HARDBOARD WEBBED WOOD I-BEAMS

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Abstract

This present paper describes the testing of hardboard webbed wood I-beams with flanges of laminated veneer lumber (LVL) and analyses their performance under third-point bending. The investigations are part of an ongoing Master's Degree study supported by ***Forest Research***. The results showed that I-beams with hardboard webs were stronger and stiffer than the plywood webbed I-beams, in spite of the plywood webs being 50% thicker than the hardboard webs.

Introduction and literature review

Currently in New Zealand there is only one locally manufactured I-beam - Twinaplate (with kiln dried, boric treated radiata timber flanges and corrugated steel webs) and one imported from CHH Wood Products, Australia (with LVL flanges and plywood webs).

Hardboard has been used as a web material in built-up beams in Europe since the mid-1930s. Research on hardboard as a shear-web material has been done in at least six countries (McNatt, 1980): Australia, Czechoslovakia, England, Germany, Sweden, and the United States. Test data on hardboard webbed I-beams showed a good correlation with theoretical predictions for deflection and bending strength (Hilson and Rodd, 1979).

McNatt and Superfesky (1983) report that I-beams with hardboard webs and LVL flanges give satisfactory performance after 5 years service in an exterior or interior environment. They also compared hardboard webbed I-beams with plywood webbed I-beams. They showed that for beams loaded to the same web shear stress, those with hardboard webs deflected less than those with plywood webs. The hardboard was stiffer by three to four times in shear through the thickness than the plywood for the plywood grain orientation used.

In Sweden, hardboard webs are more economical than plywood webs (Norlin, 1988). He also reports that shear strength properties of hardboard are comparable to those of plywood. He studied the behaviour of very slender webs ($35<H_w/t<160$) where 'H_w' is the free height of the web (the distance between the flanges) and 't' is the web thickness. He shows that the use of slender webs has the advantage of increased flexural rigidity at a low material cost by allowing an increase in depth of the beam while keeping the web thickness unchanged (or even decreasing). The main problem of slender hardboard beams is the web buckling along the beam axis. When the web height 'H_w" is below '70t' no additional stiffeners are necessary, except at supports (op. cit., 1988). For very slender webs (H_w>70t) Norlin (1988) presents a method for calculating the optimum distance between the additional stiffeners which have to be spaced along the beam.

Web materials

The plywood and hardboard were purchased locally.

<u>Oil tempered hardboard</u>

Hardboard is manufactured from wood fibre pulp by refining pine wood chips. The pulp mat and bonding agent are hot pressed into high density panels. The panels then are conditioned to a target moisture content of 10% before being trimmed to size. Hardboard has two equally smooth sides. For oil tempered hardboard the pulp mat and the bonding agents are hot pressed into a higher density panel than standard hardboard. This results in a product with improved moisture resistance. Tempered hardboard is a dark brown, strong panel product.

<u>Softwood plywood</u>

The plywood for structural purposes is made from rotary peeled veneers glued with synthetic phenolic resins which are sufficiently durable to permit the panels to be used in exterior environment. Structural plywood is the only sheet material in New Zealand with properties defined in published engineering design codes (NZS 3603:1993-Code of practice for timber design).

Modulus of rigidity and shear strength

Modulus of rigidity (G) and shear strength (V_R) of the panels used for wood I-beam webs were determined following the procedure specified in Large Panel-Shear Test, ASTM D 2719-1994 (Britton and Fitton, 1998). This test method is regarded as giving the most accurate measure of modulus of rigidity and is recommended for elastic test of materials to be used in stress analysis studies of structures. Two specimens of each material were prepared. Density at test for each of the panel materials was calculated after mechanical testing was completed. Table 1 presents the mean strength properties and density for each material. The strength values are applicable to short term loading. For other load durations, modification factors should be used. BS 5268:Part 2:1991 (Section 5) gives useful recommendations on the use of tempered hardboard for structural purposes, including suitable load duration factors.

Table 1. Results from panel testing

Web	Tempered hardboard	Plywood
Density at test (kg/m^3)	963	496
Modulus of rigidity G_{av} (MPa)	1944	460
Shear strength V_R (MPa)	7.34	4.41

The observed modes of failure during the panel shear tests were as follows:

- Hardboard - the specimens failed suddenly and catastrophically, shattering into several pieces.
- Plywood - the material did not fail catastrophically at maximum load and showed no obvious signs of failure.

