J Wood Sci (2014) 60:403–413
DOI 10.1007/s10086-014-1425-z

ORIGINAL ARTICLE

Stress estimation of roof axial members based on the release strain measured at demolition of the Japanese wooden traditional old temple

Mariko Yamasaki · Miyuki Uchida · Toshiya Torichigai · Yasutoshi Sasaki

Received: 12 February 2014 / Accepted: 10 July 2014 / Published online: 12 August 2014

Abstract In Japan, there are many large-scale old wooden buildings which show good durability. These buildings are subjected to external forces of various strength, including several disasters throughout the years. This study aims to understand the structural features of such old buildings by examining the stress state of their structural members. Targeting the dismantled old roof frame of a temple, measurement of Young's modulus on its members using the stress wave method was performed, as well as measurement of the strain of the members during the demolition, and these results were used to conduct structural analysis. Using the velocity of the strut and the girder, the average Young's moduli of both members were estimated to be 12.3 and 11.1 kN/mm^2, respectively. From these values and the strain measurement values, when removing 83 % of the total load of the roof frame and roof, the average vertical stress of the strut was estimated to be 4.4 N/mm^2; the bending stress of the girder was estimated to be 3.1–5.9 N/mm^2. These values were matching with the values of 2D-FEM analysis. Furthermore, it was suggested that the maximum bending stress of a girder could be 117 % of the allowable stress of Japanese red pine.

Keywords Traditional architecture · Stress wave velocity · Young's modulus · Stress estimation · Release strain by demolition work

M. Yamasaki (✉) · T. Torichigai · Y. Sasaki
Department of Biosphere Resources Science, Graduate School of Bioagricultural Sciences, Nagoya University, Nagoya 464-8601, Japan
e-mail: marikoy@agr.nagoya-u.ac.jp

M. Uchida
Nakamura Institute of Architecture Co. Ltd., Nagano 380-0803, Japan

Introduction

In Japan, there are many traditional old wooden buildings. This type of architecture is particularly represented in old temples and shrines; among these buildings, at over 1300 years old, Horyuji Temple especially demonstrates the form's durability. This Japanese traditional wooden architecture has a structural style that basically consists of columns and beams, but does not have braces (diagonal members). Moreover, without using adhesive or metal connections, Japanese traditional wooden architecture was built with a highly distinctive construction method in term of these joints. In these buildings, many valuable, long, and large timbers and logs are used, encapsulating a uniquely Japanese and rare traditional technique of wood utilization. Thus, preservation of such old traditional architecture has important meanings from both the perspectives of resource and culture.

Since Japan is frequently affected by massive natural disasters, such as earthquakes, typhoons, and snow accumulation, the structural safety of buildings is of utmost importance. Even for wooden architecture, particularly after the Southern Hyogo Prefecture Earthquake in 1995, the requirements for seismic performance, mainly for residential buildings, has become strict. Traditional construction methods have a different structural type to modern housing and it is not easy to analyze the structural features of these traditional methods. Therefore, study of these structural features has been actively carried out in recent years [1]. Societal demands of strengthening seismic resistance do not exempt existing wooden buildings. In recent years, scientific studies targeting existing traditional old architecture and seismic retrofitting of these structures have been carried out [2–4]. However, for the wood members used in these buildings, the mechanical properties of these members are not being evaluated quantitatively

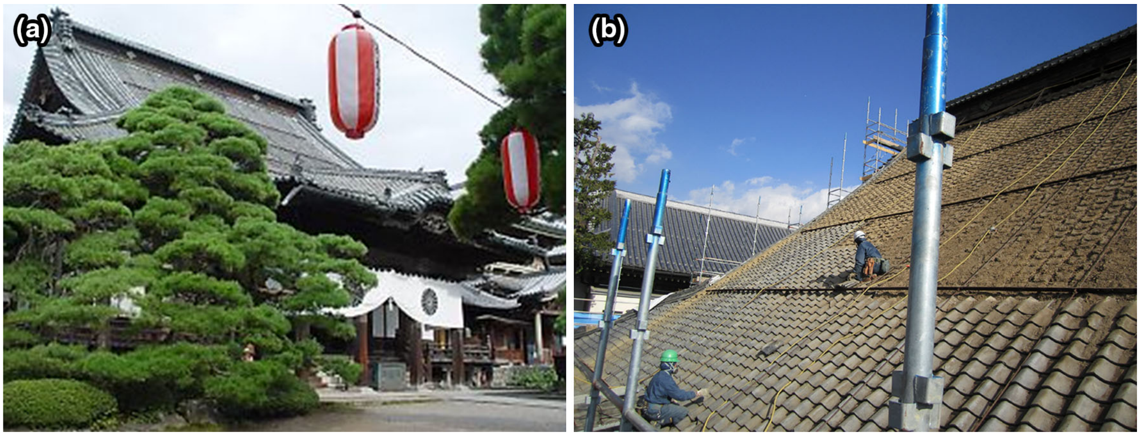

Fig. 1 Main hall of Zenkoji Daikanjin Manzendo Temple: **a** front view before the dismantling; **b** removal of the tile associated with dismantling

and scientifically due to the difficulty in measurement. In Japanese traditional wooden building, wood members are combined together by a joining method that creates mortises on the wood. Throughout the years, because a building may have been subjected to both large and small external forces, including several disasters, its framework structure (post and beam construction) might have possibilities to be distorted. In such case, the stress states of the member cannot be evaluated exactly only from calculation based on structural plans. Therefore, it is considered that the mechanical property of wooden members should be precisely evaluated to comprehend such condition.

