EUROPEAN RECOMMENDATIONS
FOR SANDWICH PANELS

PART I: DESIGN

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by the Joint Committee

European Convention for Constructional Steelwork
TWG 7.9 Sandwich panels and related structures

International Council for Research and Innovation in Building and Construction
W056 Lightweight constructions

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PREFACE

The preliminary European Recommendations for the design and testing of sandwich panels were published by the European Convention for Constructional Steelwork (ECCS) in 1991. These Recommendations, published as ECCS Publication No. 66, concentrated on sandwich panels with thin metal faces and plastic foam cores. Two years later, the Recommendations were extended by CIB Commission W56 to cover sandwich panels with cores made of mineral wool lamellas and other slabstock materials. The work of the CIB Commission was published in CIB Publication No. 148, originally in 1993 and as a reprint again in 1995.

A great deal of experience of the design and use of sandwich panels with new material combinations and new areas of application has been gained since the Recommendations were first drafted. The work of harmonization of the design rules and the standards for loads, materials and structures within Europe under the auspices of CEN has also emphasised the need to revise the recommendations for sandwich panels.

In order to bring this new experience and knowledge to interested experts, manufacturers and suppliers, CIB Commission W56 has prepared a comprehensive textbook covering all of the important considerations which arise in the design, manufacture and use of sandwich panels. This book, entitled "Lightweight sandwich construction", will be published by Blackwell Science in 2001. On the basis of the material collected for this book, the European Recommendations for the design of sandwich panels have now been completely updated by a Joint Working Group composed of the ECCS Technical Working Group TWG 7.9 and the CIB Working Commission W56.

The following individual members of ECCS TWG 7.9 and CIB W56 have taken part in the drafting of this document:

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The French producers of sandwich panels have requested that the following statement should be added to this Preface in order to clarify their position in the context of the special considerations pertaining to the French market:

"The French producers of sandwich panels view with favour the present Recommendations inasmuch that they represent a comprehensive and up-to-date description of the scientific knowledge in this field. At the same time, it is clear that several topics are still not fully understood (such as certain properties or the behaviour of certain types of products). In such cases, the Recommendations give proposals for design whose accuracy and practical and economic consequences have still to be confirmed. For these reasons, the French producers believe that they must stress that they consider it to be inopportune to introduce the Present Recommendations into any normative or regulatory document."

The technology of sandwich construction continues to advance and the work of developing the guidelines for design, testing and use continues. ECCS TC7 and CIB W56 will, therefore, welcome critical comments and proposals to improve the document.

J.M. Davies	L. Heselius
Chairman of ECCS TWG 7.9	Coordinator of CIB W56

1. INTRODUCTION

1.1 General

These Recommendations apply to roof or wall cladding, ceiling and internal wall panels in the form of a sandwich in which the inner and outer faces are formed from thin metal sheets and the core is a relatively low density material having both stiffening and insulating properties. The components of the sandwich must be bonded together in such a manner as to provide a composite load-bearing panel. Adequate bonding may be achieved by using the inherent bonding capability of certain rigid plastic foams or by the use of separate adhesives. The design equations may only be applied to fully bonded panels. If either face is only partially bonded to the core, all components of resistance must be defined by testing paying due regard to long-term effects.

One or both metal faces may be flat, lightly profiled or fully profiled. The formulae and the design rules are applicable for the range of face thicknesses of 0.5 to 2.0 mm and for panel depths of 30 to 300 mm. Outside these ranges, additional precautions are required.

The core material may be a chemically formulated foam (eg polyurethane, polyisocyanurate, polystyrene, phenolic resin), mineral wool or other material having similar mechanical and insulating characteristics. It must have sufficient strength and stiffness to contribute to the composite action and to enable the panel to adequately carry the design loads. The core itself and its bond with the face material must have adequate durability with regard to both short and long term effects including creep and ageing.

The document as a whole is concerned with structural sandwich panels designed to resist such external loading conditions as wind and snow. For internal construction, less onerous requirements are also formulated.

The sandwich panel system must include suitable fastenings to secure the composite cladding units to the supporting framework in a sound and weathertight manner without crushing the core material.

The design principles and requirements for safety are given in Chapter 2. The design of sandwich structures is based on the design equation $S_d \leq R_d$. Calculation of the effects of actions (S_d), i.e. stresses and deflections, is introduced in Chapter 3 and the evaluation of resistances against the different failure modes (R_d) in Chapter 4. In Chapter 5, test procedures to determine the basic design values and to control the quality of sandwich panels during the manufacturing process are given. The quality control process and the interpretation of the test results are also described. In Chapter 6 an alternative design method for sandwich panels based on full-scale testing is given. Chapter 7 presents the design principles for fastenings between the sandwich panels and the supporting structures. In Chapter 8, a list of references is given which provides the technical and normative background to these Recommendations. The document also includes five Appendices which give additional background information about the behaviour, design and testing of structural sandwich panels.

The design of sandwich panels is based on simplified calculation models together with basic material and structural tests. However, the use of more advanced (numerical) methods of analysis and/or full scale testing may be required in certain cases in order to take into account special properties of the panel systems. The use of such advanced methods is outside the scope of these Recommendations.

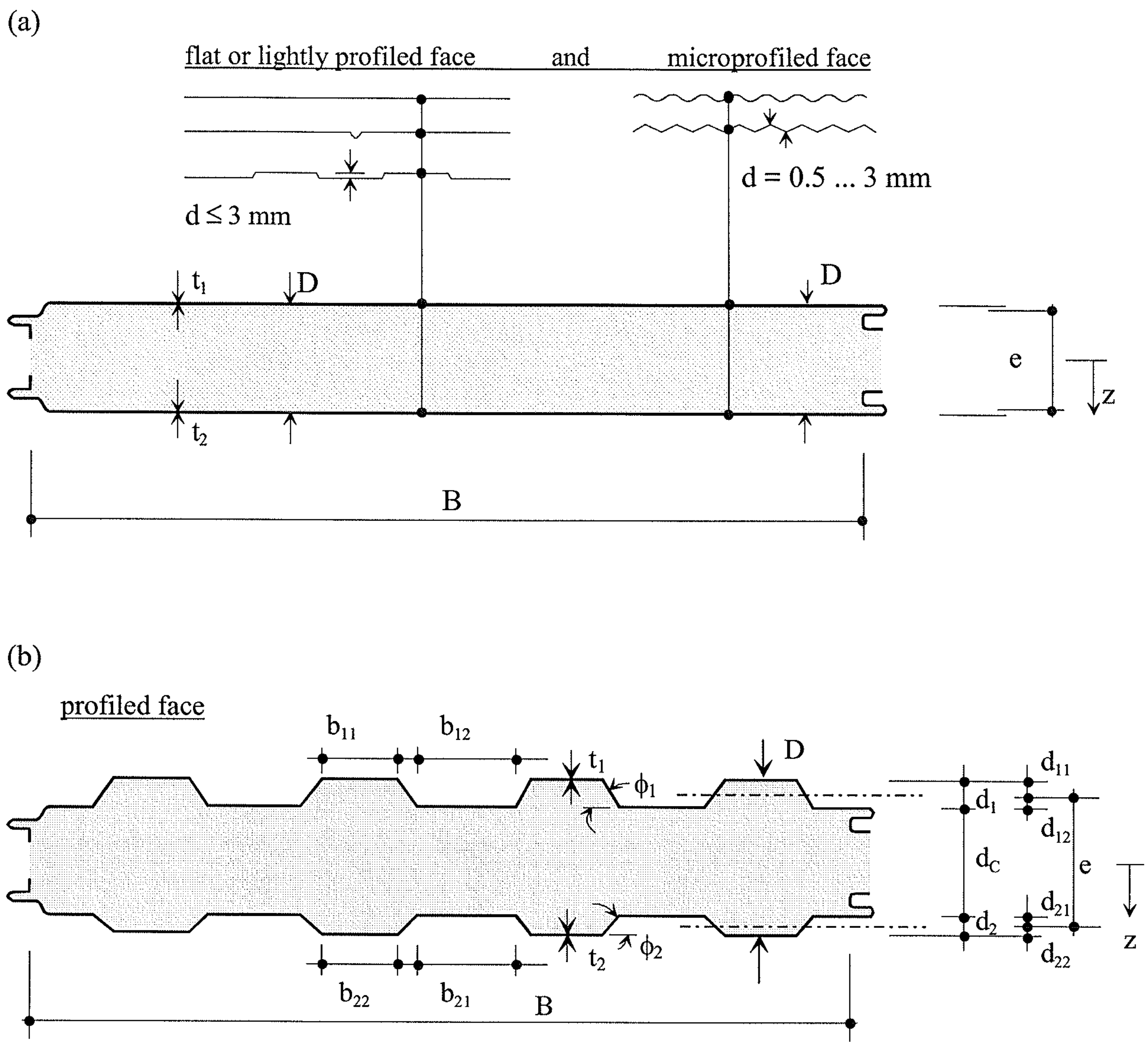
1.2 Symbols

The following symbols are used in these Recommendations.

A	cross-sectional area
B	flexural rigidity, overall width of the panel
D	overall depth of the panel, axial rigidity of face sheet
E	modulus of elasticity
F	force, load, action
G	shear modulus, self-weight, permanent action
I	moment of inertia
K	buckling coefficient
L	span, distance
M	bending moment
N	axial compressive force
Q	variable action
R	resistance, reflectivity, parameter
S	shear rigidity, value of a load effect, effect of an action
T	temperature
V	shear force
W	section modulus
a	half buckling wave length
b	width of test specimen, width of plate, width of plane part in profile
d	diameter of screw, depth of face profile, depth of core
e	distance between centroids of faces, base of natural logarithms (e = 2.718282)
f	strength
g	self-weight
i	index
k	coefficient, parameter ($k = 3B/L^2S$), spring constant
n	number of tests, number of screws, number of webs
q	live load
s	length
t	thickness of face sheet
w	deflection
x	test result
x, y, z	coordinates
α	angle of dispersion, parameter, coefficient of thermal expansion
β	parameter
ε	strain
ϕ	ratio ($\phi = a/b$), angle
γ	shear strain, partial safety factor
φ	creep coefficient
ϑ	parameter
λ	parameter
θ	parameter ($\theta = (\alpha_{F2}\,T_2 - \alpha_{F1}\,T_1) / e$)
ν	Poisson's ratio
σ	stress
τ	shear stress
ψ	combination coefficient

Subscripts

C	core
F	face, action (γ_F)
G	self-weight, permanent load, degree
M	material (γ_M)
Q	variable action
R	resistance
S	sandwich part of the cross-section
T	temperature
b	bending
c	compression, creep
cr	critical, eigenvalue, eigenmode
d	design
eff	effective
i, j	index
k	characteristic value
p	profile, buckling of a plane part of face profile
q	uniform load
s	snow, support (L_s = support width)
t	tension, time
u	ultimate
v	shear
w	wind, web
x, y, z	coordinates
0	basic value, unit width
1	external face, upper face
2	internal face, lower face



Layer	Geometry	Material properties	Structural properties
Face 1	t_1, d_1, d_{11}, d_{12}, A_{F1}, I_{F1}	E_{F1}, α_{F1}	B_{F1}
Core	d_C	E_C, G_C	S
Face 2	t_2, d_2, d_{21}, d_{22}, A_{F2}, I_{F2}	E_{F2}, α_{F2}	B_{F2}

Fig. 1.1 Definition of cross-section and material properties of a sandwich panel.

a) Thin-faced panel and

b) panel with profiled and/or thick-faces.

1.3 Definitions

Action	a load or a temperature difference between the face layers, which causes stresses and deflections in a sandwich panel.
Accidental load	an exceptional load, e.g. fire, explosion or impact.
Additional layer	fourth and fifth layers placed between the core and faces in order to improve the fire behaviour of the sandwich panel.
Ageing	the tendency for panels to suffer a change, most often a loss of strength, with time. This is usually a consequence of the effect of moisture and temperature on the core materials, adhesives and binders used during manufacture.
Ceiling	a part of the roof structure generally mounted below the load bearing frame inside the building. A ceiling may be subjected to access load. Typically, the span is the distance between the main load bearing frames.
Core	the material between the inner and outer faces which forms the contents of the sandwich panel.
Creep	the tendency for panels to suffer increasing deflection with time as a consequence of viscous flow in the core material.
Degradation factor	the ratio of the tensile strength of a core material measured in a direction normal to the faces after accelerated ageing to the tensile strength of the unaged material. These strengths should be measured with the face material in place so that the value reflects both the strength of the material itself and the bond with the faces.
Discrete core material	a core material, which is not foamed in-situ. It is supplied in finite dimensions, which are equal to or less than the dimensions of the sandwich panel. Therefore, special attention is required at the transverse joints between the parts of the core. Slabstock and lamellas are typical examples of discrete core materials.
Durability	the ability of a panel to retain its strength with time in the presence of climatic attack (moisture and temperature).

External wall	a panel which forms the external wall of a building and which is exposed to actions from the external climate such as wind, temperature from sunlight etc.
Face	the metal skin of the panel.
Factored action	combination of loads or actions, which include load and combination factors.
Fastening	a point of connection between the sandwich panel and its supporting framework. A fastening may incorporate one or more fasteners.
Fully bonded panels	The core layer is bonded to the face layers through the whole surface area between the face and the core. If the face is profiled, a core made from slabstock is shaped to follow the face profile. If the core is built up from several parts in the depth direction of the panel, i.e., from a flat part and separate parts to fill the face profile, the individual parts shall be bonded to each other over at least 90% of the whole area of their joints.
In-situ foamed core material	a core material, which is formed during the panel production process so that the adhesion between the core and the faces is established by the foam itself.
Internal wall	a panel which forms an internal wall within a building and which is not subject to actions from the external climate.
Lamella	a strip of mineral wool core material formed by cutting a laminar mineral wool slabstock and turning it through 90° so that the fibres are reorientated in a more favourable direction (i.e. at approximately 90° to the faces).
Lightly profiled face	a lightly profiled face contains rolled-in longitudinal profiling of up to 3 mm depth. A characteristic of light profiling is that the bending stiffness of the face itself can be ignored relative to the bending stiffness of the panel as a whole, so that the face may be treated as though flat for the purposes of global structural analysis. However, light profiling may be of significant benefit in enhancing the wrinkling stress of the face.

Long term action	an action of long duration whose long-term effect may cause time-dependent changes in stresses or deflections (see creep)
Micro-profiled face	a face layer containing a series of small longitudinal profiles made to add texture for architectural reasons. From a structural point of view, a micro-profiled face may be considered to be a special case of a lightly profiled face. (see above)
Mineral wool	an extrusion of molten mineral to form fine filaments which are bound together using an adhesive binder. The term mineral wool includes stone, glass and slag wools.
Permanent action	an action which is likely to act throughout a given design situation and for which the variation in magnitude with time is negligible in relation to the mean value, or for which the variation is always the same direction (monotonic) until the action attains a certain limiting value.
Profiled face	a metal skin which has been cold-formed to give it bending strength.
Repeated load	a load whose intensity varies with time and where the effect of the number of cycles of load cannot be neglected.
Sandwich panel	a composite construction of layered materials which comprises outer facings of rigid material (usually sheet metal) and an adhesively bonded lightweight core material(s) which provides the insulation and other mechanical properties. This enables panels to achieve large span/weight ratios with a high level of thermal insulation.
Self-supporting sandwich panel	see structural sandwich panel
Short term action	An action which acts for only a short period of time and whose long-term effects may be neglected.
Slabstock	core material which is pre-formed into slabs of thickness equal to the required depth of the core and then bonded to the faces using a suitable adhesive. The length and width of a slab of core material are less than or equal to the length and width of the sandwich panel.
Structural ceiling	the ceiling of an individual room inside a building and which is designed to support foot traffic and/or permanent services and/or light equipment.

Structural sandwich panel — a panel designed for use as an external wall or roof element and subject to the usual requirements for wind load, snow load etc. and also to full quality assurance.

Static load — a load which rises from zero to some more or less constant value in such a manner that the variation of load with time can be neglected.

Thermal bow — the tendency of a sandwich panel to deflect as a consequence of a temperature difference between its faces.

Thin faced sandwich panel — a sandwich panel with flat, lightly profiled or microprofiled faces. The depth of the face profile is less than 3 mm. The bending stiffness of the face layer can be neglected in the static analysis.

Thick faced sandwich panel — a sandwich panel in which one or both faces have been profiled and the depth of the profiling is more than 3 mm. If the material thickness of a flat face is large, the sandwich panel may also be classified to the group of thick faced sandwich panels, which is a very rare case for metal sheet faced sandwich panels. The bending stiffness of the face layer itself can not be neglected.

Variable action — is an action which is unlikely to act throughout a given design situation or for which the variation in magnitude with time is not negligible in comparison with the mean value nor monotonic.

Walkability — the ability of a roof or ceiling panel to resist access loads either during or after construction. A typical access load may be caused by an operative carrying a load across a roof or a ceiling.

Wrinkling — flat or lightly profiled faces in compression tend to form a series of buckling waves. As the load is increased, failure can take place when one such wave folds into a wrinkle.

2. SAFETY REQUIREMENTS AND DESIGN PRINCIPLES

2.1 General

It should be verified by means of statical analysis and/or testing that the following equation is satisfied at both the ultimate limit state and the serviceability limit state:

$$S_d \leq R_d \tag{2.1}$$

where $$S_d = \sum_i \gamma_{Fi} \psi_i S_{ki} \tag{2.2}$$

$$R_d = \frac{R_k}{\gamma_M} \tag{2.3}$$

γ_F = relevant load factor according to 2.5.2.
γ_M = relevant material factor according to 2.5.3.
ψ = combination coefficient according to 2.5.1
S_k = characteristic value of effect of an action
R_k = characteristic value of resistance at relevant limit state
S_d = design value of the effect of the actions
R_d = design value of resistance at relevant limit state

Equation (2.1) shall be equally satisfied whether the design is carried out on the basis of calculations (Chapters 3, 4 and 5) or using full-scale testing (Chapters 5 and 6).

2.2 Actions

2.2.1 Permanent actions

The permanent actions shall include the following:

(a) self weight of the sandwich panel (calculated from the nominal dimensions and mean densities)

(b) weight of any permanent components of structure which apply load to the sandwich panel

(c) permanent imposed deformations, e.g. in cold stores located within the building envelope due to the constant negative internal temperature or to pressure difference (due to high differences of temperatures and to ventilation systems).

2.2.2 Variable actions

The variable actions shall include the following:

(a) snow
(b) live loads (e.g. due to access to a roof)
(c) wind loads

(d) climatic effects
(e) construction loads

Representative values of these actions are given in Eurocode 1 /Eurocode 1 1994/ and National Application Documents.

Notes:

1. In roof and ceiling panels, access loads during construction and maintenance can be significant and should be considered.

2. In the absence of an alternative loading requirement, it is recommended that the characteristic value of the uniformly distributed load for roofs and ceilings is $q_k = 0.25$ kN/m^2 . In addition, a concentrated load of $F_k = 1.2$ kN shall be taken into account in the design of ceiling panels.

3. Sandwich panels are not generally suitable for continuous foot traffic. The influence of repeated foot traffic on the resistance of the panel has to be studied case by case.

4. These rules cover the case of occasional access.

5. Further information on walkability is given in section 4.5.

Temperature gradients due to climatic effects may be considered to be short-term actions. The influence of temperature should always be taken into account in elastic analysis but only in plastic analysis when it is relevant. The temperature gradient results from the difference between the outside temperature T_1 and the inside temperature T_2. If National Standards do not give values for the temperatures of the faces of the element, the following values for the temperature of the outside face may be used:

The temperature T_1 of the outside face has a minimum Winter value of -10 °C in a maritime climate (UK), -20 °C in Central Europe and -30 °C in the Nordic countries. The temperature of the outside face of a roof panel with an overlying snow load is 0 °C.

The temperature T_1 of the outside face has a maximum Summer value which depends on the colour and reflectivity of its surface. For ultimate limit state calculations, T_1 = 80 °C for all colours. For serviceability calculations, T_1 may be taken as follows:

(i)	very light colours,	$R_G = 75\text{-}90$,	$T_1 = +55$ °C
(ii)	light colours	$R_G = 40\text{-}74$,	$T_1 = +65$ °C
(iii)	dark colours	$R_G = 8\text{-}39$,	$T_1 = +80$ °C

where R_G = degree of reflection relative to magnesium oxide = 100%.

Note: The temperature of the outside face of a roof panel with an overlaying snow depends very much on the density and the moisture content of the snow and it may be much lower than 0 °C.

Load-span tables may be based on any of the values of T_1 = 55°C (i), T_1 = 65°C (ii) and T_1 = 80°C (iii). If T_1 = 55°C (i) is used, the designer should be sure, that the surface will retain its original reflectivity for a long time, taking into account the soiling and ageing of the coating.

Examples of the colour classifications in the different groups are:

(i)	grauweiss	light grey
	hellelfenbein	light ivory
(ii)	sandgelb	sandy yellow
	lichtblau	light blue
	reseda grün	pale green
	olivgelb	olive yellow
	olivgrau	olive grey
(iii)	brilliantblau	bright blue
	nussbraun	nut brown
	anthrazitgrau	anthracite grey

Comment: Examples of the colours indicating the face temperatures are informative, only.

The value of R_G can be obtained from data provided by the manufacturer of the coating.

In special cases, the value of R_G may be determined by testing (See references ECCA-T3 1985 and ASTM D2244-93 for the test method and the interpretation of the test data) and the surface temperature T_1 for the specific coating may be interpolated using

$$T_1 = 55°\mathrm{C} \quad \text{for } R_G \geq 75$$
$$T_1 = 65°\mathrm{C} - \frac{R_G - 40}{35} 10°\mathrm{C} \quad \text{for } 40 \leq R_G < 75$$
$$T_1 = 80°\mathrm{C} - \frac{R_G - 15}{25} 15°\mathrm{C} \quad \text{for } 15 \leq R_G < 40 \quad \text{and}$$
$$T_1 = 80°\mathrm{C} \quad \text{for } R_G < 15$$

A more precise determination of the maximum surface temperature can be obtained on the basis of the solar heat flow (depending on the location of the building and the orientation of the sandwich panel) and the coefficient of the solar absorption (α) as measured according to ASTM E903-96 and EN410 1998.

If ventilated curtain walls are placed in front of a sandwich panel wall, T_1 may be reduced to a value of not less than +40 °C (depending on the degree of shading) for both ultimate and serviceability calculations. Depending on the transparency of the curtain wall structure, and the air flow between the outside face and the curtain wall, the temperature of the outside face may either increase or decrease in comparison to the temperature of similar faces without the curtain wall.

In general, the temperature T_2 of the inside face may be taken as +20 °C in Winter and +25 °C in Summer for both ultimate limit state and serviceability limit state calculations.

In special situations, where a defined ambient temperature is maintained (e.g. buildings with air conditioning, cold stores. etc.), T_2 is the ambient operating temperature.

Comment:	Temperatures higher than those given in Section 2.2.2 may be required in Southern Europe. With an adverse combination of such factors as colour, air temperature, altitude and orientation, temperatures in excess of 90 °C are not unusual.

2.2.3 Long-term effects

Shear creep of the typical core layers used in sandwich panels causes additional deflections and, in general, also changes in the distributions of stress resultants in the course of time. Changes in stresses and deflections caused by the shear creep shall be taken into account in design. In principle, the influence of the shear creep has to be considered as an action in both the ultimate and serviceability limit state calculations.

2.2.4 Impact and dynamic actions

In buildings where impact or vibration may occur, these actions should be given special consideration. This is done usually by testing in accordance with National Standards.

Impact loading tests are generally applicable to internal and external wall panels located where they may be exposed to impact loading, for example at ground floor level in buildings accessible to the public. They are intended to simulate the performance of the panels when subject to accidental or intended impacts, for example, objects thrown or kicked against them or accidents involving people or items pushed or falling against them.

2.3 Limit states

2.3.1 Ultimate limit state

The ultimate limit state corresponds to the maximum load carrying capacity and is characterised by the following failure modes, either individually or in combination (Fig. 2.1):

(a) yielding of the face with consequential failure
(b) wrinkling of the face with consequential failure
(c) shear failure of the core
(d) shear failure of a profiled face
(e) crushing of the core or a profiled face at a support or in contact with a line load
(f) failure of the fasteners

(g) failure of the element at the points of attachment to the supporting structure

Modes (b), (c) and (g) may be influenced by considerations of durability.

2.3.2 Serviceability limit state

The verification of serviceability limit states ensures the proper functioning of the elements. The serviceability limit state is characterised by one of the following:

(a) yielding of the face layer at a single point without consequential failure
(b) wrinkling of the face layer at a single point without consequential failure
(c) shear yielding of the core layer at a single point without consequential failure
(d) shear failure of a profiled face
(e) crushing of the core or a profiled face at a support or in contact with a line load
(f) failure of the fasteners
(g) failure of the element at the points of attachment to the supporting structure
(h) the attainment of a specified limiting deflection

For roof panels and ceilings, the deflection caused by the short-term loads should not exceed the value span/200. Correspondingly, the long-term deflection of roof panels and ceilings including the effects of creep should not exceed the value span/100. For wall panels, the deflection should not exceed the value span/100. In special cases there may be other considerations in the design of sandwich panels, which necessitate more stringent deflection limits.

Comments:	The short term deflection denotes the combination in which no creep effects are included. It may include deflections caused by both short-term and long-term loads and, therefore, it represents the initial deflection. The long-term deflection consists of the short term deflection plus the additional deflection caused by the shear creep. It is not correct to associate the short-term deflection with deflection caused by the short-term loads only and the long-term deflection with deflection caused by the long-term loads only. During testing, yielding or wrinkling may be recognised by the presence of residual deflections (see Sections 5.2.8 and 5.2.9).

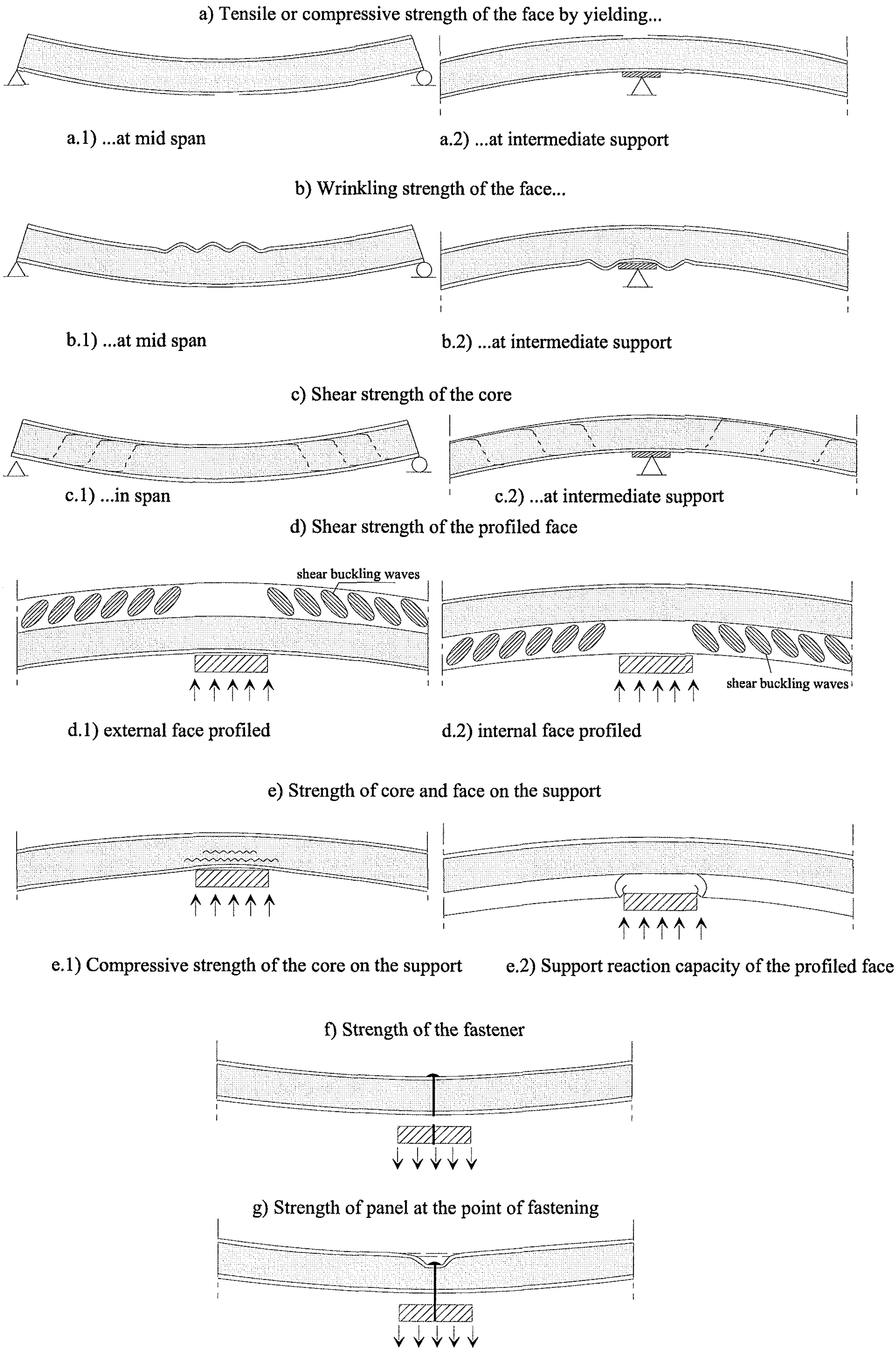


Fig. 2.1 Failure modes in sandwich panels and fastenings.

2.4 Combination rules

2.4.1 Combination of the effects of actions for the ultimate limit state

For each load case, the design value for the effects of actions shall be determined from combination rules involving the effects of separate actions, load factors and combination coefficients. In Eurocode 1 /Eurocode 1 1994/, three combinations of the effects of actions for the ultimate limit state design are given. The combination, as given in Table 2.1 and in equation (2.4), corresponds to the combination of persistent and transient actions. The second and third combinations in Eurocode 1, i.e. the combinations for accidental and for seismic design situations, are not relevant for the sandwich panels typically used to cover walls and roofs in buildings.

Permanent actions G_d	Variable actions Q_d	
(self-weight etc.)	Dominant with its characteristic value	Others with their combination value
$\gamma_G G_k$	$\gamma_{Q1} Q_{k1}$	$\gamma_{Qi} \psi_{0i} Q_{ki}$

Table 2.1 Design values of effects of actions for use when combining actions for the ultimate limit state /Eurocode 1 1994/.

The design values in Table 2.1 shall be combined in the following way:

$$S_d = \gamma_G G_k + \gamma_{Q1} Q_{k1} + \sum_{i>1} \gamma_{Qi} \psi_{0i} Q_{ki} \qquad (2.4)$$

where G_k = characteristic value of the permanent action
Q_{k1} = characteristic value of the dominant variable action
Q_{ki} = characteristic value of the non-dominant variable action i (i>1)
γ_G = partial safety factor for the permanent action
γ_{Qi} = partial safety factor for the variable action i
ψ_{0i} = combination coefficient of a variable action i (Table 2.3)

Comment: The design equation (2.4) is based on a rare combination of actions (See also the comment in section 2.4.2).

2.4.2 Combination of the effects of actions for the serviceability limit states

The combination of actions to be considered for serviceability limit states depends on the nature of the effects of the actions being checked, e.g. irreversible, reversible or long term. In Eurocode 1 /Eurocode 1 1994/, three combinations of the effects of actions for the serviceability limit state design are given. The first and second combinations, designated by the representative value of the dominant action, are given in Table 2.2 and in Equations (2.5) and (2.6). However, the second (frequent) combination has been modified to correspond to the real combination of actions to which typical sandwich panels are exposed. The third combination in Eurocode 1 is the quasi-permanent combination, which is not relevant in the design of sandwich panels. The load factors γ_G and γ_Q are taken as 1.0 except where specified otherwise.

Combination	Permanent actions G_d	Variable actions Q_d	
		Dominant	Others
Characteristic (rare)	G_k	Q_{k1}	$\psi_{0i}\ Q_{ki}$
Frequent	G_k	$\psi_{11}\ Q_{k1}$	$\psi_{0i}\ \psi_{1i}\ Q_{ki}$

Table 2.2 Design values of effects of actions for use when combining actions for serviceability limit states.

a) Characteristic (rare) combination (for resistance at intermediate supports):

$$S_d = \sum_{j\geq 1} G_{kj} + Q_{k1} + \sum_{i>1} \psi_{0i} Q_{ki} \tag{2.5}$$

b) Frequent combination (for deflections):

$$S_d = \sum_{j\geq 1} G_{kj} + \psi_{11} Q_{k1} + \sum_{i>1} \psi_{0i} \psi_{1i} Q_{ki} \tag{2.6}$$

where

ψ_{0i} = combination coefficient of a variable action i (i>1) to be used in characteristic and frequent combinations,

ψ_{11} = combination coefficient of the dominant action effect Q_{k1} to be used in frequent combinations and

ψ_{1i} = combination coefficient of the other action effects Q_{ki} (i>1) to be used in frequent combinations

Other notations are defined in 2.4.1. Values for combination coefficients ψ_{0i} and ψ_{1i} are given in Table 2.3.

Comments:

1. The ultimate limit state is always verified for all realistic combinations of permanent and variable actions (Table 2.1, Equation 2.4). The serviceability limit state is verified for characteristic or frequent combinations (Table 2.2, Equations 2.5 and 2.6).

2. Variable actions are defined on the basis of a statistical evaluation and characteristic values are chosen on the basis of a return period of (say) 50 years. The possibility of two such extreme events occurring simultaneously is remote. Therefore, equation 2.5 applies to these very rare combinations of high values of, for example, wind, snow and temperature. Conversely, equation 2.6 reflects the fact that a less extreme combination of wind, snow and temperature, acting together with permanent load, is a more frequent occurrence.

3. The following variable actions have to be considered for sandwich panels:
   - wind
   - temperature
   - snow (for roofs only)
   - concentrated (access) loads during construction and maintenance (for roofs and ceilings).

   Unless directly evident (as, for example, in the case of snow load for roofs), it is necessary to determine which action has the largest action effect.

4. For verification in the **ultimate limit state**, the dominant action effect shall be multiplied by γ_Q and all other action effects by $\gamma_Q\ \psi_0$. This calculation is always done with the characteristic (rare) combination of actions.

5. For verification in the **serviceability limit state**, bearing in mind the consideration of **deflection**, it is recommended that frequent combinations (Equation 2.6) should be applied for roof and wall panels.

6. When checking the **resistance** at an intermediate support for **the serviceability limit state,** such as **wrinkling** of the compressed face or **shear** of the core or profiled face or **crushing** of a face profile, it is recommended that this limit state should be checked under the characteristic (rare) combination of actions (Equation 2.5).

2.5 Combination coefficients and safety factors

2.5.1 Combination coefficients

Values of the combination coefficients ψ_o and ψ_1 defined in 2.4.1 and 2.4.2 are given in Table 2.3. These values have been modified from the ψ-values given in Eurocode 1 /Eurocode 1 1994/ to take account of the particular requirements for sandwich panels. These may be used in the absence of national regulations.

	Snow	Wind	Temperature
ψ_0	0.6	0.6	0.6 / 1.0 [1)]
ψ_1	0.75 / 1.0 [2)]	0.75 / 1.0 [2)]	1.0

[1)] Coefficient $\psi_0 = 1.0$ is used if the Winter temperature $T_1 = 0°C$ is combined with snow.
[2)] Coefficient $\psi_1 = 0.75$ for snow and wind is used if the combination includes the action effects of two or more variable actions and
Coefficient $\psi_1 = 1.0$ for snow and wind is used if there is, in the combination, only a single action effect representing the variable actions and it is caused by the sole snow load or the sole wind load, alone.

Table 2.3 Values of combination coefficients ψ_0 and ψ_1 (modified from Eurocode 1 1994).

Comment: Because the geographical and meteorological conditions vary considerably within Europe, the values of combination coefficients in section 2.5.1 can only be regarded as indicative values.

The value of the combination coefficient $\psi_1 \leq 1$ is based on the fact, that the characteristic values of snow and wind loads in the design are extreme value loads with a long return period. Because the deflections, if they can arise freely, do not cause any damage to the panel, a lower load than the characteristic snow or wind load can be used in the design for deflections. Where large deflections may cause damage to the sandwich panel or to other parts of the structure, a higher value than $\psi_1 = 0.75$ should be used in the frequent combination.

2.5.2 Load factors

The load factors γ_F are given in Table 2.4. The factor for permanent actions in parenthesis shall be used if the effect of the action is favourable. The values of the load factors given in Table 2.4 may be taken as indicative only and may be used if no relevant values in National Standards are available.

	Ultimate limit state	Serviceability limit state
Permanent actions G	1.35 (1.00)	1.00
Variable actions	1.50	1.00
Creep effects	1.00	1.00

Table 2.4 Load factors γ_F /Eurocode 1 1994/.

Comments: The following expressions are examples of the load combinations for sandwich panels to be used in the verification at the ultimate and serviceability limit states. The expressions have been given in detail in order to illustrate typical combinations which often determine the design of lightweight sandwich panels. The expressions given in the list do not necessarily, especially in the case of sandwich panels with profiled faces, cover all of the combinations which may be needed in the design. The designer has to study all of the combinations which may be critical and determine the potential failure modes for the specific case.

For verification in the **ultimate limit state**, typical design values of action effects for roof panels are (Eq. 2.4)

$S_d = 1.35 \cdot G + 1.50 \cdot (Q_{snow} + Q_{Temp}) + 1.50 \cdot 0.6 \cdot Q_{Wind}$ (Winter, $T_1 = 0$ °C, short-term)

$S_d = 1.35 \cdot G + 1.50 \cdot (Q_{snow} + Q_{Temp}) + 1.50 \cdot 0.6 \cdot Q_{Wind} + 1.00 \cdot (G_{Creep} + Q_{Snow,Creep})$ (Winter, $T_1 = 0$ °C, long-term)

$S_d = 1.0 \cdot G + 1.50 \cdot Q_{Temp} + 1.50 \cdot 0.6 \cdot Q_{Wind}$ (uplift; Summer temperature T_1, temperature dominant)

$S_d = 1.0 \cdot G + 1.50 \cdot Q_{Wind} + 1.50 \cdot 0.6 \cdot Q_{Temp}$ (uplift; Summer temperature T_1, wind dominant)

and for wall panels

$S_d = 1.50 \cdot Q_{Wind} + 1.50 \cdot 0.6 \cdot Q_{Temp}$ (wind dominant)

$S_d = 1.50 \cdot Q_{Temp} + 1.50 \cdot 0.6 \cdot Q_{Wind}$ (temperature dominant)

For verification the **deflection in the serviceability limit state** typical design values of deflections for roof panels are (Eq. 2.6)

$w_d = w_G + 1.0 \cdot w_{Snow}$ (Winter, short-term)

$w_d = (w_G + w_{G,Creep}) + 1.0 \cdot (w_{Snow} + w_{Snow,Creep})$ (Winter, long-term)

$w_d = w_G + 1.0 \cdot w_{Temp}$ (Winter, T_1 = -10 / -20 / -30 °C, short-term)

$w_d = w_G + 0.75 \cdot w_{Snow} + 1.0 \cdot 1.0 \cdot w_{Temp}$ (Winter, $T_1 = 0$ °C, short-term)

$w_d = w_G + 0.75 \cdot w_{Snow} + 1.0 \cdot 1.0 \cdot w_{Temp} + 0.6 \cdot 0.75 \cdot w_{Wind}$ (Winter, $T_1 = 0$ °C, short-term)

$w_d = (w_G + w_{G,Creep}) + 0.75 \cdot (w_{Snow} + w_{Snow,Creep}) + 1.0 \cdot 1.0 \cdot w_{Temp}$ (Winter, $T_1 = 0$ °C, long-term)

$w_d = (w_G + w_{G,Creep}) + 0.75 \cdot (w_{Snow} + w_{Snow,Creep}) + 1.0 \cdot 1.0 \cdot w_{Temp} + 0.6 \cdot 0.75 \cdot w_{Wind}$ (Winter,

$T_l = 0$ °C, long-term)

$w_d = w_G + 1.0 \cdot w_{Temp} + 0.6 \cdot 0.75 \cdot w_{Wind}$ (Summer temperature T_l, temperature dominant)

$w_d = w_G + 0.75 \cdot w_{Wind} + 0.6 \cdot 1.0 \cdot w_{Temp}$ (Summer temperature T_l, wind dominant)

and design values of deflections for wall panels

$w_d = 1.0 \cdot w_{Wind}$ (wind only)

$w_d = 0.75 \cdot w_{Wind} + 0.6 \cdot 1.0 \cdot w_{Temp}$ (wind dominant)

$w_d = 1.0 \cdot w_{Temp} + 0.6 \cdot 0.75 \cdot w_{Wind}$ (temperature dominant)

When checking **wrinkling** (σ_{Fd}) over an intermediate support, typical design values of the compressive stresses in the face of a roof panel are **for the serviceability limit state** the following (Eq. 2.5). Similar combinations may determine the **shear failure of a core** (τ_{Cd}) or of a **profiled face** (τ_{Fd}) at an intermediate support or the **crushing failure of a core** (σ_{Ccd}) or **profiled face** (F_d) on an intermediate support or in contact to a line load for the serviceability limit state.

$S_d = G + Q_{Snow} + 1.0 \cdot Q_{Temp} + 0.6 \cdot Q_{Wind}$ (Winter, $T_l = 0$ °C, short-term)

$S_d = (G + G_{Creep}) + (Q_{Snow} + Q_{Snow,Creep}) + 1.0 \cdot Q_{Temp} + 0.6 \cdot Q_{Wind}$ (Winter, $T_l = 0$ °C, long-term)

$S_d = G + Q_{Temp} + 0.6 \cdot Q_{Wind}$ (Summer temperature T_l, temperature dominant)

$S_d = G + Q_{Wind} + 0.6 \cdot Q_{Temp}$ (Summer temperature T_l, wind dominant)

and of a wall panel

$S_d = Q_{Wind} + 0.6 \cdot Q_{Temp}$ (wind dominant)

$S_d = Q_{Temp} + 0.6 \cdot Q_{Wind}$ (temperature dominant)

In the above expressions, terms with subscripts Snow, Wind and Temp represent the action effects which are caused by the characteristic snow, wind and temperature loads. The terms G,Creep (or Creep) and Snow,Creep describe the action effects caused by the shear creep of the core layer under permanent loads and snow load respectively.

For more details see Ref. / Lightweight sandwich construction 2000/

2.5.3 Material factors

The material safety factors γ_M for ultimate limit state and serviceability limit state verification are given in Table 2.5. These should be taken as indicative only and may be used if no relevant values in National Standards are available.

	Ultimate limit state	Serviceability limit state
yielding of a metal face (clause 4.3.1)	1.1	1.0
wrinkling of a metal face in the span (clause 4.3.2.2)	1.25	1.0
wrinkling of a metal face at an intermediate support (interaction with support reaction, clauses 4.3.2.3 and 4.4.1)	1.25 [1]	1.1
shear of the core (clause 4.3.3)	1.25	1.0
shear failure of a profiled face (clause 4.3.4)	1.1	1.0
crushing of the core (clause 4.3.5)	1.25	1.0
support reaction capacity of a profiled face (clause 4.4.2)	1.1	1.0
failure of a fastener (clause 4.3.6 and Chapter 7)	1.33 [2]	1.0 [2]
failure of an element at a point of connection (fastening) (Chapter 7)	1.33 [2]	1.0 [2]

1. The material factor for wrinkling at the ultimate limit state is needed if the design is based on elastic analysis or if a non-zero bending resistance at intermediate supports is utilized in a design based on plastic analysis.

2. If the characteristic value of the strength of a fastening is not based on a sufficient number of tests for a statistically reliable value to be obtained, higher values of the material safety factors should be used (see 5.2.18).

