STATIC AND DYNAMIC STRUCTURAL PERFORMANCE OF MODERN TIMBER BRIDGES

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Author Hideyuki Honda has experimented on actual modern timber bridges using glulam timber to investigate their structural rigidity as well as the static and dynamic characteristics of their structural performance for over 24 years. He has also evaluated the structural performance of modern timber bridges based on accumulated field test data and three-dimensional static and eigenvalue analyses. Atsushi Toyoda examined the structural performance of the static and dynamic characteristics based on those data. This study investigated the structural characteristics measured by field tests on 23 modern timber bridges immediately after completion, and then evaluated actual structural performance of those bridges based on the static and dynamic characteristics, such as static deflection, static rigidity, natural frequency, damping coefficient, dynamic increment factor (impact factor), vibration serviceability, and so on. The results clearly showed that the static flexural rigidity of modern timber bridges under actual service conditions was bigger than the rigidity in the design, and that the fundamental vertical natural frequency was almost equivalent to general highway bridges, such as steel and concrete bridges. New tables and figure have been added to the ones released in the past and an estimated equation of curve of the decrease of structural rigidity stiffness over time is proposed.

Key Words : modern timber bridge, structural performance, field test, structural analysis

1. INTRODUCTION

The timber bridge with glulam is considered a modern timber bridge. The modern timber bridges made with glulam timber have a 30-year history in Japan, and over 1,000 bridges of this type were constructed during this period. Though most bridges were pedestrian bridges, about 40 modern timber highway bridges were constructed. The reasons for construction were fundamentally driven by a large swell in modern population and social requirements, such as demand for expansion and effective utilization of local area materials, effective utilization of thinning wood, advances in wood processing technology, request for socioeconomic infrastructure investment in proportion to the upgrading of society and creation of new timber bridge technology. Timber is better than steel and concrete material, because it is a renewable resource aside from being eco-friendly. For the purpose of validity verification of safety and designed values, experiments were conducted in the laboratory and on actual bridges for new structural types and bridges using new material. However, the structural data from the measurements of actual bridges worldwide are insufficient for timber bridges. The experimental data on actual bridges will greatly contribute to the maintenance and technology development of modern timber bridges.

From this fact, the authors carried out static and dynamic field tests of actual bridges over about 24 years. The measured data have been collected to investigate the structural performance, structural rigidity as well as the static and dynamic characteristics of modern timber bridges in Japan\textsuperscript{13-17}. In this study, an actual condition evaluation of structural performance is investigated based on the measured values of 23 modern timber bridges and three-dimensional structure analysis. While actual bridge experiments on modern timber bridges are insufficient, this study seems to have presented worthwhile data and information to evaluate structural performance for modern timber bridges, though the dynamic performance of the bridge has been evaluated.
by measured data.

2. SUBJECT BRIDGES AND FIELD TESTS

Twenty-three subject bridges were measured in the field test. The structural types of the bridges were as follows. Girder-type: four bridges; slab-type: two bridges; deck arch-type: five bridges; half through arch-type: three bridges; through arch-type: five bridges; truss-type: two bridges; rigid frame-type: one bridge; and cable-stayed-type: one bridge. One subject truss bridge is shown in Fig. 1 as an example.

The field static and dynamic tests were conducted to investigate structural performance. The static test was static loading test with one or two dump trucks.

The static deflection values were measured by special auto level with accuracy over 0.1mm. The dynamic tests were done using tests such as (1) ambient vibration, (2) impact loading by dropping of sand bag of weight about 0.3 kN or by dropping of truck wheel, and (3) moving vehicles shown in Photo 1 as an example. In the dynamic test, servo-type velocity meter and displacement transducer were used. Simultaneously, a vehicle vibration test and measurement test of road surface roughness on bridge were also carried out. The weight of each vehicle used in the field test was measured beforehand. The measured data were checked and stored in a digital recorder and personal computer, and then the static and dynamic behavior and characteristics of the bridge were analyzed as the monitoring system of the field test.

(1) Static deflection shape

Fig. 2 shows an example of measured deflection and deflection shape analyzed by NASTRAN for half through arch bridge (Ohsaru Bridge). The measured values are the results in case 1-1 at $l/4$ point loading with one truck of 195.2 kN. Generally, structural characteristic of an arch type bridge is clearly demonstrated; negative deflection is generated at $3l/4$ point when the positive deflection of verticality is generated at $l/4$ point span. Since the deflection shape shows symmetry, the flexural rigidity in the bridge axial direction is deemed to be uniformly distributed. As a result, based on the investigation of static deflection shape of various types of modern timber bridges, mechanical characteristics of the structural types have individually been confirmed from the deflection shape. It appears that the structural characteristics of modern timber bridges are similar to general steel bridges and concrete bridges.

