FOLIA FORESTALIA POLONICA

©*Wyd. AR Pozn.* *Seria B, Zeszyt 36, 27-37, 2005*

BEHAVIOUR OF OSB-WEBBED I-BEAMS SUBJECTED TO SHORT-TERM LOADING

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SYNOPSIS. Seven composite I-beams made of two 38 × 65 mm pine wood flanges and 10 mm thick OSB/3 web were tested in bending. The beams of an effective span of 4.5 m and height of 240 mm were loaded at third point to obtain a wide zone of pure bending. Each beam was loaded in two cycles: six times up to 6.53 kN and six times up to 10.22 kN and next until failure. The force-deflection relationship at bending of the composite I-beams appears to be linear, if the tensile stress in flanges does not exceed 130% of the computational strength of wood. The quality of the joints in the tension flange turned out to be decisive for the load-carrying capacity of the beams.

KEY WORDS: composite I-beams, oriented strand board (OSB), pine wood, bending, rigidity, load capacity, flange-web joint, failure

INTRODUCTION

Oriented strand board (OSB) has a bigger and bigger application in Polish building industry. At the beginning OSB panels were used only as wall and floor facing and as roof boarding. The research in this area was carried out by Szyperska and Nożyński (1999). Then the attention was paid to a possible application of OSB panels for manufacturing packages, in boarding, and finally as the webs of composite I-beams (Hikiert 2001).

Composite I-beams made of wood and wood-based panels have been investigated in the Department of Engineering Mechanics and Thermal Techniques of Agricultural University of Poznań for a long time. First, the stiffness and load capacity of complex beams made of pinewood and hardboard were studied (Ganowicz and Kwiatkowski 1985, Ganowicz et al. 1990, Olejniczak and Plenzler 1993). Then the creep behaviour of the beams was also investigated (Plenzler 1993).

Wood-based materials such as plywood, particleboard, waferboard, hardboard and OSB are used in complex I-beams because they are characterized by high, in comparison to wood, shear modulus and shear strengths (Leichti et al. 1990). OSB is a cheaper replacement for plywood, although its strength and stiffness are considerably lower than those of high-quality structural grade plywood (Dinwoodie 2000). The problem of shear deflection of composite beams was analysed by Booth (1977) who found, that the effects of the load distribution, support conditions and span can be separated from the effect of the geometry and elastic moduli by means of the nondimensional shear shape factor. Ganowicz et al. (1990) predicted with the use of the shear shape factor the influence of the shear force on deflection of composite I-beams with hardboard webs.

For wider structural use of composite beams there is a need to know elastic, strength and rheological properties of wood and OSB panels. In recent years a number of the investigations was performed on the elastic properties (Wilczyński and Gogolin 1999, Plenzler and Górecki 2002), the strength properties (Szyperska and Nożyński 1999) and rheological properties of OSB panels (Pałubicki and Plenzler 2004). Smardzewski et al. (2000) made an attempt of the numerical modelling of the elastic behaviour of composite I-beams with OSB webs and to compare the results with the experimental data from Hikiert et al. (2000). However, it is necessary to make extensive investigations of the beams to prove their real rigidity, load capacity and creep compliance.

METHODS

Seven composite I-beams were constructed as illustrated in Figure 1. Each beam consisted of two 38 × 65 mm flanges made of pine wood (*Pinus sylvestris* L.) and 10 mm thick OSB/3 web. The cross-section of the composite beam was selected in the similar way as in the BK-D 240 composite rafter proposed by Hikiert (2001) or in the I-beams tested by Chen et al. (1989). The tongue-and-groove flange-web joint was similar in its type to the one earlier reported by Ganowicz et al. (1990) for composite beams with hardboard webs. In order to eliminate shear slip at the flange-web joint rigid phenol-resorcinol adhesive Dynosol S-205 was used.

The composite beams of an effective span of $L = 4.5$ m were loaded at third point along the beam to obtain a wide (i.e. 1.5 m) zone of pure bending (Fig. 2).

Because commercial OSB/3 panels (2500 mm by 1250 mm by 10 mm thick) were used, webs of the beams were joined by means of two OSB splice plates, glued on both sides of the web (Fig. 1). The width of the splice plates ($b_n = 6$ cm) was designed in accordance with the proposal of Ozelton and Baird (1976). Wood flanges were joined in about 100 cm from ends of the beams by means of finger mini-joints with 12 mm length.

