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Strength and Stiffness of Glued Laminated Timber Beams

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Strength and Stiffness of Glued Laminated Timber Beams

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Artikkelen beskriver en eksperimentell undersøkelse av limte laminerte trebjelkers styrke og stivhet. I alt ble 47 bjelker belastet til brudd. Bjelkene ble fremstillet av norsk granvirke som ble sortert etter reglene i NS 447. Til alle bjelker ble det benyttet virke av to kvaliteter, med laveste kvalitet i de indre 60 % av tverrsnittet. Det ble benyttet tre forskjellige lamelltykkelser, nemlig ¾″, 1″ og 2″.

Det ble påvist en signifikant forskjell i styrke og stivhet mellom laveste og høyeste virkeskvalitet. Variasjonene i styrke og stivhet som funksjon av lamelltykkelsen var ikke signifikant, men 2″ lameller ga i gjennomsnitt de svakeste bjelkene.

Bjelkenes styrkeegenskaper var sterkt avhengig av kvaliteten av virket i de ytre lameller på strekksiden. Svake angrep av råte eller spor av tennar syntes å kunne medføre sterkt redusert styrke. Kvister som lå så nær kanten av lamellene at fiberforstyrrelsene rundt dem var beskadiget av sagsnittet, var ofte en direkte årsak til brudd. De strengere krav som NS 447 stiller til kvister nær kanten, synes således fornuftige også for virke til laminerte konstruksjoner.

I artikkelen sammenlignes de oppnådde resultater med tilsvarende verdier funnet av Thunell for massive trebjelker av svensk furuvirke, samt med de tillatte verdier som er angitt i NS 446. De laminerte bjelkene var i gjennomsnitt ca. 25 % sterkere og stivere enn de massive furubjelkene. De tillatte bøyningsspenningene som er foreskrevet i NS 446, synes rimelige, men kan trolig forhøyes noe for virke av laveste kvalitet (T 210).

Forsøkene synes å indikere at elastisitetsmodulen for laminerte bjelker av alle virkeskvaliteter kan settes ca. 20 % høyere enn foreskrevet i NS 446.

1. *Introduction.*

Glued laminated timber is receiving increasing attention from architects and structural engineers as an attractive material for many types of structures.

In Norway, laminated timber structures are designed in accordance with the provisions of the Norwegian Standard NS 446 (1) issued in 1957, and the materials used for lamination are graded according to the structural grading rules set forth in NS 447 (2).

NS 446 and NS 447 are to a large extent based upon test results from other countries. It was therefore deemed necessary to investigate their adequacy to Norwegian species.

This report presents the results of an experimental investigation of the strength and the stiffness of glued laminated beams. Major variables were grades of timber and thicknesses of laminations.

2. *Acknowledgement.*

This investigation was carried out as a cooperative research project of the Norwegian Building Research Institute and the Norwegian Institute of Wood Working and Wood Technology. The investigation was sponsored by the Royal Norwegian Council for Scientific and Industrial Research.

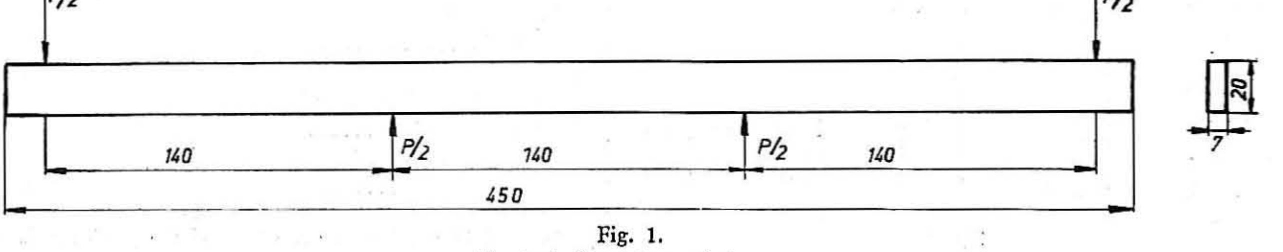

Fig. 1.
Nominal dimensions of beams.

3. *Outline of tests.*

A total number of 47 laminated beams were tested to destruction. The beams all had a nominal size of 7 × 20 × 450 cm, and were loaded through concentrated loads at the third-points with a total span length of 420 cm, see Fig. 1.

Fig. 2 shows the cross-sections of the beams. Three different series of tests were executed, according to the following scheme:

Series 0;	13	laminations of	16 mm thickness	in each beam
Series 1;	9	—»—	22 »	—»—
Series 2;	4½	—»—	45 »	—»—

Each beam consisted of two different grades of timber with the best grade in the outer 20 per cent of the laminations on each side. The Norwegian design specifications state that the allowable bending stresses prescribed for the structural grade which is used in the outer 20 per cent on each side of the cross section can be used even if the inner 60 per cent of the cross section consists of materials of the next lower structural grade.

The structural grading rules of NS 447 are briefly reviewed in Section 5. 1. According to these rules, the timber is classified into the following groups:

T 390, T 300, T 210 or «below grade», where T 390 represents the highest quality.

Table 1 shows the number of beams which were tested in each of the four major groups of grade combinations. The following symbols are introduced:

A for T 390
B for T 300
C for T 210
D for «below grade».

Table 1 contains 42 beams. The remaining five beams had combinations of grades which fell outside the outlined pattern.