(2) Maximum static deflection

Fig. 3 shows the relationship between span length and maximum static deflection obtained in the test of each bridge. In addition, even though it was difficult to determine exactly the relationship between static deflection and load capacity, because the load capacities of the dump truck used in the field test of each timber bridge were different, the relation between span length and maximum static deflection standardized by load capacity of the 196kN truck in order to determine provisional tendency, is also illustrated in this figure.
From this figure, although it can be assumed that the static deflection increases in the two trucks rather than in one truck loading and that the static deflection also increases with the increase in span length, it is recognized that the static deflection does not double even when the load capacity becomes approximately double. It also shows that the maximum deflection of modern timber bridges with span length of 25–50 m is 6–7 mm at one truck loading, and that the static deflection of through arch bridge without stringer is larger than that of the deck arch bridge due to the difference between their deck systems. In addition, when the load strength is standardized at 196 kN, though there is a fluctuation within 20–30m, the span length and maximum static deflection also tend to have a linear relation. Though it is difficult to predict the deflection value by structural type based on the data number in this case, the maximum static deflection can be estimated from the static rigidity. This corroborates with previous static field tests on modern timber bridges using 196 kN loading. Moreover, the subject modern timber bridges are also proven to have static flexural rigidity similar to previous results, if the prediction deflection is almost equal to the experiment deflection.

(3) Static flexural rigidity

In investigating the static deflection shape and maximum deflection of the preceding item, the experimental values and analytic values agree with high accuracy. From this fact, the actually retained static rigidity seems to have been reproduced in the structure analysis. Based on this knowledge, the design and actual static rigidity were investigated accordingly. With reference to Suginoki Bridge, Fig. 4 shows the live loads of \( p_1 \) and \( p_2 \) with equal load capacity used in the design calculation of the primary member applied in a three-dimensional structure analytical model. The crown point of the arch rib was used as the reference point, and then the deflection value in the design was compared with the deflection value of the analytic solution. As a result, the designed value of the design live load at the reference point was 7.0 mm, and the analyzed value was 4.2 mm, with the analytic value being 60% less than the designed value.

It appears that the static flexural rigidity of the actual bridge was 1.7 times larger than that of the design. The investigation on the static flexural rigidity for the other modern timber bridges is shown in Table 1. The actual condition of static flexural rigidity in either modern timber bridge may be larger than that in the design. Therefore, it is considered that the design of member for allowable stress has not been carried out sufficiently in modern timber bridges.

4. DYNAMIC CHARACTERISTICS

(1) Vibration mode

Investigation of dynamic characteristics of modern timber bridges has been carried out by the authors. Table 2 shows natural frequency, vibration mode, damping coefficient, and dynamic increment factor, as vibration characteristics obtained through field tests and the analyzed values by structural analysis for eight modern timber bridges as samples. Fig. 5 shows the analyzed vibration. In addition, it shows the analyzed vibration mode in proportion to the fundamental natural frequency of modern timber bridges, as examples of the first to third vibration modes. The arch height and truss height in relation to span length of modern timber bridges of the arch and truss type are generally larger than that of steel bridges and concrete bridges. Therefore, the horizontal vibration mode tends to appear in the first vibration of truss and arch bridges. Attention is necessary for the design of connection, because the load by horizontal behavior of the main truss will be concentrated at the connection part of the lower chord member. In deck, half-through, and through-type arch bridges such as the Kaminomori Bridge, Ohsharu Bridge, and Kinpo2000 Bridge, the property of the arch bridge is recognized in which the first vertical vibration becomes anti-symmetric. However, there are also cases in which the first vertical vibration by rigidity allocation between the floor structure and arch rib becomes symmetric, and the second vibration is anti-symmetric such as that of the arch-type Yahata Bridge. There is also the case in which although the first torsional vibration becomes anti-symmetric during the first vertical vibration, the first torsional vibration becomes symmetric such as that of the arch-type Suginoki Bridge. In the \( \pi \) type rigid-frame Midori Bridge, the first vertical vibration becomes anti-symmetric similar to that of the arch.
bridge. Maruyamasawa Bridge of the hybrid girder type, in which a wire rope for pre-stressing is inserted in the glulam girder, fundamentally shows the behavior of girder and slab structure because horizontal load sharing is only conducted through the RC deck system. Therefore, there is a peculiar vibration in modern timber bridges as a result of different vertical and horizontal load sharings, because of the difference. Therefore, the following figures and tables give only the results of 6 to 8 large scale bridges which have different structural types as a representative example.

