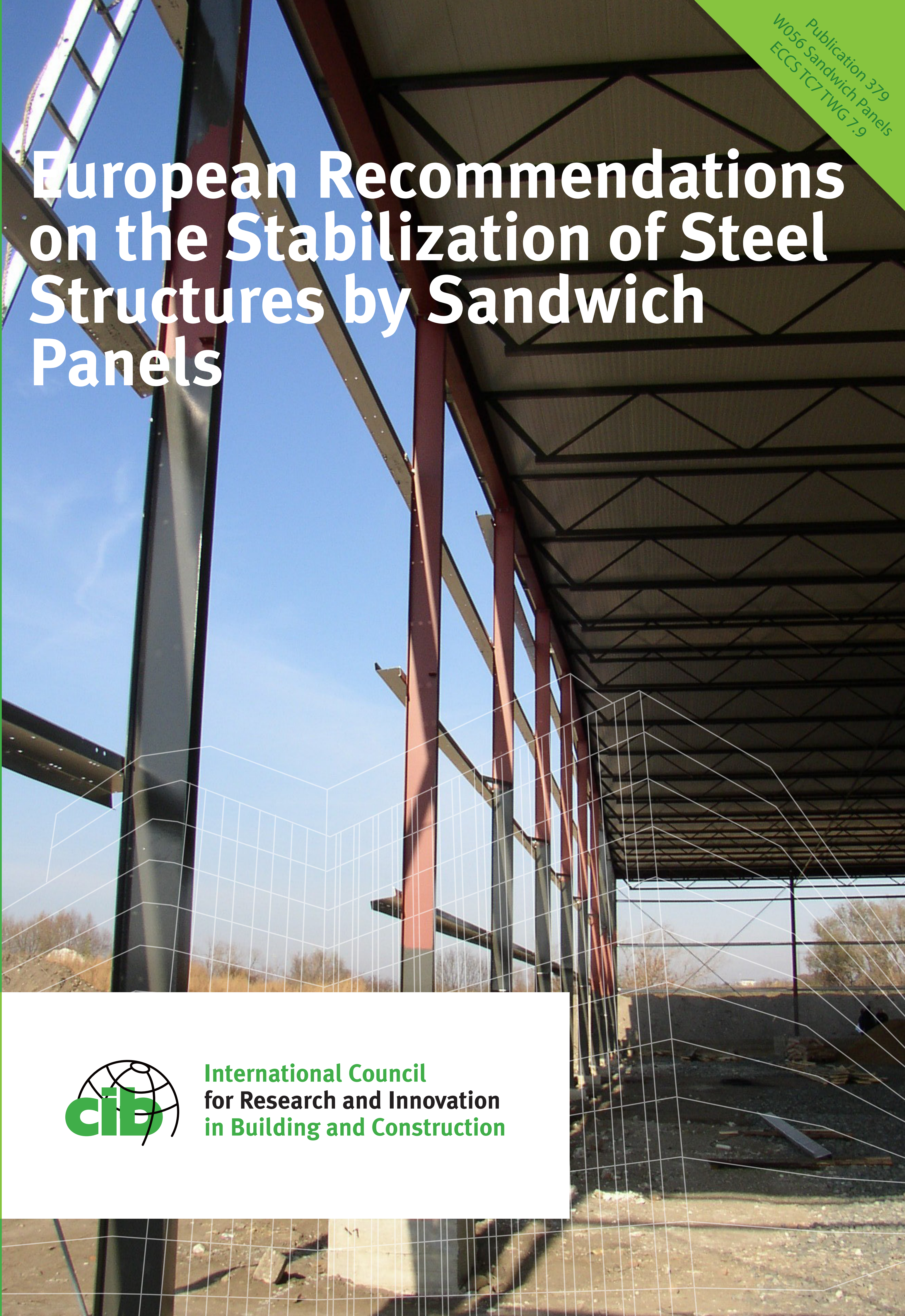
Publication 379
W056 Sandwich Panels
ECCS TC7 TWG 7.9

European Recommendations on the Stabilization of Steel Structures by Sandwich Panels



International Council
for Research and Innovation
in Building and Construction

EUROPEAN RECOMMENDATIONS
ON THE STABILIZATION OF STEEL STRUCTURES
BY SANDWICH PANELS

CIB PUBLICATION 379
ISBN 978-90-6363-081-2

ECCS / CIB JOINT COMMITTEE

ECCS TC7 – Technical Working Group TWG 7.9
Sandwich Panels and Related Structures

CIB Working Commission W056
Sandwich Panels

2nd Edition, 2014

European Recommendations on the Stabilization of Steel Structures by Sandwich Panels

Published by:
CIB – International Council for Research and Innovation in Building and Construction
secretariat@cibworld.nl
www.cibworld.nl
ECCS – European Convention for Constructional Steelwork
publications@steelconstruct.com
www.eccspublications.eu

Cover photo: Andrej Belica

PREFACE

This document gives information about the use of self-supporting sandwich panels as stabilizing elements for single steel members such as beams or columns. The document extends the application range of sandwich panels to construction class II according to EN 1993-1-3.

Sandwich panels provide stiffness against displacements in the plane of the panels and against rotation about the transverse axis of the panels. Thus, the sandwich panels may support the steel members against flexural, torsional and lateral buckling. The effect of stabilization mainly depends on the properties, location and number of fastenings installed between the individual sandwich panels and between the sandwich panels and the supporting structures. This document introduces the evaluation of rotational stiffness and shear stiffness provided by individual sandwich panels that are installed in a wall or roof of a building. In these Recommendations, the use of information is limited in order to stabilize only single structural members.

The European standard EN 14509 covers the manufacture and design of industrially made self-supporting structural sandwich panels. The use of sandwich panels as stabilizing elements such as introduced in these Recommendations extends the application area outside the scope of EN 14509. Therefore, the extended application area introduced in these Recommendations shall be regulated nationally. The sandwich panels used as stabilizing elements have to fulfil the requirements shown by the CE mark of the product.

A brief review on earlier guidelines and reports concerning the use of profiled sheeting and sandwich panels as stabilizing elements provides useful background information.

According to the knowledge of today, the sandwich panels shall be used as stiffening elements only in cases, in which the load predominantly consists of quasi-static loads, such as self-weight, snow and wind load. Repeated loads, e.g. loads caused by earthquake, are not covered by the Recommendations. Research work and further practical experience may result in new products and new ways to fasten the panels to the supporting structure in order to make stiffening technically and economically even more effective.

This document has been prepared by the European Joint Committee on Sandwich Constructions consisting of ECCS Technical Working Group TWG 7.9 and CIB Working Commission W056. The document was approved by the Technical Committee TC7. The final draft was circulated for comments to ECCS TC7.

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

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Compared to the first edition, in the present second edition, editorial corrections were made.

CONTENTS

1. INTRODUCTION

1.1 Aims of the document

Sandwich panels increase the resistance of the supporting structure (beams, purlins, columns) against lateral torsional buckling and buckling by restraining the lateral displacements and rotations.

The torsional restraint is governed by the stiffness of the connection of the sandwich panel to the supporting structure. The stiffness significantly depends on the load transferred by the sandwich panel to the supporting structure.
The high in-plane shear stiffness of sandwich panels shall be used for stabilizing the lateral displacement of the supporting structure and thus, preventing lateral torsional buckling and buckling of the supporting structure. This type of stabilization requires the exact knowledge about the in-plane shear stiffness. Special considerations are necessary for the design of the fastenings because the flexibility of the connection between the sandwich panel and the supporting structure reduces the effective shear stiffness significantly.

Transfer of horizontal loads, e.g. wind loads or loads resulting from earthquakes, is not included in the scope of the Recommendations.

These European Recommendations are founded strongly on the work done during the research project EASIE. The EASIE project has received financial support from the European Community’s Seventh Framework Programme FP7/NMP2-SE-2008 under grant agreement No 213302. Whereas these European Recommendations are limited to information and formulae necessary for the design, the EASIE reports listed in the Bibliography include some more background information and more bibliographical data.

1.2 Application range

The European standard EN 14509 [24] covers the manufacture and design of industrially made self-supporting structural sandwich panels. The use of sandwich panels as stabilizing elements extends the application area outside the scope of EN 14509. Therefore, the extended application area introduced in these Recommendations shall be regulated nationally. The sandwich panels used as stabilizing elements shall fulfil the requirements of EN 14509 shown by the CE-mark of the product.

This document covers sandwich panels with metallic faces and a core made of a PU-[1] or EPS-foam or made of mineral wool. If the shear stiffness of the panels is used for stabilization, fastening shall be done by direct fastening[2].

This document neither provides recommendations about the selection of the materials of the sandwich panels, nor of the fasteners. The compatibility of the materials of the supporting

[1] The designation PU-foam covers both polyurethane foam (PUR-foam) and polyisocyanurate foam (PIR-foam)

[2] Screw fastenings based on long screws drilled through the sandwich panel to the substructure are also termed “direct fastenings” in order to make a distinction with concealed fastenings, which are normally also based on screws but may also include other elements. An alternative term for concealed fastening is “indirect fastening”.

structure, fasteners and sandwich panels and the risk of corrosion shall be checked case by case.

1.3 Design of beams and columns with restraint

Adjacent sandwich panels may either provide full restraint or partial restraint against buckling failure of the investigated stability failure mode. If full restraint of the compressed flange is provided, the beam is not prone to lateral torsional buckling failure and no reduction of strength occurs: The full (plastic) capacity can be used in the design, provided that no other stability failure mode becomes crucial.

The shear stiffness S [kN] represents the shear force F against a shear angle γ of 1 rad.

$$S = \frac{F}{\gamma} \tag{1}$$

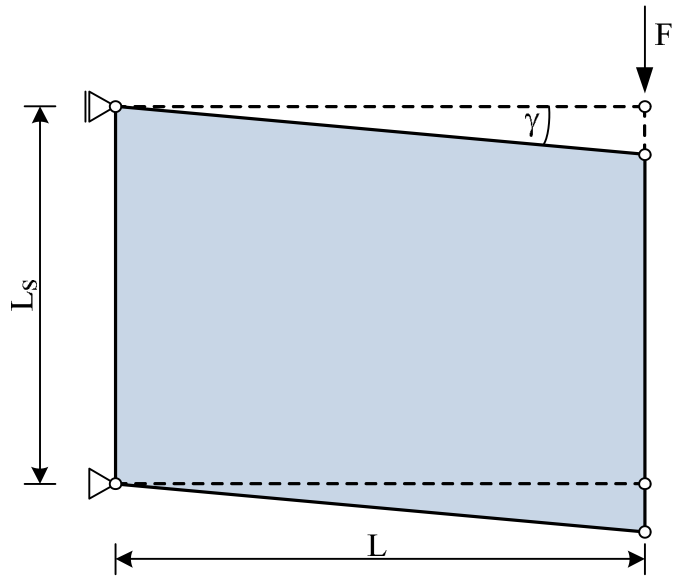


Fig. 1.1: Definition of shear stiffness S

For the stabilization of one beam the shear stiffness S_i according to formula (2) applies.

$$S_i = \frac{S}{m} \tag{2}$$

where

m number of beams to be stabilized

EN 1993-1-1 [22], Annex BB gives indications for both shear stiffness S_i and rotational spring stiffness $C_{\vartheta,k}$ required for full restraint.

$$S_i \geq \left(EI_w \cdot \frac{\pi^2}{L^2} + GI_T + EI_z \cdot \frac{\pi^2}{L^2} \cdot \frac{h^2}{4} \right) \cdot \frac{70}{h^2} \tag{3}$$

where

S_i shear stiffness available for the stabilization of one beam (formula (2))
EI_w warping stiffness
GI_T torsional stiffness
EI_z bending stiffness
L length of the beam to be stabilized
h height of cross section

and

$$C_{\vartheta,k} \geq \frac{M_{pl,k}^2}{EI_z} \cdot K_\vartheta \cdot K_\upsilon \tag{4}$$

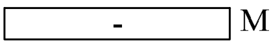
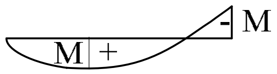
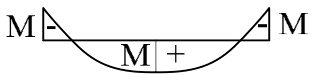
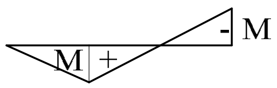
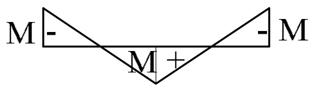
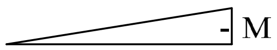
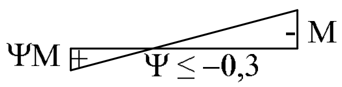
where

K_υ = 0.35 for elastic design
= 1.00 for plastic design
K_ϑ parameter depending on the moment distribution according to Table 1.1
$M_{pl,k}$ characteristic plastic bending resistance of the beam to be stabilized

If the expression (4) applies, the beam can be regarded as fully restrained against rotation. If the expression (3) is fulfilled, the beam can be regarded as restraint against lateral displacement in the plane of the sandwich panels.

If equations (3) or (4) are not fulfilled, only partial restraint is provided by the sandwich panels. If partial restraint is provided, buckling failure can be crucial, but the restraint provided by the sandwich panels will increase the elastic critical buckling moment M_{cr} and the elastic buckling load N_{cr}. In this case, the design procedure follows the usual procedures of EN 1993-1-1 [22] or EN 1993-1-3 [23], but taking into account the higher elastic critical buckling moment and the higher elastic buckling load when calculating the slenderness.

Table 1.1: Values K_9 for buckling curves b, c and d according to [43] and [17], with corrections

moment distribution	without translational restraint			with sufficient translational restraint acc. to eqn. (3)		
	buckling curve			buckling curve		
	b	c	d	b	c	d
	13.2	17.5	14.2	6.7	8.9	11.5
	6.8	10.0	10.9	0	0	0
	4.8	7.3	10.9	0.04	0.11	0.40
	4.2	6.4	9.7	0.22	0.40	0.66
	2.8	4.4	7.1	0	0	0
	1.7	2.8	4.8	0.08	0.15	0.44
	1.0	1.6	2.9	0.24	0.54	1.00
	0.89	1.4	2.6	0.33	0.71	1.60
	0.47	0.75	1.4	0.14	0.33	0.90
same as previous case but $\Psi = +0{,}5$	2.6	4.1	6.7	1.6	2.5	4.1

1.4 Review of previous guidelines and publications

The stabilizing effect of profiled metal sheeting and sandwich panels has been studied in several research projects in Europe. The stabilizing effect has been utilized to support whole buildings or cabins against horizontal and vertical loads or to support single structural members against different buckling failure modes. The brief review covers essential guidelines and reports.

Bryan, E.R.: The stressed skin design of steel buildings [3]

The book gives tools for the design of rectangular and pitched roof framed buildings to horizontal and vertical loads. The frames are covered with cladding made of trapezoidal sheets. The derivation of shear flexibility of the sheeting consisting of distortion and shear deformation of the sheet, and of the deformations in the points of connections is presented. A number of calculated examples and experimental observations show the flow of design calculations to be made when using the elastic or plastic analysis of frames and sheeting. Finally, technical limitations and practical considerations are discussed concerning design and use of stressed skin method.

Stressed skin construction – principles and practice [4]

The document is a practical introduction on the design and use of stressed skin systems made of trapezoidal steel sheeting. The document includes practical information about the role, installation and fixing of stressed skin in flat roofed and pitched roofed buildings. The static system of buildings is based on single columns and girders or on frames. The document has been good medium in marketing in use of light-weight steel structures and use of cladding to transfer horizontal and vertical loads.

Davies, J.M. & Bryan, E.R.: Manual of stressed skin diaphragm design [5]

The book is an extensive design manual for the stressed skin method and a source of experimental and theoretical background information. The book is based on the earlier work of Eric Bryan with updated and extended information. The main subject area of the book is the stiffening of flat roofed and pitched roofed buildings against horizontal and vertical loads with shear panels instead of bracing. The first part of the book introduces typical building systems, design methods and expressions used in calculations. It shows the flow of design in a number of examples and illustrative drawings and pictures from tests and analyses.

The second part gives experimental and analytical background information. It shows the derivation of design expressions. Additional cases such as folded plate structures and shells are studied. In addition, the role and effect of openings of the shear panel are investigated. Chapter 17 directly concerns the subject area of the current European Recommendations when looking for stabilizing the rafters against lateral buckling modes. The book includes an extensive list of relevant publications from the time of the book.