The beams are designated according to the grade combinations in such a way that the first letter indicates the grade of the outer laminations while the second letter denotes the grade of the inner laminations. The first number indicates the series to which the beam belongs. Hence, the number 1 AB 4 identifies beam number 4 of series 1 which had a combination of T 390 and T 300 materials.

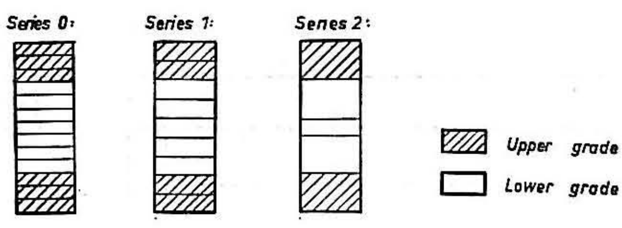

Fig. 2.
Cross-sections of the beams.

T a b l e 1.
Number of beams in different test groups.

Grades	AB	AD	BC	CD
Series 0	6	—	—	5
» 1	6	5	5	5
» 2	—	—	5	5

4. *Materials.*

Norway spruce from the district of Kirkenær in Solør was used in all beams. Kirkenær belongs to one of the best forest districts of Norway and is situated approximately 150 meters above sea level at a latitude of 61.5°.

The materials were selected at random from a sawmill in the district, and were kilndried from green condition down to a moisture content of 12 % as soon as possible after the sawing.

The materials were subsequently conditioned for several months before the production of test specimens.

It appeared that the selected materials contained a rather high percentage of compression wood.

5. *Fabrication.*

5. 1. Structural grading . . . The materials were graded after surfacing. The most important requirements of the Norwegian grading rules (NS 447) are summarized in Table 2. Additional requirements are given in NS 447 for the maximum admissible sum of knots, which is measured over a length of the piece equal to its width but not exceeding 15 cm

T a b l e 2.
Grading Rules (NS 447)

Quality Class		T 390	T 300	T 210
Width of growth rings (mm) max		3	5	—
Slope of grain . . . . max		1/14	1/10	1/7
Maximum knot size*)	Narrow face of plank max	1/4	1/3	1/2
	Wide face of plank or board middle portion . . . . . . max	1/6	1/4	1/3
	Wide face of plank or board outer portions . . . . max	1/8	1/6	1/4

*) Measured in relation to the width of the pertinent face of the piece under consideration.

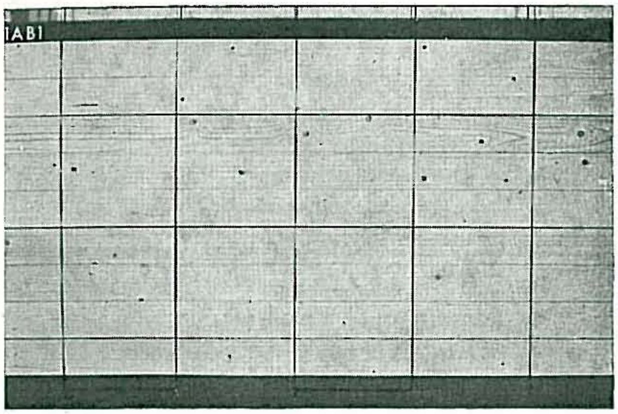

Fig. 3.
Registration of knots in laminations.

NS 447 specifies that compression wood shall not be allowed to a degree that is «worth mentioning». This is a rather vague statement that caused some trouble during the grading.

5. 2. Registration ... All the laminations of each beam were placed in a frame in the same order as in the beam, and were photographed, see Fig. 3. A number of strings identified the positions of the knots in the longitudinal direction of the beam. Color pictures were taken of both sides of the wide faces of the laminations.

5. 3. Lamination ... The moisture content of each lamination was measured by means of a Marconi electrical moisture meter. The average moisture content was 12 %, with variations not exceeding ± 2 %.

A casein glue was used. Glue was spread by means of a mechanical glue spreader at the rate of 300—350 grams per square meter, which was applied through double spreading. The gluing operation was carried out within maximum 30 hours after the surfacing of the laminations.

The beams were cured for minimum two hours at a temperature of 20 ° C and a pressure of 8—10 kg/cm^2. An additional period of curing of at least seven days was allowed before final maschining and testing.

6. Testing.

The beams were loaded at the third-points as shown in Fig. 1. The loading arrangement, including hydraulic jacks, load cells, rollers and rocker bearings, is shown in Fig. 4. The bearing blocks were made from teak and were slightly wider than the beams.

The load was applied continuously at a rate of approximately 200 kg per minute, which in the elastic region corresponds to a strain rate of approximately 225 micro-strains per minute in the extreme fibres of the central portion of the beam. This relatively slow rate of loading, which is only one third of the speed prescribed by the ASTM Designation D 198—27, was chosen in order to facilitate the strain readings.

The corresponding stress rate was approximately 30 kg/cm^2 per minute, which agrees closely with the stress rate of 28 kg/cm^2 used by Thunell (3).

The deflections in the region of constant bending moment were measured by means of a dial gage of 0.01 mm accuracy, as shown in Fig. 4. The distance between the supports of the gage was 130 cm.

The center deflection was also measured in relation to the supports of the beam. This was accomplished by means of a scale fixed to the beam in its center and a string spanning between the supports.