Additionally, pine wood web reinforcements (38 mm by 27.5 mm) were glued on both sides of the web at reaction and concentrated load points to prevent the web from buckling. The mass of the I-beams was 18.72 ± 0.32 kg.

The load was applied to the beam through a hydraulic jack and the accuracy of the load measurement was 0.12 kN. Two lateral supports with the spacing

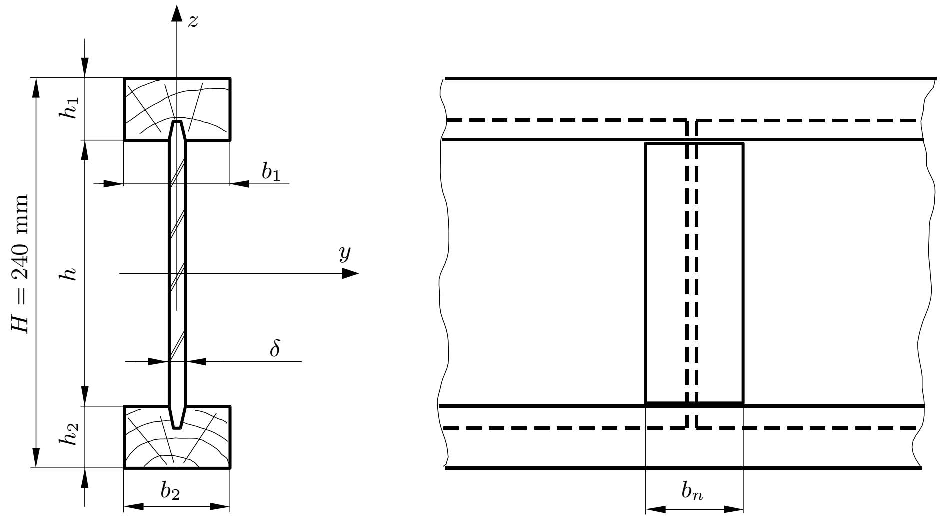

Fig. 1. Cross-section of the composite I-beam and the joint of the OSB web
Rys. 1. Przekrój poprzeczny dwuteowej belki zespolonej i połączenie środnika z płyt OSB

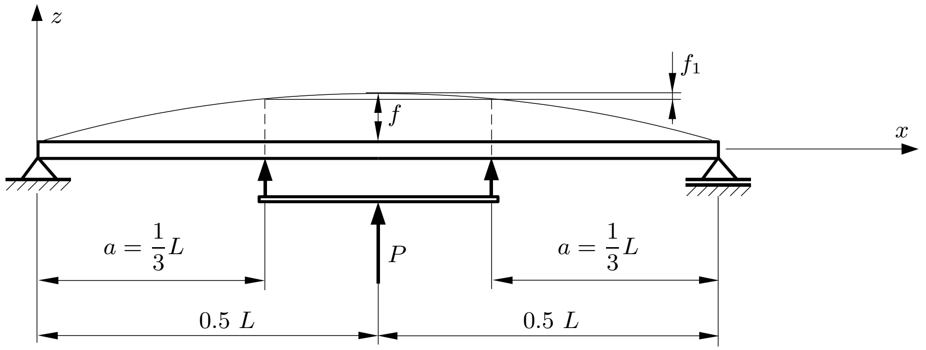

Fig. 2. Diagram of the beam load and deflections measurement
Rys. 2. Schemat obciążenia i pomiaru ugięć belki

of about 190 cm were used to avoid the lateral buckling of the beam. Temperature and relative humidity (RH) in the lab were monitored within ±1°C and ±1%, respectively.

The following procedure was used to test each beam. The beam was loaded in two cycles: six times up to 6.53 kN and next six times up to 10.22 kN with increments (apart from the first) of 2.46 kN (i.e. with increments of 1.00 MPa in the hydraulic jack of the manometer scale). These levels of load produced in the tension flange mean normal stress at the level of 83.5% and 130.7% of the computational tensile strength in the flange ($\sigma_{\mathrm{f,t,d}} = 11$ MPa), respectively, as for structural lumber grade C 30 according to the Polish Standard PN-B-03150:2000.

After completing both load cycles each beam was loaded in the same increments of the force as to failure.

