Stability of plywood webs of box girder beams

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ABSTRACT

Laboratory tests on relatively small plywood box beams are described. The results suggest that stability of the plywood webs of more realistically sized box beams is not ensured by the rules in timber design codes. A literature search failed to produce sufficient relevant material on plywood panel stability. Hence, the decision to proceed with the theoretical program whose preliminary steps are described in this paper, was made.

Critical loads and other buckling information for plywood webs are presented. The results were produced by finite element analyses in which the plywood was modelled as a layered composite material with each layer being given appropriate linear elastic orthotropic properties. Stability analyses were performed by allocating a small initial curvature to the web panels and applying incremental loads giving geometric non-linearities. Results are presented for various combinations in plane bending and in plane shear. For situations were bending actions predominate it is suggested that the face grain of the plywood should be horizontal but for locations were shear actions are significant there may be advantages in ensuring that the face grain is vertical. The results suggest that current design rules are not conservative. Further studies are planned.

INTRODUCTION

In Year 2 at the University of Auckland, Civil Engineering students make nailed plywood box beams of the geometry shown in Figure 1. The plywood box beams, along with similar span steel beams and reinforced concrete beams, are tested and students are expected to compare the laboratory load deflection curves, such as Figure 2, with solid mechanics and design rules predictions. Making box girder beams is a good exercise for young engineers because it demonstrates how easily and quickly relatively strong and stiff structural members can be made in timber from flexible small pieces of timber. Visiting groups, such as school parties, also enjoy making and testing plywood box beams.

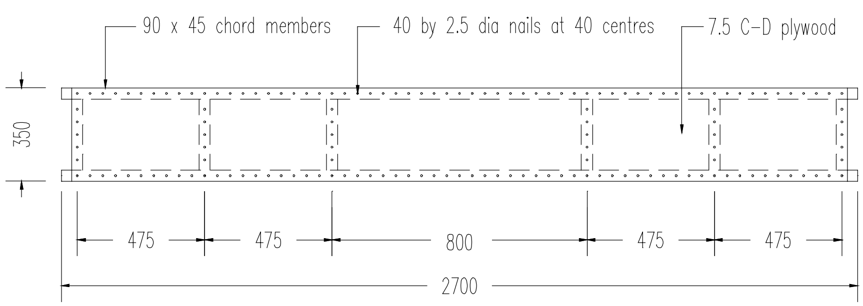

Figure 1 Details of student plywood box beams

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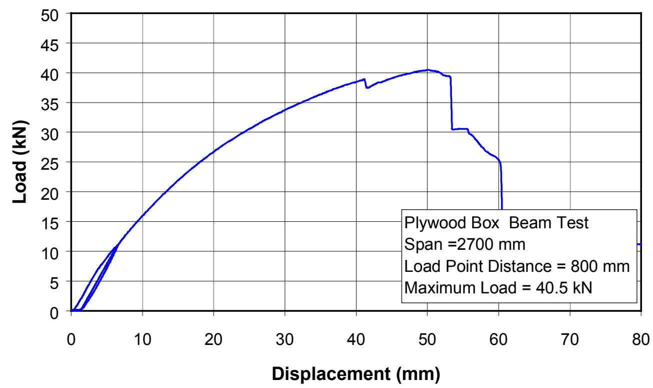

Figure 2 Load displacement behaviour of a typical student plywood box beam

Some 50 of these plywood box beams have been tested during the last 7 years. Typically, at about half the failure load, shear buckles of the plywood webs become noticeable and then failure is usually a result of a bending tension failure of the plywood or a shear failure of the plywood. If the tension flange contains significant defects, such as knots at the upper size limits allowed by grading rules, then failures initiate in the tension flange. These observations of the failure modes of relatively small and short plywood box beams as well as design calculations lead the authors to suspect that

- Plywood stability could well be a problem for larger and more realistically proportioned beams used in industry
- That timber design code rules (NZS 3603:1993, UBC 1997) do not necessarily ensure stability of plywood webs
- Plywood strength issues are unlikely to be significant.

Hence the stability studies of plywood webs, as described in this paper, were initiated.

CODE PROVISIONS FOR STABILITY OF PLYWOOD WEBS

In (UBC 1997) stiffeners are required at, and in (NZS 3603:1993) stiffeners are recommended at, a spacing given by

$$b = b'\left[1 + \frac{100 - p}{25}\right] \qquad (1)$$

where b' = actual stiffener spacing
b = stiffener spacing in a table for (UBC 1997) or in a figure for (NZS:3603)
p = percentage of maximum code value of plywood shear existing at the section.