Table 2.5 Material safety factors γ_M

Note: The total safety factor (load factor times material safety factor) with respect to the failure modes in which the properties of core and bond play an important role should be approximately 2. It is important to bear this in the mind if different material safety factors than those given in Table 2.5 are used /Lightweight sandwich construction 2000/.

3. CALCULATION OF THE EFFECTS OF ACTIONS

3.1 General

The design of sandwich elements consisting of two metal faces and a plastic foam or mineral wool core must be preceded by the determination of the internal stress resultants. This requires calculations in which some unusual features must be considered which do not generally occur with other forms of construction. The most important distinguishing feature is that the core is generally so flexible that the influence of shear deformation cannot be neglected in the determination of the stress resultants. In contrast to the classical bending theory for the calculation of composite sections, the Bernoulli hypothesis of plane sections remaining plane is not valid for the complete cross-section but only for the individual components of the section.

A constant value of the shear modulus of the core, corresponding to an average value at normal indoor temperature, may be used for all basic calculations including the calculation of stress resultants and deflections. The shear modulus shall be based on tests on the particular sandwich panel product (see Section 5.2.4).

It may be assumed that, for the range of deformations to be considered, the materials of the core and faces remain linearly elastic. It can also be assumed that the extensional stiffness of the core is so small in comparison to that of the faces that the influence of longitudinal normal stresses in the core may be neglected. The load bearing capacity of a sandwich panel can then, in general, be divided into two components (Figs. 3.1 and 3.3).

(1) For bending moments:
into a moment component M_F in the metal faces and a moment component M_S (the sandwich part) arising from the normal forces N_{F1} and N_{F2} in the faces multiplied by the lever arm e

(2) For shear forces:
into a shear force component V_F in the faces and a shear force component V_S in the sandwich part of the section.

If the faces of a sandwich panel are thin and flat or they are lightly profiled, the bending stiffness of the faces ($B_{F1} = E_{F1}\, I_{F1}$, $B_{F2} = E_{F2}\, I_{F2}$) is small and has a negligible effect on the stress distributions and deflections of the panel. In that case, the bending stiffness of the faces can be neglected ($B_{F1} = B_{F2} = 0$) in the analysis and the calculations can be based on the stress resultants M_S, N_{F1}, N_{F2}, V_S , only (Figs. 3.1 and 3.2, Eqs. 3.1 and 3.4).

Normal forces N_{F1} and N_{F2} cause a uniform compressive and tensile stress distribution over the external and internal faces, while the bending moments M_{F1} and M_{F2} result in normal stresses which vary linearly over the depths of the faces. Local buckling of a compressed web of a face profile make the normal stress distribution in the face nonlinear. The shear force V_S causes a constant shear stress distribution τ_C over the depth of the core, when the compressive and tensile rigidity of the core layer in the longitudinal direction of the sandwich panel is ignored. The shear forces V_{F1} and V_{F2} cause shear stresses τ_{F1} , τ_{F2} in the face layers with non-vanishing bending rigidity. These shear stresses τ_{F1} , τ_{F2} can be assumed to be a constant over the depths of the webs of the metal face profiles (Figs. 3.2 and 3.4, Eqs. 3.2, 3.3 and 3.5).

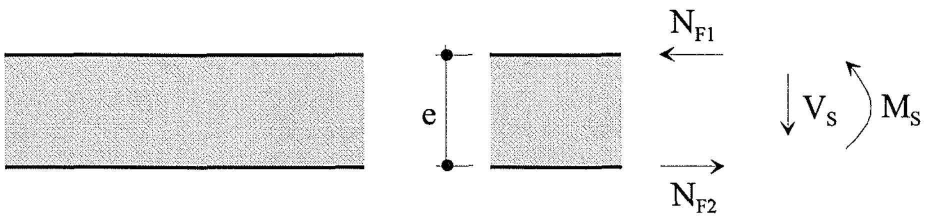


Fig. 3.1 Stress resultants in a thin (flat or lightly profiled) faced sandwich panel. In thin faced panels, the bending stiffness of the faces can be neglected ($B_{F1} = B_{F2} = 0$). The stress resultants in the cross-section are $M = M_S = e\,N_{F1} = e\,N_{F2}$ and $V = V_S$.

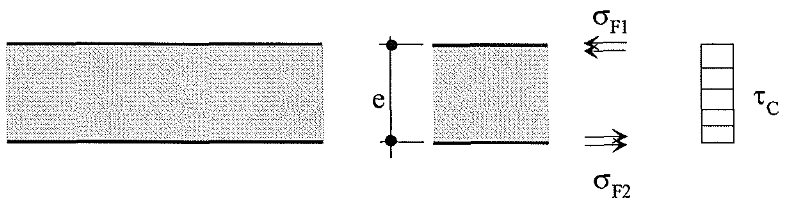


Fig. 3.2 Stress distribution over the cross-section in a thin faced sandwich panel.

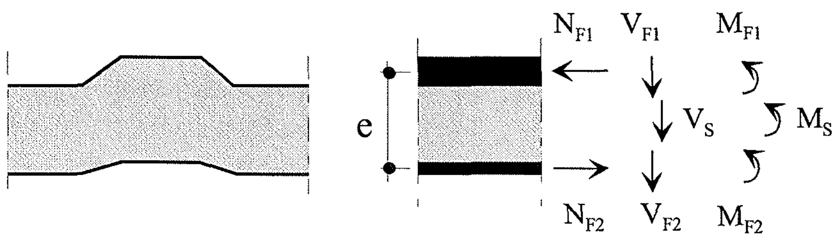


Fig. 3.3 Stress resultants in a thick faced sandwich panel. In thick faced panels, the bending stiffness of the faces can not be neglected ($B_{F1} + B_{F2} \neq 0$). The stress resultants in the cross-section are $M = M_S + M_{F1} + M_{F2}$ and $V = V_S + V_{F1} + V_{F2}$.

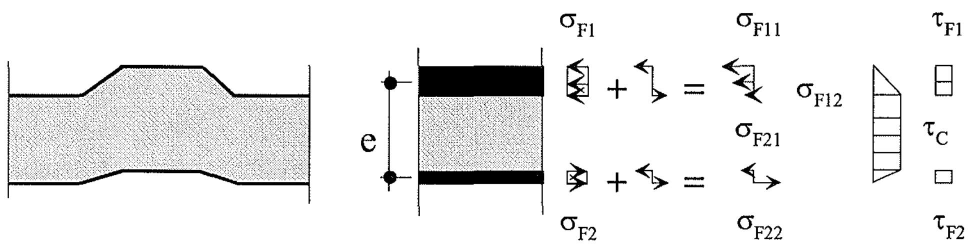


Fig. 3.4 Stress distribution over the cross-section in a thick faced sandwich panel.

$$\sigma_{F1} = -\frac{N_{F1}}{A_{F1}} = -\frac{M_S}{eA_{F1}} \;,\quad \sigma_{F2} = \frac{N_{F2}}{A_{F2}} = \frac{M_S}{eA_{F2}} \qquad (3.1a,b)$$

$$\sigma_{F11} = \sigma_{F1} - \frac{M_{F1}}{I_{F1}} d_{11} \;,\quad \sigma_{F12} = \sigma_{F1} + \frac{M_{F1}}{I_{F1}} d_{12} \qquad (3.2a,b)$$

$$\sigma_{F21} = \sigma_{F2} - \frac{M_{F2}}{I_{F2}} d_{21} \ , \ \sigma_{F22} = \sigma_{F2} + \frac{M_{F2}}{I_{F2}} d_{22} \qquad (3.3a,b)$$

$$\tau_C = \frac{V_S}{eB} \qquad (3.4)$$

$$\tau_{F1} = \frac{V_{F1}}{n_1 \, s_{w1} \, t_1} \ , \ \tau_{F2} = \frac{V_{F2}}{n_2 \, s_{w2} \, t_2} \qquad (3.5a,b)$$

In the above expressions, A_{F1} and A_{F2} are areas of the cross-sections of the faces, I_{F1}, I_{F2} second moments of the areas of the faces, $s_{w1} = d_1/\sin(\phi_1)$ and $s_{w2} = d_2/\sin(\phi_2)$ lengths of the webs of the profiled faces and n_1 and n_2 numbers of the webs in the profiled faces in the panel width B. Other symbols are introduced in Figs. 1.1 and 3.1 - 3.4.

Based on the structural properties, static system, span widths and the loads of the panel, the stress resultants M_S, M_{F1}, M_{F2}, N_{F1}, N_{F2}, V_S, V_{F1} and V_{F2}, and deflections can be determined using analytical or numerical methods. The basis of the available methods is discussed in Appendix A. In clauses 3.4 and 3.5, results for some loading cases are given based analytical and approximate methods. Further information concerning the analytical, numerical and approximate methods and solutions for different static systems and loading cases are given in the references.

<u>Comment:</u> Solutions to simply supported thin and thick faced sandwich beams exposed to external and internal loads can be found in references */Allen 1972, Stamm & Witte 1974, Hartsock & Chong 1976/*. Approximate solutions based on simplified theory of sandwich beams are presented for instance in reference */Wölfel 1987/*. Numerical methods and results for more complicated loading cases based on general and exact finite elements are introduced in references */Schwartze 1984, Davies 1986, Heinisuo 1988/*. Numerical methods are especially used to find solutions for continuous multi-span sandwich beams with thick (profiled) faces. The static behaviour of thin and thick faced sandwich panels can be simulated using finite element programmes developed for general purposes. Many modern finite element programmes contain 3-d shell elements, which include deformations due to membrane, bending and shear effects. These enable the analysis thin faced sandwich beams, plates and shells to be carried out. In references */Berner 1998 and Lightweight sandwich construction 2000/* additional tables and graphs are given with which to determine the stress resultants and deflections for some typical static systems and load cases for practical design.

3.2 Static system, geometry and thickness

The static system used in the calculation of sandwich panels shall be in accordance with the number and location of supports in the practical application for both pressure and uplift loads. The lengths of spans are generally determined as being the distances between the mid-lines of the supports. Sandwich panels are usually assumed to rotate and to move axially on the supports without restraint, thus corresponding to 'simple' support conditions between the sandwich panel and the support. If partial or full rigidity against the rotation at supports is utilized in design calculations, the validity of the assumption shall be verified experimentally.

Dimensions which are of significance for the static behaviour and resistance, such as the depth and width and the dimensions of the face profiles, shall correspond to the actual dimensions of the sandwich panel product in question. If nominal dimensions are used in calculations, the real dimensions shall agree with the dimensions used in the calculations within close tolerances.

The thickness of the metal sheet in the faces has a dominant influence on the bending stiffness and resistance of the sandwich panel. It is recommended that the design thickness of the steel sheet is taken as $t_d = t_{nom} - t_{zinc} - 0.5\ t_{tol}$, where t_{nom} is the nominal thickness of the steel sheet, t_{zinc} the total thickness of the two zinc layers and t_{tol} the normal minus tolerance according to EN10143. The design thickness of other metal facing sheets, such as those made of aluminium, stainless steel or copper shall be determined so that they represent statistically reliable minimum thickness values. For these materials the design thickness is recommended to be taken as $t_d = t_{nom} - 0.5\ t_{tol}$. In all equations in this document, the design thickness is denoted by t.

Notes:

1. EN485-4 "Aluminium alloys - sheet strip and plate-Part 4: Tolerances on shape and dimensions for cold-formed products" gives values of the thickness tolerances for aluminium sheets and EN10259 "Cold-rolled stainless steel wide strip and plate/sheet - Tolerances on dimensions and shape" for stainless steel sheets.

2. Other tolerances on geometry etc. are given in the draft CEN standard for sandwich panels, prENxxxx (no standard number available at the time of drafting these Recommendations)

3.3 Methods of analysis

The following methods of analysis may be used:

(a) elastic analysis
(b) plastic analysis

Elastic analysis should always be used for the serviceability limit state and may be used for the ultimate limit state. Plastic analysis is only applicable to the ultimate limit state and may be used whenever the design is controlled by bending stresses at an internal support. Plastic analysis should not be used when the first failure mode is a shear failure of the core, unless the core material has adequate plastic shear capacity (Section 4.4.3).

3.3.1 Elastic analysis

The action effects S (bending moments, normal and shear forces) resulting from the combination of all actions applied to the sandwich panels may be found by using the theory of elasticity taking into account of the shear flexibility of the core material. References to suitable methods of analysis are given in Chapter 8.

Equation (2.1) shall be satisfied along the whole length of the structural system.

3.3.2 Plastic analysis

The bending moment distribution at the ultimate state in a continuous sandwich element may be chosen arbitrarily, provided that the internal stress resultants are in equilibrium with the actions, which shall be equal to or higher than the most unfavourable combination of factored actions, and that the internal stress resultants nowhere exceed the plastic resistance of the cross-section.

In plastic analysis calculations at the ultimate limit state, a continuous multi-span sandwich panel may be replaced by a series of simply supported panels with zero bending resistance at intermediate supports. In this calculation model, stresses caused by the temperature difference between the faces vanish in sandwich panels with flat or lightly profiled faces.

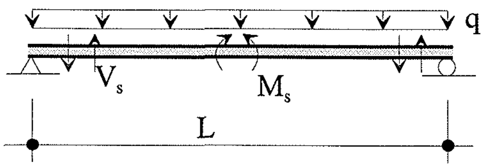
If the inelastic moment of resistance of the sandwich panel at the relevant rotation is known, the bending moments at the internal supports may be chosen to be equal to this inelastic moment of resistance with a material factor according to Table 2.5. Otherwise, hinges of zero bending moment capacity should be assumed at the supports.

3.4 Sandwich panels with plane or lightly profiled faces

In sandwich panels with flat faces or with faces which are only lightly profiled, the bending stiffness of the faces can be neglected in comparison with the bending stiffness of the sandwich part of the cross-section. No division of the global stress resultants into components is therefore necessary. The total bending moment is carried by normal forces in the faces and the total shear force by shear stresses in the core.

3.4.1 Single span panels

The static behaviour of single span sandwich panels is illustrated by the determination of stress resultants and deflections caused by a uniformly distributed load and a temperature difference (stress resultants per unit width).



$$M_{F1} = M_{F2} = 0, \; V_{F1} = V_{F2} = 0$$

$$M_S = \frac{qL^2}{8}, \; V_S = \frac{qL}{2}$$

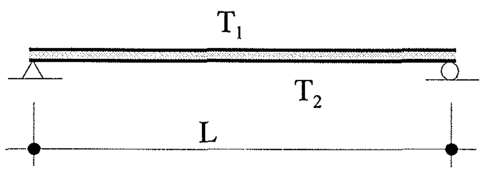
In the determination of the deformations, however, the shear deformation of the core must be taken into account. The deflection at mid-span under load q is

$$w = \frac{5}{384}\frac{qL^4}{B_S}\left(1 + \frac{16}{5}k\right)$$

where $k = \dfrac{3B_S}{L^2 G_C A_S}$, $\quad B_S = \dfrac{E_{F1}A_{F1}E_{F2}A_{F2}}{E_{F1}A_{F1} + E_{F2}A_{F2}}\dfrac{e^2}{B}$, $\quad A_S = e$

G_C is the shear modulus of the core material. The flexural rigidity B_S and the area of the core A_S are given per unit width. In the expression for B_s the parameter B denotes the width of the panel.

An unequal temperature extension in the faces does not give rise to any additional stresses.



$$M_{F1} = M_{F2} = 0, \; V_{F1} = V_{F2} = 0$$

$$M_S = 0, \; V_S = 0$$

Under a temperature difference $\Delta T = T_2 - T_1$, the deflection at mid-span is

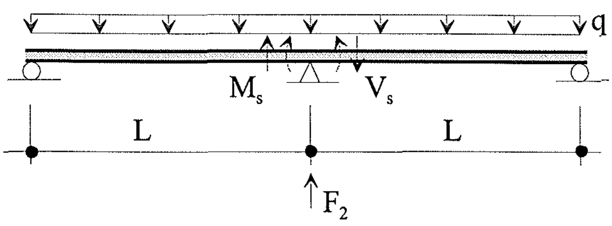
$$w = \frac{\theta L^2}{8}$$

where $\theta = \dfrac{\alpha_{F2} T_2 - \alpha_{F1} T_1}{e}$

3.4.2 Continuous multi-span panels

With continuous sandwich panels (multi-span panels), the shear flexibility of the core gives rise to smaller moments at the internal supports than would arise with a shear-stiff connection between the faces. The unit load method of elastic analysis gives rise to the following equations.

The static behaviour of continuous sandwich panels is illustrated by the determination of the bending moment, support reaction and shear force at mid-support and the deflection in the spans caused by a uniformly distributed load and a temperature difference on a continuous two-span sandwich panel (stress resultants per unit width).



$$M_{F1} = M_{F2} = 0,\ V_{F1} = V_{F2} = 0$$

$$M_S = -\frac{qL^2}{8}\frac{1}{1+k},$$

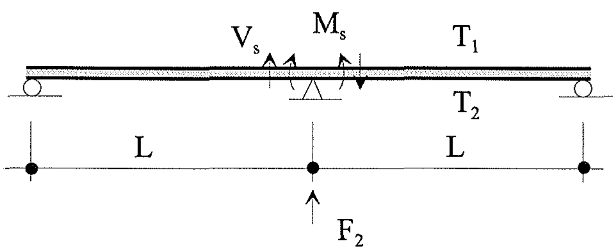
$$F_2 = qL\left(1+\frac{1}{4(1+k)}\right),$$

$$V_S = \pm\frac{qL}{2}\left(1+\frac{1}{4(1+k)}\right)$$

The point of the maximum deflection depends on the shear flexibility (parameter k) and varies between $x_{max} = 0.375$ L and 0.5 L for two-span sandwich panels with equal span lengths. The following expression gives an approximate upper bound value for deflection.

$$w = \frac{1}{48}\frac{qL^4}{B_S}\frac{0.26+2.625k+2k^2}{1+k}$$

In these expressions $k = \dfrac{3B_S}{L^2 G_C A_S}$



$$M_{F1} = M_{F2} = 0,\ V_{F1} = V_{F2} = 0$$

$$M_S = -\frac{3B_S\theta}{2}\frac{1}{1+k},$$

$$F_2 = \frac{3B_S\theta}{L}\frac{1}{1+k},$$

$$V_S = \pm\frac{3B_S\theta}{2L}\frac{1}{1+k}$$

An approximate upper bound value for deflections at mid-span (x = L/2) of a two-span sandwich panels with equal spans is

$$w = \frac{\theta L^2}{32}\frac{1.089+3.96k}{1+k}$$

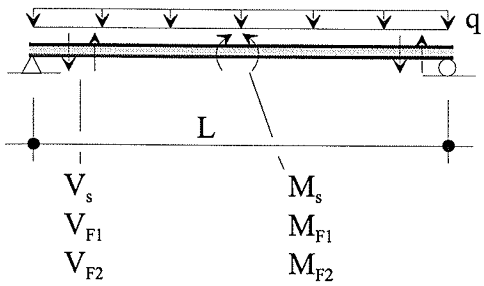
where θ is defined in section 3.4.1 above. $k = \dfrac{3B_S}{L^2 G_C A_S}$

3.5 Sandwich panels with strongly profiled faces

When the bending stiffness of a face in a sandwich panel cannot be neglected, the panel is itself statically indeterminate in addition to any global structural indeterminacy that may be present. Explicit solutions are given in the references for a few simple cases but, in general, numerical methods of analysis, e.g. the finite element method, are required.

3.5.1 Single span panels

The exact analytical solution and an approximate solution introduced in the reference /Wölfel 1987/ for a simply supported sandwich beam with strongly profiled faces or with faces having large material thickness and loaded by an uniformly distributed load are given in the following table. The stress resultants are defined per unit width.



Exact solution	Approximate solution
$M_S = \frac{qL^2}{8}\frac{1}{1+\alpha}\left[1-\frac{8}{\lambda^2}f_1(\lambda)\right]$	$M_S = (1-\beta_q)\frac{qL^2}{8}$
$M_{F1} = \frac{qL^2}{8}\frac{\alpha_1}{1+\alpha}\left[1+8\frac{1}{\alpha\lambda^2}f_1(\lambda)\right]$	$M_{F1} = \beta_1\beta_q\frac{qL^2}{8}$
$M_{F2} = \frac{qL^2}{8}\frac{\alpha_2}{1+\alpha}\left[1+8\frac{1}{\alpha\lambda^2}f_1(\lambda)\right]$	$M_{F2} = \beta_2\beta_q\frac{qL^2}{8}$
$V_S = \pm\frac{qL}{2}\frac{1}{1+\alpha}\left(1-\frac{2}{\lambda}f_2(\lambda)\right)$	$V_S = \frac{qL}{2}$
$V_{F1} = \pm\frac{qL}{2}\frac{\alpha_1}{1+\alpha}\left(1+\frac{2}{\alpha\lambda}f_2(\lambda)\right)$	
$V_{F2} = \pm\frac{qL}{2}\frac{\alpha_2}{1+\alpha}\left(1+\frac{2}{\alpha\lambda}f_2(\lambda)\right)$	
$w = \frac{qL^4}{B}\left[\frac{5}{384}+\frac{1}{8\alpha\lambda^2}\left(1-\frac{8}{\lambda^2}f_1(\lambda)\right)\right]$	$w = \frac{5}{384}\frac{qL^4}{B_S}\left(1-k_q\right)\left(1-\beta_q\right)$
Where	Where
$\alpha_1 = \frac{B_{F1}}{B_S} \quad \alpha_2 = \frac{B_{F2}}{B_S}$	$\beta_1 = \frac{B_{F1}}{B_{F1}+B_{F2}} \quad \beta_2 = \frac{B_{F2}}{B_{F1}+B_{F2}}$
$\alpha = \alpha_1 + \alpha_2 = \frac{B_{F1}+B_{F2}}{B_S}$	$\beta_q = \frac{B_{F1}+B_{F2}}{B_{F1}+B_{F2}+\frac{B_S}{1+k_q}}$
$\lambda = L\sqrt{\frac{B}{B_{F1}+B_{F2}}\frac{G_C A_S}{B_S}}$	$k_q = \frac{9.6 B_S}{L^2 G_C A_S}$
and	
$f_1(\lambda) = 1-\frac{1}{\cosh(\lambda/2)} \quad f_2(\lambda) = \tanh(\lambda/2)$	

In the above expressions

$B = B_{F1} + B_{F2} + B_S$ the flexural rigidity of the panel

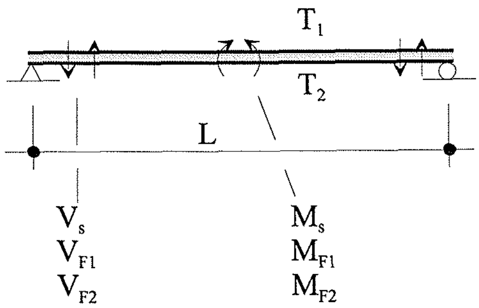
$B_S = \frac{E_{F1}A_{F1}E_{F2}A_{F2}}{E_{F1}A_{F1}+E_{F2}A_{F2}}\frac{e^2}{B}$ the bending stiffness of the sandwich part of the cross-section.

In the expression for B_S, B is the overall width of the panel

$B_{F1} = E_{F1} I_{F1}$ and $B_{F2} = E_{F2} I_{F2}$ are the bending stiffnesses of the upper and lower face, respectively. G_C is the shear modulus and $A_S = e$ the area of the core. The structural properties are defined per unit width.

The approximate solutions have been developed for manual calculations for typical loads and spans. The face stresses given by the above approximate expressions are sufficiently accurate for practical purposes. The approximate expression for the shear force V_S is less accurate.

The exact analytical solution and an approximate solution presented in the reference /Wölfel 1987/ for a simply supported sandwich beam with strongly profiled faces or with faces having large material thickness, exposed to an unequal temperature extension in the faces, are given in the following table. The stress resultants are defined per unit width.



Exact solutions	Approximate solutions
$M_S = -\dfrac{\alpha\,\theta\,B_S}{1+\alpha} f_1(\lambda)$	$M_S = -(1-\beta_T)(B_{F1}+B_{F2})\theta$
$M_{F1} = \dfrac{\alpha_1\,\theta\,B_S}{1+\alpha} f_1(\lambda)$	$M_{F1} = \beta_1\left(1-\beta_T\right)\left(B_{F1}+B_{F2}\right)\theta$
$M_{F2} = \dfrac{\alpha_2\,\theta\,B_S}{1+\alpha} f_1(\varepsilon)$	$M_{F2} = \beta_2\left(1-\beta_T\right)\left(B_{F1}+B_{F2}\right)\theta$
$V_S = \mp\dfrac{\theta\,B_S}{\alpha_S\,\lambda\,L} f_2(\lambda)$	
$V_{F1} = \pm\dfrac{\alpha_1\,\theta\,B_S}{\alpha\,\alpha_S\,\lambda\,L} f_2(\lambda)$	
$V_{F2} = \pm\dfrac{\alpha_2\,\theta\,B_S}{\alpha\,\alpha_S\,\lambda\,L} f_2(\lambda)$	
$w = \dfrac{\theta L^2}{8}\dfrac{1}{1+\alpha}\left[1-\dfrac{8}{\lambda^2} f_1(\lambda)\right]$	$w = \dfrac{\theta L^2}{8}\left(1-\beta_T\right)$
Where	where
$\alpha_1 = \dfrac{B_{F1}}{B_S} \quad \alpha_2 = \dfrac{B_{F2}}{B_S}$	$\beta_1 = \dfrac{B_{F1}}{B_{F1}+B_{F2}} \quad \beta_2 = \dfrac{B_{F2}}{B_{F1}+B_{F2}}$
$\alpha = \alpha_1 + \alpha_2 = \dfrac{B_{F1}+B_{F2}}{B_S} \quad \alpha_S = \dfrac{B_S}{L^2 G_C A_S}$	$\beta_T = \dfrac{B_{F1}+B_{F2}}{B_{F1}+B_{F2}+\dfrac{B_S}{1+k_T}}$
$\lambda = L\sqrt{\dfrac{B}{B_{F1}+B_{F2}}\dfrac{GA_S}{B_S}}$	$k_T = \dfrac{8B_S}{L^2 G_C A_S}$
and	
$f_1(\lambda) = 1-\dfrac{1}{\cosh\left(\lambda/2\right)} \quad f_2(\lambda) = \tanh\left(\lambda/2\right)$	
$B = B_S + B_{F1} + B_{F2}$ is the flexural rigidity of the panel	

The term θ is defined in Section 3.4.1 above.

The approximate solutions have been developed for manual calculations for typical loads and spans. The face stresses given by the above approximate expressions are sufficiently accurate for practical purposes. The approximate method does not give solutions for the shear forces in the core and faces.

3.5.2 Continuous multi-span panels

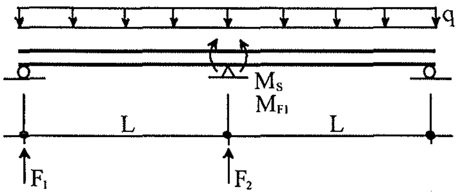
This section describes how multi-span thick-faced sandwich panels may be designed by calculation. However, more favourable results may be obtained by designing on the basis of tests as described in Chapter 6.

The stress resultants and deflections of continuous thick faced sandwich beams can be determined analytically for the most important simple cases, as shown in the following examples. However, in many cases (e. g. panels with unequal spans) the expressions become relatively complicated and require either the design charts introduced in the reference /Berner 1998/ or computer software to find numerical solutions for practical design.

The static behaviour of continuous sandwich panels is illustrated by the determination of the bending moment, support reaction and shear force at mid-support caused by a uniformly distributed load and a temperature difference between the faces on a continuous two-span panel (stress resultants per unit width).

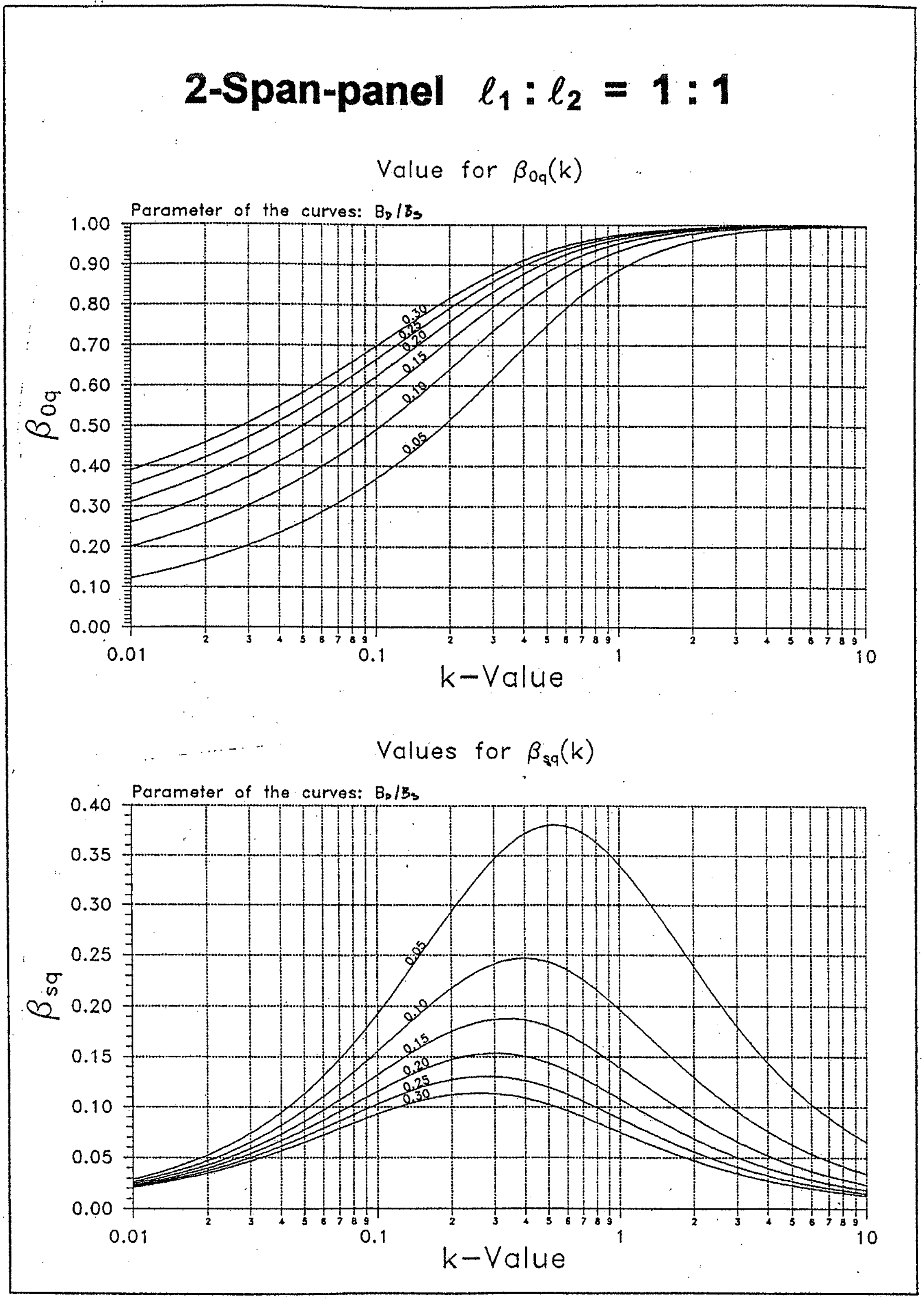
The exact analytical solution /Stamm 1984/ together with an approximate solution /Berner 1998/ for two-span panels in which one face is strongly profiled or has large material thickness [the upper face (No. 1) in this example] and which loaded by a uniformly distributed load, is given in the following table.

<u>Comment:</u>	The graphs have been produced for manual calculations. The graphs themselves are exact. The only approximation is in transforming the graphical information into the numerical form.



Exact solution	Approximate solution
$M_S = -\varepsilon_3 q L^2$	$M_S = \dfrac{(1-\beta_{0q})}{1+\beta_{Sq}} M$
$M_{F1} = -\varepsilon_{4,1} q L^2$	$M_{F1} = \dfrac{\beta_{0q}}{1+\beta_{Sq}} M$
$F_2 = \varepsilon_1 q L \;,\; F_1 = \varepsilon_2 q L$	
Where	where
$\varepsilon_1 = \dfrac{5(1+\alpha)+12\beta\left(1-2\dfrac{\cosh(\lambda)-1}{\lambda^2\cosh(\lambda)}\right)}{4(1+\alpha)+12\beta\left(1-\dfrac{\tanh(\lambda)}{\lambda}\right)}$	$M = -\dfrac{qL^2}{8}$ Values for β_{0q} and β_{Sq} are given in the graphs in Fig. 3.5.
$\varepsilon_2 = 1-\dfrac{\varepsilon_1}{2}$	
$\varepsilon_3 = \dfrac{1}{1+\alpha}\left(\left(\dfrac{\varepsilon_1}{2}\right)\left(1-\dfrac{\tanh(\lambda)}{\lambda}\right)-\dfrac{1}{2}+\dfrac{\cosh(\lambda)-1}{\lambda^2\cosh(\lambda)}\right)$	
$\varepsilon_{4,1} = \dfrac{\alpha_1}{1+\alpha}\left(\left(\left(\dfrac{\varepsilon_1}{2}\right)\left(1+\dfrac{\tanh(\lambda)}{\alpha\lambda}\right)-\dfrac{1}{2}\right)-\dfrac{\cosh(\lambda)-1}{\alpha\lambda^2\cosh(\lambda)}\right)$	
$\alpha_1 = \dfrac{B_{F1}}{B_S} \quad \alpha_2 = \dfrac{B_{F2}}{B_S} \quad \alpha = \alpha_1+\alpha_2 = \dfrac{B_{F1}+B_{F2}}{B_S}$	In graphs $k = \dfrac{B_S}{SL^2}$
$\beta = \dfrac{B_S}{SL^2} \quad \lambda = \sqrt{\dfrac{1+\alpha}{\alpha\beta}}$	$B_D = B_{F1}+B_{F2} = E_{F1}I_{F1}+E_{F2}I_{F2}$

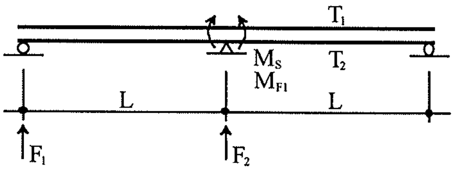
In the above expressions
$B_{F1} = E_{F1}\, I_{F1}$ is the bending stiffness of the upper face (face No. 1)
$B_S = \dfrac{E_{F1}A_{F1}E_{F2}A_{F2}}{E_{F1}A_{F1}+E_{F2}A_{F2}}\dfrac{e^2}{B}$ is the bending stiffness of the sandwich part of the cross-section
$S = G_C\, A_S$, G_C is the shear modulus and
$A_S = e$ the area of the core.
B is the overall width of the panel in the expression for B_s
The structural properties are defined per unit width.



Diagrams for 2-Span-panels with live load

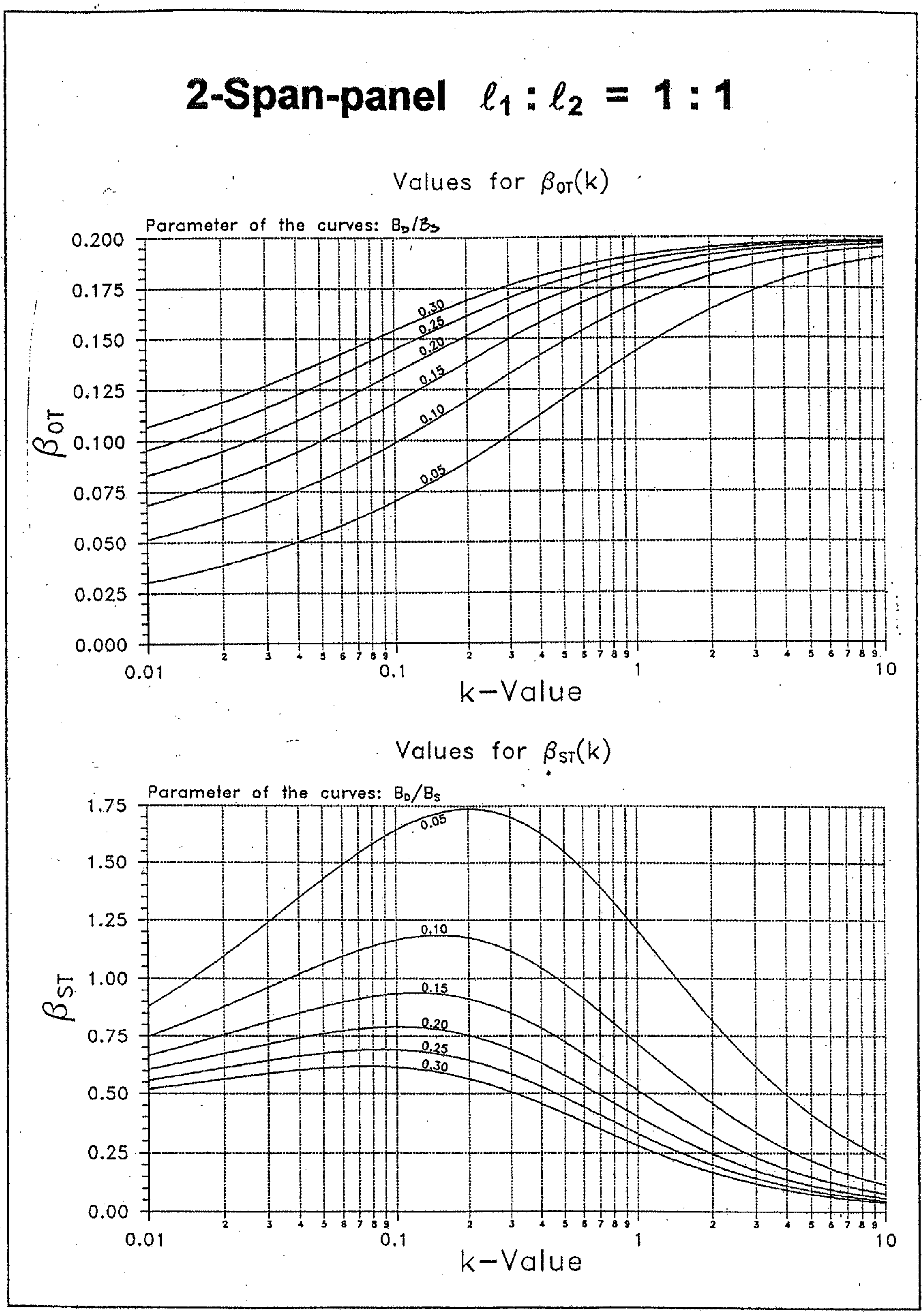
Fig. 3.5 Parameters β_{0q} and β_{Sq} for two-span sandwich panels with equal span lengths.

The exact analytical solution /Stamm 1984/ and an approximate solution /Berner 1998/ for two span sandwich panels with one strongly profiled face or one face with large material thickness (in the example the face No. 1), exposed to an unequal temperature extension in the faces, is:



Exact solutions	Approximate solutions
$M_S = -\varepsilon_7\, \theta\, B_S$ $M_{F1} = \varepsilon_{8,1}\, \theta\, B_S$ $F_2 = \varepsilon_5 \dfrac{\theta B_S}{L}$ $F_1 = \varepsilon_6 \dfrac{\theta B_S}{L}$ Where $\varepsilon_5 = \dfrac{3(1+\alpha)\left(1-2\dfrac{\cosh(\lambda)-1}{\lambda^2\cosh(\lambda)}\right)}{1+\alpha+3\beta\left(1-\dfrac{\tanh(\lambda)}{\lambda}\right)}$ $\varepsilon_6 = -\dfrac{\varepsilon_5}{2}$ $\varepsilon_7 = \dfrac{1}{1+\alpha}\left(\alpha\dfrac{\cosh(\lambda)-1}{\cosh(\lambda)} - \left(\dfrac{\varepsilon_5}{2}\right)\left(1-\dfrac{\tanh(\lambda)}{\lambda}\right)\right)$ $\varepsilon_{8,1} = \dfrac{\alpha_1}{1+\alpha}\left(\dfrac{1-\cosh(\lambda)}{\cosh(\lambda)} - \left(\dfrac{\varepsilon_5}{2}\right)\left(1+\dfrac{\tanh(\lambda)}{\alpha\lambda}\right)\right)$ parameters α_1 , α and λ are defined in section 3.5.2 above	$M_S = \left(\dfrac{1}{\beta_{0T}} - 1\right) M_{F1}$ $M_{F1} = -\beta_{ST}\, \theta\, B_D$ where $B_D = B_{F1}$ Values for β_{0T} and β_{ST} are given in the graphs in Fig. 3.6 In graphs $k = \dfrac{B_S}{S L^2}$ B_D and B_s and S as in the previous table

The parameter θ is defined in section 3.4.1 above.



Diagrams for 2-Span-panels with temperature load

Fig. 3.6 Parameters β_{0T} and β_{ST} for two-span sandwich panels with equal span lengths.

3.6 The influence of time on shear deformations of the core

Typical core materials, especially the plastic foams, are visco-elastic materials in which the deformations increase in the course of time even if the loads remain constant. In the core, long-term loading causes shear creep which may be considered as a reduction in the shear modulus G_C of the core.

In general, the reduced value of the shear modulus G_{Ct} should be determined for a time period of 2000 hours for snow load in Central Europe and 100000 hours for permanent actions (dead load). In the absence of test results, G_{Ct} may be taken as

$$G_{Ct} = \frac{G_C}{1+\varphi_t} \tag{3.6}$$

where φ_t = creep coefficient.

For rigid plastic foams (PUR, EPS, XPS)

$$\varphi_t = \begin{array}{l} 2.4 \text{ for } t = 2000 \text{ hours} \\ 7.0 \text{ for } t = 100000 \text{ hours} \end{array}$$

For mineral wool

$$\varphi_t = \begin{array}{l} 1.0 \text{ for } t = 2000 \text{ hours} \\ 2.0 \text{ for } t = 100000 \text{ hours.} \end{array}$$

More favourable values of the creep coefficient are likely to be determined by test and a testing procedure for shear creep is given in Chapter 5. The evaluation of the test results is described in Appendix B. The influence of the shear creep on the stress resultants is studied in Fig. A4 in Appendix A. If φ_t is less than 0.5, creep effects may be neglected in thin faced sandwich panels, i.e., in panels with flat or micro or lightly profiled faces.

Comments:

1. For snow load in central Europe the determination of φ_t is based on a time period of 2000 hours. Alternative values may be appropriate for other climates where snow may persist for greater or lesser periods.
2. Equation (3.6) is valid if the shear stresses causing creep are constant with time and do not exceed 30% of the ultimate characteristic shear strength. In most cases, creep causes a change in the stresses with time and this should be taken into consideration in the calculations.

4. EVALUATION OF RESISTANCES

4.1 General

The evaluation of the resistance of sandwich panels against different failure modes may be based on rational analysis in accordance with sections 4.3 and 4.4 in combination with testing according to Chapter 5.

Comments:	For the core materials which were in common use at the time of writing this standard (polyurethane, expanded and extruded polystyrene and mineral wool), it is appropriate to make the maximum use of rational analysis and to test only to determine the relevant properties of the core. The available methods of analysis may or may not be applicable to panels with other core materials. At time of drafting these Recommendations, practical experience of phenolic foam cores was limited. The design of panels utilising this material should therefore be approached with caution until more information is available. Particular caution should be exercised with regard to roofs or ceilings subject to foot traffic or other forms of repeated impact load.

Due regard shall be given to temperature effects and, where relevant, the reduction of shear strength, creep and degradation under long term loading. However, a consistent value of the shear modulus of the core, corresponding to an average value at normal indoor temperature, may be used for all global analysis, including the calculation of stress resultants and deflections. The effect of temperature on the characteristic values of material properties need only be taken into account:

a) In the determination of the wrinkling stress

b) When considering the shear failure of the core

4.1.1 Durability

The assessment of durability takes into account the long-term reduction in the strength of the sandwich element. For the purposes of this assessment, durability is defined in terms of the loss of strength properties as the result of accelerated degradation. The critical factors are considered to be the loss of strength in the core itself and its bond with the faces. The "degradation factor" is defined as the ratio of the tensile strength according to section 5.2.2 after and before accelerated ageing. A suitable procedure for accelerated ageing is given in sections 5.1.3 and 5.1.4. Figure 4.1 illustrates the possible degradation of the strength of sandwich panels with time.

