European Recommendations for the Design, Detailing and Application of Fastenings for Sandwich Panels
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Authors Wim Bakens, Ivan Balazs, Andrej Belica, Klaus Berner, Nicholas Brown, Sebastien Charton, JM Davies, Mircea Georgescu, Paavo Hassinen, Simo Heikkilä, Antti Helenius, Lars Heselius, David Izabel, Karsten Kathage, Saskia Käpplein, Jörg Lange, Philip Leach, Jan-Christer Mäki, Thomas Misiek, Youcef Mokrani, Bernd Naujoks, Ute Pfaff, Lars Pfeiffer, Ralf Podleschny, Zbigniew Pozorski, Kari Rantakylä, Keith Roberts, Daniel Ruff, Helmut Saal, Johan Schedin, Robert Xiao, Danijel Zupancic

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Van der Burghweg 1
2628 CS, Delft
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E-mail secretariat@cibworld.nl
www.cibworld.nl

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PREFACE

This document provides the guidance on design, detailing and application of fastenings for sandwich panels. As the current and existing “Preliminary European Recommendations for testing and design of fastenings for sandwich panels” has been mainly written from the manufacturer’s point of view that focuses more on the determination of resistance values against the test results. This document will provide guidance for designers and contractors and therefore is focusing more on loads and actions on fastenings of sandwich panels. For readers interested in the background information, references to other publications are given, for example to other ERs, standards or scientific papers.

This document has been prepared by the Joint Committee of ECCS TC7 TWG7.9 and CIB Working Commission W056. The members of the Joint Committee who contributed to the document are:

Wim Bakens  Netherlands
Ivan Balazs  Czech Republic
Andrej Belica  Luxemburg
Klaus Berner  Germany
Nicholas Brown  Australia
Sebastien Charton  France
JM Davies  UK
Mircea Georgescu  Romania
Paavo Hassinen  Finland
Simo Heikkilä  Finland
Antti Helenius  Finland
Lars Heselius  Finland
David Izabel  France
Karsten Kathage  Germany
Saskia Käpplein  Germany
Jörg Lange  Germany
Philip Leach  UK
Jan-Christer Mäki  Sweden
Thomas Misiek  Germany
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<tr>
<td>Bernd Naujoks</td>
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<td>Robert Xiao</td>
<td>UK</td>
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<tr>
<td>Danijel Zupancic</td>
<td>Slovenia</td>
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NOTATION

$N_{Ed}$ design value of the tensile force
$V_{Ed}$ design value of the shear force
$N_{Rk}$ characteristic value of the tensile resistance
$V_{Rk}$ characteristic value of the shear resistance
$\max u$ maximum allowable value of head deflection
$N_{Rd}$ design value of the tensile resistance
$V_{Rd}$ design value of the shear resistance

$D$ overall depth of the panel
$d_C$ continuous depth of the panel core material, $d_c = D - (t_{F1} + t_{F2})$
$D_F$ distance between top of supporting structure and top of external face at point of fastening
$b_1$ width of the ribs of the external face (see EN 14509)

$t_{F1}$ design sheet thickness of the external face
$t_{F2}$ design sheet thickness of the internal face
$t_{II}$ design sheet thickness of the supporting structure
$t_{nom,F1}$ nominal sheet thickness of the external face
$t_{nom,F2}$ nominal sheet thickness of the internal face
$t_{nom,II}$ nominal sheet thickness of the supporting structure

$E_{Cc}$ compression modulus of the core
$f_{Cc}$ compression strength of the core
$f_{u,F1}$ tensile strength of the external face
$f_{u,F2}$ tensile strength of the internal face
$f_{u,II}$ tensile strength of the supporting structure
$d$ external thread diameter of the screw, usually the nominal diameter
$d_0$ diameter of the hole
$d_w$ diameter of washer
$d_{pd}$ self-drilling screw: max $(d_1, d_{dp})$
self-tapping screw: predrilling diameter
$d_1$ core diameter of the thread
$d_{dp}$ diameter of drill-point
$d_{ef}$ effective diameter
$P$ pitch of the thread