In (UBC 1997) the maximum value allowed for b' is 48 in. (1220 mm) while in (NZS:3603) it is $3b$ or $3h_w$ where h_w is the clear distance between flanges. For 3/8 in ply (UBC 1997) gives b = 15 in. (380mm) for $h_w \geq 10$ in. (255 mm) while for ½ in. ply b = 27 in. (685 mm) or h_w = 10 in. (250 mm) and b = 22 in. (560 mm) for $h_w \geq 20$ in. (510 mm). For 75 mm ply (NZS 3603) gives b = 400 mm for $h_w \geq 250$ mm while for 12.5 mm ply $b \approx 750$ mm for $h_w \geq 600$ mm.

In Table 1 nominal, strength limit state, (NZS 3603:1993) code values of maximum shear are listed for the panels for which critical values are given in Table 4.

Table 1 Code (NZS 3603:1993) nominal strength values for 7.5 mm thick plywood webs

Clear depth between flanges (mm)	Web stiffener spacing (mm)	Maximum allowable panel shear at nominal strength	
		Action (N/mm)	Stress (MPa)
≥ 250	900	24.2	3.25
	600	30.8	4.11
	450	34.1	4.55

ANALYSIS OF STUDENT PLY BOX BEAM RESULTS

Failure loads for the 6 student beams of 1999 are listed in Table 2.

Table 2 Failure loads for 1999 student ply box beams

Date	Failure load (kN)
26/8/99	41.0
27/8/99	36.0
30/8/99	44.5
31/8/99	48.7
2/9/99	46.6
3/9/99	49.5
Average	44.4

Assumptions of linear elastic behaviour, plane sections remaining plane and the values listed in the New Zealand Timber Design Code (NZS:3603) lead to the stresses and nail forces of Table 3. According to NZS 3603, the nominal strength of the beams is controlled by the tension strength of the timber flanges and is about 25 kN (45 x 8.8/15.8) and the design strength is about 20kN (the capacity factor times the nominal strength). Note that, for a stiffener spacing of 475 mm and clear distance between webs of 275 mm as used in the student ply box beams, Equation 1 gives p as 95.3% so the allowable maximum panel shear stress of Table 3 reduces from 4.7 MPa to 4.48 MPa at the nominal strength limit state.

Table 3 Strength values for student ply box beams

	From a load of 45 kN on beam	Characteristic values in design code*
Maximum tension stress in timber flange	15.8 MPa	8.8 MPa
Maximum compression stress in timber flange	15.8 MPa	20.9 MPa
Maximum tension bending stress in ply webs	20.7 MPa	28.8 MPa
Maximum panel shear stress in ply webs	5.93 MPa	4.7 MPa
Maximum lateral nail force	587 N	567 N

*characteristic values are estimates of 5% lower limit strength values in (NZS 3603:1993)

STABILITY OF ORTHOTROPIC PLATES

Plywood webs can be regarded as orthotropic plates. A literature search found limited information that was relevant the situation for plywood webs. (Galambos, 1976), (Allen and Bulson, 1980), (Aston and Whitney, 1970) and (Whitney, 1987) proved useful and gave indications of ideas that might be relevant as well as supplying a limited number of solutions that the following finite element solutions could be checked against.

Finite element solutions.

In the Department of Civil and Resource Engineering, University of Auckland the LUSAS finite element package (FEA Ltd, 1999) is available. LUSAS has an extensive library of material models, element types and can handle non-linear geometries as well as non-linear materials.

The plywood was modelled as a material with composite geometry which means that, for the analyses of this paper, different linear elastic orthotropic plane stress properties could be allocated to each layer (i.e. ply of the plywood). Thin shell isoparametric elements, that allowed for membrane as well as bending actions and used quadratic interpolation functions, were used to model the plywood webs.

Stability analyses were performed by allocating a small initial curvature and applying incremental loads (shear, bending and direct stress actions as appropriate) to the boundaries. The resulting geometric non-linearities were accounted for using total Lagrangian formulations. For the initial analyses it was assumed that the plywood web panels were rectangular with simply supported edges. For the nailed ply box beams, that are common in New Zealand, it is felt that the boundary conditions are close to simply supported. Other analyses were performed assuming that the edges are built in and it is intended that, the actual situations in glued as well as nailed ply box beams, will be modelled in the future.

