Seismic Performance Evaluation of Traditional Wooden House
by Alternate Cyclic Loading Test

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Summary

It is important to establish the seismic design method and reinforcement method of traditional wooden buildings. To evaluate experimentally the seismic performance of a traditional wooden house built in 1898, static horizontal loading tests were carried out. The maximum bearing force was 106.6kN when the average deformation angle of wooden frame was about 1/10rad. Cracks in columns began occurring at column and tie-beam joints when the average deformation angle was about 1/25rad. After the bending fracture of all columns occurred, the wooden frame still kept the restoring force. The seismic performance of the house is also verified by an approximate response analysis. It is found that although the base shear coefficient, 0.18, is not enough because of the heavy roof, the house can be easily renovated by repairing deteriorated mud-walls and tie-beams.

1. Introduction

Wooden houses are popular in Japan, especially traditional wooden buildings are deeply part of Japanese culture. It is important to establish the structural design method and seismic reinforcement method of wooden buildings in order to inherit them to the future. In 1998 and 2000, the Building Standard Law and its Enforcement Order in Japan were, respectively, revised, and not only the calculation of effective length of bearing walls and the stress calculation but also the response-limit capacity analysis is usable to the structural design of wooden buildings. A seismic design manual for traditional wooden buildings was published in 2004 [1] based on the response-limit capacity analysis. As far as traditional wooden framework houses are concerned, there are, however, many unknown structural mechanisms and characteristics to be solved.

In this study, to evaluate the seismic performance of traditional wooden houses, an over 100 years old traditional wooden house was precisely investigated in its structural and vibration characteristics, and static horizontal cyclic loading tests were carried out [2-5]. The seismic performance of the wooden house is conformed by the response-limit capacity analysis.

2. Overview of the investigated house

2.1 Structural characteristics

The traditional framework house in Higashi-Mikawa region, Aichi Prefecture was built in 1898. Photo 1 shows the overview of the house and Figure 1 shows the plan. The house is a typical farmhouse and a single story with a loft in the roof space which is used for storage. The house
experienced two big earthquakes, 1944 Tohnankai Earthquake and 1946 Nankai Earthquake, but the residents spoke from experiences there was little damage and they did not repair the house.

It is important to know in detail the structure of the wooden house. The detailed structural investigations were carried out before loading tests. All members such as columns and beams were numbered. The section dimensions, percentage of moisture content and pilodyn value of each structural member were recorded. Tohshi-Bashira columns were named C11, C12, ... and C43. Details of the main frames X1, X6, X11 and X15 were drawn as shown in Figure 2. Main structural members are Tohshi-Bashira (continuous columns) and Sashi-Gamoi (tie-beams connecting columns), and there is few bearing walls. The section sizes of Tohshi-Bashira and Sashi-Gamoi are 175mm × 175mm and 135mm × 330mm respectively. Each column stands on a foundation stone and Ashi-Gatame (horizontal members connecting columns at the floor level) are used only in the north and south direction in X1, X6, X11 and X15 frames. It was observed that Ashi-Gatame were decaying and lost their strength. The Tohshi-Bashira columns are connected with Sashi-Gamoi in both longitudinal and span directions. Shachi-Sen (draw pin) is used at the connection of Sashi-Gamoi’s axial direction. At the end of Sashi-Gamoi, Hana-Sen (nose pin) is used to connect with each column. Above Sashi-Gamoi, mud-plastered hanging walls or transom windows are there. The roof framing is Wagoya (simplest Japanese) method and the roof is tiled thickly with roofing mud as a bond. The average thickness of tile mud is about 10cm. The sheathing materials on rafters are not wooden boards but laths made of bamboo. As for the kind of main wooden members, Tohshi-Bashira is Hinoki (Japanese cypress), Sashi-Gamoi is Matsu (pine) and Ashi-Gatame is Tsuga (Japanese hemlock).

A loft in the roof space is on the east side from X11 frame. Because the X11 frame has hanging walls, and X15 frame has a stair and the loft, it is expected the rigidity in east part of the house is higher than in west part. There are mud-plastered walls on the west side of the house, but the bearing capacity is not expected because it was so damaged by winds and rains.

2.2 Weight Measurement

It is important to know the weight of buildings exactly in the structural design and the seismic performance evaluation. In general, the unit weights described in 84th article of the Building
Standard Law Enforcement Order are used to calculate the weight of a building. It is difficult to determine the unit weights of mud-plastered walls and cover materials of the roof for the case of the investigated wooden house. In this study, the weight of the house was examined based on the measured weight of each member and element for accuracy.