Baehre, R. & Ladwein, Th.: Tragfähigkeit und Verformungsverhalten von Scheiben aus Sandwichelementen und PUR-Hartschaumkern (Diaphragm Action of Sandwich Panels) [1], [2], [38]

In the research project sandwich panels for use in shear diaphragms were investigated. In several test series with roof and wall panels, the dimensions of the shear diaphragms were varied within the practical range. The roof panels were circumferentially connected with the supporting structure and with each other in the longitudinal joints. As usual in building practice, the connections of the longitudinal joints were only realized in the area of the overlap of the external faces. Wall panels were only connected on the supported transverse edges with the supporting structure as unidirectional spanning panels corresponding to the usual application. In order to compensate the resulting reduced in-plane shear stiffness in some tests, the longitudinal edges were stiffened. A calculation model for determining the shear stiffness S, and the load-bearing capacity of the diaphragm was derived. The calculation procedure for panels without connections on longitudinal edges and joints is presented in section 3.2.1 of these Recommendations.

European Recommendations for application of metal sheeting acting as diaphragm [16]

The first edition of the European Recommendations for the stressed skin design of steel structures was published in 1977. The updated Recommendations were published in 1995 considering available new information and the role and content of Eurocodes. The document concerns the behaviour and resistance of shear panels made of trapezoidal sheets. It introduces the stressed skin method to stabilize low-rise flat roofed buildings against horizontal loads with additional application to support pitched roofed buildings for vertical loads. Annex B introduces the principles and methods to stabilize beams and columns to buckling modes with shear panels. Thus, Annex B directly concerns the subject area of the current Recommendations. The components of flexibility and modes of failure are different in trapezoidal sheets compared to those in sandwich panels. The European Recommendations are a useful document for design and shall be an important background document of the current Eurocodes as well.

Hedman-Pétursson, E.: Column buckling with restraint from sandwich wall elements [28]

In this report, the supporting of slender steel columns against buckling failure is studied. The additional support is provided using wall panels. In the investigations wall panels, which are installed in the horizontal direction and span from column to column, were considered. Experimental and numerical investigations were made in order to derive formulae for the fixing forces, for which the fastenings of sandwich panels shall be designed. These formulae are given for panels providing full restraint and panels providing partial restraint. To achieve full restraint, additional rivets were used, which connected both external faces of adjacent panels to each other above the line of support.

Lindner, J., Gregull, T.: Drehbettungswerte für Dacheindeckungen mit untergelegter Wärmedämmung (Torsional Restraint Coefficients of Roofing Skin with Thermal Insulation) [39], [41]

Lindner and Gregull were probably the first who investigated the torsional restraint provided by sandwich panels. They developed the basic approach to split up the torsional restraint into three parts - bending stiffness of the attached panel, stiffness of the connection and distortional stiffness of the beam to be stabilized. Numbers of tests and consequently application ranges were rather limited, but have been used in Germany since the late eighties.

Dürr, M.: Die Stabilisierung biegedrillknickgefährdeter Träger durch Sandwichelemente und Trapezbleche (Stabilization of beams prone to lateral torsional buckling by sandwich panels and trapezoidal sheeting) [9]

The torsional restraint provided by sandwich panels was investigated by experiments and numerical calculations. Formulae for calculating the moment-rotation relation were derived. These formulae were included into the German standard DIN 18800.

Georgescu, M. and Ungureanu V., Department of Steel Structures and Structural Mechanics, Politehnica University of Timisoara. [26], [27]

Additional research on the stabilization of thin-walled sections by rotational and lateral restraint provided by sandwich panels is planned to be performed at the Department of Steel Structures and Structural Mechanics of the Politehnica University of Timisoara. Up to now, one preliminary test on torsional restraint has been performed, whereas the influence of transverse forces was neglected. Therefore, the papers already published focus on the general design of purlins according to chapter 10.1 of EN 1993-1-3 [23], and how to implement the results of scheduled tests into these procedures.

European research project EASIE (Ensuring Advancement in Sandwich Construction through Innovation and Exploitation), 2008-2011.

The European project EASIE (European Community's Seventh Framework Programme FP7/ NMP2-SE-2008, grant agreement No 213302) treated different topics, all of them dealing with sandwich panels. Within the framework of the project, amongst other things, the stabilizing effects of sandwich panels were investigated. These investigations are the basic source for these European Recommendations. Further information on the EASIE project can be found on www.easie.eu.

1.5 Symbols and definitions

In the following list, F means a unit of a load and L a unit of a length.

α	amplification factor	[-]
γ	shear angle	[rad]
Δv	relative displacement of a fastening	[L]
ΔS_i	additional stiffness available for the stabilization of one beam resulting from a single rigid support	[F]
ϑ	rotation	[rad]
ϑ_0	initial rotation	[rad]
$\varphi_{\vartheta,t}$	parameter (duration of load)	[-]
b	width of flange of a beam	[L]
b_K	distance between governing line of fixing and contact line	[L]
B	width of a sandwich panel	[L]
c_k	distance between two fastenings of a pair	[L]
c_1	parameter for calculation of $C_{\vartheta 1}$	[-]
c_2	parameter for calculation of $C_{\vartheta 2}$	[L]
c_3	parameter for calculation of $C_{\vartheta 1}$	[L^2]
C_{sup}	stiffness of clamping in the supporting structure	[FL/rad]
C_ϑ	rotational spring stiffness	[FL/L]
$C_{\vartheta A}$	rotational spring stiffness, stiffness of the connection	[FL/L]
$C_{\vartheta B}$	rotational spring stiffness, distortional stiffness of the beam	[FL/L]

$C_{\vartheta C}$	rotational spring stiffness, bending stiffness of attached panel	[FL/L]
$C_{\vartheta 1}$	rotational spring stiffness	[FL/L]
$C_{\vartheta 2}$	rotational spring stiffness	[FL/L]
d	nominal diameter of a fastener	[L]
d_1	minor diameter of the threaded part of the fastener	[L]
d_C	depth of core	[L]
d_s	diameter of unthreaded shank	[L]
d_W	diameter of washer	[L]
D	thickness of a panel at point of fastening	[L]
e	additional imperfection	[L]
e_0	initial imperfection	[L]
e_{tot}	e_0+e	[L]
E_C	modulus of elasticity of the core	[F/L^2]
E_{Cc}	compressive modulus of the core	[F/L^2]
E_{Ct}	tensile modulus of the core	[F/L^2]
EI	bending stiffness	[F/L^2]
EI_w	warping stiffness	[FL^4]
EI_z	bending stiffness	[FL^2]
f_{Cc}	compression strength of the core material	[F/L^2]
f_{Ct}	tensile strength of the core material	[F/L^2]
f_u	tensile strength of the face sheet	[F/L^2]
F	force	[F]
F_i	normal compression force of stabilized beam	[F]
GI_t	torsional stiffness	[FL^2]
γ_F	load factor	[-]
γ_M	material safety factor	[-]
h	section height	[L]
k_c	correction factor	[-]
k_{F1}	stiffness of external face sheet	[F/L]
k_{F2}	stiffness of internal face sheet	[F/L]
k_v	shear stiffness of a fastening	[F/L]
$k_{v,1}$	shear stiffness of a fastening at a rigid support	[F/L]
$\overline{k}_v$	stiffness used for calculation of ΔS_i	[F/L]
K_ϑ, K_v	factors for the calculation of a minimum rotational spring stiffness (see EN 1993-1-1)	[-]
k_1	factor considering the reduction in the wrinkling stress caused by higher temperatures (see EN 14509)	[-]
L	length of the beam to be stabilized	[L]
L_e	overhang of sandwich panel at end support	[L]
L_S	length of stabilizing panel	[L]
m	number of beams to be stabilized	[-]
m_f	portion of stabilization moment leading to forces in fasteners	[FL/L]
$m_i(x)$	restraining moment	[FL/L]

m_k	contact moment	[FL/L]
$m_{\vartheta A}$	stabilization moment	[FL/L]
M	bending moment	[FL]
M_{cr}	elastic critical moment for lateral torsional buckling	[FL]
M^E	external moment	[FL]
M^I	internal moment	[FL]
M_{pl}	plastic bending resistance	[FL]
M_S	restraining moment	[FL]
n	number of sandwich panels	[-]
n_f	number of fasteners	[L^{-1}]
n_f	number of fasteners per panel and support	[-]
n_k	number of pairs of fasteners per panel and support	[-]
N	normal force	[F]
N_{cr}	elastic critical buckling load	[F]
q	downward load to be transferred from the panel to the beam	[F/L]
$q_i(x)$	restraining load	[F/L]
S	shear stiffness	[F]
S_i	shear stiffness available for stabilizing one beam	[F]
σ_w	wrinkling stress	[F/L^2]
t[1]	design sheet thickness of faces $t_{F1,}$ t_{F2} (aluminium) $t = t_{nom} - 0{,}5 \cdot t_{tol}$ for normal tolerances $t = t_{nom}$ for special tolerances	[L]
t_{cor}[1]	design core sheet thickness of faces $t_{F1,}$ t_{F2} (steel) $t_{cor} = t_{nom} - t_{zinc} - 0{,}5 \cdot t_{tol}$ for normal tolerance according to EN 10143 $t_{cor} = t_{nom} - t_{zinc}$ for special tolerance according to EN 10143	[L]
$t_{cor,sup}$	core sheet thickness of the supporting structure	[L]
t_{nom}	nominal thickness of the sheet	[L]
t_{tol}	normal or special tolerance	[L]
t_{zinc}	total thickness of the zinc layers (or similar protective coating); t_{zink} = 0,04 mm for Z 275	[L]
V	shear force of a fastening	[F]
V_S	shear force in the fastening resulting from stabilization	[F]
V_S^{Δ}	shear force of fastening resulting from restraining translations	[F]
V_S^{M}	shear force of fastening resulting from restraining moments	[F]

[1] The definitions of the design sheet thickness and of the design core sheet thickness correspond to the definitions given in FprEN 14509 [25]. The corresponding notation in EN 14509 is t_d or t.

V_S^Q	shear force of fastening resulting from moment equilibrium	[F]
Index F1	external face sheet	
Index F2	internal face sheet	
Index t	time	
Index θ	temperature	
Index Rk	characteristic value of the load bearing capacity	
Index Rd	design value of load bearing capacity	

2. TORSIONAL RESTRAINT

2.1 Introduction

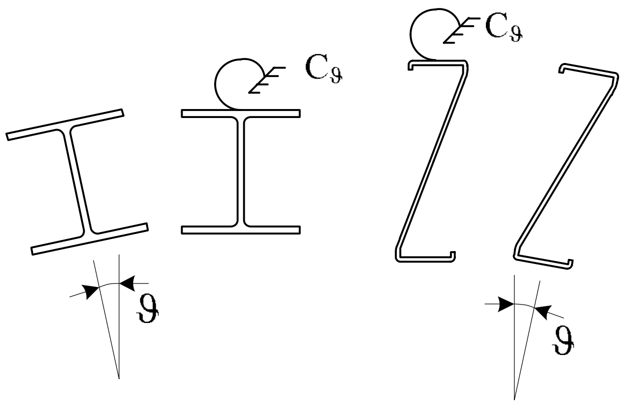
The torsional restraint is governed by the stiffness of the connection of the sandwich panel to the supporting structure. Recent research carried out showed that this stiffness significantly depends on the load transferred by the sandwich panel.

A design concept for the quantification and calculation of the stabilizing effects on beams under predominantly static loading by sandwich panels was developed within the framework of the EASIE project. Another concept is given in the German design code DIN 18800-2 [13] for steel structures and in the German national Annex to EN 1993-1-3 [14].

Formulae for calculating the parameters of this moment-rotation-relation are given for sandwich panels with three different core materials and connections through the upper or lower crimp.

2.2 General background

The torsional restraint given by sandwich panels can be calculated using the mechanical model based on a torsion spring with the spring stiffness $C_{\vartheta,k}$.



$$C_{\vartheta,k} = \frac{1}{1/C_{\vartheta A,k} + 1/C_{\vartheta B,k} + 1/C_{\vartheta C,k}}$$

Fig. 2.1: Stabilization: torsional restraint

This spring stiffness is a combination of the bending stiffness of the attached panel $C_{\vartheta C,k}$, the stiffness of the connection $C_{\vartheta A,k}$ and the distortional stiffness $C_{\vartheta B,k}$ of the beam to be stabilized. The stiffnesses $C_{\vartheta C,k}$ and $C_{\vartheta B,k}$ depend on the geometry of the sandwich panels and the type of beams used, see EN 1993-1-1 [22] and EN 1993-1-3 [23]. The calculation of $C_{\vartheta A,k}$ is explained here. For further information regarding the difference in behavior of the different types of beams (beams symmetric about minor axis, Σ-, Z- U- or C-section) see [9].

Fig. 2.2 shows a typical moment-rotation-relation and its generalized form for the design of the spring stiffness of the connection of a sandwich panel under downward loading.

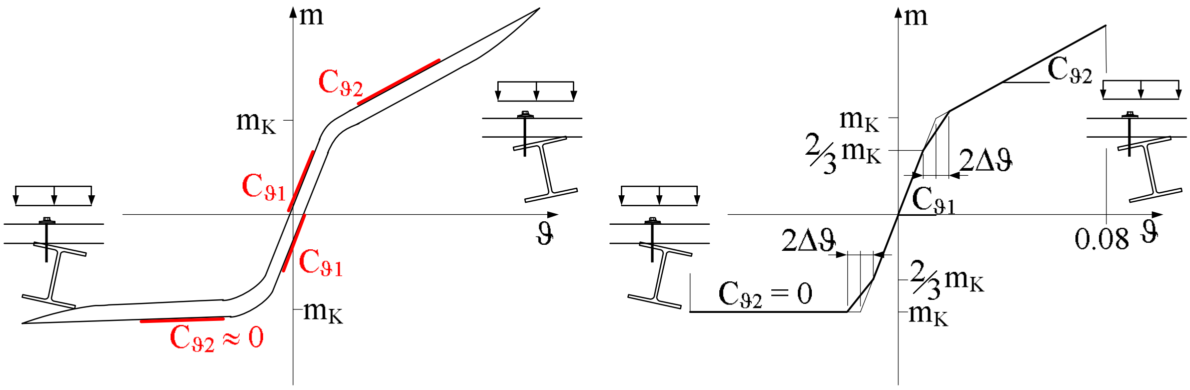


Fig. 2.2: Typical moment-rotation-relation and generalized moment-rotation-relation of the connection between a sandwich panel and a support to be used in design

There are two possibilities of using this simplified tri-linear moment-rotation relation for the design of beams:

1. A secant value (Fig. 2.3) of

$$C_{\vartheta A} = \frac{m_K}{\vartheta(m_K)} = \frac{3}{2} \cdot \frac{C_{\vartheta 1}}{\left(\frac{C_{\vartheta 1}}{C_{\vartheta 1} + C_{\vartheta 2}} + 1\right)} \qquad (5)$$

   can be taken into account. This constant value allows for a simple proof, for example with equation (4). In this case, the stabilization moment has to be limited to the contact moment m_K, see chapter 2.4. If the beams restrained by the sandwich panels are designed according to EN 1993-1-3 [23], section 10.1, the forces in sheet/purlin fasteners and reaction forces according to table 10.4 of EN 1993-1-3 shall be considered. Beyond that, no additional forces from the stabilization occur.

2. A more advanced approach is to use the tri-linear simplified moment-rotation relation shown in Fig. 2.2 and obtained from chapter 2.3, and implement them in a nonlinear computer program for the design of beams. In this case, the forces in the fasteners shall be calculated (see chapter 2.6) and the fastenings have to be designed for these additional forces.

In both cases, depending on the requirements of serviceability, an additional check should be made according to chapter 2.5, limiting the rotation and therefore the danger of leakage under serviceability load conditions.

For uplift loading, no torsional restraint is available. The in-plane shear resistance of the panels can be utilized for stabilizing both for downward and uplift loading (see section 3).

Comments:

Uplift load causes an indentation of the fastener and a gap between the upper flange of the beam and the inner face of the sandwich panel. Therefore for uplift load, no torsional restraint can be assured.

In some applications with cold-formed sections small values of torsional restraint can be assumed in case of uplift loading. These values have to be determined by tests according to chapter 2.7. Particular attention has to be paid to the forces in the fasteners and to creep effects.