Typical solutions

Shear versus out of plane maximum deflections are shown in Figure 3 for a 600 mm square simply supported 7.5 mm thick 3 ply plate with various, out of plane, initial curvatures. Uniform in-plane shear actions were applied along each boundary. The 7.5mm 3 ply was modelled as having 2.5 mm thick veneers subjected to plane stress with Young's modulus along the grain of 10 000 MPa, Young's modulus across the grain of 667 MPa, shear modulus of 667 MPa, along to across the grain Poisson's ratio of 0.375 and across to across the grain Poisson's ratio of 0.5.

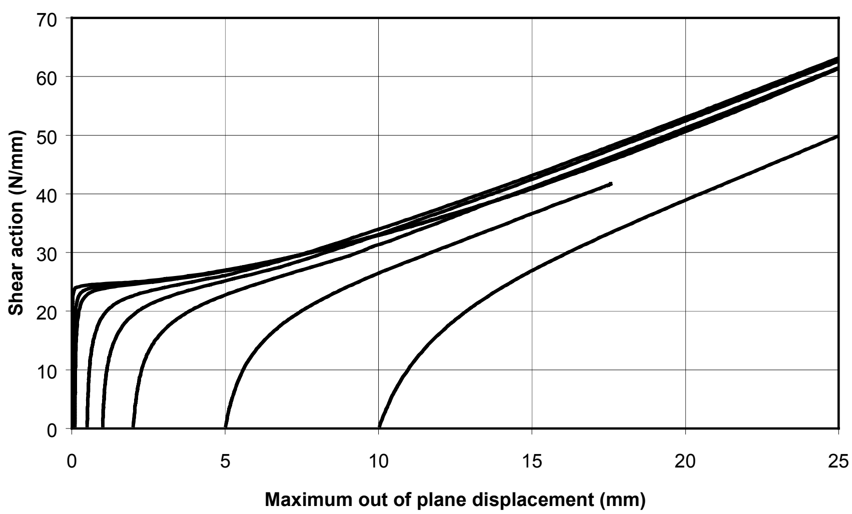

Figure 3 Out of plane displacement for various initial curvature - pure shear on simply support 600 mm square 7.5 mm 3 ply

As the shear load increases, the solutions tend towards that for a small initial curvature. A projection backwards, of the almost horizontal part of the load deflection curve for a small initial curvature in Figure 3, gives the critical load at the zero deflection ordinate. Note also that, the load keeps on increasing as the plate buckles. This does not usually happen for slender columns.

Buckled shapes

In pure shear plates buckle in the diagonal direction, that is approximately perpendicular to the principal compression direction and parallel to the principal tension direction, as shown in Figure 4. In pure bending, the longitudinal tension stresses try to keep the lower half of the plate straight while the compression stresses cause buckles to develop. For 7.5 mm 3 ply with the face grain horizontal, a single compression buckle, as shown in Figure 5, usually develops. When the panels are relatively long compared to their depth, then restraint from the top and bottom flanges has a significant effect. Local buckling, something like that shown in Figure 6, does occur when the 3 ply face grain is vertical. A buckled shape for shear plus bending is shown in Figure 7.

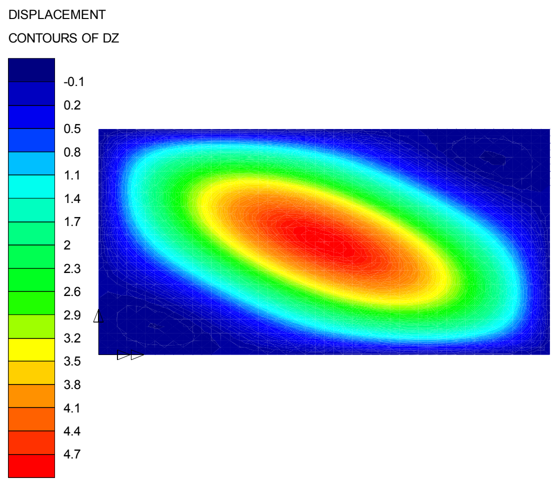

Figure 4 Buckled shape for pure shear - simply supported 1200 by 600 mm 7.5 mm 3 ply with face grain horizontal

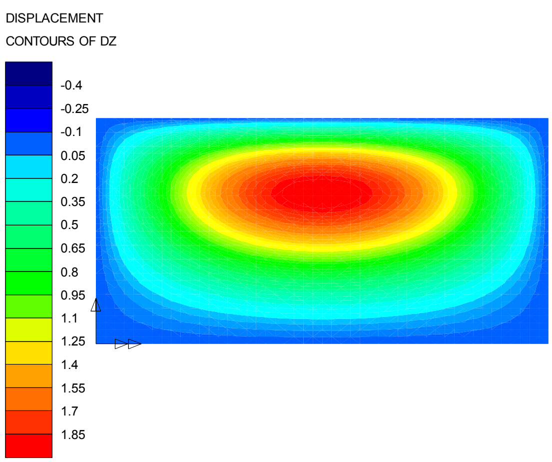

Figure 5 Buckled shape for pure in plane bending - simply supported 1200 by 600 mm 7.5 mm 3 ply with face grain horizontal