Most of the beams of Series 1 were each provided with four electrical strain gages. Gages of the types Philips PR 9812 and Huggenberger Tepic Type K were used. The effective gage lengths were 8 and 22 mm, respectively, for the two types.

The strain gages were as a rule placed across the depth of the center cross section of the beam. The strains were recorded by means of a self-balancing unit of the type Brüel & Kjær. Each set of strain readings consisted of two separate readings of each gage. The strains were read in a cyclic manner and the load was recorded at the beginning as well as at the end of the cycle. Loads as well as strains were then averaged.

After the beam had been tested to failure, a portion was cut out approximately one meter from the end of the beam. From this portion clear wood properties were determined. Clear wood compressive strength was determined for the laminations in the extreme 2″ on the compression side. Correspondingly, tensile strength properties were determined for the laminations on the tension side.

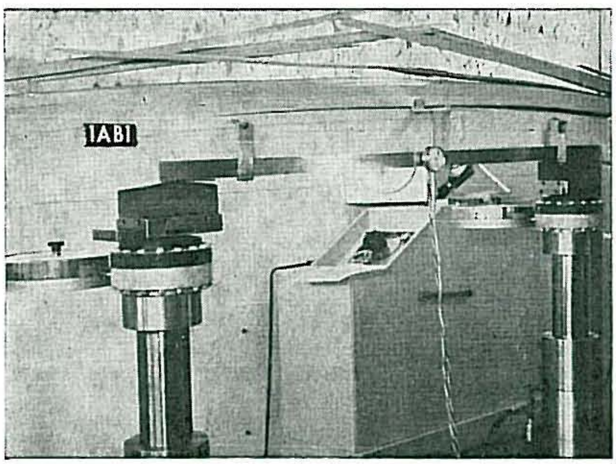

Fig. 4.
Loading arrangement.

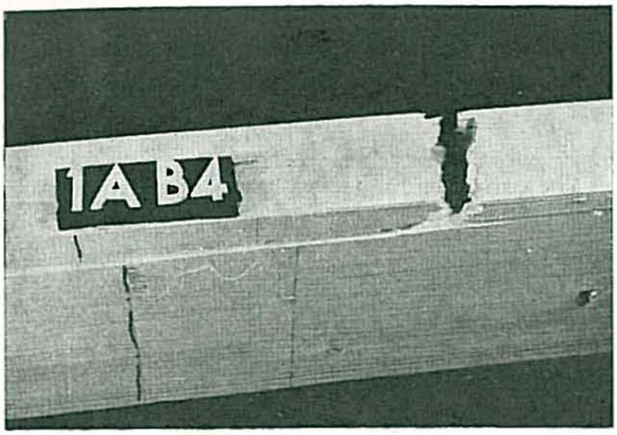

Fig. 5.
A brittle failure.

Moisture content and specific gravity was determined for all of the laminations, and shear block strength was determined for all of the glue lines.

7. *Test Results.*

A description of the observed general behaviour of the beams and the mechanism of failure is presented in a separate report (4). All of the beams except two failed by a destruction of the tension side of the beams. The two beams failed by lateral instability. In all but four beams did the failure of the tension side occur after the development of wrinkles in the compression side.

Two different types of failure in tension were observed. In most of the beams the wood was very much splintered in the zone of collapse, see Fig. 4. In other beams the zone of failure was very short, as shown in Fig. 5, and the character of the failure was rather brittle. The brittle failures were generally associated with relatively low strengths.

The most important test results are presented in Tables 3, 4 and 5. The load-deflection relationships were linear up to a load (P_E) which with few exceptions ranged between 60 and 80 per cent of the ultimate load. The first wrinkles in the compression zone usually appeared at a load (P_w) somewhere between 75 and 100 per cent of the ultimate load.

The modulus of elasticity was determined from the observed deflection on that portion of the beam which had constant bending moment. Fig. 6 shows some typical relationships between observed loads and deflections. The recorded deformations included a small amount of permanent set. Beam no. 2E2 was stepwise unloaded as indicated with dotted lines in Fig. 7 in order to determine the amount of permanent set. At the second loading to a previous maximum no permanent set was recorded except at very high loads. The slopes of the dotted curves of Fig. 7 correspond to a modulus of elasticity of approximately 150 000 kg/cm², which is nine per cent higher than the modulus computed in

Table 3.
*Test results — Series 0.**)*

Beam No.	Cross-section b	h	J	P_E	P_w	P_{ult}	P_E/P_{ult}	P_w/P_{ult}	σ_w	σ_{ult}	$E \cdot 10^{-3}$
	cm	cm	cm⁴	kg	kg	kg			kg/cm²	kg/cm²	kg/cm²
0AB 1	6,93	21.00	5350	3720	3930	4570	0.81	0.86	540	628	147.2
2	7.03	21.00	5420	3150	3900	4770*)	0.66	0.82	528	648*)	155.2
3	7.00	21.07	5460	2750	4000	4500*)	0.61	0.89	541	608*)	146.5
4	6.92	21.07	5490	2840	4050	4130	0.69	0.98	544	554	146.2
5	7.06	21.40	5760	2900	4150	4325	0,67	0.96	539	562	130.5
6	7.02	21.41	5740	3760	3900	4350	0.86	0.90	509	568	132.3
0CD 1	6.99	21.05	5430	3170	3430	3920	0.81	0.88	465	532	140.8
2	7.00	21.05	5440	2540	3000	3820	0.66	0.79	406	517	117.8
3	6.98	21.03	5410	2840	3040	3900	0.73	0.78	414	531	155.0
4	6.97	21.02	5390	2350	3150	3610	0.65	0.87	430	492	135.7
5	7.06	21.40	5760	2000	—	3020	0.66	> 1.00	—	392	120,0

*) Failure by lateral instability.
**) Notations:
b — width of beam.
h — height of beam.
J — moment of inertia.
P_E — load at proportional limit.
P_w — load at formation of wrinkles in the compression zone.
P_{ult} — ultimate load.
σ_w — maximum bending stress at the first formation of wrinkles.
σ_{ult} — modulus of rupture.
E — modulus of elasticity.