This study aimed to quantitatively evaluate the stress state of the members inside a Japanese traditional old wooden structure at the opportunity when it was dismantled and repaired. For this purpose, with the roof frame of an old temple of more than 100 years old as a target, the Young's moduli of the members were measured before the demolition work, also the strain of axial members was measured during the work. Using these measurements, the stress states of the members were evaluated. Furthermore, the calculated values of stresses using 2D-FEM analysis and the measured values were compared, and additionally comparisons between the allowable stress and the actual stress states of the members were made.

Materials and measuring methods

Measured building and wood member

The target building is a temple of Tendai sect in Nagano City, Nagano Prefecture, named Zenkoji Daikanjin Manzendo (Fig. 1a). It was built in 1894 and its roof frame was renovated from July 2010 to February 2012. The age of the roof frame during the renovation was 115 years. Figure 2 shows the cross-sectional view in the span direction (Y direction). The roof ridge was at 19.42 m (X direction), the roof span was at 19.09 m (Y direction), and the floor area was 370.75 m^2. In addition, the height under the floor was at 1.68 m; the room height was 15.18 m; the total building height was at 17.94 m. The pillars of the interior and the outer periphery of the building were round columns of Japanese zelkova (*Zelkova serrata*). Japanese zelkova was also used for the square columns in the worship area, or 'Kohai'. The cross-sectional dimension of each round column ranged between 270 and 380 mm in diameter; the square columns were 340 $\times$ 340 mm. Round logs of Japanese red pine (*Pinus densiflora* Sieb. et Zucc.) with a cross-sectional diameter of 230–540 mm were used as roof girders. The roof consisted of pantile roofing on an Irimoya (hip and gable) roof; the tiles were roofed on roof clay. The length of the overhanging eaves was about 3.6 m.

The roof frame of the study target was dismantled once for refurbishment between November 2010 and March 2011 (Fig. 1b). The net area of the roof frame was 979.0 m^2; its horizontal projection area, including the front gate (Kohai), was 793.64 m^2. Table 1 summarizes the Young's modulus of measured members obtained using stress waves prior to demolition. Descriptive statistics on stress wave propagation velocity data for all roof frame girders are listed (number of measured members was $n = 32$; average bottom end diameter was 344 mm; average top end diameter was 273 mm; average length of members was about 6 m) as well as those for the vertical roof struts (number of measured members was $n = 7$; cross-sectional side length was about 136 mm square; length of member was 934 mm while excluding the tenon "Shiguchi" parts at both end; length of member was 1176 mm, including of these parts). The vertical roof struts were resting on one girder. Both types of members have some sort of "Shiguchi" joints like tenons along the fiber

Table 1 Physical and mechanical properties of members measured stress wave velocity

Member	Number of measured member	Shape		Diameter		Side length (mm)	Length of member (mm)	Measuring distance of stress wave (mm)	Stress wave velocity, V (m/s)	Estimated Young's modulus, E (kN/mm^2)
				Bottom end (mm)	Top end (mm)					
Roof girder	32	Round	Minimum	229	191	–	1500	1421	3162.0	7.5
			Maximum	541	376		8200	8015	5454.3	15.6
			Average	344	273		6000	5829	4897.6	11.1
			SD	74	37		1500	1385	397.2	1.9
Vertical roof strut	7	Square	Minimum	–	–	136	(1) 934	320	4776.1	9.8
			Maximum				(2) 1176	760	5479.5	16.0
			Average					513	5091.3	12.3
			SD			–	–	137	233.3	2.0

(1) and (2) are the lengths excluding and including parts of joint, respectively

SD standard deviation

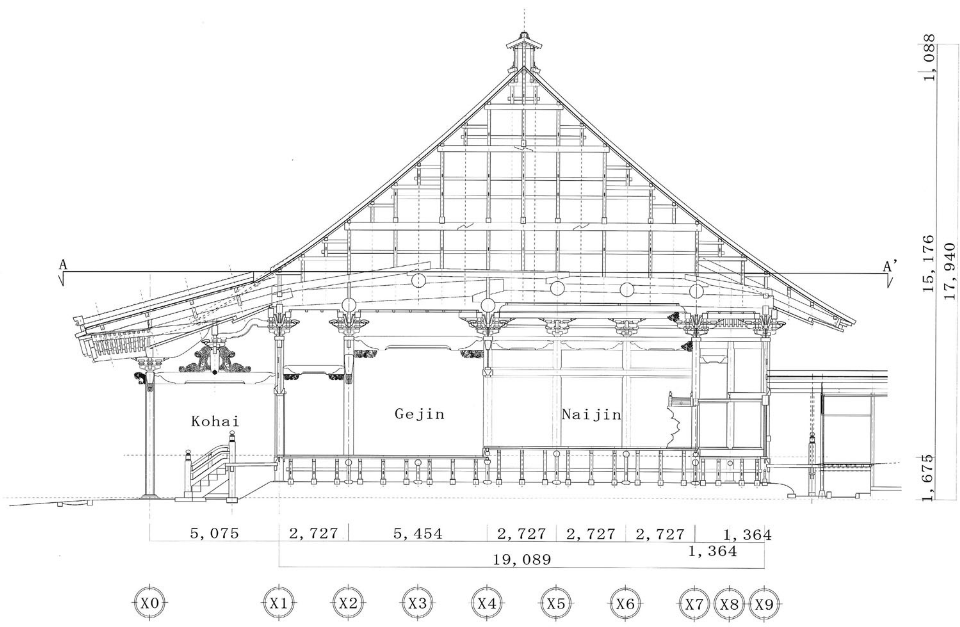

Fig. 2 Cross-sectional view in the span direction of Zenkoji Daikanjin Manzendo Temple (unit: mm)

direction. Furthermore, for members that were studied via stress analysis by measuring the strain along the demolition work, larger bending returns were expected in one roof girder (bottom end diameter was 350 mm; top end diameter was 258 mm; average diameter was 304 mm) and five roof vertical struts (136 mm in square) resting on this girder and on other girders. Both types of members, the roof girder and the vertical roof struts, were subjected to Young's modulus estimation via the stress wave method. All measured members were Japanese red pine (*Pinus densiflora* Sieb. et Zucc.). Using a moisture content measuring device (electrical resistance type) to examine the moisture content on the surface of the members, values of around 10 % or less were measured.

