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A STUDY ON SEISMIC PERFORMANCE AND SEISMIC DIAGNOSIS, SEISMIC RETROFIT OF JAPANESE TEMPLE

Shunsuke MIYAMOTO[1], Kenji MIYAZAWA[2], Yasutaka IRIE[3], Juwei WU[4], Yoshinori NOMATA[5], Osamu GOTO[6]

SUMMARY

Structural style of Japanese temple building is peculiar and different from a wooden dwelling house. This research aims to clarify seismic performance of Japanese temple building by field investigation, seismic diagnosis and seismic retrofit. The actual seismic retrofit construction of the existing temple is investigated for this aim, and the effect of seismic retrofit are confirmed from the vibration measurement after seismic retrofit. This result, roof is large and occupies the great portion of weight. The measured natural period of Japanese temple building was about 2[Hz]. It is long period as compared with a wooden construction house, because it is said that the natural period of wooden dwelling houses is 5-6[Hz]. In order to confirm the effect of seismic retrofit, we carried out the microtremor measurements before and after the seismic retrofit. The main results are following; the natural frequency of this temple has increased from 2.0[Hz] to 3.6[Hz]. The damping constant has decreased from 3.5[%] to 2.7[%]. Putting in diagonal steel braces in the horizontal ceiling structure, the effect in reduction of the torsional deformation was confirmed.

INTRODUCTION

Japanese temple building is timber structure. It has long history. Structural style of Japanese temple building is peculiar. And it has experienced many earthquakes. This research clarifies seismic performance of Japanese temple building which exists really by field investigation, seismic diagnosis and seismic retrofit. The investigated temple building is Rensho-ji which is really existing at Fujieda-city in Shizuoka-prefecture. And the vibration characteristic of the temple building comprehends by microtremor measurement. Moreover, it shows the actual example of seismic retrofit construction, and we propose effective seismic retrofit in earthquake load.

[1] Graduate School of Eng., Kogakuin Univ., Ms. Eng.
[2] Prof., Department of Architecture, Kogakuin Univ., Dr. Eng.
[3] Assoc. Prof., Department of Information and Control Systems Science, Graduate School of Eng., Utsunomiya Univ., Dr. Eng.
[4] Fujitsu FIP Corporation, Ms. Eng.
[5] Technical Official, Department of Eng., Utsunomiya Univ.
[6] Assoc. Prof., Department of Design in Architecture, Kogakuin Univ., Dr. Eng.

STRUCTURAL OF JAPANESE TENPLE BUILDING

Structural style

Rensho-ji of target temple building is in the southern part of Fujieda-city in Shizuoka- prefecture. And it is the Jodo-Shinshu-Otani group's temple building. The Rensho-ji Hondo as of now will be built in 1913. A panorama of temple building is shown in Photo.1 and a ground plan is shown in Fig. 1. Japanese temples building such as Rensho-ji are architectural structure with the very large weight of a roof truss. Therefore, roof weight occupies most gross weight, and it is influenced seismic performance greatly. Moreover, since Japanese temple building has little bearing wall to earthquake load, it is thought that seismic performance is low. However, Japanese temple building has seismic elements, such as frame and rail, wall upper than picture rail.

Photo.1 Panorama of Rensho-ji Hondo

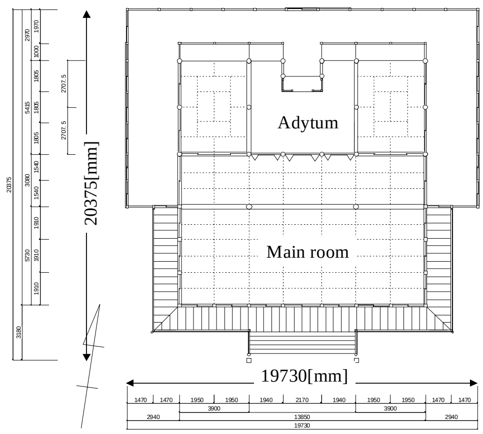


Fig.1 Ground plan of Rensho-ji Hondo

Weight

The gross weight of Japanese temple building is greatly influenced by the kind of roof. Some typical roof classification[1)] of Japanese temple building and roof weight of the temple building which exists really is shown in Table.1. Weight of roof tile roofing is heavy in roof classification, and Hongawara roofing is as the heaviest as 250 [kgf/m^2] also in some roof classification. Subsequently, Pantaile roofing is as heavy as 120 [kgf/m^2]. Although the roof tile of the temple building is heavy, clay roofing which superimpose on a roof tile also occupies 30 - 40 percent of roof weight. Fig.2 shows the roof area and unit roof weight of roofing. Although clay roofing of Rensho-ji was also heavy, clay roofing was carried away by seismic retrofit construction. As a result, roof weight of before seismic retrofit was shown 1.7 times of roof weight of after seismic retrofit.

Table.1 Roof weight of the temple building

№	Name	Construction data	The kind of roof	Roof Area	Roof Weight	Unit roof weight
		[year]		[m^2]	[kN]	[kN/m^2]
①	Jizoubu-ji Hondo	～1513	Hongawara roofing	192	48	0.250
②	Joutok-ji Entuden	1401	Hongawara roofing	151	38	0.250
③	Kyutomyo-ji Hondo	1393～1453	Hongawara roofing	519	130	0.250
④	Kansin-ji Kondo	1346～1370	Hongawara roofing	790	197	0.250
⑤	Joshin-ji Hondo	1243～1247	Hongawara roofing	452	43	0.094
⑥	Chikurin-ji Hondo	1511	Wood shingle roofing	274	5	0.019
⑦	Shokai-ji Hondo	1648～1652	Wood shingle roofing	230	8	0.033
⑧	Kozo-ji Hondo	1591	Wood shingle roofing	158	3	0.019
⑨	Horin-ji Hondo	1667	Wood shingle roofing	570	14	0.024
⑩	Kiyonizu-dera Hondo	1390～1394	Wood shingle roofing	914	23	0.025
⑪	Senpuku-ji Yakushido	1513～1573	need roofing	137	12	0.085
⑫	Tenjuin	1651	need roofing	140	10	0.072
⑬	Shoren-ji Amidado	1542	need roofing	372	25	0.067
⑭	Kuwanomi-dera Hondo	1393～1453	roofing of cypress bark shingles	472	18	0.039
⑮	Kongosho-ji Hondo	1610	roofing of cypress bark shingles	661	22	0.033
⑯	Renshoji Hondo	1913	Pantile roofing	600	124	0.207
		2003		600	72	0.120

