Glued laminated timber and orthotropic steel deck hybrid beam: ultimate bending strength

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ABSTRACT The ultimate bending strength of a non-symmetric glulam beam-orthotropic steel deck hybrid bridge model was investigated. The composite beam is composed of an orthotropic steel deck, attached to a double glulam beam, having rectangular cross sections. The beams are stiffened by two, vertically inserted, glued-in steel plates, having different size at the upper and lower surface. The deck, having its width established based on the effective width, is connected to the beams through the upper inserted steel plates. The theoretical elasto-plastic behavior of the model, determined by the composite beam theory is compared to the experimental one.

Keywords : *glulam beam, orthotropic steel deck, bridge model, composite beam theory, ultimate bending strength*

1. INTRODUCTION

The appearance of modern timber bridges, utilizing engineered wood material, made the renaissance of wooden bridges in Japan possible. Nowadays, several tens of timber bridges are believed to be constructed in a year. Although most of them are for pedestrian use, road bridges exist as well.

A very important aspect in the design of beam bridges is the ratio of beam height versus span length. Choosing this ratio to be comparable to ratios of other types of bridges helps making the timber bridge a viable option for bridge building. A glued laminated timber and orthotropic steel deck hybrid beam bridge structure, for short and medium span bridges, was proposed (with cross section shown in Fig.1) having the height-span ratio h/l = 1/16. As a comparison, in the case of reinforced concrete beam bridges h/l = 1/16, prestressed concrete beam bridges h/l = 1/24 and steel plate girder bridges h/l = 1/15 − 1/20.

This paper presents the three-point bending test of a glulam beam-orthotropic steel deck hybrid beam, being the 1/3 model of the bridge prototype and having half of its cross section. The test was conducted in the structural testing laboratory of the Institute of Wood Technology, Akita Prefectural University, situated in Noshiro City, Japan. The ultimate bending strength was investigated. The theoretical elasto-plastic behavior of the model, based on the composite beam theory, is compared to the experimental one.

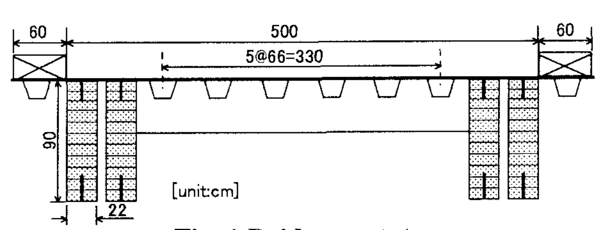

Fig. 1 Bridge prototype

2. BRIDGE MODEL

The tested hybrid beam consists of an orthotropic steel deck, attached to a double glulam beam. Fig. 2 and Fig. 3 show the geometry of the specimen, illustrated by pictures taken at the testing laboratory. The orthotropic steel deck plate has a total length of 5200 mm (the span is 5000 mm), a width of 1033 mm and a thickness of 4.5 mm. The deck plate is stiffened by four U-shaped longitudinal ribs and seven ⊥-shaped transverse ribs, the latter being arranged with an interval of 833 mm. The steel deck acts as the top flange of the double beam, having rectangular cross sections.

The beams are stiffened by two, vertically inserted, glued-in ribbed steel plates, having different size at the upper and lower surface. The size of the upper ribbed plate is 6x44 mm, while the lower one has a cross section of 6x65 mm (see Fig. 3). These steel plates are bonded by epoxy resin, after removing mill scale by sand-blasting. The deck, having its width established based on the effective width (obtained according to the Japanese shear lag formula for roadway bridges), is connected to the beams through the upper inserted steel plates. These are welded through the whole length to the bottom surface of the steel deck plate and act as shear connectors. The role of the glued-in ribbed steel plates on the lower surface of the beams is to compensate the longitudinal axial strength of beams.

