8911052

I

RAPPORT

Jürgen König

Thin-walled Wood-based Flanges in Composite Beams

Paper Presented at CIB-W18A, Meeting 22, Berlin, September 1989

Trätek

INSTITUTET FÖR TRÄTEKNISK FORSKNING

Jürgen König

THIN-WALLED WOOD-BASED FLANGES IN COMPOSITE BEAMS

Paper presented at CIB-W18A, Meeting 22, Berlin, September 1989

TräteknikCentrum, Rapport I 8911052

Nyckelord

beams
buckling
composite action
flange curling
wood

Stockholm, September 1989

Rapporter från TräteknikCentrum är kompletta sammanställningar av forskningsresultat eller översikter, utvecklingar och studier. Publicerade rapporter betecknas med I eller P och numreras tillsammans med alla utgåvor från TräteknikCentrum i löpande följd.

Reports issued by the Swedish Institute for Wood Technology Research comprise complete accounts for research results, or summaries, surveys and studies. Published reports bear the designation I or P and are numbered in consecutive order together with all the other publications from the Institute.

TräteknikCentrum betjänar de fem industrigrenarna sågverk, trämanufaktur (snickeri-, trähus-, möbel- och övrig träbearbetande industri), träfiberskivor, spånskivor och plywood. Ett avtal om forskning och utveckling mellan industrin och Styrelsen för Teknisk Utveckling (STU) utgör grunden för verksamheten som utförs med egna, samverkande och externa resurser. TräteknikCentrum har forskningsenheter, förutom i Stockholm, även i Jönköping och Skellefteå.

The Swedish Institute for Wood Technology Research serves the five branches of the industry: sawmills, manufacturing (joinery, wooden houses, furniture and other woodworking plants), fibre board, particle board and plywood. A research and development agreement between the industry and the Swedish National Board for Technical Development (STU) forms the basis for the Institute's activities. The Institute utilises its own resources as well as those of its collaborators and other outside bodies. Apart from Stockholm, research units are also located in Jönköping and Skellefteå.

SAMMANFATTNING

Denna rapport syftar till ett effektivare och säkrare utnyttjande av tunna träbaserade delar i bärande konstruktioner.

Bärförmågan av tunnväggiga flänsar av träbaserat skivmaterial i sammansatta balktvärsnitt kan reduceras på grund av skjuvdeformationer (shear lag), buckling och skålning (flange curling). Tryckta flänsar har dock en betydande bärförmåga även när flänsen bucklat. Effektiva-bredd-konceptet som är väletablerat inom tunnplåtsområdet kan användas även vid dimensionering med avseende på buckling av träbaserade skivor. Detta visas genom en utvärdering av försök genomförad vid NNI i Delft, Holland. Dimensioneringsformler för skålningens inverkan härleds och dimensioneringskriterier föreslås.

CIB-W18A/22-10-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18A - TIMBER STRUCTURES

THIN-WALLED WOOD-BASED FLANGES IN COMPOSITE BEAMS

by

J König

Swedish Institute for Wood Technology Research
Sweden

MEETING TWENTY-TWO
BERLIN
SEPTEMBER 1989

CONTENT

SUMMARY

The loadbearing capacity of thin-walled wood-based flanges in composite beams can be reduced due to shear lag, buckling and flange curling. However, flanges in compression have considerable loadbearing capacity even after the flange has buckled. The effective width concept which is well established in the field of sheet metal construction can also be used in designing wood-based board material with respect to buckling. This is shown by an evaluation of tests carried out at Delft. Design formulae for the effect of flange curling are derived, and design criteria are proposed.

1. INTRODUCTION

In the field of timber construction, structural elements comprising thin-walled components of wood-based board material have been used for some time. These structural elements are characteristic for lightweight construction, and it is expected that they will be used more extensively in the future.