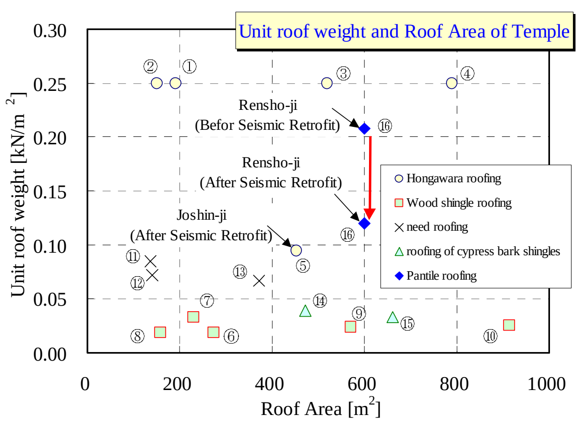


Fig.2 Roof area and unit roof weight

Vibration characteristic

Also structural and building construction Japanese temple building differ from a wooden house. Therefore, vibration characteristic of temple building, traditional wooden house and wooden house is considered by microtremor measurement. The result of microtremor measurement has draw anamnestic research[2-29]. Fig.3 shows natural frequency of ridge direction and span direction. Fig.4 shows damping factor of ridge direction and span direction. Natural frequency of Japanese temple building is 1.5[Hz] to 2.7[Hz]. Natural frequency is small in order of temple building- traditional wooden house -wooden house. Damping factor of a temple building is concentrated on 4[%] from 3[%]. And damping factor of a wooden house is concentrated on 2[%] from 1.5[%]. Therefore, deformation performance of temple building can say that it is high. Fig5 shows natural frequency and damping factor of each building. In ridge direction of the temple building, if natural frequency becomes large, damping factor will become small. In span direction of temple building, if natural frequency becomes large, damping factor will become large. However, vibration characteristic of the traditional wooden house showed the characteristic contrary to temple building. Moreover, natural frequency and damping factor of wooden house has an inverse proportion relation of the temple building.

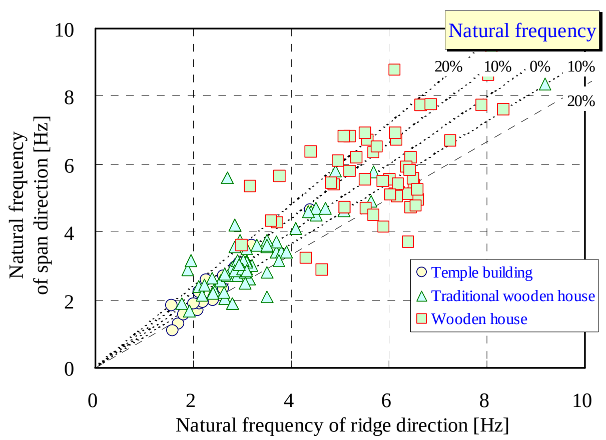


Fig.3 Natural frequency

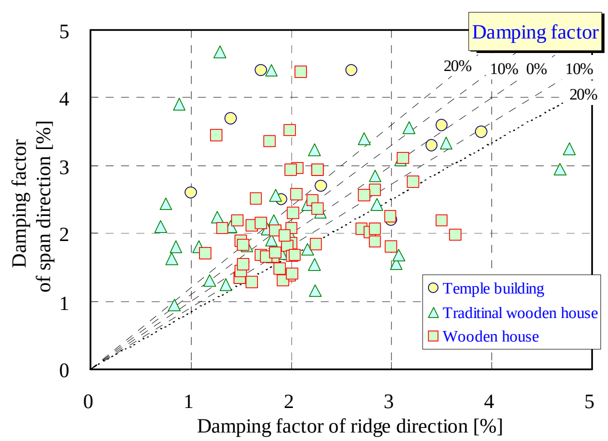
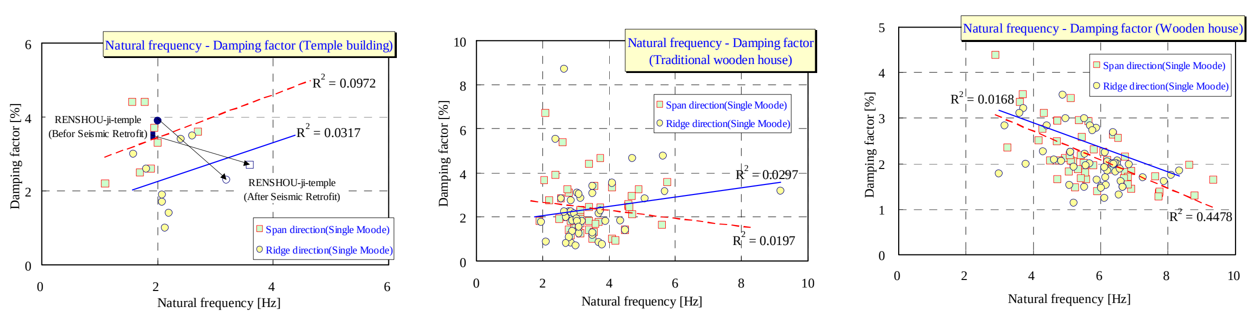