The characteristic values of modulus of rigidity and shear strength for F11 grade plywood are given in AS 1720.1-1997 (section 5):

- modulus of rigidity - 525 (MPa),
- shear strength - 5.3 (MPa).

It can be seen that the values in table 1 are below the characteristic values for F11 grade plywood. The values obtained by the shear test (table 1) are closer to those for:

- F8 grade for modulus of rigidity,
- F7 grade for shear strength (see table 5.1, AS 1720.1-1997).

The likely reason for these discrepancies is that the veneer quality from the current crop of young trees is less than that on which the characteristic stresses were established.

Flange material

Laminated veneer lumber

LVL is an engineered wood composite material produced from rotary peeled veneers, glued and assembled together so that the grain direction of the outer veneers and most of the other veneers, is in the longitudinal direction. Typically LVL is made from of 3 mm thick veneers. Hyspan, which will soon be available in New Zealand, has thicknesses of 45 or 63 mm with widths of 170 to 600 mm (see article in Issue 1 volume 7, NZ TDJ). CHH Wood Products is the only producer of LVL and I-beams in Australia. LVL is suitable for wood I-beam flanges because it has relatively low weight, it is uniform, stronger than dimension timber, and is reliable - factors which impact on the final design cost of the beams. For the wood I-beams in this study, LVL imported from Australia in 1989 was used. Table 2 shows the characteristic stresses for Hyspan.

Table 2. Characteristic stresses for Hyspan (MPa). Source: CHH Engineered Wood Products, Australia, 1998.

Bending	Tension	Compression	Shear	Modulus of elasticity	Modulus of rigidity
42	27	34	4.5	13200	650

The modulus of elasticity of the flanges was determined from bending tests over a span of 4.2 m under third-point loading. The average value of modulus of elasticity of the two (top and bottom) flanges is given in table 3 and was used for the beam calculations.

Table 3. Results from testing of modulus of elasticity of LVL

Beam code		Web	E
L=5.9m	L=3.9m		(GPa)
A1 300	K1 300	plywood	13.75
A2 300	K2 300	plywood	13.75
C1 300	L1 300	hardboard	16.15
C2 300	L2 300	hardboard	15.85
Average modulus of elasticity			14.9

The shaded cells of table 3 show that the average modulus of elasticity of the tested LVL is higher than the published value by 13% (table 2).

Adhesive

The joints of the wood I-beams (web-web and web-to-flange joints) were made using epoxy resin HPR5. HPR5 is a new generation adhesive which consists of a turbid liquid resin (part A) and a clear liquid curing agent (part B) mixed in a given ratio. Table 4 presents some of the typical cured properties of HPR5.

Table 4. Typical average cured properties of HPR5 (MPa). Source: Adhesive Technologies Ltd. Auckland

Tensile lap shear @ -55ºC	Tensile lap shear @ 25ºC	Tensile lap shear @ 50ºC	Tensile strength (room temperature)	Flexural strength (room temperature)
29.6	29	6.9	55.8	89.6

Construction of the beams

The beams are described in figure 1.

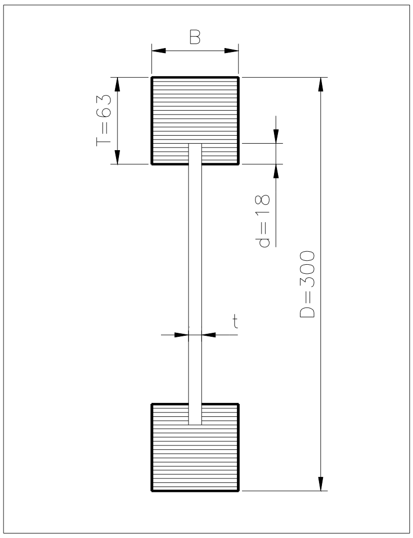

Figure 1. Cross-section of the I-beams

Wood I-beams with codes A1 300, A2 300, C1 300 and C2 300 failed at shear between the load point and the support. Because the remaining portions of those beams were intact, they were cut down to 4 m after testing and retested at a span of 3.9 m. The shorter span beams were given new codes - K1 300, K2 300, L1 300 and L2 300 (table 5).