The durability of the strength of sandwich panels is affected by variations in moisture and temperature. The critical layers are the core itself, the bonding layers between the core and the reverse side of the facings and the reverse side layer (primer layer) of the facings.

If the sandwich panels fulfil the durability requirements given in section 5.1.4.4, the panels may be used both internally and externally as wall or roof elements with no restrictions.

The tensile bond strength, defined as the characteristic tensile strength value of the unaged core and bond on the total cross-section, shall be at least 0.075 N/mm^2. A testing procedure to determine the tensile strength of the core and bond is given in Section 5.2.2.

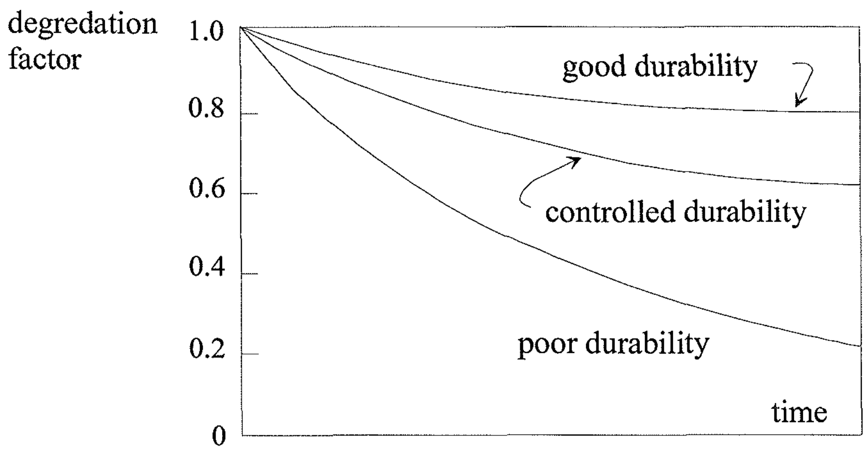


Fig. 4.1 Change of degradation factor during the service life of sandwich panel.

Comment:	When determining the degradation of the strength properties of a sandwich panel, the change in tensile strength perpendicular to the faces is chosen to be the reference property. This is because of the fundamental importance of the bond strength for good performance of the sandwich panel and because it is relatively easy to carry out the necessary testing. The test regime described in section 5.1.4 allows the durability to be controlled on this basis Another possibility is to check the change of the shear strength.

4.1.2 Particular requirements for panels formed using discrete core materials

4.1.2.1 Shear strength

When determining the shear strength of panels formed using discrete core materials with transverse joints in the core, particular account should be taken of the most unfavourable position of joints between the core elements.

Comment:	The weakest point in the core made of discrete core materials is usually the transverse joint between the adjacent core elements. In order to compensate for the weakening effect, discrete core materials can either be provided with longitudinally staggered joints or elements may be glued together at the transverse joints. Unless they are adequately strengthened, no transverse joint or line of joints should extend across more than one quarter of the width of the panel.

4.1.2.2 Wrinkling stress

With panels formed using discrete core materials with transverse joints in the core, the wrinkling stress should be determined by tests for which the panels are chosen to have the most unfavourable positioning of joints between the individual core elements within the zone of maximum bending moment. Attention shall be given to the thickness tolerances between adjacent core elements.

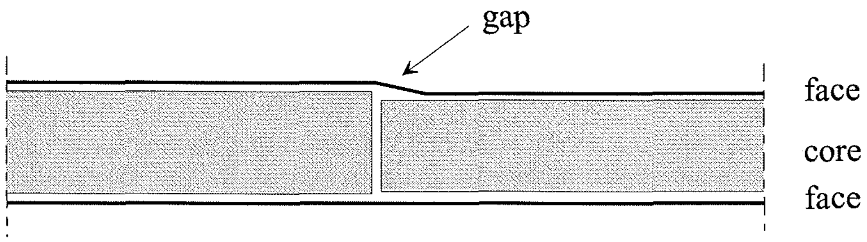


Fig. 4.2 Thickness tolerances between adjacent core elements.

> Comment: The wrinkling stress may be adversely affected by any gap between the core and the faces caused by unequal thickness or poor positioning of individual core elements as shown in Fig. 4.2.
>
> The test specimens should be chosen to reflect the worst situation likely to arise during normal production. Attention should also be given to this factor during the quality control (See Section 5.3).

4.2 Geometry and thicknesses

Definitions of the dimensions of the cross-section to be used in the evaluation of the resistance of sandwich panels are given in Section 3.2.

A definition of the thickness of the metal sheet in faces to be used in the evaluation of the resistance on the sandwich panel is given Section 3.2.

4.3 Resistance at the ultimate limit state

The appropriate methods of analysis and the conditions under which they may be used are described in Section 3.3. If elastic analysis is used, the ultimate limit state corresponds to the first attainment of one of the conditions given in section 2.3.1.

If plastic analysis is used, the ultimate limit state may be defined in terms of yield or wrinkling of the faces within the span after plastic hinge action at the internal supports.

The resistances for the potential failure modes limiting the load-bearing capacity of sandwich panels at the ultimate limit state are specified in 4.3.

4.3.1 Yielding in a face

The yield stress f_y of the face material shall be taken as the guaranteed minimum value for the metal quality used according to the appropriate standard (e.g. EN10147). Alternatively, it may be determined by testing according to section 5.2.1.

The tensile and compressive stress in the face shall satisfy equation (4.1)

$$\sigma_{Fd} \leq f_y / \gamma_M \tag{4.1}$$

where f_y is the yield stress of the face material and γ_M the material factor defined for the yielding failure mode according to Table 2.5.

4.3.2 Compression strength of a face

The methods referred to in sections 3.4 and 3.5 permit the calculation of the bending moments and forces in the faces, whether these are profiled or flat. In a sandwich panel with flat or lightly profiled faces the ultimate limit state will generally involve wrinkling of a face (see section 4.3.2.2). With profiled faces, yield or local buckling of the outer part of the section may occur (see section 4.3.2.1).

4.3.2.1 Local buckling of a profiled face

If the outermost plate element in a panel is in compression and its breadth to thickness ratio exceeds

$$\left(\frac{b}{t}\right)_{\text{local,limit}} = 1.27\sqrt{\frac{E_F}{f_y}} \tag{4.2}$$

due regard should be given to the effect of local buckling. If the breadth to thickness ratio (b/t) is higher than the limit given in Equation 4.2, the compressive stress σ_{Fcd} in the most stressed plane part of the face profile, which is bonded to the core, shall satisfy equation (4.3).

$$\sigma_{Fcd} \leq f_{Fc} / \gamma_M \tag{4.3}$$

where f_{Fc} is the strength of the compressed plane part of a profiled face and γ_M material factor defined for the yielding failure mode according to Table 2.5.

The compression strength of the profiled faces depends on the yield stress of the face material, on the breadth to thickness ratio of the most stressed plane part of the profile and on the compressive and shear stiffness of the core material. Furthermore, it depends also on the initial imperfections caused by the face, core and the bond. Therefore, it is recommended, that the compression strength f_{Fc} of the profiled faces is determined using full-scale tests according to section 5.2.6. If no test results are available, the compression strength f_{Fc} can be evaluated by calculations based on the effective width approach (Eq. 4.4 and Fig. 4.3). The effective widths of plane parts of a face profile are used only in the evaluation of the compression strength of the face (f_{Fc}). The calculation of the stresses in the face (σ_{Fcd}) according to Sections 3.1 and 3.5 is based on the full cross-section of the face profile.

$$f_{Fc} = \frac{b_{eff}}{b} f_y \tag{4.4}$$

where

$$\frac{b_{eff}}{b} = \frac{1}{\overline{\lambda}_p}\left(1 - \frac{0.22}{\overline{\lambda}_p}\right) \tag{4.5}$$

$$\overline{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} \tag{4.6}$$

$$\sigma_{cr} = K_\sigma \frac{\pi^2 E_F}{12(1 - \nu_F^2)}\left(\frac{t}{b}\right)^2 \tag{4.7}$$

If the elastic support given by the core is ignored, the buckling coefficient in Eq. 4.7 has the value $K_\sigma = 4.0$. If the elastic support provided by the core is utilised in evaluation of the compressive strength of the face, a higher value of the buckling coefficient may be calculated using Equations 4.8 and 4.9 /Davies J.M. & Hakmi M.R. 1990, Davies J.M., Hassinen P. and Hakmi M.R. 1991/.

$$K_\sigma = \sqrt{16 + 7R + 0.002R^2} \tag{4.8}$$

$$R = 0.35\frac{\sqrt{E_C G_C}}{E_F}\left(\frac{b}{t}\right)^3 \tag{4.9}$$

The limit of applicability for Expressions 4.4 - 4.7 is $b/t \leq 500$ and for Equations 4.8 - 4.9 it is $R \leq 200$ and $b/t \leq 250$. E_C and G_C are the mean values for the modulus of elasticity and the shear modulus of the core respectively.

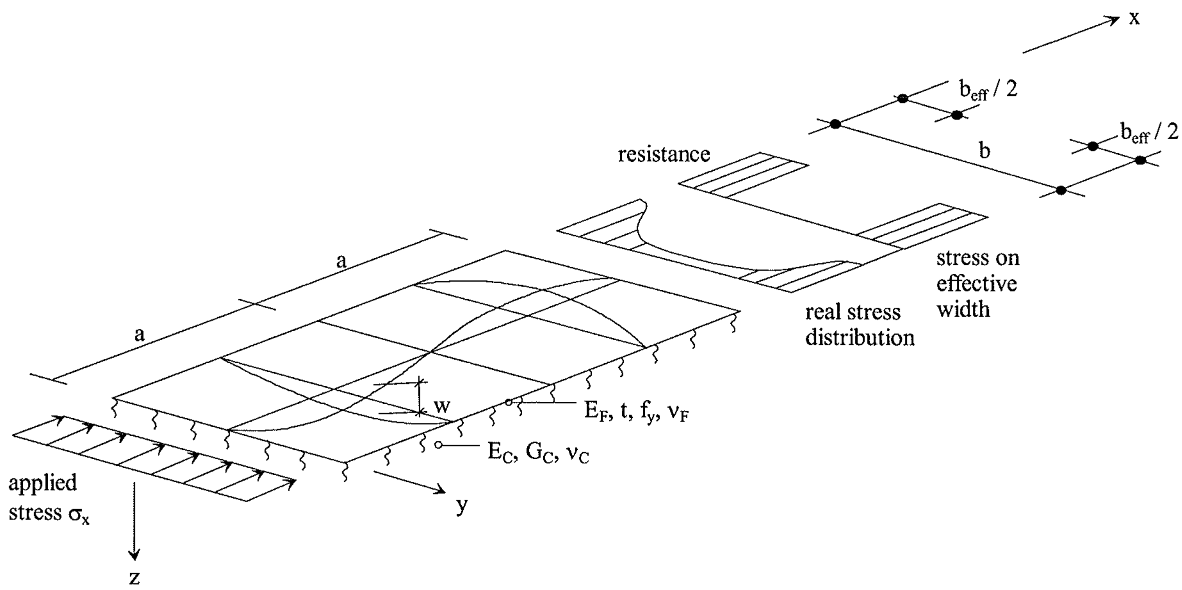


Fig. 4.3 Definition of the effective width of the compressed plane part of a face profile. The plane part is assumed to be simply supported by two webs of the profile along the longitudinal edges.

<u>Comment:</u> The compression strength of a profiled face can be determined on the basis of the test results on full-scale single span sandwich panels using the expression (3.2) given for the stress σ_F in section 3.1 and the expressions given for bending moments M_S and M_{F1} in section 3.5.1:

$$M_S = (1-\beta_q) M_u$$

$$M_{F1} = \beta_1 \beta_q M_u$$

$$f_{Fc11} = \left| -\frac{M_S}{eA_{F1}} - \frac{M_{F1}}{I_{F1}} d_{11} \right|$$

where

M_u is the ultimate bending moment in the test and

f_{Fc11} the compression strength of the outer plane part of the upper face profile.

4.3.2.2 Wrinkling stress of flat and lightly profiled faces in the span

The compressive stress in a flat, lightly profiled or micro profiled metal face shall satisfy the equation

$$\sigma_{Fcd} \leq \sigma_w / \gamma_M \qquad (4.10)$$

where γ_M is the material factor defined for the wrinkling failure mode in the span according to Table 2.5.

Because the compressive strength of flat and lightly and micro profiled faces is strongly dependent on the initial imperfections in the face, core and the bond between them, it is recommended, that the wrinkling stress σ_w of these faces is determined on the basis of full-scale tests according to Section 5.2.6.

Nevertheless, the wrinkling stress σ_w of a flat metal face may be calculated using

$$\sigma_w = k\sqrt[3]{E_{CT}\, G_{CT}\, E_F} \qquad (4.11)$$

where G_{CT} = mean value of the shear modulus of the core
E_{CT} = mean value of the elastic modulus of the core (taken as the average of the tensile and compressive moduli)
E_F = elastic modulus of metal face.

The subscript T in the expression (4.11) indicates that the formula is valid at both ambient and elevated temperatures. The values for the modulus of elasticity E_{CT} and shear modulus G_{CT} have to be chosen according to the temperature for which the wrinkling strength is evaluated.

Comments:

1. The coefficient k in Equation 4.11 is a constant and its value depends on the imperfections and quality of the face, core and bond. The typical values of the coefficient k given below have been determined experimentally.

2. The value k = 0.65 has been found to be appropriate for continuously laminated polyurethane sandwich panels of good quality. Values k = 0.5...0.65 may be appropriate for other core materials and methods of manufacture.

3. Unless the stiffness of the core is unusually small ($\sqrt{E_C G_C} < 3\ \mathrm{N/mm^2}$) an indicative value of the wrinkling stress of a lightly profiled or micro profiled metal face may be calculated using

$$\sigma_w = \frac{k_p}{A_F}\sqrt[3]{E_{CT}\, G_{CT}\, B_F} \qquad (4.12)$$

where $B_F = E_F I_F$ flexural rigidity of the face per unit width ($B_F = E_{F1} I_{F1}$ for the face 1 and $B_F = E_{F2} I_{F2}$ for the face 2, respectively) and
A_F = cross-sectional area of the face per unit width ($A_{F1} \approx t_1$ for face 1 and $A_{F2} \approx t_2$ for face 2)

4. The coefficient k_p in Equation 4.12 is a constant and its value depends on the imperfections and quality of the face profile, core and bond. The value of the coefficient k_p shall be defined experimentally.

5. If only one face thickness (t_{exp}) is used in the tests for a sandwich panel with lightly profiled faces, the wrinkling stress for larger face thicknesses shall be reduced using the formula (4.13)

$$\sigma_w = \frac{A_{F,exp}}{A_F} \sqrt[3]{\frac{B_F}{B_{F,exp}}} \sigma_{w,exp} \qquad (4.13)$$

where $A_{F,exp}$ and $B_{F,exp}$ are the cross-sectional area and flexural rigidity of the face used in the tests,
A_F and B_F the cross-sectional area and flexural rigidity of the face with a larger face thickness and
$\sigma_{w,exp}$ the experimentally determined wrinkling stress

To the wrinkling stress for the faces with a smaller thickness than that in the test ($t < t_{exp}$), the value determined in the test $\sigma_w = \sigma_{w,exp}$ shall be used.

6. The value $k_p = 0.95$ has been found to be appropriate for continuous laminated polyurethane sandwich panels of good quality with frequent folds in the compressed face. If the folds are too far apart, the value of k_p will be reduced as a result of the tendency to wrinkle between the folds.

Notes:

1. E_{CT} and G_{CT} are the mean values of the material properties of the core at the relevant temperature of the face in compression. (i.e. for Summer calculation, reduced Summer values should be used).

2. The calculated wrinkling stress for a lightly profiled face (Equation 4.12) should normally not be taken as less than that for a plain face of the same metal thickness.

3. The formula (4.12) for the wrinkling stress of a lightly or micro profiled face may be assumed to be valid provided that the depth of the profile, measured from trough to crest on one side, is not greater than five times the thickness of the metal face and $b/t < 100$ for flat parts of the face.

4. In many cases the above equations for σ_w will prove to be conservative and higher values may be obtained by testing in accordance with section 5.2.6.

5. The formula (4.12) is valid only if the bonding strength fulfils the requirements for bonding strength (Section 4.1.1).

6. For panels with a core formed of discrete core material (eg slabstock or lamellas), the wrinkling stress shall be determined by full-scale panel testing because of possible imperfections in the transverse joints between the discrete core elements (Section 5.2.6).

4.3.2.3 Determination of the wrinkling stress at a support

If elastic analysis is used in the verification for the ultimate limit state, the wrinkling stress of a flat or lightly profiled compressed face at an intermediate support shall be determined by testing (Sections 5.2.8 and 5.2.9) or may be calculated using the equations given in Section 4.4.1. If the verification is based on plastic analysis, the value of the compressive strength in the face is assumed to vanish over the intermediate supports. If the remaining inelastic resistance is utilized, the compressive strength corresponding to the actual plastic rotation over the support shall be determined experimentally.

For the wrinkling stress at a support the material factor γ_M is defined corresponding to the wrinkling failure mode at an intermediate support as given in Table 2.5.

4.3.3 Shear failure of the core

The shear stress in the core layer has to satisfy the equation

$$\tau_{Cd} \leq f_{Cv} / \gamma_M \qquad (4.14)$$

where f_{Cv} is the characteristic shear strength of the core and
γ_M material factor defined for the shear failure in the core layer according to Table 2.5.

The characteristic shear strength of the core material f_{Cv} shall be determined by testing according to Sections 5.2.4 or 5.2.5 giving due regard to the design temperatures. The ultimate limit state of shear failure of the core may be determined using the average calculated shear stress at the section of maximum shear force (Eq. 3.4).

Where panels are produced using adhesives, it must be verified that the adhesive will not fail before failure of the core itself.

When panels are produced using discrete core materials with transverse joints, the shear strength must be assessed using a test on complete panel with the most unfavourable position of the joints.

When the core layer is only partially bonded to the faces, for instance only to a flange instead of the whole face profile, the shear strength must be assessed using a test on complete panel giving due regard to the long term behaviour and strength of the sandwich panel. Such partial bonding is not recommended because of the shear stress concentrations in the bond and the potential shear failure risks in the long term.

The shear strength of the core f_{Cv} decreases under long-term loading. If a sandwich panel is designed to carry long-term permanent loads, the reduction in the shear strength should be taken into account. A test procedure for determining the long-term shear strength is given in Section 5.2.4.3.

Note: At an internal support, the fundamental equations given in Appendix A may become badly conditioned during the transition from flat face behaviour (all shear force carried by the core) to profiled face behaviour (all or most of the shear carried by the profile). Therefore

(a) for a plain or lightly or micro profiled face, all of the shear force should be carried by the core

(b) for profiled faces, it should be demonstrated that all of the shear force can be carried by either the core or by the faces acting alone.

4.3.4 Shear failure of a profiled face layer

The shear stress in the web of a profiled face layer shall satisfy the equation

$$\tau_{Fdi} \leq f_{Fvi} / \gamma_M \tag{4.15}$$

where i = 1 and 2 for faces 1 and 2, respectively.

The material factor for the shear failure in a profiled face γ_M is defined corresponding to the shear failure in a profiled face according to Table 2.5.

Methods to evaluate the shear forces, V_{F1}, V_{F2}, in the face layers are introduced in Chapter 3 and Appendix A. The shear stresses in the webs of the face profile can be calculated using the expressions (3.5a,b).

Unless it can be shown that the entire shear force at any section can be carried within the core layer according to section 4.3.3, it shall be shown that the entire shear force can be carried by the profiled face layers. Unless a proven design method which takes account of the supporting action of the core is available, the influence of the core in resisting the buckling of the web of the face profile shall be ignored and the determination based on the following expressions for trapezoidal metal sheeting /Eurocode 3 1996/:

$$f_{Fv} = \frac{f_y}{\sqrt{3}} \quad \text{if } \bar{\lambda}_w \leq 0.83$$

$$f_{Fv} = \frac{0.48\, f_y}{\bar{\lambda}_w} \quad \text{if } 0.83 \leq \bar{\lambda}_w \leq 1.40 \qquad (4.16a,b,c)$$

$$f_{Fv} = \frac{0.67\, f_y}{\bar{\lambda}_w^{\,2}} \quad \text{if } 1.40 \leq \bar{\lambda}_w$$

and

$$\bar{\lambda}_w = \sqrt{\frac{f_y}{\sqrt{3}\,\tau_{cr}}} = 0.346 \frac{s_w}{t} \sqrt{\frac{f_y}{E_F}} \qquad (4.17)$$

In expressions (4.16) and (4.17), f_y is the tensile yield stress of the particular face material.

4.3.5 Crushing of the core at a support

The compressive stress in the core layer over a support shall satisfy the equation

$$\sigma_{Ccd} \leq f_{Cc} / \gamma_M \qquad (4.18)$$

The characteristic compression strength f_{Cc} shall be determined experimentally according to Section 5.2.3. The material factor γ_M for a crushing failure in a core is defined in Table 2.5.

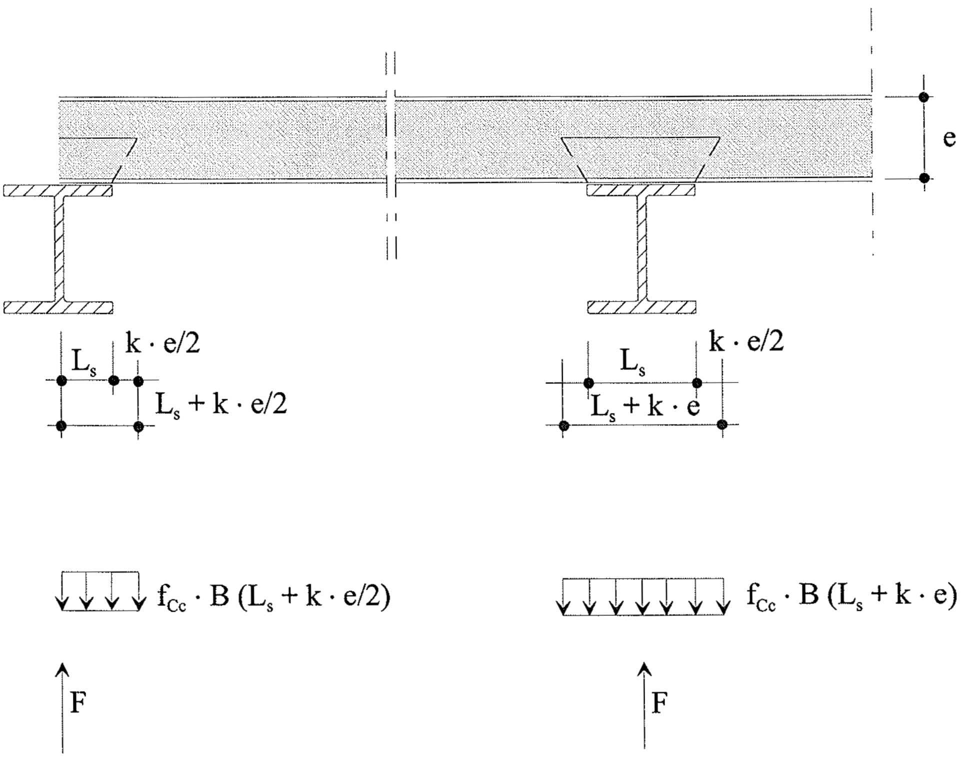


Fig. 4.4 Support reaction resistance at end and intermediate supports.

At a support, the distribution of the compressive stresses in the core is taken into account by assuming the support reaction to cause a uniformly distributed stress at mid-depth of the core (Fig. 4.4). In general, the angle of dispersion of $\tan^{-1}(k)$ shall be determined experimentally according to Section 5.2.10. The compressive stress at an end and an intermediate support are

$$\sigma_{Ccd} = \frac{F}{B\left(L_s + k\,e/2\right)} \tag{4.19a}$$

$$\sigma_{Ccd} = \frac{F}{B\left(L_s + k\,e\right)} \tag{4.19b}$$

where k is a distribution parameter.

In the absence of test results, k may be taken as k = 0.5 for rigid plastic foams. For mineral wool, k should be determined by testing or use k = 0. In these calculations, e is the distance between the centroids of the face layers. For sandwich panels with e > 100 mm, e = 100 mm should be used in the expressions (4.19).

If plastic design is used for the calculation of the ultimate limit states, that is after plastic hinge action at the internal supports, the crushing of the core shall be considered separately without any interaction with the compressive stresses in the face.

If elastic analysis is used in the verification of the ultimate limit state, the interaction between crushing of the core and compressive failure of the face which is placed against the supporting structure shall be taken into account as described in section 4.4.1.

In order that the compressive stress distribution in the core and the bending rigidity of the lightly profiled faces may be taken into account more accurately in the evaluation of the compressive stresses at an intermediate support, special design methods or tests are necessary /Lightweight sandwich construction 2000/.

Comments:

1. The distribution parameter k takes into account the effect of the bending rigidity of the face layer to distribute the support pressure in the core.
2. Calculation models for the compressive stress over a support are based on the assumption that the support plate is rigid and the support pressure is symmetrically distributed in relation to the mid-line of the support. In the case of flexible supporting structures, such as cold formed open sections, the support pressure may be assumed to be distributed over a smaller area; ($k_f L_s + k e / 2$) for an end support and ($k_f L_s + k e$) for an intermediate support, where $k_f \leq 1$.

4.3.6 Failure of the fastenings

Fastenings at supports are loaded by tensile forces caused by wind uplift loads and temperature differences between the faces of the panel. Fastenings may also be loaded by shear forces caused by the self-weight of the panels and by the weight of additional building components on a wall and roof, by the temperature expansion of the faces and further, by diaphragm action. For the verification of the failure modes of fastenings, the tensile resistance and, in the relevant cases, the shear resistance of the fastenings have to be known.

The tensile and shear resistances of the fastenings shall be determined experimentally according to section 5.2.18 (See also Chapter 7).

The material factor γ_M for the tensile or shear failure of a fastening is defined, corresponding to the relevant failure mode, in Table 2.5.

Comments:

The characteristic tensile and shear resistance of the screw and special fastenings of sandwich panels shall generally be determined by tests.

If no test results for the shear strength are available, in the preliminary phase of design, the expressions developed for screw connections between two metal sheets may be used in evaluations of the shear resistance for screw connections of sandwich panels /Eurocode 3 1996/

$$F_{Rvd} = \min\begin{Bmatrix} \alpha\, f_u d_n t/\gamma_M \\ A_n f_u/\gamma_M \end{Bmatrix} < 1.2\, F_{v,Rd} \qquad (4.20)$$

where f_u is the tensile strength of the face material
d_n the nominal diameter of the fastener
t is the thickness of the face which is against the supporting structure
t_l the thickness of the thickest sheet in connection (face or supporting structure)
A_n is net sectional area of the face
$F_{v,Rd}$ is the shear resistance of the shaft of the fastener itself

$$\alpha = 3.2\sqrt{t/d_n} \leq 2.1 \quad \text{for } t = t_1 \text{ and} \qquad (4.21)$$

$$\alpha = 2.1 \quad \text{for } t_1 \geq 2.5t \qquad (4.22)$$

The shear resistance for the thickness relationships $1 \leq t_1/t \leq 2.5$ may be determined by linear interpolation between equations (4.20), (4.21) and (4.22).

4.3.7 Failure of a panel at a point of connection

See Chapter 7.

4.3.8 Failure in transverse bending

When a sandwich panel is fastened to the supporting structure with fasteners placed only in the longitudinal joints between the adjacent sandwich panels and when the panel is subject to negative support reactions, the sandwich panel is loaded by transverse bending in addition to the longitudinal bending, as shown below.

This may give rise to unfavourable effects in panels with one or more highly profile faces. It is particularly dangerous in mineral wool panels with little transverse shear strength and stiffness between individual lamellas.

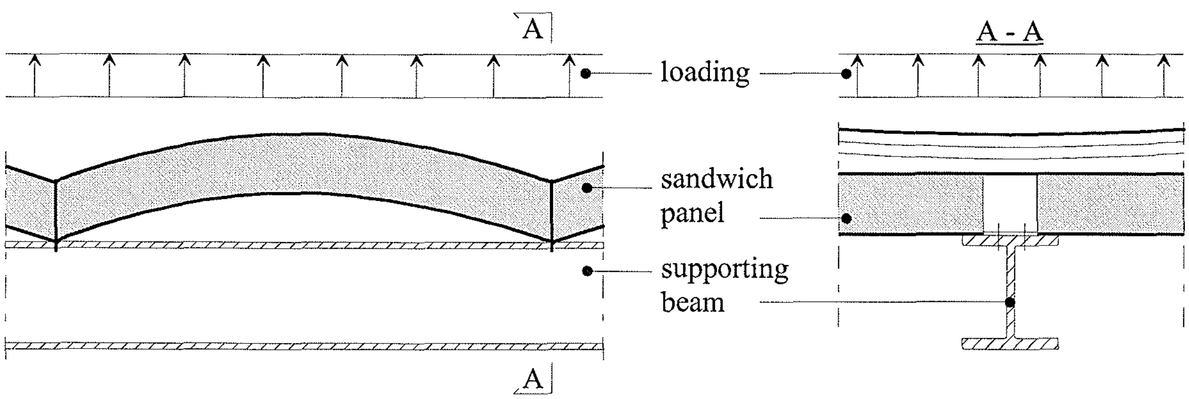


Fig. 4.5 Transverse curvature of sandwich panel.

In addition to the additional stresses caused by the transverse bending, transverse curvature results in some favourable effects which decrease the usual reduction (Sections 4.3.2.3 and 4.4.1) of the compressive strength of the face at intermediate supports. Thus, the transverse curvature may strengthen the compressed face against a buckling failure over an intermediate support. The transverse curvature of the panel and the flexibility of the fastening system may reduce the stresses caused by temperature differences and may change the bending moment and shear force distributions caused by wind uplift loading. The two loading effects may be considered separately and it should be demonstrated that there is an adequate reserve of safety with respect to transverse bending.

4.4 Resistance at the serviceability limit state

The stresses and deflections in the verification for the serviceability limit state should be based on an elastic analysis. In the verification, the failure mode corresponding to the first failure at the intermediate supports of continuous panels and in points of line or point loads has to be studied. The first failure mode may be a wrinkling failure in the face under compressive stresses, shear failure in the core or a crushing failure of the core or of a profiled face. In the serviceability limit state verifications, the value of the elastic deflections shall be compared with the deflection limits given in section 2.3.2.

4.4.1 First yield or wrinkling in flat or lightly or micro profiled faces in contact with a support or line load

In a continuous sandwich panel, the attainment of first yield or wrinkling at a support or at a line load may be considered as a serviceability condition when the ultimate limit states are defined on the basis of a plastic analysis. The considerations are as described in Sections 4.3.1 and 4.3.2.

Self weight, wind pressure and snow all cause negative bending moments and positive support reactions at the intermediate supports in continuous sandwich panels. Positive support reaction is defined as causing compressive contact stresses between the panel and the supporting structure. In this loading case, and particularly when considering the serviceability limit state, the local compressive and bending strengths of the lower face and the compressive strength of the core are critical (Fig. 4.6a).

The load case of wind suction, and some cases of temperature difference between the faces, cause negative support reactions. Here, the support reaction force causes local tensile forces in the connections between the sandwich panel and its supports and these interact with the compressive stresses in the outer face of the panel (Fig. 4.6b).

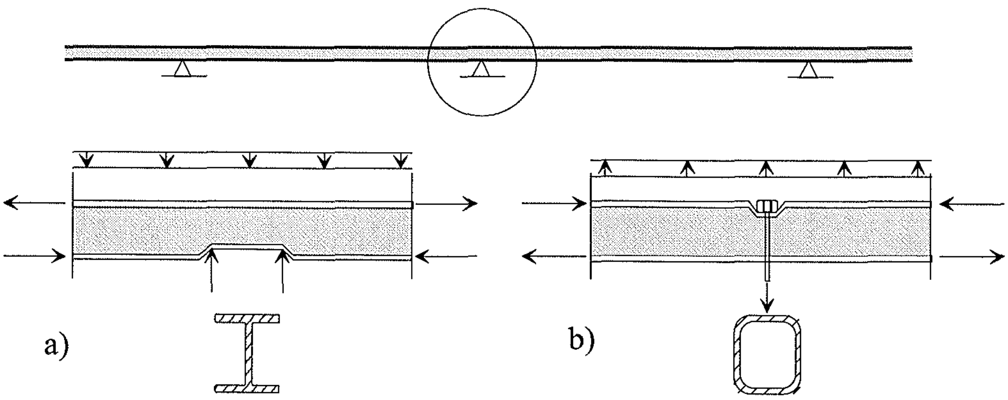


Fig. 4.6 Failure modes of a) positive and b) negative reaction at an intermediate support.

At an intermediate support in a continuous multi-span panel, the compressive strength, which is defined as the wrinkling stress or apparent yield stress of the face which is in contact with the support, shall be reduced to take account of the influence of the support force which causes local deformation of this face. This deformation has a significant influence on the compressive strength of the face.

For verification of the failure mode at an intermediate support in the serviceability limit state, the compressive strength of the faces for both the positive and negative support reaction loading cases have to be known. A test method to determine the compressive strength on the basis of full-scale panel tests is given in Section 5.2.8.

Design by testing according to Chapter 6 avoids most of the issues discussed in Section 4.4.1. However, it shall be born in mind that it is often the serviceability condition at internal support that determines the design of a continuous sandwich panel. Therefore, when designing on the basis of tests on continuous panels, particular attention must be paid to the load at which the serviceability limit state is achieved.

Comments:

If no test results are available it has been found out to be adequate:

a) for downward loads (positive support reaction) to take a uniform global decrease of 10 % of the wrinkling stress determined within the span of the sandwich panel. This is applied on the basis of the above evaluation together with the additional reduction (γ = 1.4) for the compressive stress in the core over the support which gives the requirement (σ_{Ccd} / f_{Cc} $\leq$ 1 / 1.4γ_M = 0.71 / γ_M). This method is based on experimental results for core materials with compressive strength f_{Cc} = 0.06 to 0.12 N/mm^2. Outside this range, additional precautions are required.

Therefore, for practical design, the interaction between the bending moment M and the support reaction F can be taken into account without any detailed interaction studies by reducing the strength of the compressed face at an intermediate support by 10% and by evaluating the support reaction resistance using the conservative expression with an additional reduction coefficient 1/1.4 = 0.71, thus:

$$f_{Fc,support} = 0.9\, f_{Fc} \tag{4.23}$$

$$F_{Rd} = 0.71\, L_S\, f_{Cc}/\gamma_M \tag{4.24}$$

b) for uplift loads (negative support reaction) the interaction between the transverse forces caused by screws passing through the panel and the axial compressive load in the face can be taken into account by reducing the compressive strength of the face at the intermediate supports. The reduction of the compression strength shall be 20% if the number of screws at the support is less than or equal to 3 in a panel width of 1 metre. If the number of screws is larger than 3, the compression strength of the face shall be reduced according to the equation:

$$f_{Fc,support} = 0.8\, f_{Fc} \ , \ \text{if } n \leq 3 \tag{4.25a}$$
$$f_{Fc,support} = 0.125(11-n)\left(0.8\, f_{Fc}\right) \ , \ \text{if } n > 3 \tag{4.25b}$$

where n is the number of screws in a panel width of 1 metre.

This method is based on results of tests in which the fasteners were spaced regularly in the panel. If the fasteners are not regularly spaced across the panel width, the method gives conservative results.

For sandwich panels fixed with special fasteners placed in the longitudinal joints of the adjacent panels, the interaction between the transverse forces and the axial force in the face shall be determined experimentally on the basis of tests on the whole sandwich panel (See Chapter 5).

4.4.2 Resistance of a profiled face in contact with an intermediate support or line load

If the face in contact with the supporting structure or line load is profiled, its characteristic support reaction resistance can be evaluated using the semi-empirical expression derived for trapezoidal sheets /Eurocode 3 1996/ (Fig. 4.7). This failure mode is generally only associated with the serviceability limit state.

$$F_{R2} = n0.15t^2\sqrt{E_F f_y}\left(1-0.1\sqrt{\frac{r}{t}}\right)\left(0.5+\sqrt{\frac{L_s}{50t}}\right)\left(2.4+\left(\frac{\phi}{90}\right)^2\right) \qquad (4.26)$$

where n is the number of webs per unit width, r is the radius of the bend between the lower flange and the web and ϕ the angle between the support beam and the web of the profile (Fig. 1.1). The expression (4.26) is valid for $45° \leq \phi \leq 90°$. In addition, it is assumed, that the relative difference between the transverse shear forces on each side of the support is small, i.e.,

$$\beta_v = \frac{|V_{Sd,1}|-|V_{Sd,2}|}{|V_{Sd,1}|+|V_{Sd,2}|} \leq 0.2 \qquad (4.27)$$

where $|V_{Sd,1}|$ and $|V_{Sd,2}|$ are the absolute values of the transverse shear forces on each side of the intermediate support. If the equation (4.27) is not valid, the value of the support width L_s in equation (4.26) shall be reduced /see Eurocode 3 1996/.

At an end support (distance between the end of the panel and the supporting structure is less than 1.5 times the depth of the profile) the support reaction resistance is half of the value given by Eq. (4.26).

If the lateral support provided by the core layer against the local buckling is neglected, the interaction between the bending moment M_{F2} in the profiled face in contact with the supporting structure or line load and the support reaction or line load F can be evaluated using the semi-empirical equations derived for trapezoidal sheets /Eurocode 3 1996/ (Fig. 4.8).

$$\frac{M_{F2}}{M_{2R}/\gamma_M} + \frac{F}{F_{R2}/\gamma_M} \leq 1.25 \qquad (4.28)$$

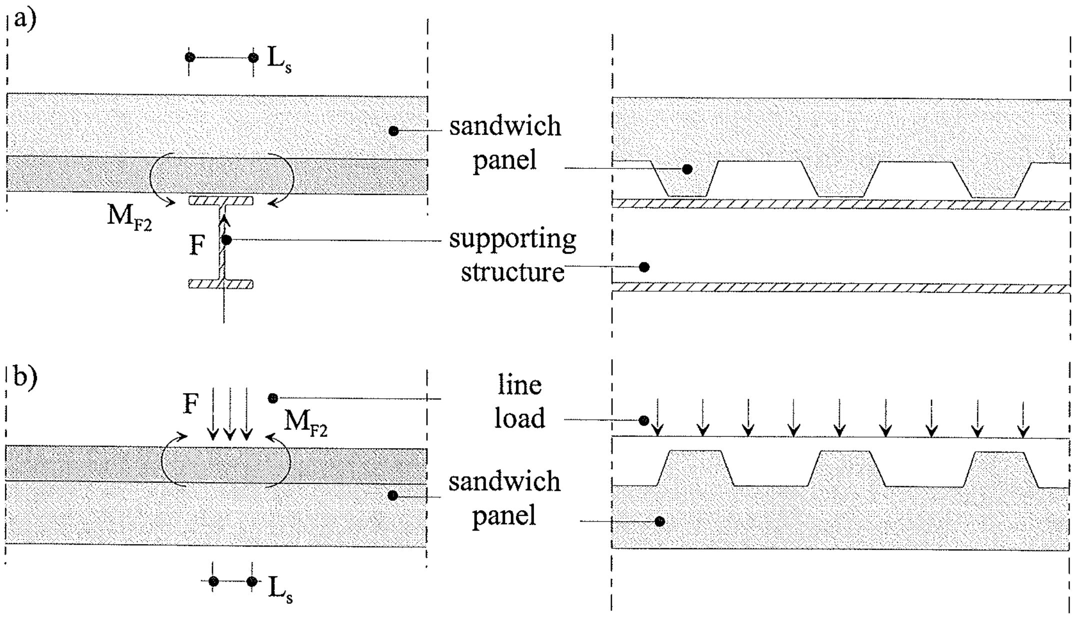


Fig. 4.7 Crushing of a face profile a) on an intermediate support or b) in contact with a line load.

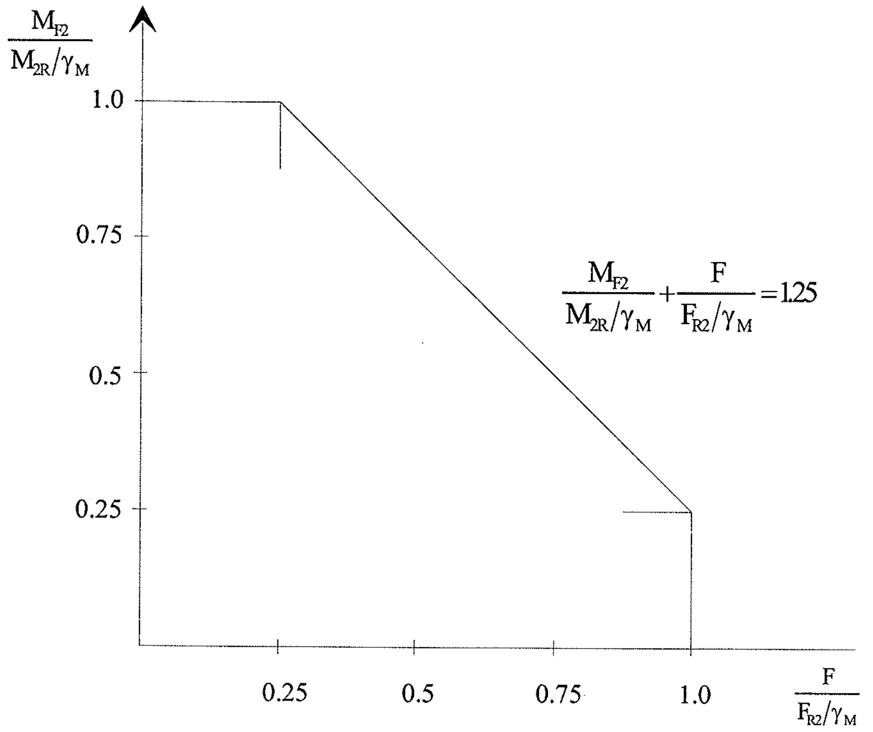


Fig. 4.8 Bending resistance of a profiled face in contact with an intermediate support or line load.

4.4.3 Shear yielding of the core layer

Shear failure of the core is usually a non-ductile failure mode which takes the structure directly to the ultimate limit state. If, however, the shear stress-strain behaviour of the core material has elasto-plastic characteristics and the cross-section of the panel has a large plastic shear resistance, shear yielding of the core may be classified to be a serviceability limit state failure. Experimental results are needed before utilizing the plastic shear resistance in design.

Comment: There is a considerable difference between core materials. Structural mineral wools and extruded polystyrene usually exhibit ductile behaviour in shear.

4.4.4 Limit state of deflection

The deflections of wall and roof panels shall be calculated using elastic theory. In such calculations, the short-term and long-term shear deformations of the core shall be taken into account. In special cases, the deflections caused by the local deformations of the core over the supports and by the flexibility of the fastening systems may also be of sufficient importance to be taken into account in calculations. Limits for maximum deflections are defined in Section 2.3.2.

4.5 Walkability

For roof and ceiling panels, in addition to the service loads, it shall be demonstrated that the panel is safe with respect to a single person walking on the panels for occasional inspection. This requirement may be satisfied by testing according to section 5.2.11 with loads according to section 2.2.2.

4.6 Impact resistance of wall panels

Where wall panels are located at ground level in buildings accessible to the public, or are otherwise liable to impact loading, it should be demonstrated that they possess adequate resistance to impact. Suitable test procedures are given in section 5.2.17.

