | 0.10 | Translational mode of frame 1st |
| 1.12 | 0.19 | Translational mode of frame 1st |
| 1.12 | 0.23 | Torsional mode 1st |
| 2.97 | 0.03 | Translational mode of frame 2nd |
| 2.98 | 0.03 | Translational mode of frame 2nd |
| 8.99 | 0.28 | Vertical mode |

Table 3 Natural frequencies of 3-dimensional model of which joints are semi-rigid

| Natural frequency (Hz) | Effective mass/Total mass | Mode |
|---|---|---|
| 0.76 | 0.22 | Translational mode of frame 1st<br>Central mast 1st |
| 0.77 | 0.25 | Translational mode of frame 1st<br>Central mast 1st |
| 0.79 | 0.05 | Torsional mode of frame |
| 0.93 | 0.03 | Central mast 2nd |
| 0.93 | 0.03 | Central mast 2nd |
| 1.72 | 0.00 | Torsional mode 2nd |
| 2.10 | 0.04 | Translational mode of frame 2nd |
| 2.12 | 0.04 | Translational mode of frame 2nd |
| 8.71 | 0.18 | Vertical |
| 8.73 | 0.08 | Vertical |



d) Central mast 1st mode 0.93Hz

b) Frame 1st mode of Translational 1.11Hz

a) Frame 1st mode of Torsional 1.12Hz

c) Frame 1st mode of Vertical 8.99Hz

Fig.11 Vibration modes of 3-dimensional model (See Table 2)

## 4.3 SEISMIC RESPONSE ANALYSES

*4.3.1 Input ground motions*

To ensure seismic safety of the structure, a total of 3 ground motions at extremely strong level(so called Level-2 which is required for human safety) , and the ground motions by the Fukuoka-ken Seiho-oki Earthquake of March 20 2005 were simulated and utilized, shown in Table 4.The procedure for simulation are summarized as follows ;

1) Earthquake motions were simulated at the engineering bedrock to fit theresponse spectra with the target spectra at Level-2 given by Japan Building Code. To synthesize the motions, the phases of the actual earthquake records were utilized ; JMA Kobe 1995 NS, Hachinohe 1986 NS, and Fukuoka-ken Seiho-oki Earthquake of 2005. (Input motions No.2,Nos,No.3, respectively)
2) Soil response analysis utilizing one-dimensional model of shear wave propagation was performed to simulate the input ground motions at the ground surface. In the soil response analysis, strain-dependency of soil stiffness and material damping were considered by using equivalent linear technique.
3) Fukuoka-ken Seiho Oki Earthquake of March 20,1995 ($M_{JMA}$=7.0)was the most severe earthquake that has hit the site during the last one hundred years. This near-field earthquake affected Fukuoka where the pagoda was constructed. Although the ground motions were not recorded at the site, it would be significant to simulate the ground motions at the site for the response analysis. As well as the prescribed procedure, the earthquake motions at the surface of the engineering bed rock were simulated from the earthquake record at JMA Fukuoka. In this analysis, soil response analysis using one-dimensional shear wave propagation theory was also applied. The ground motions at the Pagoda's site were simulated in consideration of soil response of the surface layer at the construction site, as the soil conditions at the site is different from those at JMA Fukuoka where the earthquake motions were recorded.(Input motion No.4)

Table 4 Input ground motions and numbering

| Number | Target Spectra | Phase | PGA(cm/s2) | PGV(cm/s) | PGD(cm) |
|---|---|---|---|---|---|
| 1 | Japan Building Code, Level-2 | JMA Kobe 1995 | 396 | 53 | 47 |
| 2 | | Hchinohe NS 1986 | 345 | 53 | 33 |
| 3 | | Fukuoka NS 2005 | 350 | 79 | 50 |
| 4 | JMA Fukuoka 2005 NS | | 263 | 65 | 18 |

*4.3.2 Static step-by-step analysis*

Fig. 12 shows the relationship between the base shear and the horizontal displacement at the fifth story, comparing the 2-dimensional and 3-dimensional analyses. In these analysis, the distribution of the horizontal force along the height was evaluated from the seismic force coefficient distribution given by Japan Building Code. As shown in Fig.12, the base shear corresponding to the ultimate lateral strength of the structure was evaluated to be approximately 0.5, of which story drift was 1/28. In the present structural design, the safety limit was given by the story drift angle of 1/30. Regarding the analysis result of 3-dimensional model, it can be noticed that beyond the safety limit displacement, the shear force continue to increase. Such characteristics of the force-displacement relation obtained by 3-dimensional analysis was due to the ductility of the wooden shear wall presented in Fig.7.