Fig.4 Damping factor



Japanese temple building Traditional wooden house Wooden dwelling house

Fig.5 Natural frequency and damping factor of each building

SEISMIC DIAGNOSIS

Outline of seismic diagnosis

The method of seismic performance evaluation of the wooden building generally used is dependent on wooden brace or bearing wall. This method is laid emphasis on strength. Therefore, it is thought that this method of seismic performance evaluation cannot evaluate appropriately seismic performance of temple building which hardly has wooden brace or bearing wall. However, temple building also has many earthquake resisting elements which resist shear force, such as mud wall, compressive strain inclined to the grain by beam-column(Nuki) joint, tenacity of bracket complex(Kumimono) and restoring force to column rocking produced by a thick column and roof weight. Therefore, in order to evaluate rationally a temple building with the earthquake resisting element which cannot be evaluated only by wall length ratio, it is necessary to decide the deformation domain of a wooden building and to evaluate the response in case of an earthquake quantitatively. This diagnostic method evaluates the limit state where damage as the story drift 1/120[rad.] of a building, and it evaluates shear force of building. Moreover, it reduces strength in order to take eccentricity of a building into consideration. Evaluation of eccentricity not only takes balance of alignment of a wall into consideration, but also can evaluate that it is an irregular shape in three dimensions.

Earthquake resisting element

The evaluation method of mud wall has referred anamnestic research[30), 31), 32)]. Fig.6 shows structural model of mud wall. The evaluation method of column with Kokabe frame has adopted the one where the shear strength of story drift 120[rad.] and bending strength of column is smaller. The evaluation method of mud wall is the same as that of the evaluation method shown in Fig.6. The column was calculated by assumption of cantilever beam. This outline is shown in Fig.7. Moreover, the evaluation method of beam-column(Nuki) joint, bracket complex(Kumimono) and restoring force to column rocking has draw reference[33), 34), 35), 36)]. The each earthquake resisting elements is shown Fig.8, Fig.9, and Fig.10.

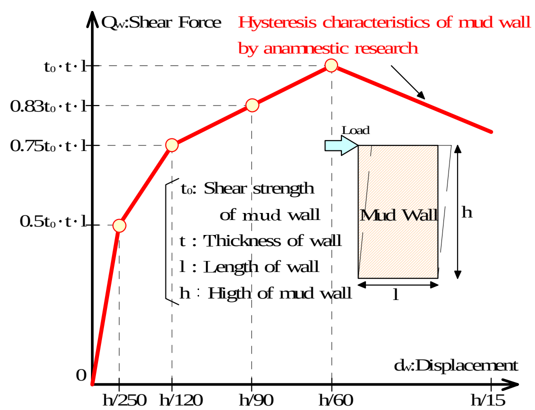


Fig.6 Structural model of mud wall

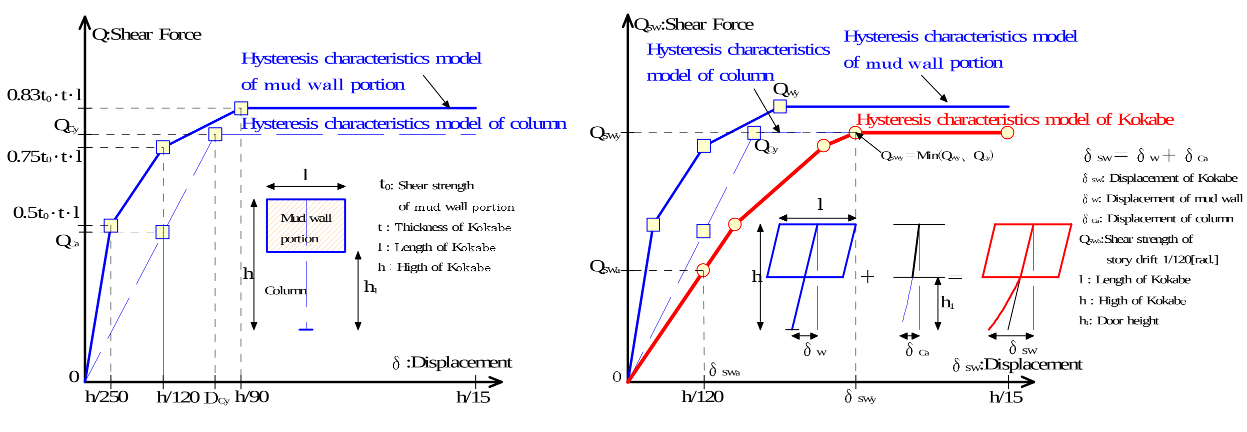


Fig.7 Structural model of column with Kokabe

Equation (1) shows the calculation method of Kokabe.

$$\delta_W + \delta_C = \delta$$

$$\frac{P \cdot h}{G \cdot l \cdot t} + \frac{P \cdot h_1^3}{3 \cdot E \cdot I} = \delta \qquad (1)$$

G: Shear modulus [N/mm^2]
E: Young modulus of a column [N/mm^2]
l : Length of wall to pay [mm]
f_b: Allowable bending stress [mm]
t : Thickness of mud wall [mm]

Equation (2) shows shear force of a column determined by deformation.

$$P = \frac{3 \cdot E \cdot I \cdot G \cdot l \cdot t}{G \cdot l \cdot t \cdot h_1^3 + 3 \cdot E \cdot I \cdot h} \qquad (2)$$

Equation (3) shows shear force determined by bending strength of a column

$$P_{cr} = \frac{f_b \cdot Z}{h_1} \qquad (3)$$

${}_CQ_a$: Allowable shear force of a column
${}_CQ_a$=Min (P, P_{cr})

The calculation method of beam-column(Nuki) joint shows below.

