Stressed skin panels

STEP lecture B10
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Objectives
To present a procedure for calculating stresses and deflections of stressed skin panels and to introduce the concept of effective flange width.

Prerequisites
B4 Shear and torsion
B11 Mechanically jointed beams and columns

Summary
The lecture begins with a general description of the layout of stressed skin panels. The concept of the effective flange width is introduced and the composite action between webs and panels, depending on the type of connection, is explained. Finally the calculation method is demonstrated.

Another form of stressed skin panel is the sandwich panel where wood-based panel flanges are separated by an intermediate core such as foam or honeycomb. The principles of the design method of this panel are outlined at the end of the lecture.

Introduction
Stressed skin panels consist of webs in the direction of the span connected with wood-based sheets forming the skins on one or both sides. In most cases the webs are made from solid timber whereas the sheets may consist of plywood, OSB, particleboard or fibreboard. The connection can either be glued or made with mechanical fasteners such as nails, staples or screws. Stressed skin panels are mostly used in prefabricated timber frame construction as bending members for floors and roofs or as walls loaded in compression, bending and racking. Due to the connection between webs and flanges the stressed skin panel acts as a composite member and consequently the bending stiffness and bending capacity will exceed the values of the webs alone.

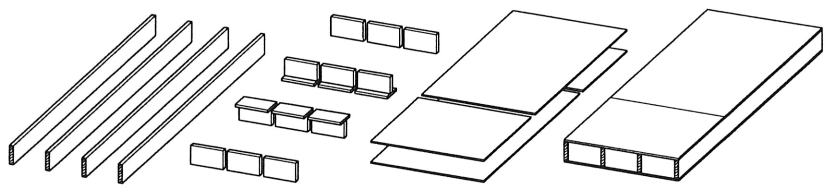

Figure 1 Construction of a stressed skin panel. (a) webs, (b) blocks and flange splices, (c) flanges, (d) stressed skin panel.

Structural layout of stressed skin panels
The dimensions of stressed skin panels are primarily limited by transport and erection. Stressed skin panels used as walls are about 2,50 *m* in height and up to 10 *m* in length, and the web units span vertically. The width of floor or roof panels lies normally between 1,25 *m* and 2,50 *m* and should correspond to the dimension of the sheets for maximum economy. With sawn timber webs the usual spans are between 5 and 6 *m*.

Flanges

The thickness of wood-based panels used as flanges is usually between 10 *mm* and 19 *mm*. If plywood is used the grain-direction of the face veneer can either be oriented perpendicular or parallel to the webs. The choice of the direction is influenced by the web spacing (bending of flooring) and the method of production. If the orientation is perpendicular to the webs, the bending capacity of the flanges between the webs is higher allowing larger web spacings. In this case, however, the strength and stiffness of the wood-based panel acting as part of the composite member is smaller compared with an orientation parallel to the webs. If large panels are prefabricated, the wood-based panel sheets have to be connected by splice joints. These joints can be made as glued scarf or finger joints or as lap joints using blocks on the inside of the panel. Fewer joints will be necessary if the longer direction is parallel to the webs. In designing stressed skin panels care needs to be taken to check the direction of the face grain in relation to the longest side of the panel.

Figure 2 Glued joints for the connection of flange panels. (a) splice joint, (b) finger joint, (c) scarf joint.

Webs

Apart from sawn timber, glued laminated timber, wood-based panels or prefabricated I-beams can be used for the webs. The thickness of sawn timber webs for wall panels is usually between 38 *mm* and 80 *mm* and the depth between 80 *mm* and 200 *mm*. For floor or roof panels the corresponding dimensions are between 38 *mm* and 63 *mm* for the thickness and 150 *mm* to 300 *mm* for the depth. The depth of the webs is not only influenced by the necessary stiffness and load-carrying capacity of the stressed skin panel but also by the thickness of insulation layers. If mechanical fasteners are used in the joints, the minimum edge distances of the fasteners have to be considered when determining the web thickness. In glued stressed skin panels the narrow edges of sawn timber webs have to be planed (regularised) before gluing. The web spacing usually lies between 300 *mm* and 625 *mm* and for efficiency should be related to the sheeting size.