Estimation of Young's modulus using stress wave velocity

The Young's moduli of all 32 roof girders and 7 roof vertical struts (6 of these were subject to strain measurement) inside the roof frame were estimated using the stress wave propagation velocity measurement prior to the demolition (2009 December). A portable resonant-type stress wave propagation timer (FAKOPP) was used for the measurement of the stress wave propagation velocity. To avoid the influence of mortises and other members for both roof girders and vertical roof struts on the stress wave measurement, it was ensured that the joints were not within the stress wave propagation range of the placed sensors. The measuring distance of stress wave was taken in a length range as far as possible. Therefore, the measuring distance for each member is different. As shown in Table 1, the average measuring distances for roof girders and roof vertical struts were 5829 and 513 mm, respectively. To obtain stable values, the average values for stress wave propagation time were determined by measuring those at each position three times. The propagation velocity, V, was calculated using the average propagation time and the measuring distance. In this study, we adopted the method of estimating the Young's modulus using only the propagation velocity without measuring the density. That is, using the Monte Carlo simulation method based on the existing database of mechanical properties of Japanese lumbers [5], the Young's modulus, E, was estimated from the propagation velocity; refer to the Ref. [5] for the detailed method. The database of mechanical properties for estimation purposes is currently being accumulated in research institutes across Japan, and data for the relationship between Young's modulus of softwood lumber in full scale and its density exist. In this study, the database of "ALL" (means the database including data of all species; $n = 12967$) was employed [6]; refer to the Ref. [6] for the detailed information. As described above, the moisture content on the surface of members was assumed to be 10 % although it was impossible to measure the overall moisture content of the material. Overall moisture content was taken to be a value lower than 15 %, a typical air-dry moisture content of Japan. It has been reported that the moisture content on the surface of old materials decreases over time [7]: therefore, the measured moisture content for old materials in this study was considered to be at the normal air-dry state. As this study intended to investigate the stress state at the present time, moisture content correction for the stress wave propagation velocity and the estimation of Young's modulus was not considered.

Strain measurement of roof girder and vertical roof strut

Preparation for the strain measurement on vertical roof struts and roof girders was carried out just prior to the demolition of the roof frame (November 2, 2010). The strain measurement was performed using strain gauges (Tokyo Sokki Co Ltd., PLW-60-11, gauge length is 60 mm) via data loggers (San-ei, Logger Mate DL1200) for observing a measured value. The preparation state for the strain measurement is illustrated in Fig. 3. Additionally, since the measured strain, ε, is strain that is associated with the demolition, it is exactly the "strain return amount". It is simply referred to as "strain" in this paper. The strain measurement positions are shown in Fig. 4. The strain measurement points

Fig. 3 Measurement of strain associated with dismantling

included a total of 3 points in one roof girder (ch1–ch3 of Fig. 4) and a total of 9 points in six roof vertical struts (ch4–ch12 of Fig. 4). On the vertical roof struts, strain was measured roughly at the center position along the longitudinal direction of the vertical roof struts, i.e. at a height of about 500 mm from the top surface of the girder underneath the vertical roof struts. As shown in Fig. 4, the vertical roof struts of measurement points ch4–ch9 rest on top of a single roof girder for strain measurement; the vertical roof struts of measurement points ch10, ch11, and ch12 are struts resting on top of different roof girders. Ch4 and ch5, ch6 and ch7, ch8 and ch9, and ch11 and ch12 are the measurement points at the opposite surface of the same vertical roof struts, respectively. Strain was measured at the roof girder's lowermost surface (tensile surface). Unfortunately, due to onsite circumstances, it was impossible to measure the strain at the position that is expected to show the largest return amount of bending. To measure the strain in a more accurate and stable manner throughout the demolition work period, the surface of members was flattened with sandpaper before attaching the strain gauges. Furthermore, after attaching the strain gauge on a member, a soft rubber plate was affixed on the surface of the strain gauge with adhesive tape for pressing and curing. During the demolition work (from November 2010 to February 2011), 7 times measurements in total were carried out at a frequency of about once a week. Table 2 shows the general demolition circumstances on strain measurement days and visual observations at that time. Incidentally, the entire roof frame was removed at a measurement day of the ninth time; the members for strain measurements had been removed, thus, the strain measurements were not conducted at this time.

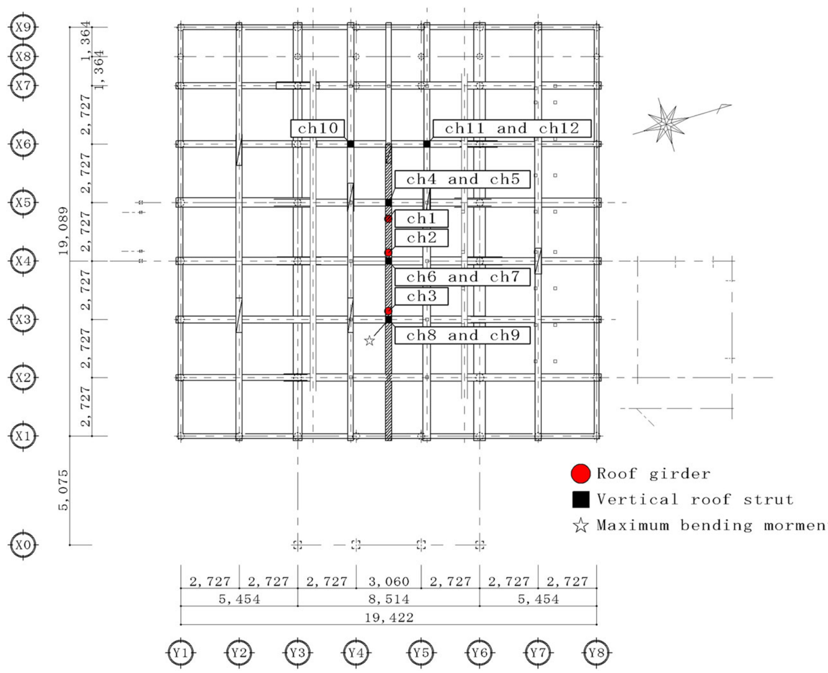

Fig. 4 Measurement position of the strain associated with dismantling (unit: mm; *A–A′* level in Fig. 2)