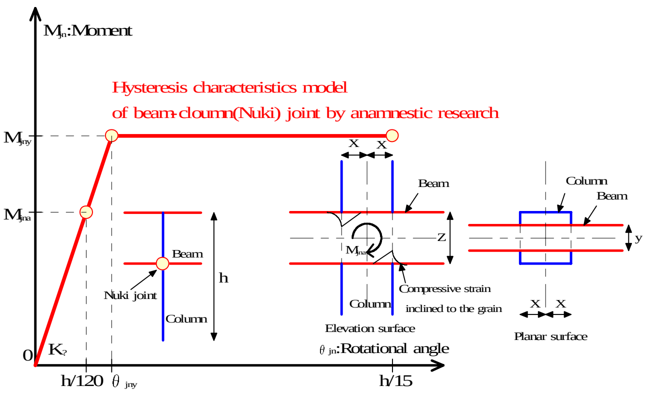


Fig.8 Structural model of beam-column(Nuki) joint

Rotational rigidity: K_θ [N・cm/rad.]

$$K_\theta = x^2 \cdot y \cdot E_C \{ \frac{x}{Z} (1 + \frac{4Z}{3x} - \frac{1}{3}) \} \qquad (4)$$

Allowable moment: M_{jna}[N・cm]

$$M_{jna} = K_\theta \cdot \frac{1}{120} \qquad (5)$$

The calculation method of bracket complex(Kumimono) and restoring force to column rocking shows below.

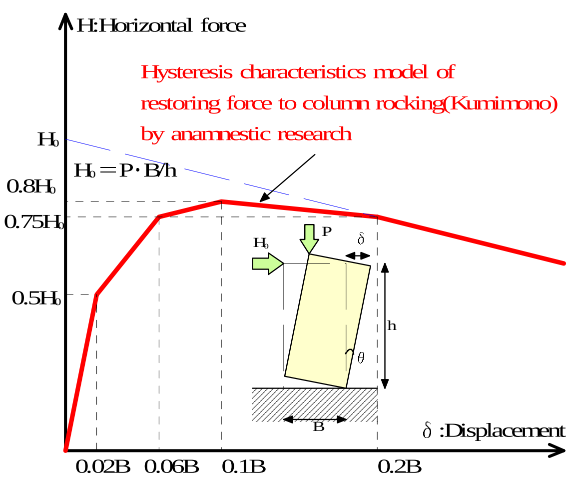


Fig.9 The restoring force to column rocking

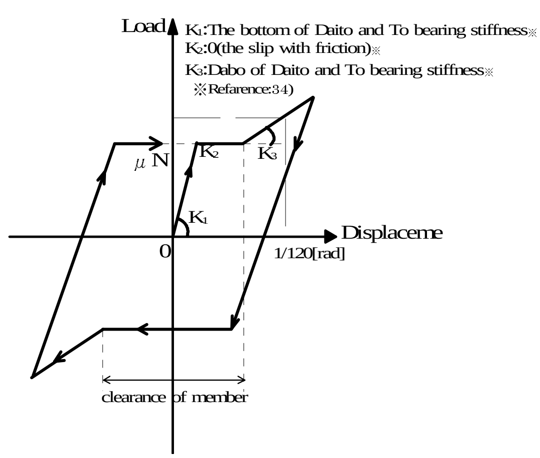


Fig.10 The bracket complex(Kumimono)

Result of seismic diagnosis

We calculated the strength of span and ridge direction at the time of story drift (1/120[rad.]) by the evaluation method of each earthquake resisting elements. The story shear force of span direction is 305[kN], and ridge direction is 93[kN]. However, when eccentricity of the building by arrangement of a wall or a column was taken into consideration, the story shear force of only the span direction showed 222[kN]. Table.2 shows the seismic diagnosis result before seismic retrofit. Table.3 shows calculation of earthquake load before seismic retrofit. The gross weight before seismic retrofit is 1198[kN], and the earthquake load is 287[kN]. Since the strength of each direction has not arrived at safety margin to earthquake load, it is evaluated that there is danger of collapse. And this building needs to reconsider seismic performance. Fig.4 shows calculation of necessary strength.

Table.2 Seismic diagnosis result (Before seismic retrofit)

Retrofit direction	Horizontal Force			Eccentricity ratio	Reduction ratio by Eccentricity	Strength in consideration of Eccentricity	Earthquake load	Strength / Earthquake	Adjudication
	burden of all column	burden of all walls	Sum total						
	[kN]	[kN]	[kN]			[kN]	[kN]		
Span(X) direction	160	145	305	0.26	0.73	222	287	0.77	NG
Ridge(Y) direction	93	0	93	-	-	-	287	0.32	NG

Table.3 Calculation of earthquake load (Before seismic retrofit)

Calculation of earthquake load	Kind of load	Unit weight	Roof Area	Weight	Shera coefficient ※1)	Earthquake load
		[kN/㎡]	[㎡]	[kN]	Ci	[kN]
Fixed load	Roof	By results of an investigation		706	0.24	169
	Roof truss ※2)	0.83	306	253		61
	fittings ※2)	0.78	306	239		57
Sum total				1198		287

(※1)The area coefficient which considered the possibility of the Tokai earthquake by specification of Shizuoka Prefecture=1.2, Ci = Co×Z = 0.2×1.2 = 0.24

(※2)References37) is referred to.

Table.4 Calculation of necessary strength (Before seismic retrofit)

Retrofit direction	Earthquake load	Strength	Necessary Strength	Retorfit wall length	Target Strength
	[kN]	[kN]	[kN]	[m]	[kN/m]
Span(X) direction	287.47	222.46	65.01	16.50	915
Ridge(Y) direction	287.47	93.10	194.37	9.40	3031

SEISMIC DIAGNOSIS AFTER WEIGHT SAVING AND SEISMIC RETROFIT PLANNING

Fig.5 shows result of seismic diagnosis after weight saving of roof, and Fig.6 shows calculation of earthquake load after weight saving of roof. Before seismic retrofit, shortage of bearing wall was conspicuous in span(X) direction and ridge(Y) direction owing to the weight of roof load. However, the gross weight decreased to 878[kN] by weight saving of roof, and earthquake load also became small with 211[kN]. Shortage of bearing wall was conspicuous only in the ridge(Y) direction with weight saving of a roof. Since the necessary strength of before seismic retrofit is 65[kN], it was thought that it was necessary to establish more 16.5[m] bearing wall of 4 times the magnification. However, the wall of span(X) direction brought a result with which safety is filled by weight saving of roof. The seismic retrofit method, which raises seismic performance from result of seismic diagnosis, is proposed.