$f_{u,SG}$ tensile strength of the material of the screw
$\alpha_{s,SG}$ ratio of shear to tensile strength
$F_{As,SG}$ Load-bearing capacity of the thread of the screw
$F_{As,MG}$ Load-bearing capacity of the inside thread in the supporting structure
$R(A_{MG,SG})$ Reduction divisor for combined failure in both threads

$t_1$ is the smaller of the thickness of the timber side member or the
penetration depth
\( n_{\text{ef}} \) effective number of fasteners
\( l_{\text{ef}} \) effective length, penetration length of the thread, \( l_{\text{ef}} = l_g - l_b \)
\( l_g \) screw-in length - part of thread into component II including length of drill point
\( l_b \) length of unthreaded part of the drill point

\( M_{y,Rk} \) characteristic yield moment of fastener
\( F_{ax,Rk} \) characteristic axial withdrawal capacity of the fastener
\( F_{v,Rk} \) characteristic shear capacity of the fastener

\( f_{ax,k} \) characteristic withdrawal strength perpendicular to the direction of grain
\( f_{h,k} \) characteristic embedment strength of timber member
\( \alpha \) angle between the force and the direction of grain
\( \rho_k \) characteristic density
\( \rho_a \) associated density related to \( f_{ax,k} \), usually \( \rho_a = 350 \text{ kg/m}^3 \) for timber of strength grade C24 as reference density
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IX | European Recommendations for the Design, Detailing and Application of Fastenings for Sandwich Panels

CONTENTS
1 INTRODUCTION AND APPLICATION RANGE

Sandwich panels for building applications are usually fastened to a supporting structure made of steel, aluminium, timber (including wood-based materials), each of different cross-sectional shapes and manufacturing processes. For fastening to concrete cast-in steel channels are often used. As a consequence, self-drilling screws and self-tapping screws are the preferred fasteners for fastening of sandwich panels to the supporting structure. Fasteners such as blind rivets or cartridge-fired pins are not covered by this document. Direct fastening to concrete is done using concrete screws or spikes driven into predrilled holes. In any case, sealing washers are used to increase resistance and to obtain tight fastenings.

For non-structural components fastened to the faces of sandwich panels see the "European Recommendations for the Design of Sandwich Panels with Point or Line Loads".

The type and material of the fastener and the washer have to be chosen depending on application, material of the supporting structure and corrosivity category of the surrounding environment. As fasteners and washers are usually exposed to weather and other environmental conditions, fasteners and washers are usually made of stainless steel: This document will start with some basic information on durability of different materials for fasteners.

Only then the basics of design of fasteners are introduced. Design values of resistance may be obtained from tests or calculated using design formulae. To allow for general application of this introductory chapter, source of applied design values of resistance is left open here.

These values and their sources are introduced in two following chapters, distinguishing between values based on tests and values determined by calculation.

Finally, information on performance of fasteners, on execution and maintenance is given, to allow for proper installation guaranteeing safety in accordance to either European or national provisions for load-bearing capacity but also serviceability.

---

1 The presentation of the supporting structures both in the text or figures must inevitably be limited to exemplary cross-sectional shapes and manufacturing processes. However, unless explicitly mentioned in individual cases, this does not result in any general limitation with regard to the scope of application of these recommendations.
1 INTRODUCTION AND APPLICATION RANGE
### 2 DEFINITIONS