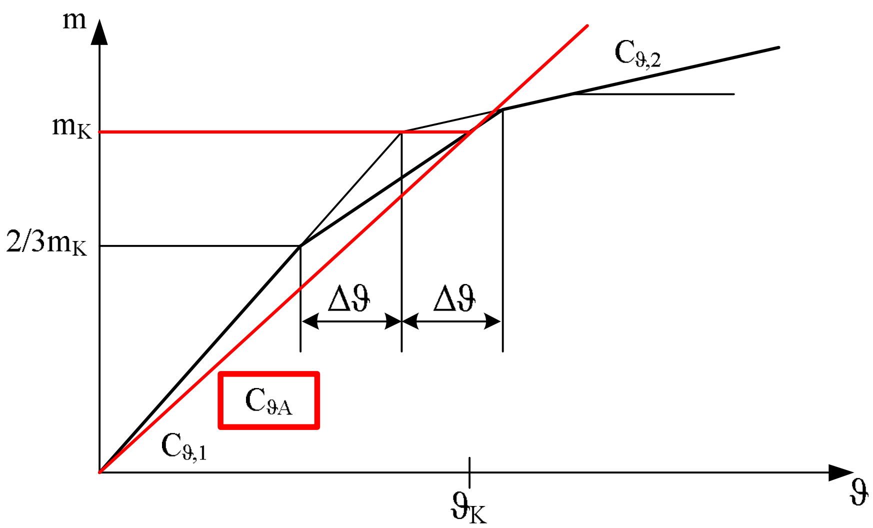


Fig. 2.3: Definition of $C_{\vartheta A}$

2.3 Determination of $C_{\vartheta 1}$ and $C_{\vartheta 2}$

This approach was developed in [33] within the framework of the European Research Project EASIE. The necessary values and parameters are given in the following tables.

Table 2.1: Values $C_{\vartheta 1}$ and $C_{\vartheta 2}$

	hot rolled beams symmetric about minor axis	cold formed ∑-, Z-, U- or C-section
$C_{\vartheta 1}$	$c_1 \cdot E_{C,t,\theta} \cdot b^2$	$c_3 \cdot E_{C,t,\theta}$
$C_{\vartheta 2}$	$c_2 \cdot n_f \cdot E_{C,t,\theta} \cdot b_k^2$ [1)]	0
$E_{C,t,\theta}$	$E_{C,t,\theta} = \frac{E_C}{1+\varphi_{\vartheta,t}} \cdot \sqrt{k_1^3}$, where $k_1 = \frac{\sigma_{w,+80°C}}{\sigma_{w,+20°C}}$ [2)]	
m_K	$q \cdot \frac{b}{2}$	$q \cdot b$
[1)] $C_{\vartheta 2} = 0$ for hidden fixings [2)] for defining the factor k_1 see also EN 14509 [24], A.5.5.5		

Table 2.2: Parameters

c_1, c_2	Parameters according to Table 2.3
$\varphi_{\vartheta,t}$	parameter depending on the duration of loading [1)] $\varphi_{\vartheta,2000}$ =1.29 core materials PU and EPS $\varphi_{\vartheta,100000}$ =1.83 core materials PU and EPS $\varphi_{\vartheta,2000}$ =1.35 core material mineral wool $\varphi_{\vartheta,100000}$ =2.31 core material mineral wool
b [mm]	width of the flange of the beam
b_k [mm]	distance between governing line of fixing and contact line, see Figure 2.4.
n_f [m^{-1}]	number of fasteners per meter length in the governing line of fixing (n_f = 0.0 for hidden fixings and for b_k < 0.5 b)
q [kN/m]	downward load to be transferred from the panel to the beam
$E_C = 0.5\ (E_{Cc} + E_{Ct})$ [N/mm²]	elastic modulus of the core, mean value of compressive modulus E_{Cc} and tensile modulus E_{Ct}
[1)] The creep coefficient $\varphi_{\vartheta,t}$ is a coefficient mainly resulting from compression stress, see [33]. The coefficients were determined experimentally. Due to the large scatter of the tests, the values are comparably high. Better values may be obtained by performing tests according to chapter 2.7.	

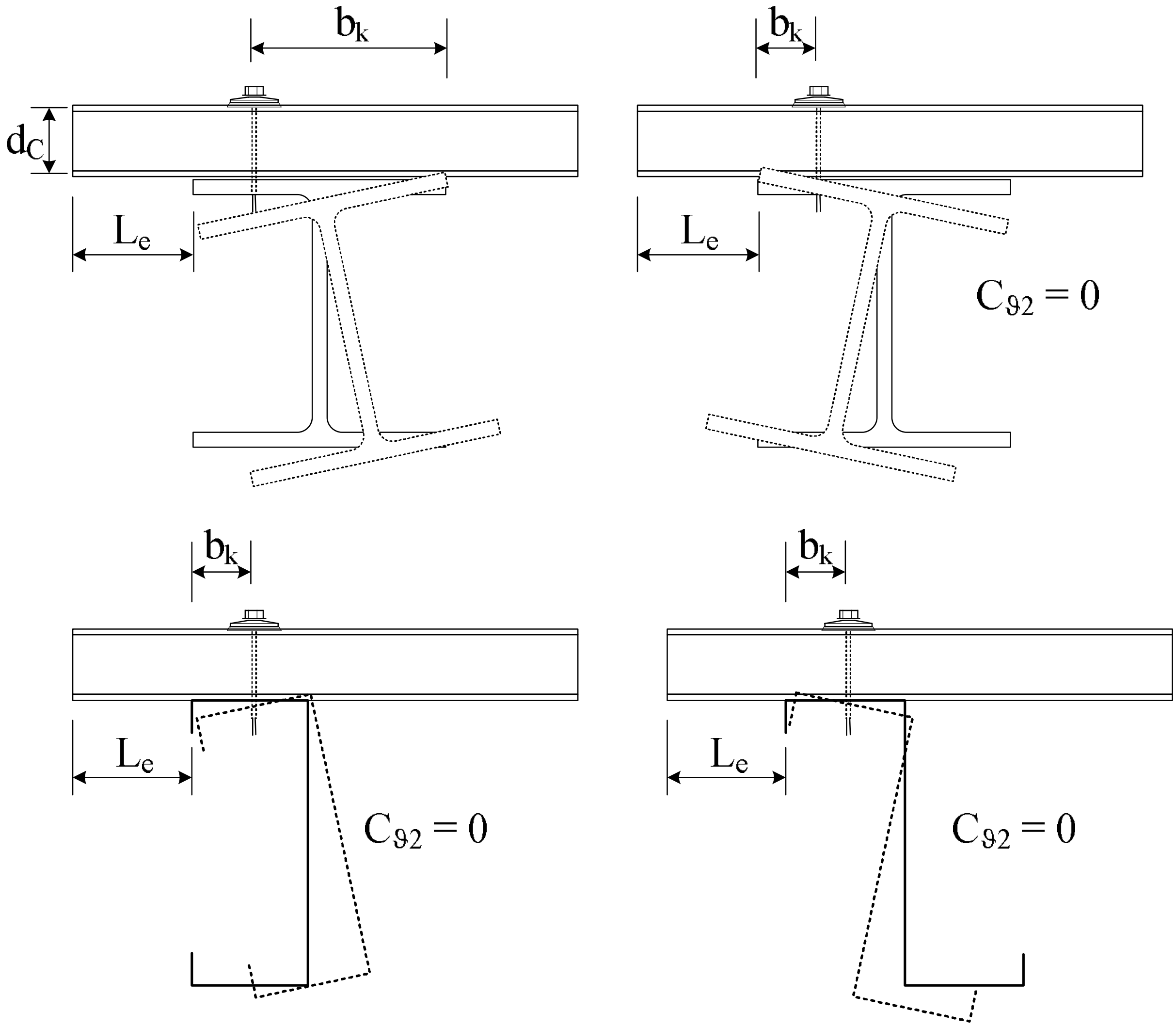


Fig. 2.4: Definition of b_k and L_e

Table 2.3: Parameters c_1, c_2 and c_3

Core material	geometry of outer face (at the head of fasteners)	c_1 [-]	c_2 [m]	c_3 [m^2]
PU/EPS	profiled[1)]	0.180	0.052	$6.48 \cdot 10^{-4}$
	slightly profiled/flat	0.142	0.040	$5.11 \cdot 10^{-4}$
Mineral wool	profiled[1)]	0.089	0.027	$3.20 \cdot 10^{-4}$
	slightly profiled/flat	0.048	0.027	$1.73 \cdot 10^{-4}$
[1)] depth of profiling ≥ 30 mm				

The application range shall be taken into account, see Table 2.4. If higher values of parameters (b, E_C, t_{cor}, t and n_f) occur, the calculation procedure is applicable, but the values should be reduced to the corresponding upper limits of the application range. If lower values occur, tests according to chapter 2.7 shall be performed.

Table 2.4: Application range

60 mm ≤ b ≤ 180 mm	hot rolled beams symmetric about minor axis
60 mm ≤ b ≤ 80 mm	cold formed ∑-, Z-, U- or C-section
2.0 N/mm² ≤ E_C ≤ 8.0 N/mm²	elastic modulus of the core material (mean value)
0.38 mm ≤ t_{cor} ≤ 0.71 mm	core sheet thickness of the face t_{F1}, t_{F2} (steel)
0.50 mm ≤ t ≤ 0.65 mm	sheet thickness of the face t_{F1}, t_{F2} (aluminium)
1 m^{-1} ≤ n_f ≤ 4 m^{-1}	number of fasteners per meter length in the governing line of fixing
q	torsional restraint is only provided with downward loading and only for predominantly static loading
f_{Cc}	compression strength of the core material (characteristic value) f_{Cc} ≥ 0.08 N/mm² core materials PU and EPS f_{Cc} ≥ 0.05 N/mm² core material mineral wool
f_{Ct} ≥ 0.06 N/mm²	tensile strength of the core material (characteristic value)
d_W ≥ 16 mm	diameter of washer
$L_e \geq d_C$	overhang of the panel (see Fig. 2.4), distance to the edge of an opening

Comments:

An alternative approach for the determination of $C_{\vartheta 1}$ and $C_{\vartheta 2}$ was developed by Dürr [9] and is included in the German standard DIN 18800-2 [13], and in the German national Annex to EN 1993-1-3 [14]. Compared to the approach presented in these Recommendations, the approach of [9] often leads to considerably higher values, but has a smaller application range.

2.4 Limitation of stabilization moment

Within the simplified design model introduced here using a secant value $C_{\vartheta A}$, the stabilization moment $m_{\vartheta A}$ shall be limited to the contact moment m_K. According to [40] the stabilization moment should be calculated using

$$m_{\vartheta A} = \frac{1}{C_{\vartheta A} \cdot \dfrac{k_c^4 \cdot E \cdot I_z}{M_{Ed}^2} - 1} \cdot C_{\vartheta A} \cdot \vartheta_0 \qquad (6)$$

where

k_c according to EN 1993-1-1 [22], table 6.6 or [17], table 3

ϑ_0 initial rotation (imperfection), to be taken to 0.06

2.5 Limitation of the rotation of the stabilized beam

According to the investigations of [9], the rotation has to be limited to

$$\vartheta \leq 0.08 \tag{7}$$

The rotation of the stabilized beam can approximately be calculated using

$$\boxed{\vartheta = \frac{m_{\vartheta A,,k}}{C_{\vartheta A}}} \tag{8}$$

and $m_{\vartheta A,k}$ according to equation (6). Since the rotation is limited for the serviceability limit state, the moment $m_{\vartheta A,k}$ is determined for the serviceability load level (characteristic combination).

2.6 Advanced analysis

A more detailed analysis using the actual moment-rotation-relation with $C_{\vartheta 1}$, $C_{\vartheta 2}$ and the transition part is possible, using an iterative approach or even a computer program allowing for the implementation of the non-linear moment-rotation-relation. In this case, the limitation of the stabilization moment to m_K is not necessary, but the load-bearing capacity of the fasteners shall be considered. The part of the stabilization moment resulting in tensile forces in the fasteners should be calculated using

$$m_f = m_{\vartheta A} - m_K \tag{9}$$

The tensile force from stabilization in a fastener is

$$N_{Ed} = \frac{m_f}{n_f \cdot b_K} \tag{10}$$

where
n_f in m^{-1} or the number of fasteners per meter length, respectively
b_K according to Fig. 2.4

If the beams restrained by the sandwich panels are designed according to EN 1993-1-3 [23], section 10.1, the forces in sheet/purlin fasteners and reaction forces according to table 10.4 of EN 1993-1-3 shall be considered. It is recalled that generally no torsional restraint is given for uplift loading.

2.7 Experimental determination of parameters

The test set-up according to EN 1993-1-3 [23], section A.5.3 is not suitable to determine the torsional restraint provided by sandwich panels. With this test set-up, the stiffness may be overestimated and the test results are not on the safe side. Instead of the set-up given in EN 1993-1-3, the set-up proposed by Lindner [39], [41], [42] should be used. This set-up was also used in the investigations of Dürr [9] and in the tests performed within the EASIE project [33].

The test set-up is shown in Fig. 2.6. It consists of a beam supported by a roller bearing. (Two) sandwich panels are fixed to the beam. At the ends of the beam, welded end plates are located, preventing a warping of the beam. Lever arms are attached rectangular to the longitudinal axis of the beam via these end plates, by means of which the beam can be twisted around the centre of rotation D. The lever arms are connected to each other through a transverse truss. Using roller bearings as well as slide bearings on the second support of the sandwich panels it is ensured that neither restraints nor resistances against twisting of the beam occur from the test set-up. During test performance, the sandwich panels are loaded by a constant load p. The transverse truss is loaded by a force F, which causes the rotation of the beam. The displacements of the upper flange and the bottom flange resulting from the rotation of the beam are measured and can be converted in a rotation.

Tests should be performed at least with the minimum and maximum panel thickness. To allow for a broad application range of the test results, stiffness of the faces should reflect the minimum stiffness (thin faces, low profiling). At least three tests should be performed with each of the two thicknesses, each of the tests with at least three load levels of gravity load p and rotation of the purlin in both directions. Evaluation of tests should follow EN 1990 [21]. The test performance and evaluation is also described in [9] and [33]. If tests were performed because the parameters b, E_C, t_{cor}, t or n_f do not meet the lower limit of the application range defined in Table 2.4, results should only be used for the verification of applicability of the design procedures given in chapter 2.3, not directly for the design.

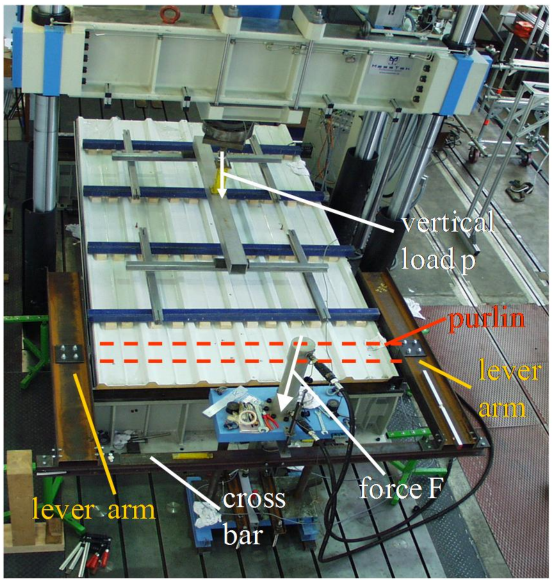


Fig. 2.5: Test set-up, [9]

