Journal of Tropical Forest Science 5(4): 492-511

STRENGTH OF PLYWOOD-WEB BOX BEAM

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Received March 1992

CHU, Y.P., ROSZALLI HJ. MOHD & CHONG, Y. T. 1993. Strength of plywood-web box beam. This paper describes the loading tests carried out on box beams of two different lengths but of the same cross section. The overall sizes of the beams were 115 *mm* × 395 *mm* × 5.4 *m* and 115 *mm* × 395 *mm* × 7.2 *m*. The box beams were assembled by nailing plywood webs to both sides of timber without the use of adhesives. The box beams were designed according to standard engineering practice and the tests were carried out to confirm the design. Examples of design calculation of the two types of box beam are given.

Key words: Web box beam - WBP plywood - timbers - strength groups - loading - performance

CHU, Y.P., ROSZALLI HJ. MOHD & CHONG, Y.T. 1993. Kekuatan papan lapis - alang kekotak. Kandungan kertas ini mengenai ujian bebanan yang dijalankan terhadap alang kekotak [box beam] yang mempunyai dua ukuran panjang yang berbeza tetapi permukaannya adalah sama. Ukuran bagi alang kekotak yang pertama ialah 115*mm* × 395*mm* × 5.4*m* dan yang berikutnya pula ialah 115*mm* × 395*mm* × 7.2*m*. Pemasangan alang kekotak telah dibuat dengan memaku kepingan papan lapis (plywood web) pada kedua-dua rangka kayu tanpa menggunakan bahan perekat. Alang kekotak ini telah direka mengikut amalan kejuruteraan dan ujian yang telah dijalankan untuk mempastikan yang ia mengikut rekaan. Contoh-contoh pengiraan mengenai rekabentuk untuk dua jenis alang kekotak turut disediakan.

Introduction

Plywood-web box beams are made by combining timber with plywood. The profiles can be of I or box-shaped sections. This paper is confined only to box-shaped beams. Such beams are highly efficient structural components similar to the "I" or "channel" sections of steel beams. When compared to solid timber beams having the same strength, less timber is required in making these beams. Moreover, the timbers required for the beam flanges (and stiffeners) are much smaller in cross-section than those of the solid beam and are, therefore, more readily obtainable. The only extra materials required are the plywood webs and nails and the extra labour in manufacturing the beams. Like glue-laminated beams, box-beams are not restricted in their total length as they can be joined virtually to any required length. The use of box-beams is another system of utilizing small-sized timbers combined with plywood.

General description

The box beam consists of top and bottom flanges to resist bending moment. The flanges are joined by plywood webs of sufficient thickness to resist shearing stress. The plywood webs are nailed to both sides of the flanges. However, for better stiffness, the beams may be assembled by gluing instead of nailing but in this case the timber flanges have to be conditioned first to a sufficiently low moisture content before gluing. Also, in gluing it is necessary to have close factory controlled environment during manufacture and it must be in accordance with glue manufacturers' recommendations.

Stiffeners are required to prevent the buckling of the webs when the flanges move towards one another as the beam is being loaded. They are needed at both ends of the beam and at intermediate points along the beam as well as at positions subjected to concentrated loads. An upward camber may be built into the beam during assembly.

Design

Plywood web beams are particularly suitable for use as roof beams to support a light-weight roof system for assembly buildings, lecture halls or similar buildings of spans 5 to 20 *m*.

In the test described here, two beams of different lengths but same cross-sectional area were tested. The design data for them are given in Table 1.

Table 1. Design data for box beams

Overall dimension *(mm)*	Effective span *(m)*	Spacing of beam *(m)*	Dead load *(kNm^{-2})*	Live load *(kNm^{-2})*	Design load *(kNm^{-1})*
115×395×5400	5.25	2.5	0.5	0.5	2.5
115×395×7200	7.05	1.2	0.5	0.5	1.2

The design calculations for the two beams are presented in the Appendix. They were based on the use of these beams as roof members without plaster ceiling and the data for design were taken from the local code of practice (Anonymous 1978) as well as foreign code such as the British Code of Practice CP 112 (Anonymous 1971) if the required data were not available in the local code.

Materials and methods

Preparation of beam

Materials

The timbers used were mostly merpauh (*Swintonia* sp.) with some pieces of machang (*Mangifera* sp.) and sepetir (*Sindora* sp.). Plywood for webs was obtained commercially and was of WBP type, 5-ply and nominal 9 *mm* thick. An examination on a few pieces of plywood revealed that they were manufactured from veneers of mixed species such as mersawa (*Anisoptera* sp.) (mostly for face veneers), kedondong (Burseraceae family), geronggong (*Cratoxylum* sp.), rambutan hutan (*Nephelium* sp.), jelawai (*Terminalia* sp.), mengkulang (*Heritiera* sp.) and perhaps others.

The nails used were ordinary wire nails of size 2 *mm* diameter x 38 *mm* long.

Fabrication

Altogether seven box-beams were fabricated for testing. They comprised three beams of 5.4 *m* and four of 7.2 *m* lengths as shown in Figures 1 and 2. Only two beams of each type were tested in accordance with design load as in Table 1, while the other beams were tested with different design loads.

All timbers were structurally graded, in accordance with Malaysian Grading Rules (Anonymous 1968) and were mostly of Standard Structural grade together with some Common Structural grade. They were air-dried to a moisture content of not more than 22 % and were planed to the required sizes.

Each plywood width of 1220 *mm* was cut into three pieces of equal widths approximately 400 *mm* each. Full lengths of plywood were used except for a few pieces of 600 *mm* length for the 5.4 *m* beam as shown in Figure 1. The plywood sizes were slightly bigger than the timber framework to allow for tolerances and also for a slight camber of the beam if feasible.