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<th>Description</th>
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<td>connecting element in a fastening</td>
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<tr>
<td>fastening</td>
<td>interaction of fastener with surrounding material</td>
</tr>
<tr>
<td>connection</td>
<td>group (one or more) of fasteners</td>
</tr>
<tr>
<td>component I</td>
<td>the component which is next to the head of the fastening screw, usually the sandwich panel to be fastened</td>
</tr>
<tr>
<td>component II</td>
<td>the component which is opposite to the head of the fastening screw, usually the supporting structure</td>
</tr>
<tr>
<td>direct fastening</td>
<td>fastening of sandwich panels which penetrates both faces of the panel (through fastening). The fasteners head is visible. Direct fastenings are sometimes referred to as visible fastenings.</td>
</tr>
<tr>
<td>hidden fastening</td>
<td>fastening of sandwich panels along their longitudinal joints, taking advantage of the geometry of the joint. The fastening is not visible. If no adapter plates are used (through fastening), the fastener is penetrating the panel sometimes also using a steel plate under the head of the fastener for increase of pull-through resistance. If adapter plates are used, the fastener is referred to as indirect fastening. Hidden fastenings are often referred to as concealed fastenings.</td>
</tr>
<tr>
<td>indirect fastening</td>
<td>fastening using special adapter plates, screw is not penetrating the panel, usually a hidden fastening, i.e. fastening is hidden in the longitudinal joint between adjacent panels.</td>
</tr>
<tr>
<td>sealing washer</td>
<td>annular washer with vulcanized sealing usually made of EPDM</td>
</tr>
<tr>
<td>saddle washer</td>
<td>prismatic washer for crest-fastening of panels with profiled faces. The width corresponds to the width of the rib of the external face. Saddle washers are usually provided with a rubber foam sealing.</td>
</tr>
<tr>
<td>self-drilling screw</td>
<td>screws with a drill-point so that the drilling of the hole, the tapping of the female and screwing-in is done in one operating sequence</td>
</tr>
<tr>
<td>self-tapping screw</td>
<td>screw which taps the female thread in an existing hole i.e. self-tapping screws require pre-drilling.</td>
</tr>
<tr>
<td>concrete screw</td>
<td>hardened screw for fastening to concrete structures, often double-threaded, taps a thread into the concrete</td>
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2 DEFINITIONS

concrete nail (no plug), i.e. requires pre-drilling.
crooked nail for fastening to concrete structures, mounted by hammering, requires pre-drilling.
3 MATERIALS AND DURABILITY

3.1 Introduction to materials

3.1.1 Carbon steel
Carbon steel screws are usually made of case-hardened steel or of heat-treatable steel. Because of heat treatment, carbon steel fasteners provide a high strength and hardness. The high hardness values are accompanied by the risk of hydrogen embrittlement.

For carbon steel screws corrosion protection is required. Most common type of corrosion protection is done by galvanic zinc plating according to EN ISO 4042. It should be highlighted that fasteners listed in a European technical approval usually have a zinc coating specified as A3K which means that the fasteners are zinc-plated with a coating thickness ≥ 8 µm. Care should be taken that a lower coating thickness shall not be used, see EN 1993-1-3, Annex B.

In addition, zinc flake coating systems are used. These coatings have an organic or inorganic matrix with dispersed zinc- and aluminium particles. Unfortunately these coatings may be damaged during transport or by abrasion during installation, if not even scraped off. Therefore an assessment of such coatings is difficult but nevertheless necessary for verification of suitability of a coating for a corrosivity category (see EN 1993-1-3, Annex B for corrosivity category of environment).

3.1.2 Stainless steel
3.1.2.1 Preliminary remarks
Stainless steels for screws can be classified according to the system given in EN ISO 3506-4. The mechanical properties which can be achieved depend on the steel group (austenitic, martensitic or ferritic stainless steel). Stainless steels obtain their corrosion resistance by a passive layer of chromium oxide.

3.1.2.2 Austenitic stainless steel
Designations such as the steel grades A2 (e.g. 1.4301 or 1.4567) and A4 (e.g. 1.4401 or 1.4578) for austenitic stainless steels are quite familiar and refer predominantly to the corrosion resistance. Austenitic stainless steels cannot be hardened by heat-treatment but by cold working. Increasing of surface hardness by nitriding is also possible. If self-drilling screws for drilling into steel are required a drill-point made of hardened carbon steel has to be welded to the screw’s tip. After installation just the stainless steel part of the screw will be a part of the load-bearing system and not the welded on drill-point.

3.1.2.3 Martensitic stainless steel
Martensitic stainless steels can be heat treated which allows producing self-drilling
screws in one part with a drill-point which has not to be welded on. Corrosion resistance is considerably lower than for austenitic stainless steel and high hardness after heat treatment makes them vulnerable to hydrogen embrittlement and stress corrosion cracking.