It is noted that due to space limitation the results of the tests on all the 25 bridges cannot be presented here. Therefore, the following figures and tables give only the results of 6 to 8 large scale bridges which have different structural types as a representative example.

Table 1 Actual condition of static flexural rigidity.

| Bridge names          | Bridge types | Design deflection (mm) | Analyzed deflection (mm) | Actual static rigidity (times) |
|-----------------------|--------------|------------------------|--------------------------|--------------------------------|
| Kinokakehashi Bridge  | Slab         | 15.1                   | 3.9                      | 3.9                            |
| Midori Bridge         | Rigid frame  | 16.8                   | 6.0                      | 2.8                            |
| Suginoki Bridge       | Deck arch    | 7.0                    | 4.2                      | 1.7                            |
| Kinpo 2000 Bridge     | Deck arch    | 6.0                    | 3.6                      | 1.7                            |
| Ohsaru Bridge         | Half through arch | 10.3   | 8.7                      | 1.2                            |
| Maruyamasawa Bridge   | Girder       | 14.0                   | 7.3                      | 1.9                            |

Table 2 An example of dynamic characteristics for modern timber bridges.

| Bridge names      | Const. data | Structural types | Design Live load | Span (m) | Width (m) | Vibratio n No. | Natural frequencies (Hz) Measured | Analyzed | Damping coefficients | Dynamic increment factors |
|-------------------|-------------|------------------|------------------|----------|-----------|----------------|-----------------------------------|----------|---------------------|--------------------------|
| Kaminomiya Bridge | 1994        | Half through arch | TL-20            | 23.0     | 5.0       | 1              | 1.08                             | 0.85     | 0.002               | 0.198                    |
| Modori Bridge     | 1996        | 5 type rigid frame | 25(A)            | 28.0     | 7.5       | 1              | 1.08                             | 0.85     | 0.002               | 0.180                    |
| Kinpo 2000 Bridge | 2000        | Deck arch        | 25(A)            | 36.9     | 10.1      | 1              | 1.08                             | 0.85     | 0.002               | 0.180                    |
| Ohsaru Bridge     | 2001        | Half through arch | 25(A)            | 25.6     | 5.4       | 1              | 1.08                             | 0.85     | 0.002               | 0.168                    |
| Kakinohara Bridge | 2003        | King post towers  | 25(A)            | 48.2     | 7.0       | 1              | 1.08                             | 0.85     | 0.002               | 0.204                    |
| Yahata Bridge     | 2003        | Through tied arch | 25(A)            | 19.6     | 5.0       | 1              | 1.08                             | 0.85     | 0.002               | 0.173                    |
| Maruyamasawa Bridge | 2003       | Girder inserted steel plate | 25(A)         | 19.0     | 7.0       | 1              | 1.08                             | 0.85     | 0.002               | 0.215                    |
| Yoshiba Bridge    | 2007        | Half through arch | 25(A)            | 28.0     | 7.0       | 1              | 1.08                             | 0.85     | 0.002               | 0.085                    |
(2) Natural frequency and span length

Generally, the fundamental natural frequency in the vertical direction of bridges decreases with the increase in span length. The decrease is estimated roughly by the equation \( f_1 = \frac{100}{L} \) (Hz), where the fundamental natural frequency is \( f_1 \) and maximum span length is \( L \) (m) \(^{(18)}\). Fig. 6 shows the relationship between vertical-fundamental natural frequency and maximum span length obtained by field test on modern timber bridges. From this figure, although there is a little dispersion in short span length, modern timber bridges have characteristics equal to steel or concrete bridges. When it is considered from the viewpoint of the vertical fundamental natural frequency, it is evaluated that the modern timber bridges have been designed for rigidity that is almost

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**Fig. 5** Fundamental natural frequency and vibration mode by eigenvalue analysis.
equivalent to those of steel or concrete bridges.