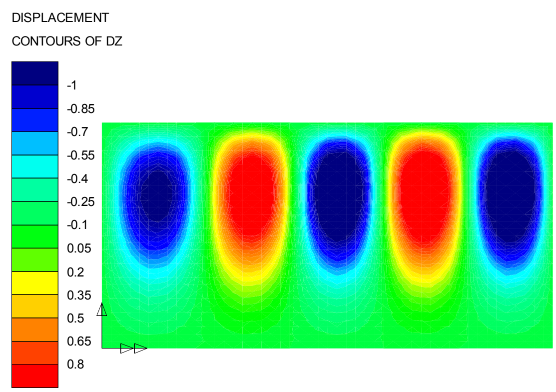

Figure 6 Buckled shape for pure in plane bending - simply supported 1200 by 600 mm 7.5 mm 3 ply with face grain vertical

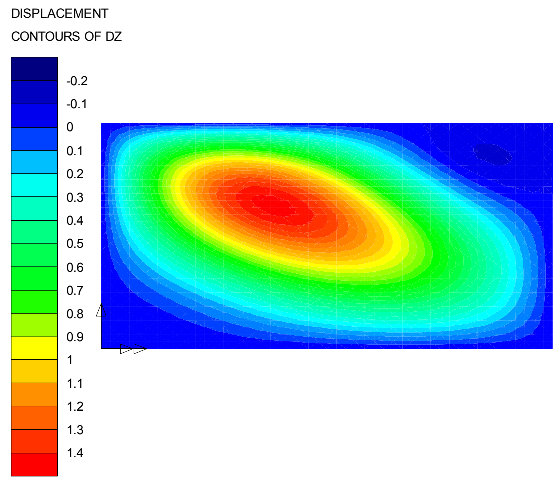

Figure 7 Buckled shape for bending plus shear - simply supported 1200 by 600 mm 7.5 mm 3 ply with face grain horizontal

Interaction of bending and shear

Critical values, for the combinations of bending and shear listed in Table 5, give the plotted points on Figure 8, and it can also be seen how well interaction formulae of the forms

$$\left(\frac{\text{actual shear}}{\text{critical shear by itself}}\right)^2 + \left(\frac{\text{actual bending}}{\text{critical bending by itself}}\right) = 1 \qquad (2)$$

and

$$\left(\frac{\text{actual shear}}{\text{critical shear by itself}}\right)^2 + \left(\frac{\text{actual bending}}{\text{critical bending by itself}}\right)^2 = 1 \qquad (3)$$

fit the results.

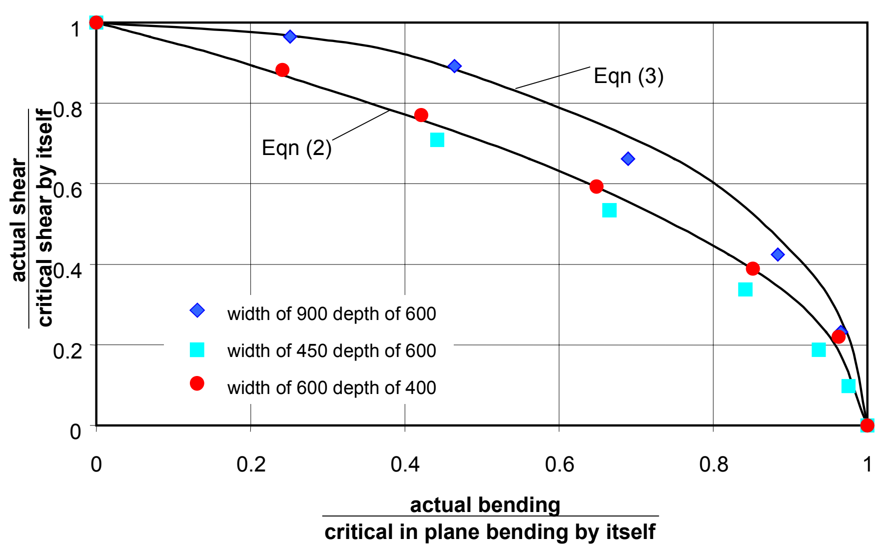

Figure 8 Interaction of shear plus bending for simply supported 7.5 mm 3 ply with face grain horizontal