Table 4.

Test Results — Series 1.

Beam No.	Cross-section			P_E	P_w	P_{ult}	P_E/P_{ult}	P_w/P_{ult}	σ_w	σ_{ult}	$E \cdot 10^{-3}$
	b	h	J								
	cm	cm	cm^4	kg	kg	kg			kg/cm^2	kg/cm^2	kg/cm^2
1AB 1	7.23	19.97	4800	3840	4600	4975	0.77	0.92	670	725	155.5
2	7.12	20.07	4800	2600	3400	3465	0.75	0.98	498	508	161,3
3	7.13	20.03	4775	2680	3275	4260	0.63	0.77	481	625	146.8
4	7.13	18.35	3670	2400	—	2700	0.89	> 1.00	—	472	136.0
5	7.19	19.83	4670	—	—	1630	1.00	> 1.00	—	242	149.0
6	7.00	20.00	4670	3020	*)	4160	0.73	*)	*)	624	137.8
1AD 1	7.04	19.88	4610	2910	3000	3920	0.74	0.77	453	592	134.6
2	7.05	19.83	4580	3380	—	3950	0.86	—	—	598	135.6
3	7.03	20.07	4740	3920	4200	4660	0.84	0.90	623	691	155.5
4	7.02	20.04	4710	3520	3600	4020	0.88	0.90	536	600	144.0
5	7.05	20.03	4720	3430	*)	4250	0.81	0.86	535	631	131.2
1BC 1	7.05	20.00	4700	2720	3600	4250	0.64	0.85	536	633	141.5
2	7.00	19.75	4490	3040	3700	4200	0.72	0.88	569	646	141.5
3	7.12	18.92	4020	2520	2700	3340	0.75	0.81	445	550	132.8
4	7.10	19.57	4390	2080	—	2590	0.80	> 1.00	—	404	119.2
5	7.05	20.06	4740	3100	3400	4020	0.77	0.85	503	595	153.5
1BD 1	7.11	19.90	4670	3040	3000	3750	0.81	0.84	447	559	144.8
2	7.06	20.09	4770	2900	3500	3960	0.73	0.88	516	584	156.8
1CD 1	7.14	19.97	4740	2720	*)	4055	0.67	*)	*)	598	131.0
2	7.13	19.90	4680	2780	*)	3370	0.82	*)	*)	501	118.2
3	7.16	19.94	4730	2200	*)	3265	0.67	*)	*)	483	115.3
4	7.14	20.01	4770	3160	3700	4360	0.73	0.85	544	641	140.0
5	7.02	20.05	4710	3200	3100	3520	0.63	0.88	461	524	130.5
1E**)	7.00	20.15	4770	2780	*)	4275	0.65	*)	*)	632	137.6

*) Not measured.

**) E stands for «extra».

Table 5.

Test Results — Series 2.

Beam No.	Cross-section			P_E	P_w	P_{ult}	P_E/P_{ult}	P_w/P_{ult}	σ_w	σ_{ult}	$E \cdot 10^{-3}$
	b	h	J								
	cm	cm	cm^4	kg	kg	kg			kg/cm^2	kg/cm^2	kg/cm^2
2BC 1	7.04	19.85	4590	2290	2650	3640	0.63	0.73	401	551	127.8
2	7.03	19.88	4600	1880	2350	3220	0.58	0.73	355	487	116,0
3	7.06	19.85	4600	2600	2850	3720	0.70	0.77	430	562	127.0
4	7.04	19.80	4550	2600	3300	3530	0.74	0.94	502	537	148,3
5	6.99	19.81	4530	2400	3300	3350	0.72	0.99	505	513	138.0
2CD 1	7.05	19.82	4570	2100	2530	3110	0.68	0.81	384	472	120,3
2	7.03	19.84	4610	2340	2500	3420	0.68	0.73	377	515	123.2
3	6.93	19.88	4540	2300	2900	3670	0.63	0.79	445	563	141.2
4	7.02	19.83	4560	2050	2750	3510	0.58	0.78	418	534	129.2
5	7.03	19.82	4560	2300	2500	3650	0.63	0.69	380	555	136.5
2E 1	7.05	19.84	4590	2100	2850	3520	0.60	0.81	431	533	123.5
2	7.03	19.84	4580	2700	2850	3910	0.69	0.73	432	593	137.2