A depth gauge was used to measure a total deflection at the mid-span of the beam and two dial gauges were positioned close to each end of the support point to enable calculation net flexural deflection. Additionally, a dial gauge at mid-span was positioned to measure a deflection at the zone of pure bending. Measuring accuracy of the depth gauge and the dial gauges was 0.1 mm and 0.01 mm, respectively. Each flexural deflection was measured during the loading of the beam.

RESULTS

Figures 3 and 4 show typical charts of the force-deflection relationship ($P \sim f$) for the composite beam during the first series of loading, i.e. for whole beam and for the zone of pure bending only, respectively. Even though the beams weren't preloaded (i.e. a mechanical conditioning was not applied) the relationship $P \sim f$ turned out to be almost linear, so the linear-regression analysis was used to determine the flexural stiffness of the beams. Permanent set after each cycle of loading-unloading was no significant, so the behaviour of the beams during the tests may be recognized as nearly elastic, even if the stress level in tensile flange exceeded 83% of the computational tensile strength of wood.

Figures 5 and 6 show typical charts of the $P \sim f$ relationship for the composite beam during the second series of loading, i.e. for the whole beam and for the zone of pure bending, respectively. Those relationships, similarly as in the first series, were nearly linear, despite of higher level of load (above 130% of the computational tensile strength of wood).

An approximation of the results of all tests was carried out using the computer program Sigma Plot. Flexibilities of all tested beams, obtained in both series of loading from total and in pure bending zone deflections, are summarized in Table 1. Those experimental data are put together with the flexibilities k_1 and k calculated from the general formulas

$$f_1 = k_1 \cdot P = \frac{a^3}{16EJ_{\text{ef}}} \cdot P \tag{1}$$

for pure bending zone deflection f_1 and

$$f = k \cdot P = \left(\frac{23}{48} \cdot \frac{a^3}{EJ_{\text{ef}}} + \frac{S \cdot a}{2A_{\text{ef}} \cdot G} \right) \cdot P \tag{2}$$

for total mid-span deflection f, where the shear shape factor

$$\begin{aligned} S \;=\; & n_1 \cdot \frac{A_{\text{ef}} \cdot b \cdot H^5}{32J_{\text{ef}}} \cdot \left\{ \frac{1}{2}\left(1 - \frac{h}{H}\right) - \frac{1}{3}\left[1 - \left(\frac{h}{H}\right)^3\right] + \frac{1}{10}\left[1 - \left(\frac{h}{H}\right)^5\right] \right\} + \\ & + \frac{2A_{\text{ef}}}{J_{\text{ef}}^2 \cdot \delta} \cdot \left[C_1^2 \cdot \frac{h}{2} - \frac{2}{3} \cdot C_1 \cdot C_2 \cdot \left(\frac{h}{2}\right)^3 + \frac{1}{5} \cdot C_2^2 \left(\frac{h}{2}\right)^5 \right] \end{aligned}$$

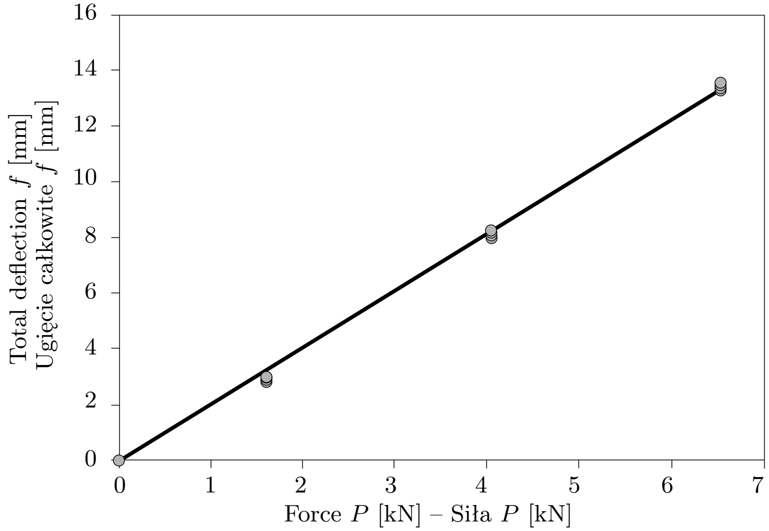

Fig. 3. Force-total deflection ($P \sim f$) relationship for the composite I-beam No. 4 during the first series of loading; $P_{\max} = 6.53$ kN
Rys. 3. Zależność siła-ugięcie całkowite ($P \sim f$) dla belki zespolonej nr 4 podczas pierwszego cyklu obciążenia; $P_{\max} = 6{,}53$ kN