Table 2 Measurement process of release strain during the demolition of roof

Number of times	Date	Demolition work	Summation of removed material			Removed weight (kN)	Measurement of strain
			Roof tile	Roof clay	Wood member		
1	2010/11/2	Before	0	0	0	0.0	Done
2	2010/12/2	In process	1/5	1/5	0	320.4	
3	2010/12/9		2/5	2/5	0	640.7	
4	2010/12/16		3/5	3/5	0	961.1	
5	2011/1/5		4/5	4/5	0	1281.5	
6	2011/1/18		All	All	0	1601.8	
7	2011/1/26		All	All	1/5	1765.1	
8	2011/2/7		All	All	1/2	2010.1	
9	2012/3/1	After	All	All	All	2418.0	None

Weight measurement of roof and roof frame

To determine the weight of the roof and the roof frame, the weights of the discharged building material were measured whenever materials were carried to the disposal site. The weight measurement was accumulated on each strain measurement day, as shown in Table 2. On the other hand, the progress of the demolition work was investigated visually at the same time. Based on this visual observation of each demolition circumstance, the weights of the roof and roof frame members were calculated, respectively.

Stress analysis of roof girder by FEM

Based on the estimated Young's modulus, *E*, via the stress wave method as described above and modeled according to the cross-sectional view of the roof frame as shown in Fig. 2, the stress state of the roof girder was analyzed using a 2D-FEM analysis program [8]. In Fig. 2, a level of the analyzed roof girder is shown by *A–A′* line. Figure 5 shows a schematic diagram of 2D-FEM analysis. In the analysis, the supporting point was assumed to be the position where the roof girder touched the lower roof girder; the load point was assumed to be the position where one touched the upper roof girder or the upper vertical roof strut. For the supporting point, in fact, the analyzed frame shown in Fig. 5 was not supported by columns but supported by orthogonal girders which were put on the nearby frames, and those nearby frames were supported by columns. Because this analysis was 2D-FEM analysis, such real situation was not expressed closely. Therefore, virtual columns were set under the girder

Analysis condition		
Member	Young's modulus (GPa)	Cross-sectional area (mm^2)
Analyzed girder	12.1	72581
Other beam-A	11.1	26002
Other beam-B	11.1	14519
Roof strut	12.0	18496
Virtual column	10.0	87089

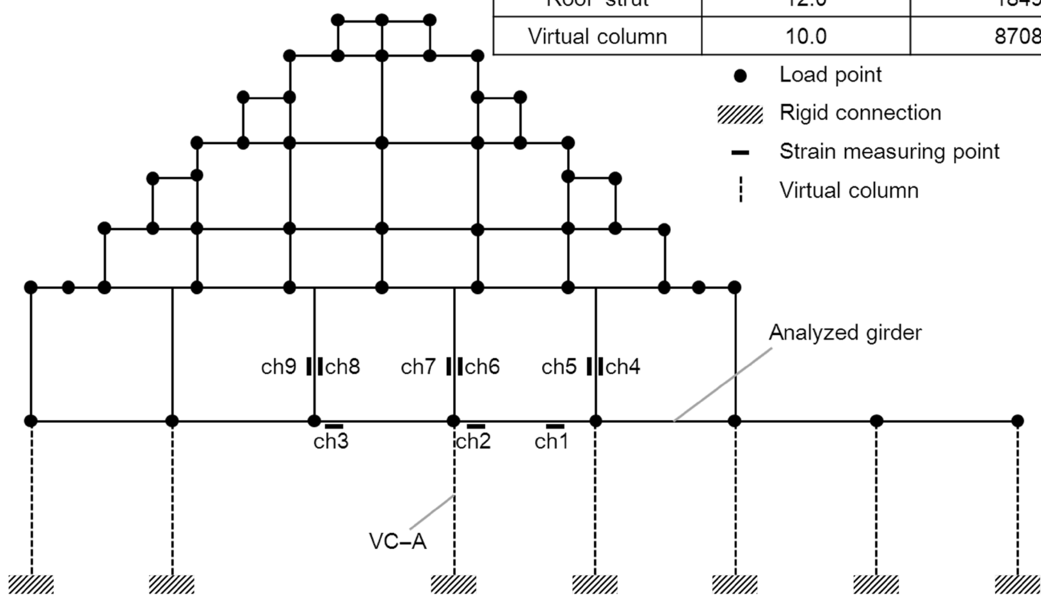

Fig. 5 Schematic diagram of 2D-FEM analysis

as the supporting points, and the girder was supported by these columns. Dimension of the columns were set as the real column, and those Young's moduli were set as 10 GPa. The other jointed positions were set as rigid joints without providing other constraint conditions. The cross-sectional shape of the girder was assumed to be circular. The circumferences were measured at several positions visually judged to have an average section size, and then the diameter of the girder was calculated from the average value of these circumferences. Although the corners of the cross-section of the vertical roof strut had been chamfered slightly, the cross-sectional shape was assumed to be rectangular; measured data were used for the dimensions. For members at high positions where the dimensions could not be measured, appropriate values were assumed by referring to general construction methods. Young's modulus of each member, E, was estimated from the respective stress wave propagation velocity. However, for the members at high positions where the velocity could not be measured, each value for the horizontal bridging member (girder) and the vertical member (vertical strut) was substituted using the respective average value of velocity measurement data for the roof girder and roof vertical strut.