When the house was demolished after the loading tests, structural members and other elements were measured. All columns (C11 - C43), Sashi-Gamoi and Ashi-Gatame, on which displacements and strains were measured, were weighed, and some numbered members in the roof frame were also weighed. As for other structural members, the weight of them was calculated by using the specific gravity obtained from the results of material tests and the volume obtained from the structure investigation. The weights of the un-numbered members in the roof frame were calculated by assuming that the specific gravity of them was as same as of other numbered members.

As for the weight of the roof, all kinds of roofing tiles were counted precisely by the photographic interpretation while the structure investigation. The number of Hira-Gawara (broad concave tiles) is 5485, the number of Oni-Gawara (goblin tiles) is 9, and the total length of Noshi and Mune-Gawara (ridge tiles) is about 49m. The unit weight of various tiles and roofing mud per unit area or length were measured. The unit weight of Hira-Gawara is 0.156kN per 10 tiles. Thus, the total weight of Hira-Gawara is 85.47kN. The unit weight of roofing mud is 0.34kN/m² and the total area of the roof is 293.20m², and the total weight of roofing mud is about 99.85kN.

As for the mud-plastered walls, a sample of 1000mm × 800mm × 75mm was taken out of the mud-plastered wall, its weight is about 0.608kN and its volume is 0.06m³. The total volume of mud-plastered wall is about 17.90m³, and the total weight of mud-plastered walls is 181.30kN.

From the above weighing and estimations, the total weight of the house is about 596.8kN. Table 1 shows the weight of the house. The weight of floor boards, ceiling boards, doors and windows were not taken into account, because they were removed before the loading test.

<table>
<thead>
<tr>
<th>Table 1 Weight of the house (kN)</th>
<th>Weight</th>
<th>Subtotal</th>
<th>Total</th>
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<tbody>
<tr>
<td>Structural members</td>
<td>132.16</td>
<td></td>
<td>313.46</td>
</tr>
<tr>
<td>Mud-plastered walls</td>
<td>181.30</td>
<td></td>
<td></td>
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<tr>
<td>Roofing tiles</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hira-Gawara</td>
<td>85.47</td>
<td>115.66</td>
<td></td>
</tr>
<tr>
<td>Oni-Gawara</td>
<td>1.05</td>
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<td></td>
</tr>
<tr>
<td>Noshi and Mune-Gawara</td>
<td>29.14</td>
<td></td>
<td></td>
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<tr>
<td>Roofing mud</td>
<td>99.85</td>
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<td></td>
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<tr>
<td>Sheathing laths made of bamboo</td>
<td>3.26</td>
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<td>Other elements</td>
<td>64.55</td>
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3. Static loading tests

3.1 Outline of static loading tests

Static horizontal cyclic loading tests were carried out in the north-south direction (span direction of the house) on September 17 and 18, 2004. On September 17, the loading tests were carried out to examine the initial rigidities of X1, X6, X11 and X15 frames in a small deformation state. In each loading test, a loading position was selected near X6, X11 and X15 frame. It was found from those tests that the center of rigidity of this house was close to X11 frame against the deformation in the north-south direction.

On September 18, to evaluate restoring force characteristics and to observe damage processes, static horizontal alternate cyclic loading tests were carried out by using two truck-crane as shown in Figure 4(a) and Photo 1. From the result on September 17, the loading positions were decided as denoted by the arrows in Figure 1. A sequence of loading is the following four steps: (1) pull to the north, (2) loosen until load is 0, (3) pull to the south and (4) loosen until load is 0. The loading was controlled based on the average displacement at the top of columns C11, C12 and C13. The test was carried out until the average displacement at the top of columns C11 - C13 exceeded 400mm.

On the columns C11 - C43, 64 displacements at the levels δct, δck and δch and 136 strains were measured (Figure 3). Figure 4(b) shows the position of measurements in X11 frame, for example.
### Table 2 Values on each column (mm)

<table>
<thead>
<tr>
<th>Column</th>
<th>a</th>
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<th>h</th>
<th>h₁</th>
<th>h₂</th>
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<td>4150</td>
<td>1573</td>
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### Table 3 Initial rigidities of frames (kN/cm)

<table>
<thead>
<tr>
<th></th>
<th>X1</th>
<th>X6</th>
<th>X11</th>
<th>X15</th>
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<td>stiffness</td>
<td>11.2</td>
<td>11.4</td>
<td>18.2</td>
<td>15.4</td>
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### Table 4 Average Young’s Modulus (kN/mm²)

<table>
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<td>7.56</td>
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### 3.2 Evaluation of initial rigidities of frames

From the result of small loading tests, the initial rigidity of each frame X1 - X15 was obtained. In each small loading test, the loading position was selected some one of near X6, X11 and X15 frames. Figure 5 shows the relationships between load and displacement of frames. In the load-displacement curves shown in Figure 5, the initial rigidity of each X1 - X15 frame was determined as the secant modulus at the displacement of 10mm. Here, the displacement means the average displacement at the top of three columns in each frame. Table 3 shows the initial rigidities.