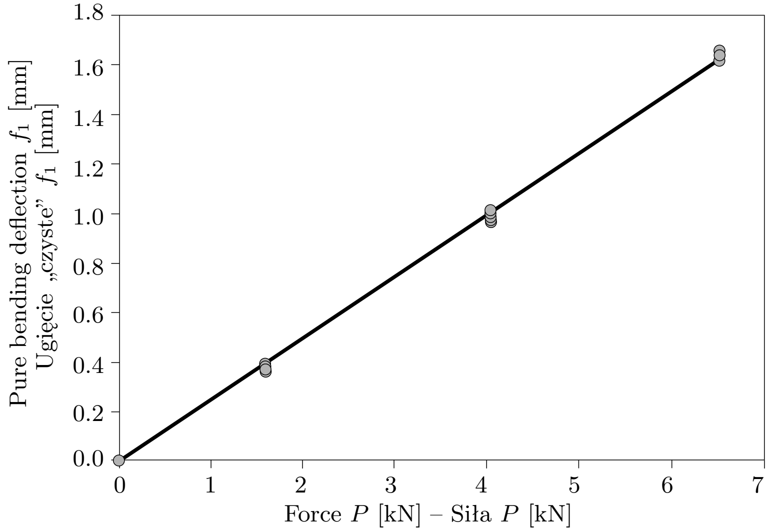

Fig. 4. Force-pure bending deflection ($P \sim f_1$) relationship for the composite I-beam No. 4 during the first series of loading; $P_{\max} = 6.53$ kN
Rys. 4. Zależność siła-ugięcie „czyste" ($P \sim f_1$) dla belki zespolonej nr 4 podczas pierwszego cyklu obciążenia; $P_{\max} = 6{,}53$ kN

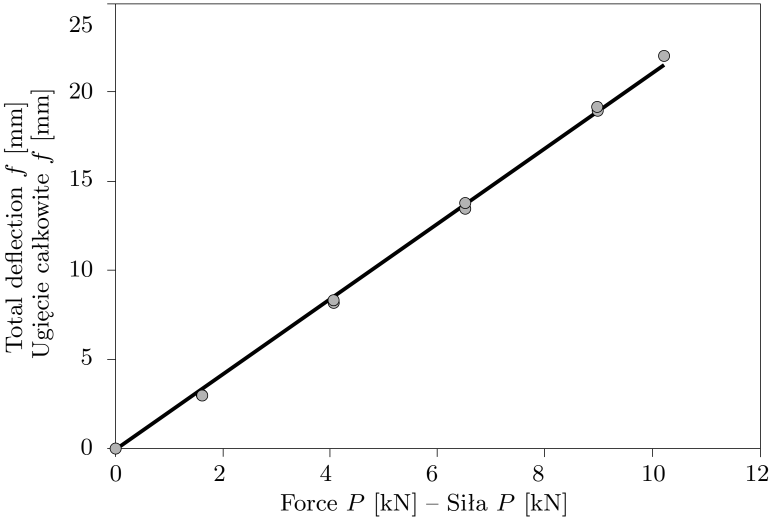

Fig. 5. Force-total deflection ($P \sim f$) relationship for the composite I-beam No. 4 during the second series of loading; $P_{\max} = 10.22$ kN
Rys. 5. Zależność siła-ugięcie całkowite ($P \sim f$) dla belki zespolonej nr 4 podczas drugiego cyklu obciążenia; $P_{\max} = 10{,}22$ kN