5. TESTING AND SAMPLING

5.1 General

In general, and unless stated otherwise, each property shall be determined on the basis of a minimum of three tests. Where a number of different core thicknesses are to be produced for a range of otherwise similar panels, it shall be permissible to determine the core-dependent properties for the thinnest and thickest panels to be produced, together with a panel near the middle of the range, and to use linear interpolation.

Material properties (tensile, compression and shear strengths and the influence of ageing) based on specimens cut from full panels shall generally be determined from a considerably larger number of tests with a minimum of five.

Unless otherwise noted, both the equipment applying the loads and the devices measuring the load applied shall be capable of working to an accuracy of at least 1 % and all deflections shall be measured with an accuracy of at least 0.1 mm.

5.1.1 Sampling

The test specimens shall be taken from a range of positions covering the width of the panel. At least one specimen shall be taken at a minimum distance from an outside edge of 10% of the cover width of the panel and at least one specimen from the middle of the panel.

5.1.2 Determination of characteristic values from tests

For each of the tests which are described in the following sections, which result in quantified design parameters, the characteristic values of the relevant properties shall be determined in accordance with the following procedure.

For each population of the test results, the mean value and the 5% fractile value shall be determined. The 5% fractile value shall be used as the characteristic value, thus:

$$\overline{x}_p = \overline{x} - k_\sigma \sigma_x \qquad (5.1)$$

where $\overline{x}_p$ = 5% fractile value of population x
$\overline{x}$ = mean value
k_σ = fractile factor given in the table below
σ_x = standard deviation

Assuming a confidence value of 75% the fractile values are /ISO 12491 1997/:

number of specimens	3	4	5	6	7	8	9	10	15	20	30	60	∞
k_σ	3.15	2.68	2.46	2.34	2.25	2.19	2.14	2.10	1.99	1.93	1.87	1.80	1.76

5.1.3 Test environment and conditioning of test specimens

In the procedures that follow, unless stated otherwise, all testing shall be carried out in a laboratory under indoor conditions.

The minimum age of the specimens for initial type tests shall be 24 hours. Specimens for quality control tests may be taken immediately after production. The date, time, temperature and relative humidity shall be recorded at the time of sampling.

In cases of dispute, or where the temperature or relative humidity are considered to be of particular significance, control tests shall be carried out under conditions:

Temperature:	23°C ± 5°C
Relative humidity	50% ± 10%

For identification purposes, the core density of all tested specimens shall be recorded with the test results. For this purpose, it is sufficient to weigh three small prismatic core specimens cut from different regions of each complete panel which is used for the preparation of test specimens.

5.1.4 Accelerated ageing tests

5.1.4.1 Preparation of test specimens

In order to determine the deterioration of strength with time as a consequence of ageing of the core material, for each set of tests a minimum of five specimens measuring 100 mm x 100 mm shall be cut from the full thickness of the panel with the faces intact.

The cut edges of the metal facing sheets in the specimens shall be protected from the effects of corrosion by the application of a suitable silicone-based protecting agent.

Prior to commencing the accelerated ageing tests, the unaged tensile strength R_0 shall be determined in accordance with 5.2.2 with the faces intact.

Prior to the accelerated ageing tests, the specimens shall be stored for at least 24 hours at + 23°C ± 5°C under normal laboratory conditions.

For tests on aged specimens, the specimens shall be prepared by subjecting them to the following ageing cycles. Between cycles, the specimens shall be transferred from one condition to another in less than 5 minutes or else they shall be enclosed in an airtight bag.

The dimensions of all test specimens in all three directions shall be measured before and after the tests and the dimensional changes shall be determined.

5.1.4.2 The basic ageing cycle, C1

The term day shall mean a time period of 24 hours. The basic ageing cycle shall consist of the following sequences which is denoted as cycle C1:

- 5 days at +70°C ± 5°C and RH 90% ± 10% followed by
- 1 day at -20°C ± 5°C followed by
- 1 day at +90°C ± 5°C and RH < 15%

Note: In the first element of the cycle, the specimen is kept on a grid over water in a closed box. It is the temperature of the air in the box that shall be controlled, not the temperature of the water.

5.1.4.3 C2 test

For the test regime denoted C2, the test specimens shall be maintained under constant conditions for 28 days at +65 ±3^{0}C and 100% RH.

Note: In this cycle, the specimen is kept on a grid over water in a closed box. It is the temperature of the air in the box that shall be controlled, not the temperature of the water.

5.1.4.4 Cycles to determine the cyclicly aged performance

One set of five specimens shall be subject to a single basic ageing cycle C1. After exposure, the test samples shall be preconditioned to the initial moisture and temperature prior to tensile testing. The tensile strength shall then be determined in accordance with 5.2.2. R_1 is the mean value of the test results.

A second set of tests shall be subject to five basic ageing cycles C1 under the same conditions as above. The mean tensile strength value so obtained is denoted R_5 .

Depending on the results obtained, it may be necessary to test a third set of test subject to ten basic ageing cycles under the same conditions as above. The mean tensile strength value so obtained is denoted R_{10}.

One set of five specimens shall be subject to the ageing cycle C2. After exposure, the test samples shall be preconditioned to the initial moisture and temperature prior to tensile testing. The tensile strength shall then be determined in accordance with 5.2.2. R_T is the mean value of the test results.

5.1.4.5 Test results and acceptance criteria

The following criteria shall be fulfilled:

- The value of R_1 shall not be smaller than 0.6 R_0
- The value of R5 shall not be smaller than 0.4 R_0
- The value of R_1 - R_5 shall be equal to or smaller than R_0 - R_1
- The value of R_T shall not be smaller than 0.40 R_0

If the third condition is not fulfilled, a third set of test specimen shall be exposed to ten basic ageing cycles C1. The acceptance criteria shall then be:

- R_5 - R_{10} shall be equal to or smaller than R_1 - R_5. If this is not fulfilled, but $R_{10} \geq$ 0.60 R_0, the test result is considered to be acceptable.

The dimensional changes after ageing in all of the test procedures described above shall be less than 5%.

Comment:	The work on the testing procedures for the durability of sandwich panels is reported in ref. /Kerkkänen & Tiainen 1999/. The test method is still under process of development and the cycles and the criteria represent the state of the art at the time of the publishing of the Recommendations.

5.2 Test procedures for material properties

5.2.1 Tensile test on the face material

Tensile tests to determine the yield strength and other properties of the face material should be carried out in accordance with the standard EN10002-1.

5.2.2 Tensile test on the core material

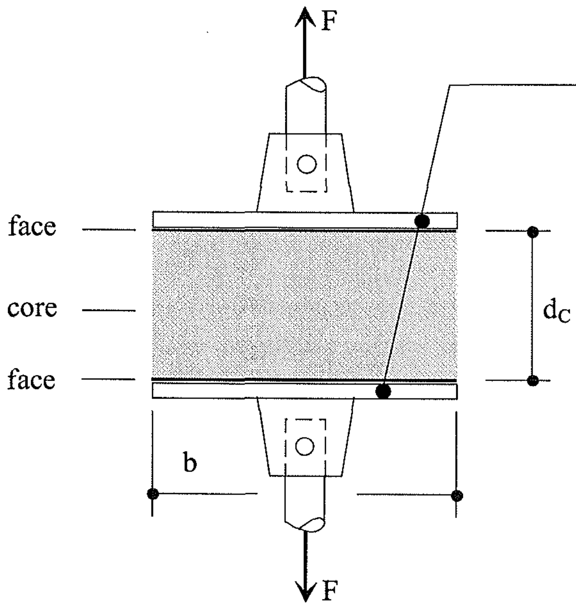
This test may be performed in one of the two ways:

(a) With the faces of the panel intact in order to determine the tensile bond strength between the faces and the core or to demonstrate adequate bond.

(b) Before the faces are attached in order to determine the tensile strength of the core.

In general, the bond with the faces is of fundamental importance and the test should be carried out with the faces intact and failure should not take place in the bond layer.

Specimens of square cross-section shall be prepared to the dimensions shown and bonded using a suitable adhesive to platens of sufficient stiffness to ensure a uniform tensile stress over the area of specimen. With lightly profiled faces, special measures may be required in order to ensure full adhesion between the platens and the faces.



$0.5\, d_C \leq b \leq 1.5\, d_C$ but not less than 50 mm

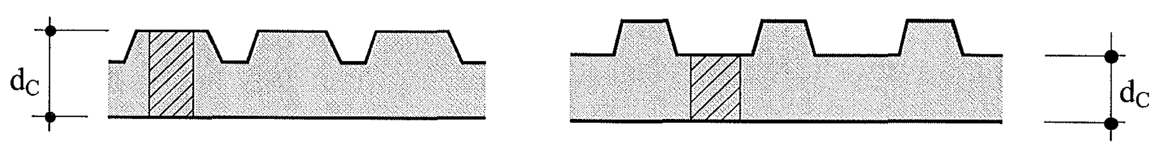
where

d_C = thickness of the core of the test specimen

b = width of test specimen

Fig. 5.1 Test arrangements in the tension test of the core and bond.

Notes: 1. For panels with profiled faces the specimens should be cut from the predominant thickness as shown.



2. Better results are generally obtained with larger specimens and it is recommended that, where possible, specimens with width b as least 100 mm should be used.

3. Cylindrical test specimens of minimum diameter 50 mm may also be used.

4. For specimens of mineral wool core material, rather larger specimens are generally required, i.e. $100 \text{ mm} \leq b \leq 2d_C$

The test shall be carried out by loading the specimen in increments in a suitable tensile testing machine. The strain rate shall have a minimum value of 1% per minute and should not exceed 3% per minute. At each increment of load, the extension shall be measured and a load-deflection curve drawn.

The tensile strength f_{Ct} is given by

$$f_{Ct} = \frac{F_u}{b^2} \tag{5.2}$$

The tensile modulus E_{Ct} is given by

$$E_{Ct} = \frac{F_u \, d_C}{w_u \, b^2} \tag{5.3}$$

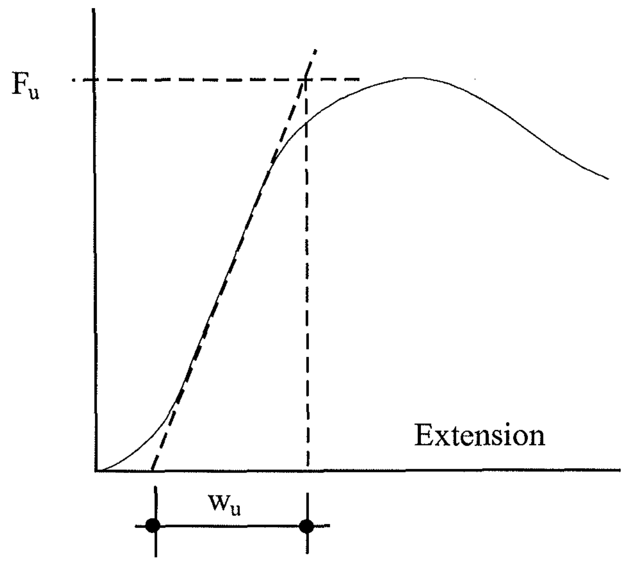


Fig. 5.2 Determination of the tensile modulus on the basis of the load-displacement curve.

The test report shall state whether failure was in adhesion or cohesion.

For design, but not for quality assurance, this test shall also be carried out on specimens heated to 80°C ± 3°C.

Notes:
1. For specimens which do not exhibit a well-defined ultimate load, F_u may alternatively be defined as the load at a specified relative deformation. For polyurethane foams, 10% relative deformation is an appropriate limit. For materials with a more rigid cell structure or of non-cellular structure, a lower value may be used.

2. The test at elevated temperature is usually carried out by heating the specimen to a temperature a little above 80°C and then carrying out the test immediately in a

conventional testing machine before it has cooled below 80°C.

3. The standard EN 1607 “Thermal insulating products for building applications – Determination of tensile strength perpendicular to faces” introduces the tensile test for insulating products. The strain rate in the test is 10 mm per minute within a tolerance of ±10% which is much higher than the rate in structural tests. The test method in EN 1607 is not applicable to measure the tensile strength and modulus of the core layer of structural sandwich panels.

5.2.3 Compression test on the core material

Specimens of square cross-section shall be prepared to the dimensions shown. The metal faces need not to be removed but, if the faces are removed, care shall be taken not to reduce the height of the core.

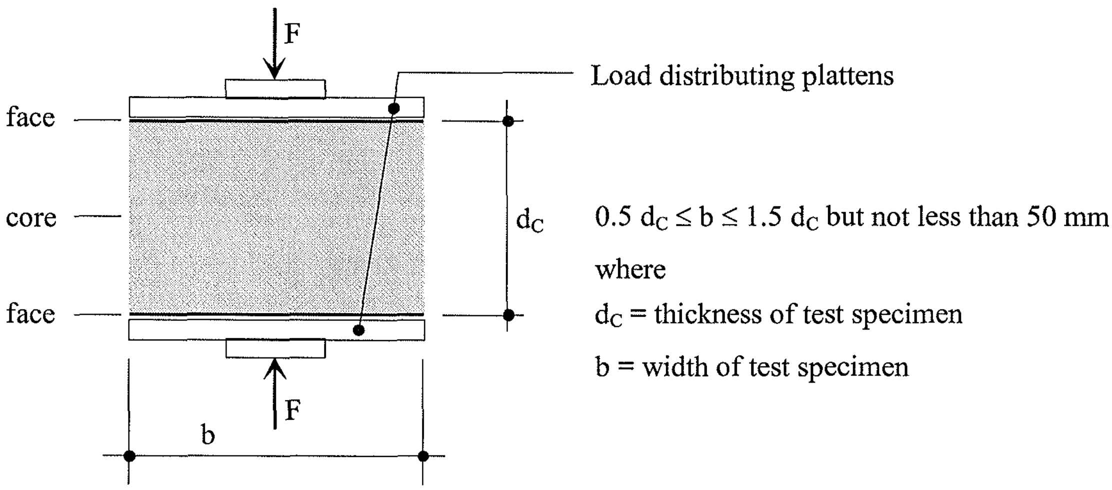


Fig. 5.3 Test arrangements in the compression test of the core layer.

Notes: 1. For panels with profiled faces, the specimens shall be cut from the predominant thickness as shown previously for tensile tests on core materials.

2. Better results are generally obtained with larger specimens and it is recommended that, where possible, specimens with width b as least 100 mm shall be used.

3. Cylindrical test specimens of minimum diameter 50 mm may also be used.

4. For specimens of mineral wool core material, rather larger specimens are generally required, i.e. $100\ \text{mm} \leq b \leq 2d_C$.

5. Apart from the dimensions of the test specimens and the use of fixed loading

platens, this test shall generally be in accordance with ISO 844 "Cellular plastics. Compression test for rigid materials" or any other standard relevant to other core materials.

The specimen shall be placed between the two parallel stiff loading plates of a suitable compression testing machine and loaded in increments. The strain rate shall have a minimum value of 1% per minute and should not exceed 3% per minute. At each increment of load, the displacement shall be measured and a load-deflection curve drawn.

The compressive strength f_{Cc} of the core material is given by

$$f_{Cc} = \frac{F_u}{b^2} \qquad (5.4)$$

The compressive modulus E_{Cc} of the core material is given by

$$E_{Cc} = \frac{F_u\, d_C}{w_u\, b^2} \qquad (5.5)$$

For design, but not for quality assurance, this test shall also be carried out on specimens heated to 80°C ± 3°C.

Notes: 1. See the figure 5.2. For specimens which do not exhibit a well-defined ultimate load, F_u may alternatively be defined as the load at a specified relative deformation. For polyurethane foams, 10% relative deformation is an appropriate limit. For materials with a more rigid cell structure or of non-cellular structure, a lower value may be used.

2. The standard EN 826 "Thermal insulating products for building applications – Determination of compression behaviour" introduces the compression test for insulating products. The strain rate in the test is $d_C/10$ per minute within a tolerance of ±25% which is much higher than the rate in structural tests. The test method in EN 826 is not applicable to measure the compressive strength and modulus of the core layer of structural sandwich panels.

5.2.4 Shear test on the core material

5.2.4.1 Short term loading

The shear strength and shear modulus of the core material shall be determined using the four-point bending test with a thin-faced specimen as shown.

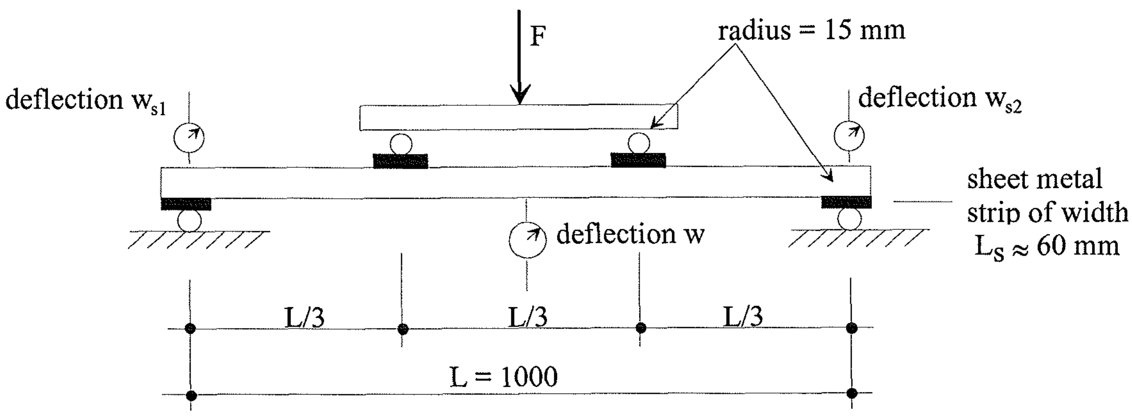


Fig. 5.4 Test arrangements in the shear test for a specimen with flat or lightly profiled faces.

The depths D and d_C, width b and the net metal thickness of both faces (t_1, t_2) of each test specimen shall be measured. For each test specimen, the shear modulus G_C shall be calculated from the slope of the straight part of the load-deflection curve $\left[\frac{\Delta F}{\Delta w}\right]$ as follows:

Flexural rigidity $B_S = \frac{E_{F1} A_{F1} E_{F2} A_{F2}}{E_{F1} A_{F1} + E_{F2} A_{F2}} e^2$ (5.6)

Bending deflection $\Delta w_b = \frac{\Delta F L^3}{56.34 B_S}$ (5.7)

Shear deflection $\Delta w_v = \Delta w - \Delta w_b$ (5.8)

Shear modulus $G_C = \frac{\Delta F L}{6 b e \Delta w_v} = \frac{L}{6 b e} \frac{\Delta F}{\Delta w_v}$ (5.9)

where E_{F1} = Young's Modulus of the top face
E_{F2} = Young's Modulus of the bottom face
A_{F1} = area of cross-section of the top face of the test panel (mm^2)
A_{F2} = area of cross-section of the bottom face of the test panel (mm^2)
e = depth between the centroids of the faces (mm)

Δw = deflection at mid-span for a load increment ΔF taken from the slope of the linear part of the load-deflection curve

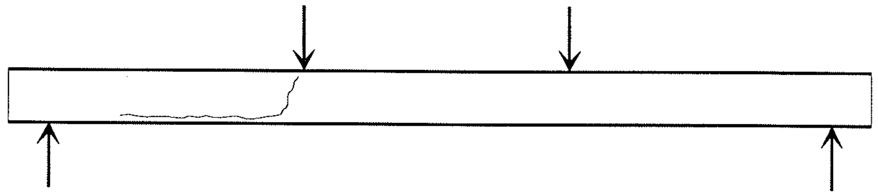
b = width of the specimen (mm)

The ultimate shear stress f_{Cv} of the core material shall be calculated from the maximum load attained in a specimen failing in shear as follows:

$$f_{Cv} = \frac{F_u}{2be} \tag{5.10}$$

where F_u = ultimate load carried by the specimen failing in shear.

Notes: 1. If the recommended span does not result in a shear failure, the span may be reduced in increments of 100 mm until a shear failure is obtained. Subsequent tests shall then be carried out at the reduced span. A typical shear failure is shown below.



2. To avoid large compressive deformations of the core on the supports compared to the deflection of the specimen, the span L shall not be reduced too much. For rigid plastic foams the span L shall conform to the limit

$$L \geq \frac{108 G_C d_C}{E_C \left(\frac{4L_s}{d_C} + 1 \right)}$$

where G_C = shear modulus of the core material.
E_C = modulus of elasticity of the core material
d_C = depth of the core of the specimen
L_s = support width

If the above condition is not satisfied, the compressive deformation of the core material at the supports may be critical and shall be measured (by gauges giving values w_{s1} and w_{s2} on the diagram). The deflection w to be used in the calculations shall then be modified by subtracting

$$\left(\frac{w_{s1} + w_{s2}}{2} \right)$$

3. As a guide, the condition that a shear failure rather than a wrinkling failure shall be obtained is

$$L < \frac{3 t_1 \sigma_w}{f_{Cv}}$$

where t_1 = the core metal thickness of the top face, excluding coatings
f_{Cv} = the shear strength of the core material
σ_w = the wrinkling stress of the top face (see section 5.2.6)

4. The width of the test specimen b shall be chosen to give flat faces without any stiffening ribs.

5. The width L_s of the sheet metal strips at the support and load points shall typically be 60 mm. This value may be increased, if necessary, in order to avoid local crushing of the core.

6. The loading rate shall be such as to result in failure between 5 and 10 minutes after the commencement of the test.

7. For the shear strength of panels with rigid foam core material, the four-point bending test will generally give more reliable values of the shear strength and stiffness than the alternatives, such as the double block method in EN 12090 (which generally gives low values for the shear modulus). However, for panels with mineral wool core material, or other slabstock, practical considerations may dictate the use of either a test on the full width of the panel with joints between the slabs or lamellas as in practice, or the use of an alternative shear test such as a lap test.

8. The width b of the test specimen may be higher for mineral wool core material.

9. Alternative methods of obtaining the shear modulus and shear strength of the core material which are in accordance with National or International Standards may be used. However, methods based on testing lapped joints in either tension or compression are not recommended as they often give inferior results to those obtained from the test which is recommended.

10. If different methods are used for design and quality control, the relationship between them shall be demonstrated.

5.2.4.2 Alternative shear test for thick mineral wool panels

The four-point bending test was originally developed for moderately thin plastic foam cored sandwich panels. When the thickness of the panel increases, a crushing failure tends to take place at the locations where the forces are transmitted to the specimen i.e. at the loading points and at the supports. In such cases, the test may be modified by using wider loading strips at these contact points. Nevertheless, very wide strips hinder the deflection and the formula (5.9) for the shear modulus G_C overestimates the value in these cases.

An alternative test method, which is suitable for thick mineral wool cored sandwich panels, is shown in the figure 5.5. In this test, relative thick platens are used to transmit the load F to the specimen. The length 'd' of the platens and the length of the specimen are chosen so that crushing failure at loading points does not take place. Instead, shear failure should occur in the narrow "shear areas". A suitable length for a shear area is 200 mm. Two displacements (w_1, w_2) approximately d_v = 100 mm apart are measured in a shear area to obtain the shear strain. The width 'b' of the specimen should be of the order of the height of the beam. No butt joints in the core material should be in the shear areas or close to them. The loading time to failure should be between 5 and 10 minutes.

The shear modulus is calculated as

$$G_C = \frac{\tau_C}{\gamma_C} = \frac{d_v}{2eb}\frac{\Delta F}{\Delta(w_2 - w_1)} \tag{5.11}$$

where

$\tau_C = \dfrac{F}{2eb}$	shear stress in the core
$\gamma_C \approx \dfrac{w_2 - w_1}{d_v}$	shear strain in the core
d_v	distance between the displacement transducers
e	distance between the centroids of the faces
b	width of the specimen
ΔF	load increment and
$\Delta(w_1 - w_2)$	corresponding deflection increment measured as the difference between w_2 and w_1

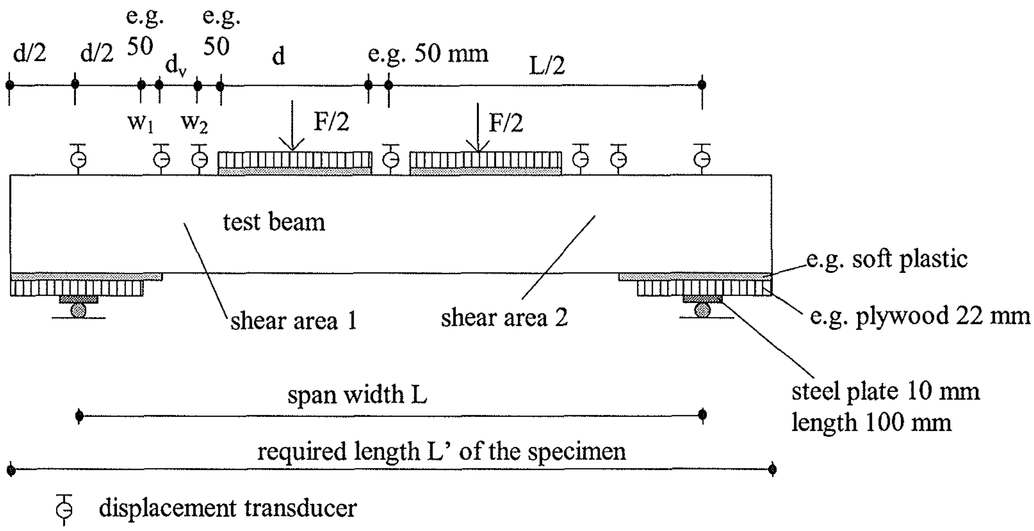


Fig. 5.5 Test arrangements in the alternative shear test.

Note: As an alternative a full-scale test is recommended as described in 5.2.5.

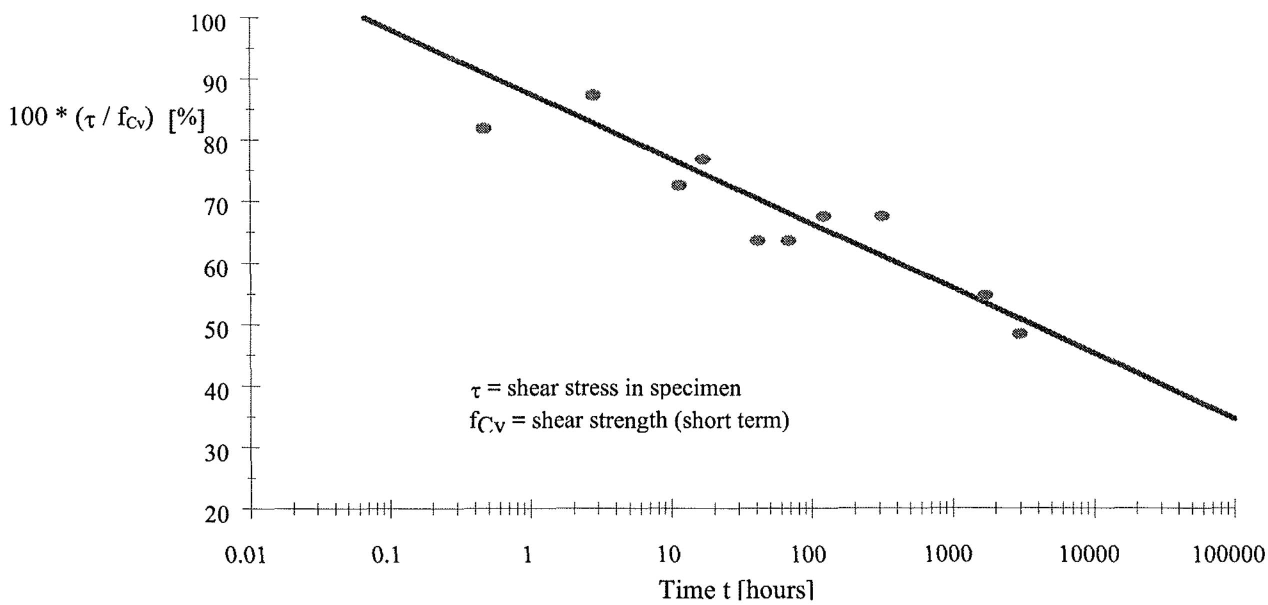
5.2.4.3 Long-term loading

In order to determine the long-term creep-rupture strength, long term loading tests corresponding to section 5.2.4.1 shall be carried out at a temperature of approximately +20°C in such a way that $n \geq 10$ samples fail within the time interval $0.1\ h \leq t \leq 10^3$ hours. Ideally, they should be equally spread out within this range.

Deformation measurements are not required.

Based on the test results, a regression line shall be drawn, as shown below, in order to show the relationship of the mean long-term shear strength to the initial shear strength (short-term strength) as a function of the loading time.

The long-term shear strength (e.g. for 2000 or 100 000 hours) shall be calculated using an expression based on the mean-value regression line.



for example:
regression line $100*(\tau / f_{Cv}) = -4.58911 * \ln(t) + 87.5289$

$100*(\tau / f_{Cv}) = -4.58911 * \ln(100\,000) + 87.5289 = 34.7\%$

Fig. 5.6 Determination of the long-term shear strength.

5.2.5 Test to determine the shear strength of a complete panel

This test offers a more reliable method of determining the shear strength of panels with a slabstock core where joints between the core elements may affect the shear properties.

For panels with discontinuous core materials, tests shall generally be carried out on the full cover width of the panel with joints in the core material in the worst arrangement that may arise in practice. The joint arrangement used in the tests shall be described in the test report.

The test shall be carried out by subjecting a short, simply supported panel to two line loads either equally spaced as shown below or applied at the 1/4 points or to air pressure caused by either a partial vacuum chamber test apparatus or air bags. The panel shall be loaded in increments up to failure and the failure load noted.

Notes:
1. If the panel has flat or lightly profiled faces, this test may be used as an alternative to that described in section 5.2.4.
2. This test is not necessary if the test described in section 5.2.6. is carried out on sufficiently small spans.
3. The span shall be sufficiently short to ensure a shear failure.
4. The loading rate shall be such as to result in failure between 5 and 10 minutes

after the commencement of the test.

5. When using air pressure loading, the load shall be measured by means of load cells, not air pressure.

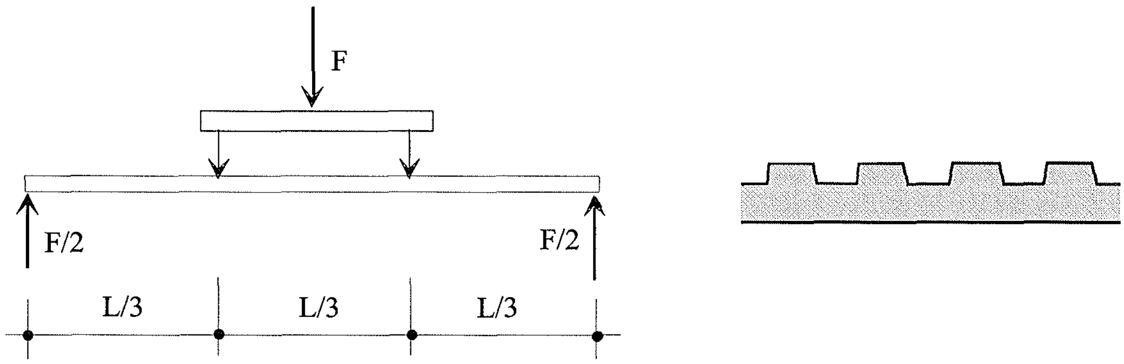


Fig. 5.7 Shear test with full-scale sandwich panels.

The depths D and d_C and the width B of the test panel and the net metal thickness of both faces of each test specimen (t_1 , t_2) shall be measured.

The load F_u at failure gives the shear strength of the complete panel including the contribution of both the core and faces.

For panels with flat or lightly profiled faces, it may be assumed that all of the shear force is carried by the core so that the ultimate shear strength f_{Cv} of the core is given by:

$$f_{Cv} = \frac{F_u}{2Be} \tag{5.12}$$

where F_u = ultimate load carried by the specimen failing in shear.
B = overall width of the sandwich panel
e = depth between centroids of the faces

5.2.6 Test to determine the bending strength and stiffness of a simply supported panel

This test is generally used to determine the wrinkling stress and, in such cases, the span L shall be sufficiently large to ensure a bending (wrinkling or face buckling) failure. The necessary span is dependent on several factors including the depth D. The following values are offered for guidance:

		D	$\leq$	40 mm	L = 3.0 m
40	<	D	$\leq$	60 mm	L = 4.0 m
60	<	D	$\leq$	100 mm	L = 5.0 m
		D	>	100 mm	L = 6.0 m

If the above values of span are found to give rise to shear failure, they shall be increased in increments of 1 metre until a bending failure is obtained.

The test shall be carried out by subjecting a simply supported panel to four line loads as shown below extending across the full width of the panel or to air pressure caused by either a partial vacuum chamber test apparatus or air bags. The panel shall be loaded in increments up to failure and the failure load noted.

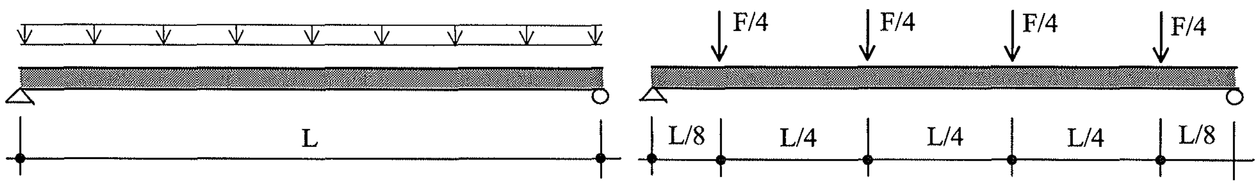


Fig. 5.8 Bending test using the air bag or partial vacuum chamber method resulting in an uniformly distributed load or hydraulic jacks causing line loads.

If line loads are applied to a profiled face, they shall be applied through timber or steel transverse loading beams together with timber loading platens placed in the troughs of the profile as shown in Fig. 5.9a. A layer of felt, rubber or other similar material may be placed between the loading platens and the panel in order to reduce the possibility of local damage. If the trough of the profile includes rolled-in stiffeners, the loading platens may be shaped appropriately as shown in Fig. 5.9b. The loads shall be maintained perpendicular to the panel throughout the test.

a)

b)

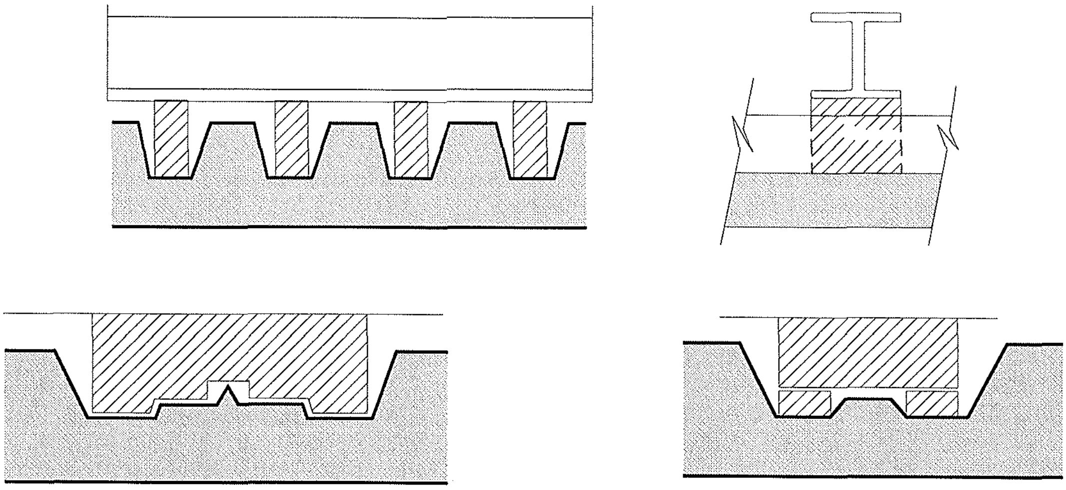


Fig. 5.9 Introduction of the line loads on the profiled face a) loading of the lower flange of the face profile and b) loading of the flange having internal rolled-in stiffeners.

5.2.6.1 Support conditions

In general, the support width shall be within the range 50 to 100 mm. Timber blocks may be used to avoid deformation of a side rib which does not contain foam.

The test panel may be attached to the supports through either the profile valleys or crests as in practice.

When this test is used to determine the wrinkling stress for use in design calculations, the support conditions shall be such as to apply no restraint to the rotation of the panel about the line of support. A suitable support detail is shown below.

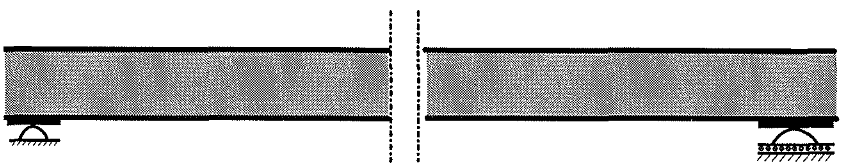


Fig. 5.10 Singly supported test specimens shall be free to rotate on the supports and free to move axially on one of the supports.

5.2.6.2 Test control

It is preferable to carry out this test by controlling the deflection rather than the load (i.e. using a constant deflection speed). However, either procedure may be used provided that the deflection speed does not exceed 1/50 of the span per minute at any time during the test. The loading rate shall be such as to result in failure between 5 and 10 minutes after the commencement of the test. The load shall be increased steadily until failure occurs. The failure load and the nature and location of the failure and the relationship between load and deflection shall be recorded.

Note: It is preferable to preface the formal test by a small preload.

5.2.6.3 Face properties

After completion of the test, the steel thickness without coatings, the profiling of the faces and the yield stress of each face shall also be determined and recorded.

5.2.6.4 Determination of the wrinkling stress

Although, in most cases, the design value of the wrinkling stress may be calculated, more favourable values of the wrinkling stress will generally be obtained by testing. In this case, the wrinkling stress shall obtained by determining the ultimate moment of resistance using the test procedure described above and then the face stress at failure shall be obtained by calculation.

For symmetrical or nearly symmetrical panels, it is essential to carry out this test with both orientations of the panel because the wrinkling stress may be greatly influenced by whether the face was at the top or bottom of the panel during manufacture. For symmetrical panels, the design shall be based on the least favourable wrinkling stress.

The wrinkling stress σ_w is only directly relevant for panels with flat or lightly profiled faces. For such cases, σ_w is given by:

$$\sigma_w = \frac{M_u}{eBt_1} \qquad (5.13)$$

where M_u = the ultimate bending moment recorded in the test, including the effect of the self-weight of the panel and the weight of the loading equipment

e = depth between centroids of the faces

B = width of the test specimen (sandwich panel)

t_1 = core metal thickness of the face in compression

If a partial vacuum chamber or air bag test apparatus is used in order to provide a uniformly distributed load over the surface of the specimen, the wrinkling stress is:

$$\sigma_w = \frac{(F_G + F_u)L}{8eBt_1} \qquad (5.14)$$

where F_G = self weight of the test panel

F_u = q_uBL = ultimate applied load

If the load is applied as four equal line loads at positions 1/8, 3/8, 5/8, 7/8 of the span, the wrinkling stress is:

$$\sigma_w = \frac{(F_G + F_u)L}{8eBt_1} \tag{5.15}$$

where F_G = self weight of the test panel
F_u = ultimate applied load plus the weight of the loading equipment

If the face under tension in this test is profiled, the wrinkling stress of the flat or lightly profiled face in compression may be determined using

$$\sigma_w = \frac{M_u - M_{F2}}{eBt_1} \tag{5.16}$$

where M_u = the ultimate bending moment recorded in the test including the effect of the self weight of the specimen and the weight of the loading equipment and
M_{F2} = the bending moment carried by the profiled face. The value of M_{F2} shall be determined by calculation (see Chapter 3 and Appendix A)

<u>Notes:</u> 1. The formulae for the calculation of the wrinkling stress are based on an analysis of elastic buckling with an empirical correction factor to take account of imperfections etc. The determination of the wrinkling stress by test takes full account of:

- non-homogeneity and anisotropy of the core
- non-linear material behaviour
- lack of flatness of the faces
- the true buckling and post buckling behaviour of the face material in compression.

2. The distribution of the bending moment at the ultimate limit state, $M_u = M_{Su} + M_{F2}$ has to be calculated according to the load distribution; uniformly distributed load or four line loads. If the test panel is loaded by an uniformly distributed load, the expressions given in 3.5.1 can be used to evaluate M_S and M_{F2} corresponding the ultimate load ($F_G + F_u$).

$$\sigma_w = \frac{M_{Su}}{eBt_t}$$

where $M_{Su} = (1 - \beta_q)M_u$

Typically, in the equation for σ_w, M_{F2} is much less than the bending resistance of the lower face profile.

5.2.6.5 Determination of the shear modulus of the core material

This bending test may also be used in order to determine a reliable value for the shear modulus of the core material.

If both faces of the test panel are flat or lightly profiled, the total deflection at the centre of the test panel may be divided into two parts:

$$w = w_b + w_v \tag{5.17}$$

where w = measured deflection at mid-span of the test panel
w_b = deflection due to axial deformation in the faces
w_v = deflection due to shear deformation of the core material

The shear modulus of the core may be determined from w_v .

If a partial vacuum chamber or air bag test apparatus is used in order to provide a uniformly distributed load over the surface of the specimen, the deflection increments at mid-span may be expressed as:

$$\Delta w_b = \frac{5}{384}\frac{\Delta F\,L^3}{B_S} \quad \text{and} \quad \Delta w_v = \frac{1}{8}\frac{\Delta F\,L}{G_C\,A_S} \tag{5.18a,b}$$

The shear modulus of the core may then be expressed as:

$$G_C = \frac{\Delta F\,L}{8A_S\left(\Delta w - \Delta w_b\right)} = \frac{L}{8A_S}\frac{\Delta F}{\Delta w - \Delta w_b} \tag{5.19}$$

In these expressions, the deflection increment Δw is taken from the linear part of the load deflection curve and ΔF is the corresponding increment of the applied load and:

$$B_S = \frac{E_{F1}\,A_{F1}\,E_{F2}\,A_{F2}}{E_{F1}\,A_{F1} + E_{F2}\,A_{F2}}\,e^2 \quad \text{and} \quad A_S = e\,B \tag{5.20a,b}$$

If the load is applied as four equal line loads at positions 1/8, 3/8, 5/8, 7/8 of the span, the expressions for the deflection components at mid-span become:

$$\Delta w_b = \frac{41}{3072}\frac{\Delta F\,L^3}{B_S} \quad \text{and} \quad \Delta w_v = \frac{1}{8}\frac{\Delta F\,L}{G_C\,A_S} \tag{5.21a,b}$$

The calculation procedure is otherwise unchanged.

5.2.7 Test to determine the creep coefficient φ_t

A single test shall be sufficient to determine the creep coefficient for a particular core material.

The test shall be carried out on a complete panel of span equal to that used for the bending test in section 5.2.6. The core thickness shall be the maximum in the test series.

The test shall be carried out under a constant load which shall be sustained undisturbed for a minimum of 1000 hours. During this time, the deflection shall be regularly monitored to give a continuous relationship between deflection and time. The load used for the creep test shall correspond to approximately 30% of the average load for shear failure at ambient temperature determined from the tests carried out according to section 5.2.4.

The test shall be carried out by subjecting a simply-supported panel to uniformly distributed dead load. During the placing of the load, the panel shall be propped from below in such a way that the propping can be removed quickly and smoothly in order to initiate the test. Deflection measurements shall commence the instant that the full load is applied.

Alternatively, the initial deflection may be calculated from the slope of the load deflection curve obtained during the corresponding bending test in section 5.2.6. In this case, the dead load may be applied more gradually in the conventional manner.

The creep coefficient for the core of a thin-faced sandwich panel shall be determined using the expression:

$$\varphi_t = \frac{w_t - w_0}{w_0 - w_b} \qquad (5.22)$$

where w_t = the deflection measured at time t,
w_0 = the initial deflection at the time t = 0 and
w_b = the deflection caused by the elastic extension of the faces.