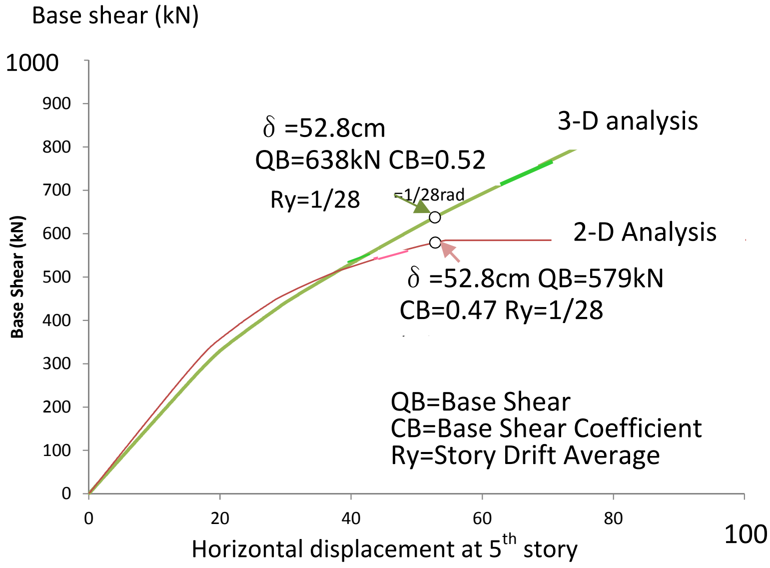



Fig.12 Relationship between base shear and horizontal displacement at 5th story ; ultimate lateral strength

*4.3.3 Seismic response analysis*

Seismic response analyses by employing 2-dimensional and 3-dimensional models were successfully conducted for the input ground motions described in 3.3.1. They were performed with Rayleigh damping of which factor was 5% for 1st and 2nd modes. Stiffness and damping coefficient of the damper was evaluated through the preliminary parametric analysis. The most appropriate parameters were evaluated as K=0.147kN/cm and C=0.3kNs/cm. In the present paper, the analysis results by the 3-dimensional model using such parameters are shown. As mentioned in 1. INTRODUCTION, the purpose of the coupled control vibration system introduced was to reduce the dynamic response of the central mast, in particular, the decorative pole that is its upper part. Therefore, the story drift angle of the central mast was focused on and described in the present paper. Fig.13 shows the resulted peak drift angle of central mast, comparing the response with and without vibration control device. In Fig.13, case 1 denotes the response without the damper, on the other hand, case 2 denotes the response of the model with the damper installed for coupled vibration control. It should be noticed that the response of the central mast with damper (Case 2) was rather reduced in comparison with that without damper (Case 1). Table 5 summarizes the effectiveness of the damper in reduction of the deformation of the central mast, indicating that good performance would be made against such extremely strong ground motions. By installing the damper, the peak story drift angle at the top of the central mast was reduced by 34% on average.

Table 5 Peak drift angle of central mast to show effectiveness of damper

| Ground motion (See Table 4) | Peak story drift angle (rad) | | Reduction (%) |
|---|---|---|---|
| | Case 1 (without damper) | Case 2 (with damper) | |
| No.1 | 0.226 | 0.147 | 35 |
| No.2 | 0.240 | 0.128 | 46 |
| No.3 | 0.234 | 0.154 | 34 |
| No.4 | 0.255 | 0.199 | 22 |

## 5. MONITORING

### 5.1 Microtremore measurements

After completion of the five-storied pagoda, the miccrotremore measurements and free vibration tests