1) Weight saving of roof, and extension of a ceiling brace.

The intendment fortify of horizontal plane of structure. And abatement of weight decreases the imposition of horizontal load.

2) Extension and earthquake retrofit of the bearing wall

The strength of a building is made to increase.

3) Fixation of a column base

By shear capacity of a wall is strengthened, the tensile force of a column base becomes strong.

Table.5 Seismic diagnosis result (Weight saving of roof)

Retrofit direction	Horizontal Force			Eccentricity ratio	Reduction ratio by Eccentricity	Strength in consideration of Eccentricity	Earthquake load	Strength / Earthquake	Adjudication
	burden of all column	burden of all walls	Sum total						
	[kN]	[kN]	[kN]			[kN]	[kN]		
Span(X) direction	160	145	305	0.26	0.73	222	212	1.05	OK
Ridge(Y) direction	93	0	93	-	-	-	212	0.44	NG

Table.6 Calculation of earthquake load (Weight saving of roof)

Calculation of earthquake load	Kind of load	Unit weight	Roof Area	Weight	Shera coefficient ※1)	Earthquake load
		[kN/㎡]	[㎡]	[kN]	Ci	[kN]
Fixed load	Roof	By results of an investigation		386	0.24	93
	Roof truss ※2)	0.83	306	253		61
	fittings ※2)	0.78	306	239		57
Sum total				878		211

(※1)The area coefficient which considered the possibility of the Tokai earthquake by specification of Shizuoka Prefecture=1.2, Ci = Co×Z = 0.2×1.2 = 0.24

(※2)References37) is referred to.

Table.7 Calculation of necessary strength (Weight saving of roof)

Retrofit direction	Earthquake load	Strength	Necessary Strength	Retorfit wall length	Target Strength
	[kN]	[kN]	[kN]	[m]	[kN/m]
Span(X) direction	212	222	Strength > Earthquake load		
Ridge(Y) direction	212	93	119	9.4	21.47

ACTUAL EXAMPLE OF SEISMIC RETROFIT

Repair work of existing mud wall and existing column with Kokabe

Fig.11 and Fig.12 show earthquake retrofit position of existing mud wall and existing column with Kokabe. Photo.2 shows condition of earthquake retrofit. Since the defect junction with a boundary frame was seen on the whole by deterioration of existing mud wall and existing column with Kokabe used as earthquake resisting element, we thought that sufficient seismic performance was not demonstrated. Therefore, a mud wall is removed, and it is made replacement the structural plywood of thickness: 12[mm].

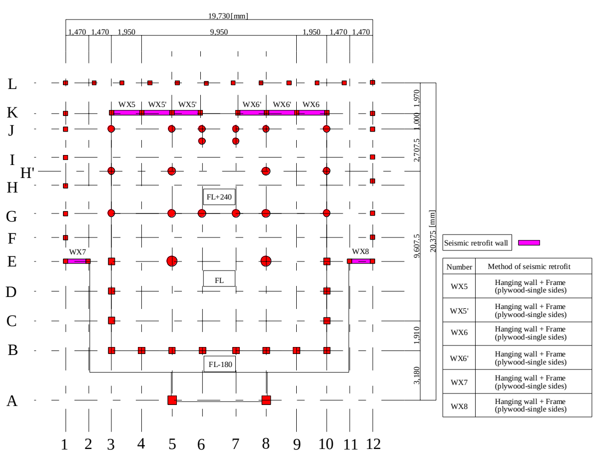


Fig.11 Earthquake retrofit alignment of existing mud wall

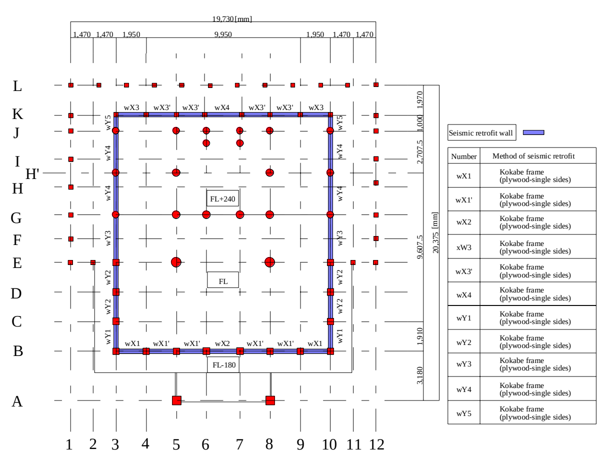
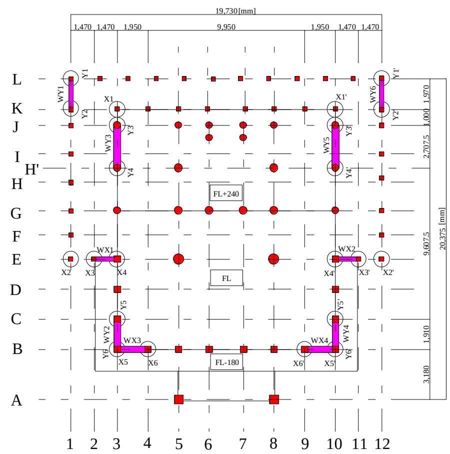


Fig.12 Earthquake retrofit alignment of existing column with Kokabe

Expansion of bearing wall

In order to compensate the absence of shear capacity to earthquake load, six bearing walls were extended in north and south direction and, five hanging walls were extended in east and west direction. The extended bearing wall constructed a stud and a bracket of plywood in lattice-shaped, and jointed the structural plywood of thickness: 12[mm] to both sides at the interval of 8-9 [cm] with nail(CN50). Moreover, the extension bearing wall prepared the wooden sill of sectional area: 72×72[mm] newly, and joined it on RC underground beam with the anchor bolt. And a metallic material (Hold-down) is attached in the column base, in order to forfend pull-out of the column base. Fig.13 shows alignment of extended bearing wall. Fig.14 shows detail drawing of extended bearing wall.