The deflections caused by the bending and shear deformations of a sandwich panel with strongly profiled faces can not be separated in the expression for the deflection because the distribution of the bending moment into the sandwich component M_S and the flange components M_{F1}, M_{F2} depends on the shear stiffness of the core (see Section 3.5 and Appendix A). Therefore, the creep coefficient shall to be evaluated iteratively on the basis of the measured deflections as a function of the time.

On the basis of the results of the tests within $t \geq 1000$ h, creep coefficients may be extrapolated using a semi-logarithmic diagram in order to determine the creep coefficients required in the design (i.e. $\varphi_t(t = 2000\ \text{h})$, and $\varphi_t(t = 100\,000\ \text{h})$, see Appendix B).

Notes: 1. The creep test may alternatively be carried out on a complete panel using a uniformly distributed load.

2. More exact and usually advantageous values of the creep coefficient φ_t will be obtained if the duration of the test is extended to (say) 2000 hours.

3. The development of new core materials may give rise to increased susceptibility to creep and may therefore require longer tests.

4. The load used for creep tests is not unduly critical and similar results will be obtained for any load in the range 30 % to 40% of the failure load.

5.2.8 Interaction between bending moment and support force

5.2.8.1 Wrinkling stress at an intermediate support

The test arrangement for the interaction between bending moment and support reaction force shall be a single span panel subject to a line load. This is often referred to as the "simulated central support test" because it simulates the conditions in the central support of a two-span beam. In the test, the complete panel with the full width B shall be used.

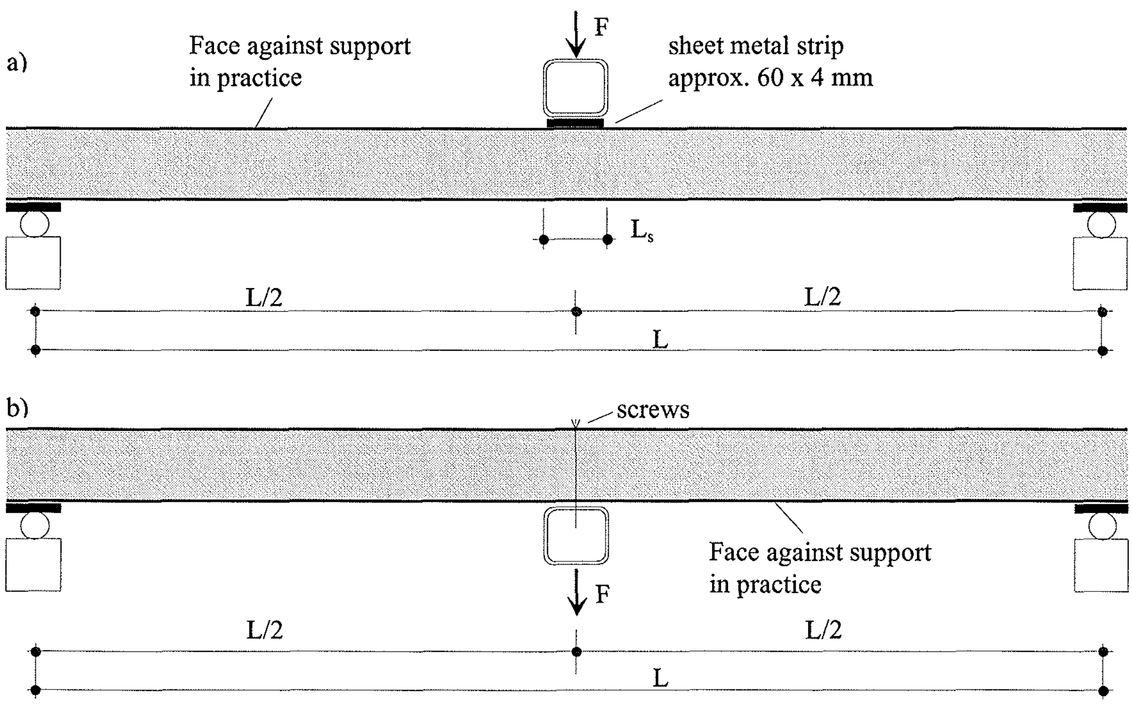


Fig. 5.11 Simulation of the interaction at intermediate supports of continuous sandwich panels a) for pressure loads and b) for uplift loads.

In order to determine the wrinkling stress at an intermediate support it is necessary to carry out two types of test:

(a) tests which simulate downward load

(b) tests which simulate uplift load

Both of these tests shall be carried out on a full panel width.

It is important that the span (which may in some cases be 5 metres or more) shall be sufficient to ensure that:

- for tests (a), the compressive stress between the panel and the support (under the line load) at the time of wrinkling failure is less than the compressive strength of the panel core material

- and for test (b), the forces in the fasteners at wrinkling failure of the panel are less than their design values.

This ensures that, for the prototype panel, all failure modes (wrinkling of the face, compressive failure of the core and tensile failure of the connection) are designed for approximately equal levels of safety.

Note: If the test is carried out on a shorter specimen than that described above, the failure mode is likely to be dominated by core crushing and a conservative value of the wrinkling stress will be obtained.

The wrinkling stress σ_w is only directly relevant for panels with flat or lightly profiled faces. For such cases, σ_w is given by:

$$\sigma_w = \frac{F_u L}{4 e B t_1} \qquad (5.23)$$

where F_u = ultimate load carried by the specimen
e = depth between centroids of the faces
B = width of the test specimen (sandwich panel)
t_1 = core metal thickness of the face in compression

5.2.8.2 Remaining bending resistance at an intermediate support

If the load-deflection curve is of type (a), the attainment of maximum bending moment at an internal support corresponds to a serviceability limit state. Furthermore, a non-zero rest moment may be determined and incorporated into the calculations at the ultimate limit state. If the load-deflection curve falls away suddenly, as shown in (b), the attainment of maximum bending moment at an internal support may be better considered to correspond to the ultimate limit state.

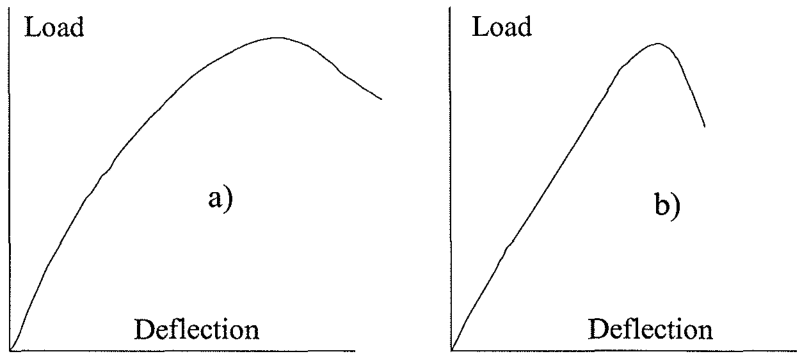


Fig. 5.12 Load-deflection curve of sandwich panel with a remaining (curve a) and a vanishing (curve b) bending resistance at the intermediate support.

A suitable value for the non-zero rest moment M_{rest} may be determined from a load-deflection curve type (a) by subtracting the elastic component of deflection and choosing M_{rest} as the moment on the drooping part of the curve corresponding to a "plastic hinge" rotation of 3° .

5.2.9 Test on a two-span panel

This test may be used as an alternative to the test described in section 5.2.8 in order to investigate the interaction between the bending moment and reaction force at an internal support.

In carrying out this test, one of the arrangements shown below shall be used, following the principles described in section 5.2.8.

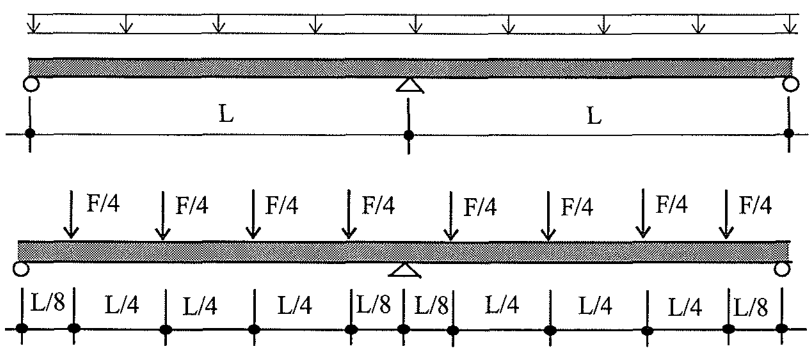


Fig. 5.13 Bending test arrangements of two-span sandwich panels a) using air bag or partial vacuum chamber method and b) using line loads caused by hydraulic jacks.

Care should be taken to identify the onset of permanent deformations corresponding to buckling or yielding of the face or crushing of the core at the internal support. This will generally require that the panel is unloaded after certain increases in the applied load in order to determine the residual deflection.

Fastenings at the end and intermediate supports have some flexibility, especially against the uplift loads caused by wind suction and temperature differences between the faces. Thus, the calculations based on immovable supports overestimate the stresses caused by the temperature differences and are approximate for the stresses caused by wind suction loads. However, the flexibility of the fastenings increases the deflections of the panel.

The flexibility of the fastening system, and its influence on the bending moment and shear force distributions and on deflections, shall be determined experimentally. In such tests, mechanical loading can be used in the way shown above. The panel shall be supported by the fastenings against the load, thus, the fastenings will be loaded by tensile forces. The flexibility of the fastenings can also be tested using loading arrangements which causes a temperature difference between the faces of the panel as shown ($T_1 > T_2$) in the figure. Results obtained from the measurement of the support reactions and the deflections in the spans and at the supports make it possible to evaluate the flexibility of the fastening systems and their influence on the stresses and deflections of the panel system. A possible outcome from the analysis may be a spring coefficient, which can be used to model the flexibility of the fastening system against negative support reactions.

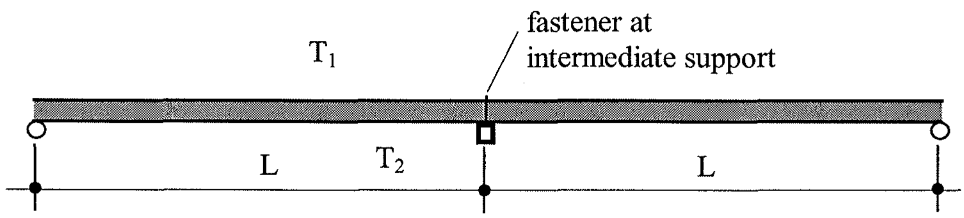


Fig. 5.14 Testing of the influence of the flexibility of the fastening system on the stress distribution and resistance of two-span sandwich panel using temperature loading.

5.2.10 Support reaction capacity

As an alternative to basing the support reaction resistance at an internal support solely on the compressive strength determined in section 5.2.3, this test may also be used to determine the support reaction resistance at internal support.

A suitable test with which to determine the capacity with respect to an end support reaction is shown in the figure 5.15. The end support reaction capacity is defined as

$$F_{R1} = \frac{L_2}{L_1 + L_2} F \qquad (5.24)$$

where F is the maximum load measured in the test or the load corresponding to a deflection w = 0.1 e (where e is the distance between the centroids of the faces of the panel) if this is lower than the maximum load and on the rising part of the load deflection curve as shown in Fig. 5.16.

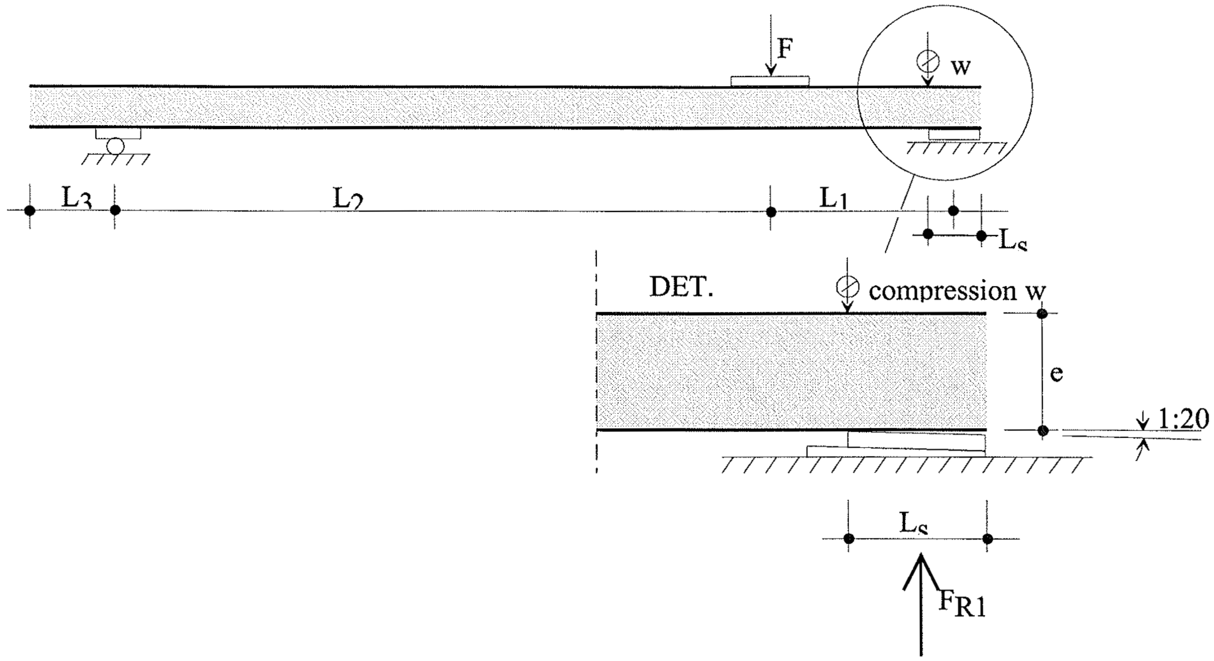


Fig. 5.15 Test arrangements for the determination of the end support reaction capacity.

The dimensions L_1, L_2 and L_3 shall be chosen in such a way that the failure mode of the specimen is a compression failure at the support. If the failure mode is a shear failure between the loading plate (F) and the support plate (L_s), the end support reaction capacity may be taken to be the value of the support reaction force at the time of shear failure.

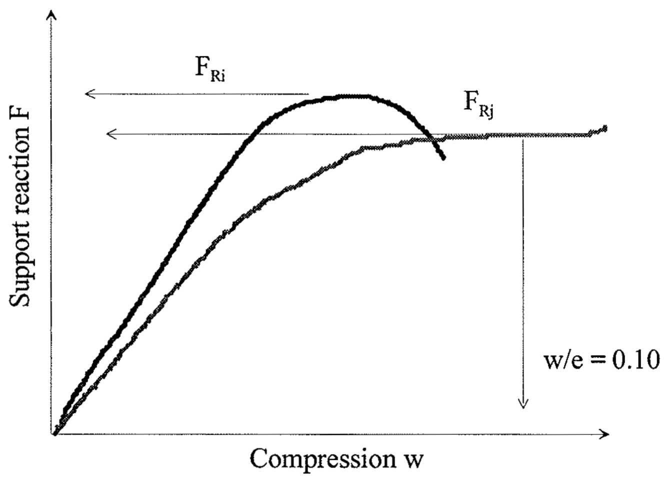


Fig. 5.16 Definition of the ultimate load from the load-deflection curve in an end support reaction test.

The support reaction capacity at an intermediate support is defined as:

$$F_{R2} = f_{Cc}\, B\left(L_s + k\,e\right) \tag{5.25}$$

where $$k = 2\frac{F_{R1} - f_{Cc}\, B\, L_s}{f_{Cc}\, e\, B} \tag{5.26}$$

In the expressions, B is the width of the test specimen (sandwich panel).

The compression strength f_{Cc} of the core shall be determined according to section 5.2.3.

The loading rate shall be such as to result in failure between 5 and 10 minutes after the commencement of the test.

5.2.11 Test for walkability

There are two parts to this test. Part 1 provides information regarding the safety and serviceability of roof and ceiling panels with respect to a single person walking on the panel both during and after erection. Part 2 provides information regarding the durability of the panel with regard to repeated foot traffic during the life of the panel.

Notes: 1. Most types of sandwich panel are unsuitable for use as walkways or working platforms without some form of protection from repeated foot traffic.

2. Part 1 is sustained for occasional access. Part 2 is provided access on a regular basis but not frequent.

Part 1:

The tests shall be carried out on single span panels of full width. The span shall be the largest envisaged in practice. The load shall be applied through a timber block measuring 100 x 100 mm. In order to avoid local stresses, a 10 mm thick layer of rubber or felt may be placed between the timber block and the metal skin of the panel.

A load of 1.2 kN shall be applied at mid-span on the edge rib or on the edge of a flat panel.

There are three possible outcomes from this test:

(1) If the panel carries the applied load without permanent visible damage, there are no access restrictions onto the roof or ceiling either during or after erection.

(2) If the panel supports the load but with permanent visible damage then measures shall be taken to avoid damage during erection (e.g. walking boards). Furthermore, there shall be no provision for access to the roof after building work is completed.

(3) If the panel fails to support the load then it shall not be used in practice.

For multi-span panels, the maximum allowable span indicated by the above test may be increased by 25 %.

Part 2:

This test requires a minimum of 10 tensile specimens with a size of 100 mm x 100 mm according to section 5.2.2 with the faces intact. 50% of the specimens shall be tested to failure in tension according to section 5.2.2. The remaining 50% of the specimens shall be first subject to 250 cycles of compressive loading with a stress level varying between limits of zero and 0.08 N/mm^2 applied as a central load 0.45 kN over a loading area of 75 mm x 75 mm. The rate of testing is not critical but shall not exceed 1 Hz. These specimens shall then be tested to failure in tension as previously.

If the average of the tensile strength results obtained after cyclic loading falls below 80% of the average value obtained without cyclic loading, the panels shall be considered to be unsuitable for regular access without added protection.

Note: The Part 2 test for walkability is still under debate within CEN. The above Recommendation may, therefore, be subject to subsequent change.

5.2.12 Blistering test

Blistering may affect the outer faces of dark coloured foam-filled panels exposed to sunlight or other panels subject to relatively high temperatures. Blisters are usually associated with imperfect bond between the core and the faces or voids or imperfections introduced during manufacture.

The test shall be conducted by heating the outer (or otherwise exposed) face of the panel up to a uniform temperature of 85°C ± 3°C and then maintaining the panel at that temperature for two hours. The panel shall then be carefully examined for visible blisters before it is allowed to cool. The test is successful if no visible blisters are observed.

For continuously produced panels, it is sufficient to test a specimen of width equal to the full width of the panel and minimum length 1 metre.

For panels which are individually moulded by an injection process or manufactured by other methods which result in a non-uniform distribution of foam, the complete panel shall be tested.

Notes: 1. Once the foam formulation and method of manufacture have been shown to be sound with respect to blistering the test shall be repeated as part of the quality control.

2. Batch-produced panels tend to be more prone to blistering than those manufactured on a continuous production line and this shall be borne in mind when agreeing quality control procedures.

3. When, after some months of continuous production, consistent properties have been established and the design has been shown to be basically sound with respect to blistering, the frequency of blistering tests for quality control may be reduced on the basis that this property will be controlled by tensile bond tests.

5.2.13 Thermal shock test

This test is a development test.

The panel to be tested shall be a little over 6 metres long and shall be fixed by the specified fastenings to a rigid steel frame arranged so that membrane forces or fixing moments are avoided and to give two equal spans of 3 metres.

The temperature of the outer face shall then be raised in steps of 10°C up to a maximum of 80°C while the other face is maintained at ambient temperature (≈20°C). When the panel has stabilized at the maximum temperature it shall be suddenly cooled by spraying with cold water (≤ ambient temperature).

During the test, any visible damage (wrinkling, buckling, blistering, etc) shall be recorded.

Note: This test does not provide any information regarding the design strength of the panel. However, it is not unusual for otherwise satisfactory panels to fail this thermal shock test.

5.2.14 Test for thermal stability

The dimensional stability of the foam core shall be tested at 80°C and -20°C.

The test specimens shall be 100 mm x 100 mm and of the full thickness of the panel with the faces removed. Their thickness shall be precisely measured. One set of specimens shall be placed in a heating chamber and the other in a freezing chamber and the requisite temperature maintained for three hours. They shall then be allowed to return to ambient temperature and the thickness re-measured. The difference in thickness before and after thermal attack shall be less than 3% as a result of the high temperature and 1% as a result of the low temperature.

5.2.15 Tests for controlling foam reaction and the reaction of adhesives

Predetermined quantities of the foam or adhesive components, according to the recipe, should be mixed in a disposable container and the phases of the reaction timed and recorded, e.g.:

- starting time
- mixing time or stirring time
- cream time
- rising time
- rate of setting or tack-free time
- end of active bonding.

Specimens should be taken from the specimen of free rise foam for the determination of:

- density
- compression strength
- behaviour under temperature attack (+80°C, -20°C - see section 5.2.14)

The evaluation of these data, together with the size and appearance (cell structure) of the foamed specimen indicate a number of characteristics of the foam which influence the laminating process. The successful use of this information is a matter of experience.

Note: These tests are usually carried out before the commencement of a shift. Because free rise is allowed, the results obtained bear no direct relationship to the characteristics of the cores of the panels to be produced. They do, however, provide the engineer with qualitative information which is essential to the successful operation of the plant.

5.2.16 Adhesive bond between faces and prefabricated core material

The adhesive bond between the faces and the core is vital to the satisfactory performance of the panel and it is essential that every attention is given to the long term properties of the completed joint. The properties of the adhesive are fundamental and these must be agreed by the supplier and the manufacturer of the panel. The following shall be carefully controlled during manufacture of the panels:

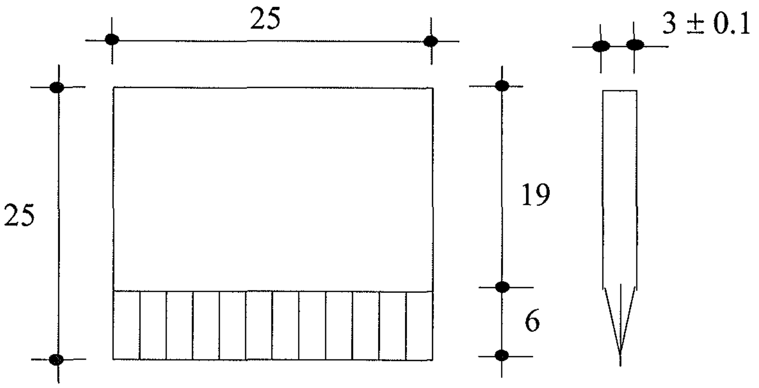
- preparation of surfaces
- temperature and humidity
- quantity and uniformity of application of adhesive
- curing pressure and duration

A suitable test with which to control the adhesive bond is the wedge test carried out in accordance with ASTM D3762. The test specimens are fabricated from two strips of the face material with a width of 20 mm and a length of 100 mm. These strips may either be cut from the coil material to be used in the manufacturing process or, in the case of panels produced using autoadhesive bonding, from the manufactured panels. When cutting from completed panels, the core material shall be carefully removed without damaging the bonding layer with the surface of the metal face. The strips of face material are then bonded together as shown in the Figure 5.17.

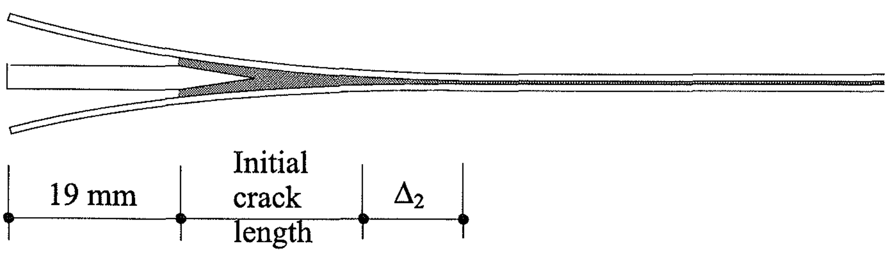
The wedge is pressed between the two faces, thus causing an initial crack whose length shall be measured. The wedge should be loaded with a force of 3N. The specimen is then immersed for 24 hours in water heated to 70°C.

The initial crack should not extend for more than approximately 20 mm and should not grow by more than approximately a further 20 mm after immersion for 24 hours in heated water. The crack should appear within the adhesive material itself not in the bond with the face material.

Dimensions of the aluminium or stainless steel wedge, measures in mm



Wedged crack extension specimen



Δ_2 = crack growth after exposure

Fig. 5.17 Dimensions of the specimen and test arrangements in adhesive bond test.

Notes: 1. The small force is not included in the ASTM D3762 method and is needed here to compensate for the use of thin plates in sandwich construction.

2. This modified ASTM method has not been validated for plate thickness less than 0.5 mm.

In addition regular tensile tests should be carried out in accordance with section 5.2.2 in order to evaluate the strength of the joint in the manufactured panels.

5.2.17 Impact loading tests

Impact loading tests are according to ISO 7893 and ISO/DIS 7893. They are generally applicable to internal and external wall panels located where they may be exposed to impact loading, for example at ground floor level in buildings accessible to the public. They are intended to simulate the performance of the panels when subject to accidental or intended impacts, for example, objects thrown or kicked against them or accidents involving people or items pushed or falling against them.

The type and magnitude of impact will depend on the building use and this makes it difficult to recommend a universal test(s) suited to all building uses.

The tests shall consist of single impacts applied in several locations.

Where impact loading tests are necessary, the tests shall be carried out with a hard body and/or a soft body impactor, whichever corresponds most closely to the actual application (ISO 7892).

Impact loading tests shall be carried out on an assembly of at least two panels incorporating a vertical joint. The width of the panel assembly shall be not less than 2 m. The height of the panels shall be as close as possible to the maximum design height, but not less than 2 m. The panels shall be secured at their upper and lower ends to a suitable support frame using the attachment method and fasteners according to the end-use condition. The support frame shall be of rigid construction (a flexible support frame will, by elastic deformation, absorb some of the impact energy so reducing the impact sustained by the panel assembly).

The impactor shall be suspended down the face of the panel by a wire at least 3 metres in length. The position of the impactor shall be adjusted so that it rests against the face of the panel at the point to be tested. The test shall be conducted by raising the impactor in pendulum fashion to the required height and then releasing it. The test shall be carried out in one or more positions on the face of the assembly which are considered to the most critical.

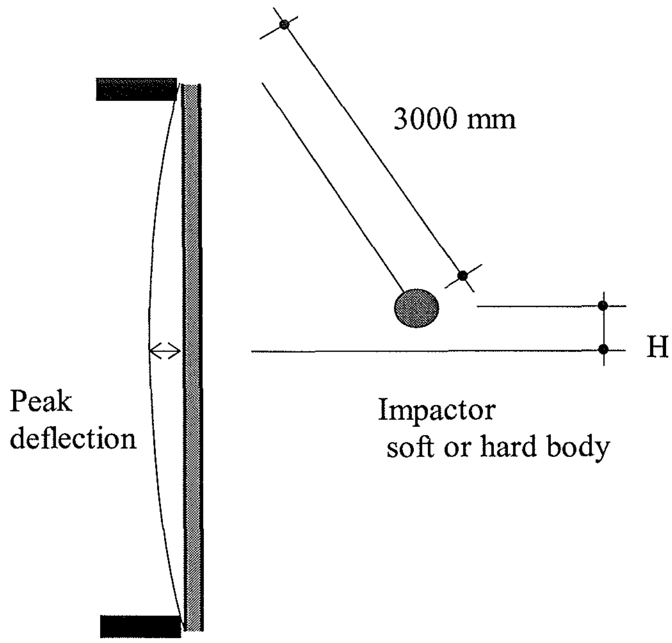


Fig. 5.18 Principles of impact loading test.

These tests should demonstrate that, when subject to impact loads, the performance of the panel is such that:

- there is no risk to people either inside or outside the building
- there is no reduction in the structural safety of the building
- there is no risk of panels becoming detached from the supporting framework

Dents, or other superficial damage are acceptable subject to aesthetic considerations.

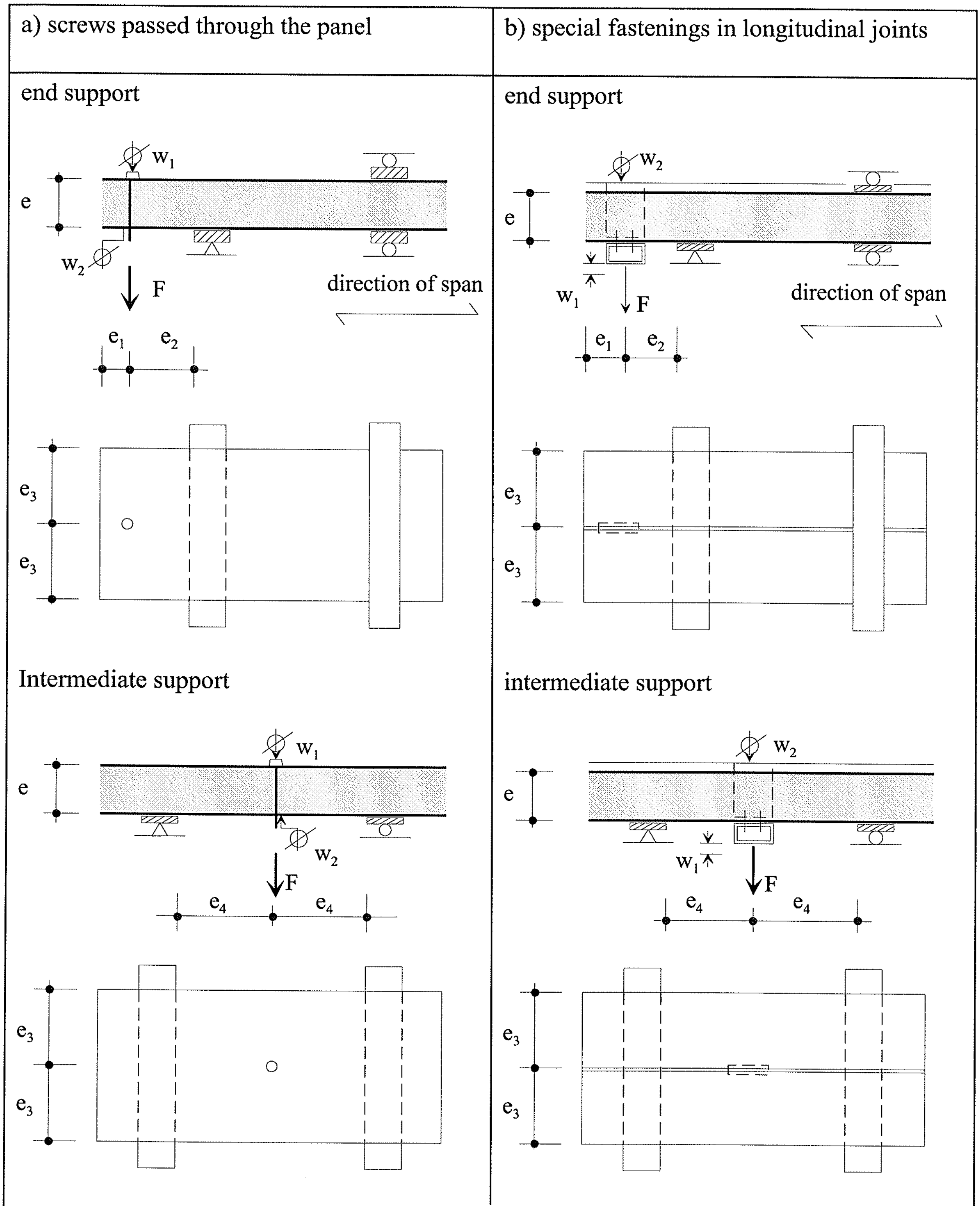
In the absence of other guidance the following data may be used:

The hard body impactor shall comprise a steel ball of 67.5 mm diameter and weight approximately 1.0 kg

The soft body impactor shall comprise a spherical canvas bag of diameter 400 mm filled with 3 mm diameter glass spheres to give a total weight of approximately 50 kg.

The test shall be carried out by applying the impactor at increasing heights until failure is deemed to have occurred. The initial impactor height H shall be 0.30 m and this shall be increased by 0.30 m between each impact. At each impact, it is important to catch the impactor so that only one impact from each height is allowed to occur. The panel assembly shall be inspected for visible damage after each impact. The failure criteria may be one of the following:

- first visible damage
- maximum allowable deflection
- limit of integrity



e_1 corresponds the minimum edge distance defined by the manufacturer
$e_2 \geq \max\{e, 100\text{ mm}\}$
$e_3 \geq B/4$ where B is the overall width of the panel
$e_4 \geq \max\{e, 100\text{ mm}\}$

Fig. 5.19 Testing of tensile resistance of a fastening a) screws passed through the panel and b) special fastenings placed in longitudinal joints between the panels.

Note: The German guidelines, in addition to 10 static tests on each arrangement (with average resistance $F_{0,static}$), also require 5 dynamic tests to be carried out with the following load limits:

Upper load limit:	F_{max}	$= 0.5\ F_{0,static}$
Lower load limit:	F_{min}	$= 0.1\ F_{max}$
Number of load cycles:	5000	

After cyclic loading, the connection should be loaded statically to failure and the load capacity so attained should be not less than 1.3 F_{max}. If this requirement is fulfilled, then:

The design capacity $F_{t,Rd} = 0.5\ F_{R,u}$ (5% fractile with a material factor $\gamma_M = 2.0$)

Or, alternatively, the design of the connections should be based on :

$$2.0\ S_L + 1.3\ S_T \leq F_{R,u}$$

where S_L is the tension due the load actions
S_T is the tension due to the temperature actions

In all other cases and if, during the cyclic loading tests, damage is observed (for example tearing of the metal at the edges of the holes in the faces) then the value determined by an expert should be adopted. In this case, the design of the connections should be based on at least:

$$3.0\ S_L + 2.0\ S_T \leq F_{R,u}$$

In all tests, any noticeable permanent deformations should be recorded.

5.2.18.2 Testing of fasteners in shear

If the distance between the end of the panel and the fastener is less than 30 mm, separate tests shall be carried out for fastenings at the end of a panel and those at an intermediate support.

For each screw type, the tests shall be carried out for the largest panel depth to be used in practice.

The load or displacement shall be increased monotonically up to the ultimate load. The use of a displacement-controlled testing machine is to be preferred. The shear displacement at the fastening shall be measured in order to determine the shear stiffness and the shear displacement corresponding to the maximum load in the fastening. The loading rate shall be such as to results in failure between 5 and 10 minutes after the commencement of the test.

The ultimate load is defined to be the smallest of:

- the maximum load recorded during the test

- the load at which the first fall in load is observed on the load deflection curve
- the load corresponding to a displacement of 3 mm if this occurs on the rising portion of the load deflection curve.

The ultimate failure load and the mode of failure (pull through, pull out, failure of fastener itself etc.) shall be recorded.

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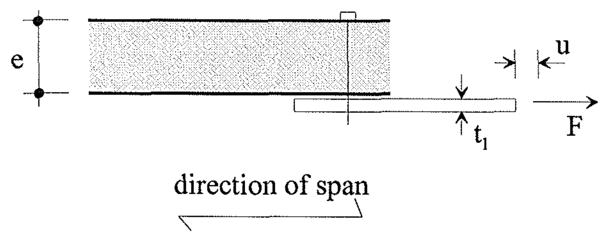
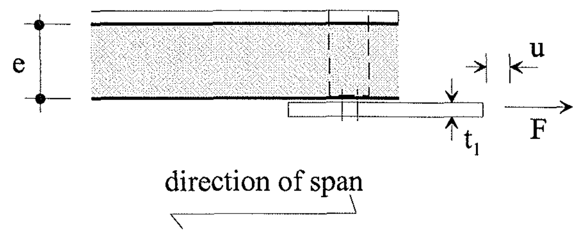
<tr><td>a) screws passed through the panel</td><td>b) special fastenings in longitudinal joints</td></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td colspan="2">for shear loads at an end of the panel, e_1 corresponds the minimum end distance and
for shear loads inside the ends of the panel, $e_1 \geq e$
t_1 is the thickness of the supporting structure</td></tr>
</table>

Fig. 5.20 Testing of shear resistance of a fastening a) screws passed through the panel and b) special fastenings placed in longitudinal joints between panels.

5.2.18.3 General notes with regard to the testing of fasteners

The above procedures may be used to give general guidance when local tensile and/or shear loads are transferred to a panel through other similar types of fastening.

The test arrangements described in Ref. /ECCS 1993a/ may require modification in order to accommodate the fixings used in sandwich panel construction. When this is the case, the test set up must be representative of the real situation. Particular attention should be given to:

- type of loading
- thickness of supporting member
- fastener head and washer
- quality of core
- quality of face material and
- end and edge distances

The following test is an additional requirement that is specific to sandwich panels.

5.2.18.4 Repeated bending test on a fastener

The fastener shall be tested as a cantilever of length l equal to the depth of the panel to the point of connection. The details of the test arrangement shall correspond as closely as possible to those in the actual structure, in particular, it shall be fixed into material of the same thickness 't' as the supporting member in the actual structure. The head of the fastener shall not be restrained during the test.

The fastener shall be tested in a displacement-controlled testing machine and shall be subjected to the following unilateral displacement spectrum where 'u' is the maximum lateral displacement calculated at the point of attachment

(1) 20000 cycles at 4/7 u
(2) 2000 cycles at 6/7 u
(3) 100 cycles at u

The frequency shall not exceed 5 Hz.

After application of the above displacement spectrum, the fastener shall be tested to failure in tension.

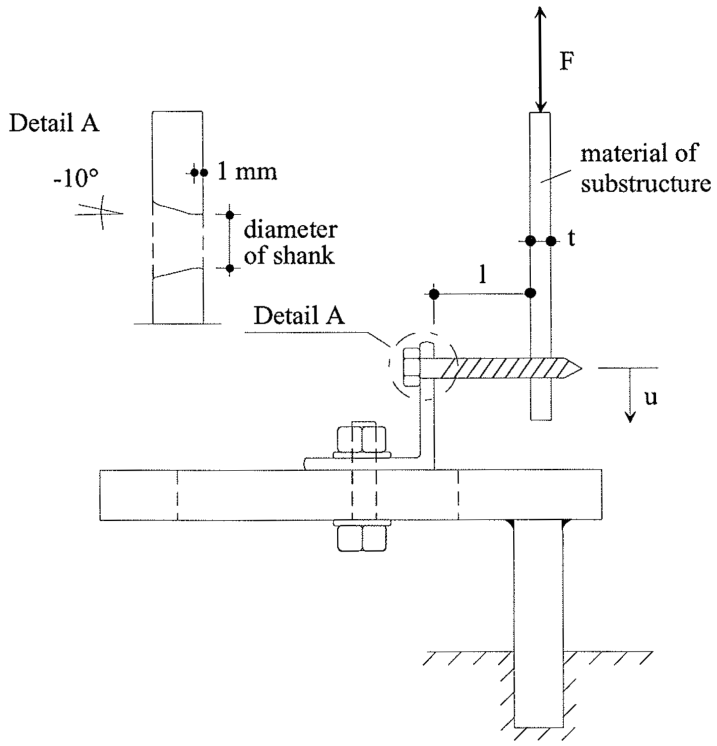


Fig. 5.21 Test arrangement for the repeated bending test of a fastener.

Notes: 1. The arrangement shown represents a suitable arrangement for this test.

By varying l and u, this test can be used to determine the maximum allowable value of u for different panel thicknesses. In systematic testing of a particular fastener, the thickness t of the sub-structure material shall also be varied because a greater thickness results in increased bending restraint and therefore reduced deformation capacity. Further details of the interpretation of this test are given in Appendix E.

2. The recommended displacement spectrum is based on a procedure that is accepted in Germany. This, in turn, is based on the following temperature variations during a 50 year working life:

number of cycles	ΔT
20000	40°C
2000	60°C
100	70°C.

3. Detail A is designed to allow the head of the screw to rotate freely during the test. If this detail does in practice not allow free rotation, thus putting the screw into double curvature, detail A may require modification.

5.2.19 Tests to determine other aspects of physical behaviour

Tests in addition to those described in these recommendations will generally be necessary in order to determine other aspects of the behaviour of the panel, e.g.

- corrosion resistance
- thermal conductivity
- resistance to humidity
- tightness of joints between panels
- resistance to fire
- reaction to fire
- acoustical behaviour

The required tests will normally be specified in the relevant Building Regulations.

Notes: 1. Thermal conductivity. The thermal performance of metal sheet faced sandwich panels shall be determined by either measurement or calculation. Methods to determine the thermal resistance of core materials are given in Refs. /prEN13162, 13163, 13164, 13165/.

With many rigid-foams, in particular PUR, the measured values of thermal conductivity need to be increased by an ageing factor in order to take account of the slow exchange of cell gases with air.

For PUR, panels which have vapour-tight faces and edge details, the ageing factor may be taken to be 10%. For panels with lesser degrees of vapour tightness, the ageing factor may vary between 10 and 50% depending on the details.

2. Fire tests. The tests required in order to determine the behaviour in fire vary considerably between the different countries and the results given by the different national test procedures are often not comparable. European harmonisation of fire test procedures is under consideration but will probably take several years.

5.2.20 Recording and interpretation of test results

For each test series, formal documentation should be prepared giving all the relevant data so that the test series could be accurately reproduced. In particular, in addition to the results of the tests, the specimens should be fully and accurately described in terms of dimensions and material properties. Any observations made during the tests should also be recorded.

Note: The following may serve as a check list for the information to be recorded:

- Date and time of manufacture
- Method of manufacture and orientation of panel during manufacture (e.g. which face was uppermost, which was the leading edge during continuous foaming, etc.)
- Date and time of testing
- Conditions during testing (temperature and humidity)
- Method of loading and details of instrumentation
- Support conditions (number and length of spans, width and details of supports, number and details of connections to supporting structure etc.)
- Orientation of panel during testing
- Properties of face material (thickness, yield stress, geometry etc)
- Properties of core material (density, strength, moduli etc)
- Measurements made during testing (load, deflection readings, temperature etc)

The analysis of the results of a test shall be based on the measured dimensions and material properties of the test specimens rather than the nominal values assumed in the design.

For the following failure modes:

- Failure of the profiled metal face in compression
- Failure of the fastenings in tension and shear

the test results shall be adjusted according to the following procedure /Eurocode 3 1996/:

$$R_{adj,i} = R_{obs,i}\left(\frac{f_y}{f_{y,obs}}\right)^{\alpha}\left(\frac{t}{t_{obs}}\right)^{\beta} \qquad (5.27)$$

where
$R_{obs,i}$ = the result of test number i
$R_{adj,i}$ = test result of the test i modified to correspond to the design values of metal thickness and yield stress
f_y = design yield stress
$f_{y,obs}$ = the yield stress measured in the test specimen
t = design metal thickness
t_{obs} = the metal thickness measured in the test specimen
α = 0 if $f_{y,obs} \leq f_y$
α = 1 if $f_{y,obs} > f_y$

except that, for the compression failure mode of a profiled face:

$$\alpha = 0.5 \quad \text{if} \quad f_{y,obs} > f_y \quad \text{and} \quad \frac{b}{t} > 1.27\sqrt{\frac{E_F}{f_y}}$$

In general:

$$\beta = 1.0$$

except that, for the compression failure mode of a profiled face:

$$\beta = 1.0 \quad \text{if} \quad t_{obs} \leq t$$

$$\beta = 1.0 \quad \text{if} \quad t_{obs} > t \quad \text{and} \quad \frac{b}{t} \leq 1.27\sqrt{\frac{E_F}{f_y}}$$

$$\beta = 2.0 \quad \text{if} \quad t_{obs} > t \quad \text{and} \quad \frac{b}{t} > 1.27\sqrt{\frac{E_F}{f_y}}$$

where $\frac{b}{t}$ = width to thickness ratio of the dominant part of the profiled face

The values $R_{adj,i}$ shall be used to represent the individual test results in the evaluation of characteristic strengths and resistances.

5.3 Test procedures for quality control

5.3.1 General

Every production facility shall have an adequate quality control procedure which shall include regular testing of the running production in accordance with the following paragraphs. The aim of the quality control procedures is to check:

- whether the properties of an individual production batch are such that the panels produced will satisfy all the functional expectations made in the design and are adequate for the batch to be acceptable

- whether, during the longer production period, the mean value and the standard deviation of the relevant properties are in accordance with the assumptions made in the design.

