In-situ evaluation of timber roof structures of historic buildings

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Abstract

Condition assessment is an important step before repairs of timber structure. In the past, the experienced master carpenters carried out these works in Taiwan. This paper addresses the inadequacy of the traditional evaluation method, and proposes a new methodology to evaluate a roof structure by taking into account the structural performance. The method can be divided into three steps, first the visual inspection was conducted to record necessary information for further structural studied. Secondly, the structural analysis of the trusses was carried out in order to obtain the stresses in members. Comparing the resultant stress under loading with allowable stress, safety factor in each member was calculated. Finally, the unsafe and suspect members are designated for further nondestructive testing – the drill resistance. After considering the structural performance of the roof structure and condition in each member, the members that cannot meet the requirement will be found and recommended for replacement. Furthermore, the proper minimum section of each member that should be replaced is also proposed in this paper.

1 Introduction

The evaluation of a historic building before renovation is needed in many countries, especially seismic-prone area like Taiwan. Experience learned from
the earthquake occurred on September 21st, 1999, shows that the collapse of roof structure caused many injuries. Therefore, the issue of evaluating a historic building has attracted much attention recently. However, few papers emphasized the evaluation of roof structure, which is very important in these seismic-prone areas.

In the past, the master carpenters took the duty of evaluating the roof structures, these carpenters tapped the member surface by hammers and evaluate the condition of the member by hearing the sounds. The main disadvantage for this kind of evaluation is that this method is experience-based rather than knowledge-based, and the correctness of the evaluation often vary within the experience of these carpenters. Another disadvantage for this evaluation is that only conditions of individual element were considered. Recently, nondestructive testing technologies are involved into the roof structure evaluation. To sum up the methodology carried out in roof structure evaluation, only conditions of each member were evaluated, i.e. evaluation of global structural systems were missed in the past.

The evaluation of roof structure contains two phases, said preliminary evaluation and final evaluation phases. Usually, preliminary evaluation should be carried out to decide whether components might have urgent dangerous, and the contemporary shoring should be established. In other words, if a component cannot meet the requirement under dead load, which might have urgent danger, the contemporary shoring should be established for further assessment and ensure safety during the final evaluation and design phases. The purpose of final evaluation is to decide final condition of timber roof structure, and propose solution for this condition. This paper focuses on preliminary evaluation of roof structure, and proposes a new approach to accomplish this goal by taking into account the structural system of the whole roof structure. The procedure can be divided into three steps, including visual inspection, structural analysis, and in-situ nondestructive testing; which will be detail described later.

2 Evaluation procedure

Firstly, the visual inspection was carried out, several important parameters are recorded for following research. Secondly, base on the data from visual inspection and actual loading condition of the structure, the structural performance of the structure was examined by using finite element code, and decided the “suspected members” and “unsafe members” for nondestructive testing. Lastly, nondestructive testing was executed by using drill resistance method to check the interior deterioration and internal condition of the member. Base upon the procedure prementioned, the members that are unable to meet the requirement would be chosen.

2.1 Visual inspection

It is inevitable to carry out a visual inspection during evaluating the roof structure in the first step. The results of visual inspection can be used and
applicable during both preliminary and final evaluation of roof structure. There are several purposes for visual inspection:

- Inspect each member within the structure, including a detailed description of defect location, type, and extent, etc.
- Detailed record the geometry condition of each element, including the diameter and section information of each element. This will enable the structural analysis step, said 2nd step, to analyze a "more-real structure".

The parameters that recorded in this paper including the crack patterns, knot, the actual size of each member, visible local defects of the roof structure, and biological degradation, etc.

### 2.2 Structural analysis

The historic building is single floored and was built in 1901, which is designed as restaurant in the beginning. The superstructure of building consists of unreinforced masonry walls and timber roof trusses. In this study, only roof structure is presented. Six king post trusses compose the roof structure, which are linked together by purling. Each king post truss consists of two inclined rafters and struts, one horizontal tie beam, king post, and four inclined bracings. Figure 1 shows the roof structure and its structural modeling. The material of the members is Taiwan Cedar. Figure 2 shows the picture of the historic building studied in this paper.