During fabrication, all timber members (flanges and stiffeners) were held together by slant nailing to make the timber framework. On top of this framework, plywood webs with face grain parallel to span were fixed in position by approximately half the required number of nails. The whole assembly was then turned over as another set of plywood was placed over the framework and the full set of nails were driven in. The remaining nails for the opposite side were then driven in to complete the assembly. A nailing template was used to mark the position of the nails.

Method of test

Each plywood box beam was simply supported over a bearing length of 100 *mm* at each end. The effective span as given in Table 1 was the distance between the centres of the bearing length. Lateral restraint was provided to the beam at the top flange of the beam by using battens placed at 800 *mm* intervals and nailed to a fixed support at one end and at the other end to the beam through over-sized holes to eliminate vertical restraint. Loading in the form of dead weights was applied to the top flange. They were placed at eight equally spaced points for the 5.4 *m* beam and ten equally spaced points for the 7.2 *m* beam. These equally spaced loading points were meant to simulate uniformly distributed load (UDL) on the beam. Deflection readings were obtained using five dial gauges spaced approximately equally along the bottom of the beam with one dial gauge positioned at mid-span.

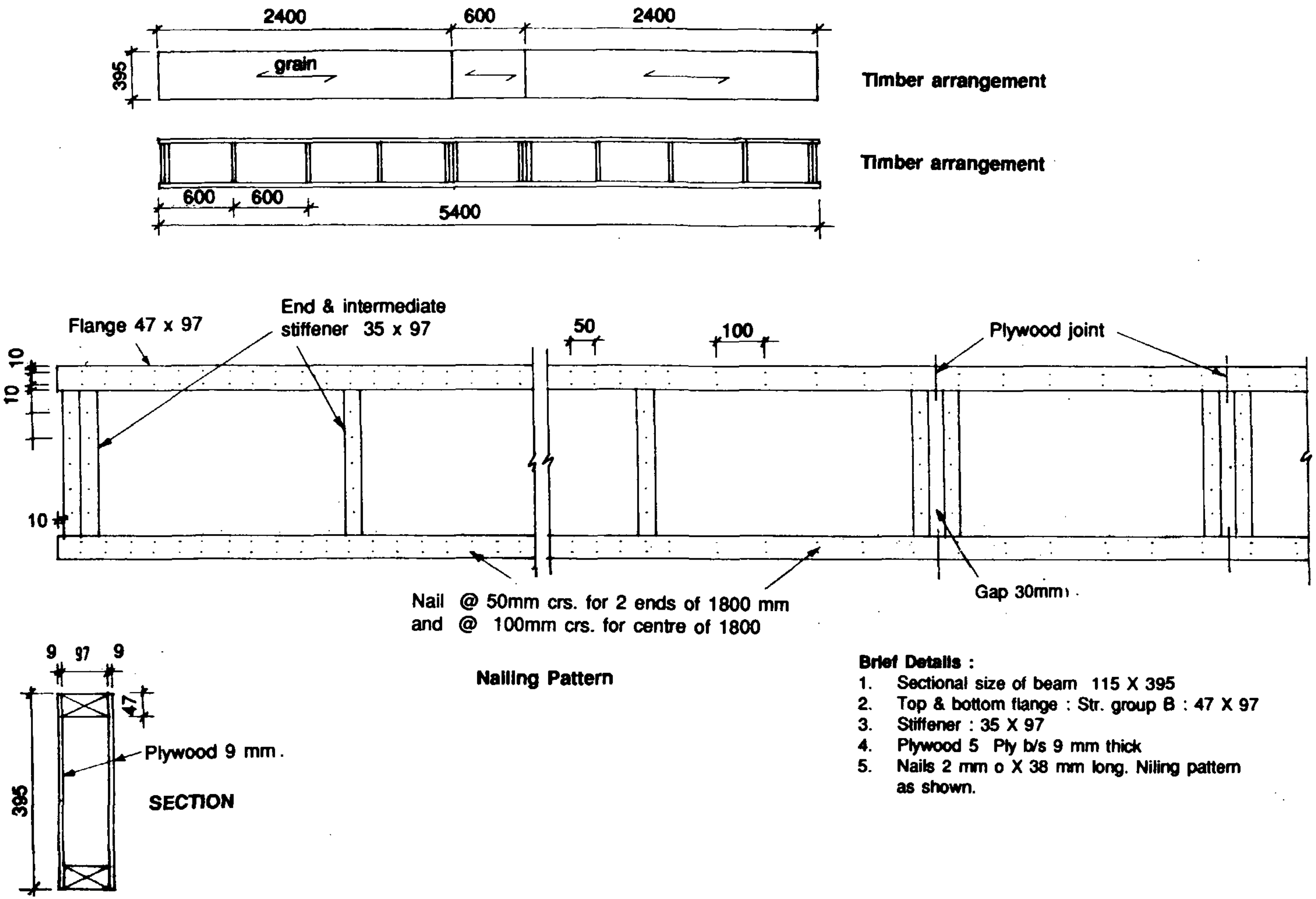

Figure 1. Details of tested box beam 5.4 *m* long

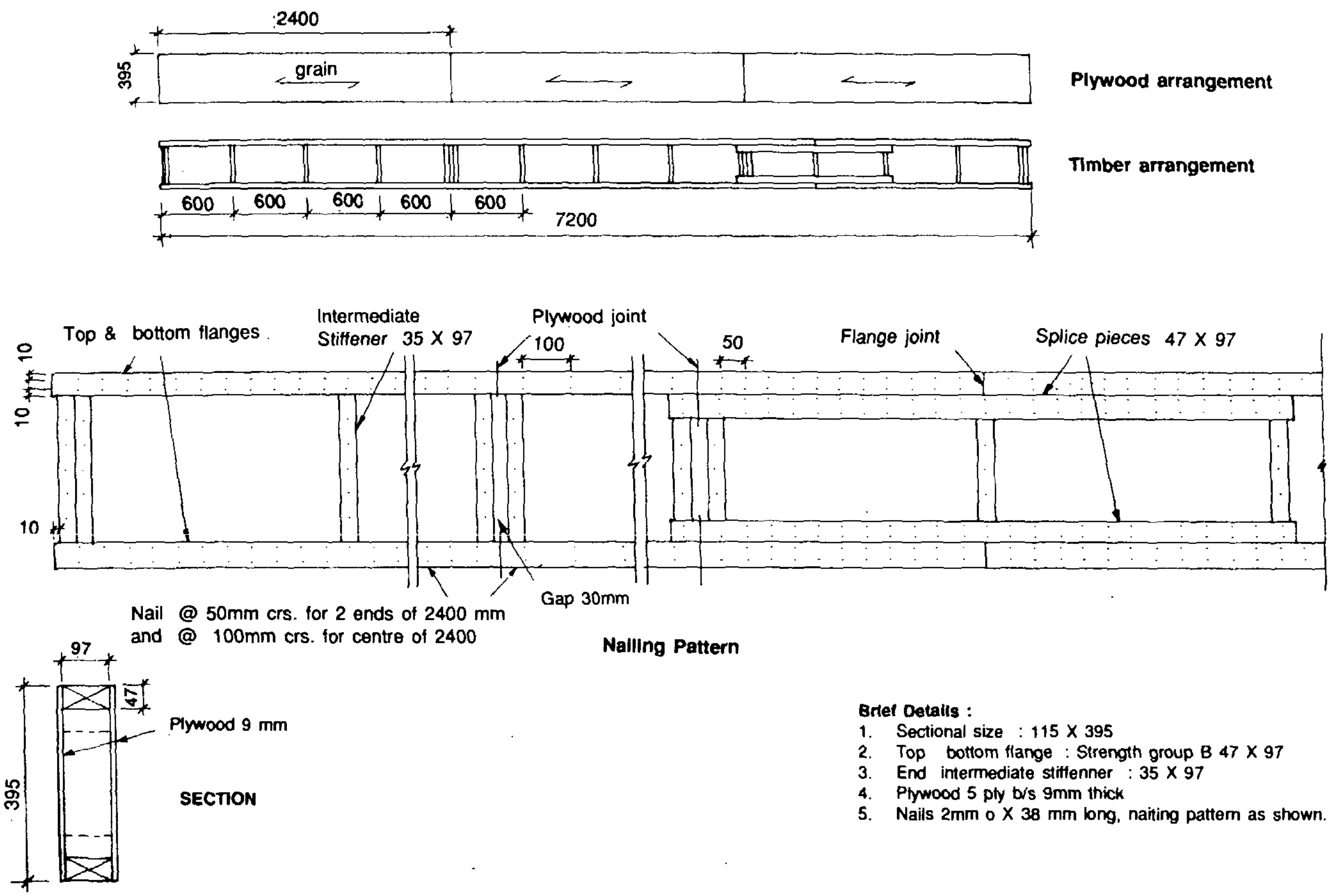

Figure 2. Details of tested box beam 7.2 *m* long

The procedure of testing, based on Malaysian Standard MS 544:1978 (Anonymous 1978), was as follows:-

(1) *Preload test*
A preload equal to the long-term load (dead load) was applied. This was maintained for 30*min* and then released. Deflection readings were taken 15*min* after release of load to establish a datum for the deflection test as follows:

(2) *Deflection test*