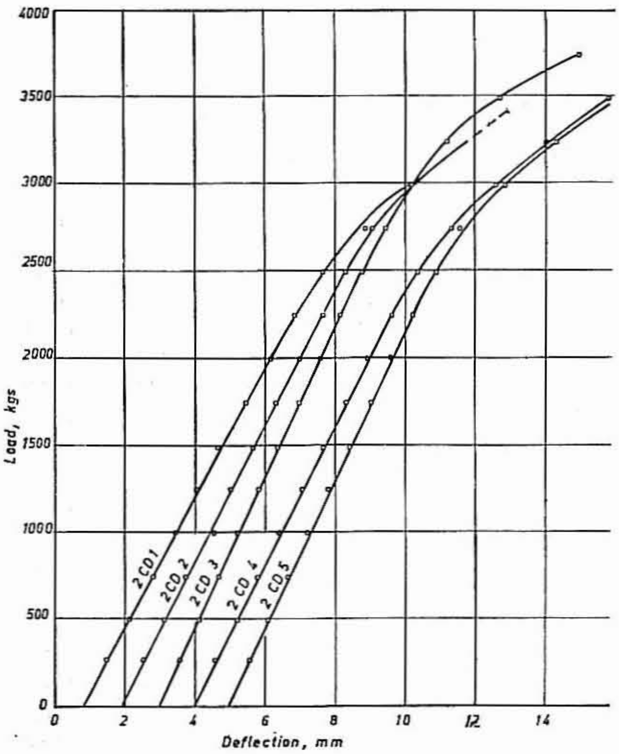

Fig. 6.
Load-deflection curves for beams 2 CD.

the regular manner from the full line deflection curve of Fig. 7.

A summary of the observed values of the bending strength (modulus of rupture) and the modulus of elasticity is presented in Table 6, which also contains computed average values and standard deviations for the individual groups of specimens as well as the total series. Frequency distributions of the bending strength and the modulus of elasticity are presented in Figs. 8 and 9, respectively.

An analysis of variance indicated that the differences between the structural grades AB and CD are significant on the 95 per cent level of probability, while no significant difference was found between the different thicknesses of laminations.

Two beams were excluded from the statistical study, viz. beams no. 1 AB 4 and 1 AB 5. These beams need further comment. They both failed suddenly in tension, as shown in Figs. 5 and 10. In beam no. 1 AB 4, the early collapse was clearly caused by an attack of fungi. The area around the zone of failure in the outer lamination on the tension side was weakly colored in dark blue. It is doubtful whether such a discolor would be detected during structural grading.

Beam no. 1 AB 5 apparently had more compression wood in the outer tension lamination than the other beams. The amount of compression wood was, however, not so large that it is likely that such a lamination would be excluded from the highest structural grade on the basis of the rules of NS 447.

The tensile strength of the wood on each side of the zone of failure of this lamination was determined by means of small clear wood specimens. These tests did not reveal any abnormality. The low strength of the beam may possibly be ascribed to minute cracks in the fibres caused by rough handling during logging or transportation of the timber or by wind forces.

Unfortunately, the test was discontinued when the specimen broke as indicated in Fig. 10. It seems reasonable to assume that the major part of the beam was still undamaged at that time. If it is assumed that the effective cross-section of the beam was reduced by two laminations and that the strength of the other laminations corresponded to the average value for timber of grade B, it is found that the ultimate load, through a continued loading, probably could be raised from 1630 kg to 2350 kg. This would give a modulus of rupture, computed on the total cross section, of approximately 350 kg/cm^2 instead of the value of 242 kg/cm^2 which is listed in Table 6.

A statistical study of the bending strength of all the beams of grade combinations AB and AD

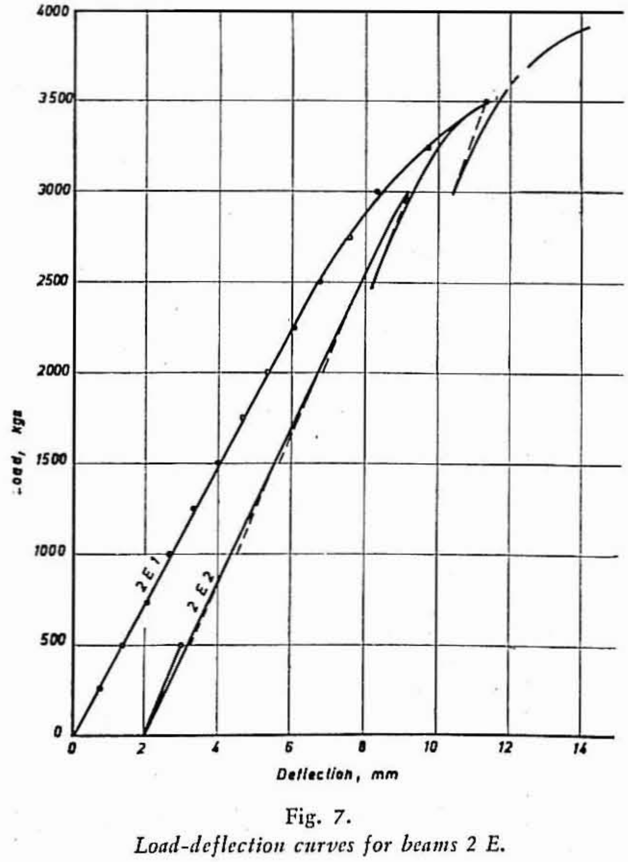

Fig. 7.
Load-deflection curves for beams 2 E.