Typical examples are composite sections comprising webs of solid wood and flanges of board material. The webs may also be made up of components of board material and solid wood, see figure 1. However, owing to the small thickness of the board material, there are a number of properties which demand more accurate analysis if the material is to be utilised in the optimum manner. The flange often has the duty of distributing load in the transverse direction. In floors, consideration must also be given to the stiffness requirement which determines the thickness of the flange. In these structures it is generally only the shear lag in the board material which limits the loadbearing capacity of the flange when the flanges are very wide in proportion to the span of the beam. In other structures, for instance in roof cassettes, the load distribution task may be of subordinate importance and certain deformations may be tolerated. Finally,

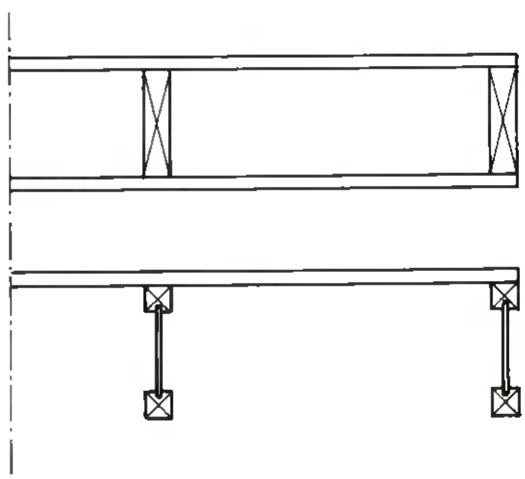

Figure 1. Examples of composite sections with thin walled components.

direct application of load on the flange can often be avoided, for instance by application of the load over the web of the composite section or on the bottom flange of the section. Flanges in compression may also be subject to buckling. When the beam is curved, or becomes curved owing to the external load, the flanges are deformed inwards towards the neutral axis of the cross section (flange curling), and the contribution of the flanges to the loadbearing capacity and stiffness of the box section is therefore reduced.

It is well known that thin-walled plates in compression have considerable postcritical loadbearing capacity, i.e. the load can be further increased after the critical buckling stress of the plate has been reached. This favourable behaviour has been utilised for a long time, first in aeroplane construction and since the end of the 1940s also in building construction, mainly in thin-walled structural elements of steel and aluminium sheeting. In the field of timber construction also the postcritical loadbearing capacity of buckled plate elements has been known. In order that this phenomenon may be utilised to some extent, the coefficient of safety is slightly reduced in the structural regulations for timber in most countries.

In accordance with the CIB Structural Timber Code /1/, it is permissible to utilise the postcritical loadbearing capacity. The code lays down some values regarding the effective width of the flange, but these values are undifferentiated since neither the critical buckling load of the flange nor its compressive strength is taken into consideration. As far as the author is aware, the effect of these is reflected only in the Swiss timber code SIA 164 (1981) /5/. It is however not known whether the rules in the code had been verified for wood-based board materials when the code was written. In /6/, which gives the background to the Swiss code, no reference is made to this.

The research reports which deal with the buckling of wood-based boards or thin-walled components in structural elements mainly confine them-selves to the determination of the critical buckling load, see e.g. /2/, /3/ and /4/. While most structural regulations for timber structures take account of the fact that the loadbearing capacity of thin-walled flanges is limited due to shear lag in the board material, there is no requirement, as far as the author is aware, that the effect of flange curling on the loadbearing capacity of the composite section should be checked.

It is shown below that the effective width concept in utilising the post-critical region can also be applied to wood-based thin-walled flanges. Design rules are also given which take the effect of flange curling into consideration. The effect of shear lag in wood-based board materials is already well documented and requires no further elucidation.

2. POSTCRITICAL LOADBEARING CAPACITY AFTER BUCKLING OF A FLANGE IN COMPRESSION

2.1 Definition of the effective width

The effective width concept is well established in the field of sheet metal construction. The effective width b_{ef} is defined so that the following relationship holds (see figure 2):

$$\int_0^b \sigma_x(y)dy = b_{ef}\sigma_e$$

The formula which is most widely used at present for determination of the effective width is that determined empirically by Winter /7/ and later somewhat modified in the AISI Code (1968) /8/:

$$\frac{b_{ef}}{b} = \sqrt{\frac{\sigma_{cr}}{\sigma_e}}\,(1 - 0.22\sqrt{\frac{\sigma_{cr}}{\sigma_e}}) \qquad (2)$$

where σ_{cr} is the critical buckling stress
σ_e is the compressive edge stress in the buckled state, see figure 1.