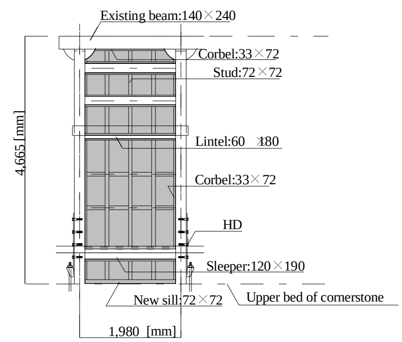


Joint metallic material of seismic retrofit

number	Name of joint metallic material
X1 , X1'	HD-B10
X2 , X2'	HD-B20
X3 , X3'	HD-B20
X4 , X4'	HD-B20 and HD-B20
X5,Y6 , X5',Y6'	HD-B20 and HD-B20
X6 , X6'	HD-B20
Y1 , Y1'	HD-B20 and HD-B20
Y2 , Y2'	HD-B20 and HD-B20
Y3 , Y3'	HD-B20 and HD-B10
Y4 , Y4'	HD-B15
Y5 , Y5'	HD-B20 and HD-B20

Seismic retrofit wall

Number	Method of seismic retrofit
WX1	Bearing wall(plywood-both sides)
WX2	Bearing wall(plywood-both sides)
WX3	Bearing wall(plywood-both sides)
WX4	Bearing wall(plywood-both sides)
WY1	Bearing wall(plywood-both sides)
WY2	Bearing wall(plywood-both sides)
WY3	Bearing wall(plywood-both sides)
WY4	Bearing wall(plywood-both sides)
WY5	Bearing wall(plywood-both sides)
WY6	Bearing wall(plywood-both sides)



Common element

The structual plywood is stretched to both sides using a Douglas fir.(Thickness of 12mm)
The structual plywood is joined with nail of CN50.(@8-9cm)
Anchor of a lower hole down fixed to RC base.
Corbel is binded with a coach bolt.
The junction part of components, such as brace, a column, and Kamoi, use Z mark ironmongery goods.
(or equivalent of Z mark ironmongery goods)

Fig.13 Alignment of extended bearing wall

Fig.14 Detail drawing of extended bearing wall

Repair work of footing

The column base of the existing mud wall of east and west direction at northern end of the temple building and bearing wall newly extended in a circumference of the temple building is on upheaped earth retaining with the stone. It has the danger of crash by an earthquake load. In order to utilize bearing wall effectively, seismic retrofit of a stone is needed. The method of seismic retrofit undertakes construction reverse of L-type retaining wall of RC at circumference of upheaped stone. Moreover, the footing and a column base are made to connect. Fig.15 shows alignment of repair work of footing. Photo.3 shows condition of repair work of footing.

Seismic retrofit of horizontal plane of structure in roof truss

The setting of horizontal brace utilize existing mud wall and extended bearing wall effectively as opposed to an earthquake, and it makes horizontal rigidity high. We installed a steel joint plate(PL-6) which carried out a bolt(M16) fasten in a capital of roof truss, and it connected columns and neighboring beams of 30 places by plain bar(M16). Fig.16 shows alignment of horizontal brace. Photo.4 shows condition of setting.

Photo.2 Column with Kokabe

Photo.3 Repair work of footing

Photo.4 Setting of horizontal

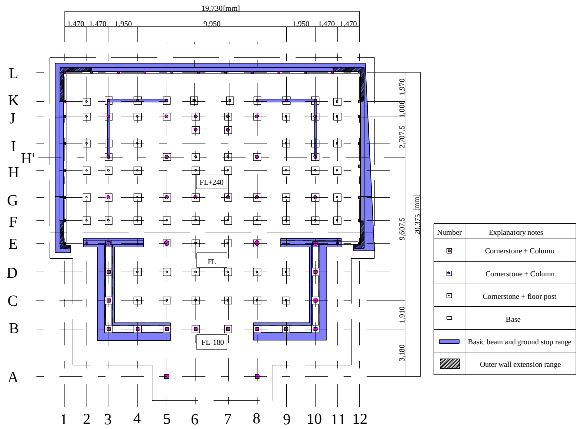


Fig.15 Alignment of repair work of footing

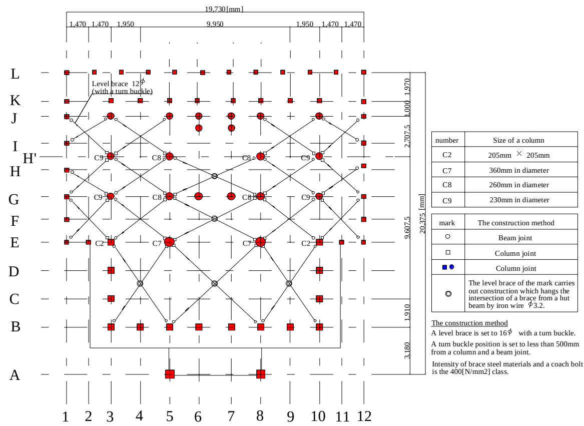


Fig.16 Alignment of horizontal brace

Repair work of thatching

The ratio of the roof weight occupied in earthquake load is large. For the purpose of mitigation of roof weight, this seismic retrofit also carried out conversion of the thatching of the whole roof. The situation is shown in Photo.5. The roof was deconstructed from the ridge. Work was removed in order of plain roof tile, cedar bark roofing, and sheathing roof board. And the new sheathing roof board was affixed densely simultaneously. Roofing, pantile, plain roof tile and the ridge were constructed. By this construction, roof weight was able to be decreased compared with seismic retrofit before. It shows below.