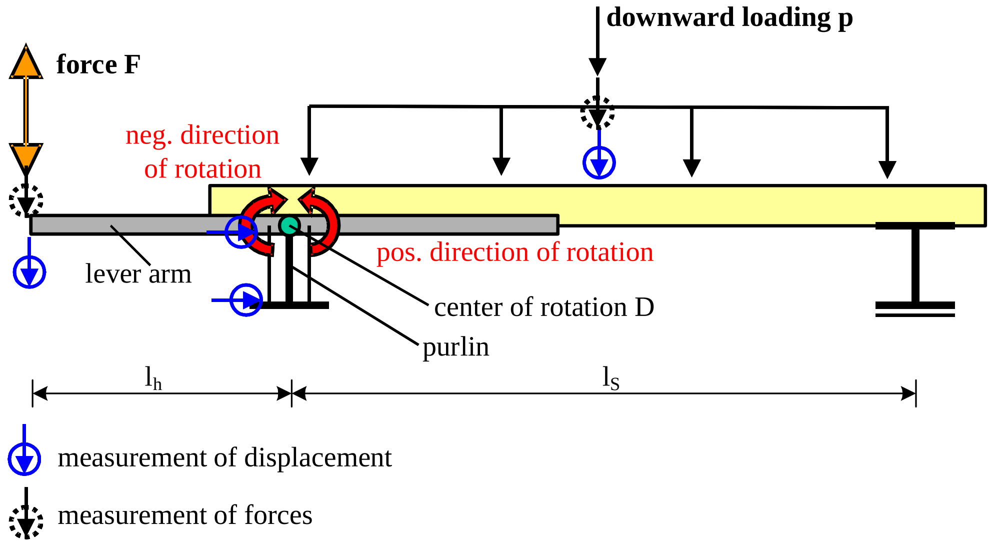
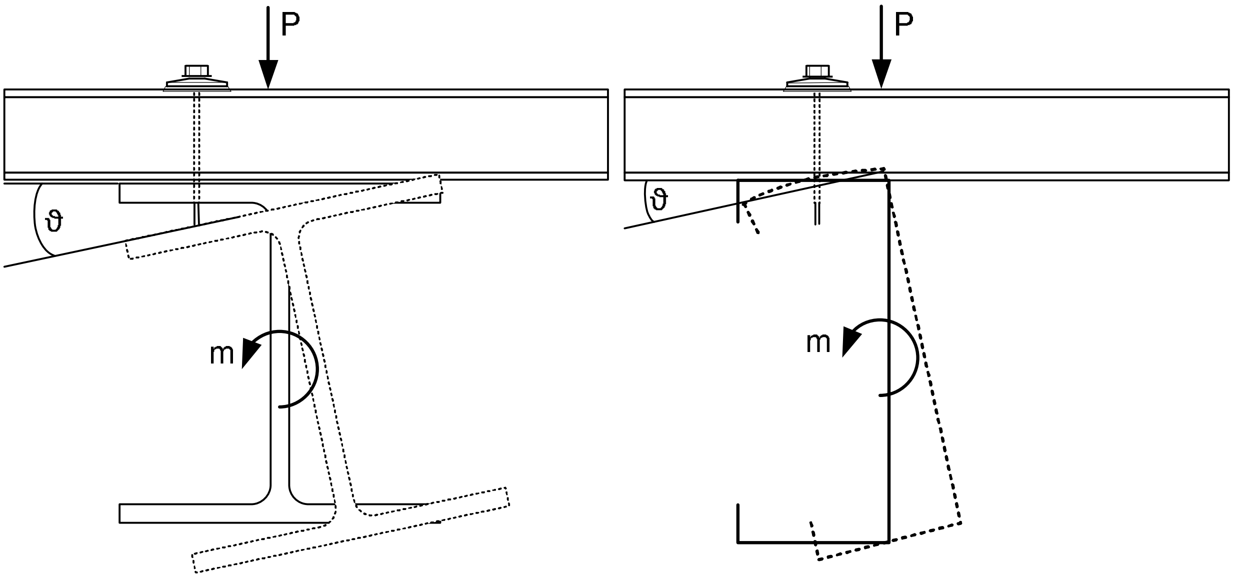


Fig. 2.6: Test set-up, [9]



$$C_{\vartheta} = \frac{m}{\vartheta}$$

remove the influence of distortion of the profile

Fig. 2.7: Principle of tests on torsional restraint

For the determination of the creep coefficient $\varphi_{\vartheta,t}$, creep bending tests with a simply-supported panel subjected to a uniformly distributed dead load should be performed. The load used for the creep test shall correspond to between 30 % and 40 % of the average load for shear failure at ambient temperature. The indentation at the edges of the supports should be measured with dial gauges placed directly over these edges.

Analysis of the test results should be as follows

$$\varphi_{\vartheta,t} = \frac{u_{C,t} - u_{C,0}}{u_{C,0}} \tag{11}$$

where
$u_{C,0}$ initial compressive displacement at the edge corresponding to the time t = 0
$u_{C,t}$ displacement corresponding to the time t

Extrapolation of test results should follow the principles of EN 14509 [24], note in chapter A.6.5.2.

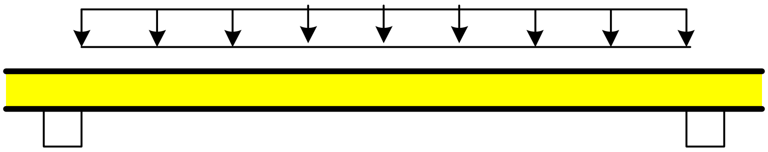


Fig. 2.8: Creep bending tests according to EN 14509

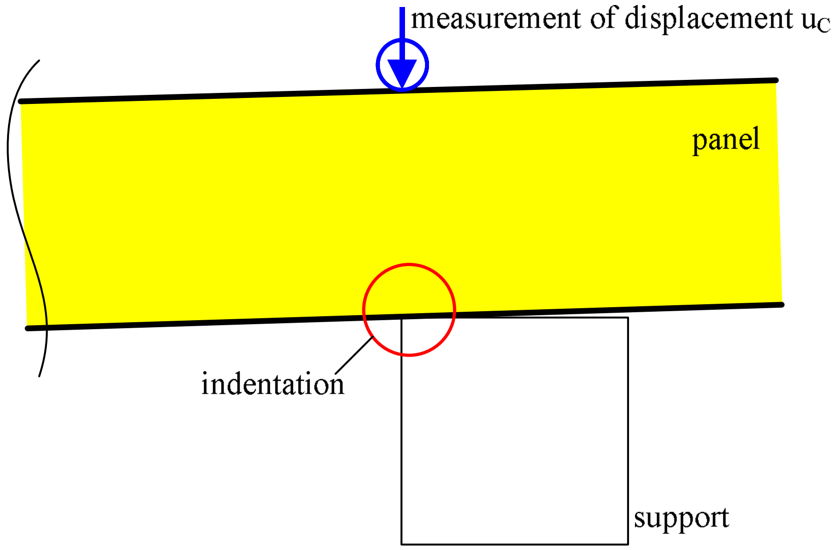


Fig. 2.9: Measurement of indentation at the support during creep bending tests

If the displacement at the edge has been measured, existing test data from creep bending tests according to EN 14509 shall be utilized.

In any case, material properties both of the core and the faces should be determined according to EN 14509 to allow for the evaluation of test results.

3. LATERAL RESTRAINT - IN-PLANE SHEAR RESISTANCE

3.1 Introduction

Sandwich panels have a high stiffness and strength when loaded in the plane of the panel. This can be used to stabilize the supporting structure of the panels (beams, purlins, columns).

The deformation of sandwich panels themselves caused by in-plane shear load may normally be neglected. The flexibility of the fixings usually dominates the shear flexibility. The fixings must be designed for the in-plane shear load. In typical cases, it is not necessary to design the sandwich panels for this additional load, but it is sufficient to design the panels for their primary loading consisting of the distributed snow and wind load and against the forces resulting from the difference of the temperature between the faces. However, this rule resulted from current experiments, which shall not be generalized. The resistance of the individual sandwich panels to in-plane shear load shall be studied in each case. The shear resistance of the individual panels is influenced by imperfections such as incomplete bonding, in addition to material properties and thicknesses.

Sandwich panels shall be used to stabilize steel members only under the following conditions:

- The sandwich panels are treated as a structural component that cannot be removed or modified without proper consideration.
- The project specification, including the calculations and drawings, gains attention to the fact that the sandwich panels are designed to stabilize steel members.
- Direct fastenings with edge distance ≥ 20 mm in direction of span are used. For panels with concealed fastenings, no lateral restraint is available. However, the torsional restraint can be utilized for stabilization of the supporting structure, see chapter 2.
- The properties of the core material are not needed to design the sandwich panels for lateral restraint. Although, it should be ensured that the materials have a “good quality”. It is recommended to use the application range for the core material given in chapter 2 also for panels which are utilized for lateral restraint.

The basis of the approach presented in the following sections was developed in [1] and further elaborated and amended in [34] within the framework of the European Research Project EASIE.

Comments:

In the calculation procedures given in the following chapter 3.2 connections at the longitudinal joints of the sandwich panels are neglected. Considering connections at the longitudinal joints often leads to a considerably higher shear stiffness S. But it is on the safe side to neglect these connections and to calculate the shear stiffness according to the formulae of chapter 3.2.

With the calculation procedure presented in [1] connections at the longitudinal joints of the sandwich panels can also be taken into account.

3.2 Determination of the shear stiffness S

3.2.1 Uni-directionally spanning panels

Sandwich panels are normally connected to the supporting structure at the transverse edges only. They usually do not have connections at the longitudinal edges. This is common practice, especially for wall panels. Each panel acts as an individual element. When loaded by in plane shear forces, each panel rotates around a reference point P, which is located in the centre of the panel (centre of gravity of the fasteners). The panels remain parallel to the longitudinal edges and they are parallel to each other (Fig. 3.1).

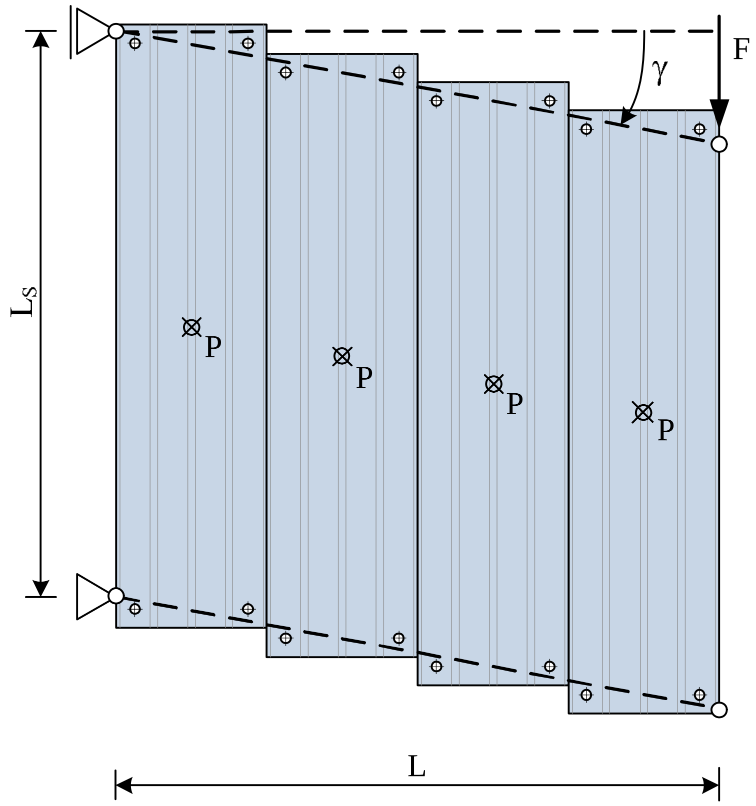


Fig. 3.1: Displacement of shear loaded uni-directionally spanning sandwich panels

The forces and displacements at the fastenings take place in direction of the longitudinal edges. The relative displacement Δv of a fastening can be defined by the angle of the shear γ and the distance to the reference point (Fig. 3.2).

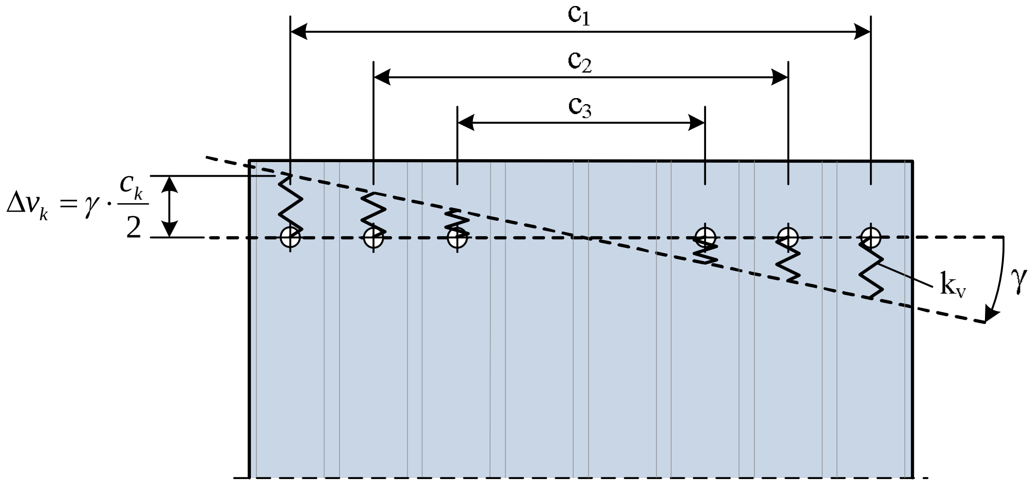


Fig. 3.2: Displacement of fastenings

With the stiffness k_v of the fastening the shear force V of a fastening shall be determined.

$$V_k = k_v \cdot \Delta v_k = k_v \cdot \gamma \cdot \frac{c_k}{2} \tag{12}$$

where
k_v stiffness of the fastenings, see section 3.3
c_k distance between the two fasteners of a pair

The directions of the forces of a pair of fasteners are opposite to each other. For one pair of fasteners, the internal moment shall be written as

$$M_k^I = V_k \cdot c_k = k_v \cdot \gamma \cdot \frac{c_k^2}{2} \tag{13}$$

The internal moment of the system is determined by addition of the moments M_k^I over all pairs of fasteners.

$$M^I = n \cdot m \cdot \frac{k_v \cdot \gamma}{2} \cdot \sum_{k=1}^{n_k} c_k^2 \tag{14}$$

where
n number of sandwich panels
m number of beams to be stabilized
n_k number of pairs of fasteners per panel and support

The internal moment has to counteract the external moment.

$$M^E = F \cdot L = S \cdot \gamma \cdot L \tag{15}$$

The equalization of internal and external moments provides an expression for the shear stiffness S.

$$S = \frac{k_v}{2 \cdot L} \cdot n \cdot m \cdot \sum_{k=1}^{n_k} c_k^2 \tag{16}$$

For stabilization of each beam the shear stiffness

$$S_i = \frac{k_v}{2 \cdot L} \cdot n \cdot \sum_{k=1}^{n_k} c_k^2 \tag{17}$$

is available. Equation (17) can be simplified to

$$\boxed{S_i = \frac{k_v}{2 \cdot B} \cdot \sum_{k=1}^{n_k} c_k^2} \tag{18}$$

where
B width of a sandwich panel

3.2.2 Panels with a single rigid support

If beams and columns to be stabilized are connected with a rigid support by the panels, i.e. the panels are supported along a rigid line for example with a concrete basement (Fig. 3.3) or a rigid ridge purlin, the shear stiffness S_i according to equation (18) can be increased using the following value.

$$\Delta S_i = \frac{n_f \cdot \bar{k}_v}{B} \cdot \left(\frac{L}{\pi}\right)^2 \tag{19}$$

with

$$\bar{k}_v = \frac{1}{\frac{1}{k_v} + \frac{m}{k_{v,1}}} \tag{20}$$

where
m number of beams to be stabilized
k_v stiffness of the fastenings with the beams to be stabilized
$k_{v,1}$ stiffness of the fastenings with the rigid support
n_f number of fasteners per panel and support

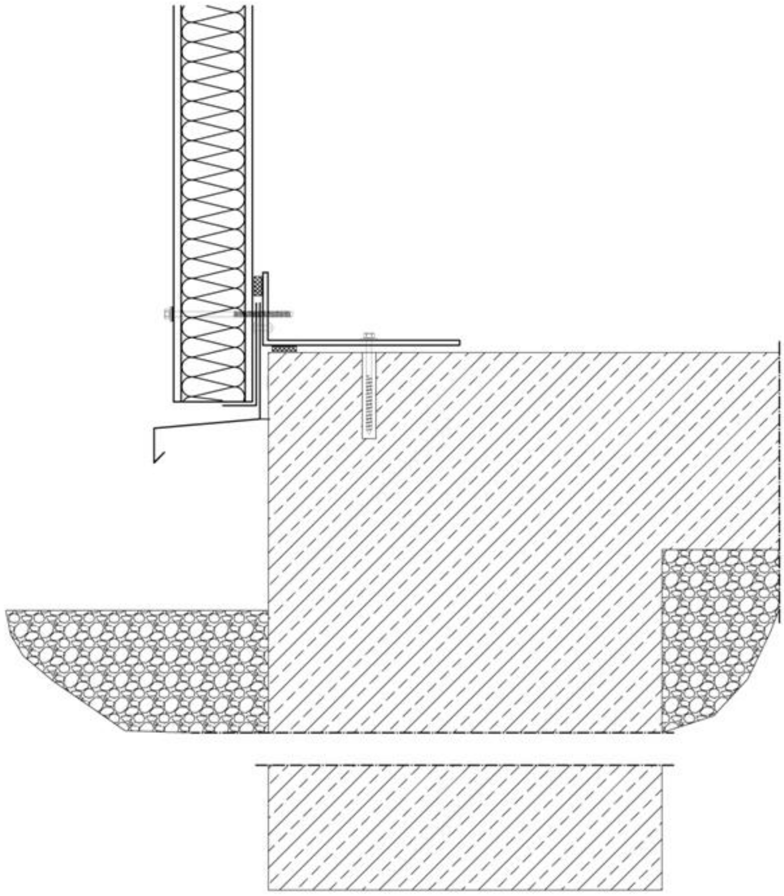


Fig. 3.3: Example for a rigid support of a panel at the basement (figure: IFBS, [31])

3.3 Stiffness of fastenings

3.3.1 Determination of the stiffness by calculation

Following the investigations of [35], the stiffness of a fixing of a sandwich panel to a supporting steel structure is influenced by the following stiffnesses or parameters:

(1) Bending stiffness EI of the fastener
(2) Clamping of the head of the fastener (rotational spring)
(3) Clamping of the fastener in the supporting structure (rotational spring with stiffness C_{sup})
(4) Hole elongation of the internal face sheet (longitudinal spring with stiffness k_{F2}).
(5) Hole elongation of the external face sheet (longitudinal spring with stiffness k_{F1})

The mechanical model of a fastening with its individual parameters is shown in Fig. 3.4.