Table 5. Description of the wood I-beams

Beam code	Web	t (mm)	B (mm)	I_x (mm^4)	Z (mm^3)	Web stiffeners	Buckling restraints
A1 300	plywood	9	63	118792832	791952	no	4 @ 1200 mm
A2 300	plywood	9	63	118792832	791952	no	4 @ 1200 mm
C1 300	hardboard	6	63	118723374	791489	no	4 @ 1200 mm
C2 300	hardboard	6	63	118723374	791489	no	4 @ 1200 mm
K1 300	plywood	9	36	69895886	465973	at supports	3 @ 1000 mm
K2 300	plywood	9	36	69895886	465973	at supports and at loads	3 @ 1000 mm
L1 300	hardboard	6	36	69826428	465510	at supports	3 @ 1000 mm
L2 300	hardboard	6	36	69826428	465510	at supports and at loads	3 @ 1000 mm

Section properties were calculated from the following expressions:

I_x = the total net moment of inertia (second moment of area) about X-X axis, which was calculated without using a transformed section analysis.

$$I_x = I_w + I_{fl} \ (mm^4) \tag{1}$$

I_w = the net moment of inertia of the web

$$I_w = \frac{t_{eff}(D - 2T + 2d)^3}{12} \tag{2}$$

For the computation of bending stress of plywood webbed wood I-beams, the net moment of inertia is based on parallel plies plus 0.03 times plies perpendicular to the span (AS 1720.1-1997, section 5, table 5.4 and Appendix J).

$$t_{eff} = 6 + 0.03 \text{ x } 3 = 6.09 \text{ (mm)} \tag{3}$$

I_{fl} = the net moment of inertia of the flange

$$I_{fl} = \frac{B[D^3 - (D - 2T)^3]}{12} \tag{4}$$

Z = the section modulus (a geometrical property of the beam)

$$Z = \frac{2I_x}{D} \ (mm^3) \tag{5}$$

Flange-to-web joints

The flange-to-web joint was a square-edged web glued into routed groove (slot).

Web-web joint

The web ends were butt jointed. Splice plates were centred and glued over the full contact area without staples. The splice plates were made of the same material as the webs with lengths twelve times the panel thickness. A gap of about 6 mm was left between the plates and the flanges.

Web stiffeners

Web stiffeners were cut from plywood with thickness 18 mm and width 50 mm. The arrangement can be seen from table 5. A gap of about 3 mm was left between the web stiffener and the compression flange at bearing ends and between the web stiffener and the tension flange at load points. The other end of the web stiffener was installed in tight contact with the bottom or top flange. The gap prevents the web stiffener from pushing the flanges apart when the bending deflection is large (Leichti *et al.*, 1990).

Design method

Deflection formulas

The deflection of a wood I-beam is taken as a sum of the calculated deflections due to bending and to shear. This can be expressed as:

$$\Delta = \Delta_b + \Delta_s \text{ (mm)} \quad (6)$$

where

$$\Delta_b = \frac{23PL^3}{1296EI_x} \text{ (mm)} \quad (7)$$

Δ	=	total deflection
Δ_b	=	bending deflection
P	=	total load
L	=	span
E	=	modulus of elasticity of the flange material (table 3).

Shear deflection was calculated using APA, supplement 2, formula.

and

$$\Delta_s = \frac{KPL}{6G_w A} \text{ (mm)} \quad (8)$$

Δ_s = shear deflection

A = cross-sectional area of the beam

$$A = 2(BxT) + t(D\text{-}2T) \text{ (mm}^2) \quad (9)$$

G_w = modulus of rigidity of the web (table 1).

K = factor determined by the beam cross-section and is given below:

$$K = \frac{\frac{9}{2}\left[\frac{1}{p}(1-s)+s\right]\left\{\frac{1}{p^2}\left[\frac{s^5}{2}-s^3+\frac{s}{2}\right]+\frac{1}{p}\left[-s^5\left(\frac{3}{30\beta}+\frac{2}{3}\right)+s^3\left(\frac{1}{3\beta}+\frac{2}{3}\right)-\frac{s}{2\beta}+\frac{8}{30\beta}\right]+\frac{8s^5}{30}\right\}}{\left[\frac{1}{p}\left(1-s^3\right)+s^3\right]^2} \quad (10)$$

$$\beta = \frac{G_{fl.}}{G_w} \quad (11)$$

$$p = \frac{t}{B} \quad (12)$$

$$s = \frac{D\text{-}2T}{D} \quad (13)$$

G_{fl} = modulus of rigidity of the flange (table 2)

The other notations are shown in figure 1.
Table 6 summarises the factor 'K' for the I-beams:

Table 6. Section factor, K.