1) The clay roofing of the whole roof was removed and the tile was transposed to the light tile.
2) The ridge part was made smaller than seismic retrofit before, and was made light.
3) Onigawara was made smaller than C before, and was made light.

Photo.6 shows panorama of the temple building(Rensho-ji) after seismic retrofit.

Photo.5 Repair work of thatching

Photo.6 Rensho-ji after seismic retrofit

CONFIRMATION OF THE EFFECT OF SEISMIC RETROFIT BASED ON MICROTREMOR MEASUREMENT

In order to confirm the effect of seismic retrofit mentioned above, we carried out the microtremor measurements before and after this seismic retrofit.

Outline of microtremor measurement

The sensing instruments for microtremors are one component moving-coil velocity type seismometer having a natural period of 1.0[sec]., damping constant of 0.7 sensitivity of 3[volts/kine]. All of the data are recorded by digital form at sampling frequency of 100[Hz]. Two types of seismometer locations are shown in Fig.17.

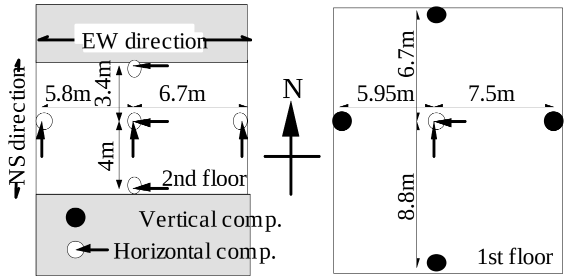


Fig.17 Two types of seismometer locations

Natural frequency

As an example, microtremors and its Fourier spectra in NS component and EW component are shown in Fig.18, Fig.19 and Fig.20, respectively. From Fig.19, we can recognize a natural frequencies around 2.0[Hz] in both sides of component and a torsional frequency in 2.9[Hz]. Comparing the Fig.3 and Fig.4, we can recognize an increase of natural frequency from 2.0[Hz] to 3.5[Hz] in NS component, from 2.0[Hz] to 3.2[Hz] in EW component, and from 2.9[Hz] to 4.9[Hz] in torsional component. Moreover, the effect of reduction of the torsional deformation can be seen from the decrease of the spectral value around the 4.9[Hz].

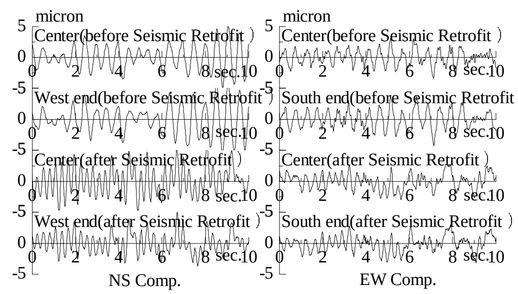


Fig.18 Microtremors

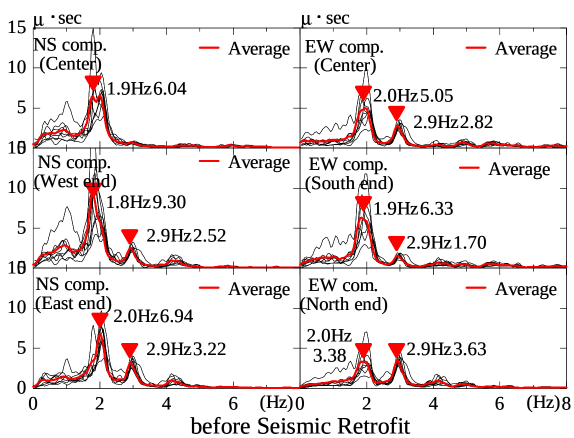


Fig.19 Fourier spectra (before retrofit)

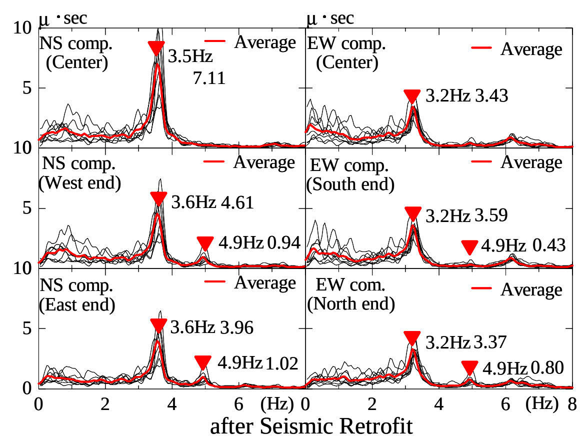


Fig.20 Fourier spectra (after retrofit)

Damping constant

Free vibration waves produced by man power are shown in Fig.21. The damping constant has decreased owing to the seismic retrofit from 3.5[%] to 2.7[%] in NS component, and from 3.9[%] to 2.3[%] in EW component. The reason of this reduction is caused by the tightening of the connection of members.

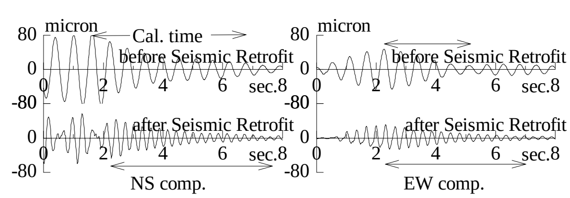


Fig.21 Free vibration waves

Deformation of torsional mode and soil-building interaction mode

To investigate the mode shape around the natural frequency, smoothing of waves are carried out. The Fourier amplitude spectra (F.A.S.) around the natural frequency are cut out by a band pass filter. A band width of the filter is 0.5[Hz] and height of it is 1.0[Hz], and both side of this rectangular filter having the cosine taper of 0.5[Hz]. The smoothed waves are obtained by transforming these cut out F.A.S. to the waves of time domain. Deformation of a translational motion and a torsional motion of the 2nd floor level of the temple are shown in Fig.22. The translational motion is larger in NS component than in EW component, and this motion becomes a bow shape after the seismic retrofit. The torsional motion included the translational one in EW component is recognized before the seismic retrofit, and this translational motion almost vanishes after the seismic retrofit. The reason of this phenomenon is explained by the tightening of horizontal structure caused by putting in diagonal horizontal steel braces.