Since the weight of the roof and the roof frame members correspond to vertical load, as described above, these loads were determined from the measured weight values of those members discharged during the demolition. As shown in Figs. 1a and 2, the front eaves consisted of a large overhang from the roof frame. The load of the overhanging section was supported with a horizontal bridging member, called "Hanegi". When examining the structure of the roof frame, it was confirmed that this load did not affect the strain measurement on the roof girder. In other words, since both the load of the roof frame and the load of the overhanging roof are included in total discharge weight above, those should be separated. The overhanging sections were partitioned in proportion to the projected areas ratio on the horizontal plane from the roof frame and non-overhanging section. The area portion of the non-overhanging section is 47.8 % and from this, the load of the non-overhanging section was estimated. By multiplying this value with the 14.3 % load-sharing percentage of the vertical frame including the roof girder used in strain measurement, the vertical load used in the analysis could be calculated. Corresponding to each strain measurement day shown in Table 2, the stress analysis was carried out by allocating loads to the roof and the roof frame according to the

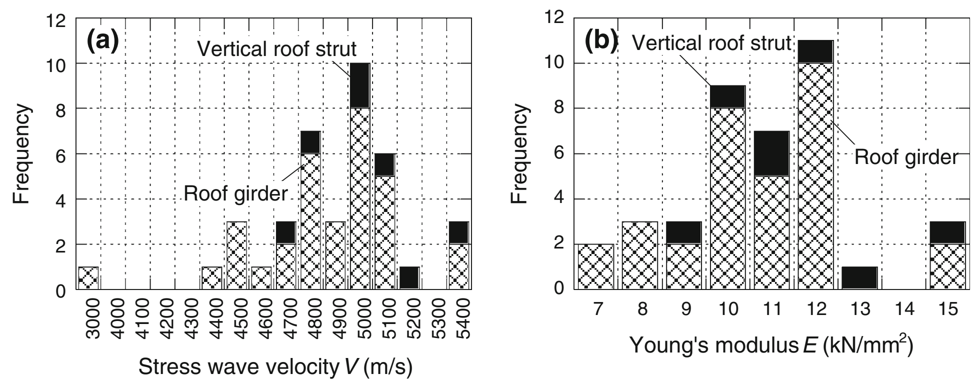

Fig. 6 Distribution of stress wave propagation velocity of roof frame member and estimated Young's modulus; **a** measured stress wave velocity; **b** estimated Young's modulus

demolition conditions during the strain measurement day. Sequentially from the top of the roof frame, the weight of a vertical member and the load between the vertical members at one layer above were taken as the vertical load. Although the actual demolition progressed locally and independently in both the vertical and horizontal planes rather than evenly (on average), this point was not considered in this analysis.

Results and discussion

Mechanical property of wood member in roof frame

The distributions of stress wave propagation velocity of the members of the frame, V, and Young's modulus, E, as estimated from V are shown in Table 1 and Fig. 6. As shown in Table 1, the average stress wave propagation velocity of the roof strut was 5091.3 m/s and the average value for the roof girder was 4897.6 m/s. As shown in Fig. 6a, the value of the stress wave propagation velocity, 3162.0 m/s, is much smaller than the other measured values. At a later date, this girder was examined in detail. Decay was observed in the roof girder and this decay is considered the reason for the resulting small stress wave propagation velocity. By running the Monte Carlo simulation method with these propagation velocities, the average Young's moduli of the vertical roof struts and the girder were estimated to be 12.3 and 11.1 kN/mm^2, respectively (shown in Table 1 and Fig. 6b). According to the mechanical properties database for lumber, the value of minimum–average ± standard deviation–maximum for the bending Young's modulus of Japanese red pine lumber (visually graded material) is 5.0–10.3 ± 2.3–17.8 kN/mm^2 (adjusted moisture content of 15 %) [9]. The Young's modulus distribution for Japanese red pine members as measured in this study is slightly larger when compared with the database, but it is still within the Young's modulus distribution range of Japanese red pine materials in general.

Weight of roof and roof frame

In Table 2, for each day of strain measurement, the cumulative values of the removal loads for the dismantled roof and the roof frame are tabulated. Moreover, the demolition status at each time point was also shown. Unintentionally, the removal load of the roof and roof frame increased proportionally with respect to demolition status in general, as shown in Table 2. The final removal load, that is the total weight of the roof and roof frame, was 2418 kN (5841 N/m^2). When examining the breakdown of the discharged total weight, the total weight of the roof was 1601.8 kN (66.2 % of the total) and the total weight of the wooden roof frame was 816.2 kN (33.8 % of the total). Since the volume of the wooden roof frame that had been discharged was 166.63 m^3, the density of the wooden roof frame member was calculated to be 499.3 kg/m^3. Its members were mostly Japanese red pine. According to the database of building materials (FFPRI 2005 [9], visually graded material, at 15 % moisture content), the average density of Japanese red pine lumber with standard deviation is 526 ± 56.8 kg/m^3. This result is, therefore, considered to be reasonable.