Beam code	K
A1 300, A2 300	3.29
C1 300, C2 300	4.84
K1 300, K2 300	2.19
L1 300, L2 300	3.18

The expected stiffness of the beam, $\frac{P}{\Delta}$, is calculated from equations 6, 7 and 8:

$$\frac{P}{\Delta} = \frac{1}{\frac{23L^3}{1296EI_x} + \frac{KL}{6G_w A}} \text{ (kN / mm)} \quad (14)$$

Bending test arrangements

Third point bending tests on the beams were carried out on the laboratory strong-floor at ***Forest Research***. The beams were loaded upwards and were restrained at their ends by attachments connected to the strong-floor. Figure 2 shows the arrangement of the testing equipment. Total specimen deflection was measured below the beam at mid-point and recorded as a load/deflection curve on a x-y plotter. Buckling restraints were provided at appropriate intervals along the span (see table 5). The rate of loading of the beams was such that failure would be achieved in 3 to 5 minutes.

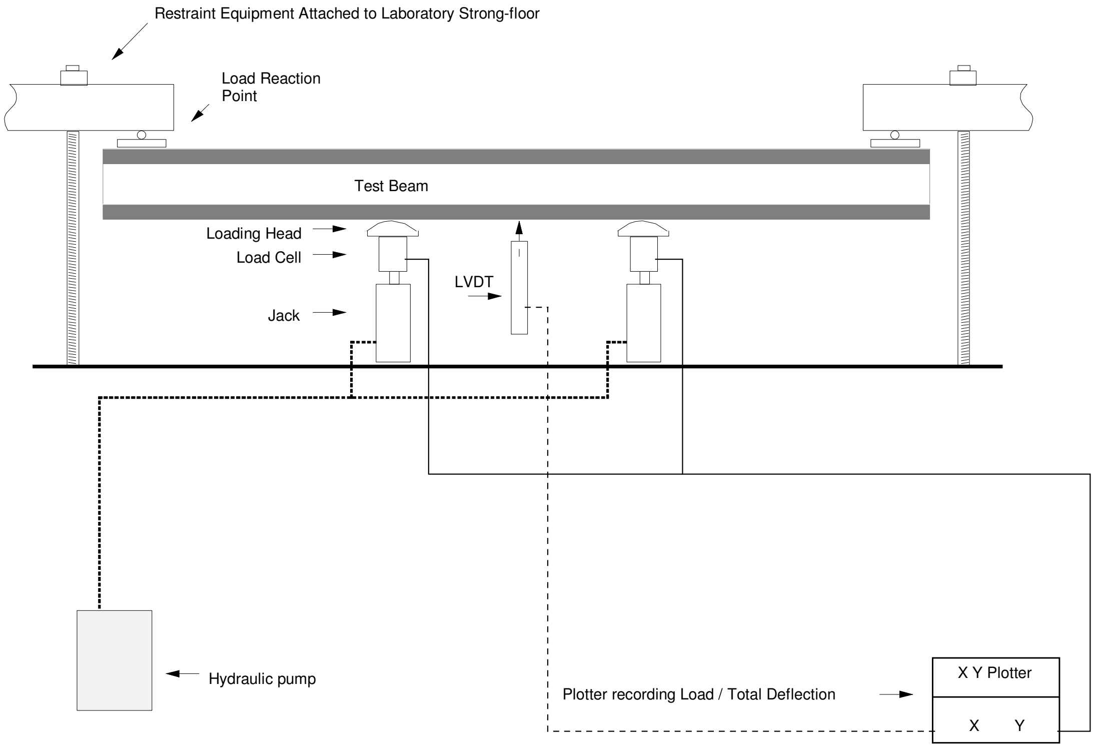

Figure 2. Third-point loading arrangement

Wood I-beams failure modes

<u>I-beams with span 5.9 m</u>

a) Plywood webbed wood I-beams

The webs of the two I-beams buckled at the end reaction. The beams failed near the tension flange by web crushing (figure 3). It was assumed that the web buckling and crushing was caused by a lack of vertical stiffeners at the supports.