The long-term load was now re-applied in four equal increments and maintained for 15*min*. The load was then increased up to full design load (dead load plus live load) in another four equal increments and this was maintained for 24*h* and then released. The rate of loading was fairly uniform and the time taken to reach full design load from zero load was not less than 30*min*. This test was to study the load/deflection as well as the deflection/time characteristics of the beam.

(3) *Strength test*
After 15*min* in the unloaded condition, full design load was re-applied as under the deflection test. The load was then increased in increments up to failure load if the beam failed prematurely or to 2 times the design load and then maintained for 15*min*. The beam was then loaded to destruction.

Deflection readings for the above tests were taken at each change of load as well as at constant load during the 24-*h* interval. Once the reading at the mid-span approached the maximum range of the dial gauge, the dial gauge was removed and readings were taken from a ruler attached there.

After the beams had been tested to failure, small clear specimens (20 × 20 *mm*) were cut from the timber flanges in order to determine the timber properties in static bending and compression parallel to the grain based on British standard BS 373: 1957 (Anonymous 1957). Moisture content and specific gravity of the timber were also assessed from these specimens. The plywood webs for two beams were also tested to determine some plywood properties in accordance with British standard BS 4512 : 1969 (Anonymous 1969).

Results and discussion

The test results of the seven box beams are summarised in Table 2. As mentioned previously, only beams 2, 3, 6 and 7 follow the design loads as given in Table 1. The other beams were loaded at different design loads in order to observe their performance. The average strength properties of timbers used are given in Table 3 and strength properties of plywood in Table 4. Figure 3 shows the load-deflection relationship of the beam during loading and Figure 4 gives the deflection/time curve during the 24 *h* while the design load was on the beam.