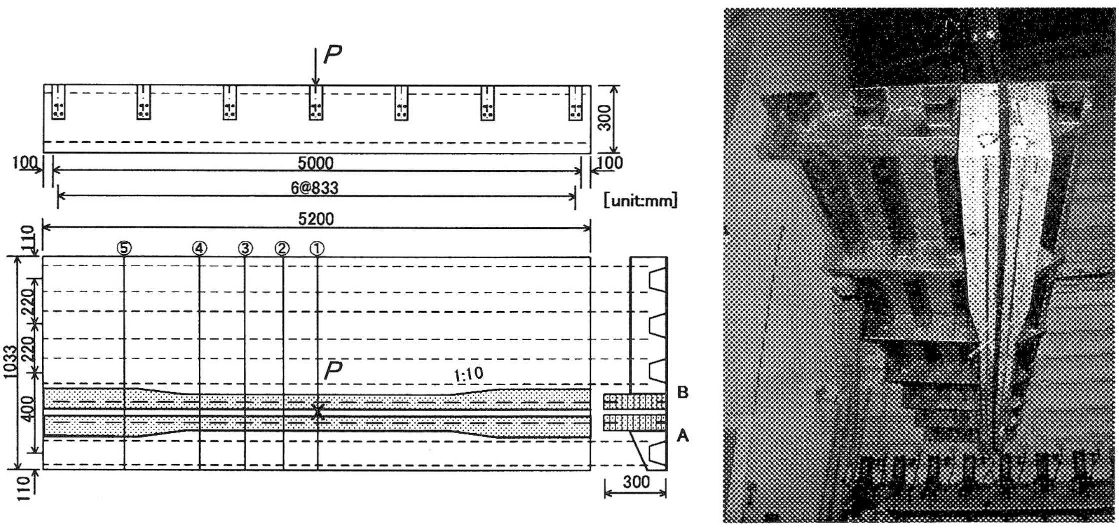

Fig. 2 Strain gage location, side and plan view of the test specimen

The beam width of the double beam varies from 73.5 mm to 113 mm at near the beam ends (in order to overcome the shear forces developed by the reactive forces on the support), with a gradient of 1:10, but has a constant beam depth of 300 mm (see Fig. 2).

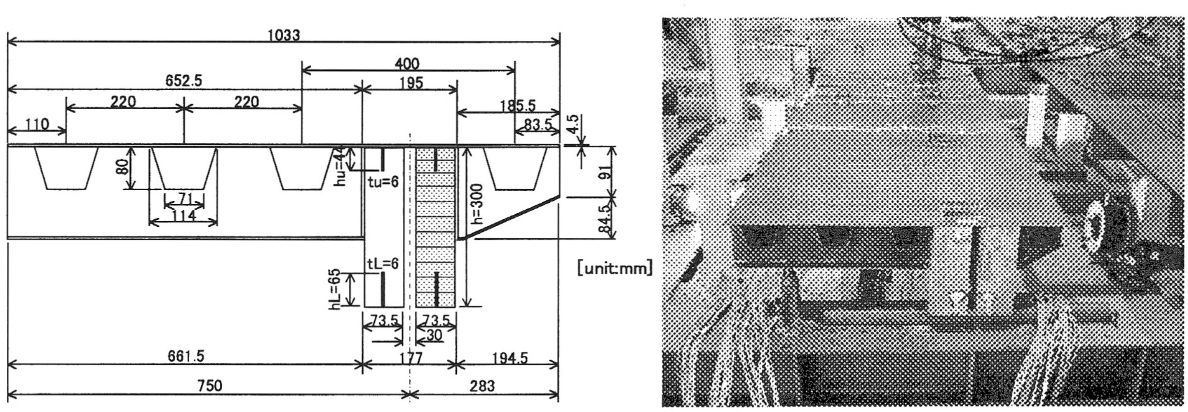

Fig. 3 Cross section of the composite beam

3. BENDING TEST

3.1 MATERIAL PROPERTIES

Three-point bending tests of glulam beam-orthotropic steel deck hybrid bridge model were conducted in the structural testing laboratory of the Institute of Wood Technology, Akita Prefectural University, situated in Noshiro City, Japan. Two bridge models were tested, however in this paper only one is presented in detail.