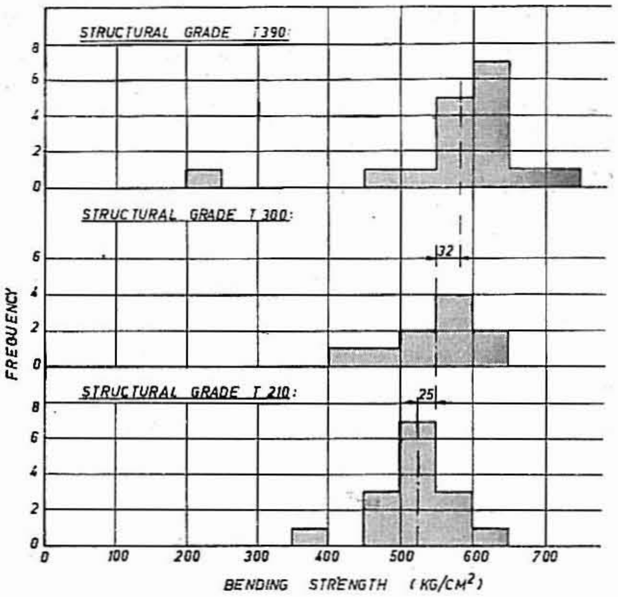

Fig. 8.
Frequency distribution of bending strength.

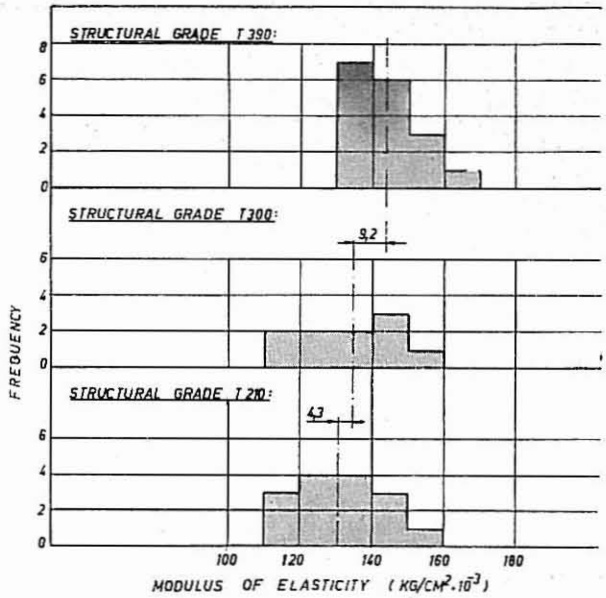

Fig. 9.
Frequency distribution of modulus of elasticity.

Table 6.
Summary of Test Results.

Series No.	Test No.	Grade: Bending strength (kg/cm^2)					Modulus of elasticity ($kg/cm^2 \cdot 10^{-3}$)				
		AB	AD	BC	CD	ALL	AB	AD	BC	CD	ALL
0	1	628			532		147.2			140.8	
	2	648**)			517		155.2			117.8	
	3	608**)			531		146.5			155.0	
	4	554			492		146.2			135.7	
	5	562			392		130.5			120.0	
	6	568					132.3				
	Average	593			493	548	143.0			133.9	138.8
	St. dev.	36			57	70	9.6			15.4	12.8
1	1	725	592	633	598		155.5	134.6	141.5	131.0	
	2	508	598	646	501		161.3	135.6	141.5	118.2	
	3	625	691	550	483		146.8	155.5	132.8	115.3	
	4	472*)	600	404	641		136.0*)	144.0	119.2	140.0	
	5	242*)	631	595	524		149.0*)	131.2	153.5	130.5	
	6	624					137.8				
	Average	621	622	566	549	588	150.3	140.2	137.7	126.9	139.3
	St. dev.	84	40	99	70	64	10.3	9.8	12.7	10.4	12.4
2	1			551	472				127.8	120.3	
	2			487	515				116.0	123.2	
	3			562	563				127.0	141.2	
	4			537	534				148.3	129.2	
	5			513	555				138.0	136.5	
	Average			530	528	529			131.4	130.1	130.8
	St. dev.			30	36	31			12.2	8.8	10.1
ALL	Average	604	622	548	523	562	145.9	140.2	134.6	130.3	136.5
	St. dev.	57	40	71	58	70	10.0	9.8	12.2	11.3	12.4

*) Excluded from the statistical study.
**) Failure by instability.

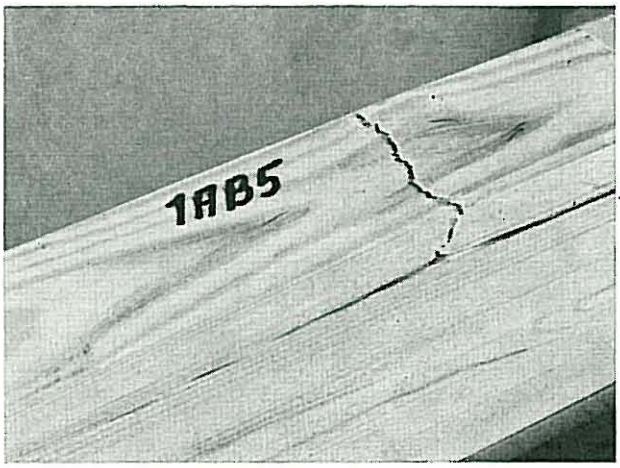

Fig. 10.
Failure in beam no. 1 AB 5.

assuming 350 kg/cm^2 for 1 AB 5, yielded an average value of 587 kg/cm^2 and a standard deviation of 86 kg/cm^2, instead of the values presented in Table 6.