Regular quality assurance testing should commence at the stage of prototype manufacture and should continue throughout the duration of manufacture.

Quality assurance tests should be carried out using established procedures. These procedures should be fully documented when the quality assurance scheme is established. Only the details particular to any batch of specimens needs to be recorded at the time of the tests.

Quality assurance tests provide a clear documentation of the quality and consistency of production. They should be regularly used to update the statistical data base of the production process in accordance with section 5.3.5. Quality assurance test results should be kept available for at least five years.

Note: The following may serve as a check list for the information to be recorded:

- Date and time of manufacture
- Type of product
- Material specification
- Significant results with respect to the quality assurance scheme (e.g. strength, modulus etc.)

5.3.2 Number of quality assurance tests

A typical minimum quality assurance procedure for a continuous production line for the manufacture of panels containing polyurethane foamed-insitu shall include testing according to the following table:

Type of test	Number of specimens	Frequency
Core material		
Density	3	1 per shift
Cross-panel tensile strength and modulus	3	1 per shift
Compressive strength and modulus	3	1 per shift
Shear strength and modulus	3	1 per week
Dimensional stability under temperature attack	1	1 per week
Face material		All deliveries unless attested by material certificates
Tensile strength		
Thickness		
Complete panel		
Strength and stiffness	1	1 per week
Dimensional control	1	1 per shift
Ageing/durability	5	1 per year

A typical minimum quality assurance procedure for panels formed from slabstock shall include testing according, to the following table:

Type of test	Number of specimens	Frequency
Discrete core material		
Density (unless certificated)	3	1 per shift
Cross-panel tensile strength and modulus	3	1 per shift
Compressive strength and modulus	3	1 per shift
Shear strength and modulus	3	1 per week
Dimensional stability under temperature attack	1	1 per week
Face material		All deliveries unless attested by material certificates
Tensile strength		
Thickness		
Adhesive		
Shear strength	3	1 per week
Wedge test	3	1 per week
Complete panel		
Strength and stiffness of panel	1*	1 per week
Dimensional control	1	1 per shift
Ageing/durability	5	1 per year

* Wrinkling and shear strength shall be tested alternately

Notes: 1. For panels produced by horizontal batch moulding or other processes whereby the properties may not be consistent within the area of the panel, a greater number of tests will be necessary. These tables may therefore be regarded as a minimum procedure for a "structural sandwich panel".

2. When, after some months of continuous production, an established data base has been produced, and consistency of properties established, the above frequencies may be reduced.

3. Although no frequency is specified, the control of the thickness of slabstock and lamellas and the positioning of the joints between individual slabs is of fundamental importance and should be continuous.

Typical allowable differences in cutting thickness between adjacent lamellas or slabstock are as follows:

for fabrication with stiff platens 0.5 mm

4. When several different panel types are produced or when different deliveries of slabstock are used during a single shift, the manufacturer shall use his own judgement regarding the number of tests to be carried out.

5. If the producers of pre-formed core materials provide the panel producer with certificated values of the relevant core material properties, determined in accordance with the above table, the panel producer may substantially reduce the quality requirements for those properties so certificated. He shall, however, still carry out spot checks to confirm that there is no deterioration of these properties as a result of handling or storing.

5.3.3 Quality control of prefabricated slabs of core material

Material for the cores of sandwich panels shall only be obtained from suppliers of standardised material who operate an adequate quality control procedure and are prepared to guarantee the consistency and properties of their products. The following properties shall be guaranteed by the supplier. Nevertheless, it is recommended that they should be subject to random checks in accordance with this chapter.

- tolerances (particularly consistency of thickness)
- density
- tensile strength and modulus
- compression strength and modulus
- shear strength and modulus
- thermal conductivity
- reaction to fire

Notes: 1. The manufacturer should be able to provide statistical data regarding the variability of the properties of the core material.

2. In order to control the bonding process, particular attention should be given to the following:

- preparation of surfaces which are to be bonded to other materials (e.g. cutting marks, contamination by dust or other materials etc)
- temperature and humidity
- quantity and uniformity of application of foam or adhesive
- curing pressure and duration

3. The bond between the face and the core can effectively be checked by carrying out peel tests.

5.3.4 Dimensional control

All panel dimensions that are of significance for a faultless installation process, for effective tightness of joints and for satisfactory performance in service should be regularly checked during production. The main dimensions to be controlled are:

- thickness
- length
- width
- flatness of surfaces
- profile geometry (including depth of light profiling)
- edge details

The dimensions to be controlled and the applicable tolerances may be dependent on the nature of the panel and its intended usage. Minimum tolerances, together with measuring procedures, for wall and roof panels are given in the European Standard under preparation by CEN TC128/SC11.

More stringent tolerances may be agreed between the manufacturer and user as appropriate for a particular application. In particular, tolerances on length may be advantageously positive or negative depending on the usage.

5.3.5 Evaluation of quality assurance tests

In general, a production batch may be accepted with respect to a particular property x if:

$$\overline{x} - k_\sigma \sigma_x \geq \overline{x}_p \qquad (5.28)$$

where $\overline{x}_p$ = the 5 % fractile value of the total population according to section 5.1.2 (i.e. the characteristic value)
$\overline{x}$ = the average of the values for the batch
σ_x = the standard deviation for the batch

The values of the fractile factor k_σ are given in section 5.1.2.

Notes: 1. Where only a single test is carried out, evaluation is only possible over a longer period.

2. It is permissible for individual test results to fall below the characteristic value $\overline{x}_p$ provided that the above equation is satisfied for all test results in the longer term.

5.3.6 General control of production

The production process, and therefore the quality of the final product, may be influenced by the conditions existing during production. It is therefore essential that the quality control procedures described previously are accompanied by equally stringent checking of the relevant items in the production process. This checking should include the following list:

- Ambient temperature and humidity
- Temperature of the raw materials
- Temperature of the moulds
- Amount of adhesives
- Temperature of complete panel during and after production
- Adherence to specifications for foaming and glueing
- Control of delivery of core material
- Time and pressure during curing of panels.

6. DESIGN OF PANELS BY TESTING

6.1 General

6.1.1 Scope

This chapter describes the principles whereby sandwich panels may be designed on the basis of a comprehensive test programme as an alternative to design on the basis of calculations.

For design on the basis of calculation, as described in previous chapters, the procedure requires the determination of stresses and deflections for all of the relevant load cases. It is then checked that the most unfavourable combination of factored loads does not result in stresses or deflections which exceed the design values in either the serviceability or ultimate limit states.

This approach is also possible for design by testing. The test program yields values for deflections, failure loads and, in certain cases, reaction forces. A statistical analysis of the test results allows these to be reduced to characteristic values. In order to be able to combine load cases it is necessary to use test arrangements or evaluation methods, which are suitable to take in account the action effects due to the temperature changes and the shear creep of the core.

6.1.2 Selection of test specimens

A series of test results may be used to derive design data for a given panel or range of panels only if the thickness and material properties of the facings and the core material of all panels included in the series are within the following limits compared to the nominal values:

thickness of core	between +2% and -4%
density of core	± 10%
yield stress of face material	the correct grade shall be used and shall have a yield stress within -5 and +25% of the nominal value
thickness of face material	between ±10%

6.1.3 Treatment of test results

Before commencing the statistical analysis, individual test results shall be adjusted to correspond to the nominal values of the geometry and material properties of the specific sandwich panel product under consideration. The adjustment procedure described in section 5.2.20 shall be used.

6.2 Limit states

6.2.1 Ultimate limit state

In any test, the ultimate limit state load for the tested panel is defined as the maximum load sustained by the panel during the test. The failure mode corresponding to this limit state may be in the faces, in the core or in the connections as described in section 2.3.1.

For design at the ultimate limit state, the forces and bending moments corresponding to failure must first be subject to statistical analysis to give characteristic values and must then be divided by the appropriate material factor given in section 2.5.3 to give design values.

6.2.2 Serviceability limit state

In any test, the serviceability limit state may correspond to any one of the conditions described in section 2.3.2.

For design at the serviceability limit state, the actions giving rise to the limiting condition must first be subject to statistical analysis to give characteristic values and must then be divided by the appropriate material factor given in section 2.5.3. to given design values.

Comment: Material factors for the serviceability limit state verification are generally 1.0 (See 2.5.3).

6.3 Test procedures

All testing shall be carried out using the procedures described in chapter 5.

The following tests are particularly applicable to design by testing:

- Test to determine the ultimate failure load and stiffness of a simply supported panel (section 5.2.7)
- Test on two-span panels (section 5.2.9). To combine the action effects due to the temperature, a distance piece over the middle support should be used (see Appendix D)
- Thermal load test (section 5.2.12), if necessary
- Test to determine the creep deflection (section 5.2.5)
- Test for walkability (section 5.2.10)
- Tests to determine the degradation factor (sections 5.1.4 and 5.2.2)

These primary tests may be supplemented by

- Tests to determine the material properties of the core of all tested panels:
  Density
  Tensile strength or bond (section 5.2.2)

Compression strength (section 5.2.3)
Shear strength (section 5.2.4)

- Tensile tests on the face material of all tested panels (section 5.2.1)
- Tests for fasteners (section 5.2.18).

It is implicit that the primary tests in any programme of design by testing should be carried out on complete panels or assemblies of complete panels which reproduce as nearly as possible the loading and support conditions likely to be realised in practice. Where various supporting arrangements may be used, that which gives the least restraint to the panel should be used for the test programme.

6.4 Number of tests

6.4.1 Number of short-term tests

The total number of tests shall be sufficient to obtain a valid statistical correlation.

In general, the test series shall embrace the extreme values of all of the variable parameters and it is permissible to interpolate between test results. However, extrapolation should only carried out with caution and over a limited range of values.

The test series shall include, where relevant to the practical conditions of use, for each of the following conditions:

- statical system (one or two spans)
- direction of load (downward or uplift)
- thickness and metal quality of face material
- thickness and material properties of core
- width of supporting member
- type and frequency of fastenings
- ageing

It is generally sufficient to test each combination of the above conditions at three different spans:

- the longest span likely to be used in practice
- the shortest span likely to be used in practice
- an intermediate span

When a panel is manufactured in more than two core thicknesses, the tests may be carried out for only three thickness, namely the thinnest, the thickest and one of the intermediate thicknesses with a maximum difference of 30 mm between two tested thicknesses.

6.4.2 Number of creep tests

It is normally sufficient to carry out a single creep test in order to define the creep behaviour for a given core material. If the design is to embrace a series of panels with different core thicknesses, the test should be carried out on the panel with the thickest core. The span is not critical and should be within the normal range of use.

6.5 Evaluation of test results

6.5.1 General conditions

The evaluation of the test result shall be carried out in terms of the nominal values of the thickness and material properties of the facings and core materials.

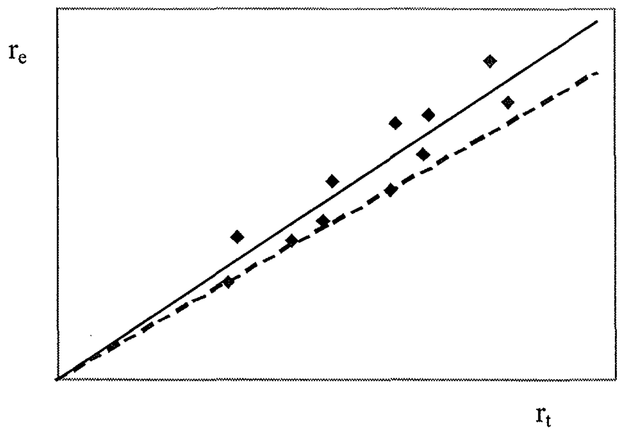
Before embarking on any evaluation of test results, it is necessary to have a suitable analytical model which defines a relationship between the relevant test result and all of the relevant parameters in the test series. Thus, if different core or face thicknesses, as well as spans, are included in the tests, the mathematical model must include terms to cater for these parameters.

In general, designers may choose either a model based on the appropriate equations of structural mechanics or an entirely empirical approach. However, all methods are required to show the same level of safety.

Comment:	It is implicit in this clause that the more accurately the model reflects the real behaviour, the more favourable will be the characteristic values resulting from the evaluation.

The characteristic values shall be obtained from the test results by carrying out a statistical analysis of the differences between the test results and the values given by the mathematical model. A suitable method is described in Appendix Z to Eurocode 3.

Comment:	The method described in Appendix Z is rather complicated and may be simplified for this application provided that the implied level of safety is not reduced. A possible simplification is as follows: A comparison between the test results r_e and the corresponding theoretical values given by the mathematical model r_t can be plotted on a diagram.



The mean value of the test results is a straight line passing through the origin. After adaptation of the model, this should be approximately the line $r_e = r_t$

The mathematical model is then treated as the mean of the test results. In order to obtain a unified population of values, each test result r_e is divided by the corresponding r_t. The standard deviation, s, of this normalised population r_n can then be found using

$$s = \sqrt{\frac{\sum (r_n - r_{mn})^2}{n-1}}$$

where n is the number of test results and $r_{mn} = \frac{\sum r_n}{n}$

The characteristic value obtained from the test results is then given by

$$r_k = r_t (1 - k_\sigma s)$$

for any panel within the range covered by the tests where k_σ is given in section 5.1.2.

Characteristic values r_k are shown by the broken line in the above figure.

6.5.2 Ultimate limit state

For the ultimate limit state verification, the design load is determined using the combination rules given in section 2.4.1. The design load shall be less than the design value of the corresponding resistance of the panel. Thus

$$\gamma_G G_k + \gamma_{Q1} Q_{k1} + \sum_{i>1} \gamma_{Qi} \psi_{0i} Q_{ki} \le R_d \qquad (6.1)$$

where

G_k = characteristic value of the permanent load, e.g. self-weight

Q_{k1} = characteristic value of the dominant live load

Q_{ki} = characteristic values of the non-dominant live loads (i > 1)

$R_d = \frac{R_k}{\gamma_M}$ design value of the resistance load of the panel

The load Q in the expression (6.1) denotes wind and snow loads and the load caused by the temperature difference ΔT between the faces of the panel.

If the resistance of the sandwich panel is determined by tests in which the effects of the uniformly distributed load q and the temperature difference ΔT between the faces of the panel are combined directly, the expression (6.1) can be written as (see Appendix D2)

$$\gamma_G G_k + \gamma_{Q1} Q_{k1} + \sum_{i>1} \gamma_{Qi} \psi_{0i} Q_{ki} \leq q_{rd} = \frac{q_{rk}}{\gamma_M} \tag{6.2}$$

In the expression (6.2) Q denotes the wind and snow loads, only, because the effect of the temperature difference ΔT is included in the experimental resistance value $q_{rk} = q_{rk}(q,\Delta T)$.

Comments:
The resistance load q_{rd} is a function of temperature and creep time (See Appendix D2). Therefore, the resistance load q_{rd} used in the ultimate limit state verification has a different value in different combinations and it has to correspond the combination of the loads on the left side of the design equation (6.2).

For the ultimate limit state verification, the design expression (6.2) can typically be written for roof panels as

$1.35 \cdot G + 1.50 \cdot Q_{snow} + 1.50 \cdot 0.6 \cdot Q_{Wind} \leq q_{rd}$ (Winter, T_1 = 0°C, short-term)

$1.35 \cdot G + 1.50 \cdot Q_{snow} + 1.50 \cdot 0.6 \cdot Q_{Wind} \leq q_{rd}$ (Winter, T_1 = 0°C, long-term)

$1.0 \cdot G + 1.50 \cdot 0.6 \cdot Q_{Wind} \leq q_{rd}$ (uplift; Summer temperature T_1, temperature dominant)

$1.0 \cdot G + 1.50 \cdot Q_{Wind} \leq q_{rd}$ (uplift; Summer temperature T_1 , wind dominant)

and for wall panels

$1.50 \cdot Q_{Wind} \leq q_{rd}$ (wind dominant)

$1.50 \cdot 0.6 \cdot Q_{Wind} \leq q_{rd}$ (temperature dominant)

6.5.3 Serviceability limit state

6.5.3.1 Limit state of deflection

For the serviceability limit state verification, the design deflection shall be determined using the combination rules given in section 2.4.2 The value of the design deflection shall be less than the allowable deflection. Thus

$$\sum_{j\geq 1} w_{Gj} + \psi_{11} w_{Q1} + \sum_{i>1} \psi_{0i} \psi_{1i} w_{Qi} \leq w_{allowable} \qquad (6.3)$$

where

w_G = short-term deflection due to dead load

w_{Q1} = short-term deflection due to dominant live load

w_{Qi} = short-term deflection due to non-dominant live load ($i > 1$)

$w_{allowable}$ = allowable deflection, e.g., L/200 or L/100 (See section 2.3.2)

<u>Comment:</u> Examples of the typical combinations for deflections are given in section 2.5.2.

6.5.3.2 Limit state of resistance

The verification of the resistance at the serviceability limit state is normally necessary to multi-span panels because, for single span panels, the ultimate limit state verification governs the design with respect to resistance.

$$\sum_{j\geq 1} G_{kj} + Q_{k1} + \sum_{i>1} \psi_{0i} Q_{ki} \leq R_d \qquad (6.4)$$

The important failure mode for this verification is the failure causing permanent deformations at internal supports (e.g. buckling or yielding of the face or crushing of the core). Test arrangement to combine the effects of the uniformly distributed load q and the temperature difference ΔT on the resistance load value q_{rd} are given in Appendix D1.

If the resistance of the sandwich panel is determined by tests in which the effects of the uniformly distributed load q and the temperature difference ΔT between the faces of the panel are combined directly, the expression (6.4) can be written as (see Appendix D1)

$$\sum_{j\geq 1} G_{kj} + Q_{k1} + \sum_{i>1} \psi_{0i} Q_{ki} \leq q_{rd} = \frac{q_{rk}}{\gamma_M} \qquad (6.5)$$

In the expression (6.5) G_k denotes the characteristic value of the self-weight and other permanent loads and Q_k the wind and snow loads, only, because the effect of the temperature difference ΔT is included in the experimental resistance value $q_{rk} = q_{rk}(q,\Delta T)$.

Comment:

The resistance load q_{rd} is a function of temperature (thickness of the distance piece, See Appendix D1). Therefore, the resistance load value used in the serviceability limit state verification, has to correspond the combination of the loads on the left side of the design equation (6.5).

The verification of the resistance at the serviceability limit state includes the checking **wrinkling of the compressed face**, the **shear failure of a core** or of a **profiled face** and the **crushing failure of a core** or **profiled face** on an intermediate support or in contact to a line load.

If the effect of the temperature difference ΔT is included in the experimental resistance value q_{rd}, the design equation can in typical cases be expressed for roof panels as

$$G + Q_{Snow} + 0.6 \cdot Q_{Wind} \leq q_{rd} \quad \text{(Winter, } T_1 = 0°\text{C, short-term)}$$

$$G + Q_{Snow} + 0.6 \cdot Q_{Wind} \leq q_{rd} \quad \text{(Winter, } T_1 = 0°\text{C, long-term)}$$

$$G + 0.6 \cdot Q_{Wind} \leq q_{rd} \quad \text{(Summer temperature } T_1 \text{, temperature dominant)}$$

$$G + 1.0 \cdot Q_{Wind} \leq q_{rd} \quad \text{(Summer temperature } T_1 \text{, wind dominant)}$$

and for a wall panel as

$$Q_{Wind} \leq q_{rd} \quad \text{(wind dominant)}$$

$$0.6 \cdot Q_{Wind} \leq q_{rd} \quad \text{(temperature dominant)}$$

7. FASTENINGS

7.1 Basic principles

7.1.1 Loads on fastenings

Fastenings at supports are loaded by tensile forces caused by wind uplift loads and temperature differences between the faces of the panel. Fastenings may be loaded also by shear forces caused by the self-weight and weight of additional building components on a wall and roof, by the temperature expansion of the faces and, additionally, by possible diaphragm action (see Chapter 2).

Whereas the sandwich panel itself has to be designed against static loads, the fastenings may be more sensitive to repeated loading than the panels themselves and repeated loading should usually be considered in the design.

Comment:	The following loads may arise in the fastenings connecting a sandwich panel to to supporting substructure: All are likely to be repeated: Tensile forces - static or repeated Shear forces - these occur mainly in the inner face Imposed deformations - imposed differential movements of the faces causing bending of the fasteners and/or yield/folding of the outer face and/or deformations or yield in the supports.

7.1.2 Types of fastenings to supports

The following principles of fastening may be distinguished and are embraced by these Recommendations:

(a) Fastening within the width of the panel
1. Fastening through the overall depth of the panel
2. Fastening through the substructure into an insert

(b) Fastening at the sidelap
1. Fastening through the overall depth of the sidelap with attachment of the outer face
2. Fastening of the inner face only
3. Fastening of some part of the panel without attachment of the outer face

(c) Proprietary designs not covered by (a) or (b)

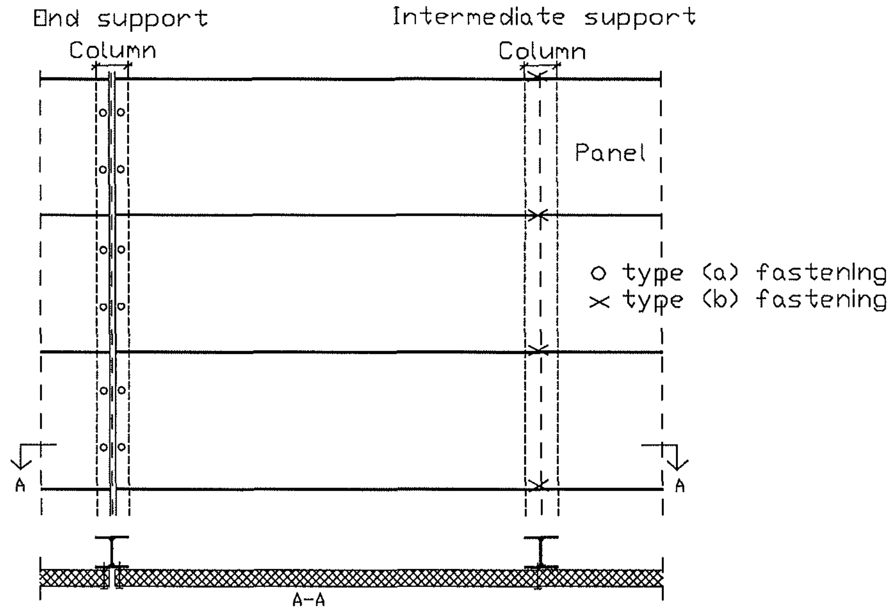


Fig. 7.1 Types of fastenings to supports.

End support = support of the end of a panel
Intermediate support = support in between the length of a panel

Comments:

1. When fastening is through one skin only, local strengthening of the panel may be necessary in order to avoid crushing, delamination or peeling of the fixed face at the point of fixation.

2. Both types of fastenings are common at end supports as well as at intermediate supports.

3. The characteristic strength of type (b) fastenings at end supports is usually not so high as at intermediate supports.

4. The outer face shall preferably be fixed to the framework.

7.1.3 Types of fasteners

In sandwich panel construction, the following fastener types are commonly used for connecting sandwich panels to light-gauge metal supporting structures:

a), c) self-tapping screws

b), d) self-drilling screws

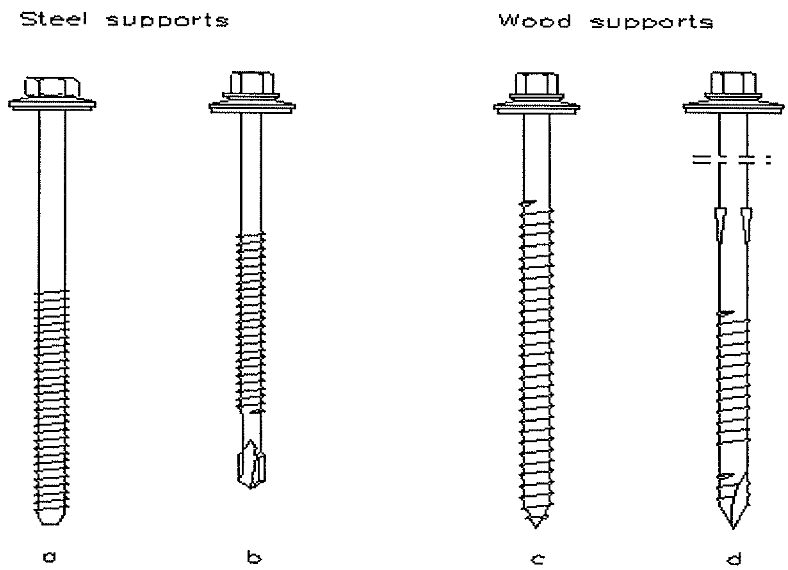


Fig. 7.2 Fasteners for common applications in light-gauge metal constructions.

In addition to these fasteners, nowadays, special fasteners for sandwich panels which have a face-supporting thread beneath the head of the screw are often used. The types of screw are the same as above:

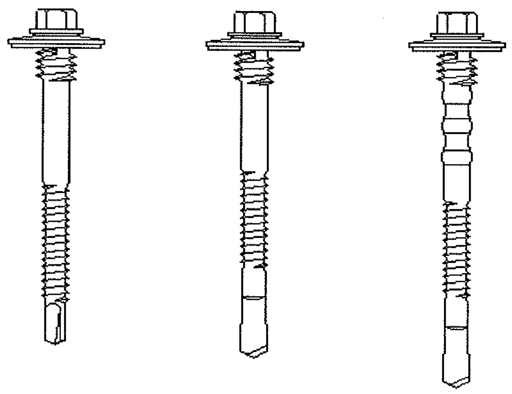


Fig. 7.3 Special fasteners for fastening sandwich panels.

For connecting sandwich panels, as well as conventional sheeting, to concrete supports, a special nail has been designed, which already has been approved in some European countries. This nail is hammered through the panel into a predrilled hole in the concrete.

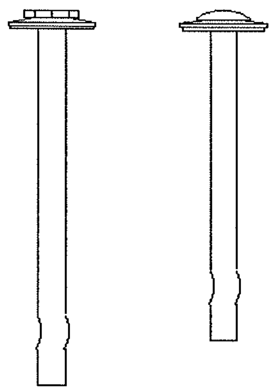


Fig. 7.4 Special fasteners for fastening to concrete frames.

7.2 Failure modes of fastenings

7.2.1 Failure modes of fastenings loaded in shear

In sandwich construction, the same failure modes can arise as for fastenings in sheeting, as described in the European Recommendations for the design and testing of connections in steel sheeting and sections; Ref. /ECCS 1983a, Section B1.1.1/:

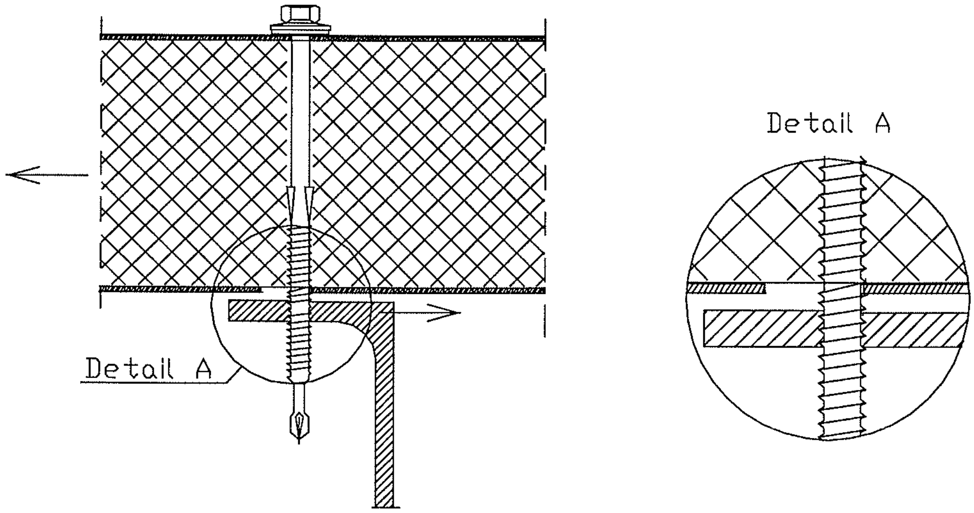


Fig. 7.5 Yield of inner panel sheet only.

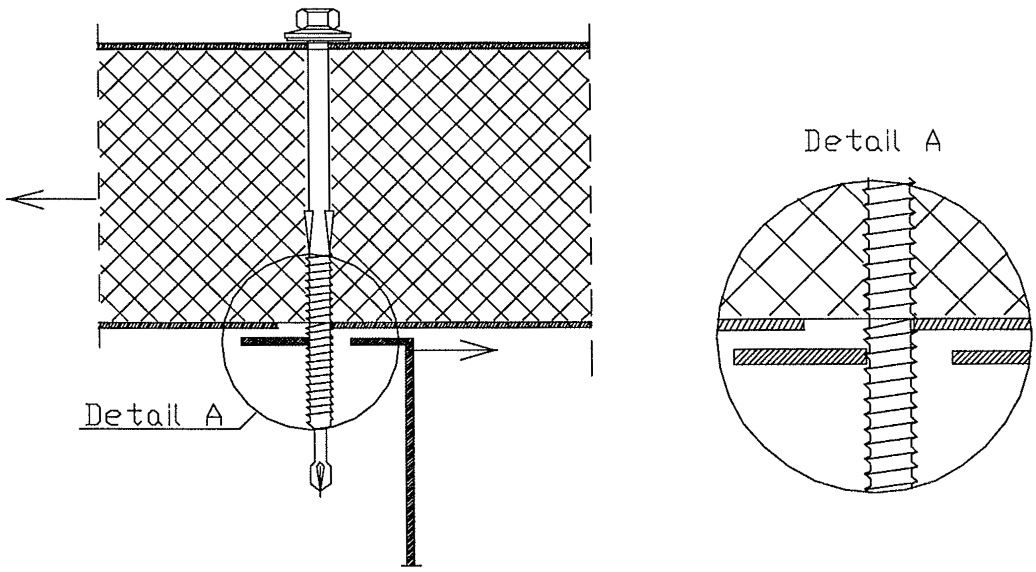


Fig. 7.6 Yield of inner panel sheet and/or supporting structure.

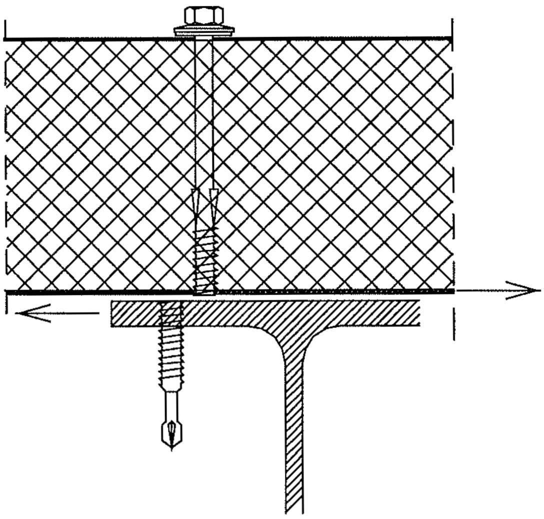


Fig. 7.7 Shear of the fastener.

<u>Comment:</u>	This mode may occur when the sheets are thick in comparison to the fastener diameter, or when an unsuitable fastener is used.

In addition, the following failure mode may also occur in fastenings of sandwich panels:

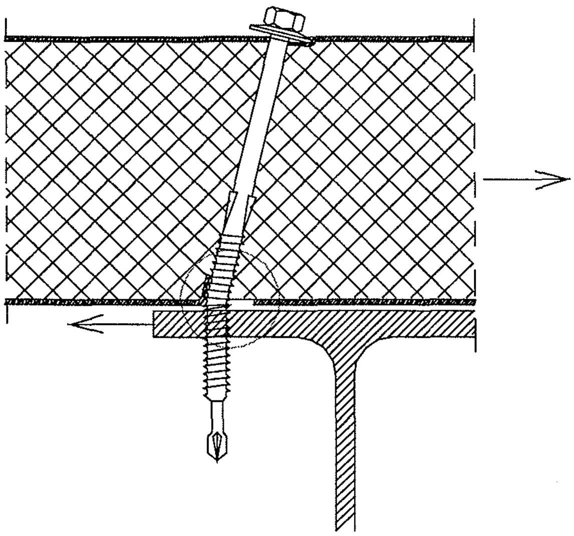


Fig. 7.8 Tilting of the fastener with folding of the inner face of the panel.

Comment:	The failure mode involving tilting of the fastener together with folding or tearing of the inner face of the panel usually arises when the fasteners are relatively flexible. The inner face will carry most of the shear load and can be assumed to carry all of the shear load for the purposes of design.

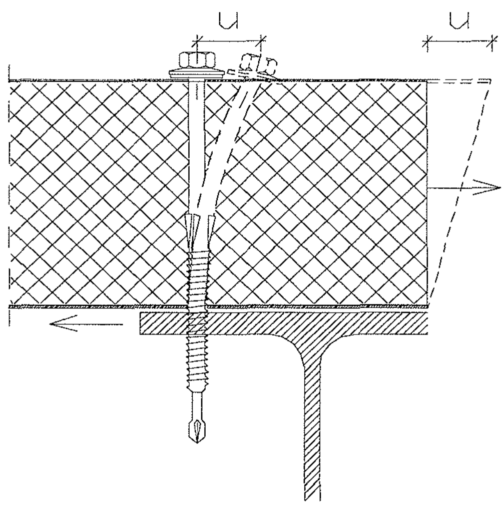


Fig. 7.9 Bending of fastener due to imposed deformation, u.

Comment:	A temperature difference between the faces of the panels causes a relative displacement u which, in turn can cause yield and/or folding of the outer face of the panel and bending of the fastener. In thin supports (e.g. cold-formed members) deformations in the supports may also be caused. At a certain value of the displacement 'u', the fastener may break.

7.2.2 Failure modes of fastenings loaded in tension

Similar failure modes can arise as for fastenings in sheeting as described in Ref. /ECCS 1983a, Section B1.1.2, Figures 5-11/.

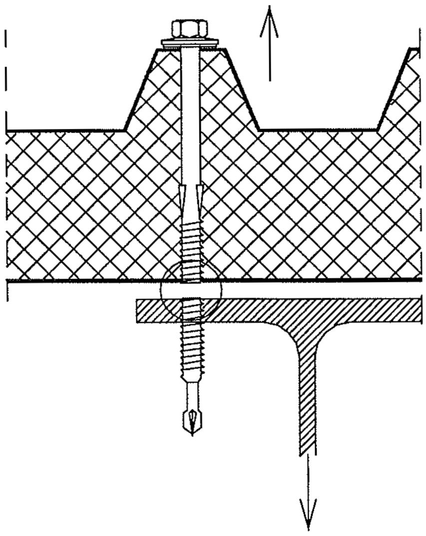


Fig. 7.10 Tension failure of the fasteners.

Comment:	This mode may occur when the strength of panel is high in comparison with the fastener or when an unsuitable fastener is used.

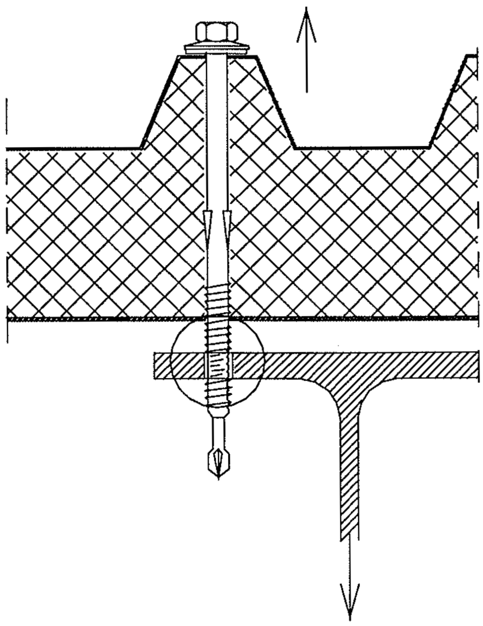


Fig. 7.11 Pull-out of the fastener by disturbing the thread in the substructure.

Comment:	This mode may occur when the support member is thin, or when there is insufficient anchorage of the fastener.

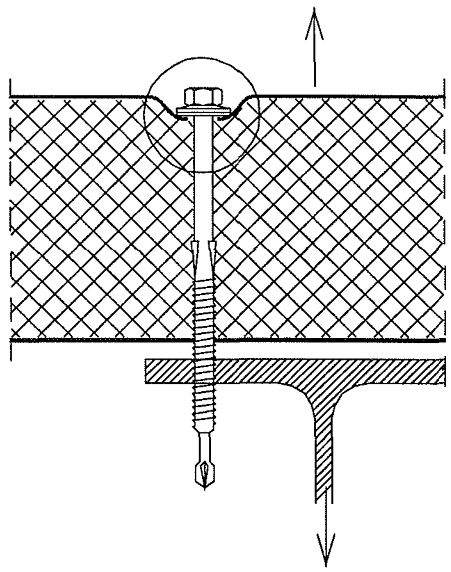


Fig. 7.12 Pull-over of the outer face of the panel.

<u>Comments:</u>	1. In sandwich panels, the failure mode of "pull-over" is influenced by the stiffness of the core material, as shown above. 2. Pull-over of the outer face of the panel may cause loss of weathertightness.

In addition, the following can also occur in sandwich panels:

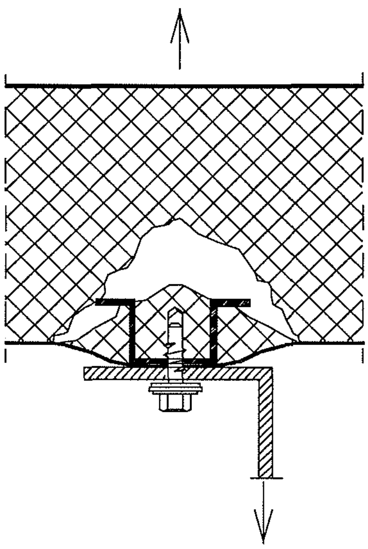


Fig. 7.13 Delamination of the inner face (fastening of some part of the panel on inserts without attachment to the outer face).

<u>Comment:</u>	Delamination of the inner face may occur when the pull-out strength of the insert is higher than the tension strength of the affected core area. This detail is not generally recommended although it is sometimes used when hygeinic requirements preclude the use of fasteners which are visible on the non-fastened face.

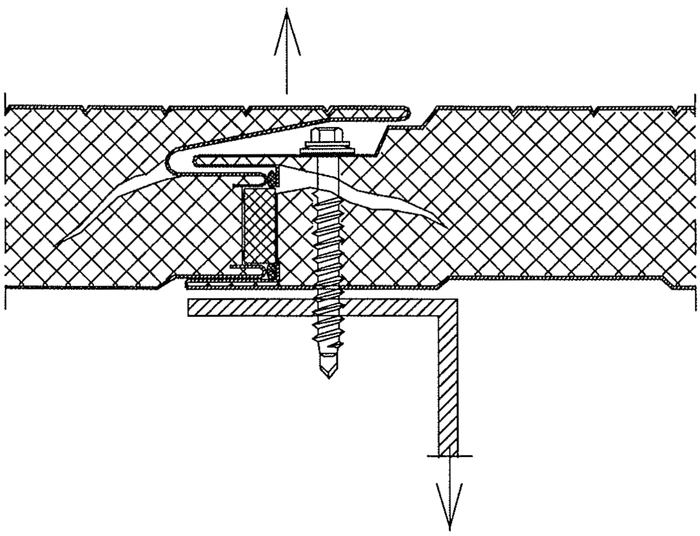
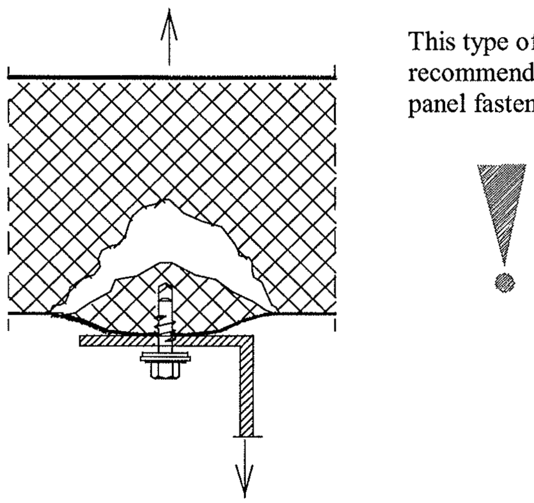


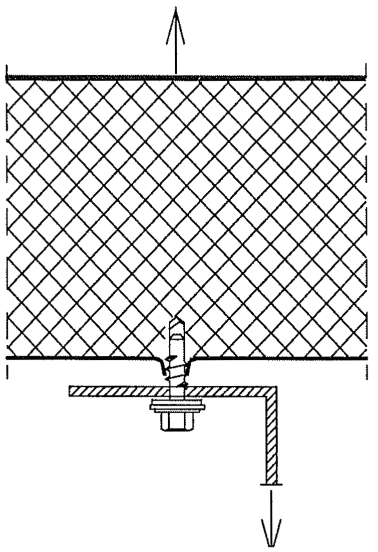
Fig. 7.14 Failure of the core.

Comment:	Failure of the core may occur in side-lap fastenings when only part of the panel without the outer skin is attached.



This type of fastening is not recommended as a sandwich panel fastening system

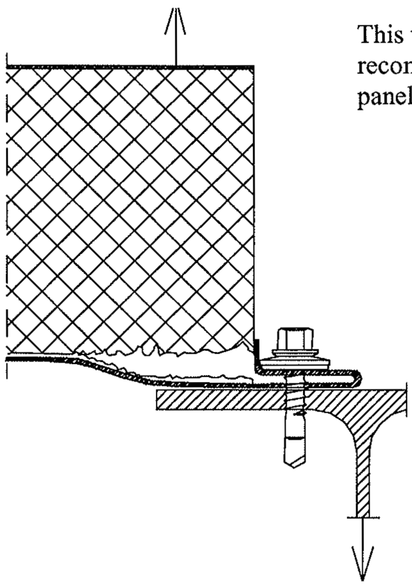
Fig. 7.15 Delamination of the inner face.



This type of fastening is not recommended as a sandwich panel fastening system

Fig. 7.16 Pull-out of the inner face.

<u>Comment:</u> Attention should be paid that the fastening systems in the figures 7.15 and 7.16 above have very low characteristic strength. In some European countries they are not recommended.



This type of fastening is not recommended as a sandwich panel fastening system

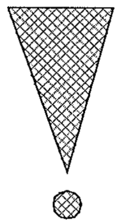


Fig. 7.17 Peeling of the inner face.

<u>Comments:</u>

1. Delamination of the inner face or pull-out are the likely modes of failure when only the inner skin is fastened and there is no local strengthening of the panel (see also note on "peeling" below)

2. Peeling of the inner face is the likely mode of failure when only the inner skin is fastened at the sidelap and there is no local strengthening of the panel. Except in special cases, the design of fastenings which fail by delamination or peeling is <u>not</u> recommended.

7.3 Characteristic resistance of fastenings

7.3.1 Characteristic resistance of fastenings under static load

The characteristic resistance of a fastening shall be obtained by testing. At least five, and usually ten, tests should be carried out on a given arrangement and the results interpreted in accordance with clause 5.1.2. The strength of a fastening should be taken as the lower of:

the ultimate load

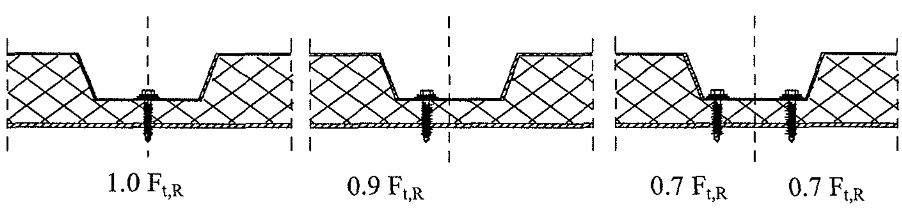
the load achieved with a deformation of 3 mm at the inner face in case of shear.