Why does the first natural frequency of modern timber bridges fall in the category of general bridges? The reason is considered here. When the Young’s modulus of structural glulam timber is 9.8 kN/mm² as a general value, this value is about 1/2 of concrete and about 1/20 of steel. When the unit product weight of structural glulam timber is 7.85 kN/m³, the structural glulam timber is light; about 1/3 of concrete and about 1/10 of steel.

Therefore, although it is difficult to do a quantitative evaluation, the geometrical moment of inertia of member increases because the Young’s modulus and unit product weight of modern timber bridges are small. As a result, the flexural rigidity (EI) of modern timber bridges seems to become equivalent to that of the general bridges. Therefore, it is possible for the flexural rigidity of modern timber bridges to become equivalent to that of general bridges.

Fig. 7 shows the relationship between natural frequency and vibration number. Though there is a tendency of linear increase in both relations, a quantitative evaluation is difficult since the data number is small.

(3) Damping coefficient

The calculation of the damping coefficient (h) is calculated by the equation \( \delta = \ln(X_i + 1/X_i) \), \( h \approx \delta/2\pi \), where \( \delta \) is logarithmic damping coefficient, and \( X_i \) is amplitude. The relationship of span length and damping coefficient (\( h_i \)) at vertical first natural frequency of general steel and concrete bridges is estimated roughly by the equation \( h_i = 0.12/\sqrt{L} \), where \( L \) (m) is maximum span length. **Fig. 8**

(4) Damping coefficient and natural frequency

Fig. 9 shows the relationship between damping coefficient (\( h_i \)) and natural frequency (\( f_1 \)) of the vertical first vibration obtained by field test. The value of damping coefficient may also increase, when the fundamental frequency increases. This is because there is a correlation that is resistant to span length and fundamental frequency.

(5) Damping coefficient and vibration number

Fig. 10 shows the relationship between damping coefficient and vibration number. Although the damping coefficient tends to decrease when vibration number increases, a general judgment will be
difficult in the current study. An investigation by analysis is nearly impossible for the evaluation of damping coefficient. Therefore, for further investigation of damping coefficient, many glulam timber bridges should be tested in the future, and the evaluation must be quantitatively done by statistical analysis based on accumulated data.

(6) Impact factor

The dynamic increment factor \(i\) is calculated by the equation \(i = \frac{\gamma_{dy,\text{max}} - \gamma_{st,\text{max}}}{\gamma_{st,\text{max}}}\), where \(\gamma_{dy,\text{max}}\) is maximum dynamic deflection and \(\gamma_{st,\text{max}}\) is maximum static deflection. The impact factor used in bridge design is a substantial load factor which increases the design live load. On the other hand, this impact factor seems to respond to the dynamic effect which a moving load gives to the bridge. Considering this impact factor to be a response coefficient is an effective analytical approach in modern timber highway bridges. This is because timber highway bridges with short span length reach full loading condition with merely one or two vehicles. Therefore, it makes sense not to consider moving vehicle condition as design live load, rather just to consider the dynamic effect on the bridge due to daily traffic flow, which leads to the practical approach of evaluating the impact factor as a dynamic response coefficient.

Fig. 11 shows the relationship between maximum span length and dynamic increment factor (impact factor) obtained in the field test. The equation \(i = \frac{16}{30 + L}\) shown in the figure is a design impact factor for steel highway bridges in Japan since 1940 regardless of structural type and span length. It can be recognized, from the above figure, that some modern timber bridges with short span exceed the design impact factor of 0.25. As a result of investigation on the design impact factor, although basic data for investigation are not sufficient at present, the equation \(i = \frac{16}{30 + L}\) shown in this figure is proposed by JSCE as design impact factor for modern timber highway bridges in Japan.

Fig. 12 shows the relationship between dynamic increment factor (impact factor) and vertical first natural frequency obtained by field test. In the future, the impact factor based on vertical first natural frequency as it is adopted by the Ontario Highway Bridge Design Code in Canada is expected to become main-stream worldwide.

However, since necessary basic data for investigation are insufficient, it is currently difficult to find a code for the design impact factor like that of the Ontario State, and accumulation and analysis of the data on dynamic increment factor for modern timber highway bridges are necessary in the future.