Connections

In the case of glued panels, the connection between flange and web is assumed to be infinitely stiff. Consequently, a linear strain distribution over the depth of the composite cross-section may be assumed. In the case of mechanically jointed panels, however, the slip between flange and web has to be taken into account (see STEP lecture B11).

Effective flange width

Due to shear deformations, the normal stresses in the centre plane of the unsupported area of the flanges are not uniformly distributed (see Figure 3). The contributions of the flanges to the bending stiffness and bending capacity of the composite cross-section consequently decrease with increasing distance from the nearest web. The extent of the stress decrease mainly depends on the ratios b_f/l and E/G. Here, b_f is the web spacing, l is the span, E is the modulus of elasticity of the flange in the direction of the span of the stressed skin panel and G its shear modulus. The effective flange width decreases with increasing ratios E/G and b_f/l. A mathematical derivation of the effective flange width, taking into account the shear deformation in the flange can be found in Möhler et al. (1963). The resulting ratio between the effective and actual flange width b_{ef}/b_f for uniformly loaded beams on two supports is:

$$\frac{b_{ef}}{b_f} = \frac{(\lambda_1 \tanh\alpha_1 - \lambda_2 \tanh\alpha_2)}{\pi \, (\lambda_1^2 - \lambda_2^2)} \frac{2\,l}{b_f} \tag{1}$$

where

$$\alpha_1 = \frac{\lambda_1 \, \pi \, b_f}{2\,l} \tag{2}$$

$$\alpha_2 = \frac{\lambda_2 \, \pi \, b_f}{2\,l} \tag{3}$$

$$\lambda_1 = \sqrt{a + \sqrt{a^2 - c}} \tag{4}$$

$$\lambda_2 = \sqrt{a - \sqrt{a^2 - c}} \tag{5}$$

$$a = \frac{E_y}{2\,G} - \mu_{xy} \tag{6}$$

$$c = E_y \,/\, E_x \tag{7}$$

and where μ_{xy} is Poisson's ratio.

In order to be able to use the elementary beam theory in the calculation of stressed skin panels, the concept of the effective flange width is used. The effective flange width b_{ef} is defined as the width of an idealised flange cross-section where the normal stress in the centre of the flange resulting from elementary beam theory equals the maximum stress according to the correct theory, taking into account the shear deformations in the flanges. The total flange force thus remains the same and gives the same moment of resistance.

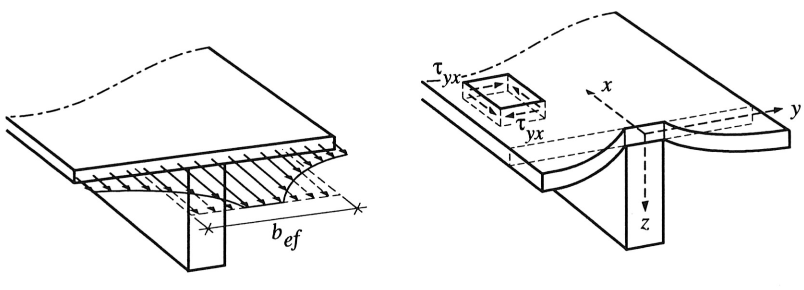

Figure 3 Stress-distribution in the flange.

EC5: Part 1-1: 5.3.2

EC5 gives the following approximation for the effective flange width b_{ef} for I-beams (or internal beams), respectively:

$$b_{ef} = b_{c,ef} + b_w \text{ (or } b_{t,ef} + b_w) \tag{8}$$

and for C-beams (or edge beams), respectively:

$$b_{ef} = 0{,}5\; b_{c,ef} + b_w \text{ (or } 0{,}5\; b_{t,ef} + b_w) \tag{9}$$

The values of $b_{c,ef}$ and $b_{t,ef}$ should not be greater than the maximum value calculated for the shear lag. In addition the value of $b_{c,ef}$ should not be greater than the maximum value calculated for plate buckling. The values according to EC5 are given in Table 1.