7.3.1.1 Characteristic resistance of fastenings loaded in tension

The characteristic resistance $F_{t,R}$ under both static and repeated tensile load shall be determined by testing in accordance with the procedures described in section 5.2.18.1. Depending on the particular fastening arrangement, the tests may be carried out using either a representative portion of the complete sandwich panel or material from the outside face only.

It is assumed that load is applied centrally and that the diameter of the head of the fastener or washer minus the diameter of the fastener is at least 7 mm and the washer has sufficient rigidity to prevent it from being appreciably deformed or pulled over the head of the fastener. When this is not the case and there are no specific data available from tests on sandwich panels, the following values (obtained from profiled sheeting) may conservatively be taken.

When the attachment is at a single quarter point, the design value of the resistance shall be 0.9 $F_{t,Rd}$ where $F_{t,Rd}$ is the design value of the characteristic resistance (after application of the material factor). When the attachment is at both quarter points and the distance apart of the screws is less than 70 mm, it shall be 0.7 $F_{t,Rd}$, otherwise $F_{t,Rd}$ may be used.



Note: For the diameter of pre-drilled holes for screws, the manufacturer's guidelines shall be observed.

If the individual fasteners are too close together (typically less than about 250 mm), the characteristic resistance may be reduced by interaction between them. In such cases, the characteristic resistance shall be determined by tests on the complete fastener group.

7.3.1.2 Characteristic resistance of fastenings loaded in shear

The characteristic resistance under both static and repeated load should normally be obtained by testing in accordance with the procedures described in section 5.2.18.2. Depending on the particular fastening arrangement, the tests may be carried out using either a representative portion of the complete sandwich panel or material from the inner face only.

7.3.2 Deformation requirements for fasteners under static load

The requirements given for sheeting in clauses B1.3 and B1.4 of Ref. /ECCS 1983a/ are valid. The requirements for shear must be related to the inner face.

7.3.3 Requirements for fastenings under repeated load

Under repeated tension load, the requirements given for sheeting in clause B2 of Ref. /ECCS 1983a/ are valid.

Under repeated shear load the requirements given for sheeting in clause B2 of Ref. /ECCS 1983a/ are valid.

7.3.3.1 Imposed deformations under repeated load

The fastenings should be capable of sustaining the repeated lateral deformations associated with movements of the structure under load (e.g. thermal bow, inclination of the panel at the end support). A suitable test for this requirement is described in section 5.2.18.4.

The imposed lateral deformation 'u' should be taken to be the difference between the displacement of the inner and outer faces at the point of attachment. In calculating this displacement, the most unfavourable combination of loads at the ultimate limit state should be considered (see section 2.4.1).

Comments: 1. Due primarily to thermal expansion and contraction of the outer face, the heads of fasteners which pass through the full depth of the panel may be subjected to repeated cycles of imposed deformation. The test requirements should therefore be based on the expected cycles of temperature variation during the life of the panel.

2. A method of calculating 'u' is given in Appendix E. In most cases, it is sufficient to take account only of temperature differences when calculating 'u'.

7.3.3.2 Characteristic strength with regard to peeling and delamination

The characteristic strength with regard to peeling and delamination shall be based on tests which take into account the effect of repeated loading.

This test should follow the loading spectrum given in the appropriate National Standard. In the absence of such a spectrum, satisfactory performance at the serviceability limit state may be demonstrated by a preliminary test in which the fastening is subject to 100 cycles of load up to the serviceability load. This preliminary test is satisfactory if the residual deformation is not greater than 10% of the maximum deformation achieved during the test.

Comments: 1. Investigation of these modes of failure will usually require testing of a complete panel assembly which should reflect the real situation as closely as possible.

2. Peeling and delamination should normally be regarded as serviceability limit states.

7.4 Design strength of fastenings

The design strength of a fastening should be obtained by dividing the characteristic strength by the material factor given in section 2.5.3.

7.5 Forces and deformations in fastenings

Primary forces in fastening are those caused directly by the actions (load or temperature). These must always be calculated and compared with the design strength of the fastening.

Secondary forces are those caused indirectly by the actions (e.g. by rotation of the point of attachment). They need only be calculated when the deformation requirements given in clause 7.3.2 are not satisfied. The combined effects of primary and secondary forces must then be considered.

7.5.1 Shear forces in fastenings

It may be assumed that shear forces are only present in the inner face of the panel. Primary shear forces may be caused, for example, by:

- dead load (eg of facades)
    - temperature changes in the face material

- secondary shear forces may be caused, for example, by:
    - rotation at the end of a panel fastened eccentrically with respect to its neutral axis
    - unintentional diaphragm action.

Comments: 1. Stressed skin (diaphragm) design in which sandwich panels carrying in-plane shear forces are used to replace wind bracing is beyond the scope of these Recommendations.

2. Shear forces in fastenings arising from variations in the temperature of the faces of the panel may be calculated taking into account slip in the fastenings, strain in the faces and deflection of the supporting framework. These shear forces may be neglected when there is sufficient deformation capacity.

3. Appendix 1.B.2 of Ref. /ECCS 1983a/ provides a calculation method for the shear forces caused by rotation and membrane action of the sheeting.

7.5.2 Tensile forces in fastenings

Primary tensile forces will generally be caused either by uplift loads or by temperature differences between the faces.

Secondary tensile forces may be caused by prying action under either upward or downward loads. A method for determining prying forces is given in Appendix I.IA of Ref. /ECCS 1983a/.

7.6 Additional considerations with regard to fastening systems

- It should be verified that the performance of the fastening system will not be adversely affected by creep.
- It should be verified that the performance of the fastening system is not impaired by a rise in temperature of the core material.
- When designing the fastening system, the possible deleterious effects of corrosion should be considered. (See ECCS 1983a and ECCS 1983b).
- When designing the fastening system of panels for cold stores, it should be verified that the lateral deformations of the supporting substructures during the cool-down phase

cause additional forces in fastenings and panels and the additional forces in the fastenings and panels can be accommodated (see Fig 7.18).

Comments: 1. Creep occurs primarily in roofs. It can only occur in wall cladding if there is permanent externally applied load.

2. In general, the mechanical properties of core materials are adversely affected by a rise in temperature and this may influence the performance of fastenings. Where relevant, this may require testing at elevated temperature.

3. To avoid additional forces in cold store panels due to lateral deformations of the supporting substructures special designed clamps are recommended (see ref. Lightweight sandwich construction 2000).

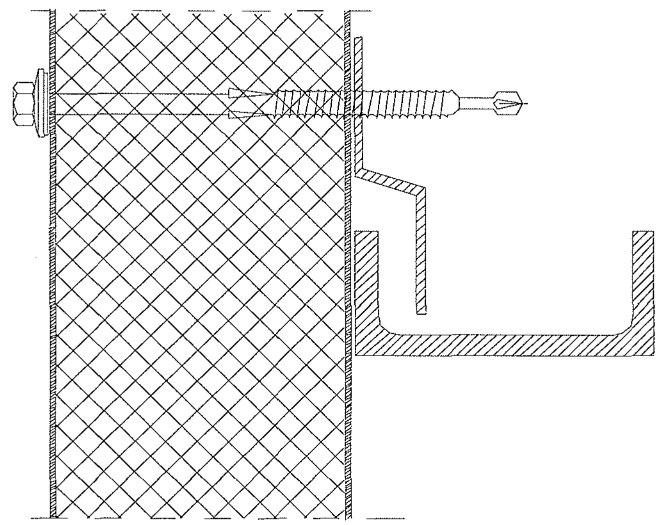


Fig. 7.18 Example of special clamp in cold stores.

8. REFERENCES

8.1 Standards, guidelines, recommendations

CIB W56 / ECCS 1993, Preliminary European Recommendations for Sandwich Panels with Additional Recommendations for Panels with Mineral Wool Core Material, Part I: Design. CIB Report, Publication 148, 1993, Reprint 1995.

ECCS 1991, Preliminary European Recommendations for Sandwich Panels, Part I, Design. ECCS Publication N 66, 1991.

ECCS 1983a, European Recommendations for the design and testing of connections in steel sheeting and sections. ECCS Committee TC7, ECCS Publication No. 21, 1983.

ECCS 1983b, European Recommendations for Steel Construction: Mechanical fasteners for use in steel sheeting and sections. ECCS Committee TC7, ECCS Publication No. 42, 1983.

Eurocode 1 1994. Basis of Design and Actions on Structures, Part 1 Basis of Design, ENV 1991-1, 1994.

Eurocode 3 1996. Design of Steel Structures, Part 1.3 Cold formed thin gauge members and sheeting. ENV 1993-1-3, 1996.

Institut für Bautechnik 1996. Design Loads and statical analysis for sandwich panels - core of polyurethane rigid foam with metal faces. Version 3.93.

UEAtc 1996, Rapport technique UEAtc sur l'Agrément des ouvrages réalisés en panneaux sandwich avec mousse de polyuréthanne sans CFC. Juillet 1996.

ASTM D2244-93, Standard Test Method for Calculation of Color Differences from Instrumentally Measured Color Coordinates.

ASTM D3762-98, Standard Test Method for Adhesive-Bonded Surface Durability of Aluminium (Wedge Test).

ASTM E903-96, Standard Test Method for Solar Absorptance, Reflectance and Transmittance of Materials Using Integrating Spheres.

ECCA-T3 1985, Color difference.

EN 410 1998, Glass in building. Determination of luminous and solar characteristics.

EN 485-2 1995, Aluminium and aluminium alloys. Sheet, strip and plate. Part 2: Mechanical properties.

EN 10002-1 1990, Metallic materials. Tensile testing. Part I: Method of test (at ambient temperature).

EN 10088-2 1995, Stainless steels. Part 2: Technical delivery conditions for sheet/plate and strip for general purposes.

EN 10143 1993, Continuously hot-dip metal coated steel sheet and strip. Tolerances on dimensions and shape.

EN 10147 1995, Continuously hot-dip zinc coated structural steel strip and steel. Technical delivery conditions.

EN 10214 1995, Continuously hot dip zinc-aluminium (ZA) coated steel strip and sheet. Technical delivery conditions.

EN 10215 1995, Continuously hot dip aluminium-zinc (AZ) coated steel strip and sheet. Technical delivery conditions.

EN 12090 1997, Thermal insulating products for building applications. Determination of shear behaviour.

ISO 844 1984, Cellular plastics. Compression test for rigid materials.

ISO 12491 1997, Statistical methods for quality control of building materials and components.

ISO 7892 1988, Vertical elements - Impact resistance tests - Impact bodies and general test procedures.

8.2 Textbooks

Allen, H.G., Analysis and design of structural sandwich panels. Pergamon Press Ltd. UK, 1969.

Jungbluth U. & Berner K., Verbund und Sandwichtragwerke. Springer-Verlag, 1986. Pp. 336-559. (in German)

Lightweight sandwich construction. Ed. J.M. Davies, Blackwell Science Ltd., Oxford. 2000.

Stamm K. & Witte H., Sandwichkonstruktionen-Berechnung, Fertigung, Ausführung. Springer-Verlag, Wien and New York, 1974. (in German)

Zenkert, D., An Introduction to Sandwich Construction. EMAS Ltd. 1995

8.3 Papers and reports on design procedure and calculation models

Bark, H., Eberhardt, C. & Sheikh-Ali, G., Zur Bestimmung der Deckschichttemperatur von Sandwichelementen infolge klimatischer Einwirkungen. Bauingenieur 67 (1992) pp. 77-81. (in German)

Bayramoğlu, G., Büyüktaşkin, H.A. & Arda, T.S. 1997. Comparison between experimental and calculation results of structural sandwich panels. Proc. of EC'97 Conference held in Strassbourg, 22-23 May 1997. 7 p.

Berner, K., Davies, J.M., Helenius, A. & Heselius, L., The durability of structural sandwich elements. Materials and Structures, 1994, Vol. 27, pp. 33-39.

Berner, K., Research Report: Erarbeitung vollständiger Bemessungsgrundlagen im Rahmen bautechnischer Zulassungen für Sandwichbauteile, Teil 2: Berücksichtigung zusätzlicher Beanspruchungen bei der Bemessung von durchlaufenden Sandwichplatten im Zwischenstützenbereich, Fachhochschule Rheinland-Pfalz. (in German). 1995

Berner, K., Praxisgerechte Nachweise zur Trag- und Gebrauchsfähigkeit von Sandwichelementen. Stahlbau 67(1998) H12, pp 910 - 925. (in German)

Burkhardt, S., Tragfähigkeitsermittlung von Sandwichelementen mit ebenen und schwach profilierten Deckschichten. Stahlbau 58 (1989) H1, pp. 23-27. (in German)

Chong, K.P., Engen, K.O., & Hartsock, J.A., Thermal stresses and deflection of sandwich panelsl. Proc. ASCE, J. Struct. Div. Vol. 103, No. ST1, January 1977.

Davies, J.M., The analysis of sandwich panels with profiled metal faces. Proc. Eighth Int. Speciality Conf. on Cold-formed Steel Structures. University of Missouri-Rolla, 1986. (This reference also quotes the most important equations of Stamm and Witte, 1974 above.)

Davies, J.M., Axially loaded sandwich panels. Proc. ASCE, J. Struct. Div. Vol. 113, No. 11, November 1987.

Davies, J.M. & Hakmi, M.R. 1990, Local buckling of sandwich plates. IABSE Report, Vol. 60, pp. 533-538.

Davies, J.M., Hassinen, P. & Hakmi, M.R. 1991, Face buckling stresses in sandwich panels. Proc. of the Nordic Steel Colloquium held in Odense Denmark September 9-11 1991. pp. 99-110.

Hartsock, J.A. & Chong, K.P., Analysis of sandwich panels with formed faces. Proc. ASCE. J. Struct. Div. Vol. 103, No. ST4, April 1976.

Heinisuo, M., An exact finite element technique for layered beams. Computers and Structures, Vol. 30, No. 3, 1988, pp 615 - 622.

Just, M. 1992, Zum Langzeitverhalten von Sandwichplatten mit einem Polyurethan-Hartschaumstoffkern. Bauingenieur 67(1992)83-89. (in German)

Kilpeläinen T. & Hassinen, P. 1996, Long-term and fire behaviour of an eps-foam sandwich panel product. In Sandwich Construction 3. Engineering Materials Advisory Services Ltd., London (Ed. H.G. Allen) 1996, Volume II, pp. 911-920.

Kerkkänen, T. & Tiainen, T. 1999, Durability of different types of sandwich panels under climatic conditions. Tampere University of Technology, Institute of Materials Science, Report 15/1999. Tampere. 27 p.

Lindberg R., Suonketo J., & Hassinen P. 1994, Long Term Tests on Isora-Elements, Test Report. Tampere University of Technology, Department of Civil Engineering, Publication 63, 1994. 56 p. + app. 62 p.

Linke, K.-P., Zum Tragverhalten von Profilsandwichplatten mit Stahldeckschichten und einem Polyurethan-Hartschaum-Kern bei kurz- und langzeitiger Belastung. Dissertation D17, Darmstadt 1978. (in German)

Martikainen, L., & Hassinen, P., Load-bearing capacity of continuous sandwich panels. Helsinki University of Technology, Department of Structural Engineering, Publication 135, Espoo 1996. 179 p. + app. 43 s.

Schulz, U. & Ummenhofer, T. 1995, Tragverhalten von Sandwich-elementen mit Minaralfaserkernen. Bautechnik 72(1995) H10, pp. 642-654. (in German)

Schwarze, K., Numerische Methoden zur Berechung von Sandwichelementen. Stahlbau 12/1984, pp 363 - 370. (in German)

Stamm, K., Sandwichelemente mit metallischen Deckschichten als Wandbauplatten im Bauwesen. Stahlbau 53(1984) H5, pp 135 - 143. (in German)

Stamm, K., Sandwichelemente mit metallischen Deckschichten als Dachbautafeln im Bauwesen. Stahlbau 53(1984) H8, pp 231 - 236. (in German)

Wölfel, E., Nachgiebiger Verbund: Eine Näherungslösung und deren Anwendungsmöglichkeiten. Stahlbau 56(1987) H6, pp 173 - 180. (in German)

APPENDIX A

DISCUSSION OF THE DIFFERENTIAL EQUATIONS

The basic differential equations for the calculation of sandwich elements using the theory of elastic composite action are given, for example, in references /Allen 1969 and Stamm & Witte 1974/. The equations for the deflection w and the shear strain γ for the general case of a continuous sandwich element with faces having bending stiffness and subject to direct load and temperature are:

$$-\frac{B_{F1}+B_{F2}}{G_C A_S}\cdot w^{VI}+\frac{B}{B_S}\cdot w^{IV}=\frac{q}{B_S}-\frac{q''}{G_C A_S} \qquad \text{(A1a)}$$

$$-\frac{B_{F1}+B_{F2}}{G_C A_S}\cdot \gamma^{IV}+\frac{B}{B_S}\cdot \gamma''=-\frac{q'}{G_C A_S} \qquad \text{(A1b)}$$

With this representation, the temperature stresses are considered by the inclusion of the temperature term in the formulation of the internal stress-deformation equations. For single-span sandwich elements with bending-stiff faces, by integrating the above equation twice and observing the equilibrium conditions, the following 4th order equations may also be derived:

$$-\frac{B_{F1}+B_{F2}}{G_C A_S}\cdot w^{IV}+\frac{B}{B_S}\cdot w''=-\frac{M}{B_S}-\frac{q}{G_C A_S}-\theta \qquad \text{(A2a)}$$

$$-\frac{B_{F1}+B_{F2}}{G_C A_S}\cdot \gamma''+\frac{B}{B_S}\cdot \gamma=\frac{Q}{G_C A_S} \qquad \text{(A2b)}$$

Here the temperature term $\theta = (\alpha_{F2}T_2 - \alpha_{F1}T_1)/e$ appears as an inhomogeneity in the differential equation for the deflection w.

Various methods are given in the literature for the solution of these differential equations. The classical solution involves integration of the equations with the integration constants determined by considering the support conditions. A solution using the method of finite differences is given in reference /Berner 1978/. In reference /Schwarze 1984/, a method is described in which the classical solution of differential equations is combined with the unit load method; continuous elements are split up into a row of single span elements by means of cuts and their deformation terms determined by integration of the differential equations. The compatibility of deformations at the cuts is then satisfied by the use of stress resultant components which are determined by means of the unit load method. Reference /Davies 1986/ includes an exact and completely general finite element solution. In addition, general purpose finite element methods may also used to find solutions for sandwich panels. They are applied especially in the cases where the panels are to subjected to loads in three dimensions, for example in the case of sandwich plates and shells which are, of course, beyond the scope of equations (A1) and (A2).

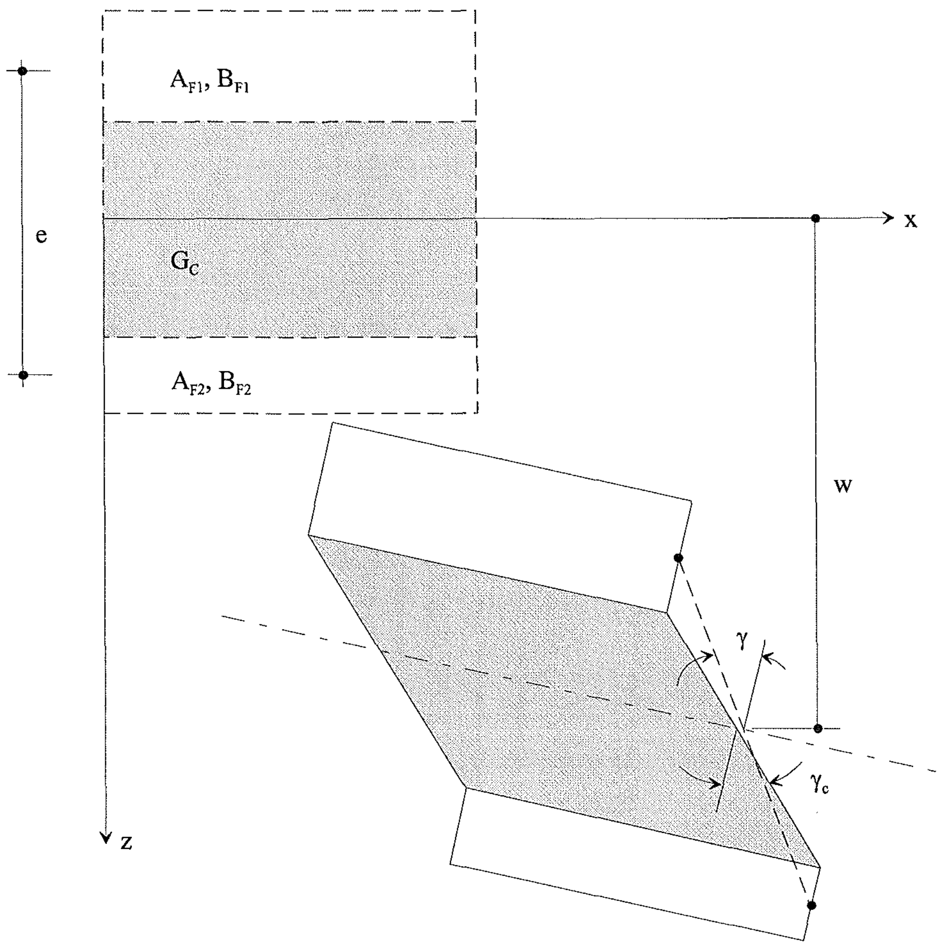


Fig. A1. Deflection w and shear strain γ in a cross-section of a sandwich beam.

The following basic information can be derived from the solution of the differential equations:

A1 For panels which are statically indeterminate by virtue of their supports, the flexibility of the core influences the total stress resultants. The distribution of the stress resultants in a given span is changed in such a way that the total moment at an internal support becomes smaller in comparison to the theoretical value for a rigid connection between the flanges. For a given shear stiffness, the reduction is the more pronounced the bigger the component of bending stiffness of the sandwich part B_S is in comparison to the total bending stiffness B. As the support moments decrease, the span moments become correspondingly larger.

A2 The division of the stress resultant components at a given cross-section follows from the relationship of the bending stiffness of the faces to the stiffness of the sandwich action. The stiffness of the sandwich action is significantly influenced by the shear stiffness of the core. The smaller the shear stiffness of the core, the larger is the load component which must be carried by the faces. The load component in the faces (and thus the load component in the sandwich part) varies over the length of the element.

A3 As a result of time-dependent influence (creep of the core under load) the load component carried by the sandwich part reduces with the passage of time and the component carried by the faces increases.

A4 Temperature differences between the upper and lower faces lead to both a change in the total stress resultants over the whole span of sandwich elements with statically indeterminate supports as well as a change in the components of the stress resultants at a particular cross-section. These changes are so large that they cannot be neglected in the design of sandwich elements.

The following example illustrates the distribution of bending moments and shear forces in a statically indeterminate three-span sandwich beam, the upper face of which is profiled (Fig. A2). Therefore, the sandwich panel is classified to be a thick-faced sandwich beam in which the stress resultants M and V are divided into the part carried by the sandwich structure, M_S and V_S , and the part carried out by the profiled face, M_{F1} and V_{F1}. The example shows the response of the sandwich panel under a uniformly distributed load, q = 1 kN/m, a temperature difference between the faces of the panel, $T_1 = 0$ and $T_2 = 20°C$, and a line load in a span, F = 1.2 kN. The influence of the shear creep of the core is described by reducing the shear modulus, $G_{Ct} = G_C/(1+\varphi_t)$, by the use of a creep coefficient $\varphi_t = 2.4$. The results have been computed using software based on the exact finite element method for sandwich beams.

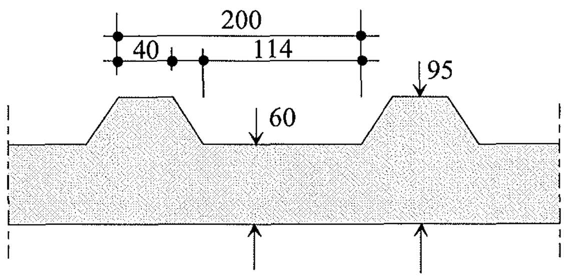


Fig. A2. Cross-section of the sandwich beam in Example A1. The total depth and width of the panel are D = 95 mm and B = 1000 mm, respectively.

Table A1. Geometrical and material properties of the sandwich panel in Example A1.

Face 1	Core	Face 2
$E_{F1} = 210000$ N/mm^2 $t_1 = 0.59$ mm $A_{F1} = 701.4$ mm^2 $I_{F1} = 143560$ mm^4 $B_{F1} = 30.15\ 10^9$ Nmm^2 $d_{11} = 22.95$ mm $d_{12} = 12.05$ mm $\alpha_{F1} = 0.000012$ 1/°C	$G_C = 4.00$ N/mm^2 e = 71.5 mm $A_S = 71050$ mm^2 $\varphi_t = 0$ and $\varphi_t = 2.4$ $B_S = 317.06\ 10^9$ Nmm^2	$E_{F2} = 210000$ N/mm^2 $t_2 = 0.51$ mm $A_{F2} = 510$ mm^2 $I_{F2} = 0$ $B_{F2} = 0$ $\alpha_{F2} = 0.000012$ 1/°C

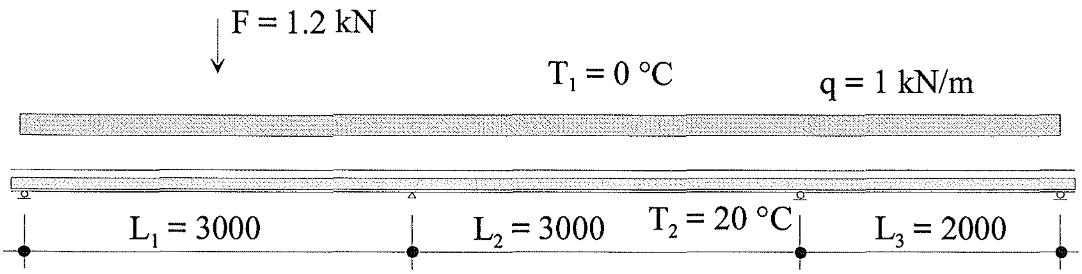


Fig. A3. Static system and loads for the sandwich panel in Example A1. The span lengths are $L_1 + L_2 + L_3 = 3000 + 3000 + 2000$ mm.

Table A2. Support reactions for the four loading cases in Example A1.

Support reaction / Load	F_1 , kN	F_2 , kN	F_3 , kN	F_4 , kN
Uniform load q = 1 kN/m ($\varphi_t = 0$)	1.209	3.391	2.687	0.713
Uniform load q = 1 kN/m ($\varphi_t = 2.4$)	1.224	3.364	2.697	0.716
Temperature difference $T_1 = 0$ and $T_2 = 20$°C	-0.377	0.426	0.441	-0.491
Line load F = 1.2 kN at $L_1/2$	0.504	0.807	-0.131	0.021

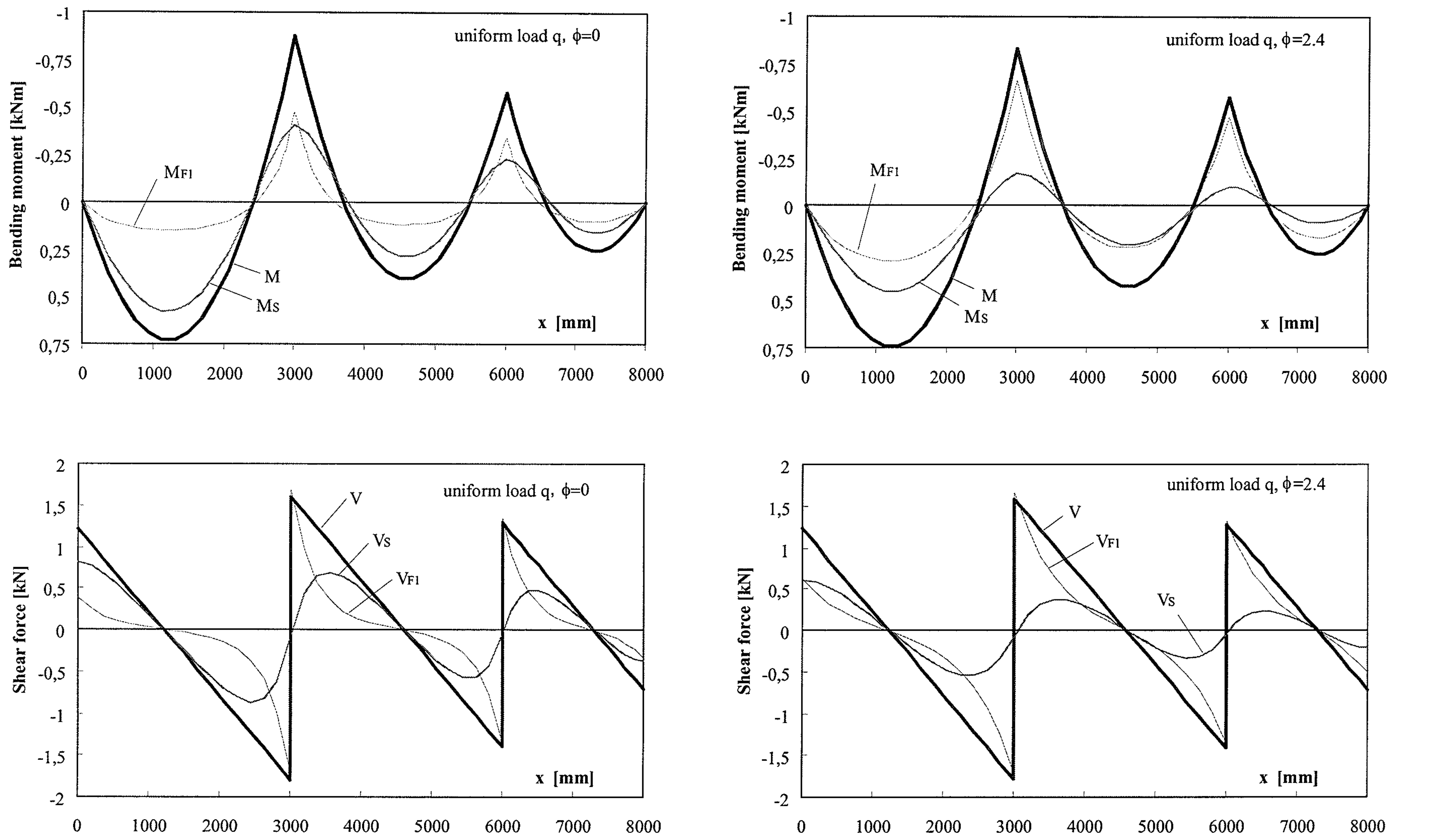


Fig. A4. Bending moments and shear forces of a three-span sandwich beam loaded by an uniformly distributed load of q=1 kN/m^2. Resultants for creep coefficients φ_t=0 and φ_t=2.4.

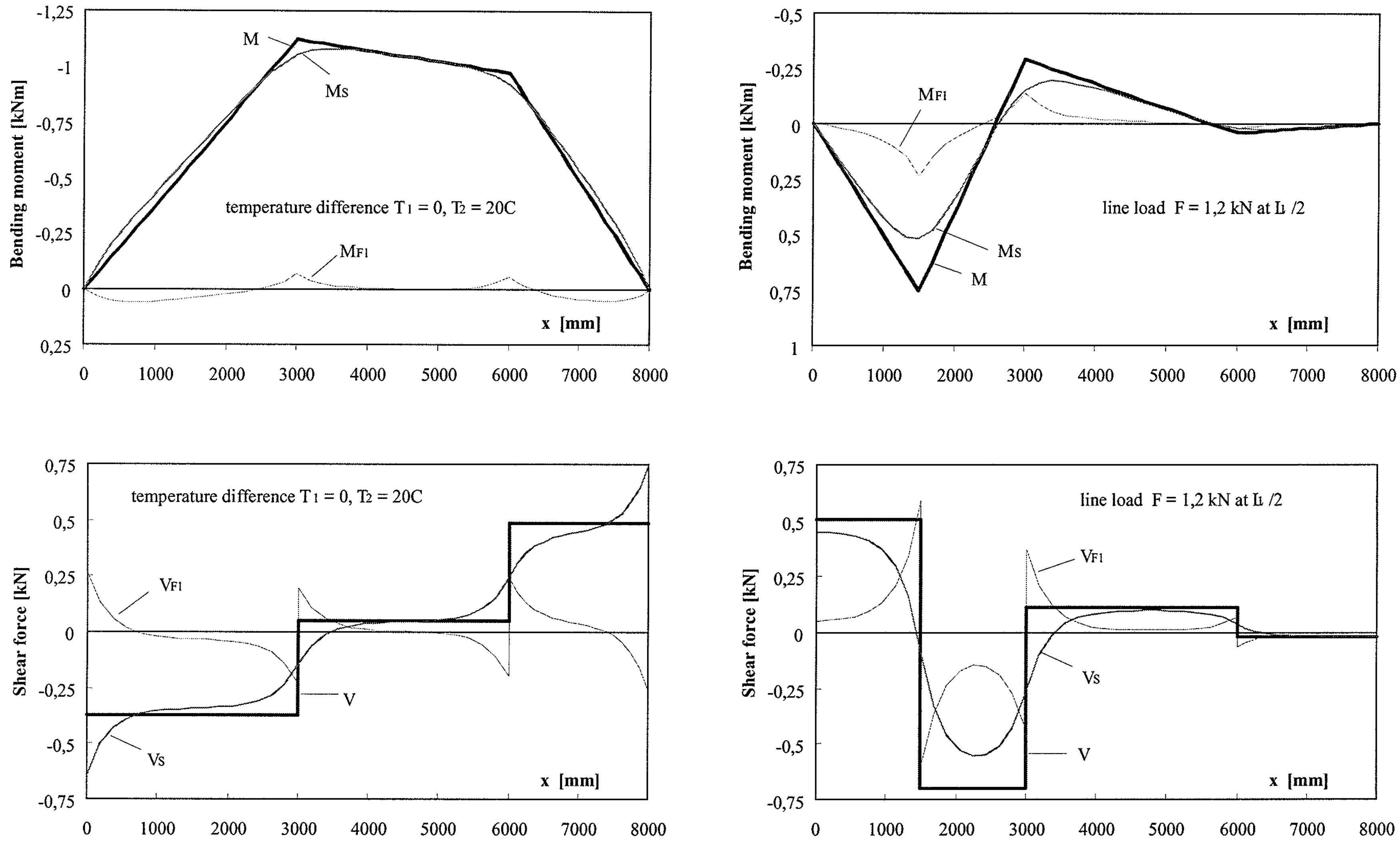


Fig. A5. Bending moments and shear forces of a three-span sandwich beam loaded by a temperature difference of (T_1 = 0 °C and T_2 = 0 °C) and a line load of F = 1.2 kN in the middle of the first span.

References for Appendix A

Allen, H.G. 1969, Analysis and design of structural sandwich panels; Pergamon, Oxford 1969.

Berner, K. 1978, Stahl/Polyurethan-Sandwichtragwerke unter Temperatur und Brandbeanspruchung. Dissertation D17. Darmstadt 1978. (in German)

Davies, J.M. 1986, The analysis of sandwich panels with profiled metal faces. Proc. Eighth Int. Speciality Conf. on Cold-formed Steel Structures. University of Missouri-Rolla, 1986.

Linke, K.-P. 1978, Zum Tragverhalten von Profilsandwichplatten mit Stahldeckschichten und einem Polyurethan-Hartschaum-Kern bei kurz- und langzeitiger Belastung. Dissertation D17, Darmstadt 1978. (in German)

Schwarze, K. 1984, Numerische Methoden zur Berechnung von Sandwichelementen. Stahlbau 53 (1984) H. 12, pp. 363-370. (in German)

Stamm, K. & Witte, H. 1974, Sandwichkonstruktionen - Berechnung, Fertigung, Ausführung. Springer-Verlag, Wien and New York, 1974. (in German)

APPENDIX B

TIME-DEPENDENT STRESS DISTRIBUTION

B1 Creep deformations of sandwich panels

The deformations of sandwich elements increase with time under long term actions. The increase in deformation is caused by the creep of the core under the action of permanent shear stresses. With constant shear stresses, the shear strain increases in time t from

γ_o to γ_t

where $\gamma_t = \gamma_o (1 + \varphi_t)$ (B1)

In equation (B1), φ_t is the creep coefficient and γ_o is the initial value of the shear strain which may be measured at a nominal start time of t_o = 0.1 hours = 6 minutes from the commencement of loading.

The hypothesis concerning the proportionality between time-dependent deformation and magnitude of load which underlies equation (B1) is generally satisfied when the stresses in the core are those at the expected working loads. The magnitude of φ_t depends essentially on the type of the core material, on the time of initial loading, on the temperature and on the humidity. Measurements /Just 1992/ show that increases of deformation still appear after over 3 years of continuous loading. Creep coefficients of up to 7 must be expected.

B2 Thin faced sandwich panels

If the both faces of the sandwich panel are flat or lightly or micro profiled, the initial deflection w_0 of the panel can be expressed as the sum of the deflections caused by the elastic extensions of the metal sheet faces w_b and by the shear deformations of the core w_v

$$w_0 = w_b + w_v \tag{B2}$$

Only the shear deformations of the core increase with time so that the deflection after a loading time of t is

$$w_t = w_b + w_v \left(1 + \varphi_t\right) \tag{B3}$$

From (B2) and (B3) follows

$$w_t = w_o + \varphi_t \left(w_o - w_b\right) \tag{B4}$$

Using equation (B2) in the form $w_b = w_0 / \left(1 + k_1\right)$ the expression (B3) can also be written in the form

$$w_t = w_o\left(1+\frac{k_1}{\left(1+k_1\right)}\varphi_t\right) = w_o\left(1+\varphi_{S,t}\right) \tag{B5}$$

where $k_1 = w_v/w_b$ and $\varphi_{S,t} = \varphi_t\,(k_1/(1+k_1))$ (B6a,b)

The creep coefficient φ_t used in the expressions (B3), (B4) and (B5) has to be determined from expression (B7) based on the experimentally measured initial deflection w_0 and the deflection w_t after the loading time of t. The partial deflection w_b in the expression (B7) has to be determined by calculations (See Clause 5.2.6). The expression (B7) is valid for sandwich panels with plane or lightly or micro profiled faces.

$$\varphi_t = \frac{w_t - w_o}{w_o - w_b} = \frac{1+k_1}{k_1}\frac{w_t - w_o}{w_o} \tag{B7}$$

B3 Thick faced sandwich panels

If one or both faces of a sandwich panel are profiled, the creep coefficient can be evaluated from the expression (B8). The expression is derived from the equation for the mid-span deflection of a thick faced sandwich beam. The method is approximate because it assumes the relationship between the stress resultants in the sandwich part of the structure and in the profiled faces to be a constant and, secondly, because it takes into account the change of stress resultants from the sandwich part of the structure to the profiled faces due to the creep by means of a relaxation coefficient ρ. The background to this method is presented in the reference /Wölfel 1987/.

$$\varphi_t = \frac{\beta\left(C_D - 1\right)}{\beta_1\left(1-\beta-\beta\rho\left(C_D-1\right)\right)} \tag{B8}$$

where

$C_D = \frac{w_t}{w_0}$ is the relationship between the deflection after a loading time of t and the initial deflection (B9)

$\rho = 0.5$ is a relaxation coefficient, having here the value of 0.5 (B10)

$$\beta = \frac{B_{F1} + B_{F2}}{\overline{B}} \tag{B11}$$

$$\beta_1 = \frac{k_2\,\beta}{1+k_2} \tag{B12}$$

$$\overline{B} = B_{F1} + B_{F2} + \frac{B_S}{1+k_2} = B_{F1} + B_{F2} + \gamma\, B_S \quad \text{simulated bending rigidity} \tag{B13}$$

$$\gamma = \frac{1}{1+k_2} \tag{B14}$$

$$k_2 = \frac{\pi^2 B_S}{L^2\, G_C A_S} = \frac{\pi^2 E_F A_{F2}\, e^2}{\left(\left(\frac{A_{F2}}{A_{F1}}\right)+1\right) G_C A_S L^2} \tag{B15}$$

$$A_S = e\,B \tag{B16}$$

The parameter k_2 given in (B15) is exactly valid for a sinusoidal $\left(q \sin(\pi x/L)\right)$ load on the sandwich beam.

Instead of using the expression (B8), the creep coefficient of the sandwich panels with profiled faces may be determined numerically on the basis of the analytical or numerical methods. In these procedures, the experimental and calculated deflections are equated by varying the creep coefficient in the expression for the shear modulus $G_{Ct} = G_C/\left(1+\varphi_t\right)$ used to describe the shear deformations of the core. Iterative methods are required to find creep coefficients for the different periods of time.

B4 Determination of the creep coefficients for design

The creep coefficients for a certain loading time, which are needed in the design of sandwich panels are interpolated or extrapolated from the test results. Extrapolation of test observations over a greater time period than about ten times the period of observation is problematical because rules on the basis of simple representative models are not available at present. Using a linear extrapolation in a semi-logarithmic diagram on the basis of experimental results φ_{exp1} and φ_{exp2} at times t_1 and t_2 leads to the extrapolation formula (B17). In this expression, the coefficient $\gamma_c = 1.2$ is a factor which increases the experimentally defined creep coefficient by 20% in order to take into account the fact that a part of the creep deformations of typical core materials is not recovered during the unloading phase, i.e., in Summer time.

$$\varphi_t(t) = \frac{1.2\left[\varphi_{exp2}\left(\lg(t)-\lg(t_1)\right) - \varphi_{exp1}\left(\lg(t)-\lg(t_2)\right)\right]}{\lg(t_2)-\lg(t_1)} \tag{B17}$$

where φ_{exp1} and φ_{exp2} are the experimental creep coefficients at times t_1 and t_2.

If the measurement times are defined as $t_1 = 200$ h and $t_2 = 1000$ h, the expression (B17) can be written as follows

$$\varphi_t(t) = \frac{1.2\left[\varphi_{exp2}\left(\lg(t) - \lg(200)\right) - \varphi_{exp1}\left(\lg(t) - \lg(1000)\right)\right]}{\lg(1000) - \lg(200)} \tag{B18}$$

The extrapolation of the creep coefficient on the basis of the experimental values of $\varphi_{exp,1}$ and $\varphi_{exp,2}$ at $t_1 = 200$ h and $t_2 = 1000$ h for snow load persisting for 2000 hours gives the following expression for φ_{2000}

$$\varphi_{2000} = 1.2\left(1.43\varphi_{1000} - 0.43\varphi_{200}\right) = 1.7\left(\varphi_{1000} - 0.3\varphi_{200}\right) \tag{B19}$$

The creep coefficient for 2000 hours is used in the calculations of the creep deformations caused by the snow load, assuming that the Winter period lasts about 3 months and that most of the creep deformations recover during the summer period.

The extrapolation of the creep coefficient for the permanent load lasting 100000 hours on the basis of the experimental values of $\varphi_{exp,1}$ and $\varphi_{exp,2}$ at $t_1 = 200$ h and $t_2 = 1000$ h gives the following expression for φ_{100000}

$$\varphi_{100000} = 3.86\varphi_{1000} - 2.86\varphi_{200} \tag{B20}$$

In the expression (B20), there is no additional factor to increase the creep coefficient because of non recovered creep deformations ($\gamma_c = 1.0$) because the load is constant during the whole period of time.

The creep coefficient for 100000 hours ($\approx$ 12 years) is used for permanent loads. The creep coefficient for loading of such large duration cannot be reliably predicted from tests with a practical time scale. For this reason, the indicative values of φ_{100000} given in section 3.6 of these Recommendations should normally be used. As permanent load (self-weight) is usually a small proportion of the total load, the approximations involved in this procedure are not significant.