Expression (1) is a development of the formula according to von Karman:

$$\frac{b_{ef}}{b} = \sqrt{\frac{\sigma_{cr}}{\sigma_e}} \qquad (3)$$

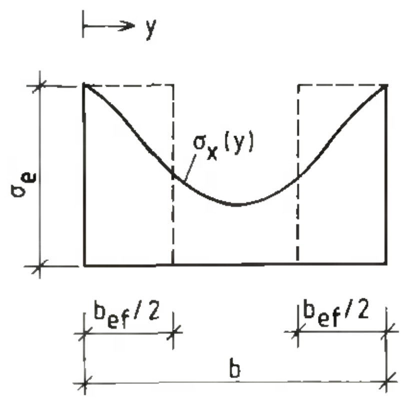

Figure 2. Definition of the effective width be. The condition $\int_0^b \sigma_x(y)dy = b_{ef}\sigma_e$ must be satisfied.

According to this formula, the loadbearing capacity of a plate in compression is not reduced until the critical buckling stress is reached, while Winters formula takes account of imperfections which reduce the loadbearing capacity at compressive stresses which are lower than the critical buckling stress.

2.2 Evaluation of tests carried out at Delft

Dekker et al (1978) /9/ carried out a test series on 100 sheets of plywood loaded in compression. The object of this test series was to investigate whether the linear buckling theory could be applied to plywood also. In the following, only the data and results which are important with regard to evaluation with respect to the postcritical loadbearing capacity are given.

The material in the tests was Canadian Oregon Pipe Plywood, Select Sheathing, Ext. 1". Two different widths, 600 and 400 mm, and two different nominal thicknesses, 8 and 13 mm, were used. The aspect ratio a/b varied between 0.5 and 4.5. One half of the test specimens were simply supported on four sides, while the unloaded edges of the other half were subjected to some restraint. Since there are no data regarding the degree of restraint, these specimens are not included in this evaluation.

The test report sets out the measured thickness t of the test specimens and their stiffnesses N_x, N_y and N_{xy} which are defined as follows:

$$N_x = \frac{E_x t^3}{12(1-v_x v_y)}$$

$$N_y = \frac{E_y t^3}{12(1-v_x v_y)}$$

$$N_{xy} = \frac{Gt^3}{6} + \frac{1}{2}(v_x N_x + v_y N_y)$$

The theoretical critical buckling stress can then be calculated as

$$\sigma_{cr} = k \frac{4\pi^2}{b^2 t} \sqrt{N_x N_y} \qquad (4)$$

with the buckling coefficient

$$k = \frac{m^2}{4\alpha_v^2} + \frac{\eta}{2} + \frac{\alpha_v^2}{4m^2} \qquad (5)$$

where m is the number of half wavelengths in the longitudinal direction in the buckled state. It is assumed that in the transverse direction there is only one half wave. We further have

$$\alpha_v = \frac{a}{b} \sqrt[4]{\frac{N_y}{N_x}} \qquad (6)$$

$$\eta = \frac{N_{xy}}{\sqrt{N_x N_y}} \qquad (7)$$

The critical buckling load N_{cr} was determined for each test specimen in accordance with Equation (4) using the experimental stiffness values. The experimental critical buckling stress was also determined by means of the relationship between the load and the perpendicular displacement of the plate. The critical load was defined by the point of intersection of the load axis and the extension of the practically straight portion of the curve in the postcritical region.

The ultimate load N_u was also recorded for each test specimen, but the compressive strengths $f_{c,0}$ and $f_{c,90}$ of the sheets of plywood were not recorded. In spite of this omission in the investigation, it is possible to study the properties of the specimens in the postcritical region as shown below.