![Figure 1: roof structure and its modeling](image1)

![Figure 2. Historic building studied](image2)
2.2.1 Structural modeling
Because of the high degree of indeterminacy of the roof truss system, finite element analysis was involved to study the structure performance (see Fig.3). The structure is modeled with 3-D beams and spars. The 3-D beams behave as a uniaxial element with tension, compression, torsion, and bending capacity, which have 6 degrees of freedom at each node. The 3-D spars behave as a uniaxial tension-compression element with 3-D of freedom at each node. The following assumptions were made:
1. The connections of two incline rafters and horizontal tie beam are assumed as hinges; two struts, king post, and four inclined bracings are pined with incline rafters and horizontal tie beam, which are regarded as continuous beams.
2. The weight of roof tile is assumed to be 60 kg/m², and transferred into pressure, uniformly distributed on purling. The stiffness of the roof board is eliminated.
3. Material properties are given in accordance with previous research [1], the modulus of elasticity is $8 \times 10^4$ kgf/cm², and a density of 450 kg/m³ is used in the analysis.
4. The allowable stresses of the material are 60 kgf/cm² and 45 kgf/cm² in compression and tension, respectively.

![Figure 3: 3-D structural modeling of the roof structure](image)

Since this paper focus on preliminary evaluation of timber roof structure, only dead load is considered. In the static structural analysis, the geometry of each member in roof structure is given according to the visual survey, i.e. each member in the same position may have difference section. The size ranges of each component are given in Table 1, notice that the section sizes vary largely in each member.
Table 1: Range of member sizes

<table>
<thead>
<tr>
<th>Member</th>
<th>Size (cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Incline rafters</td>
<td>143.2-380.1</td>
</tr>
<tr>
<td>Horizontal tie beams</td>
<td>113.1-298.6</td>
</tr>
<tr>
<td>King post</td>
<td>152.3-168.0</td>
</tr>
<tr>
<td>Incline bracing</td>
<td>26.4-113.1</td>
</tr>
<tr>
<td>Struts</td>
<td>74.3-128.7</td>
</tr>
</tbody>
</table>

2.2.2 Verification of modeling

The modal analysis was carried out to verify the assumption described above. The results obtained from modal analysis were compared with those from measuring the vibration of the structure. The vibration signals were recorded by using four velocity detectors, modal VSE-15D manufactured by TOKYO SOKUSHIN Company, with frequency range from 0.1 Hz to 70Hz. Spartan-L, manufactured by IMC Company, was selected as data acquisition during experiments. In order to compute the averaged Fourier spectrum of the roof structure, Fast Fourier Transfer, FFT, analyzed the signals recorded before averaging. The sampling rate is 200 Hz and the duration of the recording was long enough to eliminate any non-stationary force that might appear during the test. Table 2 shows the comparison of the natural frequencies of the first two modes of the roof structure obtained from modal analysis and measurement. The results show that the results obtained from modal analysis have good agreement with those from vibration measurement, which indicates that the assumptions taken in this paper are valid in this case.

Table 2: Comparison of natural frequency

<table>
<thead>
<tr>
<th></th>
<th>Modal</th>
<th>Experiment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Mode</td>
<td>4.79 Hz</td>
<td>4.88 Hz</td>
</tr>
<tr>
<td>2nd Mode</td>
<td>6.62 Hz</td>
<td>6.59 Hz</td>
</tr>
</tbody>
</table>

2.2.3 Results of static analysis

The results of static analysis were obtained and compared with Taiwan’s building code for wooden construction, which is under revision now[2]. Incline rafters, horizontal tie beams are subjected to combined moment and axial force, whereas the struts, king posts and incline bracings are subjected to pure axial force. According to the technical code, the members subjected to pure axial forces, and combined stresses should meet the requirements, which are listed below:

For members in compression:

\[
\frac{N}{A_g} \leq \eta \cdot f_c \tag{1}
\]
in which \( N \) is working compression force, \( A_g \) represents the gross area of member; \( \eta \) represents buckling reduction factor; and \( f_c \) is allowable compression stress.

For members in combined bending-tension:

\[
\frac{N + \eta f_c}{A_e} \cdot \frac{M}{f_b \cdot Z_e \cdot C_f} \leq f_t
\]  

(2)

where \( N \) is axial force; \( M \) represents moment; \( A_e \) is the effective area of the member; \( Z_e \) is sectional modulus; \( C_f \) is the size factor; \( f_b \) and \( f_t \) are allowable bending stress and tension stress, respectively.