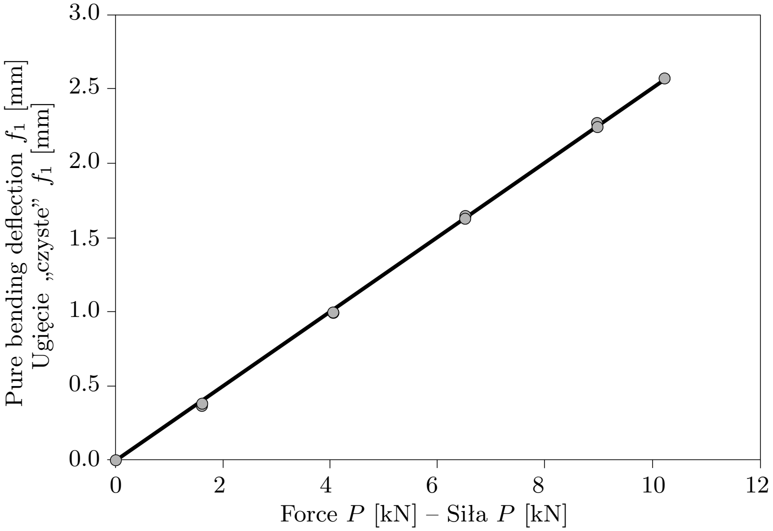

Fig. 6. Force-pure bending deflection ($P \sim f_1$) relationship for the composite I-beam No. 4 during the second series of loading; $P_{\max} = 10.22$ kN
Rys. 6. Zależność siła-ugięcie „czyste” ($P \sim f_1$) dla belki zespolonej nr 4 podczas drugiego cyklu obciążenia; $P_{\max} = 10{,}22$ kN

and

– A_{ef}, J_{ef} – effective area and moment of inertia, respectively,

– $C_1 = n \cdot \dfrac{\delta \cdot h^2}{8} + \dfrac{b}{2} \cdot \left(\dfrac{H^2}{4} - \dfrac{h^2}{4}\right)$,

– $C_2 = n \cdot \dfrac{\delta}{2}$,

– $n = \dfrac{E_{\mathrm{OSB}}}{E}$,

– $n_1 = \dfrac{G_{\mathrm{OSB}}}{G}$,

– $E = E_{0,\mathrm{mean}} = 12$ GPa and $G = G_{\mathrm{mean}} = 750$ MPa – moduli of elasticity and rigidity, respectively, of structural lumber grade C 30 according to PN-B-03150:2000,

– H, h, b, δ, a – cross-section and beam dimensions according to Figure 1 and Figure 2.

Equation (2) was derived in the similar way as proposed by Ganowicz et al. (1990) i.e. using the virtual-work method for deflections in the following form:

$$f = \int_0^L \frac{M\overline{M}}{EJ_{ef}} ds + S \int_0^L \frac{T\overline{T}}{GA_{ef}} ds \tag{3}$$

however taking into consideration different values of the G moduli of wood and OSB panels. In equation (3), M is the bending moment and T is the shear force due to the real loads, while $\overline{M}$ and $\overline{T}$ are the bending moment and the shear force due to the virtual loading.

The values of the moduli of elasticity and rigidity of 10 mm thick of OSB/3 panels $E_{\mathrm{OSB}} = 6323$ MPa and $G_{\mathrm{OSB}} = 1996$ MPa were taken from the earlier investigation (Plenzler and Górecki 2002). The additional deflection due to shear forces as calculated from the formula (2), was about 6% of the total mid-span deflection of the composite beam.

Observing the values of the parameters k and k_1 in Table 1, we can see that the actual flexibilities of composite beams turned out to be about 15% less than the calculated from the formulas (1) and (2). Presumably, the modulus of elasticity of wood flanges was higher than 12 GPa as assumed for structural lumber grade C 30 and the effective moment of inertia of composite beam cross-section was higher too.

All the beams were destructively tested in bending using the same loading system as earlier. Failures occurred in two modes only, either in tension flange (6 beams) or in compression flange (1 beam). None glue joint between flange and OSB web failed which indicated that a construction of the flange-web joint and their workmanship were adequate. In contrast to that, the finger mini-joints in wood flanges can't be recognized as successful, because 6 beams were damaged in the same manner – as a result of the failure of a joint in tension flange. Samson (1983) has earlier found that there is a relation between the load-carrying capacity of the composite beam made of wood and waferboard with a quality of its tension

Table 1. Flexibility of the composite I-beams
Tabela 1. Podatność zespolonych belek dwuteowych