In accordance with the "Implementation Guidelines for Basic Diagnosis of Seismic Performance of Important Cultural Properties (Buildings)" published by the Agency for Cultural Affairs, Ministry of Education, Culture, Sports, Science and Technology in Japan [10], the roof load per floor area in the case of "Clay-containing pantile roof" is 2400 N/m^2. Since the floor area of Manzendo Temple, investigated in this study, is 370.75 m^2, the roof load based on the guidelines from the Agency for Cultural Affairs is calculated to be 889.8 kN, which is much smaller than the actual roof load (1601.8 kN). As mentioned above, the eaves in the Manzendo Temple were greatly extended. Moreover, the roof height was considered high and even the weight of the ridge-end tiles was also greater than expected. Because the value calculated using the guidelines from the Agency for Cultural Affairs uses weight per floor area, under these conditions, it is considered that the actual

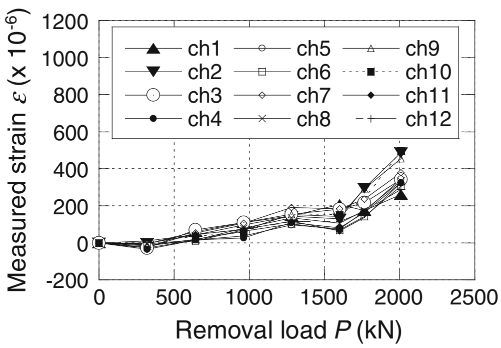

Fig. 7 Relationship between the removal load and the strain; ch1–3, roof girder; ch4–12, vertical roof strut

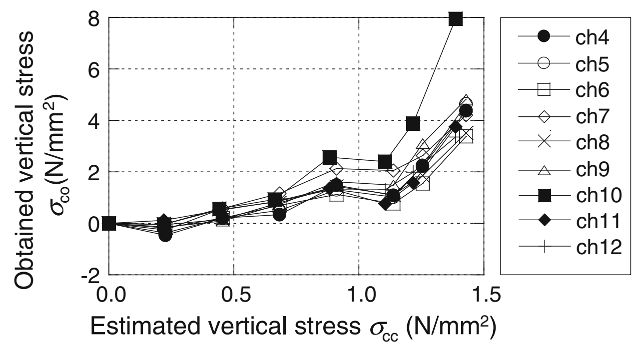

Fig. 8 Relationship between the measured value and the estimated value of the vertical stress acting on the vertical roof strut

measured value was much greater than the calculated value. Miyamoto et al. [2] also measured the volume of waste from a temple when performing seismic retrofitting on the building architecture according to a seismic diagnosis. According to their research, the roof ridge was at 19.73 m; the roof span was at 20.38 m; the floor area was 306 m^2; the building height was 8.8 m; the roof weight of the temple was 706 kN; the weight of its roof frame was 253 kN. On the other hand, for Manzendo Temple, the roof ridge was at 19.42 m; the roof span was at 19.09 m; the floor area was 370.75 m^2; the height was at 17.94 m; the roof weight was 1601.8 kN; the weight of its roof frame was 816.2 kN. Despite the fact that the dimensions for the roof ridges and roof spans as well as the floor areas in both buildings are substantially similar, the roof heights are significantly different. Due to this, it is impossible to simply compare these buildings. The roof weight and the weight of the roof frame in Manzendo Temple were significantly larger than the temple examined by Miyamoto et al. Comparing the values calculated by dividing the weight by the floor area for both buildings, the roof load and the roof frame load of the temple examined by Miyamoto et al. are 2.31 and 0.83 kN/m^2, respectively, whereas those are 3.46 and 2.20 kN/m^2, respectively, in the case of Manzendo Temple. The roof load and the roof frame weight of Manzendo Temple were 1.5 times and 2.7 times those of the temple measured by Miyamoto et al. From the above result, especially in the case of the temple architecture whose roof shape has infinite variety, it is understood that the roof load can vary greatly between buildings. Since the roof load affects the stress state of the members below, this finding is, therefore, important.

Strain of vertical roof strut and estimation of vertical load

Figure 7 shows the strain change arising from the removal of the roof load during the demolition of roof frame. As mentioned above, the measured strain, ε, is the return amount as a result of the removal load P; the strain that has been actually caused by the roof load. The plots in this figure correspond to the strain measurement positions, illustrated in Fig. 4. Plots of ch4–ch12 show the measurement results for the vertical roof strut. As can be seen, regardless of measurement positions, the strain measured at the removal load of 1601.8 kN (No. 6 in Table 2) was smaller than the strain measured at the removal load of 1281.5 kN (No. 5 in Table 2). The cause of this is not known for certain; however, a scaffold which was placed during the process of dismantling the roof may have affected the results. Excluding this data, the strain in the vertical roof strut increased with increasing removal load. For the removal load at 2010.1 kN (No. 8 in Table 2), the value that was measured when half of the wooden members forming the roof frame remained was taken as the final result for strain measurement, and the value of minimum–average $\pm$ standard deviation–maximum for the strain in the vertical roof strut was 289–361.1 $\pm$ 70.4–496 ($\times 10^{-6}$).

Depending on the load-shared area of each member, the vertical stress, σ_{cc}, acting on each vertical roof strut was estimated from the removal load. In Fig. 8, comparisons between the σ_{cc} and the obtained vertical stress, $\sigma_{co} = E\varepsilon$, by multiplying the strain measured value, ε, and the estimated Young's modulus, E, from stress wave propagation velocity, are shown. Trends in the plot are due to the increase in the removal load, and associated with the progress of demolition. As shown in Fig. 8, the orders of both values were substantially matched. However, the obtained vertical stress from strain, σ_{co}, for most of measurement positions was greater than the estimated stress, σ_{cc}, from the shared load taken from the plans. The difference between both values increased with increasing removal load. In the case of final strain measurement (No. 8 in Table 2, removing 83 % of the total load), the average value with standard deviation of obtained vertical stress, σ_{co}, from strain measurement was 4.4 $\pm$ 1.4 N/mm^2; the estimated vertical stress, σ_{cc}, from shared load taken from the plans was 1.4 ± 0.02 N/mm^2. The former was about 3 times larger than the latter. The reason for this result is considered to be non-uniformity in sharing the vertical load, which is in turn due to the error of construction precision of the roof frame structure, degree of the precise fit of the "Shiguchi" joints, and the heterogeneity in mechanical properties of the members.