The criteria for accepting the beam as a sound structural member are:-

Table 2. Test results of plywood box beams

Box beam No.	Effect-ive span (*m*)	Weight of beam (*kN*)	Preload (DL) (*kN*)	Design* load (DL+LL) (*kN*)	Deflection at preload: After 30 *min* at preload (*mm*)	Deflection at preload: 15 *min* after preload is removed (*mm*)	Deflection at design load: Immediate-ly (*mm*)	Deflection at design load: After 24 *h* (*mm*)	Deflection at design load: Percent-age in-crease (%)	Ratio of deflection to span (9) / (2)	Ultimate load: Amount (*kN*)	Ultimate load: Load factor (ult./ design)	Mode of failure
1	2	3	4	5	6	7	8	9	10	11	12	13	14
(A) Size : 115 × 397 × 5400 *mm*													
1	5.25	0.64	5.78	11.55	4.49	0.32	8.97	10.59	18.1	0.0020	43.41	3.76	Failure at knot at bottom flange, shear in plywood web
2.	5.25	0.66	6.56	13.12	5.34	0.40	9.86	11.08	12.4	0.0021	52.53	4.00	At top and bottom flanges and shear in plywood near centre
3	5.25	0.62	6.56	13.12	6.50	0.66	11.84	13.49	13.9	0.0026	53.35	4.07	At top flange at plywood joint
(B) Size : 115 × 397× 7200 *mm*													
4	7.05	0.97	7.05	21.15	10.61	1.31	37.18	43.50	17.0	0.0062	33.65	1.59	Failure at top flange joint and opening up of plywood joint
5	7.05	0.90	6.70	13.40	12.26	1.64	25.84	30.48	18.0	0.0043	28.42	2.12	At top and bottom flanges at plywood joint
6	7.05	0.90	4.23	8.46	6.84	0.42	12.75	14.67	15.1	0.0021	28.91	3.42	- ditto -
7	7.05	0.97	4.23	8.46	8.52	0.67	16.33	20.13	23.3	0.0029	29.53	3.49	Failure at knot at top flange near mid-span and followed by plywood shearing

* Only beams Nos. 2, 3, 6 and 7 follow the design loads as given in Table 1.

Table 3. Strength properties of timbers

Box beam No.	Timber species	No. of test specimens	Moisture content %	Specific gravity (O.D wt./ vol. at test)	Modulus of rupture (MOR) (Nmm^2)	Modulus of elasticity (MOE) (Nmm^2)	Compression parallel (Nmm^2)
1	Not tested	-	-	-	-	-	-
2	Merpauh	2	12.6	0.50	63.1	11,400	41.0
3	Merpauh	1	16.1	0.48	81.4	11,100	34.0
4	Merpauh	4	18.7	0.58	67.1	11,900	35.3
5	Machang	3	17.8	0.50	89.6	13,600	39.6
6	Merpauh	2	12.3	0.59	69.8	13,100	42.5
7	Merpauh	2	15.9	0.61	72.0	9,400	38.8

Table 4. Strength properties of plywood

Box beam No.	Veneer species (5-ply)	No. of test specimens	Moisture content %	Specific gravity (O.D wt./ vol. at test)	Maximum panel shear stress (Nmm^2)	Static bending test MOR (Nmm^2) Parallel	Static bending test MOR (Nmm^2) Perpendicular	Static bending test MOE (Nmm^2) Parallel	Static bending test MOE (Nmm^2) Perpendicular
3	Mersawa/mersawa/ mengkulang/mersawa/ mersawa	2 for panel shear, 4 each for static bending	13.1	0.62	6.9	47.5	46.5	7120	5210
7	Mersawa/kedondong/ geronggang/ kedondong/mersawa	2 for panel shear	-	-	5.9	-	-	-	-
	Mersawa/rambutan hutan/jelawai/ rambutan hutan/ mersawa	1 for panel shear	-	-	6.5	-	-	-	-

(1) The maximum deflection of the beam at the end of a 24-*h* loading period should not exceed 0.8 times the permissible amount of the design. The permissible amount depends on its functional requirement, and since this was a roof beam without plaster ceiling or other finishings that might be damaged by the deflection, the amount was set at 0.004 of the span. Therefore, the maximum deflection should not exceed 0.0032 of the span. In addition, the rate of increase in deflection during the 24-*h* period should tend to decrease.

(2) The beam should be able to sustain a load of $2\frac{1}{2}$ times the design load for 15 *min* without failure.