The strength grade of Douglas fir glulam used for the bridge model was E120-F330, meaning a modulus of elasticity of 120 tf/cm^2 (11.8 GPa) and a bending strength of 330 kgf/cm^2 (32.4 MPa). To verify the actual parallel-to-grain MOE as well as the bending moment capacity of the timber material, twelve specimens were tested. These pieces were cut out from the double glulam beam of the hybrid model, after the failure occurred. The cross section of these small beams was 45x65 mm, having a span length of 1300 mm.

The statistical analysis of the test data resulted in higher bending capacity and smaller MOE compared to the nominal values. Test data of bending moment capacity fit a 2-parameter Weibull distribution. The non-parametric, 5% tolerance limit with 75% confidence level is $\sigma_{Y,W}$ = 47 MPa. The allowable bending strength is equal to 1/3 of the bending strength, i.e. $\sigma_{A,W} = \sigma_{Y,W}/3$ = 16 MPa. In the case of the bending modulus, the lower 95% confidence limit on the mean value of the Young s modulus was used to establish the value of E_W = 10.4 GPa. The shear modulus was taken as G_W = 648 MPa, determined during an earlier bending test.

The steel material used for the inserted steel plates as well for the orthotropic steel deck is SMA490W. The yield strength of this material is $\sigma_{Y,S}$ = 353 MPa, the allowable strength is $\sigma_{A,S}$ = 206 MPa, while the bending modulus is E_S = 200 GPa.

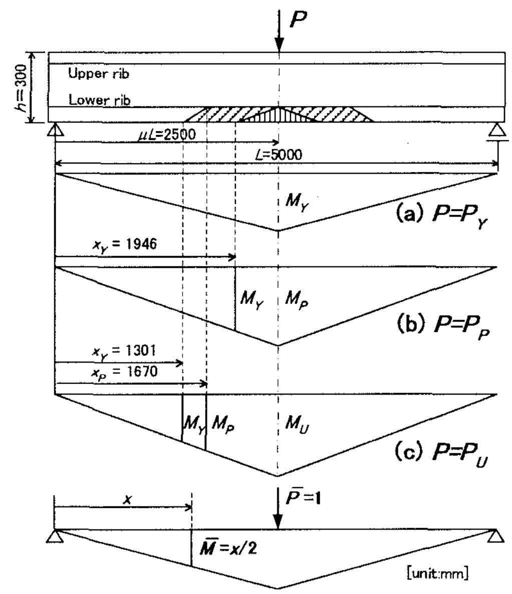

Fig. 4 Loading scheme and method of virtual work

3.2 COMPOSITE BEAM THEORY

Using the concept of transformed section and converting all steel to an equivalent wood area, the properties of the composite cross section are calculated. First, the ratio of moduli of elasticity is determined in order to obtain the modified section: $n = E_S/E_W$ = 200 / 10.4 = 19.

Since the steel deck (having its width nearly equal to the effective width), the ribbed steel plates and the double glulam beam constitute a composite beam, the composite beam theory can be used to calculate the specific moments and corresponding loads occurring during load history (see Table 1).

Table 1 Calculated bending moments and loads

Case	M_i & P_i		Unit	P_i/P_A
Allowable	M_A	75.8	kN·m	1
	P_A	60.6	kN	
Yield	M_Y	129.9	kN·m	1.7
	P_Y	103.9	kN	
Plastic	M_P	166.9	kN·m	2.2
	P_P	133.5	kN	
Ultimate	M_U	249.9	kN·m	3.3
	P_U	199.9	kN	

These specific moments are based on some theoretical assumptions. The yield moment M_Y is the bending moment for which yielding of lower ribbed steel plate starts. At this phase, the tensile stress in this lower inserted steel plate reaches the yield strength of steel, i.e. $\sigma_x = \sigma_{Y,S}$ = 353 MPa. The plastic moment M_P occurs when at mid-span, the cross section of the lower inserted steel plate is in fully plastic state. Determination of the ultimate moment M_U is based on the assumption that failure of the composite beam occurs when the tensile stress in the outer fibers of the double glulam beam reaches the ultimate bending strength of timber, i.e. when $\sigma_x = \sigma_{Y,W}$ = 47 MPa.