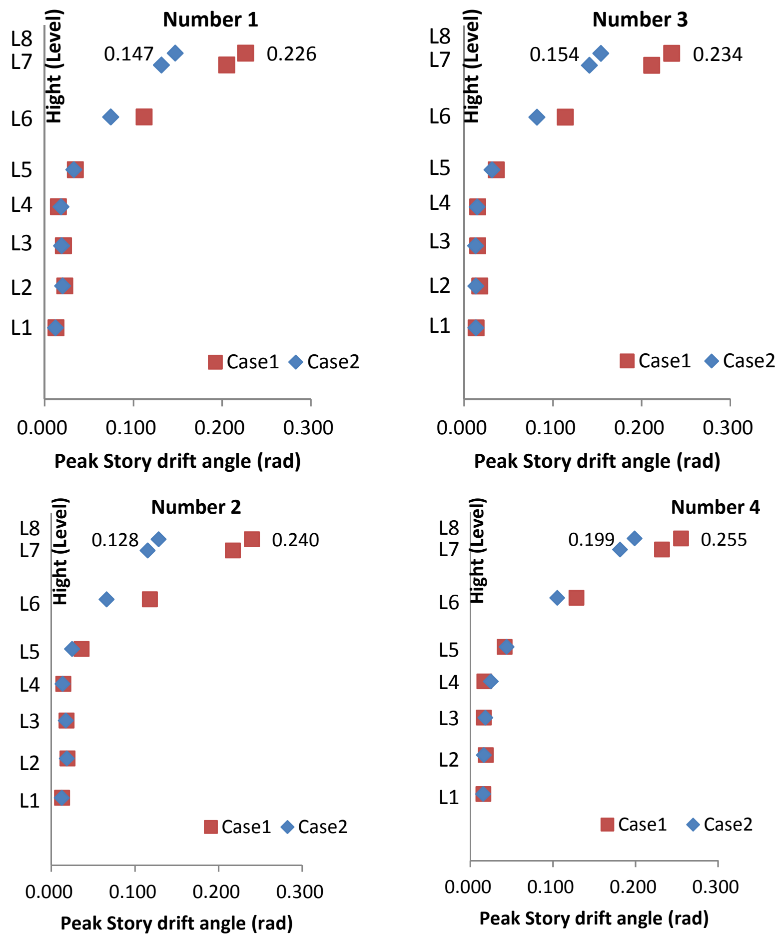



Fig.13 Peak drift angle of central mast ; comparison of the response with and without vibration control device

were conducted to investigate the fundamental dynamic characteristics of this new-built structure, as well as, to verify the analyses models. Table 6 summarizes the natural frequency and the damping factor obtained from those measurements. The mictotremore measurements shown in Table 6 indicated that natural frequency measured with mode is well consistent with the 3-dimensional frame model without embedment effect at joints, shown in table 2. Good comparison can be found in not only translational but also torsional and vertical vibration modes. This comparison was reasonable because embedment phenomena at joints would not appear at such quite small strain level of microtremor. As mentioned above, the employed 3-dimensional analysis model was verified by the microtremor measurements that might not cause embedding effect at joints. It is well known that, in general, the natural period is correlated with the height of buildings. Fig.14 shows the relationship between the height of the main frame of the five-storied pagoda and the natural periods of 1$^{st}$ and 2$^{nd}$ modes. In addition to the natural periods obtained by the past studies, the measurement results of the present five-storied pagoda were plotted in this figure. This figure shows that the natural period of the

Table 6 Natural frequency and damping factor evaluated by measurements of microtremor and free vibration

| | Microtremor | | | Free vibration | | | | Micro-tremor |
|---|---|---|---|---|---|---|---|---|
| | Translational mode | | | Translational mode | | Torsional mode | | Vertical mode |
| Direction | 1$^{st}$ mode | 2$^{nd}$ mode | 3$^{rd}$ mode | 1$^{st}$ mode | Damping factor | 1$^{st}$ mode | 2$^{nd}$ mode | |
| NS | 1.12 | 3.34 | 5.69 | 1.08 | 1.6% | 1.49 | 3.63 | 10.50 |
| EW | 1.15 | 3.30 | 5.59 | 1.09 | 1.6% | | | |
| Ave. | 1.14 | 3.32 | 5.64 | 1.09 | 1.6% | 1.49 | 3.63 | 10.50 |

present pagoda is well correlated with the empirical equation of relation between the building height and the natural period.

### 5.2 Earthquake monitoring

In order to examine the effectiveness of the coupled vibration system in reduction of the response of the central mast, as well as, to verify the analysis model at higher strain level, earthquake monitoring was initiated after the completion of the pagoda. Shown in Fig.1, seismograms were installed to record motions during earthquakes at both the main frame from the base to the top and the central mast. Photo 5 shows the seismogram installed at the frame of 2nd story

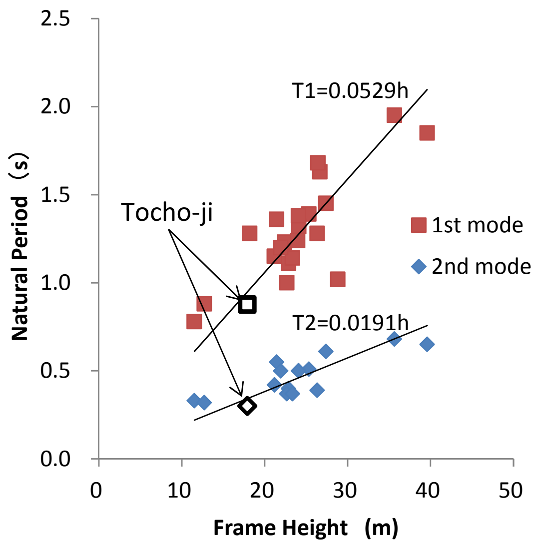



Fig.14 Relationship between frame height and natural period by microtremore measurements

## 6. CONCLUDING REMARKS

Photo 5 Seismogram installed on the beam at 2nd story

A recent technology of the coupled vibration control system was successfully introduced to reduce dynamic response of the central mast of the new-built traditional timber five-storied pagoda to earthquakes and typhoons. For its purpose, a new device with damper to connect the central mast and the frame was developed and practically installed. Both 2-dimensional and 3-dimensional non-linear analysis models were employed to conduct seismic response analyses. The eigenvalue analysis of 3-dimensional model was in good agreement with the microtremore measurements. Seismograms were also installed to verify both the device's effectiveness and the analysis model at larger strain level.

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