A new timber beam bridge with an orthotropic steel deck

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ABSTRACT

The construction of a new type of timber bridge is planned and now manufactured in the Akita Prefecture, Japan. The bridge is on a forest roadway and has the bridge length of 55.0 m with two continuous spans and the total width of 12.3 m with a 7.0 m two lane roadway and a 2.0 m one-side footway. The orthotropic steel deck is attached to two double beams to preserve perfectly beams from rainfall and snowfall. The composite mechanism between steel deck and glulam beams, the enlargement of beam width near intermediate support, the special cross section of supporting members and shear stresses in beams are presented.

INTRODUCTION

A renaissance of wooden bridges arose in mid 80’ with the appearance of a modern timber bridge utilizing engineered wood material, in Japan. Nowadays, several tens of timber bridges are believed to be constructed in a year. Although most of them are for pedestrians, road bridges count seventeen until now.

The construction of a new type of timber bridge is planned and now manufactured in the Akita Prefecture, Japan. The bridge site is located at an entrance into the Sirakami Mountains registered as one of the World Heritage. This is the main reason why the timber bridge is employed, rather than a pre-stressed concrete or a steel bridge. An old, deficient steel girder bridge will be replaced by a new timber bridge.

The bridge is on a forest roadway and has the bridge length of 55.0 m with two continuous spans and the total width of 12.3 m with a 7.0 m two lane roadway and a 2.0 m one-side footway. Two double glulam beams with center to center spacing of 8.55 m are used due to low head room and the beam depth of 1.0m is kept constant along the bridge length. The orthotropic steel deck is attached to beams to preserve perfectly beams from rainfall and snowfall. The reinforced concrete deck is rejected due to its heavy dead load and to design requirement of low head room crossing.

The beams of each span is stiffened by supporting timber structure that is similar to the King post truss used in Finland (Gustafsson, 1998). In this paper, the composite mechanism between steel deck and glulam beams, the enlargement of beam width near intermediate support, the special cross section of supporting members and shear stresses in beams are presented.

THE DESIGN PROJECT DESCRIPTIONS

The proposal for a new type of timber bridge in the Akita Prefecture located in the northwestern of Japan is not the result of a design competition. The local government of Akita Prefecture has made an optional contract with the Japan Forest Civil Engineering Consultants Foundation. This is due to the fact that the bridge is located on the forest roadway of Yonesiro Route in Akita Prefecture and that the Foundation has some experienced designer on timber bridges.

The Akita Branch of the Foundation organized two kinds of committees. One is to hearing on the questions about the type of timber bridge, service life and maintenance needs, from the citizens of Fujisato city in which the bridge is constructed.

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The other is to discuss detailed design procedures on the bridge adopted in the former committee and the committee member consists of bridge designers of the Foundation, researchers of the Institute of Wood Technology, Akita Prefectural University and professors of the Department of Civil Engineering, Akita University.

As a result of a public tender, the Japan Steel Works Ltd. has been awarded a contract. The productive members of the company join in the later committee and discuss the field welding method of steel deck plates and of inserted steel plates of glulam timber beams, at joints.

**BRIDGE SYSTEMS**

Over the past decade, emphasis has been places on improving design efficiency related to structural and serviceability performance, economics, and longevity. Emphasis has especially been placed on improving longevity. In Japan, timbers treated by CCA and the creosote are considered not to be environment-friendly. Then the work in the committee is focused on wood protection, especially wood protection by design and use of environmental-friendly chemicals like AAC. This is one of the reason why the orthotropic steel deck is employed. In addition, it is considered steel products are not expensive in Japan, as an industrial country.

**General Views**

An old, deficient steel and reinforced concrete girder bridge constructed before forty years will be replaced by a new timber bridge. Figure 1 shows elevation. The total length of the bridge is 55m having continuous two glulam timber beam spans of 27 m each and the width of the roadway 7.0 m. The supporting timber structure is not truss frame but rigid frame with pin connection at centroids of beams. The vertical and diagonal timber members are connected by pins at centroids of supporting members and those of beams.

![Fig.1  Side view](image)

The beams consist of three kinds of beams (from beam ① ③ with different lengths, as shown in Fig.1. One beam of double beams,① ② is 350 mm wide and 1000 mm depth. The clear span of double beams is also 350 mm and the lower parts of the supporting structure, vertical and diagonal members are inserted into this clear span. The lower parts of these structure and members are stiffened by steel box and connected with beams by pins penetrating double beams. The cross section of double beam ③ is different from that of double beams,① ②.

The supporting structure consists of AAC-treated poles with a length of 14.7 m. The poles are supported laterally at n e a r the pin connection and cross-braced at the top, as shown in Fig.2. The vertical, diagonal members and cross beams are also treated by AAC. In Fig.2, the orthotropic steel deck is supported by two double glulam beams. This is due to the lack of head room.