From Table 3, it is found that the east part of the house is more rigid than the west part. This rigidity deflection is corresponding to the structural characteristics of the house expected by the result of structure investigation described in section 2.1. Although the X6 and X11 frames have mud-plastered hanging walls which are effective on the stiffness of the house, the height of the hanging wall in X6 frame is half of the hanging wall in X11 frame. So the stiffness of X6 frame is lower than that of X11 frame.
3.3 Load-deformation relationship and damage process

As the result of static horizontal alternate cyclic loading tests, the relationships between load and deformation angle of X1, X6, X11 and X15 frames are shown in Figure 6. The abscissa axis is the average deformation angles of three continuous columns, $\gamma_{ct}$ and $\gamma_{ck}$. The deformation angles $\gamma_{ct}$ and $\gamma_{ck}$ are calculated by the following equations:

\[ \gamma_{ct} = (\delta_{ct} - \delta_{ch})/h \]  
\[ \gamma_{ck} = (\delta_{ck} - \delta_{ch})/h_i \]  

Here, $h$ and $h_i$ are the distances between displacement sensors shown in Figure 3 and Table 2.

The deformation in the west part of the house (X1 and X6) is larger than that in the east part (X11 and X15), that is, the house was torsionally deforming. The maximum load $P_{max}$ was 106.6kN, when $\gamma_{ct}$ of X1 frame was about 0.09rad. The weight of the house is 596.8kN, and the base shear coefficient $C_0$ is then 0.18 at the maximum load. When the loading tests was finished, the resisting force was about 80kN which is 75% of $P_{max}$ and the deformation angle $\gamma_{ct}$ of X1 frame was greater than 0.11rad. The wooden frame structure of the house has a great capability in deformation while columns had been damaged by bending crack as described below.

As for damage process, while $\gamma_{ct}$ of X1 frame was less than 1/300rad, no particular damage was recognized. After more than 1/100rad, the cracks of plastered mud were recognized at the corner of mud-plastered walls on the west side of the house, and some shear cracks of plastered mud on the same walls were recognized in between 1/60rad and 1/50rad. At 1/30rad, the bending of nose-pins, which are used to connect Tohshi-Bashira columns and Sashi-Gamoi tie-beams, were getting remarkable. At 1/25rad, bending cracks began occurring just below the column and tie-beam joints.
in some *Toshi-Bashira* columns. At large deformation more than 1/10 rad, bending cracks were recognized in all *Tohshi-Bashira* columns. Photo 2 shows the bending cracks in the column C21.

### 3.4 Moment and shear force

The moments and shear forces in columns and tie-beams were calculated from strains measured on them. Figure 7 shows the strain measurement on column and tie-beam. In Figure 7, $E_c$, $I_c$, $E_k$, and $I_k$ are Young’s modulus and the moment of inertia of column and tie-beam, $\varepsilon_U$ and $\varepsilon_L$ are strains at upper and lower surfaces of *Sashi-Gamoi* tie-beam, and $\varepsilon_n$ and $\varepsilon_s$ are strains at northern and southern surfaces of *Toshi-Bashira* column. The moment of column $M_c$ and of beam $M_k$ are calculated with these strains. Here, the depth of column and tie-beam are, respectively, $d_c = 17.5$ cm and $d_k = 33.0$ cm. The average values of Young’s modulus of *Tohshi-Bashira* ($E_c$) and *Sashi-Gamoi* ($E_k$) are listed in Table 4.

Figure 8 shows the relationship between $\gamma_{ck}$ and $M_c$ of the column C21 at the position K shown in Figure 3. It is fund that before the load reached to $P_{\text{max}}$, the moment $M_c$ of C21 at K reached the maximum value 839.1 kNcm, and the moment then decreased significantly. This distinguished decrease of the moment means that the column C21 damaged. Actually, as described in the previous section 3.3, the bending cracks occurred at C21 after the deformation angle of X1 frame became larger than 1/25 rad. The similar phenomena were observed in other damaged columns C11 - C33. Although all the columns cracked when the deformation angles became large, the house as a whole was keeping the bearing capacity because the cracks of columns were held in shallow depths.