3.2 Choice of materials with respect to corrosion

Choice of material has to be made depending on the material of the structural parts to be connected, taking into account both corrosivity of the surrounding and the intended service life. Most important factor is corrosion which is also influenced by the material of the parts to be connected. It has to be mentioned that the high strength of the fasteners does not have a significant effect on the resistance of the connection, because with thin-walled sheeting and sections, failure of the building components to be connected is the governing parameter. Stressing by forces therefore normally does not become decisive for the choice of material for the fasteners.

Corrosivity of the surrounding depends on moisture conditions, air pollution (dust which may dissolve in water, chloride in industrial and marine environment or from road de-icing salts, sulphur dioxide exhausted from power plants and traffic etc.) and time of exposure. Moisture may have access to the screws by weather, but also by condensation at thermal bridges. Some thermal insulation materials such as mineral wool are able to work like a sponge, absorbing water. If screws are installed through a sandwich panel with soaked mineral wool, corrosion is directly affecting the load-bearing part of the screw. It is also important if conditions may get worse by accumulation of corrosive agents. Typical example is the accumulation of both corrosive agents such as de-icing salt and moisture behind external walls ventilated at rear. On the other hand, occasional rain may even have a cleaning effect on the screw. So detailing is also a very important consideration.

The following recommendations are backed by a long-time experience:

- Screws completely or partly exposed to weather or comparable moisture conditions shall be made of austenitic stainless steel. This does not refer to welded drill points, but it has to be checked that the screw-in length is big enough so that carbon steel parts are not part of the load-bearing system.
- The period of construction shall be taken into account, also with respect of time of construction.
- In applications, where concentrations of corrosive agents may accumulate or surroundings with higher corrosivity, austenitic stainless steels of higher grades (for example A4) are necessary. This may be the case with screws
in the cavity or clear space of external walls ventilated at rear or otherwise shielded from direct rain and a regular cleaning is not foreseen or not possible. A good assistance for choosing the right stainless steel grade regarding corrosion resistance for fastening elements offers EN 1993-1-4/A1, Annex A.

- Screws made of carbon steel or martensitic stainless steel are not suitable in case that minimum requirements on corrosion resistance exist, see EN 1993-1-3, Annex B. Carbon steel screws, including electrolytically galvanized or coated fasteners, and screws made of martensitic stainless steel shall only be used where moisture does not affect them. This covers:
  - fastening of internal shells of multi-shell roof and wall structures (decking profiles or cassettes) of dry and predominantly closed rooms, provided the external shell prevents entry and accumulation of corrosive agents and rain (external shell made of sheeting).
  - fastening of decking profiles of unventilated single shell roofs of dry and predominantly closed rooms with insulation on the external side (typical flat roof applications with insulation membranes)
  - ceiling systems over dry and predominantly closed rooms
- Fastening of aluminium sheeting shall only be done with screws made of stainless steel except in heavy maritime environments. In this case additional protection measures of the connections are required.
- Galvanic corrosion shall be taken into account when designing and detailing structures, see EN 1090-4, Annex E or EN 1090-5, Annex E.
- While repeated bending of fasteners due to thermal elongation and movement usually becomes not decisive for design of austenitic stainless steel fasteners, care should be taken when using hardened martensitic or carbon steel screws.
3 MATERIALS AND DURABILITY
4 DESIGN OF FASTENINGS

4.1 Actions and loads

Shear forces in fasteners are mainly caused by self-weight loading\(^2\) of the panel or roof thrust caused by self-weight or snow (Fig. 4.1). Other possible sources are forces from lateral stabilisation of the supporting beams, purlins or rails (see ECCS publication no. 135/CIB publication no. 379 (2014)), diaphragm action (including seismic actions) or forces from restraint thermal movements. The design value of the shear force caused by action effects is denoted with \(V_{Ed}\). Determination of action effects is based on the different parts of EN 1991. For a complete overview on actions acting on sandwich panels see ECCS publication no. 136/CIB publication no. 401 (2015).

Shear loads in connections are predominantly transferred by the internal face of the panel (\(t_{F2}\)), therefore failure occurs by elongation of the hole in the internal face. Tilting of the fastener is prevented by sandwich panel itself. With the usual sheet thicknesses of sandwich panels, shear failure of the fastener itself rarely occurs.