The time histories of swaying and rocking ratios (Rs and Rr) are shown in Fig.23. The Rs and Rr are defined as Rs=Xb/Xt and Rr=Xr /Xt where Xb is the base displacement including ground motion and the horizontal deformation of the soil, Xt is the absolute displacement of a temple at the 2nd floor level and Xr is the displacement caused by the rotational deformation of the soil at the 2nd floor level. From this figure, Rs is about 20[%] and Rr is about 1[%] and these values don't transit after the seismic retrofit.

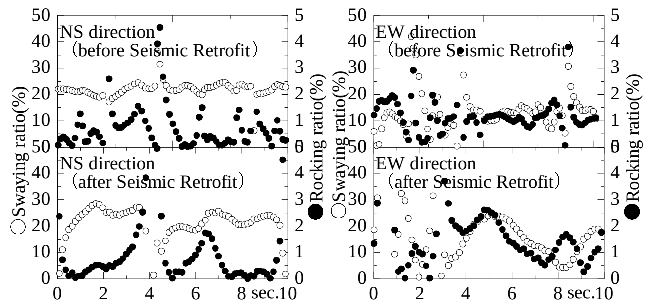


Fig.23 Swaying and rocking ratios

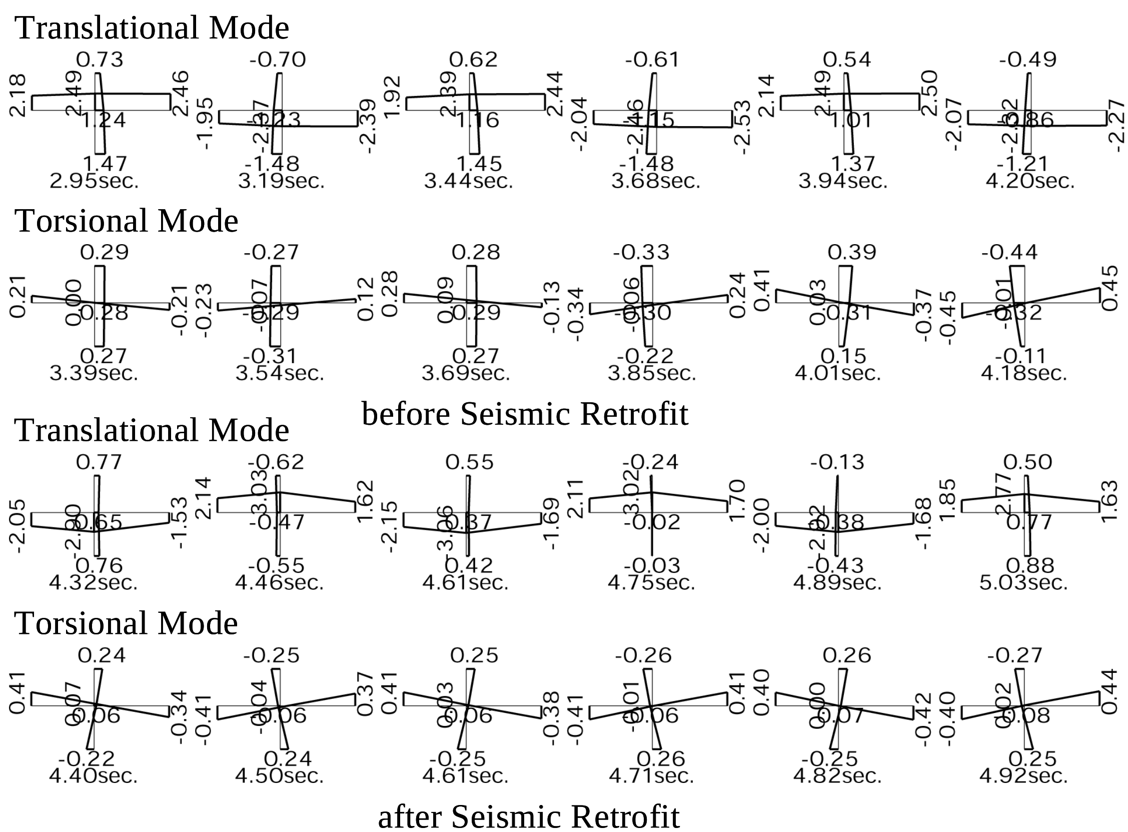


Fig.22 Deformation of a translational and torsional motion

CONCLUSIONS

1) Structural style of Japanese temple building is peculiar, and the weight of a roof is very large. As a heavy reason, the roof tile of the temple building is heavy. And clay roofing which superimpose on a roof tile also occupies 30 - 40 percent of roof weight. Moreover, a coiumn exists mostly. Howevre, it has the characteristic that wall length ratio is little.
2) Natural frequency of Japanese temple building is 1.5[Hz] to 2.7[Hz]. Natural frequency is small in order of temple building- traditional wooden house -wooden house. Damping factor of a temple building is concentrated on 4[%] from 3[%]. And damping factor of a wooden house is concentrated on 2[%] from 1.5[%]. Therefore, deformation performance of temple building can say that it is high.
3) Seismic retrofit of Japanese temple building needs consideration of design. Therefore, it is difficult to secure shear capacity of a temple building only by bearing wall. Consequently, in order to reduce earthquake load, it is effective to make weight light. And weight saving of roof carried out as the seismic retrofit method.
4) As a confirmation of the seismic retrofit effect, natural frequency of translation first mode and natural frequency of torsional mode became large 1.7 times. However, rigidity of temple building was high by seismic retrofit. Therefore, since deformation of a joint etc. was controlled, damping factor became small with 0.7 times.

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