The method of virtual work, or sometimes referred to as the unit-load method, is one of the several techniques available that can be used to solve for displacements and rotations at any point on a structure. The mid-span deflection is calculated for each specific moment and load, mentioned before. Here, only the theoretical expression of the ultimate deflection δ_U is included:

$$\delta_U = \delta_M + \delta_V$$

where δ_M and δ_V are the deflection due to bending and shear effects, respectively. As the behavior of the composite beam changes from elastic to plastic, the neutral axis moves from its initial position towards the compression side, i.e. towards the steel deck. As a consequence, the equivalent moment of inertia $I_{V,i}$ of the composite beam decreases as the bending moment increases (see Table 2). This is the reason why in the following equation, the member δ_M is divided into three portions, corresponding to the three limit cross sections marked by M_Y, M_P and M_U, as shown in Fig. 4(c).

Table 2 Calculated moments of inertia

Case	$I_{V,i}$		Unit
Yield	$I_{V,Y}$	16.19	$\times 10^4$ cm^4
Plastic	$I_{V,P}$	15.18	
Ultimate	$I_{V,U}$	13.38	

$$\delta_U = \frac{M_U}{E_W}\left(\frac{x_Y^3}{3I_{V,Y}\mu L} + \frac{1}{\mu L}\int_{x_Y}^{x_P}\frac{x^2}{I_{V,P}}dx + \frac{\mu^3 L^3 - x_P^3}{3I_{V,U}\mu L}\right) + \frac{\kappa P_U}{2G_W A_V}\mu L \quad \text{, in which}$$

M_U ultimate bending moment, calculated by composite beam theory

P_U ultimate load, corresponding to the ultimate bending moment

x_Y coordinate of cross section position where the bending moment is $M = M_Y$, used when $M_{max} \geq M_Y$

x_P coordinate of cross section position where the bending moment is $M = M_P$, used when $M_{max} \geq M_P$

μL coordinate of the cross section being under the applied load, acting closest to the support; in the case of a three-point bending test, as considered in this paper, $\mu = 0.5$

$I_{V,Y}$ elastic moment of inertia of the composite beam, used when $M \leq M_Y$

$I_{V,P}$ plastic moment of inertia of the composite beam, used when $M_Y \leq M \leq M_P$

$I_{V,U}$ ultimate moment of inertia of the composite beam, used when $M_P \leq M \leq M_U$

A_V equivalent cross section area of the composite beam, here being $A_V = 1747$ cm^2

κ shear factor of cross section, being $\kappa = 3.79$ for the bridge model considered

All the above-mentioned coordinates are measured from the considered origin, in this case the fixed support of the beam. Table 3 shows the calculated mid-span deflections at the specific moments. It can be seen that while the bending moment increases from the yield moment towards the ultimate moment, at the same time the shear effect decreases, resulting in smaller shear deflections δ_V compared to the total deflections δ_i. The reason of this decrease can be explained using the above expression of the ultimate deflection δ_U. The shear component contains only one variable term (the applied load), while all other terms are constants (κ, G_W, A_V). In contrast to this, the bending component of δ_U increases thanks to several variables. One of these is the bending moment, which continues to increase until failure. At the same time, the equivalent second moment of inertia $I_{V,i}$ of the composite beam decreases (see Table 2), these value being inverse proportional to the deflection. The smaller the coordinates x_Y and x_P become, the larger the effect of the variable $I_{V,i}$ becomes. These coordinates change due to the spreading of plastic region in the lower inserted steel plate as the moment increases. The plastic zone starts spreading from the mid-span toward the ends of the beam, until the bending moment reaches its ultimate value. This is illustrated in Fig. 4, from (a) to (c).