Fig. 4.1: Shear forces in fasteners caused by self-weight and roof thrust

Tensile forces in fasteners are predominantly caused by wind suction (Fig. 4.2) or

\(^2\) Self-weight loading of wall panels may also be transferred via support angles at base points or above large openings by reinforced girts.
restraint thermal deformation (Fig. 4.3). Other possible sources for tensile forces are forces from torsional stabilisation of the supporting beams, purlins or rails, see ECCS publication no. 135/CIB publication no. 379 (2014) or inertia forces caused by seismic actions. The design value of the tensile force caused by action effects is denoted with $N_{Ed}$. Determination of action effects is based on the different parts of EN 1991. For normal applications increased action effects from wind gusts need not to be taken into account when calculating the action effects. These are usually considered when determining the resistance values and therefore covered by the resistance values, see ECCS publication no. 127/CIB publication no. 320 (2009). EN 14509 gives additional information on temperature loads. For a complete overview on actions acting on sandwich panels see ECCS publication no. 136/CIB publication no. 401 (2015).

Possible failure modes are pull-out failure from the supporting structure ($t_{ll}$) or pull-over/pull-through failure through the external face of the panel ($t_{F1}$). With the usual sheet thicknesses of sandwich panels, tensile failure of the fastener itself rarely occurs.

![Diagram of tensile forces in fasteners caused by wind]

Fig. 4.2: Tensile forces in fasteners caused by wind
Temperature differences $\Delta T$ of the surface temperatures of the internal and external face (Fig. 4.4) cause thermal movement of the panel. This includes shear deformations and rotation at supports. As temperature of the external face changes from night to day, temperature differences do so as well. The repeated changes lead to repeated deformations and rotations at supports causing a repeated movement of the fasteners (head displacement $u$). The fastening is therefore prone to fatigue failure. Although the head deflection corresponds to a cyclic load, the action effect is usually denoted with $u$ and calculated using a partial safety factor of $\gamma_F = 1.0$. EN 14509 gives additional information on temperature loads. For a complete overview on actions and loads acting on sandwich panels, see ECCS publication no. 136/CIB publication no. 401 (2015).

The load spectrum caused by the change of temperature difference and the number of cycles are taken into account when determining the resistance values and are therefore covered by the resistance values, see ECCS publication no. 127/CIB publication no. 320 (2009).

Fatigue failure can be observed as unwinding from the supporting structure ($t_{u1}$), reduced pull-out resistance from the supporting structure ($t_{u2}$) or cracking of the fastener itself.
Exposure of roof panels to high snow loads might lead to a transverse compression deformation (shortening) of the core material at supports. This might cause a gap between the external face and the washer or an improperly compressed sealing, both causing leakage. Roof panels with indirect fastenings or wall panels in general are not affected. Some national regulations and/or customers require checking the value of transverse compression deformation, for example for altitudes above 1000 m (see also EN 1991-1-3). The verifications may be done using the formulas and requirements given in Annex B. See RAGE (2014a) for further requirements and additional verifications.

The tensile and shear loads in fastenings shall be calculated for ultimate limit state (ULS) using the theory of elasticity. This approach is different to the one used in
the design of the panels itself, where plastic hinges at intermediate supports are usually assumed for ULS design (see EN 14509, chapter E.7.2.1). If this would be done in design of fastenings, tensile forces caused by restraint thermal deformation would be supressed. See references for detailed information on structural analysis of sandwich panels. For ULS, the combination according to Eq. (4.1) applies.

\[ E_d = \sum_{j=1}^{3} \gamma_{G,j} G_{k,j} + \gamma_{Q,k,1} Q_{k,1} + \sum_{j=1}^{3} \gamma_{Q,j} \psi_{0,j} Q_{k,i} \]

Eq. (4.1)

The value of head deflection shall be calculated for fatigue limit state (FLS) using the formulas given in Annex A and a characteristic value of temperature difference of \( \Delta T = T_2 - T_1 = -70 \) K only. This value is based on an assumed installation temperature of 10° C and a temperature of the outer face of 80° C. The later assumption is conservative and has become firmly established as good practice. The value of transverse compression deformation of the core at supports shall be calculated for serviceability limit state (SLS) using the theory of elasticity for calculation of support reactions and by means of the compressive modulus \( E_{Cc} \) according to EN 14509. For SLS, the characteristic (rare) combination according to Eq. (4.2) applies.

\[ E_d = \sum_{j=1}^{3} G_{k,j} + Q_{k,1} \]

Eq. (4.2)

where \( Q_{k,1} \) is the characteristic value of actions caused by snow load. For the rare combination, creep effects do not need to be considered.