The orthotropic steel deck plate with thickness of 12 mm is stiffened by U-shaped longitudinal ribs and L-shaped transverse ribs. The later is arranged with a typical interval of 2.075 m. The steel deck is designed for truck loading with one axle of 196 kN in deck bending and for distributed live loading in beam bending, as the deck acts as the top flange of double beams.
An orthotropic steel deck will not normally compete in cost with reinforced concrete if viewed only as a slab. It may be competitive if allowance is made for its axial force capacity and reduced dead load (1.86 kN/m², whereas 6.13 kN/m² for a concrete deck), such as this bridge. The side walk steel deck with the width of 2.0 m is also designed for a uniformly distributed load of 4.9 kN/m². This is supported by tapered glulam beams in which vertical top and bottom steel plates are inserted and the top plates are welded to the bottom of steel deck (see also Fig.6). Steel brackets are used only at abutments, as shown in Fig.2.

The glued-in ribbed steel plates on the upper surface of the beams transfer the shear forces between deck and beams. In other words, the ribbed steel plates welded to the bottom surface of steel deck plate act as shear connecters. The glued-in ribbed steel plates on the bottom surface of the beams compensate the longitudinal axial strength of beams. Thus a part of the steel deck with an effective width, the ribbed steel plates and the double glulam beams constitute a composite beam and we can use the composite beam theory to calculate the bending and the shearing stresses in the beams.

The reactive forces acting at abutments and pier are 1,127 kN and 2,440 kN, respectively, for the double beams attaching side walk. For these reactive forces, elastometric bearing pads are used to allow seismic movements between the superstructure and the substructure, as shown in Fig.2. The lengths of sole plates along the beams for uniformly distributing the concentrated reactive forces are 800 mm and 920 mm, for abutments and pier, respectively. These are needed to assure bearing areas sufficient to give adequate safety with respect to crushing of the supporting glulam beams. The allowable normal stress perpendicular to grain is only 196 N/cm², for this material.

Cross Sections of Members
Fig. 3 (a)-(c) show cross sections of supporting structures, vertical members and diagonal members in Fig.1, respectively. Two kind of laminar glued edge to edge are used to form the member width. The thickness of each laminar made of Japanese-cedar is 30 mm. The supporting member shown in Fig.3 (a) is stiffened by four inserted steel plates. The width and the depth of each slit are larger 3mm and 1 mm than those of steel plates, respectively. The steel plates are bonded by epoxy resin after removing mill scale by sand-blasting. Satisfactory composite behaviors in compressing and bending are observed due to reduced model tests in laboratory.
For example, the design axial force and bending moment of supporting member, a-b in Fig.1 are 1,480 kN and 27.4 kNm, respectively (at pin end a, the bending moment vanishes). Members subjected to both flexure and axial compression shall be so proportioned that

$$\frac{f_c}{F'_c} + \frac{f_b}{F'_b} \leq 1$$  \hspace{1cm} (1)

where \(F'_c\) is modified allowable compressive stress, determined by using the slenderness ratio and \(F'_b\) modified allowable bending stress, determined by using slenderness factor or size factor. \(f_c\) and \(f_b\) are actual unit stresses in compression and in bending, respectively. Using the concept of transformed section and converting all steel to an equivalent wood area, the ratio, factors and actual unit stresses are calculated. Allowable stresses and moduli of elasticity are as follows:

<table>
<thead>
<tr>
<th>Steel (SM400) (kN/cm²)</th>
<th>Timber (kN/cm²)</th>
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</thead>
<tbody>
<tr>
<td>(E_s = 20.6 \times 10^3)</td>
<td>(E_w = 735)</td>
</tr>
<tr>
<td>(\sigma_{ba} = 13.7)</td>
<td>(F'_b = 0.78)</td>
</tr>
<tr>
<td>(\tau_a = 7.8)</td>
<td>(F_b = 0.088)</td>
</tr>
<tr>
<td>(\sigma_{ba} = 13.7)</td>
<td>(F_{c\perp} = 0.196)</td>
</tr>
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</table>

In Table 1, \(\sigma_{ba}\) is allowable bearing stress and the steel of SM400 is most used for this bridge, including the steel deck. The ratio of moduli of elasticity to obtain modified section is

$$n = \frac{E_s}{E_w} = \frac{20.6 \times 10^3}{0.735 \times 10^5} = 28$$

This value of \(n\) is not an actual one but a design one and the former will be larger than this, as the modulus of elasticity \(E_s\) is estimated comparatively lower. For the section in Fig.3 (a), it is sufficient to check Eq.(1) in the plane of bending and is not necessary to check a design formula for steel beam-column, because the ratio of cross sectional steel area to its timber area is small (4.9%, Usuki, et.al.2000). The over all lateral buckling of supporting structure is also checked, in which the structure is supported laterally and elastically, at nodes by vertical and diagonal members.
The design axial force of vertical member in Fig.3 (b) is 1,068 kN (tension) and that of diagonal member in Fig.3 (c) is 53.9 kN (compression). In Fig.3 (b),(c), the x axis is directed to roadway width. Fig.4 shows the cross section of cross-bracing beams at near the top of supporting structure. Two beams are attached at near the top and tightened together with the supporting structure by pre-stressing rods with a diameter of 26 mm. The pre-stressing is needed to form a rigid frame that has a high load-carrying capacity to resist lateral buckling of supporting structure.