In the evaluation, only the specimens whose modified aspect ratios α_v according to Equation (6) are larger than 1 were used. The experimental effective width of the test specimens in the ultimate state was determined as

$$\frac{b_{ef}}{b} = \frac{\sigma_u}{f_c}$$

where $\sigma_u = \frac{N}{bt}$

In calculating the slenderness ratio $\alpha = \sqrt{\sigma_e/\sigma_{cr}}$, the extreme fibre stress σ_e was put equal to the compressive strength $f_{c,0}$ or $f_{c,90}$. The compressive strength here is an effective value, i.e. plywood is considered to be homogeneous. In view of the uncertainty in determining the experimental critical buckling stress $\sigma_{cr,test}$, the values $\sigma_{cr,calc}$ calculated using the experimental stiffness values N_x, N_y and N_{xy} in accordance with Equation (4) were used instead. When $\sigma_{cr,calc}$ and $\sigma_{cr,test}$ are compared in Table 1, it is seen that the latter exhibit a consideraly larger scatter. In order to simplify calculations, the mean values of each test series, set out in Table 1, were used. The slenderness ratio α thus contains a small error which is however negligible.

Since the compressive strength of the plywood material had not been determined, both b_{ef} and α were calculated for several assumed values of f_c, see Table 2. The results are set out in figure 3. Curves in accordance with Equations (2) and (3) are also plotted in this figure.

An estimate of the compressive strength of the plywood material can be made on the basis of the characteristic compressive strengths of similar materials which it is understood /9/ will be included in the Canadian code for timber structures to be published in 1990. The coefficient of variation of the strength values is approximately 0.13. This means that the mean values are approximately 25% greater than the characteristic values, which approximately corresponds to the 5th percentile. Table 2 also sets out the effective widths and slenderness ratios for the mean values of the compressive strength determined in this way.

It can be seen that the experimental results are in good agreement with the curves and that the results are not particularly sensitive to the choice of the correct compressive strength. Most of the values lie above the curve in accordance with Equation (3) by von Karman. It is only for slenderness ratios greater than 3.6 that there are values noticeably below the curve. One reason for this may be that the value $\sigma_{cr,calc}$ used is somewhat too large owing to some errors in determining the stiffness values of the plywood sheets. In test series 600/8 in which load was applied parallel and perpendicular to the direction of the grain, considerably smaller critical buckling stresses $\sigma_{cr,test}$ were measured. The corresponding slenderness ratio α is then much higher, so that even the lowest values of b_{ef}/b will be very near the curve (3).