Roof girder strain and estimation of bending stress

Plots of ch1–ch3 in Fig. 7 show the strain change arising in the roof girder while removing the roof load during the demolition of the roof frame. These three strain gauges were placed at different positions on the one girder described above, as shown in Fig. 5. As shown in the figure, the strain on any measurement positions of the roof girder increased in proportion to the progression of demolition. That is, the strain increased with increasing removal load. At a removal load of 2010 kN (No. 7 in Table 2), when half of the wooden members forming the roof frame remained during the strain measurement, ch1 and ch3 had strains of 262×10^{-6} and 342×10^{-6}, respectively; whereas ch2 had a strain of 487×10^{-6}.

The measured strain value of the roof girder (ε) and the Young's modulus of the relevant member ($E = 12.1$ GPa) was used to calculate the obtained bending stress, $\sigma_{bo} = E\varepsilon$, at the position of strain measurement and the obtained bending stress was compared with the estimated bending stress ($\sigma_{bc} = M/Z$) of the same section using 2D-FEM analysis. The model of 2D-FEM analysis is shown in Fig. 5. This comparison is shown in Fig. 9 with white symbols. Similar to Fig. 8, trends in the plot are due to the increase in the removal load, associated with the progress of demolition. As shown in Fig. 9, the bending stress, σ_{bo}, calculated from the measured strain value is significantly larger when compared to the estimated stress, σ_{bc}, from the FEM analysis in the measurement positions ch1–ch3. The σ_{bo} of the final strain measurement (No. 8 in Table 2, 83 % of the total load removed) was 3.1–5.9 N/mm^2. The σ_{bo} value at ch3 was found to be closest to the σ_{bc} value, where σ_{bo} was 0.9 times of σ_{bc}; the largest difference between these two values was at ch1 where σ_{bo} was 22.7 times of σ_{bc}. Because the building has been loaded repeatedly with horizontal forces such as wind pressure and seismic force as well as vertical load due to snow accumulation, it has received various external forces during its long-term use. The frame structure is believed to have gradually changed by the influence of such external forces; as a result, the joints of the entire skeleton structure are not necessarily well joined. Thus, differences were found between the FEM-analyzed value, σ_{bc}, and the measured value, σ_{bo}. For example, the roof girder and the columns under it are joined with only the tenon in the vertical direction: such a

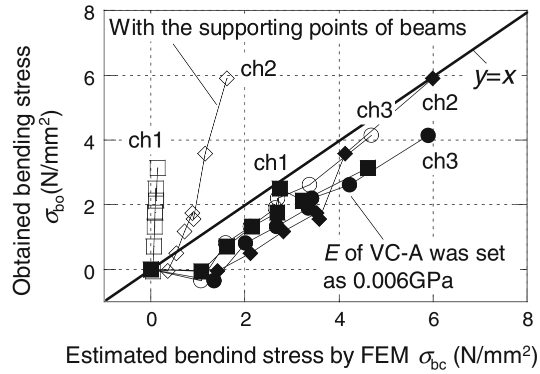

Fig. 9 Comparison between the measured value and the estimated value via 2D-FEM analysis of the bending stress acting on the girder; *white symbols* analysis with the supporting points of columns; *black symbols* analysis in which the Young's modulus of VC-A was set as 0.006 GPa

joint is in an unfastened state. That is, it is highly conceivable that the girder was possibly in a floating state. Then, in the analysis model shown in Fig. 5, the Young's modulus of one virtual column, VC-A, was set extremely small, and the FEM analysis was conducted again. The results when the Young's modulus of VC-A was set as 0.006 GPa are shown in Fig. 9 with black symbols. As shown in Fig. 9, the estimated stress value, σ_{bc}, by FEM analysis was substantially close to the bending stress, σ_{bo}, calculated from the measured strain value. In the case of the final strain measurement, the bending stress, σ_{bo}, calculated from the measured strain value was about 0.7–1.0 times of the estimated stress, σ_{bc}, from FEM. This result supports the hypothesis relating to the joint as described above. In the case of a building built using Japanese traditional construction methods such as Manzendo Temple, the intra-individual variation and the inter-individual variation of Young's moduli in wood members, the human-made non-uniformity during the construction and changes of the structural frame characteristics due to external forces over time are considered to be inevitable. Therefore, it is easy to assume that some deviation occurs in the building.

Lastly, the relationship between the estimated maximum bending stress of this girder and the vertical load was examined. The estimated maximum bending stress σ_{max} was determined from the measured strain value. As described above, it is possible to determine the stress by substituting the measured strain, ε, and the estimated Young's modulus, E, from the stress wave propagation velocities into $\sigma_{bo} = E\varepsilon$, but this strain was not intended to have measured the position of the maximum bending moment. In other words, the σ_{bo} value is the bending stress on the position of strain measurement, not the maximum bending stress. Therefore, the bending moment via FEM analysis on the strain measurement position (ch2), M_{bc}, and