It was not considered reasonable to assume that weaknesses such as those described above tend to appear more often in the highest structural grade than in the lower ones. It therefore seems justified to exclude the two above mentioned beams from the comparison between the different types of beams which is presented in Table 6.

The results of these two tests as well as others clearly demonstrated the importance of high strength in the outer laminations on the tension side. The theory of the mechanism of failure outlined by the author (4) also predicts that the strength of the extreme tension fibres greatly influences the strength of the member.

Great care should always be taken in order to select the very best materials for use on the tension side. The author believes that existing statistical analyses of the effects of knots in laminated beams, which are based on the assumption of a linear stress distribution across the beam at failure, greatly underestimates the effect of knots in the tension side.

The tests also clearly demonstrated that knots in the neighbourhood of the edges of the laminations were more critical than those in the central part, especially when the grain disturbances around the knots were damaged by the saw cuts, see Fig. 11.

A comparison between Series 1 AB and 1 AD shows no decrease in strength when timber of the medium structural grade in the interior of the beams was replaced with «below grade» timber. A larger number of tests might, of course, have revealed a minor difference between the two groups.

Table 7 shows average moisture content and specific gravity of the different types of beams. The variations are very small and the beams with the highest average moisture content also had the highest average specific gravity. The results in Tables 3, 4, 5 and 6 are, therefore, directly comparable without any corrections for moisture content or specific gravity.

No failure was observed in the glue lines during the testing of the beams. Also the tests of the glue lines by means of conventional shear block specimens indicated that strong and dependable glue lines had been obtained in all the beams.

8. *Comparison with Solid Beams.*

A relatively large investigation of the strength properties of solid beams of Norway spruce will be executed at the Norwegian Institute of Wood Working and Wood Technology in the near future. Before the results of that investigation are available, very little information is at hand for comparative purposes.

The allowable stresses given in the Norwegian design standard for timber structures NS 446 (1) are problably to some extent based on the test results obtained by Thunell (3) in an investigation of Swedish Redwood. It may, therefore, be of interest to compare the strength properties of the laminated beams with those obtained by Thunell. Such a comparison is shown in Table 8, where the data for the solid beams have been corrected for a small difference in the moisture content between the two types of beams.

The coefficients of variance are listed in the parentheses of Table 8. It should be noted here that the variances of the two series are noe directly comparable. The tests by Thunell included materi-

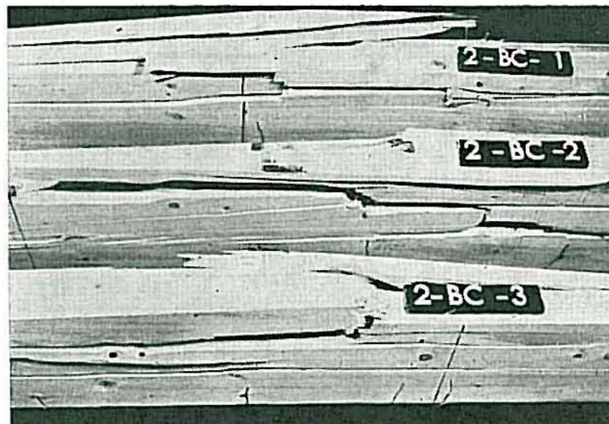

Fig. 11.
Failures through edge knots.

Table 7.

Moisture Content and Specific Gravity.

	Moisture Content (%)					Specific Gravity* (kg/dm^3)				
	AB	AD	BC	CD	ALL	AB	AD	BC	CD	ALL
Series 0	11.2			11.5	11.3	0.409			0.398	0.403
Series 1	11.9	12.0	12.0	11.9	12.0	0.417	0.407	0.419	0.407	0.412
Series 2			13.6	13.2	13.4			0.428	0.434	0.431
ALL	11.6	12.0	12.8	12.2	12.2	0.413	0.407	0.424	0.413	0.416

*) $\dfrac{\text{Weight}}{(1 + \frac{p}{100}) \times \text{Volume}}$ when p is the moisture content.

als from different districts of Sweden, while all the materials for the author's investigation came from a single district i Norway.

This comparison is also problematic since the investigation by Thunell included beams of different heights. His tests clearly indicated increasing strength with decreasing height. Since the laminated beams were higher than the majority of the solid beams, Table 8 tends to underestimate the relative strength of the laminated beams.

Table 8 indicates that the modulus of rupture as well as the modulus of elasticity of the laminated beams made from Norway spruce are on the average approximately 25 per cent higher than the corresponding values for solid timber beams of Swedish Redwood. The coefficients of variance seem to be lowest for the laminated beams.

9. *Comparison with NS 446.*

In order to compare the test results with the Norwegian design specifications (NS 446) it will be necessary to make some assumptions with respect to factors of safety, effects of sustained load etc. The allowable bending stresses for sustained load will be computed from the following formula

$$\sigma_a = \frac{1}{n_s} \cdot n_h \cdot n_l \, (\sigma_m - 2_{s\sigma})$$

where

σ_m = the observed average modulus of rupture
s_σ = the observed standard deviation
n_s = factor of safety
n_h = height factor
n_l = sustained load factor.

The values of σ_m and s_σ have been determined by means of the tests and are listed in Table 6. In a normal distribution only 2.5 per cent of the total number of tests results will fall below the value $(\sigma_m - 2\, s_\sigma)$.