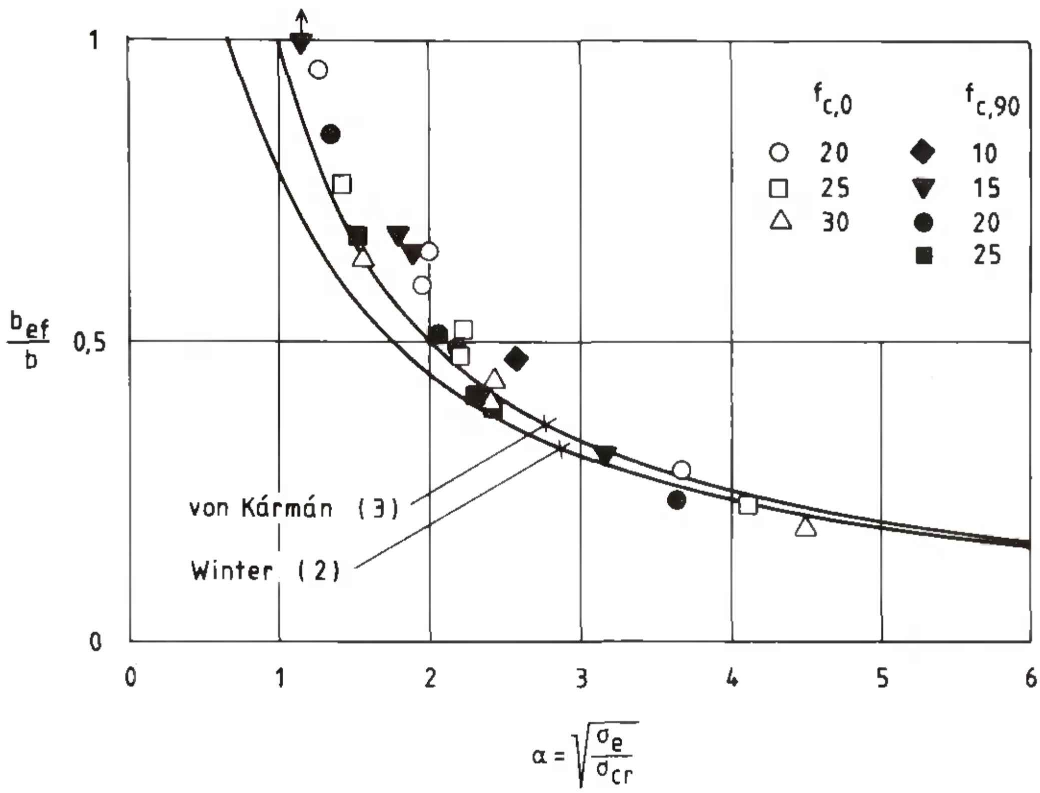

Figure 3. Effective widths determined using test results by Dekker et al (1978) /8/.

In the other test series $\sigma_{cr,calc}$ is a little smaller than $\sigma_{cr,test}$. In these cases the value of the slenderness ratio is reduced to some extent, i.e. the plots in the diagram are moved to the left by a small distance. The difference is however small and has no significance.

2.3 Proposed design regulations

For large values of the slenderness ratio, there is very little difference between the formulae according to Winter and von Karman. However, for slenderness ratios near 1 and below, the difference is large. In the test series under consideration there are no test specimens with slenderness ratios as low as this, and it is therefore not possible to verify Winter's formula in this region.

With reference to the procedure at present applied for sheet metal structures, it is proposed that in conjunction with design in the ultimate limit state the effective width of thin-walled wood-based flanges supported on two webs should be calculated as

$$b_{ef} = b \qquad \text{for } \alpha \leqslant 0.67$$

$$b_{ef} = b\sqrt{\frac{\sigma_{cr}}{\sigma_e}}\,(1-0{,}22\sqrt{\frac{\sigma_{cr}}{\sigma_e}}) \qquad \text{for } \alpha \geqslant 0.67 \qquad (8)$$

where $\alpha = \sqrt{\sigma_e/\sigma_{cr}}$

TABLE 1. Mean values, standard deviations and coefficients of variation for the test data. Only test specimens with $\alpha_v > 1$ are included.

b/t_{nom}	Direction of load	No of specimens with $\alpha_u > 1$		$\sigma_{cr,calc}$ N/mm^2	$\sigma_{cr,test}$ N/mm^2	σ_u N/mm^2	t mm
			$\bar{x}$	5.02	5.20	13.00	8.01
400/8	∥	7	s	0.545	1.34	2.91	0.113
			δ	0.109	0.258	0.224	0.014
			$\bar{x}$	12,11	12.26	19.03	11.97
400/13	∥	7	s	0.876	1,82	1.05	0,111
			δ	0,072	0.148	0,052	0,093
			$\bar{x}$	1.47	1,13	5.65	7.55
600/8	∥	3	s	0,176	0.234	0.522	0.035
			δ	0.120	0.207	0.092	0.005
			$\bar{x}$	5.14	5.21	11.88	12.2
600/13	∥	4	s	0.16	1.06	1.66	0
			δ	0.032	0.203	0.140	0
			$\bar{x}$	4,15	4.44	9.74	7,78
400/8	⊥	5	s	0.488	0.937	0,466	0,129
			δ	0.117	0.211	0.048	0.017
			$\bar{x}$	10.82	12.76	16,86	12.24
400/13	⊥	5	s	0.576	1.36	1,39	0.152
			δ	0,053	0,107	0.083	0,012
			$\bar{x}$	1.51	1,073	4.72	7,85
600/8	⊥	3	s	0.097	0,158	0.544	0,551
			δ	0,064	0.147	0.115	0.070
			$\bar{x}$	4,66	5.26	10.23	12.37
600/13	⊥	3	s	0.099	0.108	1,069	0.058
			δ	0.021	0.020	0.104	0,005