Table 3 Calculated deflections in mm

Case	δ_M	δ_V	$\delta_i = \delta_M + \delta_V$		δ_V / δ_i(%)
Yield	16.1	5.0	δ_Y	21.1	23.5
Plastic	21.4	5.6	δ_P	27	20.7
Ultimate	35.9	8.3	δ_U	44.2	18.9

3.3 EXPERIMENTAL RESULTS

Fig. 5 reports the theoretical (with and without shear effect taken into account) and experimental diagrams of the load P versus the mid-span deflection δ . The experimental value of the maximum load, corresponding to the ultimate bending moment $P_{U,EXP}$ = 238 kN. This value is 19% higher than P_U = 200 kN, determined by the composite beam theory. For P = 200 kN, the experimental deflections of the component beams of double glulam beam are measured as δ_A = 44.5 mm and δ_B = 46.1 mm, corresponding to glulam beam A and B, respectively (see Fig. 2). These values are slightly higher than the theoretical value of δ_U = 44.2 mm (see Table 3). The theoretical $P-\delta$ curve follows closely the curves of beams A and B, however it fails to predict the maximum deflections of $\delta_{A,MAX}$ = 61.3 mm and $\delta_{B,MAX}$ = 64.2 mm. It can be seen that the $P-\delta$ curve of deck is separated from those of the double glulam beam. This is due to the fact that the steel-glue-timber interaction is not sufficient, i.e. the composite action between the deck and double beam fails to work properly.

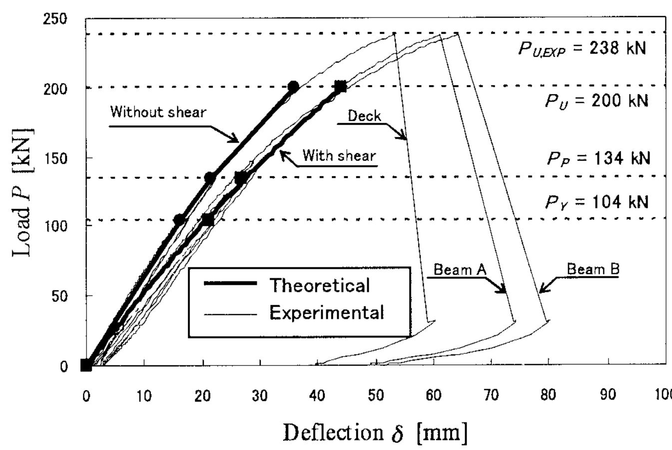

Fig. 5 Theoretical and experimental curves of load versus mid-span deflection

Strain gages were installed along the depth of the double glulam beam, at the bottom of lower inserted steel plates as well as on the upper surface of orthotropic steel deck. Gages are located along line ①−⑤, line ① being at the mid-span of bending test specimen (see Fig. 2). Also, at mid-span, bottom of U-ribs were provided with strain gages. Strain state of some of the composite beam components (steel deck plate, glulam beam and lower inserted steel plate) at specific loads is presented in Fig. 6. The strains included here were measured at line ②, positioned 306 mm from the mid-span. It can be seen that on one hand, at the upper part of the glulam beam strain values of the deck and timber are relatively close to each other. On the other hand, in the case of the ultimate load, at the lower part of beam the strain value of the lower rib and timber are separated. This is due the separation of the steel plate from the timber. Theoretical values of strain manage to closely predict the experimental values.

Failure of the test specimen occurred in the outermost fibres of the timber, failure starting at structural finger-joints. Fig. 7 shows the failure of the bridge model.