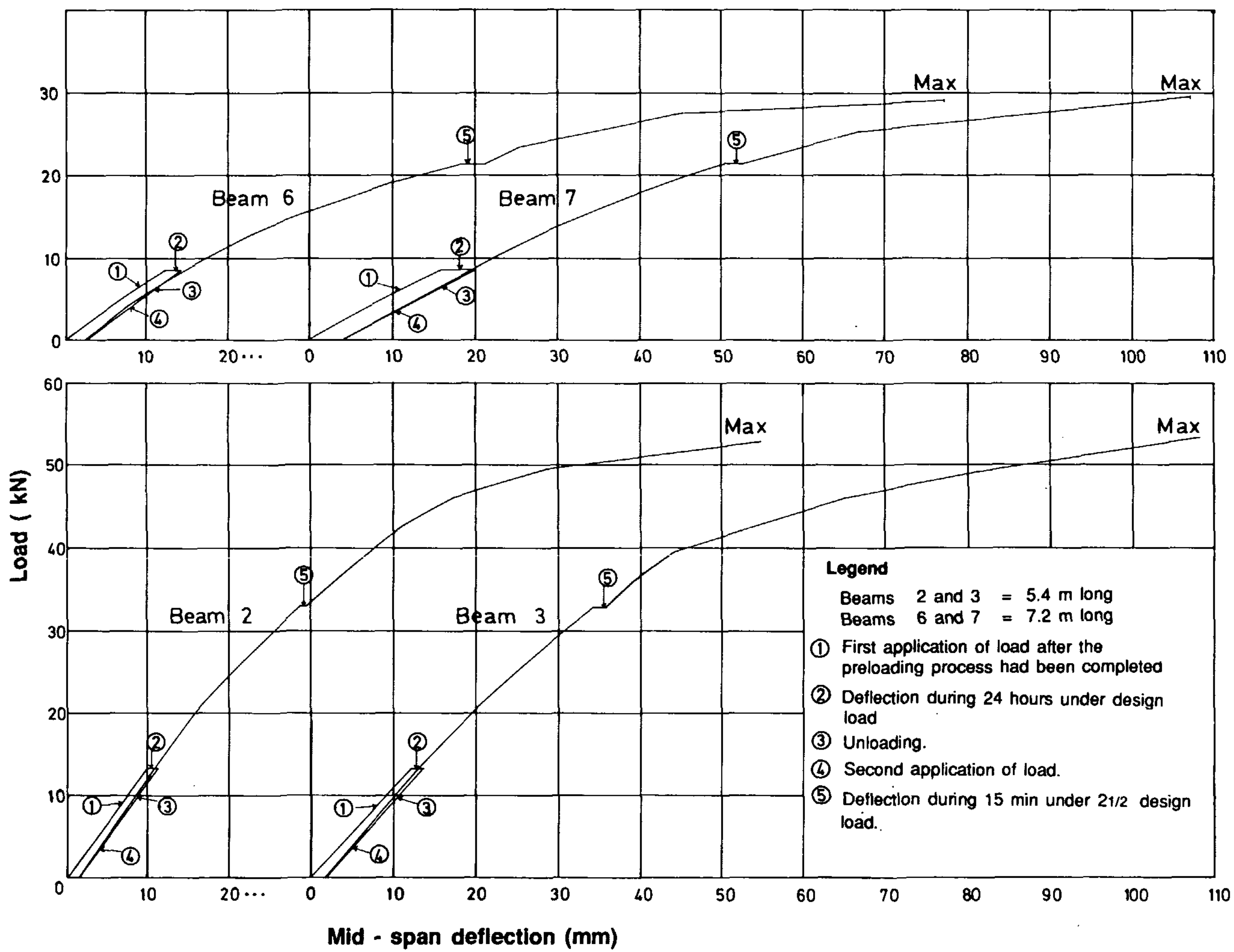

Figure 3. Load-Deflection curve for box beams

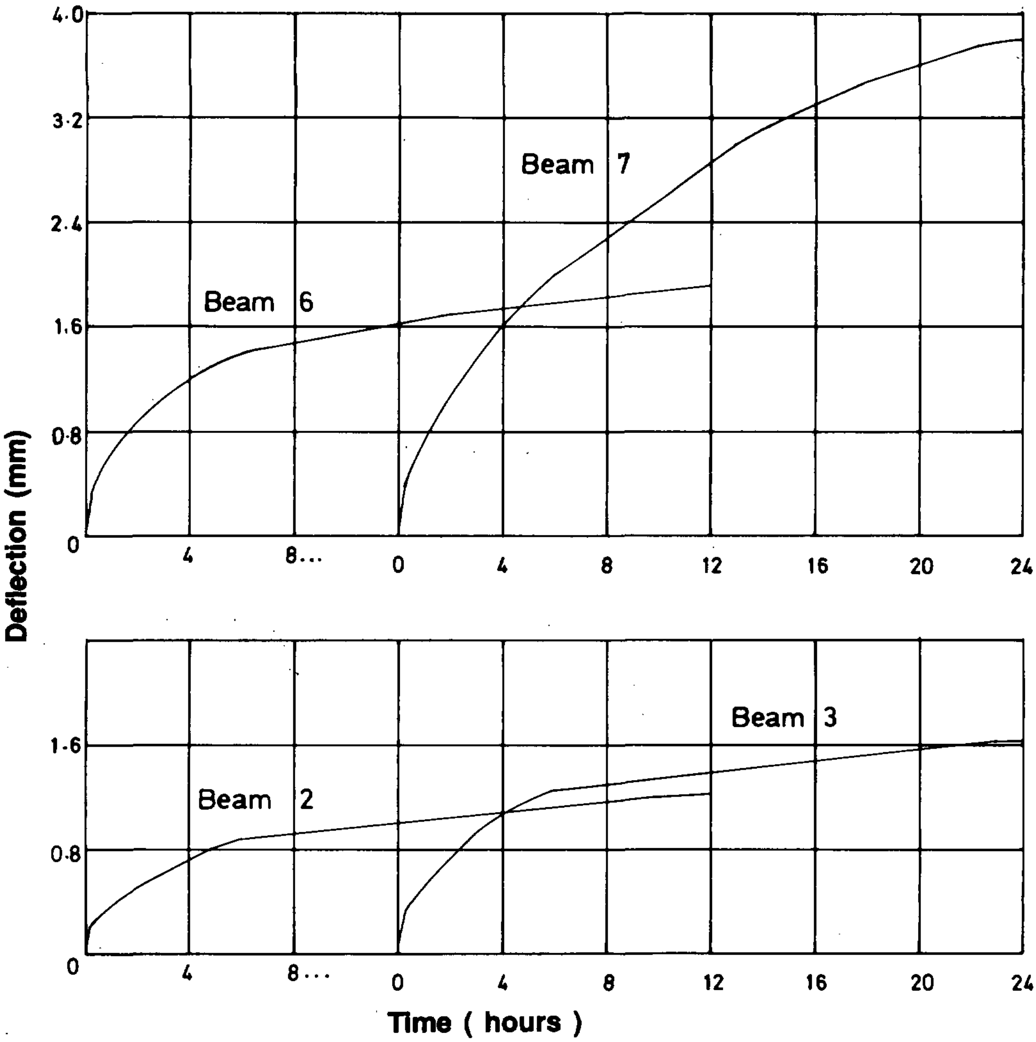

Figure 4. Deflection - Time curve

From the results shown in Table 2, it can be seen that all the beams that had been loaded with the intended design load, that is beams 2, 3, 6 and 7, passed the test in both the deflection requirement as indicated in column 11, and the ultimate load requirement as shown in column 13. While the deflection requirement was somewhat close to the allowable limit of 0.0032 of the span, the ultimate to design load ratio was much higher than the allowable figure of 2.5. This was also reflected in the theoretical calculations given in the Appendix where the permissible stresses of the material in bending, tension and shear were much higher than the applied forces whereas the calculated deflections (based on an arbitrary increase of 50% over the bending deflection to allow for shear deflection and nail slip) were close to the allowable deflection.