TABLE 2. Experimental effective width b_{ef} and slenderness ratio α for different values of the compressive strength f_{cb}.

b/t_{nom}	Direction of load	f_c N/mm2	$\frac{b_{ef}}{b}$	α
400/8	∥	20	0.650	1.996
		25	0.520	2,232
		30	0.433	2.445
		21.7 a)	0,600	2,079
400/13	∥	20	0.952	1.285
		25	0.761	1,437
		30	0.634	1,574
		17.0 a)	1,119	1,185
600/8	∥	20	0.282	3,685
		25	0,226	4,120
		30	0.188	4,513
		21,7 a)	0.260	3,842
600/13	∥	20	0.594	1.974
		25	0.475	2,207
		30	0.396	2,417
		17,0 a)	0,699	1,819
400/8	⊥	15	0,649	1.901
		20	0.487	2.195
		25	0.390	2.454
		6.7 a)	1.454	1.271
400/13	⊥	15	1.177	1.121
		20	0,843	1.360
		25	0,675	1.520
		7.9 a)	2.130	0.855
600/8	⊥	10	0.472	2.576
		15	0.315	3,155
		20	0.236	3.643
		25	0.189	4,073
		6.7 a)	0.705	2.106
600/13	⊥	15	0.682	1.794
		20	0.512	2.071
		25	0.409	2.316
		7.9 a)	1.295	1.302

a) Estimated mean strength according to /9/.

When there is a glue joint between the flange and the webs, b is put equal to b_f, i.e. the clear distance between the webs. For nailed or screwed connections, $b = b_f + t_w$ where t_w is the thickness of the web.

In a composite section according to figure 1, the strengths of the flange and web materials are normally different. When the compressive strength of the flange material is lower than that of the web material, σ_e in the above formula is put equal to the compressive strength f_c of the flange material. When, on the other hand, ultimate stress in the web is reached before this occurs in the flange, σ_e is put equal to the compressive edge stress which is obtained in the flange. Investigations on sheet metal structures have shown that the formula according to Winter yields good accuracy in the ultimate limit state while it is conservative for lower loads, see e.g. /11/, /12/ and /13/. Pending the availability of test results for wood-based boards, it is proposed however that the above formula should also be used for stresses which are lower than the compressive strength.

Thin-walled flanges which are supported along one edge and have a free edge also have a postcritical loadbearing capacity. For wood-based flanges no test results are available at present. It is proposed therefore that the postcritical loadbearing capacity should not be utilised in such cases.

The compressive strength of plywood is normally quoted as loadbearing capacity per unit width (e.g. N/mm). The value of f_c can be obtained by dividing this value by the thickness of the plywood.

It is proposed that in determining the critical buckling stress σ_{cr}, simply supported conditions should be assumed unless a more accurate investigation is carried out. Full fixity is very difficult to achieve in sheet metal structures. However, restraint has a favourable effect in the postcritical region also. This was shown in /11/ for hinged supports. When thin-walled flanges are bonded to solid wood webs, restraint is likely to be very effective. It is therefore proposed that some restraint should be permitted in determining σ_{cr} if the degree of restraint can be ascertained.

In conjunction with design for the serviceability limit state, it is proposed that f_c in the above expression should be replaced by the actual compressive stress in the flange. Imperfections which can give rise to visible buckling already in the subcritical region can be allowed for in this way.

3. FLANGE CURLING

In a curved beam, or beam which becomes curved owing to the external load, components of force which are directed towards the neutral layer arise in the axially loaded flanges (figure 4). These forces cause both the flange in compression and the flange in tension to curl inwards towards the centre of the section (figure 5). When this deformation of the flange becomes excessive, the loadbearing capacity of the composite section as a whole is affected. The Swedish Code for Light Gauge Metal Structures, StBK- N5 /14/ specifies that deflection shall not exceed 5% of the depth of the cross section in order that flange curling may be ignored in calculating the loadbearing capacity.