(7) Maximum response acceleration

Fig. 13 shows the relationship between maximum response acceleration and span length obtained in the field test of modern timber highway bridges. There is a value of about 250 cm/s\(^2\) in some arch bridges. This is the effect of the impact force from the vehicle at extension joint part. Most mod-
ern timber bridges tend to be within the general bridge value similar to other highway bridges, because they are mostly within 50 - 180 cm/s². From this figure, it is deemed that the maximum response acceleration may not particularly increase even though modern timber bridges are lighter than steel or concrete bridges.

(8) Maximum response velocity (vibration serviceability)

Fig. 14 shows the relationship between maximum response velocity and span length obtained in the field test. The maximum response velocity obtained in the field test is mostly within 0.4 – 2.4 cm/s. ○ and ■ symbols shown in this figure refer to modern timber pedestrian bridges. Response velocities of pedestrian bridges are the value measured in resonance state induced by one person walking perpendicularly to the direction of the vertical first natural frequency.

Especially, vibration serviceability is an important issue for pedestrian bridges. In Japan, there are many cases in which they are investigated by response velocity based on the research by Kajikawa and Kobori on vibration serviceability for highway bridges and pedestrian bridges. There are two types of evaluation methods for vibration serviceability. One is the method using the maximum value of response velocity, and the other method is the method using root mean square (RMS) value of response velocity.

Table 3 shows the limit values of response velocity for vibration serviceability. Vibration serviceability of bridges is investigated by the limit values of 2.4 cm/s for maximum value and of 1.7 cm/s for RMS value. These limit values show the category equivalent to when it is lightly hard for the pedestrian to walk, when he senses bridge vibration.

Fig. 15 shows an example of the investigated vibration serviceability for the hybrid truss timber highway bridge. Based on the results of vibration tests for a large number of modern timber bridges, in the bridges that exceeded the limit values of 2.4 cm/s and 1.7 cm/s, vibrations could hardly be recognized in usual moving vehicle speed. Presently, there seems to be no problem regarding vibration serviceability for modern timber bridges under daily traffic volume.

5. STRUCTURAL RIGIDITY OVER ELAPSED YEARS

Soundness of modern timber bridges practices was monitored over many years, and the changes in fundamental natural frequency in horizontal direction obtained from vibration test were investigated. It is considered that the condition of structural rigidity in the entire bridge can be represented by the fundamental natural frequency due to the fact that this fundamental natural frequency reflects the influence of the local degradation of wood over time.

| Content of category                        | Limit values (cm/s) |
|--------------------------------------------|---------------------|
|                                           | Max. values | Max. values |
| Not perceptible                            | 0.6 under  | 0.42 under  |
| Light perceptible                          | 0.6 – 1.2  | 0.42 – 0.85 |
| Definitely perceptible                     | 1.2 – 2.4  | 0.85 – 1.7  |
| Lightly hard to walk                       | 2.4 – 3.8  | 1.7 – 2.7   |
| Extremely hard to walk                     | 3.8 above  | 2.7 above   |

Table 3 Limit values of response velocity

For vibration serviceability. [21]

Fig. 13 Maximum response acceleration.

Fig. 14 Maximum response velocity.

Fig. 15 Vibration serviceability.
From this, the fundamental natural frequency is defined as the structural rigidity of the entire bridge in this study.

Fig. 16 shows the decreasing rate of structural rigidity of ten modern timber bridges over fifty years. The horizontal axis refers to the elapsed years, and the vertical axis is the decreasing rate of structural rigidity. Change is not seen in the first ten years after modern timber bridge is constructed. However, after ten years have passed, the structural rigidity is seen to decrease rapidly. The estimated equation of curve is proposed based on the obtained result in the figure. The accuracy of this predicted decreasing rate of structural rigidity can be increased by accumulating more data in the future.

6. CONCLUSIONS

This study investigated the structural characteristics of 23 timber bridges measured by field tests, and then evaluated actual structural performance for glulam timber bridges based on the static and dynamic characteristics such as static and dynamic performance. As the result, the actual condition of modern timber bridges became clear; that is, the static flexural rigidity was larger than the rigidity in the design, and that the fundamental vertical natural frequency was almost equivalent to that of general highway bridges, such as steel and concrete bridges.

The content investigated in this study seems to have presented worthwhile test data and information to evaluate structural performance and to develop a design method for modern timber bridges.