The comparatively high deflection of the beam might be due to the fact that the pieces of plywood were butt jointed and located at the same position on both sides of the timber framework. It was thought that if the plywood joints were scarf-jointed or the joints were staggered at a minimum distance of 600*mm* on opposite sides of the beam, the stiffness of the beam would improve and the deflection would be lower.

Beams 1, 4 and 5 were loaded at different design loads. Beam 1 was loaded slightly lower than the intended design load. While the deflection criterion for this beam was slightly better than beams 2 and 3, the load factor was even lower. This was perhaps due to variability of timber strength and the defect of knot in the flange as stated in the mode of failure in Table 2.

Beams 4 and 5 were loaded much higher than the intended design load and as such they did not meet the minimum requirement for acceptance. However, the magnitude of the absolute ultimate loads of these two beams, as given in column 12 of Table 2, were comparable to beams 6 and 7.

The strength properties of the timber and plywood given in Tables 3 and 4 are only to show approximately the strength of the materials used to make the beams and not for design purposes. The stresses for design are given in the Appendix. The timber of merpauh is in strength group B and machang and sepetir in strength group C (lower strength than group B) as given in the Code of Practice (Anonymous 1978).

Typical load-deflection curves given in Figure 3 illustrate the behavior of the beam when subjected to the loading procedures described previously. The plotting of the graph starts after the completion of the preloading process which allowed the connections of the beam to take up the slack. Figure 4 shows the deflection-time curve during the 24-*h* period while under the design load. The curve shows there was a decrease in the rate of increase of deflection.

During the experiment, it was difficult to observe whether the plywood would fail first and thereby cause the failure of the timber flange or the failure first occurred in the flanges causing the plywood to shear off. However, it was believed that on the whole the flanges would fail first at a place where there were some timber defects near the centre of the beam or near the plywood joint and this immediately sheared off the plywood web. Figure 5 shows a general arrangement of the test and Figure 6 the failure of the beam after test.

Conclusion

From the results of the test, it can be seen that the two different lengths of nailed ply-web beams as described in this paper can be safely used as roof beams without plaster ceiling or similar finishings to take the specified design load as given in Table 1. The timbers to be used should be of Strength group B, dry and of Standard Structural grade. The plywood should be of WBP type and obtained from a reputable factory.

However, it is possible that the performance of the beam as regards deflection could be improved by staggering the plywood joints with a minimum of 600 *mm* stagger on opposite sides of the beam.