Flange material	Shear lag	Plate buckling
Plywood, with grain direction in the outer plies:		
- parallel to the webs	0,1 l	25 h_f
- perpendicular to the webs	0,1 l	20 h_f
Oriented strand board	0,15 l	25 h_f
Particleboard or fibreboard with random fibre orientation	0,2 l	30 h_f

Table 1 *Maximum effective flange widths due to shear lag and plate buckling.*

Figure 4 shows the effective flange width according to equation (1) and the corresponding approximation of EC5 for shear lag. Most stressed skin panels in practice show ratios b_f/l smaller than 0,3.

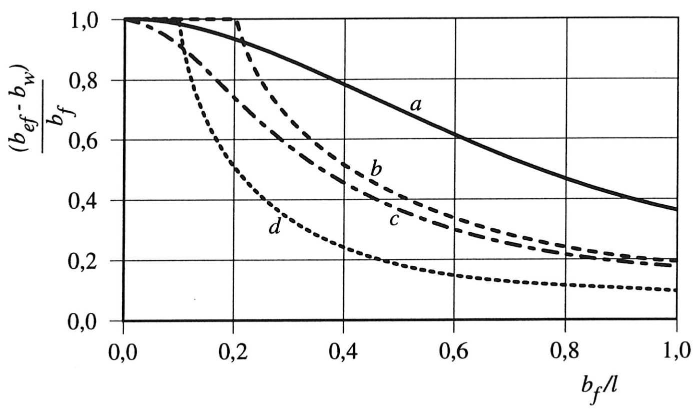

Figure 4 *Effective flange width according to equation (1) and EC5. (a) particleboard equation (1), (b) particleboard EC5, (c) plywood equation (1), (d) plywood EC5.*

Flanges loaded in compression are prone to buckling. A detailed buckling analysis can be carried out for example according to von Halász and Csiesielski (1966). If a detailed buckling investigation is not made, the clear flange width b_f should not be greater than twice the effective width to avoid plate buckling. For nailed or stapled stressed skin panels the withdrawal capacity of the nails has to be sufficient to anchor the sheets against buckling.

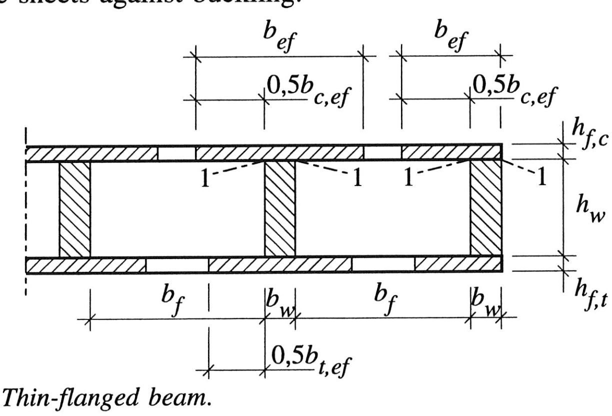

Figure 5 *Thin-flanged beam.*

Glued stressed skin panels

The equations needed to calculate the bending stiffness and the stresses in the different components of a stressed skin panel with a flange on the top are given below. Glued stressed skin panels are calculated assuming rigid joints between flange and web.

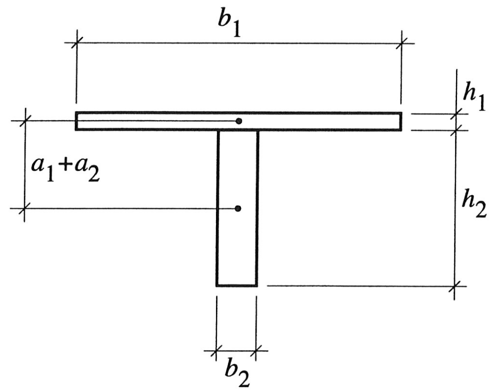

Figure 6 Cross section of a glued stressed skin panel. 1 denotes the flange, 2 the web.