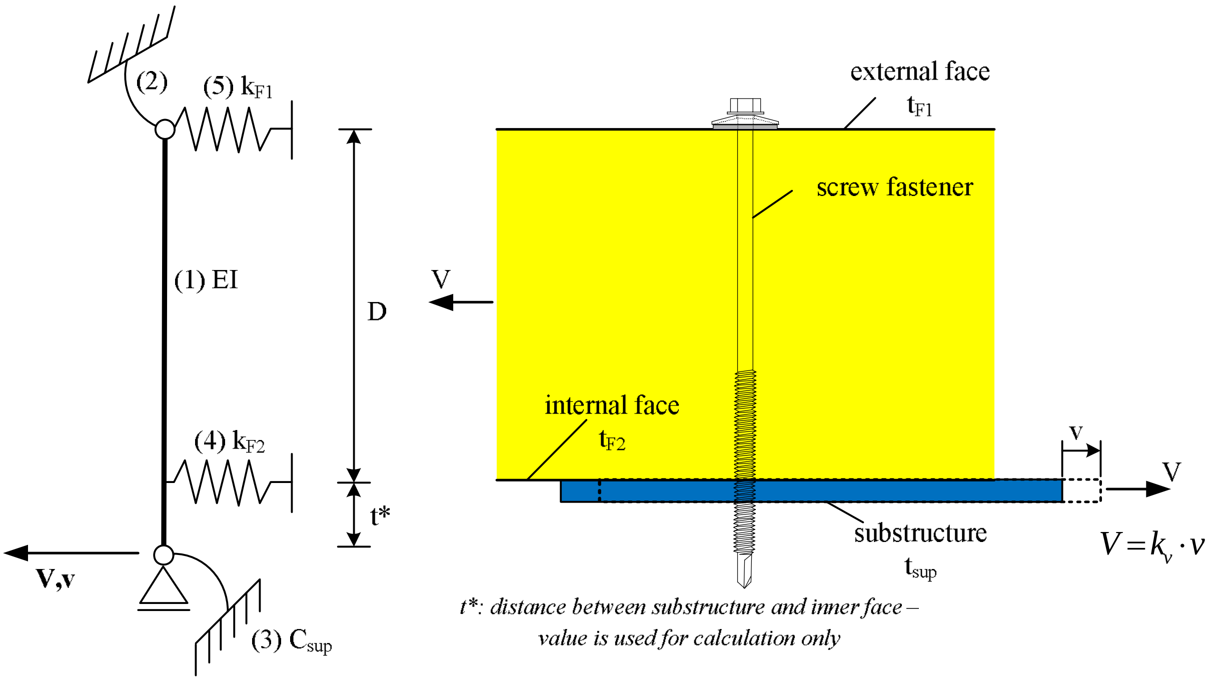


Fig. 3.4: Individual components of a fastening

The translational stiffness of a connection with a self-drilling or self-tapping screw fastener can be calculated with

$$k_v = \frac{1}{\dfrac{x_F}{k_{F2}} + \dfrac{t_{cor,\sup}^2 + 2\cdot(1-x_F)\cdot D\cdot t_{cor,\sup}}{4\cdot C_{\sup}} + \dfrac{3\cdot(1-x_F)\cdot D\cdot t_{cor,\sup}^2 + t_{cor,\sup}^3}{24\cdot EI}} \tag{21}$$

and

$$x_F = 1 - \frac{\dfrac{1}{k_{F2}} - \dfrac{D\cdot t_{cor,\sup}}{2\cdot C_{\sup}} - \dfrac{D\cdot t_{cor,\sup}^2}{8\cdot EI}}{\dfrac{1}{k_{F2}} + \dfrac{D^2}{C_{\sup}} + \dfrac{D^2\cdot(2\cdot D + 3\cdot t_{cor,\sup})}{6\cdot EI}} \tag{22}$$

where

$t_{cor,F2}$ core thickness of internal face
$t_{cor,sup}$ core thickness of the supporting structure
d_1 minor diameter of the threaded part of the fastener
d_S diameter of the unthreaded shank
$f_{u,F2}$ tensile strength of the internal face
D thickness of panel at point of fastening
and the following parameters:

Bending stiffness of the fastener

$$EI = 200000 N/mm^2 \cdot \frac{\pi \cdot d_S^4}{64} \tag{23}$$

Stiffness of clamping in the supporting structure

$$C_{\sup} = 2400 N/mm^2 \cdot \sqrt{t_{cor,\sup} \cdot d_1^5} \tag{24}$$

Stiffness of internal face sheet (hole elongation)

$$k_{F2} = 6.93 \cdot \frac{f_{u,F2} \cdot \sqrt{t_{cor,F2}^3 \cdot d_1}}{0.26mm + 0.8 \cdot t_{F2}} \text{ for } 0.40mm \leq t_{cor,F2} \leq 0.70mm \tag{25}$$

$$k_{F2} = \frac{4.2 \cdot f_{u,F2} \cdot \sqrt{t_{cor,F2}^3 \cdot d_1}}{0.373mm} \text{ for } 0.70mm \leq t_{cor,F2} \leq 1.00mm \tag{26}$$

These parameters and equations apply within the application range given in Table 3.1.

Table 3.1: Application range

5.5 mm ≤ d ≤ 8.0 mm	nominal diameter of the fastener
40 mm ≤ D	panel thickness
0.40 mm ≤ $t_{cor,F2}$ ≤ 1.00 mm	core sheet thickness of the face layers (steel)
1.50 mm ≤ $t_{cor,sup}$ ≤ 10.0 mm	core thickness of the supporting structure (steel)

The application range has to be taken into account, see Table 3.1. If higher values of parameters d, t_{F2} or t_{sup} occur, the calculation procedure is applicable, but the values used in calculations should be reduced to the upper limits of the application range.

For connections not included in the application range, e.g. sandwich panels with faces made of aluminium, the shear stiffness and resistance shall be determined by tests (see chapter 3.3.2).

Comments:

The clamping of the head (2) and the hole elongation of the external face (5) have only a minor influence. Therefore, these components have been ignored in expressions (20) and (21).

The stiffness of a fastening given above corresponds to the load level at the serviceability limit state, which is assumed not to exceed half of the characteristic value of the shear resistance provided by the internal face sheet. This characteristic value can be estimated using

$$V_{Rk} = 4.2 \cdot \sqrt{t_{cor,F2}^3 \cdot d_1} \cdot f_{u,F2} \tag{27}$$

The design value is:

$$V_{Rd} = \frac{V_{Rk}}{\gamma_{M2}} \tag{28}$$

The material safety factor γ_{M2} is given in the national specifications. According to EN 1993-1-3 [23], γ_{M2} = 1.25 is recommended. If the load bearing capacity is determined by tests according to different approvals γ_{M2} = 1.33 shall be used.

Comments:

Being on the safe side, the stiffness k_v given in Table 3.2 can be used. The values apply for thicknesses of the steel supporting structure 1.5 mm ≤ $t_{cor,sup}$ ≤ 4.0 mm.

Table 3.2: Stiffness k_v of fastenings [kN/mm]

nominal thickness of the inner face sheet t_{F2}	*S220GD*	*S280GD*	*S320GD*
0.40 mm	*1.6*	*1.9*	*2.0*
0.50 mm	*2.0*	*2.3*	*2.5*
0.63 mm	*2.4*	*2.9*	*3.1*
0.75 mm	*2.8*	*3.3*	*3.6*

3.3.2 Determination of the stiffness by tests

The test set-up and performance of the tests is described in [19]. The evaluation of the tests is described in chapter 2.9 of [18].

3.4 Stabilization forces

An imperfection (initial deflection) of the beams following a sinusoidal half-wave is assumed.

$$e_0(x) = e_0 \cdot \sin\left(\frac{\pi \cdot x}{L}\right) \tag{29}$$

The imperfection e_0 at x = L/2 shall be determined according to EN 1993-1-1 [22]:

$$e_0 = \frac{L}{500} \cdot \sqrt{0.5 \cdot \left(1 + \frac{1}{m}\right)} \tag{30}$$

where

m number of components to be stabilized

In addition, the compression force F_i in the component to be stabilized is assumed to be constant in the longitudinal direction of the beam. If a beam without an axial loading is considered, the normal force resulting from the bending moment M_d in the flange subjected to compression is

$$F_i = \frac{M_d}{h} \tag{31}$$

For the stabilization of a compression member subjected to an axial force (31) shall be modified to

$$F_i = N_d \tag{32}$$

and for the stabilization of a beam-column with axial force and bending moment

$$F_i = N_d + \frac{M_d}{h} \tag{33}$$

or

$$F_i = \frac{N_d}{2} + \frac{M_d}{h} \tag{34}$$

depending on whether flexural buckling (equation (33)) or lateral torsional buckling (equation (34)) is concerned.

Due to the effects of 2nd order theory, the axial compression force leads to an amplifiction of the deformation. This effect is usually considered by an amplification factor. If only the shear stiffness of the stabilizing sandwich panels considered and the bending stiffness of the stabilized component is neglected, the amplification factor can be written as

$$\alpha = \frac{1}{1 - \frac{F_i}{S_i}} \tag{35}$$

Comment:

In formula (35) given above, the bending stiffness of the stabilized component (e.g. the compressed flange of the beam) is neglected. The bending stiffness can be considered by modification of the amplification factor. In addition, the normal force F_i was assumed to be constant in longitudinal direction of the beam. Both assumptions are on the safe side. To consider an inconstant normal force - resulting from a bending moment on a single-span beam – in [45] an adjustment of the amplification factor with the factor 1/2 is proposed.

$$\alpha = \frac{1}{1 - \frac{1}{2} \cdot \frac{F_i}{S_i + EI \cdot \left(\frac{\pi}{L}\right)^2}} \tag{36}$$

The resulting deflection can be written as

$$e_{tot}(x) = e_0(x) + e(x) = \alpha \cdot e_0(x) = e_0 \cdot \frac{1}{1 - \frac{F_i}{S_i}} \cdot \sin\left(\frac{\pi \cdot x}{L}\right) \tag{37}$$

The additional deflection e results in a rotation of the sandwich panels in relation to the supporting structure.

$$\gamma(x) = e(x)' = e \cdot \frac{\pi}{L} \cdot \cos\left(\frac{\pi \cdot x}{L}\right) \tag{38}$$

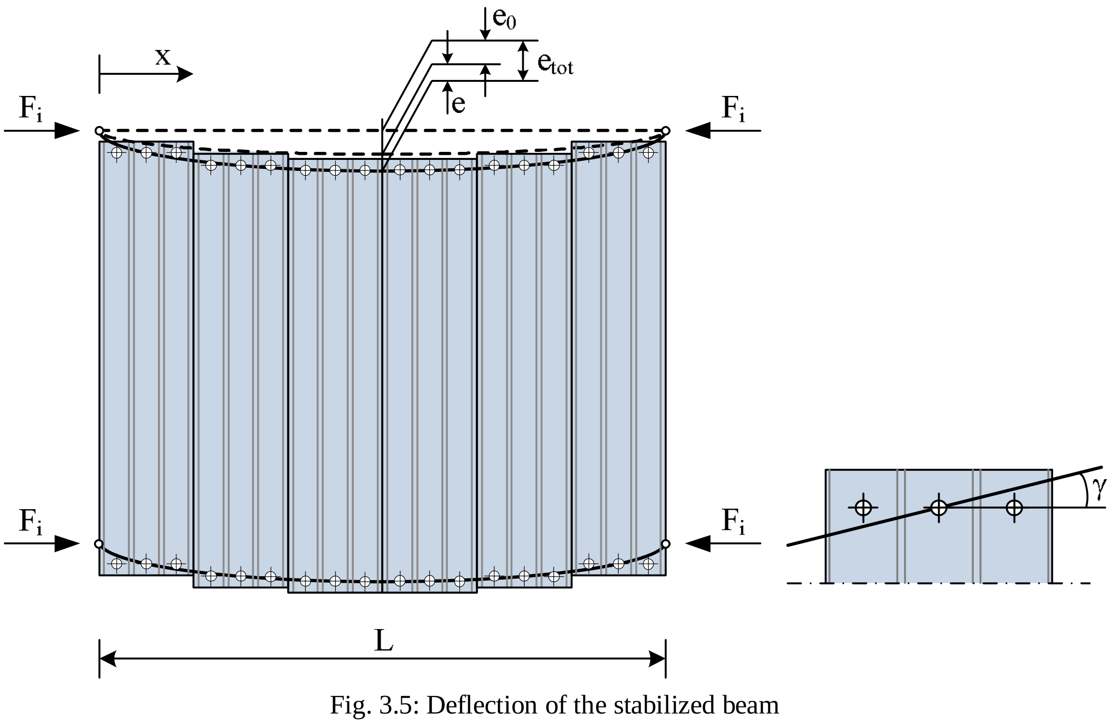


Fig. 3.5: Deflection of the stabilized beam

The deflection and the normal force result in the moment M_i.

$$M_i(x) = F_i \cdot e_{tot}(x) = F_i \cdot e_0 \cdot \frac{1}{1 - \frac{F_i}{S_i}} \cdot \sin\left(\frac{\pi \cdot x}{L}\right) \tag{39}$$

With this moment, the restraining load q_i acting on the stabilizing panels shall be determined.

$$q_i(x) = -\left(M_i\ (x)\right)'' = F_i \cdot \left(\frac{\pi}{L}\right)^2 \cdot e_0 \cdot \frac{1}{1 - \frac{F_i}{S_i}} \cdot \sin\left(\frac{\pi \cdot x}{L}\right) \tag{40}$$

The load $q_i(x)$ is the restraining load, which prevents a transverse displacement of the beam or of the compressed flange.

Instead of preventing the transverse displacement, a beam can also be restraint by preventing the rotation about the z-axis. To prevent rotations, a moment $m_i(x)$ is assumed. The moment $m_i(x)$ represents a restraining moment per unit length [kNm/m] along the longitudinal axis of the beam.