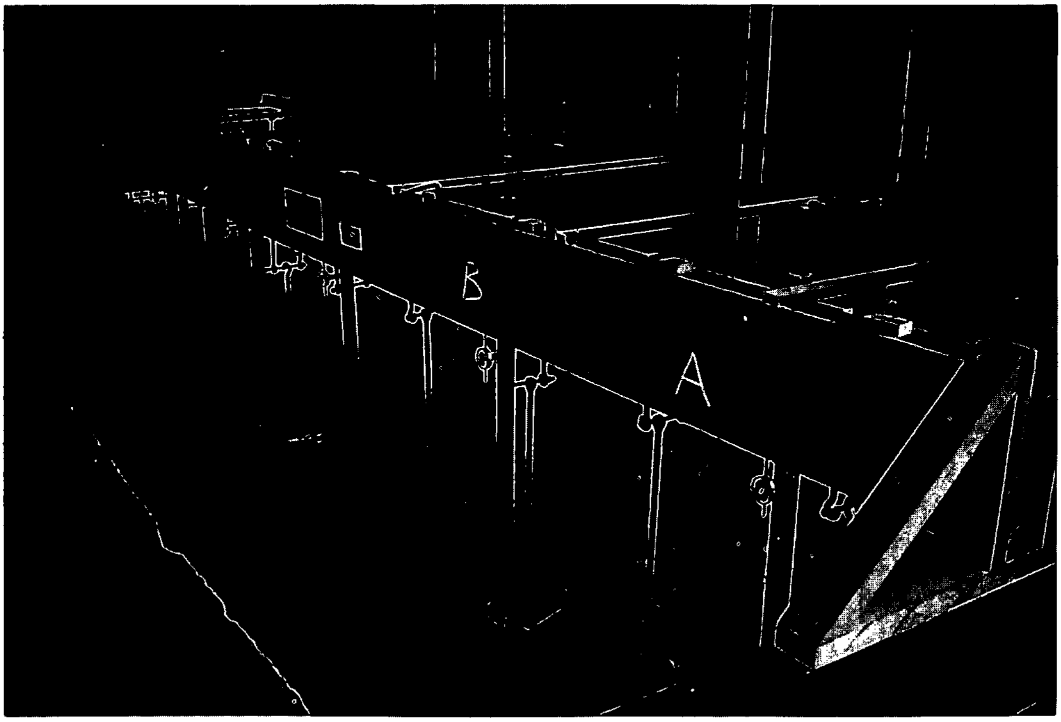

Figure 5. General view of test

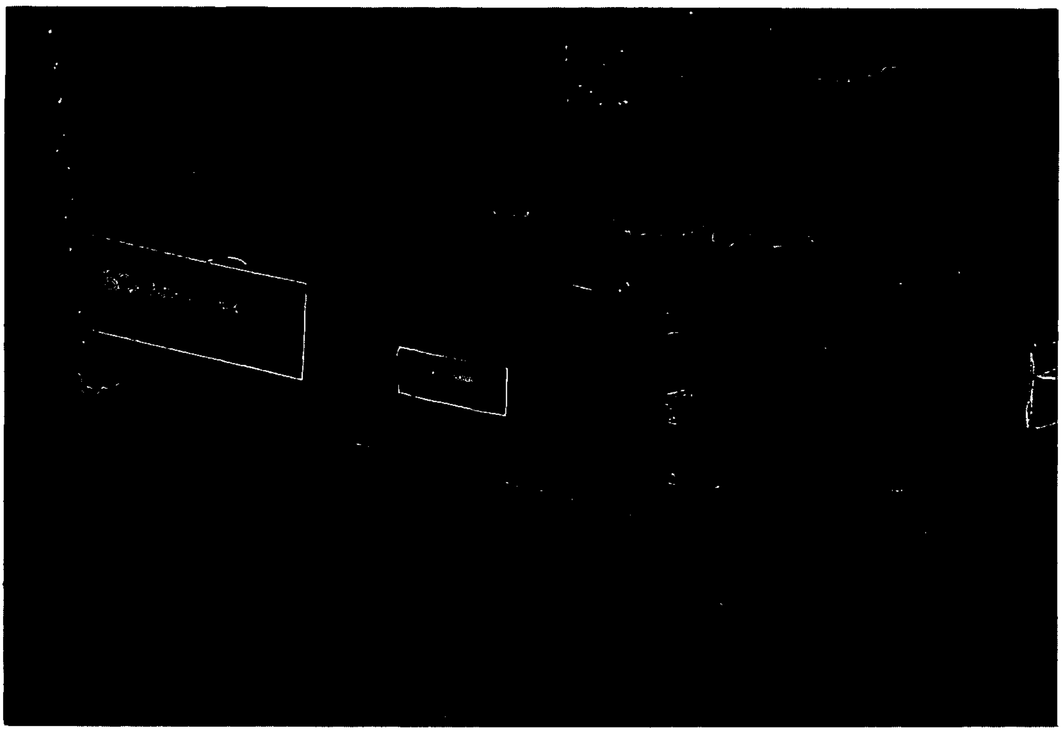

Figure 6. Failure of beam after test

For other design loads and spans, box beams can be designed by methods similar to those given in the example or by prototype testing.

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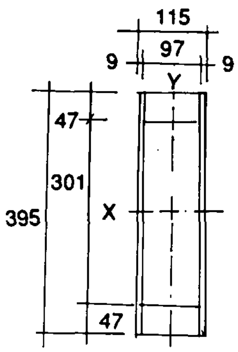
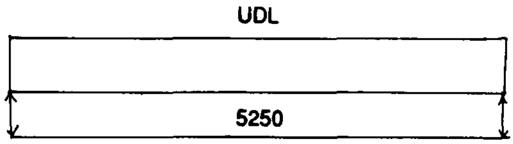
Appendix

(I) Design calculation for 5.4 *m* box beam





Given

Effective design span	=	5.25 *m*
Spacing of beams	=	2.5 *m*

Loading

Dead load (DL)	=	0.5 kNm^{-2} including self weight of beam
Live load (LL)	=	0.5 kNm^{-2}
Total	=	1.0 kNm^{-2}

Timber flanges

Strength group B, dry, standard grade

The following permissible stresses are the grade stresses (except tension stresses) taken from Engku (1980) and increased for medium term loading where applicable.

Bending (f) = 12.4 × 1.25 = 15.5 Nmm^{-2}

Tension (t) = 7.4 × 1.25 = 9.2 Nmm^{-2}
(based on 0.6 × bending and not from Engku (1980)

Compression parallel ($c_{//}$) = 10.0 × 1.25 = 12.5 Nmm^{-2}

Compression perpendicular ($c_{\perp}$) = 1.24 × 1.25 = 1.55 Nmm^{-2}
(based on basic stress with no wane)

Minimum E (E_{min}) = 6.600 Nmm^{-2}

Plywood web

The following stresses are estimates only since there were no available stresses for local plywood.

Panel shear stress = 1.72 × 1.25 = 2.15 Nmm^{-2}
(from Table 48 of CP 112 (Anonymous 1971)

E = 6,600 Nmm^{-2} (assume same value as timber)

Calculations

UDL on beam = 1.0 × 2.5 = 2.5 kNm^{-1}

$$\text{Bending moment, } M = \frac{wl^2}{8} = \frac{2500 \times 5.25 \times 5250}{8}$$

$$= 8.61 \times 10^6 \ N\,mm$$

$$\text{Shear } V = \frac{2.5 \times 5.25}{2} = 6.56 \ kN$$

Section properties

$$I_{xx} = \frac{115 \times 395^3}{12} - \frac{97 \times 301^3}{12}$$

$$= 590.6 \times 10^6 - 220.4 \times 10^6 = 370.2 \times 10^6 \ mm^4$$

$$\text{Max. compressive stress for top flange} = \frac{M}{Z} = \frac{My}{I}$$

$$= \frac{8.61 \times 10^6 \times 197.5}{370.2 \times 10^6} = 4.6 \ Nmm^{-2} < 12.5 \text{ (OK)}$$

Maximum tensile stress for bottom flange = 4.6 Nmm^{-2} from symmetry < 9.2. Both the above stresses are also less than the permissible bending stress of 15.5 Nmm^{-2}. Hence beam is satisfactory.

Deflection

The above beam is also checked for deflection. The deflection of a box beam consists of bending deflection as well as shear deflection. Since the attachment of plywood to timber was by nails alone and not by gluing, a certain amount of joint slip would occur. Bending deflection is calculated in a similar way as solid beams. For shear deflection and joint slip, allow an extra of 50% of the bending deflection as was done in the British TRADA (Johnson 1966).

$$\text{Hence total deflection} = \frac{5\ wl^4}{384\ EI} \times 1.5$$

$$= \frac{5 \times 2500 \times 5.25 \times 5250^3 \times 1.5}{384 \times 6600 \times 370.2 \times 10^6} = 15.2\ mm$$

This is within the limit of $0.004 \times 5250 = 21.0\ mm$

Web thickness

The panel shear stress at the X-X axis is

$$q = \frac{V \times Q}{I \times t}$$

where V = shear force = 6560 *N*

Q = first moment of area of the flange and web above the X-X axis $= 97 \times 47 \times 174 + 18 \times 197.5 \times \frac{197.5}{2}$