Beam number Numer belki	Flexibility (pure bending) Podatność (czyste zginanie) k_1 [mm/N] $\cdot 10^{-3}$		Flexibility (bending and shear) Podatność (zginanie ze ścinaniem) k [mm/N] $\cdot 10^{-3}$	
	first cycle of loading pierwszy cykl obciążenia $P_{\max} = 6.53$ kN	second cycle of loading drugi cykl obciążenia $P_{\max} = 10.22$ kN	first cycle of loading pierwszy cykl obciążenia $P_{\max} = 6.53$ kN	second cycle of loading drugi cykl obciążenia $P_{\max} = 10.22$ kN
	mean values – wartości średnie			
1	0.2401	0.2437	2.0307	2.0520
2	0.2405	0.2525	2.1423	2.2566
3	0.2421	0.2436	2.0914	2.1228
4	0.2472	0.2492	2.0190	2.0942
5	0.2792	0.2851	2.3910	2.4515
6	0.3038	0.3142	2.6946	2.7905
7	0.2705	0.2713	2.2228	2.2372
Mean value Wartość średnia	0.2605 ± 0.0229	0.2657 ± 0.0244	2.2274 ± 0.2245	2.2864 ± 0.2400
Values calculated from formulas (1) or (2) Wartości obliczone ze wzorów (1) lub (2)	0.3228		2.6336	

flange. Only one beam was damaged as a result of an occurrence of a large knot in compression flange.

The destructive forces varied between 10.96 kN and 20.07 kN for different beams, at the average of 14.18 kN, i.e. the effective factor of safety, relative to the computational destructive force (7.82 kN), averaged out at 1.81. Therefore, the average value of the breaking moment was 10.64 kNm. The obtained flexibilities and failure modes of the tested I-beams turned out to be similar to ones reported by Chen et Al. (1989) but their load capacities were significantly lower.

After the composite beams were destructively tested, wood and OSB samples were collected from each beam to determine their moisture content. Moisture content for all wood flanges and OSB webs ranged from 6.5% to 8.3% and from 6% to 7.5%, respectively. Average moisture content of wood flanges was 7.3% when for OSB webs 6.5%.

CONCLUSIONS

1. The force-deflection relationship at bending of the composite I-beams with OSB/3 webs appears to be nearly linear if the tensile stress in the flanges don't exceed 130% of the computational strength of wood.
2. The flexibility of the composite beams was slightly (15%) less than that calculated from the formulas (1) and (2) as a result of the underestimation of the actual modulus of elasticity of wood.
3. The glue tongue-and-groove flange-web joint made with the use of the phenol-resorcinol adhesive turned out to be successful and did not fail during the destructive tests of the beams.
4. The web splices made of OSB plates glued on both sides of the web transmitted very well vertical shear and bending moment.
5. The quality of the joints in the tension flange turned out to be decisive for the load-carrying capacity of the composite beams.
6. The occurrence of big knots in the compression flange with not large cross-section may be the reason of the lateral buckling and failure of composite I-beams made of wood and OSB plates.

Acknowledgement

The authors wish to thank Dr. A. Makowski and Mr. S. Matuszak from the Department of Engineering Mechanics and Thermal Techniques, Agricultural University of Poznań for their contribution in experiments.

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SZTYWNOŚĆ I NOŚNOŚĆ DWUTEOWYCH BELEK ZESPOLONYCH O ŚRODNIKACH Z PŁYT OSB W PRÓBIE ZGINANIA

Streszczenie

Przedstawiono wyniki badań pracy zginanych belek zespolonych z drewna i płyt OSB. Siedem belek dwuteowych o rozpiętości 4,5 m i wysokości 240 mm sklejono z pasów sosnowych o przekroju 38 × 65 mm i środników z płyty OSB/3 o grubości 10 mm. W połączeniu klinowo-wpustowym pasów ze środnikiem zastosowano klej rezorcynowo-fenolowy. Belki zginano w schemacie 4-punktowym w dwóch cyklach po 6 obciążeń w każdym. Następnie belki były obciążane aż do zniszczenia. Praca belek okazała się praktycznie liniowo sprężysta przy obciążeniu wywołującym w pasie rozciąganym naprężenia rzędu 83% jak również 130% naprężenia obliczeniowego dla drewna konstrukcyjnego klasy C 30. Podatność belek na zginanie okazała się przy tym mniejsza niż przewidywano, ponieważ rzeczywisty moduł sprężystości drewna okazał się wyższy od założonego. Stwierdzono, że powodem zniszczenia sześciu belek było rozerwanie pasa rozciąganego w miejscu połączenia klinowego.

Received in July 2005

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