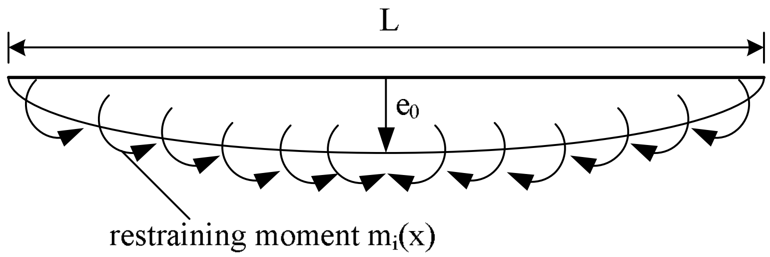


Fig. 3.6: Restraining moment

The restraining moment shall be calculated with

$$m_i(x) = -\left(M_i\ (x)\right)' = F_i \cdot \left(\frac{\pi}{L}\right) \cdot e_0 \cdot \frac{1}{1 - \frac{F_i}{S_i}} \cdot \cos\left(\frac{\pi \cdot x}{L}\right) \tag{41}$$

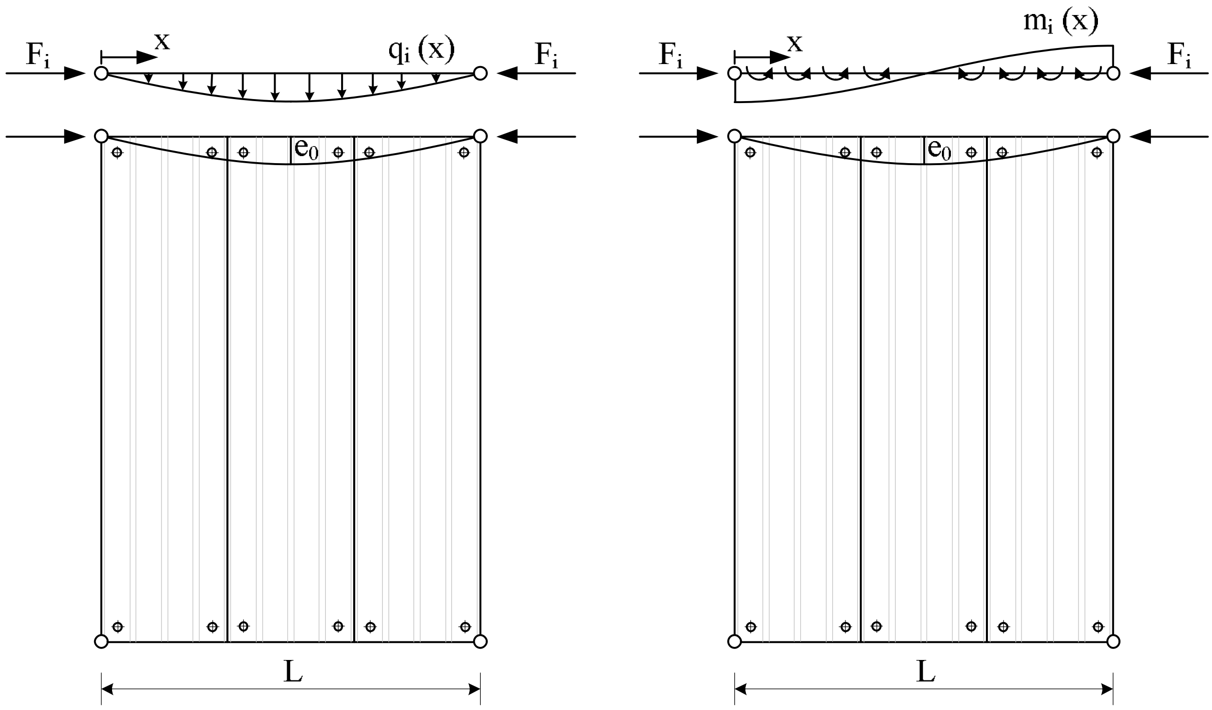


Fig. 3.7: Load resulting from the stabilization of a single component

Comments:

Further explanation of the derivation of the restraining moment $m_i(x)$ can be found in [37].

An alternative derivation of the restraining moment $m_i(x)$ is given in [28]. If a beam is restraint by sandwich panels, the fasteners introduce a force V in the panel; e.g. we have two fasteners on a transverse edge at the distance of c; we get the moment

$$M = V \cdot c \tag{42}$$

The moment M is exposed in each panel. The forces V, and therefore also the moments M depend on the rotation γ of the beam. So the highest moment acts at the ends of the beam and M = 0 in the mid-span of the beam. If the moments M are smeared over the length of the beam, we get a moment $m_i(x)$ per unit length.

3.5 Forces in fastenings

3.5.1 Introduction

If sandwich panels are used for the stabilization of single components, in the design of the fastenings additional shear forces have to be considered.

The forces resulting from the stabilization shall be considered in the design of the fastenings in any case, even though full restraint is shown by the stiffness S_i according to formula (3). If the beams restrained by the sandwich panels are designed according to EN 1993-1-3 [23], section 10.1, the forces in sheet/purlin fasteners and reaction forces according to table 10.4 of EN 1993-1-3 shall be taken into account additionally.

3.5.2 Uni-directionally spanning panels

The restraining moment $m_i(x)$ (expression (41)) has its maximum $m_{i,max}$ at the ends of the beam (x = 0, x = L).

$$m_{i,\max} = F_i \cdot \left(\frac{\pi}{L}\right) \cdot e_0 \cdot \frac{1}{1-\frac{F_i}{S_i}} \tag{43}$$

The forces of the fastenings of the panel shall withstand the moment $m_i(x)$. So the moment m_i (per unit length) is converted to the moment M_S acting on one panel. For the panels at the ends of the beam the moment M_S is approximately

$$M_{S,\max} = m_{i,\max} \cdot B = F_i \cdot \left(\frac{\pi}{L}\right) \cdot e_0 \cdot \frac{1}{1-\frac{F_i}{S_i}} \cdot B \tag{44}$$

where

B width of the panel

The moment M_S results in the shear forces V_S^M in the fastenings (Fig. 3.8). These forces act in longitudinal direction of the panel. The highest forces arise in the outer fastenings of a panel. The force in the highest stressed fastenings is

$$V_{S,\max}^{M} = \frac{M_{S,\max}}{\sum \frac{c_k^2}{c_1}} \tag{45}$$

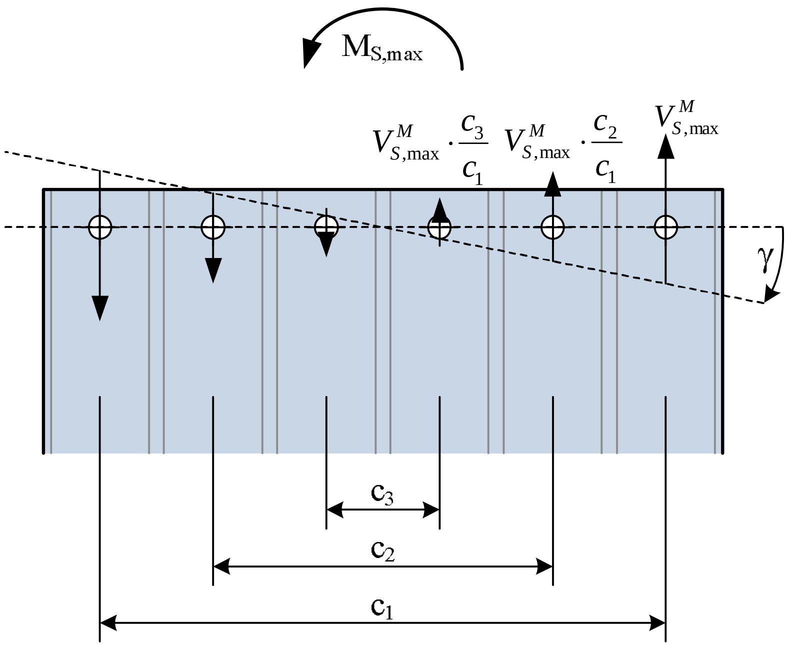


Fig. 3.8: Forces resulting in the moment M_S

Fig. 3.9 shows a panel with the moments M_S resulting from stabilization of the supporting structure. At the end supports additional forces Q_S in transverse direction are derived from moment equilibrium. If a constant distribution on the fastenings of the transverse edge is assumed, for one fastening the force V^Q in transverse direction is

$$V_{S,\max}^{Q} = \frac{m \cdot M_{S,\max}}{L_S \cdot n_f} \tag{46}$$

where

n_f number of fasteners per panel and support

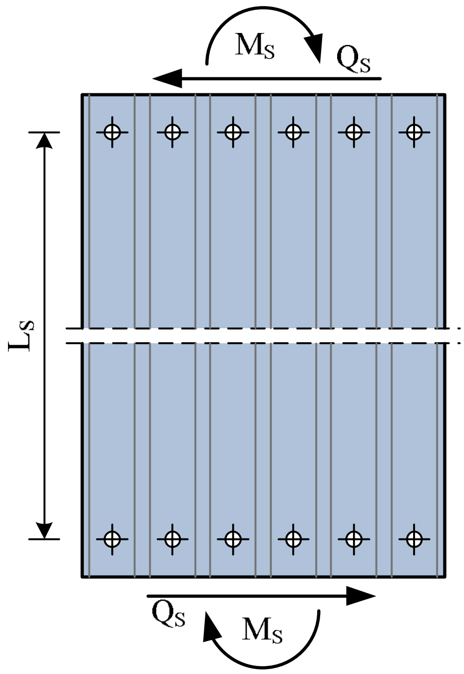


Fig. 3.9: Moment equilibrium of a panel

The resulting force of one fastening is determined by a vector summation. So in the most stressed fastening, the shear force resulting from stabilization is

$$V_{S,\max} = \sqrt{\left(V_{S,\max}^{M}\right)^2 + \left(V_{S,\max}^{Q}\right)^2} \qquad (47)$$

The shear force $V_{S,max}$ together with the shear and tensile forces caused by the primary loading of the sandwich panels shall be considered in the design of the fastenings.

Comments:

The loads of the ordinary structural behaviour expose forces in perpendicular direction to the faces. These loads cause normal loads in the fastenings, which shall be considered simultaneously with the in-plane shear loads.

Stabilization effects are usually taken into account for loads resulting from self-weight, snow or wind pressure, which do not cause normal forces in the fastenings. Thus, in the design of the fastenings the interaction between normal force resulting from wind suction and shear force resulting from stabilization need not be considered in many cases.

3.5.3 Panels with a single rigid support

In addition to the forces V_S^M resulting from the rotation of the panel, forces V_S^Δ occur. The forces V_S^Δ are caused by translational restrain of the panels. Both forces shall be added. The force in the highest stressed fastenings shall be calculated using

$$V_{S,\max}^{M+\Delta} = F_i \cdot \left(\frac{\pi}{L}\right) \cdot e_0 \cdot \frac{1}{1-\dfrac{F_i}{S_i + \Delta S_i}} \cdot B \cdot \sqrt{\left(\frac{1}{n_f} \cdot \frac{\pi}{L}\right)^2 + \left(\frac{c_1}{\sum c_k^2}\right)^2} \tag{48}$$

For determining the forces in the fastenings at the rigid support, the forces resulting from the translational restrain of the sandwich panels shall be added over the beams to be stabilized. The force in the highest stressed fastenings at the rigid support shall be calculated using

$$V_{S,\max}^{\Delta} = F_i \cdot \left(\frac{\pi}{L}\right)^2 \cdot e_0 \cdot \frac{1}{1-\dfrac{F_i}{S_i + \Delta S_i}} \cdot \frac{B}{n_f} \cdot m \tag{49}$$

At the end supports of the panels additional forces in transverse direction are derived from moment equilibrium (see Fig. 3.9). For one fastening the force V^Q in transverse direction is

$$V_{S,\max}^{Q} = \frac{m \cdot M_{S,\max}}{L_S \cdot n_f} \tag{50}$$

with

$$M_{S,\max} = m_{i,\max} \cdot B = F_i \cdot \left(\frac{\pi}{L}\right) \cdot e_0 \cdot \frac{1}{1-\dfrac{F_i}{S_i + \Delta S_i}} \cdot B \tag{51}$$

The resulting force of one fastening is determined by vector summation. So, in the most stressed fastening at the stabilized beams, the shear force resulting from stabilization is

$$V_{S,\max} = \sqrt{\left(V_{S,\max}^{M+\Delta}\right)^2 + \left(V_{S,\max}^{Q}\right)^2} \tag{52}$$

In the most stressed fastening at the rigid support of the shear force resulting from stabilization is

$$V_{S,\max} = \sqrt{\left(V_{S,\max}^{\Delta}\right)^2 + \left(V_{S,\max}^{Q}\right)^2} \tag{53}$$

The shear force $V_{S,\max}$ together with the shear and tensile forces caused by primary loading of the sandwich panels shall be considered in the design of the fastenings.

Comments:

The comment at the end of chapter 3.5.2 also applies here.

In some cases the stabilizing effect of the sandwich panels may reduce the buckling length of the stabilized beams. A reduction of the buckling length can cause higher forces in the fastenings. So the influence of the buckling length of the beams must be checked carefully.

3.6 Limitation of deformations

In addition to the design of the fastenings, the displacements resulting from the stabilization should be limited. The angle between sandwich panel and stabilized beam (formula (38)) has its maximum value at the ends of the beam.

$$\gamma_{\max} = e_0 \cdot \frac{\pi}{L} \cdot \frac{1}{\frac{S_i}{F_i} - 1} \tag{54}$$

It is recommended to limit the angle γ_{max} to

$$\gamma_{\max} \leq \frac{1}{750} \tag{55}$$

In [47], this limitation was proposed for the design of diaphragms made of trapezoidal sheets.

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ANNEX 1: PRACTICAL CONSIDERATIONS

1. GENERAL REMARKS

This document introduces the design principles of sandwich panels, which are used as stabilizing elements for single members of the supporting structure such as beams, purlins and columns. The role of the sandwich panels is more demanding compared to the traditional use of the sandwich panels, in which the panels carry the pressure and suction loads exposed perpendicular to the face of the panels only. It shall be noted, that the document introduces rules to stiffen single steel members against buckling and lateral buckling using sandwich panels. The document does not cover the cases in which whole buildings or units are stiffened with wall and roof panels.

The practice of design, building, proofing and inspection varies in different countries. However, principally the same tasks have to be made during the building process and shall be considered during the use of the building. This chapter makes remarks on some points, which have practical importance. The use of sandwich panels and the ways and methods of practice are under development phases in the near future. Therefore, the documentation of the design and building work is highly important concerning the later use of the building.

2. RESPONSIBILITIES AND TASKS

2.1 General Information

Sandwich panels used to stabilize steel structures become an integral part of a load-bearing system. Therefore, the design of such a structure requires a constant flow of information between

- the owner and user of the building
- the designer of the steel structure
- the designer of the sandwich panels
- the manufacturer of the panels
- the installer of the steel structure and sandwich panels on site

Each of the mentioned partners has the responsibilities and obligations towards the others. The responsibility may also cover the tasks of another partner, or there are additional partners such as an external supervisor. The following paragraphs give hints to assure the flow of information and to help to define the responsibilities and obligations.

2.2 Owner or User

The owner or user has to be aware of the role of the sandwich panels as stiffening components in the building. He shall consider the role of the sandwich panels in later modifications and updates of the sandwich panels, steel structures and the whole building. He has to take care of the updates of the information in the documents and drawings.

2.3 Designer

Additional information about the sandwich panels and the fastening needed for the stabilization of the supporting structure shall cover the rotational stiffness and the in-plane shear stiffness, and the strength of the sandwich panels and connections and the shear stiffness and shear resistance of the fasteners used to fix the sandwich panels to the supporting structure.

The **designer of the sandwich panels** shall verify the resistance of the sandwich panels and fastenings to meet the requirements given by the primary loads to the panels and by the additional loads caused by the stiffening of the single steel members. The designer shall prepare drawings which show the types, materials and dimensions of the sandwich panels, and the types, materials, dimensions and positions of all fasteners and possibly the order to be followed in installing the fasteners on site. He shall define the tolerances to be fulfilled in fastenings. The design shall be based on relevant national and European standards and technical approvals.

The **designer of the steel structure** needs to know the properties of the sandwich panels such as the allowed span length in longitudinal direction, required support width, shear stiffness, and torsional restraint, and the required type, number and positions of fasteners. He shall define the stiffness needed to support the supporting structure against buckling failure on an acceptable level. On the other hand, the number of fasteners might affect the load-bearing capacity of the sandwich panel such as the support reaction capacity and the reduction of the wrinkling stress at the intermediate supports under uplift loadings.

Thus, the **designer of the sandwich panels** shall know the structural requirements resulting from the stabilization of the supporting structure and possibly, based on the requirements of the design of the steel members of the supporting structure, re-check the resistance of sandwich panels and fastenings.

At the end of design process full assembly documentation shall be prepared and delivered to the owner of the building and/or to the partner, who is responsible for the assembly.

2.4 Manufacturer of sandwich panels

The sandwich panels used as stabilizing elements shall fulfil the requirements of EN 14509 [24] shown by the CE-mark of the product (see chapter 1.2) and the application range given in this document. Depending on the case and practice, the manufacturer of the sandwich panels shall provide additional information about the stiffness and resistance of the fastenings or about the shear stiffness and torsional restraint of the stabilizing system. The possible document with additional information compared to that given in CE marking according to EN 14509 shall be based on test results and calculations.

2.5 Installer

Before starting the assembly, the installer shall perform the following controls:

- Check and verification of documentation for assembly: relevant drawings, list of fastening elements, details for fixing, assembly and fixing sequence

- Check of possible temporary supports of the steel structure during the installation work
- Check of the load bearing structural components considering the project documentation, technical standards and tolerances
- During the assembly the following activities shall be performed:
    - Check of the impact of assembly, weather conditions on the panels and supporting structure. In the case of extreme conditions during assembly, the installer shall contact the designer.
    - Supervision of the mounting work: type, number and location of the fasteners
- At the end of the assembly works, written statement of work compliance with design documentation shall be issued through the responsible person of the installation.

Depending on the practice, products and the assembly work may be also checked through an external partner or independent person instead of the installer himself. The external partner shall control the incoming products and verify the correspondence of the mounting work to that required by the designer. The final inspection document shall be prepared, including possible deviations, which may results in a need for special investigations.

Comments:

If the external partner is not involved in the assembly process but all responsibilities for incoming products and the mounting work are the obligation of the installer, a written report shall be issued at the end of the assembly work through the responsible person of the installer.