$= 1.14 \times 10^6\ mm^3$

I = 370.2×10^6 as before

t = $2 \times 9 = 18\ mm$

$$\therefore\ q = \frac{6560 \times 1.14 \times 10^6}{370.2 \times 10^6 \times 18} = 1.12\ Nmm^2 < 2.15,\ \therefore\ \text{OK}$$

Nail spacing

The spacing of nails along the flange on each side of the beam is given by:-

$$s = \frac{2\ P \times h}{V}$$

where s = spacing

P = allowable load on each nail = 170 *N* (extrapolation of Table 10 in Chu (1978) based on 2 *mm* nails and J3 timber joint group) × 1.125 (medium term)

= 191 *N*

h = distance between centres of flanges = 348 *mm*

V = shear force = 6560 *N*

$$\therefore \quad s = \frac{2 \times 191 \times 348}{6560} = 20.3 \ mm$$

For 2 rows, spacing = 41 *mm*

For the test beam, spacing provided was 50 *mm* for a length of 1.75 *m* at each end and 100 *mm* spacing for the central 1.75 *m* portion.

Stiffeners

For stiffeners at supports or end stiffeners.

Thickness parallel to length of beam,

$$t = \frac{v}{bc} = \frac{6560}{97 \times 1.55} = 44 \ mm$$

Thickness provided = 2 ×35 = 70 *mm*

For intermediate stiffeners, the thickness is recommended to be at least $\frac{1}{6}$ the width of the flange, *i.e.* $\frac{1}{6} \times 97 = 16$ *mm* and spacing to be equal to twice the clear distance between flanges (Pearson *et al.* 1968), *i.e.* 2 × 301 = 602 *mm.*

Intermediate stiffeners provided = 35 × 97 at 600 *mm* centres

Lateral stability

$$I_{xx} = 370.2 \times 10^6 \ mm^4 \text{ as before}$$

$$I_{yy} = \frac{395 \times 115^3}{12} - \frac{301 \times 97^3}{12}$$

$$= 27.2 \times 10^6 \ mm^4$$

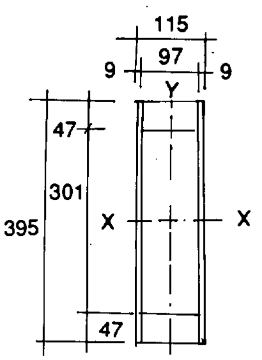
$$\text{Ratio of } \frac{I_{xx}}{I_{yy}} = 13.6$$

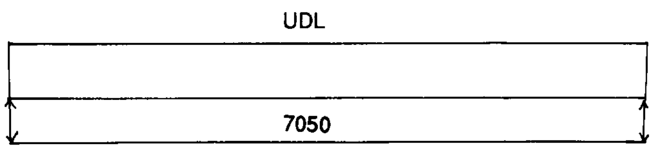
According to clause 4.9 of CP 112 (Anonymous 1971), the beam should be held in line at the ends to prevent buckling of the compression flange and overturning of the beam.

The above beam is also suitable for other combinations of loadings and beam spacings as given below (based on design loading of 2.5 kNm^{-1} on beam):-

Total design load (kNm^{-2})	*Spacing of beam* (m)
0.6	4.17
1.0	2.50 (as in example)
1.5	1.67
2.0	1.25

(II) Design calculation for 7.2 *m* box beam





Given

Effective design span = 7.05 m
Spacing of beams = 1.2 m

Data on loading, timber flange stresses and plywood stresses are as given for 5.4 m box beam.

Calculations

UDL on beam, w = 1.0 × 1.2 = 1.2 kNm^{-1}

$$\text{Bending moment, } M = \frac{wl^2}{8} = \frac{1200 \times 7.05 \times 7050}{8}$$

$$= 7.46 \times 10^6 \; N\,mm$$

$$\text{Shear } V = \frac{1.2 \times 7.05}{2} = 4.23 \; kN$$

$$\text{Max. compressive stress for top flange} = \frac{M_y}{I}$$

$$= \frac{7.46 \times 10^6 \times 197.5}{370.2} = 4.0\ Nmm^{-2} < 12.5\ \text{(OK)}$$

$$\text{Max. tensile stress for bottom flange} = 4.0\ Nmm^{-2}\ \text{from symmetry} < 9.2\ \text{(OK)}$$

Both stresses are also less than the permissible bending stress of 15.5 Nmm^{-2}. Hence satisfactory.

Deflection

$$\text{Total deflection} = \frac{5\ wl^4}{384\ EI} \times 1.5$$

$$= \frac{5 \times 1200 \times 7.05 \times 7050^3 \times 1.5}{384 \times 6600 \times 370.2 \times 10^6} = 23.7\ mm$$

This is within the limit of $0.004 \times 7050 = 28.2$ *mm*

Web thickness

Panel shear stress at the X-X axis

$$q = \frac{V \times Q}{I \times t} = \frac{4230 \times 1.14 \times 10^6}{370.2 \times 10^6 \times 18} = 0.72\ Nmm^{-2} < 2.15\ \text{(OK)}$$

Nail spacing

$$s = \frac{2\ P \times h}{V}$$

$$= \frac{2 \times 191 \times 348}{4230} = 31.4\ mm$$

For 2 rows, spacing = 63 *mm*

Spacing provided at both ends of beam of 2.4 *m* long was 50 *mm* while at the centre of 2.4 *m*, spacing was 100 *mm*.

Stiffeners

Thickness of end stiffeners

$$t = \frac{V}{b \times c} = \frac{4230}{97 \times 1.55} = 28\ mm$$

Thickness provided $\doteq$ 2 × 35 = 70 *mm*

The size of intermediate stiffeners were 35 × 97 at 600 *mm* centres. Splice plates for the flange joints at top and bottom flanges were provided as shown in Figure 2.

Lateral stability

As given in 5.4 *m* box beam.