$$a_1 = \frac{E_2 A_2 (h_1 + h_2)}{2 (E_1 A_1 + E_2 A_2)} \tag{10}$$

$$a_2 = \frac{h_1 + h_2}{2} - a_1 \tag{11}$$

The effective bending stiffness is:

$$(EI)_{ef} = \sum_{i=1}^{2} (E_i I_i + E_i A_i a_i^2) \tag{12}$$

The compression stress acting at the centre of the flange is:

$$\sigma_{1,c} = \frac{M}{(EI)_{ef}} E_1 a_1 \tag{13}$$

The compression stress at the top of the flange is:

$$\sigma_{1,c} = \frac{M}{(EI)_{ef}} E_1 (a_1 + \frac{h_{1,c}}{2}) \tag{14}$$

The tension stress at the centre of the web is:

$$\sigma_{2,t} = \frac{M}{(EI)_{ef}} E_2 a_2 \tag{15}$$

The bending stress at the bottom of the web is:

$$\sigma_{2,m} = \frac{M}{(EI)_{ef}} E_2 (a_2 + \frac{h_{2,t}}{2}) \tag{16}$$

The shear stress at the joint between the web and the flange is:

$$\tau_{max} = \frac{V E_1 A_1 a_1}{(EI)_{ef} b_2} \tag{17}$$

For stressed skin panels with flanges at both top and bottom and for mechanically jointed stressed skin panels reference is made to EC5 Appendix B and STEP lecture B11.

Design example

A stressed skin panel used as a flat roof bending member on two supports, span $l = 4{,}7\ m$, web spacing $b_f + b_w = 625\ mm$, nailed connection between flanges and web. The design example only covers the stresses at the instantaneous deformation. A method for determining the stresses at the final deformation, using the appropriate values of k_{def} is shown in STEP lecture B9.

Characteristic and design values of permanent and variable load per web for the governing load case:

permanent load:	$g_k = 0{,}31\ kN/m$	$g_d = 0{,}42\ kN/m$	(permanent)
variable load:	$q_k = 1{,}25\ kN/m$	$q_d = 1{,}88\ kN/m$	(medium-term)

Top flange: US Plywood C-C, Exterior, Group 1, unsanded according to EN 112.406 "Wood-based panels. Characteristic values for established products." $d = 16\ mm$, three layers, orientation of the face veneer perpendicular to the webs.

Webs: Strength class C22 according to prEN 338 "Structural timber. Strength classes". b x h = 40 x 180 mm.

Bottom flange: US Plywood C-C, Exterior, Group 1, unsanded according to EN 112.406. $d = 11{,}5\ mm$, 5 layers, orientation of the face veneer perpendicular to the webs.

EC5: Part 1-1: 3.1.7

Service class 1: $k_{mod} = 0{,}8$ (solid timber and plywood)

prEN 338: 1991

EC5: Part 1-1: B2(1)

Characteristic material properties:
The characteristic strength values as well as the modulus of elasticity are taken from prEN 338 and EN 112.406, respectively. For the modulus of elasticity and the slip modulus of the nails, the mean value is used in the design although an ultimate limit state is considered.

Top flange:	$f_{m,90,k}$	$= 12{,}1\ N/mm^2$	$f_{c,90,k}$	$= 8{,}4\ N/mm^2$
	$E_{c,0,mean}$	$= 5600\ N/mm^2$	$E_{c,90,mean}$	$= 4400\ N/mm^2$
	$G_{v,mean}$	$= 500\ N/mm^2$	ρ_k	$= 410\ kg/m^3$
Webs:	$f_{m,k}$	$= 22{,}0\ N/mm^2$	$f_{v,k}$	$= 2{,}4\ N/mm^2$
	$E_{0,mean}$	$= 10000\ N/mm^2$	ρ_k	$= 340\ kg/m^3$
Bottom flange:	$f_{m,90,k}$	$= 12{,}9\ N/mm^2$	$f_{t,90,k}$	$= 7{,}2\ N/mm^2$
	$E_{t,0,mean}$	$= 7000\ N/mm^2$	$E_{t,90,mean}$	$= 4700\ N/mm^2$
	$G_{v,mean}$	$= 500\ N/mm^2$	ρ_k	$= 410\ kg/m^3$
Nails:	d	$= 4{,}0\ mm$	s	$= 40\ mm$
	$M_{y,k}$	$= 6620\ Nmm$		

EC5: Part 1-1: 6.2.1

The design load-carrying capacity per shear plane per fastener of the nailed panel-to-timber joints is:

Top flange - web: $R_d = 903\ N$

Bottom flange - web: $R_d = 842\ N$

EC5: Part 1-1: 5.3.3

The instantaneous slip modulus per shear plane per fastener of the nailed panel-to-timber joints results as:

$K_u = 583\ N/mm$

EC5: Part 1-1: 5.3.2

Effective flange width:
$b_{c,ef} = 20 \cdot 16 = 320\ mm$
$b_{t,ef} = 0{,}1\ l = 0{,}1 \cdot 4700 = 470\ mm$

top flange: $b_{ef} = b_{c,ef} + b_w = 320 + 40 = 360\ mm < 625\ mm$
bottom flange: $b_{ef} = b_{t,ef} + b_w = 470 + 40 = 510\ mm < 625\ mm$

According to Möhler et al. (1963), an effective flange width for the top and the bottom flange of $b_{ef} = 567\ mm$ and $b_{ef} = 564\ mm$, respectively, results.

The effective bending stiffness of the cross-section is calculated according to EC5 Annex B. Table 2 shows the results of the calculation including the corresponding equation numbers taking into account the effective flange width according to EC5 as well as according to Möhler et al. (1963).

	Equation No.	EC5	Möhler et al. (1963)
h_1 (mm)		16	16
h_2 (mm)		180	180
h_3 (mm)		11,5	11,5
b_1 (mm)		360	567
b_2 (mm)		40	40
b_3 (mm)		510	564
A_1 (mm^2)	B2b	5760	9070
A_2 (mm^2)	B2b	7200	7200
A_3 (mm^2)	B2b	5860	6480
E_1 (N/mm^2)		4400	4400
E_2 (N/mm^2)		10000	10000
E_3 (N/mm^2)		4700	4700
γ_1	B2e	0,56	0,45
γ_2	B2d	1,00	1,00
γ_3	B2e	0,54	0,52
a_1 (mm)		99	93
a_2 (mm)	B2f	-1	5
a_3 (mm)		95	100
$(EI)_{ef}$ (Nmm^2)	B2a	$469 \cdot 10^9$	$512 \cdot 10^9$

Table 2 Calculation of effective bending stiffness according to EC5 Annex B.

Design shear force:

$$V_d = \frac{(g_d + q_d)\ l}{2} = \frac{(0{,}42 + 1{,}88)\ 4700}{2} = 5400\ N$$

Design bending moment:

$$M_d = \frac{(g_d + q_d)\, l^2}{8} = \frac{(0{,}42 + 1{,}88)\, 4700^2}{8} = 6{,}35 \cdot 10^6 \ Nmm$$

For the calculation of the design stresses, the effective bending stiffness based on the effective flange width according to EC5 is used.

EC5: Part 1-1: B3a

Design compression stress in the top flange:

$$\sigma_{1,d} = \frac{M_d}{(EI)_{ef}} \gamma_1 E_1 a_1 = \frac{0{,}56 \cdot 4400 \cdot 99 \cdot 6{,}35 \cdot 10^6}{469 \cdot 10^9} = 3{,}30 \ N/mm^2$$

$$\sigma_{1,d} < f_{c,90,d} = \frac{k_{mod}\, f_{c,90,k}}{\gamma_M} = \frac{0{,}8 \cdot 8{,}4}{1{,}3} = 5{,}17 \ N/mm^2$$

EC5: Part 1-1: B3a, B3b

Design bending stress in the web:

$$\sigma_{2,d} + \sigma_{m,2,d} = \frac{M_d}{(EI)_{ef}} \gamma_2 E_2 a_2 + \frac{M_d}{(EI)_{ef}} 0{,}5\, E_2 h_2$$

$$\sigma_{2,d} + \sigma_{m,2,d} = (1{,}0 \cdot 10000 \cdot 1 + 0{,}5 \cdot 10000 \cdot 180) \frac{6{,}35 \cdot 10^6}{469 \cdot 10^9}$$

$$\sigma_{2,d} + \sigma_{m,2,d} = 12{,}2 \ N/mm^2 < f_{m,d} = \frac{k_{mod}\, f_{m,k}}{\gamma_M} = \frac{0{,}8 \cdot 22}{1{,}3} = 13{,}5 \ N/mm^2$$