Table 4 Critical values for shear plus bending for a simply supported 7.5 mm 3 ply panel with face grain horizontal

Width of 900 and depth of 600				Width of 450 and depth of 600				Width of 600 and depth of 400			
Shear		Maximum bending		Shear		Maximum bending		Shear		Maximum bending	
Action (N/mm)	Stress[1] (MPa)	Action (N/mm)	Stress[2] (MPa)	Action (N/mm)	Stress[1] (MPa)	Action (N/mm)	Stress[2] (MPa)	Action (N/mm)	Stress[1] (MPa)	Action (N/mm)	Stress[2] (MPa)
11.4	1.52	0	0	35.4	4.72	0	0	26.8	3.57	0	0
11.0	1.47	11.0	2.20	25.1	3.35	25.1	5.02	23.7	3.16	23.7	4.74
10.2	1.36	20.4	4.07	18.9	2.52	37.8	7.56	20.7	2.76	41.4	8.28
7.6	1.01	30.2	6.04	12.0	1.59	47.8	9.56	15.9	2.12	63.7	12.74
4.8	0.65	38.7	7.74	6.7	0.89	53.2	10.64	10.5	1.39	83.6	16.72
2.6	0.35	42.3	8.46	3.5	0.46	55.4	11.08	5.9	0.79	94.5	18.90
0	0	43.8	8.76	0	0	56.8	11.36	0	0	98.2	19. 6

[1] based on shear thickness of 7.5 mm
[2] based on total ply with the grain parallel to stress of 5 mm.

Direction of face grain plywood

Three ply plate bending stiffness associated with the direction of the face grain is about 10 times the plate bending stiffness associated with the direction perpendicular to the face grain so it should not be surprising that, when the face grain is orientated in a vertical direction, the buckling situation can be somewhat different from that when the face grain is horizontal. Changes in critical buckling actions can be judged from the values in Table 5. When interpreting the values in Table 5, keep in mind that a maximum bending action of 20 N/mm for horizontal face grain occurs at approximately the same timber flange stress as a maximum bending action of 10 N/mm for vertical face grain.

Table 5 Effects of direction of face grain on critical values for simply supported 7.5 mm 3 ply

Panel size		Direction of face grain	Pure shear		In plane pure bending	
Width (mm)	Depth (mm)		Action (N/mm)	Stress[1] (MPa)	Maximum action (N/mm)	Maximum stress[2] (MPa)
1200	600	horizontal	7.88	1.05	50.70	10.10
		vertical	20.95	2.79	44.07	17.80
600	600	horizontal	24.35	3.25	46.35	9.27
		vertical	24.35	3.25	42.20	16.90

[1] based on shear thickness of 7.5 mm
[2] based on total ply with the grain parallel to stress of 5 mm.

Boundary conditions

Critical values for selected fixed edge and simply panels are listed in Table 6. Real panels might well be somewhat better than simply supported but not as good as fixed. For nailed ply box beams, it should be appreciated that the edges of the panels are partially fixed against rotation by the nails thus increasing critical loads over the simply supported case. But, the loads are also applied to the panels by nails so the loads are not applied down the centrelines of the plywood panels, which makes the buckling situation somewhat worse than that assumed for the simply supported analyses of this paper.

Table 6 Effects of boundary conditions on critical values for simply supported and fixed 7.5 mm 3 ply panels with face grain horizontal

Panel size		Boundaries	Pure shear		In plane pure bending	
Width (mm)	Depth (mm)		Action (N/mm)	Stress[1] (MPa)	Maximum action (N/mm)	Maximum stress[2] (MPa)
1200	600	Simply supported	7.88	1.05	50.70	10.10
		Fixed	13.85	1.85	97.40	19.50
600	600	Simply supported	24.35	3.25	46.35	9.27
		Fixed	40.20	5.36	130.0	26.00

[1] based on shear thickness of 7.5 mm
[2] based on total ply with the grain parallel to stress of 5 mm.

CONCLUSIONS

The results of limited experimental testing of small sized plywood box beams as well as the preliminary results given by finite element analyses of the stability of plywood panels suggest that existing code rules for the design of plywood webs may not be conservative. Further finite element studies, with more realistic bound conditions, are warranted.

A major advantage of the finite element stability analyses is that the effects of various parameters can easily and quickly be studied. As an example, the results suggest that the face grain of the plywood should be horizontal for situations where bending stresses predominate but for situations where shear stresses are significant there may be advantages in ensuring that the face grain of the plywood webs is vertical.

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