Table 8.

Comparison of Laminated and Soalid Timber Beams.

	Modulus of rupture (kg/cm^2)				Modulus of elasticity (kg/cm^2)				Average moisture content %	Average specific gravity kg/dm^3
	T 390	T 300	T 210	ALL	T 390	T 300	T 210	ALL		
A) Solid*	480 (22)	450 (23)	385 (22)	438	116 000 (17)	113 000 (18)	103 000 (18)	111 000	14.6	0.410
B) Laminated	604 (9.5)	548 (13)	523 (11)	558	146 000 (7)	134 000 (9)	130 000 (8.5)	137 000	12.2	0.416
Ratio B/A	1.26	1.22	1.36	1,27	1.26	1.19	1.26	1.24		

*) From reference (3). Corrected to 12.2 per cent moisture content.
In parentheses: Coefficients of variance.

A factor of safety of 1.5 will be used. Several empirical formulas exist which account for the decrease in strength of wood beams with increasing height. In laminated beams, which may differ very much in height from case to case, this effect should preferably be considered separately in each case. However, since NS 446 does not specify an allowable strength that varies with the height of the beam it seems advisable to base the allowable stresses on a fairly large beam height.

Wood Handbook (6) gives the following empirical formula for the height factor

$$n_H = 0.81\ (H^2 + 143)\ /\ (H^2 + 88)$$

where H is the height of the beam, measured in inches.

It seems reasonable to base the allowable stresses for laminated beams on an average height of 75 cm (30″). Since the experimental investigation included beams of approximately 20 cm (8″) height, a reduction factor

$$n_h = n_{30}\ /\ n_8 = 0.86\ /\ 1.10 = 0.8$$

should be introduced to account for the reduced strength of higher beams.

The ratio of the sustained load strength to the strength as found in tests of approximately half an hour duration may according to Wood (5) be estimated to 0.60.

Hence we compute the allowable bending stress from the following formula

$$\sigma_a = \frac{1}{1.5} \cdot 0.80 \cdot 0.6\,(\sigma_m - 2\,s_\sigma) = 0.32\,(\sigma_m - 2\,s_\sigma)$$

For the modulus of elasticity it seems sufficiently conservative to compute the design value by the formula

$$E_a = E_m - 2\,s_E$$

Table 9 shows a comparsion between the computed allowable stresses and moduli of elasticity and those at present prescribed in NS 446.

The bending stresses given in NS 446 are indeed reasonable. A minor increase in the allowable stress for laminated timber of the lowest structural grade (T 210) seems justified.

The moduli of elasticity specified in NS 446 seem to be too conservative for laminated timber. An increase in E of 20 000 kg/cm² seems to be justified for all the grades.

10. *Summary and Conclusions.*

This report presents the results of an experimental investigation of forty-seven laminated timber beams. It should be emphasized that the materials for all the beams were purchased from a single sawmill in one of the best forest districts of Norway.

On the basis of the test results, the following conclusions seem to be justified:

Table 9.

Comparison of Test Results with NS 446.

A) Bending Stress (kg/cm²).

	NS 446			From tests			Suggested	
Structural grade	Solid timber σ_{NS}	Laminated timber σ^1_{NS}	Ratio $\sigma^1_{NS}/\sigma_{NS}$	Average modulus of rupture σ_m	Standard deviation s_σ	Computed allowable stress σ_a *	Allowable stress σ_s	Ratio σ_s/σ_{NS}
T390	130	156	1.20	604	57	157	156	1.20
T300	100	130	1.30	548	71	130	130	1.30
T210	70	98	1.40	523	58	130	105	1.50

B) Modulus of Elasticity (kg/cm²).

	NS 446			From tests			Suggested	
Structural grade	Solid timber E_{NS}	Laminated timber E^1_{NS}	Ratio E^1_{NS}/E_{NS}	Average modulus of elasticity E_m	Standard deviation s_E	Computed design M.E.** E_a	Design M.E. E_s	Ratio E_s/E_{NS}
T390	100 000	100 000	1.0	146 000	10 000	126 000	120 000	1.20
T300	90 000	90 000	1.0	134 000	12 000	110 000	110 000	1.22
T210	70 000	70 000	1.0	130 000	11 000	108 000	90 000	1.28

*) $\sigma_a = 0.32\,(\sigma_m - 2\,s_\sigma)$

**) $E_a = E_m - 2\,s_E$

1) A significant difference in strength and stiffness was observed between the beams of the highest and the lowest grade.

2) No significant difference was found for the different thicknesses of laminations. Beams with laminations of 2″ thickness had, however, slightly lower average strength and stiffness than beams with thinner laminations.

3) The strength of laminated beams depends very much on the quality of the outer laminations on the tension side. Even very weak traces of fungi may reduce the strength of the beam considerably. Care should also be taken in order to avoid compression wood in these laminations.

4) The allowable stresses specified in NS 446 are quite reasonable. A minor increase in the allowable bending stress for laminated timber of the weakest grade seems justified.

5) The moduli of elasticity given in the standard specifications are too conservative. An increase of 20.000 kg/cm^2 for laminated beams seems justified for all the structural grades.

6) Additional tests should be carried out in order to substaniate the validity of the two preceding conclusions for materials from other districts.

11. REFERENCES

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