### 4.2 Resistance of fastenings

The characteristic resistance values are denoted as follows:

- \( N_{Rk,I} \): characteristic value of pull-through resistance
- \( N_{Rk,II} \): characteristic value of pull-out resistance
- \( V_{Rk,I} \): characteristic value of shear resistance of the internal face
- \( V_{Rk,II} \): characteristic value of shear resistance of the supporting structure
- \( u_{Rk} \): characteristic value of allowable head deflection
- \( w_{S,Ck} \): characteristic value of allowable transverse compression deformation
The characteristic values of tensile and shear resistance are defined as

\[ N_{R_k} = \min \{ N_{R_k, I}, N_{R_k, II} \} \]

Eq. (4.3)

and

\[ V_{R_k} = \min \{ V_{R_k, I}, V_{R_k, II} \} \]

Eq. (4.4)

respectively. Failure of the fastener itself is usually covered by the resistance related to component II, however it will not become decisive with usual applications in sandwich panels. The resistance values can be determined by tests (see chapter 5) or by calculation (see chapter 6 and table 4.1).

<table>
<thead>
<tr>
<th>Resistance</th>
<th>Component</th>
<th>Material of supporting structure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
<td>Steel</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>6.2.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Aluminium</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>6.2.2</td>
</tr>
<tr>
<td></td>
<td>I</td>
<td>Timber</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>6.2.2</td>
</tr>
</tbody>
</table>

max. \( u \) 6.5.2

max. \( w_s \) c)

a) Determination of resistance of fastenings of sandwich panels on aluminium supporting structures has to be based on tests.
b) Within the application range given in chapter 6.4.2, no further verification of the characteristic shear resistance of the supporting structure has to be done.
c) The value of transverse compression deformation is usually limited to \( w_{S,Ck} = 1 \) mm.

The characteristic values of resistance depend on spacing, edge distance and arrangement of the fasteners and have to be checked against the test provisions or application range of calculation formulas.
4.3 Design of fastenings

Partial safety factors for resistance values apply in design. The design values of resistance are calculated by dividing the characteristic values with the partial safety factor $\gamma_M$:

$$N_{Rd.I} = \frac{N_{Rk.I}}{\gamma_M}$$
Eq. (4.5)

$$N_{Rd.II} = \frac{N_{Rk.II}}{\gamma_M}$$
Eq. (4.6)

$$V_{Rd.I} = \frac{V_{Rk.I}}{\gamma_M}$$
Eq. (4.7)

$$V_{Rd.II} = \frac{V_{Rk.II}}{\gamma_M}$$
Eq. (4.8)

$$\max u \equiv u_{Rd} = \frac{u_{Rk}}{\gamma_{M,I}}$$
Eq. (4.8)

$$\max w_S \equiv w_{S,cd} = \frac{w_{S,ck}}{\gamma_{M,ser}}$$
Eq. (4.8)

Usually, a partial safety factor of $\gamma_M = 1.33$ applies for ultimate limit state design of fastenings of sandwich panels, but deviating values may be given in some approvals or in some national specifications. EN 1995-1-1 gives $\gamma_M = 1.3$ for timber supporting structures, but since differences are negligible, $\gamma_M = 1.33$ is applied uniformly for ultimate limit state design.

For serviceability limit state and fatigue limit state, $\gamma_{M,ser} = 1.00$ and $\gamma_{M,I} = 1.00$ apply. As partial factors are set to unity, the designation "max u" and "max ws" usually apply.