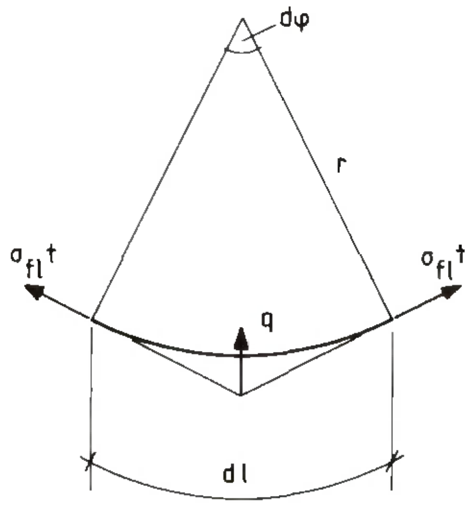

Figure 4. Forces in the flange of a curved beam.

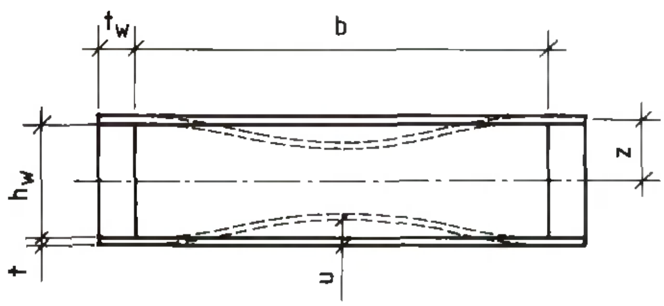

Figure 5. Curling of thin walled flanges.

The inward force component may be determined as (figure 4)

$$q = \frac{\sigma_f t d\varphi}{d\ell}$$

With $d\ell = r d\varphi$ and $\frac{1}{r} = \frac{M}{E_b I_b}$, we have

$$q = \frac{\sigma_f t}{r} = \frac{\sigma_f t M}{E_b I_b}$$

and with $\sigma_f = \frac{M}{I_b} z$ we finally have

$$q = \frac{\sigma_f^2 t}{E_b z} \tag{9}$$

where z is the distance between the neutral layer and the midplane of the flange (figure 5). The moduli of elasticity of the web and flange in a composite section are usually different. Conventionally, this is taken into consideration by e.g. putting the modulus of elasticity of the web equal to E_b and using a modified flange area in calculating the second moment of are a I_b of the composite section.

This load causes the flange to deflect, and under simply supported conditions the value of this at the midpoint of the flange is

$$u = \frac{5}{384} \frac{qb^4}{(EI)_f}$$

where $(EI)_f$ is the flexural rigidity of the flange in the transverse direction of the beam.

Substitution of (9) yields

$$u = \frac{5}{384} \frac{\sigma_f^2 tb^4}{E_b \ (EI)_f \ z} \tag{10}$$

In the same way, when the web is fully restrained, we have

$$u = \frac{1}{384} \frac{\sigma_f^2 tb^4}{E_b \ (EI)_f \ z} \tag{11}$$

In box sections with solid webs and glue joints between webs and flanges, full fixity can probably be assumed. When the webs and flanges are joined by mechanical fasteners, the degree of restraint is very uncertain, and Equation (9) for the simply supported case should be used.

For open sections, i.e. where the section comprises only a compression or tension flange which is bonded to the web, the degree of restraint depends on the torsional rigidity of the web. It should be possible to make use of some fixity by taking the mean value of Equations (10) and (11).

In the same way as in StBK-N5, the following is proposed as the limiting value of the maximum flange displacement where this must be taken into consideration:

$$u_{max} = 0.05 \ h$$

where $h = h_w + t$ for a box section

and

$$h = h_w + \frac{t}{2}$$

When the maximum displacement of the flange exceeds u_{max}, this must be allowed for in calculating the second moment of area of the composite section. Approximately, the effective distance between the flange and the neutral layer of the cross section is

$$z_{fl} = z \frac{2}{3} u$$

See figure 6.