3. WORK ON SITE

Sandwich panels used for the stabilization of structural members are an integral part of the load-bearing system of the building. To assure the special care required in the mounting, these panels should be highlighted and marked in the layout drawings.

All instructions on the detailing resulting from the design shall be strictly followed. On-site processing and rework of the sandwich panels for example the cutting of holes for openings shall be defined in the layout drawings. Subsequent changes shall be re-checked through the designers both of the sandwich panels and of the supporting structure.

It is recommended to label the sandwich panels used for the stabilization of structural members in a clearly visible and durable way. The role of the sandwich panels as stabilizing members shall be understood and taken care during the later replacement and updates of the sandwich panels and supporting structure.

The supporting structure shall be checked before the assembly of the façade / roof panels considering project documentation, technical standards and tolerances. The structural safety of the non-stabilized steel members shall be considered during the installation work. Sandwich panels and fixing products shall be checked before starting assembly work. Proper installation shall be recorded and confirmed through the responsible person of the installer or through external proofing.

4. FASTENING OF SANDWICH PANELS

The designer of the fastenings used to fix the sandwich panels to the supporting structure shall consider two kinds of forces:

- Forces resulting from loads acting directly on the panel such as self-weight, wind load and snow load, and the temperature difference load. These forces are usually determined through the designer of the sandwich panels.
- Forces and required stiffness resulting from stabilization of the supporting structure. These requirements are usually determined through the designer of the supporting structure.

All loads and combinations of loads regarding the valid standards and specifications shall be considered in the design of the fastenings.

All fasteners used in fixing the sandwich panels to the supporting structure and in the connections between the adjacent sandwich panels shall have a European (ETA) or national technical approval, which state the dimensions, materials and mechanical properties of the fasteners as well as characteristic values of the resistance of the connections.

Screws completely or partly exposed to weather or comparable moisture conditions shall be made of austenitic stainless steel or have a proper corrosion protection system corresponding to the corrosivity category of the environment. The sealants of the washers shall be made of EPDM.

The type of fasteners, the diameters of sealing washers, the number of fasteners and the positions of fixing shall correspond to the requirements made in design. Special care shall be taken to the positioning of the fasteners with respect to the supporting structure. The position of the fastener in the upper flange of the beam to be stabilized has tight tolerances for example. Depending on the requirements defined in the design, the fasteners shall be arranged in a straight line or in an alternating fixing pattern positioned on both sides of the beam's flanges. The pre-drill diameter of the self-tapping screws shall comply with the value given in the technical approval.

A minimum edge distance of 20 mm in direction of the span is required, if no larger value is given by an approval or by the manufacturer.

For a proper fixation of sandwich panels, the fasteners shall be tightened in a way that the sealing washer has a slight deformation, required for a tight connection. This causes a light indentation of the outer face of the sandwich panel which is unavoidable in practice. For flat and lightly profiled faces, the indentation should be less than 2 mm (Fig. A.1). Larger deformations shall be avoided. A correct tightening of the screws attaching two steel plates is also important in order to achieve correct resistance and air and water tightness (Fig. A.2).

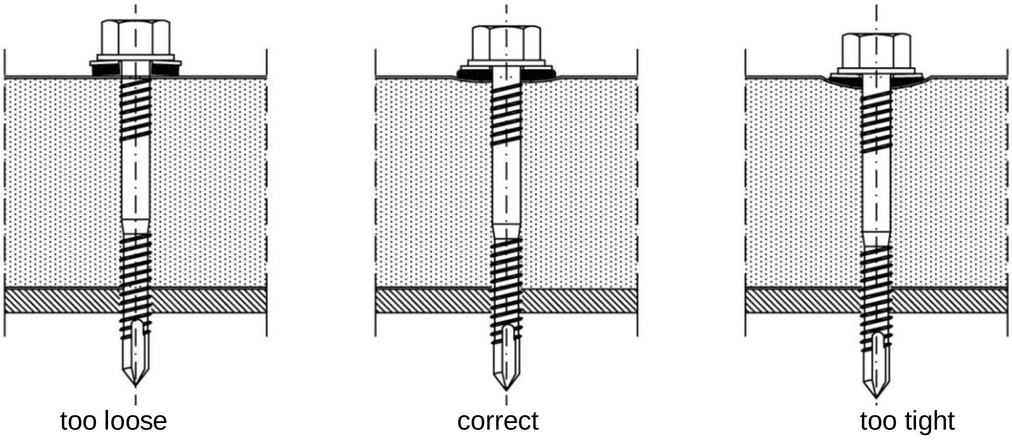

Fig. A.1: Tightening of the screws of sandwich panels to the supporting structure (figure: IFBS [30])

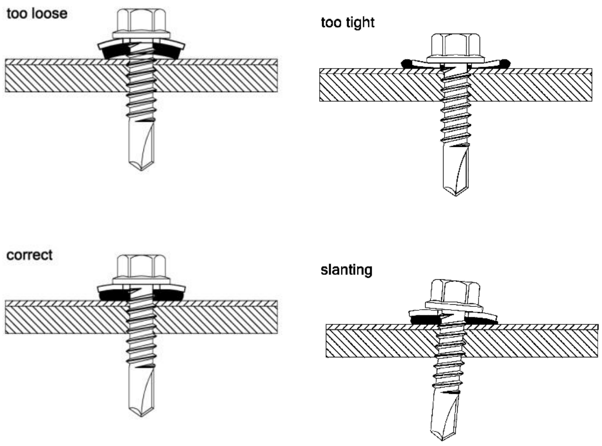

Fig. A.2: Tightening of the screws between two steel sheets (figure: IFBS [30])

Fasteners shall be placed perpendicular to the surface of the panel to obtain a safe, water- and air-tight connection. This requirement might be a challenge in cases in which the depth of the sandwich panels is large (Fig. A.3). Further information about the mounting of regular sandwich panels can be found in the documents [29], [30], [15].

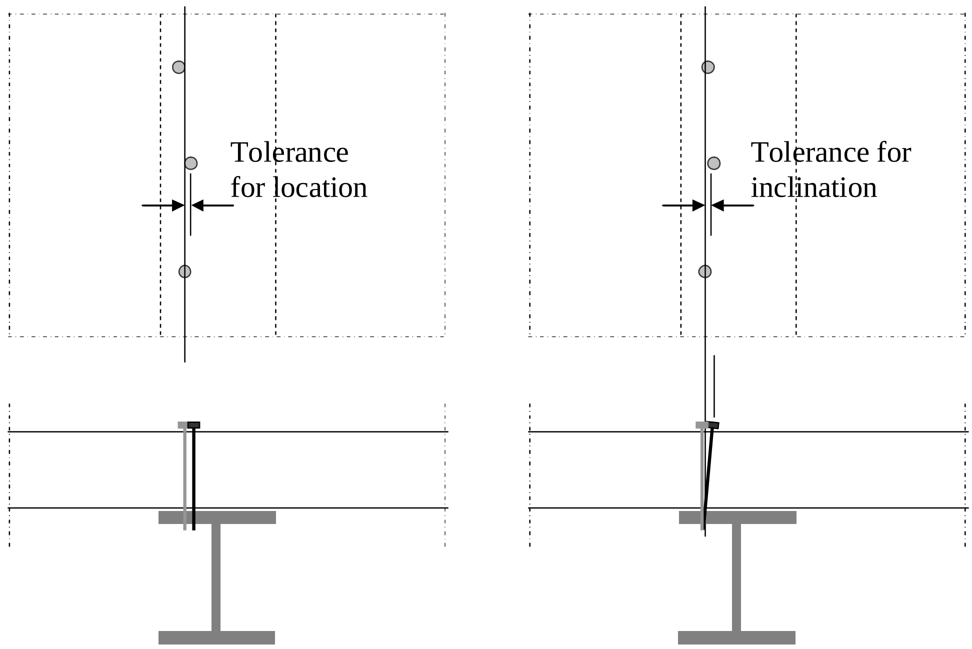

Fig. A.3: Tolerances concerning the location and inclination of the screws drilled through the sandwich panels

Location of the connection point between the internal face of the sandwich panel and the surface of the supporting structure is highly important concerning the effect of stabilization. However, for mounting, the location is defined in the external face of the sandwich panel. Therefore, special care shall be given to the definition of the allowed tolerances concerning the deviation of the fastener in the plane of the sandwich panel and the deviation of the angle of fastener from 90 degrees.

- It is important for the end result that the panels themselves are well aligned when lifted on the final positions. This assures that installers easily find the correct fastening points. If the installers use the panel manufacturer's recommendation about the minimum end distance, e.g. 20 mm, they are able to define the center line of the purlin or the other target point in the purlin. The fixing problems may come from the fact that fasteners hit a wrong location in the purlin, either too close to the welding or the edge. The correct alignment of the sandwich panel would help in many cases. The allowable deviations of the fasteners from the target line shall be defined by the designer.
- The installers do not use additional fastening tools besides a standard screw gun to keep the angle to 90 degrees to the face of the sandwich panel. The inclination itself does not affect the stabilization, but the hit in an incorrect location in the supporting structure due to the inclination may lead to a reduction of the effect of stabilization, especially in the case of thick sandwich panels. The use of an additional guide in the screw gun to guarantee the perpendicular direction of the screws in case of thick panels (> 100 mm) is recommended.
- The limiting factors for tolerances may be the tightness of the sealants of the washer under static and dynamic loads, the performance of the drilling operation and the positioning of the screw end in the correct place in the supporting structure.

ANNEX 2: EXAMPLES

The calculated examples given in this Annex illustrate the calculation procedures introduced in the Recommendations. Examples 1 and 2 deal with the torsional restraint provided by sandwich panels, examples 3 and 4 deal with the lateral restraint. Example 3 was originally published in [34]. An additional example dealing with lateral restraint can be found in [37].

Example No. 1: Torsional restraint of a hot-rolled section

Sandwich roof panels are used to stabilize a purlin IPE200. The second moment of inertia about the weak axis of the purlin is 142 cm^4. The purlin is loaded by a transverse load (self-weight and snow)

q_d = 3.7 kN/m (ultimate limit state)

q_d = 2.7 kN/m (serviceability limit state)

which results in a bending moment. The purlin is installed as a multi-span-system with a span of 6.0 m. The purlins have to be stabilized against lateral torsional buckling.
For the sandwich panels, the following parameters apply:

- Core material mineral wool, E_{Cc} = 4 N/mm², E_{Ct} = 6 N/mm², f_{Cc} = 0.06 N/mm².
- Steel faces with nominal thicknesses t_{F1} = 0.75 mm , t_{F2} = 0.63 mm

The purlin IPE200 has a width of the flange of b = 100 mm. The sandwich panel is fixed to the upper flange of the purlin at every crest with a distance of 333 mm (n_f = 3 m^{-1}).

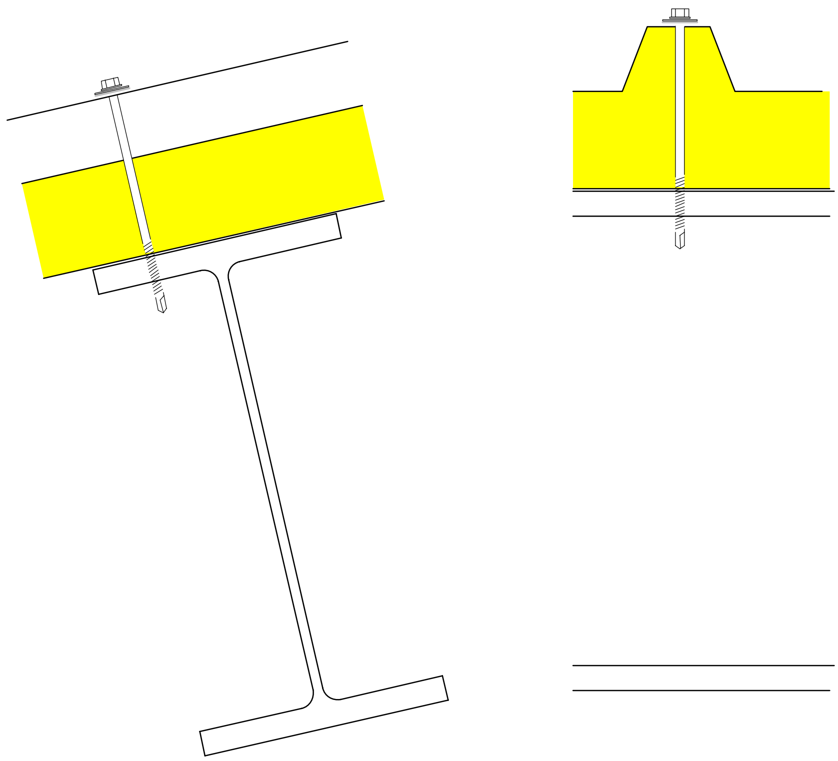

Fig. A.4: Purlin, sandwich panel and details of fastening

The requirements according to Table 2.4 are fulfilled.

Modulus of elasticity of the core:

$$E_C = 0.5 \cdot \left(4 \frac{N}{mm^2} + 6 \frac{N}{mm^2} \right) = 5 \frac{N}{mm^2}$$

The modulus shall be reduced to consider the duration of the loading (creep). The action with the shortest time of load duration becomes crucial. For the action of snow load on mineral wool panels, $\varphi_{\vartheta,t}$ = 1.35 (Table 2.2) applies. During the time of loading, a temperature of 0°C shall be assumed, and no further reduction of the elastic modulus because of the temperature is necessary.

$$E_{C,t} = \frac{E_C}{1+\varphi_{\vartheta,t}} = \frac{5 \frac{N}{mm^2}}{1+1.35} = 2.1 \frac{N}{mm^2}$$

Based on the position of the fasteners in the direction of slope,

$$b_k = 0.75 \cdot b = 75 mm$$

applies (Fig. 2.4). From Table 2.3, we obtain c_1 = 0.089 and c_2 = 0.027 m.

$$C_{\vartheta 1} = c_1 \cdot E_{C,t} \cdot b^2 = 0.089 \cdot 2.1 \frac{N}{mm^2} \cdot (100 mm)^2 = 1869 \frac{Nmm}{mm} = 1.869 \frac{kNm}{m}$$

and

$$C_{\vartheta 2} = c_2 \cdot n_f \cdot E_{C,t} \cdot b_K^2 = 0.027 m \cdot 3 \frac{1}{m} \cdot 2.1 \frac{N}{mm^2} \cdot (75 mm)^2$$
$$= 957 \frac{Nmm}{mm} = 0.957 \frac{kNm}{m}$$

The value of the secant stiffness of $C_{\vartheta A}$ is calculated as follows

$$C_{\vartheta A} = \frac{3}{2} \cdot \frac{C_{\vartheta 1}}{\left(\frac{C_{\vartheta 1}}{C_{\vartheta 1} + C_{\vartheta 2}} + 1 \right)} = \frac{3}{2} \cdot \frac{1.869 \frac{kNm}{m}}{\left(\frac{1.869 \frac{kNm}{m}}{1.869 \frac{kNm}{m} + 0.957 \frac{kNm}{m}} + 1 \right)} = 1.687 \frac{kNm}{m}$$

Limitation of the stabilization moment at the ultimate limit state is calculated on the basis of the expressions in chapter 2.4.

Contact moment (ultimate limit state)

$$m_K = q_d \cdot \frac{b}{2} = 3.7\frac{kN}{m} \cdot \frac{0.1m}{2} = 0.185\frac{kNm}{m}$$

Maximum bending moment (here: at intermediate supports)

$$M_{Ed} = q_d \cdot \frac{l^2}{12} = 3.7\frac{kN}{m} \cdot \frac{(6.0m)^2}{12} = 11.10kNm$$

With the correction factor $k_c = 0.91$, the stabilization moment is

$$m_{\vartheta A} = \frac{1}{C_{\vartheta A} \cdot \frac{k_c^4 \cdot E \cdot I_z}{M_{Ed}^2} - 1} \cdot C_{\vartheta A} \cdot \vartheta_0$$

$$= \frac{1}{1.687\frac{kNm}{m} \cdot \frac{0.91^4 \cdot 21000\frac{kN}{cm^2} \cdot 142cm^4}{(11.10kNm)^2} - 1} \cdot 1.687\frac{kNm}{m} \cdot 0.06$$

$$= 0.056\frac{kNm}{m}$$

which is lower than the contact moment. Thus, the limitation is fulfilled.

Limitation of the rotation moment at the serviceability limit state is calculated on the basis of the expressions in chapter 2.5.