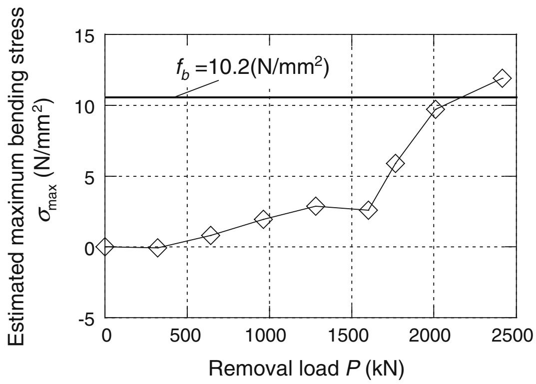

Fig. 10 Estimated maximum bending stress of the roof girder

the maximum bending moment acting on the relevant roof girder, M_{max}, were determined, respectively, then the maximum bending stress of roof girder was estimated using $\sigma_{max} = \sigma_{bo} \times M_{max}/M_{bc}$. Here, because it's given that the above results are related to the stress in the roof girder, FEM analysis was conducted as if the Young's modulus of VC-A was set as 0.006 GPa. Moreover, at the time when the roof and roof frame were all removed (No. 9 in Table 2, removal load of 2418 kN), it was not possible to measure the strain. Therefore, the maximum bending stress at that time, $\sigma_{max(9)}$, was estimated by multiplying the ratio of the $\sigma_{max\text{-}FEM(8)}$ at a removal load of 2010 kN (No. 8 in Table 2) from FEM analysis to the estimated maximum bending stress, $\sigma_{max(8)}$, based on strain measurement value, $\sigma_{max(8)}/\sigma_{max\text{-}FEM(8)}$, with the $\sigma_{max\text{-}FEM(9)}$ at the removal load of 2418 kN (No. 9 in Table 2). Figure 10 shows the relationship between the removal load during the demolition of the roof frame and the maximum bending stress, σ_{max}, of girders that were subsequently released. The allowable stress in bending of Japanese red pine lumber, f_b = 10.2 N/mm^2 [11], is also shown in the figure. As shown in Fig. 10, it was inferred from the estimation based on the FEM analysis and strain measurements values that the maximum stress acting on the girder, σ_{max}, was 11.91 N/mm^2 (117 % of f_b). In other words, it was suggested that the measured girder could have a stress state more than or equal to the allowable stress, f_b.

Conclusions

This study aimed to investigate the stress state of the structural members inside the roof frame of an old temple that is exemplary of Japanese traditional wooden architecture and is more than 100 years old. Using the Young's modulus estimated from the stress wave propagation velocity as well as the measured strain along with the demolition of the roof and the roof frame, the stress acting on members was examined. Furthermore, using these results, the estimated stress value from skeleton structural analysis was compared with the standard allowable stress of Japanese red pine material, determined by the Japanese Building Codes. The findings are as follows.

- The floor area of the temple's roof was 370.75 m^2; the total weight of the roof frame was 2418 kN.
- The average stress wave propagation velocities of the vertical roof struts and the roof girder, made of Japanese red pines, were 5091.3 and 4897.6 m/s, respectively. The average Young's moduli of both were estimated to be 11.1 and 12.3 kN/mm^2, respectively, from the propagation velocity via Monte Carlo simulation method.
- When 83 % of the total load of roof and roof frame had been removed, the stress acting on the members was determined from the estimated Young's modulus and the strain measurement value. The average vertical stress of the vertical roof struts was 4.4 N/mm^2; the bending stress on the strain measurement positions of the roof girder were determined to be 3.1–5.9 N/mm^2. The order of these values matched the results from 2D-FEM analysis when it was supposed that the some joint did not work enough.
- Based on the strain measurement values and FEM analysis, the estimated maximum bending stress of the roof girder was at 11.91 N/mm^2, which was 117 % of the allowable stress of Japanese red pine. From this, it is suggested that there existed a stress state on the girder that was greater than or equal to the allowable stress for Japanese red pine.

Studies like this one, in which a traditional building was examined mechanically from the viewpoint of its members to elucidate its features more scientifically, will achieve progress that will more certainly ensure the succession of building systems of the traditional style.

References

1. Mukai Y, Yonezawa M, Koshihara M, Saito Y, Suzuki Y (2012) Numerical modeling of traditional Japanese wooden structures based on full-scale shaking table test results. In: Proc world conference on timber engineering (WCTE), Auckland, pp 429–436
2. Miyamoto S, Miyazawa K, Irie Y, Goto O (2005) Case study of seismic diagnosis and retrofit for Japanese old temple (in Japanese). J Struct Construct Eng AIJ 593:87–94
3. Suda T, Suzuki Y, Shimizu H, Ogasawara M (2006) Dynamic tests of traditional wooden house in Kyoto using large-scale shaking table. In: Proc WCTE 2006, Portland, pp 1354–1359
4. Imanishi Y (2009) Documentation on the restoration work at Toshodai-ji temple Kondo a national treasure (in Japanese). GBRC (J Gene Build Res Corp Jpn) 34:6–20
5. Yamasaki M, Sasaki Y (2010) Determining Young's modulus of timber on the basis of a strength database and stress wave

propagation velocity I: an estimation method for Young's modulus employing Monte Carlo simulation. J Wood Sci 56:269–275
6. Yamasaki M, Sasaki Y, Iijima Y (2010) Determining Young's modulus of timber on the basis of a strength database and stress wave propagation velocity II: effect of the reference distribution database on the determination. J Wood Sci 56:380–386
7. Yokoyama M, Gril J, Matsuo M, Yano H, Sugiyama J, Clair B, Kubodera S, Mistutani T, Sakamoto M, Ozaki H, Imamura M, Kawai S (2009) Mechanical characteristics of aged Hinoki wood from Japanese historical buildings. CR Phys 10:601–611
8. Yamashita S, Tanaka S (2004) Introduction of FEM analysis using EXCEL—analysis for the frame construction (in Japanese). INDEX Press, Tokyo
9. Forestry and Forest Products Research Institute (2005) Database of strength performance of commercial lumber (in Japanese). Database No. 7. The Institute, Ibaraki
10. Agency for Cultural Affairs (2012) Implementation guidelines for basic diagnosis of seismic performance of important cultural properties (Buildings) (in Japanese). Agency for Cultural Affairs, Ministry of Education, Culture, Sports, Science and Technology in Japan, Tokyo, p 9
11. Architectural Institute of Japan (2006) Standard and explanatory for the design of wooden construction—allowable stress design and allowable proof stress design method (in Japanese). Maruzen, Tokyo, pp 395–412