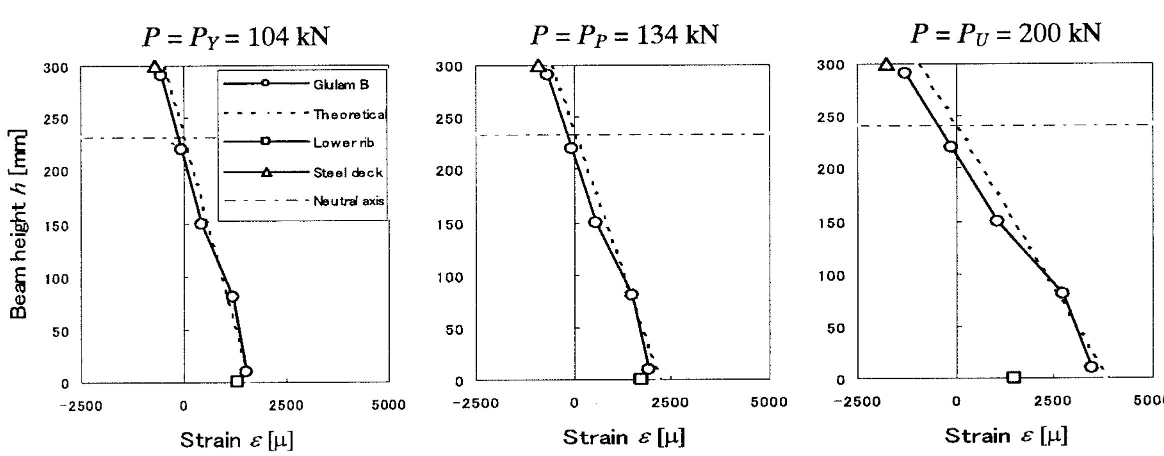

Fig. 6 Strain state at specific load steps

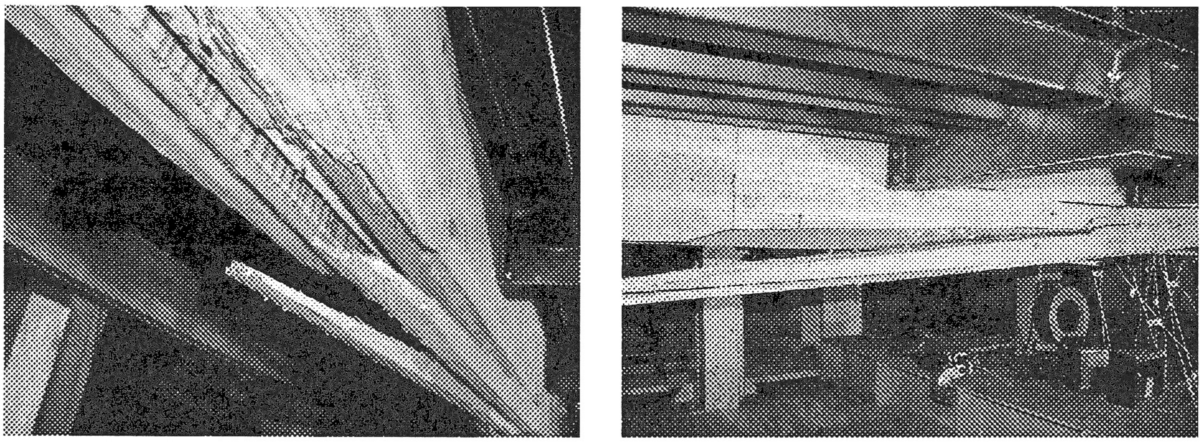

Fig. 7 Failure of bridge model

4. DISCUSSIONS AND CONCLUSIONS

Results of the three-point bending test of glulam beam-orthotropic steel deck hybrid bridge model show that emphasis on interaction between steel, glue and timber must be improved, taking extra care on the manufacturing of the hybrid structure.

Since the composite beam theory could not give a complete view about the overall behavior of the composite beam model, a finite element analysis has to be carried out to obtain a better structural model, able to correctly predict the structural behavior in the plastic region too. This FEM analysis is now in progress, the results of which will be presented in a future paper.

REFERENCES

1) Usuki, S., Gotou, H. and Kiss, L.: Bending capacity of glued-laminated timber stiffened with inserted steel plate, *Journal of Structural Engineering, JSCE*, Vol. 49A, pp. 889-894, 2003 (in Japanese)
2) Usuki, S., Atsumi, A., Sudo, S. and Iijima, Y.: A new timber beam bridge with an orthotropic steel deck, *Proceedings of 6th World Conference on Timber Engineering*, pp. 8.3.3-1 8.3.3-7, 2000
3) Usuki, S., Sasaki, T., Atsumi, A., Sharma, M.P.: Shearing stress of longitudinal rib plate of hybrid timber beam and orthotropic steel deck, *Journal of Structural Engineering, JSCE*, Vol. 47A, pp. 1221-1227, 2001 (in Japanese)
4) Bob, L., Gruin, A., Kiss, L.: Alternative reinforcements of wooden beams, *Conference Report-Lahti, IABSE*, pp. 501-506, 2001