EC5: Part 1-1: B3a

Design tension stress in the bottom flange:

$$\sigma_{3,d} = \frac{M_d}{(EI)_{ef}} \gamma_3 E_3 a_3 = \frac{0{,}54 \cdot 4700 \cdot 95 \cdot 6{,}35 \cdot 10^6}{469 \cdot 10^9} = 3{,}27 \ N/mm^2$$

$$\sigma_{3,d} < f_{t,90,d} = \frac{k_{mod}\, f_{t,90,k}}{\gamma_M} = \frac{0{,}8 \cdot 7{,}2}{1{,}3} = 4{,}43 \ N/mm^2$$

EC5: Part 1-1: B4

Design shear stress in the web:

$$\tau_{2,max,d} = \frac{(\gamma_3 E_3 A_3 a_3 + 0{,}5\, E_2 b_2 h^2)\, V_d}{b_2\, (EI)_{ef}}$$

With $h = 0{,}5\, h_2 + a_2 = 89\ mm$ the design shear stress results in:

$$\tau_{2,max,d} = \frac{(0{,}54 \cdot 4700 \cdot 5860 \cdot 95 + 0{,}5 \cdot 10000 \cdot 40 \cdot 89^2)\, 5400}{40 \cdot 469 \cdot 10^9}$$

$$\tau_{2,max,d} = 0{,}87 \ N/mm^2 < f_{v,d} = \frac{k_{mod}\, f_{v,k}}{\gamma_M} = \frac{0{,}8 \cdot 2{,}4}{1{,}3} = 1{,}48 \ N/mm^2$$

EC5: Part 1-1: B5

Design fastener load in the top flange:

$$F_{1,d} = \frac{\gamma_1 E_1 A_1 a_1 s_1 V_d}{(EI)_{ef}} = \frac{0{,}56 \cdot 4400 \cdot 5760 \cdot 99 \cdot 40 \cdot 5400}{469 \cdot 10^9} = 647 \ N$$

$$F_{1,d} < R_{1,d} = 903 \ N$$

EC5: Part 1-1: B5

Design fastener load in the bottom flange:

$$F_{3,d} = \frac{\gamma_3\ E_3\ A_3\ a_3\ s_3\ V_d}{(EI)_{ef}} = \frac{0{,}54 \cdot 4700 \cdot 5860 \cdot 95 \cdot 40 \cdot 5400}{469 \cdot 10^9} = 654\ N$$

$$F_{3,d} < R_{3,d} = 842\ N$$

EC5: Part 1-1: 5.3.2(5)

A detailed buckling analysis is not necessary since the clear flange width $b_f = 585$ *mm* is smaller than twice the effective width due to plate buckling:

$b_f < 2 \cdot 20\ h_f = 640\ mm$

Instantaneous deflection:

$$u_{inst} = \frac{5\ (g_k + q_k)\ l^4}{384\ (EI)_{ef}} = \frac{5\ (0{,}31 + 1{,}25) \cdot 4700^4}{384 \cdot 469 \cdot 10^9} = 21{,}2\ mm = \frac{l}{222}$$

Sandwich panels

Sandwich panels with faces consisting of wood-based panels and a core of expanded foam are increasingly used as walls or roofs in timber frame buildings and as roof elements for industrial buildings. The faces often consist of particleboard, the core of polyurethane or polystyrene foams.

EC5: Part 1-1: Annex B

Using the following assumptions, three layer sandwich panels can be calculated as mechanically jointed components:

- the normal stresses in the foam core in the direction of the member axis are disregarded,

- the shear deformations in the foam core are taken into account by replacing the joint stiffness K/s in a mechanically jointed component by G_{mean}/h for the sandwich panel. Here, K is the slip modulus and s the fastener spacing, G_{mean} is the shear modulus and h the thickness of the foam.

A detailed description of the calculation of sandwich panels is given in Aicher and von Roth (1987) and in Aicher (1987).

Concluding summary

- Stressed skin panels are primarily used as bending members in floors and roofs and as compression members in walls.

- Due to shear deformations in the wood-based panel sheeting the flanges contribute only partly to the composite cross-section. In stressed skin panels where the connections between web and flange are made with mechanical fasteners, the slip in the connections has also to be taken into account.

- For maximum economy, the size of stressed skin panels as well as the web spacing should correspond to the dimensions of the wood-based panels.

References

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