Figure 9 shows the bending moment diagrams of X1 - X15 frames at $P_{\text{max}}$. The moment of column became large just below the column and tie-beam joints, especially at the center column of the frame. The shear force borne by each column is calculated from the moments at the positions K and F in Figure 3 as follows:

$$Q = \frac{M_k - M_f}{a}$$  \[3\]

Here, $a$ is a distance between positions K and F as shown in Figure 3 and Table 2. The shear force $Q$ of each column at $P_{\text{max}}$ is shown in Figure 10. The summation of shear forces of all columns is about 40% against the load until $P_{\text{max}}$. The total shear force decreases to about 30% after $P_{\text{max}}$ and damages of columns occurred.

From Figure 10, it is also found that the the shear forces borne by the X1 - X15 frames are different, and the shear forces in the east part of the house are greater than those of the west part.

### 3.5 Torsional deformation of the house

For the case of the traditional wooden houses, the horizontal shear deformation occurs generally for lack of horizontal shear stiffness. As for the investigated house, the large torsion deformation was caused by the loading, although the loading position was very close to the center of rigidity in the house.

In Figure 11, the measured deformation $\delta_c$ and $\delta_k$ at $P_{\text{max}}$ are plotted, and the torsion deformation is shown. The average torsion angles calculated from $\delta_c$ are $0.48 \times 10^{-2}$ rad between X1 and X6 frames, $1.29 \times 10^{-2}$ rad between X6 and X11 frames, and $2.14 \times 10^{-2}$ rad between X11 and X15 frames.

### 4. Seismic performance evaluation

The seismic performance of the house was examined by the response-limit capacity analysis regulated by the Building Standard Law Enforcement Order after the loading tests. The analytical results are shown in Table 5.
In this analysis, the two criteria of seismic performance are used. One is the limit deformation against damage for moderate earthquakes of rare occurrence, and another is the limit deformation against collapse for severe earthquakes of extremely rare occurrence. The maximum deformation angles are 1/93 rad and 1/14 rad, respectively, for the moderate and severe earthquakes by using the restoring force obtained from the loading test. The criteria for limit deformation angles for the cases of traditional wooden houses are supposed, respectively, 1/100 rad and 1/15 rad. The seismic reinforcement is needed if the house leaves for use as it stands.

For the seismic reinforcement design, 1/120 rad and 1/20 rad are set for limit deformation angles. As an example of seismic reinforcements, if the deteriorated mud-plastered walls on the west gable end (X0 frame) of the house are repaired, the maximum deformation angles decrease to 1/125 rad and

<table>
<thead>
<tr>
<th></th>
<th>Present condition</th>
<th>After repair</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moderate earthquake</td>
<td>1/93 rad</td>
<td>1/125 rad</td>
</tr>
<tr>
<td>Severe earthquake</td>
<td>1/14 rad</td>
<td>1/21 rad</td>
</tr>
</tbody>
</table>

Figure 7 Strains used in calculation of moment
Figure 8 M-\(\phi\) relationship on C21
Figure 9 Moment distributions on wooden frames at \(PB_{max}\)
Figure 10 Shear force \(Q\) of each column at \(PB_{max}\)
Figure 11 Torsional deformation at \(PB_{max}\)
Figure 12 Seismic performance evaluation after repairing mud walls on X0 frame
1/21rad respectively as shown in Figure 12. The required seismic performance of the repaired house is satisfied. It is found that the traditional wooden house investigated here can be easily renovated by the seismic reinforcements, namely, repairing deteriorated mud-plastered walls, installing more seismic resisting walls or elements, and lightening the weight by removing the roofing mud.

5. Conclusion

In general, the seismic performance of old wood house, especially traditional wooden houses is considered to be low for lack of bearing walls. From the structure investigation and cyclic alternate static loading tests on the traditional wooden house built in 1898, it is found that although the base shear coefficient of the house, 0.18 is not enough because of the heavy roof, the traditional wooden frames have a large capacity in deformation and keeps the bearing force capacity as 75% of its maximum force even if large deformation angle beyond 1/10rad. The structural members are mainly mud-hanging walls and tie-beams with large cross section except the mud-plastered wall in the gable end. These structural members give the efficiency on the initial stiffness and the bearing capacity up to large deformation, even if all the columns damaged such as bending cracks.

The seismic performance of the house is evaluated by the response-limit capacity analysis regulated by the Building Standard Law Enforcement Order. The two criteria for moderate and severe earthquakes are not satisfied because the structural members deteriorated due to aging. The house can be easily renovated by repairing deteriorated mud-plastered walls or by seismic reinforcements. The results obtained here are useful to the renovation and conservation of traditional wooden houses as well as historic and cultural wooden buildings.

6. Acknowledgement

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7. References