For members in combined bending-compression:

\[
\frac{N + \eta \cdot f_c}{A_e} \cdot \frac{M}{f_b \cdot Z_e \cdot C_f} \leq f_c
\]  

(3)

The buckling reduction factor \( \eta \) is influenced by slender ratio, \( \lambda \), of the member, and should be calculated as:

\[
\begin{align*}
\lambda \leq 30 & \quad \eta = 1 \\
30 < \lambda \leq 100 & \quad \eta = 1.3 - 0.01 \lambda \\
100 < \lambda & \quad \eta = 3000 / \lambda^2
\end{align*}
\]  

(4)

After obtaining the stress in each member, the safety factors of each member were calculated. The safety factor represents the relation between working stress and allowable stress, if the safety factor (S.F.) value is less than 1, it means the member cannot meet the requirement of code, and is overstressed. The S.F of each stress combination is defined as:

For compression members:

\[
SF_c = \frac{\eta \cdot f_c}{N / A_g}
\]  

(5)

For combined bending-tension members:

\[
SF_{bt} = \frac{f_t}{N + \eta \cdot f_c} \cdot \frac{M}{A_e \cdot f_b \cdot Z_e \cdot C_f}
\]  

(6)

For combined bending-compression members:

\[
SF_{bc} = \frac{f_c}{N + \eta \cdot f_c} \cdot \frac{M}{A_e \cdot f_b \cdot Z_e \cdot C_f}
\]  

(7)

The safety factor distributions in each type of element are shown in Figure 4. Notice that safety factors of 39% of incline rafters are less than 1, i.e. 39% of incline rafters, totally 7 members, are overstressed under dead load. Additionally,
4 incline rafters that safety factor exactly equal to 1. As for horizontal tie beams, the working stresses in most of these members are about one third of allowable stress. The working stresses in half of the king posts are less than one third of allowable stress. The stresses in most of the struts are very small, compared with allowable compression stress. The stresses in incline bracings seldom exceed half of the allowable stress, however, one incline bracing cannot meet the requirement, 6G-2, i.e. the safety factor of that member is less than 1.

![Figure 4: Safety factor distributions in each type of element](image)

The members that have safety factor less than 1 and their stress are listed in Table 3. The negative sign in Table 3 indicates compressive stress. The position of maximum stress in these listed members are shown in Figure 3, emphasized by the circle. Notice that there might be 2 or even more unsafe points within one components, however, Table 3 only demonstrates the maximum stress points. Since the importance of the historic structure, and indetermination of the material decay, the members that have a SF less than 2, totally twenty four members, will be regarded as "suspected members" and undergone progressive nondestructive testing, which will be stated below.

<table>
<thead>
<tr>
<th>Loading</th>
<th>Member</th>
<th>Maximum Stress (kgf/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R3</td>
<td>-67.98</td>
<td></td>
</tr>
<tr>
<td>R8</td>
<td>-102.31</td>
<td></td>
</tr>
<tr>
<td>R10</td>
<td>-74.07</td>
<td></td>
</tr>
<tr>
<td>R12</td>
<td>-92.82</td>
<td></td>
</tr>
<tr>
<td>R14</td>
<td>-60.67</td>
<td></td>
</tr>
<tr>
<td>R15</td>
<td>-66.96</td>
<td></td>
</tr>
<tr>
<td>R17</td>
<td>-64.38</td>
<td></td>
</tr>
<tr>
<td>6G-2</td>
<td>-63.98</td>
<td></td>
</tr>
</tbody>
</table>

* (-): compressive sign of stress

For deflection of each member, the deflections in horizontal tie beams are usually less than those in incline rafters; the maximum deflection in the roof structure is 0.3%. After examining the deflection of each member, it is found that two of the members, R6 and R14, were deflected to 0.3% under dead load, the
deflections of the rest members are less than 0.1%. Although deflections of the members R6 and R14 do not exceed the limitation of 0.33%, the results of visual inspection show that the member R6 has a large crack. The crack in member R6 has a depth that half the diameter and the orientation is perpendicular to loading. The results of static structural analysis will be the basis for the progressive inspection.

2.3 Nondestructive Testing

In this study, the equipment RESISTOGRAPH, which measures drill resistance of wooden components was used to study the interior conditions of suspected members. These tested members include members with a safety factor less than 2, and members that suspected have interior deteriorations in visual inspection step. The purposes for this nondestructive testing are to detect the interior condition of the member and to ensure no deterioration inside the member.