B5 Examples

Two long-term loading tests, A and B, were made with sandwich panels having profiled faces. Table B1 gives the span and geometrical, material and structural dimensions of the simply supported one-span test specimens. The measured mid-span deflections and the analysis of the creep coefficients are presented following the procedure given in Clauses B3 and B4.

Table B1. Dimensions, loads, deflections and creep coefficients of single-span test panels as in Example B1.

<table>
<tr><th></th><th>Test panel A</th><th>Test panel B</th></tr>
<tr><td>Span, L (mm)</td><td>4000</td><td>4000</td></tr>
<tr><td>Load, q (kN/m)</td><td>3.625</td><td>3.635</td></tr>
<tr><td>Shear stress, τ_C (N/mm^2)</td><td>0.049</td><td>0.049</td></tr>
<tr><td>Relative shear stress, τ_C / f_{Cv} (%)</td><td>28</td><td>28</td></tr>
<tr><td>Flexural rigidity of faces, B_1+B_2 (Nmm^2)</td><td>$23.24\ 10^9$</td><td>$23.24\ 10^9$</td></tr>
<tr><td>Distance, e_1 (mm)</td><td>39.86</td><td>39.86</td></tr>
<tr><td>Distance, e_2 (mm)</td><td>66.34</td><td>66.34</td></tr>
<tr><td>Distance, $e=e_1+e_2$ (mm)</td><td>106.2</td><td>106.2</td></tr>
<tr><td>Total width, B (mm)</td><td>1000</td><td>1000</td></tr>
<tr><td>Modulus of elasticity of face, E_F (N/mm^2)</td><td>210000</td><td>210000</td></tr>
<tr><td>Area of face1, A_1 (mm^2)</td><td>684</td><td>684</td></tr>
<tr><td>Area of face 2, A_2 (mm^2)</td><td>411</td><td>411</td></tr>
<tr><td>Area of core, $A_S = e\ B$ (mm^2)</td><td>1062</td><td>1062</td></tr>
<tr><td>Shear modulus, G_C (N/mm^2)</td><td>3.29</td><td>3.29</td></tr>
<tr><td>Parameter, k_2 (Eq. B15)</td><td>1.074</td><td>1.074</td></tr>
<tr><td>Flexural rigidity, $\overline{B}$ (Nmm^2) (Eq. B13)</td><td>$316.493\ 10^9$</td><td>$316.493\ 10^9$</td></tr>
<tr><td>Parameter, β (Eq. B11)</td><td>0.073</td><td>0.073</td></tr>
<tr><td>Parameter, β_1 (Eq. B12)</td><td>0.038</td><td>0.038</td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td>Initial deflection, w_0 (mm)</td><td>35.04</td><td>35.68</td></tr>
<tr><td>Increase of deflection, ($w_{200} - w_0$) (mm)</td><td>11.69</td><td>12.53</td></tr>
<tr><td>Increase of deflection, ($w_{1000} - w_0$) (mm)</td><td>20.12</td><td>23.15</td></tr>
<tr><td>Parameter, $C_{D,200} = w_{200} / w_0$</td><td>1.334</td><td>1.351</td></tr>
<tr><td>Parameter, $C_{D,1000} = w_{1000} / w_0$</td><td>1.574</td><td>1.649</td></tr>
<tr><td>Experimental creep coefficient, φ_{200}</td><td>0.705</td><td>0.742</td></tr>
<tr><td>Experimental creep coefficient, φ_{1000}</td><td>1.225</td><td>1.388</td></tr>
<tr><td>Extrapolated creep coefficient, φ_{2000}</td><td>1.738</td><td>1.999</td></tr>
<tr><td>Extrapolated creep coefficient, φ_{100000}</td><td>2.712</td><td>3.235</td></tr>
<tr><td>Mean creep coefficient, φ_{2000}</td><td colspan="2">1.869</td></tr>
<tr><td>Mean creep coefficient, φ_{100000}</td><td colspan="2">2.974</td></tr>
<tr><td>Creep coefficient for design, φ_{2000}</td><td colspan="2">2.0</td></tr>
<tr><td>Creep coefficient for design, φ_{100000}</td><td colspan="2">7.0</td></tr>
</table>

References for Appendix B

Berner, K. 1978, Stahl/Polyurethan-Sandwichtragwerke unter Temperatur und Brandbeanspruchung. Dissertation. Darmstadt 1978. (in German)

Burkhardt, S. 1989, Zeitabhängige Verformungen von Sandwichelementen. Bauingenieur 64(1989) pp. 327-331. (in German)

Jungbluth U. & Berner K. 1986, Verbund und Sandwichtragwerke. Springer-Verlag, 1986. (in German) pp. 336-559.

Just, M., Ergebnisse experimenteller Untersuchungen zum Langzeitverhalten von PUR-Hartschaumstoff-Stützkernbauteilen und Schlussfolgerungen für die Anwendung; 1030. Mitteilung aus dem Institut für Leichtbau und ökonomische Verwendung von Werkstoffen, Dresden. (in German)

Just, M. 1992, Zum Langzeitverhalten von Sandwichplatten mit einem Polyurethan-Hartschaumstoffkern. Bauingenieur 67(1992) pp. 83-89. (in German)

Kilpeläinen, T. & Hassinen, P. 1995, Long-term behaviour and strength of EPS foam sandwich panels. Proc. of Nordic Steel Construction Conference '95 held in Malmö Sweden June 19-21, 1995. Swedish Institute of Steel Construction. Publication 150. 1995. pp. 645-652.

Linke, K.-P. 1978, Zum Tragverhalten von Profilsandwichplatten mit Stahldeckschichten und einem Polyurethan-Hartschaum-Kern bei kurz- und langzeitiger Belastung. Dissertation D17, Darmstadt 1978.

Stamm, K. & Witte, H. 1974, Sandwichkonstruktionen; Springer-Verlag, Wien 1974. (in German)

Wölfel, E. 1987, Nachgiebiger Verbund: Eine Näherungslösung und deren Anwendungsmöglichkeiten. Stahlbau 56(1987) H6, pp 173 - 180. (in German)

APPENDIX C

THE INFLUENCE OF EXTRA LAYERS BETWEEN THE FACES AND THE CORE

C1 Four and five layer sandwich panels

In some cases, extra layers of various boards are used between the face and the core. The primary reason is to improve the fire resistance of panels by protecting the core from exposure to high temperature and flames. Gypsum, chip boards or other building boards may be used for this purpose. A typical cross section of such a panel is shown in Fig. C1. The extra layer is glued to the core and to the thin metal face. The board and the metal face act as the face when properly bonded together. This face is naturally much stiffer and stronger than the metal face alone. It is not susceptible to wrinkling and it also distributes point loads in a smoother way to the core. This increase in load bearing capacity and rigidity is usually not taken into account in design.

There are nevertheless some specific features that need to be considered when assessing this type of panel. This appendix introduces these features briefly and the view is limited to panels with flat faces as in Fig. C1.

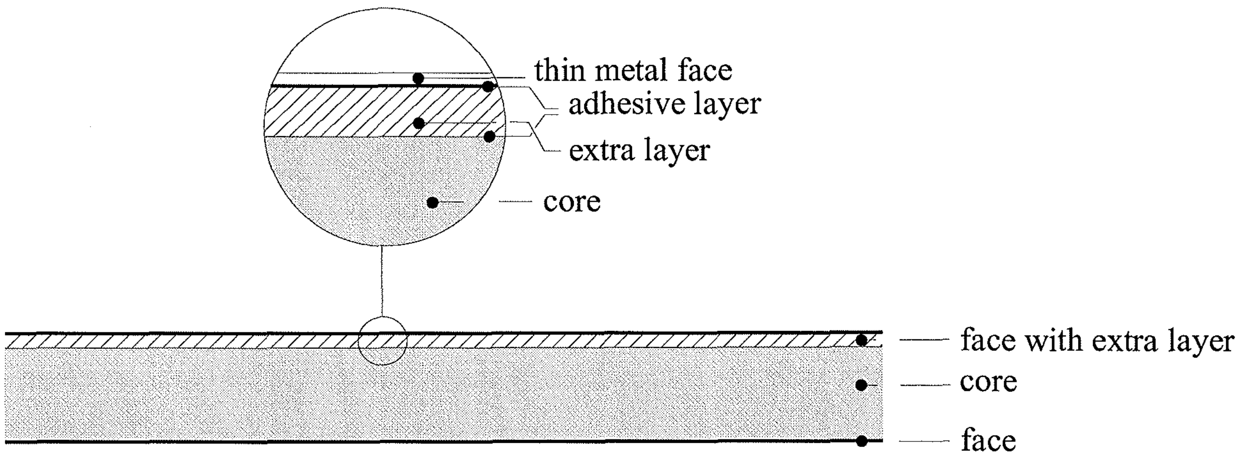


Fig. C1. Cross-section of a four layer sandwich panel.

C2 Joints between boards

The building boards are produced with fixed dimensions that usually are smaller than the size of the panels. This means that extra layers consist of separate boards and joints between them cannot be avoided (see Fig. C2). Boards are usually butt jointed across the panel. Joints can transfer compressive forces but not tensile forces unless special joining techniques are used. The combined face has some bending stiffness of its own. The joints are, however, discontinuities in the face and the face is susceptible to kink along a joint if it is subjected to a moment that tends to 'open' the joint. The following aspects should be noted in the manufacture, the quality control and the design:

- The boards should be positioned carefully so that they form a smooth and even surface for the metal face.
- Butt jointing should be made tightly without any gaps between boards.
- The joint should be placed in a panel so that it is not subjected to a moment that opens the joint. For example a joint should not be near a support if the lower face has a butt jointed extra layer in order to avoid kinking in the face.

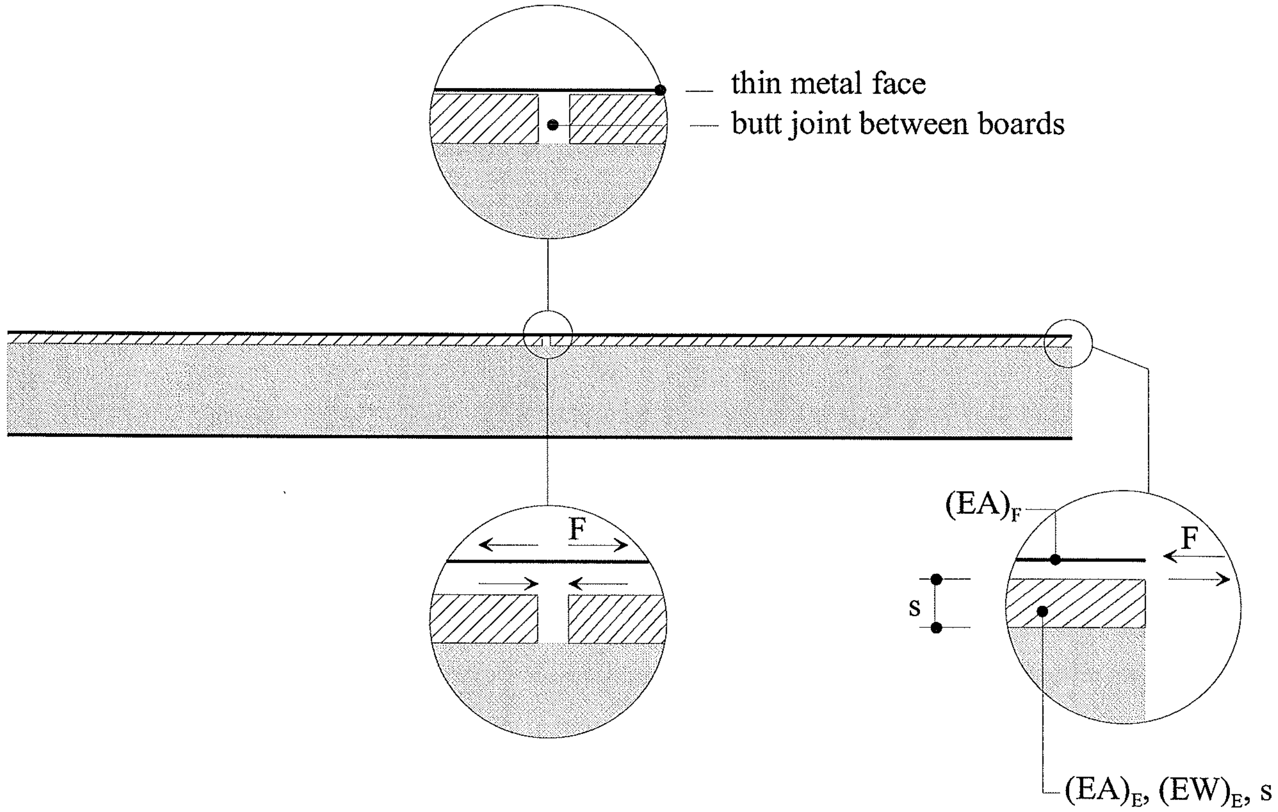


Fig. C2. Stress resultants in the extra layer of a four layer sandwich panel.

C3 Different material properties

The boards have a different behaviour than the metal face when subjected to variations in temperature and humidity. They may cause high stress concentrations in the combined face. A panel with flat metal faces only does not react on the variations in humidity, but wooden based boards would exhibit very large strains in this case without the restraint from the metal face. The boards are usually beneath diffusion tight metal faces but if variations in humidity of the boards can take place this will result in considerable stresses, as in the case of temperature loading.

The theoretical solution for the stresses between the metal face and the boards when the flexibility of the adhesion layer is neglected is in the form of two point loads at the ends of the panel (see Fig. C2). The magnitude of the point load F follows from the compatibility of the strains:

$$F = \frac{\Delta\varepsilon}{\frac{1}{(EA)_F} + \frac{1}{(EA)_E} + \frac{s}{2(EW)_E}} \tag{C.1}$$

where

$$\Delta\varepsilon = \alpha_F \Delta T_F - \alpha_E \Delta T_E + \kappa_F \Delta H_F - \kappa_E \Delta H_E \tag{C.2}$$

	is the difference between the strains in the metal face and the board due to the variation in temperature or humidity
α_F, α_E	are coefficients of thermal expansion for face and board, respectively
κ_F, κ_E	are coefficients of linear expansion for face and board caused by the change of humidity
ΔT_F, ΔT_E	are the change of temperature of the face and board
ΔH_F, ΔH_E	are the change of relative humidity of the face and board
$(EA)_F$	is the axial stiffness of the metal face.
$(EA)_E$	is the axial stiffness of the board.
s	is the thickness of the board.

$(EW)_E$ is the product of the modulus of elasticity and the section modulus of the board.

Note that the force is independent of the lengths of the board or the panel. It acts at the ends of the panel and also in the joints if the board tends to shorten relative to the metal face (as shown in Fig. C2). There is some resilience in the adhesion layer but nevertheless high stress concentrations exist at the ends of a panel. In the assessment of the panel, the reliability of the bonding between the metal face and the boards should be tested:

- The testing should be made using the anticipated conditions i.e. the variations in temperature and humidity with the expected number of cycles.

In general, the following rules should be followed:

- The boards used as extra layers should be such that the strength and the rigidity of the panel will not be diminished due to them.

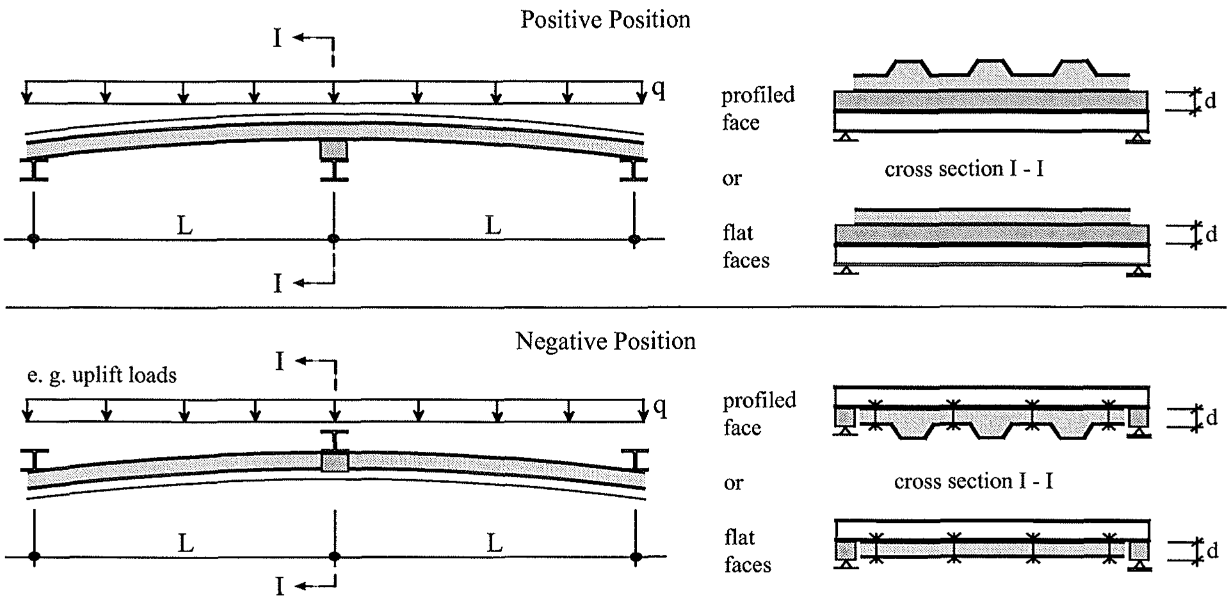
- Extra boards should not be placed on the side where condensation is anticipated.

APPENDIX D

DIRECT COMBINATION OF LOAD CASES q AND ΔT IN FULL-SCALE TESTS

D1 Two span panels, general remarks

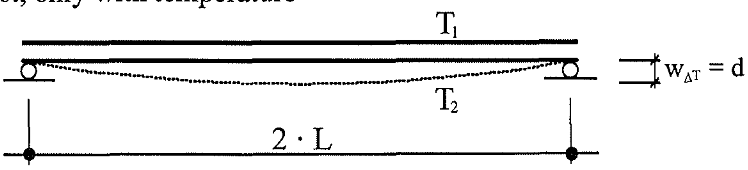
Test arrangement with simulated temperature actions with the help of a distance piece with thickness "d".



Fixing of d

a) On the base of a test, only with temperature

$\Delta T = T_2 - T_1$



or

b) On the base of the formulas (section 3.4.1 and 3.5.1)

for panels with plane or lightly profiled faces:

$$d = \frac{\Theta \cdot (2 \cdot L)^2}{8}$$

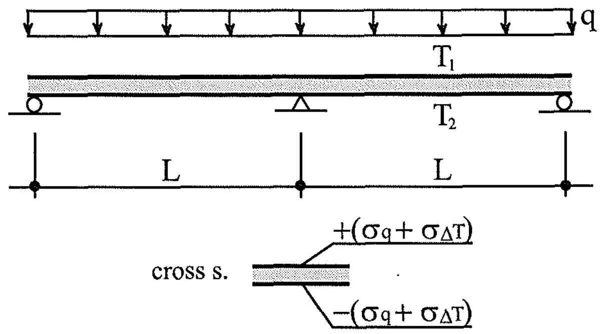
for panels with strongly profiled faces (section 3.5.1)

$$d = \frac{\Theta \cdot (2 \cdot L)^2}{8} \cdot (1 - \beta_T)$$

$$\Theta = \frac{\alpha_{T2} \cdot T_2 - \alpha_{T1} \cdot T_1}{1}$$

Result $\boxed{\textbf{load } q_{rk} = q_{rk}(q, \Delta T)}$ q_{rk} is the load of first wrinkling or yielding of the face

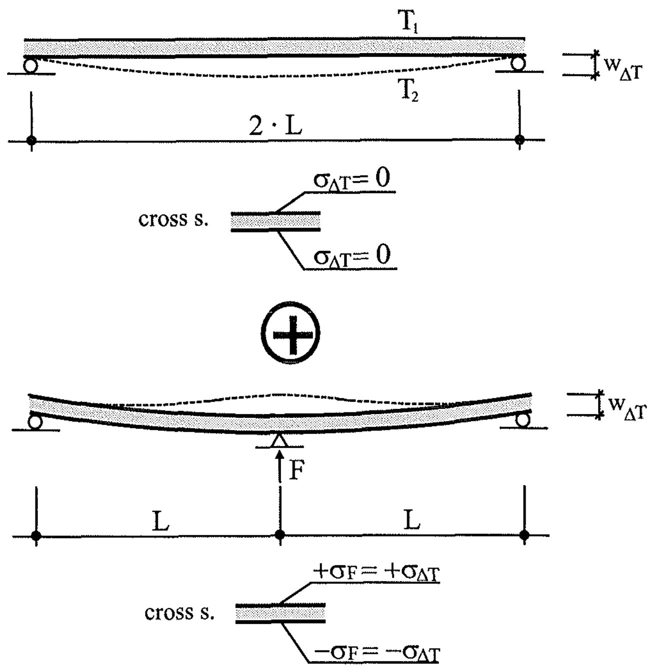
D1.1 Panels with flat or lightly profiled faces



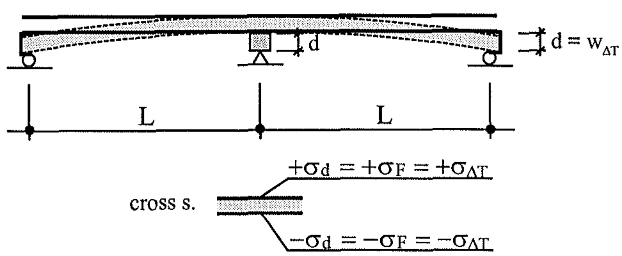
Full scale test with load and temperature at the same time (most realistic test for combination of both action efforts).

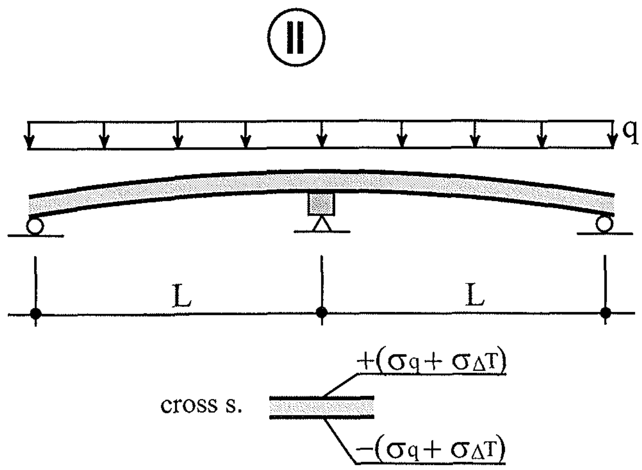
This test can be replaced by this test

Full scale test with load and simulated temperature effect with the help of a distance piece with thickness "d".

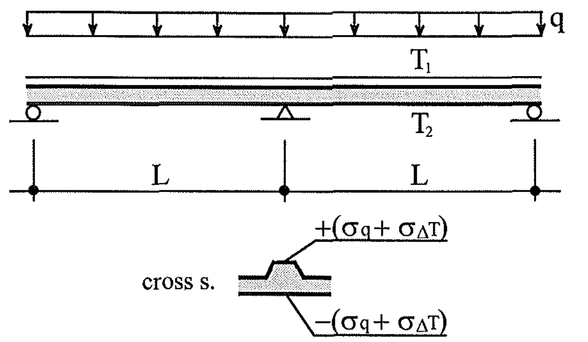


The same stresses due to temperature ΔT are caused by a test arrangement with distance piece over the middle support.





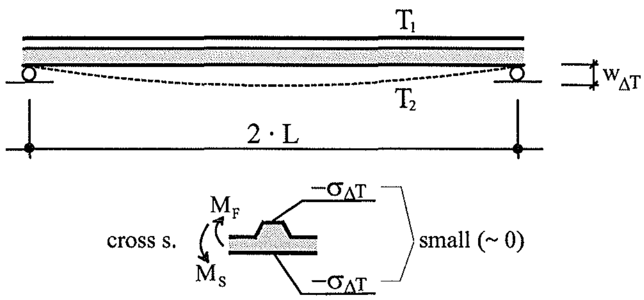
D1.2 Panels with profiled faces



Full scale test with load and temperature at the same time (most realistic test for combination of both action efforts).

This test can be replaced by this test

Full scale test with load and simulated temperature effect with the help of a distance piece with thickness "d".



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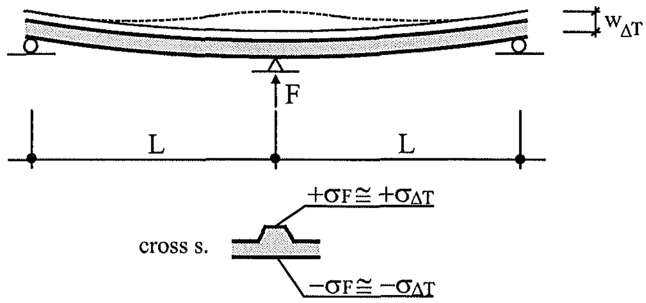
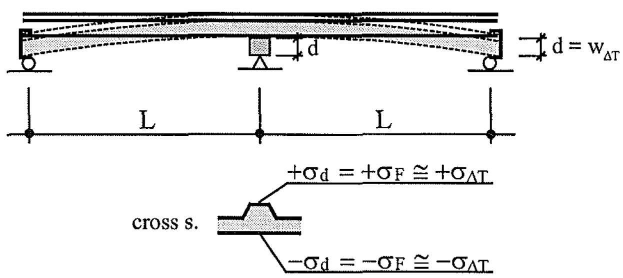


The same stresses due to temperature ΔT are caused by a test arrangement with distance piece over the middle support.



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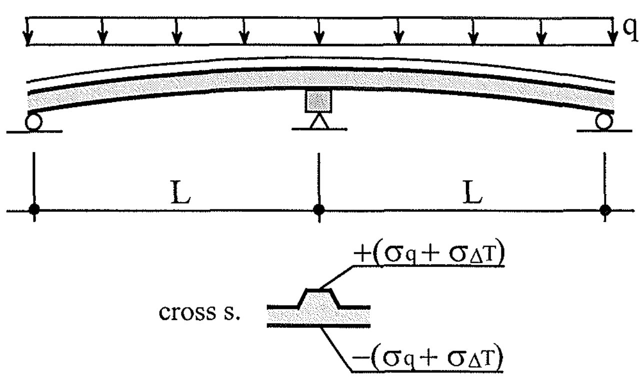


D2 One span panels

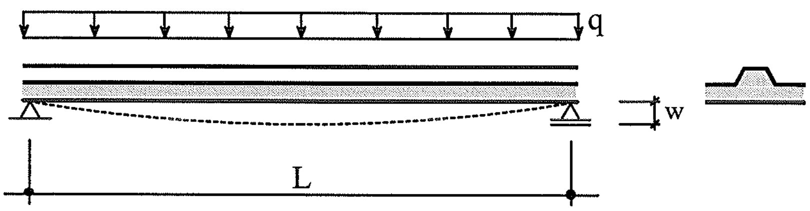
Full scale tests , with simulated actions by considering of $\Delta q_{\Delta T}$

D2.1 Panels with plane or lightly profiled faces

no influence of ΔT

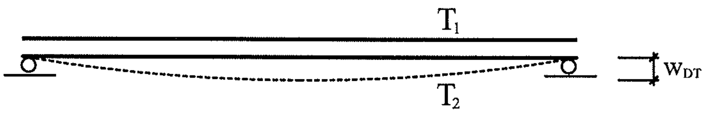
D2.2 Panels with strongly profiled faces:

Positive position



fixing of $w_{\Delta T}$

1) on the base of a test, only with temperature

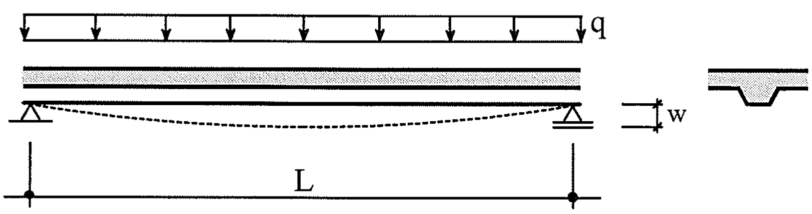


2) on the base of the formulas (section 3.5.1)

$$w_{\Delta T} = \frac{\Theta \cdot L^2}{8} \cdot (1 - \beta_T)$$

Θ, β_T see section 3.5.1

Negative position



fixing of $w_{\Delta T}$

$$w_{\Delta T} = w_{\Delta T,\mathrm{flatfaces}} - w_{\Delta T,\mathrm{prof.faces}}$$

1) on the base of a test, only with temperature

$w_{\Delta T,\mathrm{prof.\ faces}}$

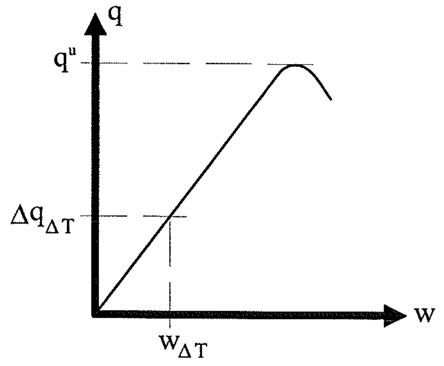
$$w_{\Delta T,\mathrm{flat\ faces}} = \frac{\Theta \cdot L^2}{8}$$

2) on the base of the formulas (section 3.5.1)

$$w_{\Delta T} = \frac{\Theta \cdot L^2}{8} \cdot (\beta_T)$$

β_T see section 3.5.1

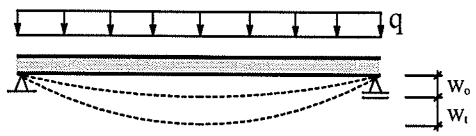
Definition of $\Delta q_{\Delta T}$: Test-result, load q

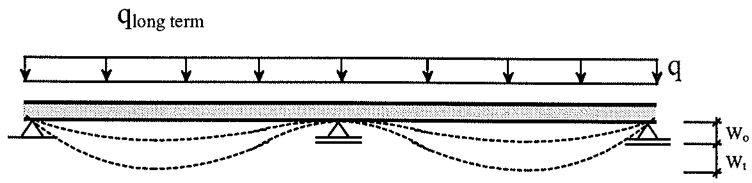


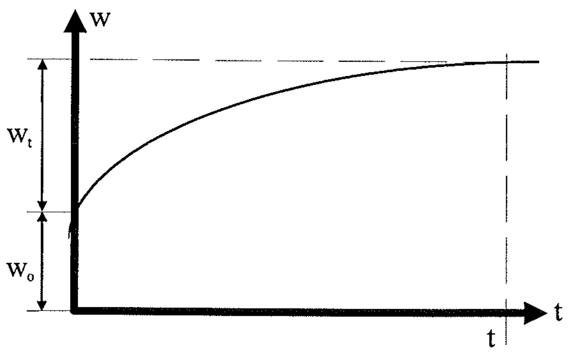
$$q_{rk} = q^u - \Delta q_{\Delta T}$$

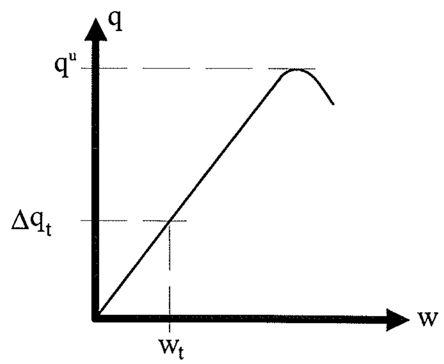
D3 Creep

Positive Position









$$q_{rk} = q^u - \Delta q_t$$

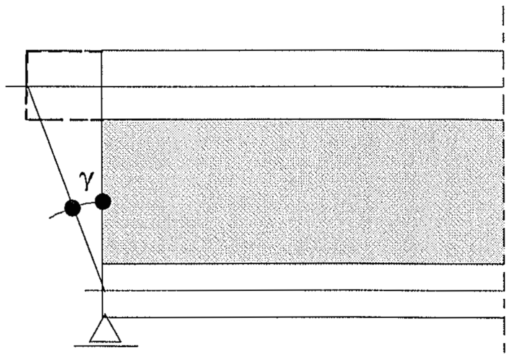
APPENDIX E

METHOD TO DETERMINE THE RELATIVE DISPLACEMENT BETWEEN THE INTERNAL AND EXTERNAL FACE AT THE HEAD OF A FASTENER

E1 Mechanisms which cause deflections

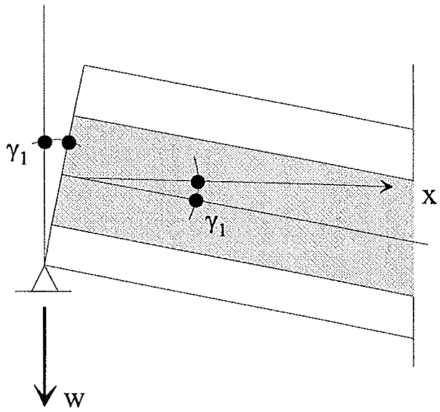
Shear-strain γ due to elastic shear deformation in the core

$$\gamma = \frac{\tau}{G_C}$$



Bending rotation at the support

$$\gamma_1 = \frac{dw}{dx}$$



u = imposed deflection of the fastener, being the relative displacement between the inner and the outer skins at the head of the fastener. u is related to reference point A in figure E1.

u = $u(\gamma) - u(\gamma_1)$

In accordance with 5.2.17 the requirement is $u < \Delta f$

The displacement u of the head of the fasteners can be evaluated on the basis of the superposition of displacements $u(\gamma)$ and $u(\gamma_1)$ as shown in figure E1.

$$u = \gamma_1 \cdot l' - \gamma \cdot h$$

In the following, calculation methods are given to determine γ and γ_1 for different static systems and load cases.

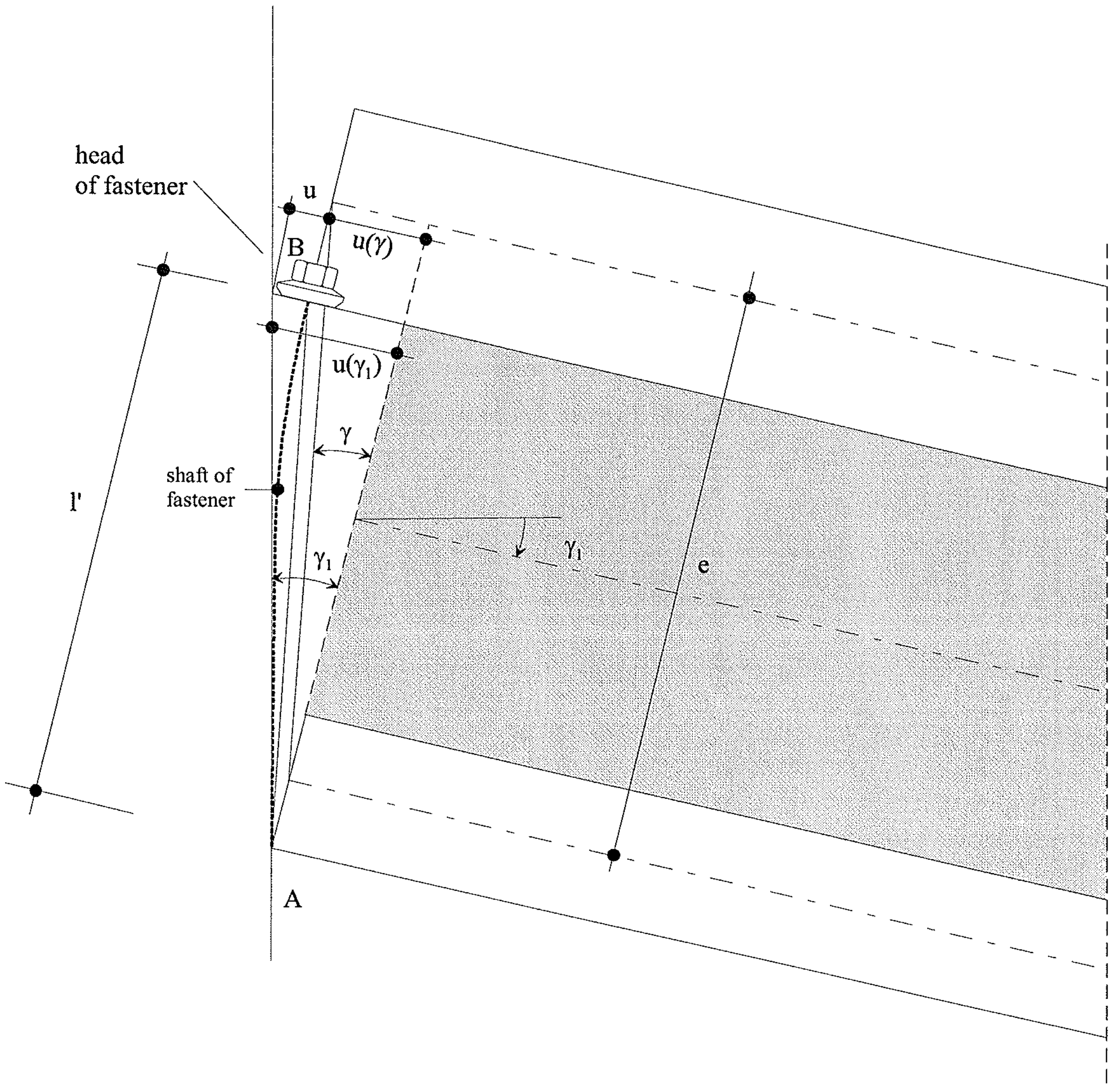


Fig. E1. Displacements at the end of sandwich panel.

E2 Calculation methods for γ and γ_1

The parameters and symbols which are used in these methods are given in Clause 1.2. In addition:

$$\delta = e B G_C$$

$$\alpha = \frac{B_{F1} + B_{F2}}{B_S}$$

$$\beta = \frac{B_S}{\delta L^2}$$

$$\lambda^2 = \frac{1+\alpha}{\alpha\beta}$$

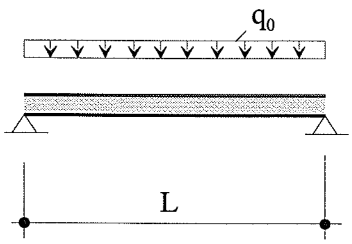
E2.1 Sandwich panels with flat faces

E2.1.1 Simply supported panel with uniformly distributed load q_o

$$Q = \frac{q_o L}{2}$$

$$\gamma = \frac{Q}{S}$$

$$\gamma_1 = \frac{q_o L^3}{24 B_S} = \frac{q_o L}{2S}$$



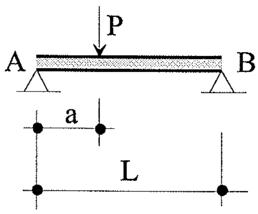
E2.1.2 Simply supported panel with a line load parallel to the supports

At support A

$$Q = \frac{P(L-e)}{L}$$

$$\gamma = \frac{Q}{S}$$

$$\gamma_1 = \frac{PL^2}{6B_S}\left[2\varepsilon - 3\varepsilon^2 + \varepsilon^3\right] + \frac{P}{S}(1-\varepsilon)$$



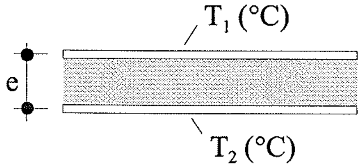
$$\varepsilon = \frac{a}{L}$$

$$a < \frac{L}{2}$$

E2.1.3 Simply supported panel with a temperature difference between the faces

$$\gamma = 0$$

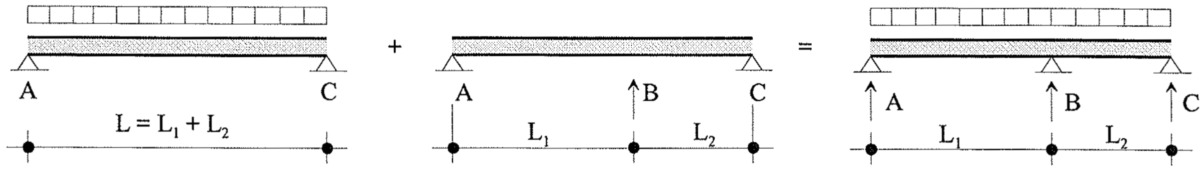
$$\gamma_1 = \frac{\left[\alpha_{F2} T_2 - \alpha_{F1} T_1\right] L}{2e}$$



where α_{F1} and α_{F2} are the coefficients of thermal expansion of faces 1 and 2, respectively.

E2.1.4 Continuous panels

These are solved by the superposition of the simply supported system and the statically indeterminate forces as shown below.



E2.2 Sandwich panels with profiled faces

E2.2.1 Simply supported panel with uniformly distributed load q_o

$$\gamma = \frac{q_o L^3 \beta}{B_F}\left[\frac{1}{2} - \frac{1}{\lambda}\tan\frac{\lambda}{2}\right]$$

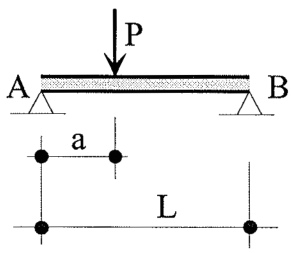
$$\gamma_1 = \frac{q_o L^3}{B_F}\left[\frac{1}{24} + \frac{1}{2\alpha\lambda^2} - \frac{1}{\alpha\lambda^2}\tanh\frac{\lambda}{2}\right]$$

E2.2.2 Simply supported panel with a line load parallel to supports

At support A

$$\gamma_F = \frac{PL^2\beta}{B_F}\left[1 - \varepsilon - \frac{\sinh\lambda(1-\varepsilon)}{\sinh\lambda}\right]$$

$$\gamma_1 = \frac{PL^2}{B_F}\left[\frac{1}{6}\left[2\varepsilon - 3\varepsilon^2 + \varepsilon^3\right] + \frac{1}{\alpha\lambda^2}\left[(1-\varepsilon) - \frac{\sinh\lambda(1-\varepsilon)}{\sinh\lambda}\right]\right]$$



$$\varepsilon = \frac{a}{L}$$

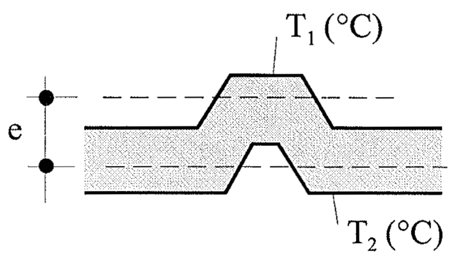
$$a < \frac{L}{2}$$

E2.2.3 Simply supported panel with a temperature difference between the faces

$$\theta = \frac{\alpha_{F2}\,T_2 - \alpha_{F1}\,T_1}{e}$$

$$\gamma = -\frac{\theta L}{\lambda}\tan\frac{\lambda}{2}$$

$$\gamma_1 = \frac{\theta L}{1+\alpha}\left[\frac{1}{2} - \frac{1}{\lambda}\tanh\frac{\lambda}{2}\right]$$



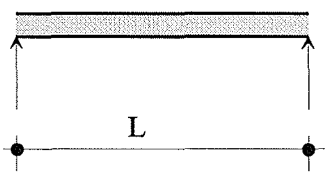
E2.2.4 Continuous panels

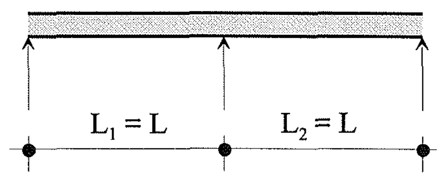
The same procedure shall be followed as described for the panels with flat faces (see E2.1.4)

E2.3 Approximate calculation for single or multi-span panels with flat or profiled faces

$$u[q_o] = \frac{q_o L^3 e}{24 B_F}$$

$$u[\Delta T] = \frac{\theta L e}{2}$$





The limits of application of the above formulae are given by

$$L_1 = L_2 = L_3 = \cdots = L$$

$$L\sqrt{\frac{BeG_C}{B_S}} > 2.5$$

Reference for Appendix E

Toma A.W. 1989, Proposal for chapter 4 of the European Recommendations for sandwich panels: Design of fastenings for sandwich panels. TNO Report BI-84-012. Delft 1989.