b) Hardboard webbed wood I-beams

The I-beams failed explosively in shear breaking the web into several pieces (figure 4) between one of the load points and the end of the beam. The cracks ran at a slope of about 45º. The difference in the failure modes between C1 300 and C2 300 was that the web of C2 300 buckled at one of the end reactions. The reason is that the beam was not supplied with vertical stiffeners at support to prevent the buckling.

Figure 3. Beam A1 300. Web crushing near the tension flange.

Figure 4. Beam C1 300. Hardboard web failure.

I-beams with span 3.9 m

a) Plywood webbed wood I-beams

The compression flanges of the two I-beams buckled sideways between the load points. The sideway buckling of the beams caused adjacent buckling of the web without any web-web or web-to-flange joints failure. Further load increase caused an ‘S’ curve to develop in the beams and a significant end rotation at reaction points (figure 5). Beam K1 300 failed by buckling of the compression flange while the web of beam K2 300 crushed near the compression flange as can be seen from figure 6. It is believed that the significant end rotation and the buckling of beam K2 300 caused the web crushing (see figure 6). It was observed that the gap left between the stiffener at the end of the beam K2 300 and the compression flange was bigger than normally specified by most of the manufacturers (about 3 mm) and the web crushing continued through the gap.

b) Hardboard webbed wood I-beams

The two I-beams presented a typical shear failure located between one of the load points and the end reaction of the beam (figure 7). The web failure did not cause web-to-flange failure or pulling out the stiffeners. The compression flange of the beam L2 300 buckled between one of the supporting ends and the lateral restraint

The failure modes of the wood I-beams with plywood and hardboard webs (span 3.9 m) suggests that an additional lateral support was required - a fully restrained compression flange and a lateral support at bearings to prevent rotation and displacement. In practice buckling of I-beams supporting flooring is not a problem as the compression (the top) flange is fully restrained.

The results showed good bonding for all the beams as there were no failures in the web-to-flange or in the web-web joints.

Figure 5. Beam K1 300. The figure shows the beam buckling and end rotation.

Figure 6. Beam K2 300. End rotation and buckling.

Figure 7. Beam L1 300. Shear failure between end reaction and load point.

Results and discussion

Table 7 summarises the experimental and design data of the wood I-beams. The shaded cells show the ratio of the observed properties of the I-beams and their design values. The results were in good agreement with the design data. The ratios of observed to expected properties ranged from 0.84 to 1.54 with an average of 1.10. The critical failure values (the beams failed in shear or buckling) are indicated in bold.

Table 7. Summary of experimental and design data.

Beam code	P_{ult} (kN)	Stiffness $\frac{P}{\Delta}$ (kN/mm)			Strength of flanges (MPa)			Shear strength (N)		
		Expected	Observed	Obs./exp	F_t	F_{obs}	F_{obs}/F_t	V_R	V_{obs}	V_{obs}/V_R
A1 300	19.2	0.3365	0.3933	1.17	27	23.8	0.88	4.41	**4.35**	0.99
A2 300	18.2	0.3365	0.3746	1.11	27	22.6	0.84	4.41	**4.13**	0.94
C1 300	26.0	0.4601	0.5063	1.10	27	32.3	1.20	7.34	**8.95**	1.22
C2 300	22.8	0.4526	0.5049	1.12	27	28.3	1.05	7.34	**7.85**	1.07
K1 300	20.0	0.6240	**0.6588**	1.06	27	27.9	1.03	4.41	4.50	1.02
K2 300	17.9	0.6240	0.6701	1.07	27	25.0	0.93	4.41	**4.03**	0.91
L1 300	29.9	0.8896	0.9831	1.11	27	41.7	1.54	7.34	**10.3**	1.41
L2 300	24.7	0.8759	1.0686	1.22	27	34.5	1.28	7.34	**8.52**	1.16
				1.12			1.09			1.09
Comparison of the average values for hardboard and plywood webbed wood I-beams										
(1)	25.85	0.67	0.77	1.14		34.2	1.27	7.34	8.90	1.21
(2)	18.82	0.48	0.52	1.10		24.8	0.92	4.41	4.25	0.96
(1)/(2)	1.37	1.40	1.48			1.38		1.66	2.09	