The density is one of the most significant material properties of wood, which gives importance information on the interior condition and material degradation. The decay of wood density is usually considered as an indicator of the degradation of wood strength. The equipment used can detect the local density of wood, and has advantages of nondestructive, portable, and visualization of the interior condition of the members. The equipment drives a fine needle into the member and measures the resistance as it rotates, the needle has a shaft diameter of 1.5mm, and the width of the tip of needle is 3mm.

Totally thirty suspected members were tested in this paper. In each suspected member, the spacing of each drill is 30 cm. After carrying out the testing and examining the results, none deterioration was found in those suspected members. Figure 5 shows typical resistance diagram of critical point, where the safety factor is less than 1, in member R8.

![Figure 5. typical resistance diagram](image)

3 Discussion

The visual inspection is commonly conducted when evaluating a roof structure. In this case, the most evident deterioration found including cracks in member, missing of steel connectors, etc. If the orientation of the crack parallel to loading, the crack will not obviously affect the deflection; however if perpendicular to
loading, the crack will enlarge the deflection. Besides, the stress will concentrate near the tip of the crack, which will result in local failure of the member. However, the extent that how a crack affects the stress concentration and results in local failure, and enlarge the deflection need further investigation in the future.

The results of structural analysis show that none of the tie beams is overstressed. The allowable tensile stress, compressive and shear stress parallel to grain are provided in the Taiwan’s building code for wooden construction, and are also examined. However, the shear stress perpendicular to grain is not provided in code, therefore, this term was not examined in this paper. After examining the stresses in all the members, totally eight “unsafe” members were found in the roof structure.

The only one incline bracing has smallest diameter, said 5.8cm, compared with any other incline bracing, said average 9cm, therefore, the member is overstressed. Aside from the “unsafe” incline bracing, seven incline rafters are found unsafe. In these unsafe members, the positions of the maximum stress in the members are all located near the lowest purling in each incline rafter (see Figure 3). The main reason for these members to be overstressed is the inadequacy of the section, therefore, contemporary shoring should be established on these members, and after final evaluation, these overstressed members are recommended to be replaced.

The nondestructive testing technology of drill resistance can show much information that needed, however, the frequency of the drilling will strongly influence the resolution and reliability of the output of testing. In the past, a big question for inspectors and engineers is how frequent in a member should be drilled? One answer for this question is the inspectors should carry out test under every purling, since the internal forces change in these positions [3]. However, after carrying out structural analysis, only positions that have safety factors less than 2 is needed, since the vulnerability of these positions are higher than those have safety factor more than 2. Therefore, the results of the structural analysis can be the basis of nondestructive testing technology. After carrying out nondestructive testing, no additional members were found to be deterioration and should be replaced.

After carrying out structural analysis and nondestructive testing, totally eight members should be replaced due to overstressed. Table 4 lists the origin size and size that recommended size for replacing after final phase of evaluation.

<table>
<thead>
<tr>
<th>Stressed Member</th>
<th>Origin Diameter (cm)</th>
<th>Recommended Minimum Diameter (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R3</td>
<td>17.0</td>
<td>18.0</td>
</tr>
<tr>
<td>R8</td>
<td>14.8</td>
<td>18.0</td>
</tr>
<tr>
<td>R10</td>
<td>17.5</td>
<td>19.0</td>
</tr>
<tr>
<td>R12</td>
<td>13.5</td>
<td>18.0</td>
</tr>
<tr>
<td>R14</td>
<td>16.0</td>
<td>17.0</td>
</tr>
<tr>
<td>R15</td>
<td>15.5</td>
<td>17.0</td>
</tr>
<tr>
<td>R17</td>
<td>18.0</td>
<td>16.0</td>
</tr>
<tr>
<td>6G-2</td>
<td>5.8</td>
<td>6.0</td>
</tr>
</tbody>
</table>
4 Conclusion

This paper addresses some blind spots on in-situ evaluation of timber roof structures in Taiwan. Besides, a new approach that combined visual inspection, numerical modeling and nondestructive test was proposed and carried out. After static analysis, eight members were consequently found to be unsafe, and a contemporary shoring should be established to assure the safety of structure during the assessment and design phase. Besides, the shear stresses perpendicular to grain should be examined in the further study.

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References