Contact moment (serviceability limit state)

$$m_K = q_d \cdot \frac{b}{2} = 2.7\frac{kN}{m} \cdot \frac{0.1m}{2} = 0.135\frac{kNm}{m}$$

With this moment the rotation of the beam is

$$\vartheta = \frac{m_K}{C_{\vartheta A}} = \frac{0.135kNm/m}{1.687kNm/m} = 0.08rad$$

which fulfils the limitation $\vartheta \leq 0.08$.

Example No. 2: Torsional restraint of a cold-formed C-section

Sandwich wall panels are used to stabilize a cold-formed C-section. The second moment of inertia about the weak axis is I_z = 101.9 cm^4. The section is loaded by a transverse load (wind pressure)

q_d = 2.7 kN/m (ultimate limit state)

q_d = 1.8 kN/m (serviceability limit state)

which results in a bending moment $M_{E,d}$. The section is installed as a single span system with a span of 4.5 m. The section shall stabilized against lateral torsional buckling.

For the sandwich panels, the following parameters apply:

- Core material PUR, E_{Cc} = 4 N/mm², E_{Ct} = 3 N/mm², f_{Cc} = 0.12 N/mm².
- Steel faces with nominal thicknesses t_{F1} = 0.50 mm, t_{F2} = 0.40 mm

The C-section has a width of the flange of b = 60 mm. Fixing is done with at least two fasteners per panel with a width of one meter ($n_f \geq 2$ m^{-1}).

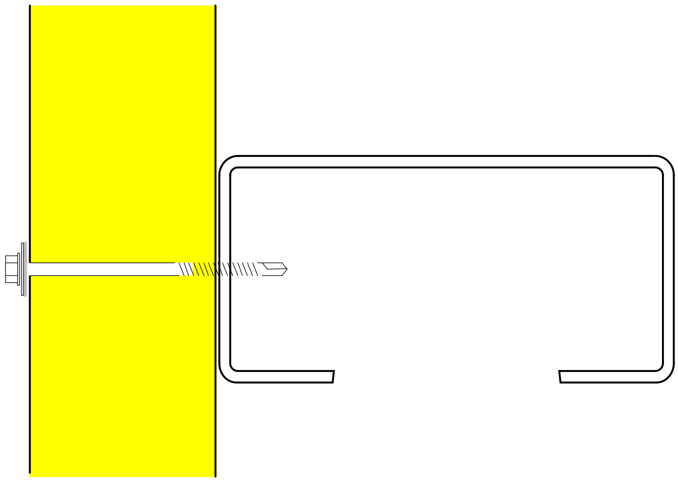

Fig. A.5: C-section, sandwich panel and fastening

The requirements according to Table 2.4 are met.

Modulus of elasticity of the core:

$$E_C = 0.5 \cdot \left(3 \frac{N}{mm^2} + 4 \frac{N}{mm^2} \right) = 3.5 \frac{N}{mm^2}$$

No reduction for duration of loading (creep) is required. The action with the shortest time of load duration becomes crucial. For the action of the wind load, $\varphi_{\vartheta,t}$ = 0.0. During the time of loading, a temperature of 0°C is assumed in this example. From Table 2.3, we obtain c_3 = 5.11·10^{-4} m^2.

$$C_{\vartheta 1} = c_3 \cdot E_{C,t} = 5.11 \cdot 10^{-4} m^2 \cdot 3.5 \frac{N}{mm^2} = 1789 \frac{Nmm}{mm} = 1.789 \frac{kNm}{m}$$

and

$$C_{\vartheta 2} = 0$$

for cold-formed sections. The secant value of the rotational stiffness is

$$C_{\vartheta A} = \frac{3}{2} \cdot \frac{C_{\vartheta 1}}{\left(\frac{C_{\vartheta 1}}{C_{\vartheta 1} + C_{\vartheta 2}} + 1 \right)} = \frac{3}{2} \cdot \frac{1.789 \frac{kNm}{m}}{\left(\frac{1.789 \frac{kNm}{m}}{1.789 \frac{kNm}{m} + 0 \frac{kNm}{m}} + 1 \right)} = 1.342 \frac{kNm}{m}$$

Limitation of stabilization moment at ultimate limit state

Contact moment (ultimate limit state)

$$m_K = q_d \cdot b = 2.7 \frac{kN}{m} \cdot 0.06m = 0.162 \frac{kNm}{m}$$

Bending moment

$$M_{Ed} = q_d \cdot \frac{l^2}{8} = 2.7 \frac{kN}{m} \cdot \frac{(4.5m)^2}{8} = 6.83 kNm$$

With the correction factor $k_c = 0.94$, the stabilization moment is calculated as

$$m_{\vartheta A} = \frac{1}{C_{\vartheta A} \cdot \frac{k_c^4 \cdot E \cdot I_z}{M_{Ed}^2} - 1} \cdot C_{\vartheta A} \cdot \vartheta_0$$

$$= \frac{1}{1.342 \frac{kNm}{m} \cdot \frac{0.94^4 \cdot 21000 \frac{kN}{cm^2} \cdot 101.9 cm^4}{(6.83 kNm)^2} - 1} \cdot 1.342 \frac{kNm}{m} \cdot 0.06$$

$$= 0.021 \frac{kNm}{m}$$

which is smaller than the contact moment. Thus, the limitation is fulfilled.

Limitation of the rotation at the serviceability limit state

Contact moment

$$m_K = q_d \cdot b = 1.8 \frac{kN}{m} \cdot 0.06m = 0.108 \frac{kNm}{m}$$

With this moment the rotation of the beam is

$$\vartheta = \frac{m_K}{C_{\vartheta A}} = \frac{0.108 kNm/m}{1.342 kNm/m} = 0.08 rad$$

which fulfils the limit $\vartheta \leq 0.08$.

Example No. 3: In-plane-shear stiffness

The sandwich panels shown in Fig. A.6 are used to stabilize the purlins supporting them. The purlins are loaded by a transverse load q_d = 7.33 kN/m, which results in a bending moment. Therefore, the purlins have to be stabilized against lateral torsional buckling.

At the transverse edges the panels are connected to the supporting structure as shown in Fig. A.7. The fastenings have the stiffness

$$k_v = 2.34 kN / mm$$

(The stiffness of the fastenings can be determined according to section 3.3.)

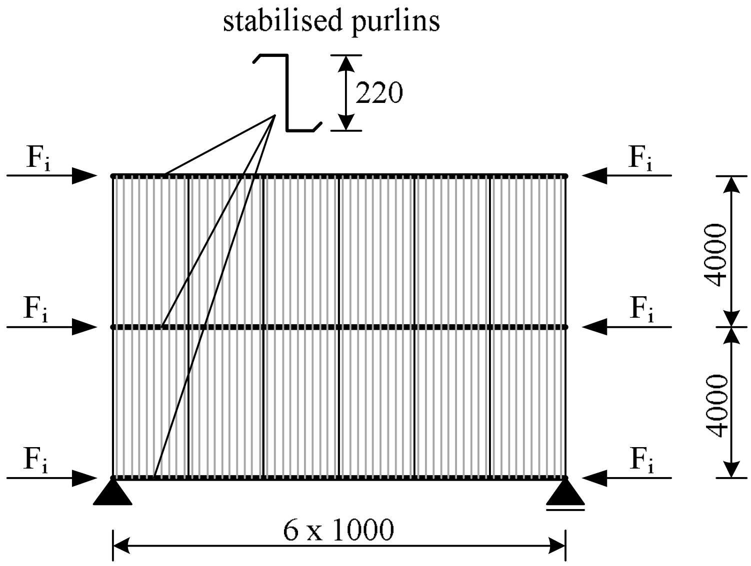

Fig. A.6: System

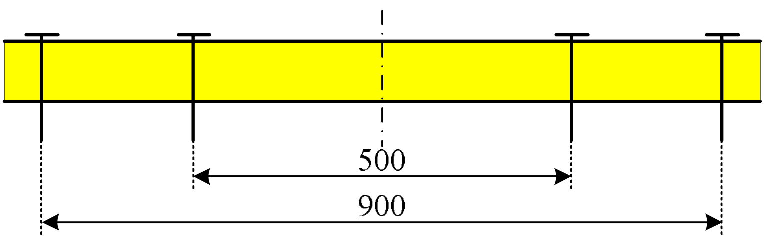

Fig. A.7: Position of fastenings at the supports of the sandwich panels

Initial deflection (equation (30)):

$$e_0 = \frac{6000mm}{500} \cdot \sqrt{0.5 \cdot \left(1 + \frac{1}{3}\right)} = 9.8mm$$

Compression force in the upper flange of the purlin (equation (31)):

$$M_{i,d} = \frac{1}{8} \cdot 7.33 \frac{kN}{m} (6m)^2 = 33kNm$$

$$F_{i,d} = \frac{33kNm}{0.22m} = 150kN$$

Determination of the shear stiffness S_i

The shear stiffness available for the stabilization of one beam is (equation (18))

$$S_i = \frac{2.34kN/mm}{2 \cdot 1000mm} \cdot \left((900mm)^2 + (500mm)^2\right) = 1240kN$$

Forces of the fastenings

The maximum bracing moment (x = 0 and x = l) is (equation (44))

$$M_{S,\max} = 150kN \cdot \left(\frac{\pi}{6000mm}\right) \cdot 9.8mm \cdot \frac{1}{1 - \frac{150kN}{1240kN}} \cdot 1000mm = 875.6kNmm$$

The moment $M_{S,max}$ results in forces V_S^M on the fastenings. The force on the outer fasteners of the panel is (equation (44))

$$V^M_{S,\max} = \frac{875.6kNmm}{\frac{(900mm)^2}{900mm} + \frac{(500mm)^2}{900mm}} = 0.743kN$$

The fastenings are also loaded through shear forces in transverse direction (equation (46)).

$$V^Q_{S,\max} = \frac{3 \cdot 875.6kNmm}{8000mm \cdot 4} = 0.082kN$$

The resulting force for which the fastenings have to be designed for is (equation (47))

$$V_{S,\max} = \sqrt{0.743^2 + 0.082^2} = 0.75kN$$

Each fastening shall be designed for the shear force $V_{S,max}$ (design value, ultimate limit state). If there are further shear forces, they shall be added in the sum. If there are also tensile forces, the interaction between shear and tensile forces shall be considered.

Limitation of deflection

The maximum shear angle is (equation (54))

$$\gamma_{\max} = 9.8mm \cdot \frac{\pi}{6000mm} \cdot \frac{1}{\frac{1240kN}{150kN} - 1} = 0.71 \cdot 10^{-3}$$

which is less than the limit angle

$$\gamma_{\max} \le \frac{1}{750}$$

Example No. 4: In-plane-shear stiffness - Panels with a single rigid support

The sandwich panels shown in Fig. A.8 are used to stabilize the beams supporting them. On the end support, the panels are connected with a concrete basement, i.e. with a rigid support (see Fig. 3.3). The beams are loaded through a transverse load q_d = 7.33 kN/m, which results in a bending moment. Therefore, the beams shall be stabilized against lateral torsional buckling.

The panels are connected to the beams and to the rigid support as shown in Fig. A.7. The fastenings on the stabilized beams have the stiffness

$$k_v = 2.34 kN/mm$$

The fastenings on the rigid support have the stiffness

$$k_{v,1} = 2.00 kN/mm$$

(The stiffness of the fastenings can be determined according to section 3.3.)

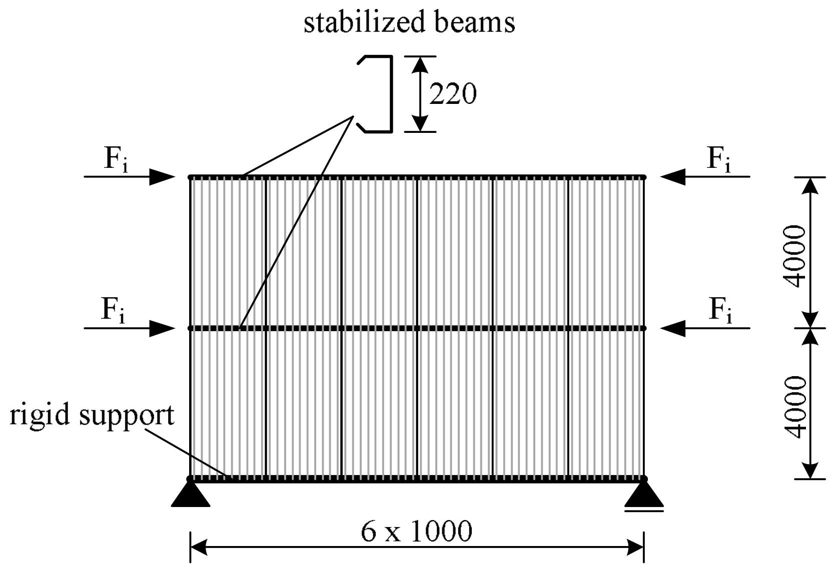

Fig. A.8: System

Initial deflection (equation (30)):

$$e_0 = \frac{6000mm}{500} \cdot \sqrt{0.5 \cdot \left(1 + \frac{1}{2}\right)} = 10.4mm$$

Compression force in the upper flange of the beam (equation (31)):

$$M_{i,d} = \frac{1}{8} \cdot 7.33 \frac{kN}{m} (6m)^2 = 33kNm$$

$$F_{i,d} = \frac{33kNm}{0.22m} = 150kN$$

Determination of stiffness $S_i + \Delta S_i$

The shear stiffness available for the stabilization of one beam is (equations (18) and (19))

$$S_i = \frac{2.34kN/mm}{2 \cdot 1000mm} \cdot \left((900mm)^2 + (500mm)^2\right) = 1240kN$$

$$\bar{k}_V = \frac{1}{\frac{2}{2,00kN/mm} + \frac{1}{2,34kN/mm}} = 0,70kN/mm$$

$$\Delta S_i = \frac{4 \cdot 0,70kN/mm}{1000mm} \cdot \left(\frac{6000mm}{\pi}\right)^2 = 10213kN$$

Forces of the fastenings on the stabilized beams

Force in direction of the panel´s span (equation (48))

$$V_{S,\max}^{M+\Delta} = 150kN \cdot \left(\frac{\pi}{6000mm}\right) \cdot 10.4mm \cdot \frac{1}{1 - \frac{150kN}{1240kN + 10213kN}} \cdot 1000mm \cdot$$
$$\cdot \sqrt{\left(\frac{1}{4} \cdot \frac{\pi}{6000mm}\right)^2 + \left(\frac{900mm}{(900mm)^2 + (500mm)^2}\right)^2} = 0,711kN$$

Force in transverse direction (equation (50))

$$M_{S,\max} = 150kN \cdot \left(\frac{\pi}{6000mm}\right) \cdot 10,4mm \cdot \frac{1}{1 - \frac{150kN}{1240kN + 10213kN}} \cdot 1000mm$$
$$= 827.7kNmm$$

$$V_{S,\max}^{Q} = \frac{2 \cdot 827.7kNmm}{8000mm \cdot 4} = 0.051kN$$

Resulting force (equation (52))

$$V_{S,\max} = \sqrt{0.711^2 + 0.051^2} = 0.71kN$$

Forces of the fastenings on the rigid support

Force in direction of the panel´s span (equation (49))

$$V_{S,\max}^{\Delta} = 150kN \cdot \left(\frac{\pi}{6000mm}\right)^2 \cdot 10.4mm \cdot \frac{1}{1-\frac{150kN}{1240kN+10213kN}} \cdot \frac{1000mm}{4} \cdot 2$$

$$= 0{,}217kN$$

Force in transverse direction (equation (50))

$$M_{S,\max} = 150kN \cdot \left(\frac{\pi}{6000mm}\right) \cdot 10.4mm \cdot \frac{1}{1-\frac{150kN}{1240kN+10213kN}} \cdot 1000mm$$

$$= 827.7kNmm$$

$$V_{S,\max}^{Q} = \frac{2 \cdot 827.7kNmm}{8000mm \cdot 4} = 0.051kN$$

Resulting force (equation (53))

$$V_{S,\max} = \sqrt{0.217^2 + 0.051^2} = 0.22kN$$

Each fastening shall be designed for the shear force $V_{S,max}$ (design value, ultimate limit state). If there are further shear forces, they shall be added in the sum. If there are also tensile forces, the interaction between the shear and the tensile forces shall be considered.

Limitation of deflection

The maximum shear angle is (equation (54))

$$\gamma_{\max} = 10.4mm \cdot \frac{\pi}{6000mm} \cdot \frac{1}{\frac{1240kN+10213}{150kN}-1} = 0.72 \cdot 10^{-4}$$

which is less than the limit angle

$$\gamma_{\max} \leq \frac{1}{750}$$

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CIB Publication 379 / ISBN 978-90-6363-081-2