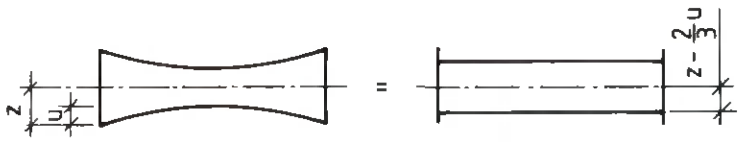

Figure 6. Effective cross section when flange curling is taken into consideration.

When the loadbearing capacity of the flange is reduced due to buckling or the effect of shear deformations, the displacement u of the flange for the mean stress in the flange is determined as

$$\sigma_f = f_c \frac{b_{ef}}{b}$$

When the flange is in the ultimate limit state, the compressive edge stress in the flange, σ_e, is put equal to the compressive strength f_c.

REFERENCES

/1/ CIB Structural Timber Code. CIB Report, Publication 66, 1983.

/2/ Buckling of Flat Plywood Plates in Compression, Shear, or Combined Compression and Shear. USDA, Forest Products Laboratory, Madison, Report No. 1316 with Supplements 1316A-1316J.

/3/ Foschi, R.O. 1969. Buckling of the compressed skin of a plywood stressed-skin panel with longitudinal stiffeners. Canadian Forestry Service, Publ. No. 1265.

/4/ Halasz, R. von & Cziesielski, E.: Berechnung und Konstruktion geleimter Träger mit Stegen aus Furnierplatten. Berichte aus der Bauforschung, Heft 47.

/5/ SIA 164, 1981, Swiss Timber Code, Schweizerischer Ingenieur- und Architekten-Verein, Zürich.

/6/ Einführung in die Norm SIA 164 (1981), Holzbau, Publikation Nr 21-1, Baustatik und Stahlbau, Eidg. Techn. Hochschule, Zürich.

/7/ Winter, G. 1947. Strength of Thin Steel Compression Flanges. Trans. ASCE, Vol. 112, 1947.

/8/ AISI. 1968. Specification for the Design of Cold-formed Steel Structural Members. 1968 Edition. American Iron and Steel Institute, Washington.

/9/ Dekker, J., Kuipers, J. & Ploos van Amstel, H. (1978). Buckling strength of plywood, Stevin-Laboratorium, Delft.

/10/ Foschi, R., private communication.

/11/ König, J. 1978. Transversally loaded thin walled C shaped panels with intermediate stiffeners. Swedish Council for Building Research, Stockholm. Document D7:1978.

/12/ Graves-Smith, T.R. 1971. A variational method for large deflection elasto-plastic theory in its application to arbitrary flat plates. Proc. Int. Conf. on Structures, Solid Mechanics and Engineering Design, Southampton University 1969, John Wiley, London (1971), pp 1249-1256.

/13/ Thomasson, P-O. 1978. Thin walled C-shaped panels in axial compression. Swedish Council for Building Research, Stockholm. Document D1:1978.

/14/ StBK-N5, Swedish Code for Light Gauge Metal Structures, Stockholm, 1980.

Detta digitala dokument
skapades med anslag från

Stiftelsen Nils och Dorthi Troëdssons forskningsfond

TräteknikCentrum
INSTITUTET FÖR TRÄTEKNISK FORSKNING

Box 5609, 114 86 STOCKHOLM
Besöksadress: Drottning Kristinas väg 67
Telefon: 08-14 53 00
Telex: 14445 tratek s
Telefax: 08-11 61 88
Huvudenhet med kansli

Åsenvägen 9, 552 58 JÖNKÖPING
Telefon: 036-12 60 41
Telefax: 036-16 87 98

931 87 SKELLEFTEÅ
Besöksadress: Bockholmsvägen 18
Telefon: 0910-652 00
Telefax: 0910-652 65
Telex: 65031 expolar s

ISSN 0283-4634