(1) = average value for hardboard webbed wood I-beams (C 300+L 300)
(2) = average value for plywood webbed wood I-beams (A 300+K 300)

In table 7 the following formulas and definitions apply:

P_{ult} = ultimate load

$\frac{P}{\Delta}$ is from equation (14)

F_t = characteristic stress in tension for LVL (table 2)

F_{obs} = observed modulus of rupture

$$F_{obs} = \frac{P_{ult} L}{6Z} \text{ (MPa)} \tag{15}$$

Z is from equation (5)

V_R = strength in shear (table 1)

V_{obs} = observed shear stress in web

$$V_{obs} = \frac{P_{ult.} Q}{2tI_x} \text{ (N)} \tag{16}$$

For I_x see equations (1), (2) and (4).

Q = statical moment (first area moment)

$$Q = Q_w + Q_{fl} \, (\text{mm}^3) \tag{17}$$

Q_w = statical moment of the web

$$Q_w = t_{eff} \, x \, \frac{(D - 2T + 2d)^2}{8} \tag{18}$$

t_{eff} is from equation (3)

Q_{fl} = statical moment of the flanges

$$Q_{fl} = A_{fl} \, x \, \frac{D - T}{2} \tag{19}$$

A_{fl} = net flange area (all web material removed). Table 8 shows the net flange areas of the beams.

Table 8. Net flange area of the beams.

Beam code	Net flange area A_{fl} (mm^2)
A1 300, A2 300	3807
C1 300, C2 300	3861
K1 300, K2 300	2106
L1 300, L2 300	2160

Stiffness

The average observed stiffness of the beams, as a ratio of load to deflection, was higher than the expected stiffness by 12%. There was a good agreement (see the last row of table 7) between the expected difference in stiffness of hardboard and plywood webbed I-beams (ratio of 1.4) and the observed difference in stiffness (ratio of 1.48). The fact that the hardboard webbed I-beams were stiffer than those with plywood webs is thought to be due to two main effects:

- The higher stiffness of the flanges used for hardboard webbed I-beams

As can be seen from table 3 the difference in the flange stiffness is 16%. The influence of flange stiffness on the load capacity was investigated by different authors. For example, Samson (1983) showed that more than 50% of the variation in load-carrying capacity of I-beams is due to the variation of the tension flange. He compared the load capacity in flexure of 100 double-web I-beams with flanges of three lumber laminations and waferboard webs. He found that the maximum load was correlated linearly to the average stiffness of the tension flange laminations.

- The web material

Leichti *et al.* (1990) found that the web material had a significant effect on I-beam stiffness and load capacity. Superfesky and Remaker (1978) investigated 6.4 mm hardboard and 6.4 mm plywood webbed I- beams under short-term destruction loading. They reported that the strength and stiffness of plywood webbed I-beams was about half that of the hardboard webbed beams.

Bending strength

The average ratio of observed bending strength to expected characteristic strength was 1.09. Hardboard webbed I-beams were stronger than plywood webbed I-beams by 38%. As can be seen from the last rows of table 7 the observed bending strength of hardboard webbed I-beams was higher than the expected flange stress by 27% while for the plywood webbed I-beams the bending strength was lower by 8%. This lower value, however, did not cause failure in bending.

Shear strength

The ratio of design shear strength of hardboard to shear strength of plywood was 1.66 while the observed average ratio was 2.09. The average shear strength of plywood (4.25 MPa) was lower than the theoretical by 3.6%. This tends to show that the short term shear strength value for F11 plywood is probably close to the observed value. On the other hand the good shear strength of hardboard indicates that probably the average value derived from the panel shear test underestimates the panel properties.

Conclusions

The conclusions below are based on short-term third point loading of four hardboard webbed wood I-beams and four plywood webbed I-beams.

- Thin oil tempered hardboard manufactured in New Zealand performs well compared to plywood.
- The shear strength was higher for hardboard than for plywood in spite of its thinner dimension.
- Hardboard has potential as a web material in manufacturing wood I-beams. However, further study is required due to the limited test data.

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