REFERENCES

1) Honda, H., Fujino, Y. and limura Y.: Structural rigidity of cable stayed bridge using glue laminated timber based on test and analysis, The 15th IABSE Congress Report, pp. 1191-1192, 1996.
2) Usuki, S., Sasaki, T., Iijima, Y. and Honda, H.: Field dynamic performance of stress laminated continuous timber deck bridge, Proc. of the SEWC, T134-2, 1998.
3) Honda, H., Usuki, S., Iijima, Y. and Mishina, Y.: Field test and analysis of four spans continuous SLTD highway bridge, Proc. of the WCTE, Vol. 1, pp. 746-747, 1998.
4) Mishina, Y., Usuki, S. and Honda, H.: Timber road bridges in Japan and their structural characteristics, Proc. of the WCTE, Vol. 2, pp. 93-99, 1998.
5) Sasaki, T., Honda, H. and Usuki, S.: Field tests and analysis of π-type rigid frame timber highway bridge, Proc. of the 1st RILEM Symposium on Timber Engineering, pp. 223-232, 1999.
6) Sasaki, T., Usuki, S., Honda, H. and Sudo, S.: Field tests and in-service performance of the new glulam tied arch roadway bridge (Hyakume-ishi), Proc. of the WCTE, pp. 8-3-4, 2000.
7) Sasaki, T., Usuki, S., Sharma, M., Iijima, Y., Honda, H. and Atsumi, A.: Field experiment of a hybrid timber-steel deck roadway bridge, Proc. of the IABSE Conference LAHTI 2001-Innovative Wooden Structures and Bridges, Vol. 85, pp. 181-186, 2001.
8) Honda, H., Usuki, S., Sasaki, T. and Mishina, Y.: Structural performance of deck arch timber highway bridge, Proc. of the IABSE Conference LAHTI 2001-Innovative Wooden Structures and Bridges, Vol. 85, pp. 187-192, 2001.
9) Honda, H., Usuki, S. and Sasaki, T.: Structural characteristics of deck arch timber highway bridge, IABSE Symposium Melbourne 2002 Report, Vol. 86, pp. 1-8 (CD-R), 2002.
10) Honda, H., Kusanagi, T. and Kabemura, H.: Structural performance of timber arch highway bridge, Proc. of the WCTE, pp. 259-264, 2004.
11) Honda, H. and Sasaki T.: Dynamic performance of glulam timber arch highway bridge, Proc. of the EVACES 05, pp. 553-561, 2005.
12) Honda, H., Sasaki, T. and Usuki, S.: Structural performance of hybrid timber truss highway bridge, Proc. of the WCTE, pp. 125-132, 2006.
13) Kusaka, M. and Honda, H.: Vibration serviceability of large-scale timber pedestrian bridge, Proc. of the EVACES 09, pp. 483-490, 2009.
14) Honda, H.: Structural performance of large timber king post truss highway bridge, IABSE Symposium Venice 2010 Report, Vol. 32, pp. 1-8 (CD-R), 2010.
15) Honda, H.: Structural performance of hybrid timber girder highway bridge, IABSE-IASS 2011 London Symposium Report, pp. 1-8 (CD-R), 2011.
16) Ministry of Transportations and Communications Highway Engineering Division: Ontario Highway Bridge Design Code, 34, 1983.
17) Honda, H.: Dynamic performance of timber truss pedestrian bridge using steel ball joint, IABSE Symposium Madrid 2014 Report, pp. 1-8 (CD-R), 2014.
18) Bridge Vibration Monitoring Committee: Structural Engineering Series 10: Guidelines for Bridge Vibration Monitoring, Japan Society of Civil Engineers (JSCE), 2000 (in Japanese).
19) Honda, H., Kobori, T. and Yamada, Y.: Dynamic factor of highway steel girder bridges, IABSE Proceedings, P-98/86, pp. 57-75, 1986.
20) Technical Committee of Timber Bridges of JSCE: Technical Guideline on Timber Bridge 2005 - Part of Design of Timber Bridge, Japan Society of Civil Engineers, 2005 (in Japanese).
21) Kajikawa, Y. and Kobori, T.: Ergonomic evaluation methods for bridge vibration, Journal of the Structural Mechanics and Earthquake Engineering, JSCE, No. 230, pp. 65-73, 1974.
22) Shinohara, M., Toyoda, A., Kato, S. and Honda, H.: Structural rigidity based on dynamic performance of timber bridges by passing age, Abstracts of the 17th Symposium on Use of Wood for Civil Engineering, JSCE, pp. 42-47, 2018.

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