**Framing Plan**

The framing plan is shown in Fig.5. U-shaped longitudinal ribs shown in Fig.2 are abbreviated in the figure. The lines of SSP-1 etc. show longitudinal field welding lines of deck plate and the lines of J1 etc. show transverse field welding lines of deck plate and of glulam beam joints, as mentioned on Fig.1. Steel box section beams are used for the end floor beams of S1 and S2 and the floor beam P on pier, instead of \( \perp \)-shaped transverse ribs.

Two double beams, G1 and G2 in Fig.5 consist of three kinds of beams (from ① to ③), as mentioned earlier. The cross sections of beam, ① ② are as shown in Fig.2. The beam width of double beams ③ varies from 350 mm to 525 mm at near the beam ends, with a gradient of 1:5, but have same beam depth of 1000 mm as beams, ① ②. The beam length having the width of 525 mm is 7.6 m and this width is needed to overcome shear forces developed by reactive forces on pier. In addition, low head room of the bridge sight does not permit the variation of beam depth.

**Fig.5 Plan view**

The cross section of beam G2 on intermediate support is shown in Fig.6. The widths, \( \lambda_2 \) and \( \lambda_3 \) of steel deck are effective width obtained by applying Japanese shear lag formula for roadway bridge. The effective width is lineally widened from

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**Fig.4 Section of Cross-bracing**

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the intermediate support to points at distance of 0.2 \( L \) from the intermediate support (\( L = 27.0 \) m). From those points to supports (\( S_1 \) and \( S_2 \) in Fig.5) on abutments, uniform cross section having effective widths of Error! 2,891\( \text{mm} \) and Error! 2,219\( \text{mm} \) is held and a beam width in this section is 350\( \text{mm} \) as shown in Fig.2. The superstructure is analyzed as a planar framing structure by the use of computer program available.

**Fig.6 Cross section at intermediate support**

**SHEARING STRESSES IN BEAMS**

The design bending moment occurs at intermediate support and the shear is checked at the position with a distance from the intermediate support equal to the depth of beam. These values for beam G2 with a side walk are as follows;

\[ M = -2,087 \text{kNm} \]
\[ V = 696 \text{kN} \]

The shear flow \( q \) due to transverse shear \( V \) is shown in Fig.7 (a). The cross section of double beams at this position considered is modified as an axe symmetric cross section with a deck plate thickness \( t \). The thickness \( t \) is determined so that the moment of inertia is equal to that of actual cross section having longitudinal ribs and effective widths at this position.

**Fig.7 Shearing stresses**

Denoting the shear stress by \( \tau_b \), the shear flow \( q \) is \( \tau_b b \) in deck plate and \( \tau_b b \) in glulam beam where \( b \) is beam width. Using \( I_v = 1.73 \times 10^7 \text{cm}^4 \) as the moment of inertia transformed to timber, we obtain maximum shear stress of 87\( \text{N/cm}^2 \) at
the centroid of cross section. This value is slightly lower than allowable along-the-grain (horizontal) shear stress \( F_v = 88 \text{ N/cm}^2 \) in Table 1. It is cited that the shear stress distributes uniformly in the direction of beam width, because no shear strain is accounted at this stage.

The shear stress acting on the surface of inserted steel plate is also obtained, by considering equilibrium equations for the element of steel plate. Denoting the shear stress in the direction of beam axis (x-axis) by \( \tau_{zx} \) and assuming its linear distribution in the direction of plate depth (y-axis), we obtain results as shown in Fig.7 (b). The absolute stress intensity on the upper plate is lower than that of lower plate, because the upper plate is near the centroid of cross section (the cross section rotates about z-axis). In fact, the shear stress on upper plate vanishes at the centroid.

The absolute maximum value of \( \tau_{zx} = 41 \text{ N/cm}^2 \) occurs at the bottom of lower plate and this is lower than \( F_v = 88 \text{ N/cm}^2 \) in Table 1. In Fig.7 (b), the depth of upper steel plate varies linearly near this position. The upper plate has to be pre-drilled together with double beams, to pass through a pin on the intermediate support.

REFERENCES