The following proofs have to be made:
4 DESIGN OF FASTENINGS

\[
\frac{N_{Ed}}{N_{Ed, I}} \leq 1.0
\]
Eq. (4.9)

\[
\frac{N_{Ed}}{N_{Ed, II}} \leq 1.0
\]
Eq. (4.10)

\[
\frac{V_{Ed}}{V_{Ed, I}} \leq 1.0
\]
Eq. (4.11)

\[
\frac{V_{Ed}}{V_{Ed, II}} \leq 1.0
\]
Eq. (4.12)

\[
\frac{u}{\max u} \leq 1.0
\]
Eq. (4.13)

\[
\frac{w_s}{\max w_s} \leq 1.0
\]
Eq. (4.14)

In case of repeated shear forces with load reversal (shear forces with changing direction), interaction has to be checked if \( V_{Ed} \geq 0.25 \cdot V_{Rd, I} \) using

\[
0.75 \cdot \frac{N_{Ed}}{N_{Rd, I}} + \frac{V_{Ed}}{V_{Rd, I}} \leq 1.0
\]
Eq. (4.15)

Deviating interaction approaches may be used if proven by tests. See also Kilian et al. (2015) for enhanced interaction approaches. Care should be taken in cases were equivalent static forces are used to design for effects actually associated with a load reversal (e.g. wind loads or seismic loads in structures with sandwich panels serving as bracing). In these cases, interaction should be checked using Eq. (4.15).
In any case, for the supporting structure (component II) the interaction equation

\[ \frac{N_{Ed}}{N_{Rd,II}} + \frac{V_{Ed}}{V_{Rd,II}} \leq 1.0 \]

Eq. (4.16)

applies. For timber supporting structures EN 1995-1-1 Eq. (8.28) leads

\[ \left( \frac{N_{Ed}}{N_{Rd,II}} \right)^2 + \left( \frac{V_{Ed}}{V_{Rd,II}} \right)^2 \leq 1.0 \]

Eq. (4.17)

However, application of Eq. (4.16) is generally not covered by the ETAs of the fasteners.
4 DESIGN OF FASTENINGS
5 RESISTANCE DETERMINED BY TESTING

5.1 General

Performance and evaluation of tests is described in ECCS publication no. 127/CIB publication no. 320 (2009), CUAP (2010) and prEN 14509-2 (2017). In case resistance values are based on tests, they are often published in approvals. These are either national approvals, European Technical Approvals or European Technical Assessments. They are usually published on request of a fasteners manufacturer, but sometimes also on request of a manufacturer of sandwich panels, taking into account the effects of geometry and material properties of the panel on the pull-through resistance of the fastening, something which is particularly relevant for indirect fastenings. The latter also applies for resistance values based on tests according to prEN 14509-2 (2017). They are essential characteristics declared by the manufacturer of the sandwich panel with the resistance value specified in the CE-mark and Declaration of Performance (DoP).

5.2 Design based on an approval for fasteners

5.2.1 Sandwich panels on steel or aluminium supporting structures

Design of fasteners for sandwich panels on steel or aluminium supporting structures is based on tabulated values. Applicability of design tables depends on material properties (steel grade for example) of components I and II. Nominal sheet thicknesses are input values for determining the resistance values: Shear resistance is given depending on the nominal thickness $t_{\text{nom,II}}$ of the supporting structure and nominal thickness $t_{\text{nom,F2}}$ of the internal face. Tensile resistance is given depending on the nominal thickness $t_{\text{nom,II}}$ of the supporting structure and nominal thickness $t_{\text{nom,F1}}$ of the external face. Values for tensile resistance are not applicable for hidden fastenings, but pull-out resistance may be deduced from the tables (see chapter 5.3). In addition to division by a safety factor, no further calculation procedures are necessary.

Allowable head deflection is given depending on the nominal thickness $t_{\text{nom,II}}$ of the supporting structure and distance $D_F$ between top of supporting structure and top of external face at point of fastening. Fig. 5.1 and 5.2 show examples of European Technical Approvals for fasteners for fastening sandwich panels on steel supporting structures.
### 5 RESISTANCE DETERMINED BY TESTING

**Fig. 5.1:** Example of a table from an ETA - steel supporting structure

<table>
<thead>
<tr>
<th>( t_{\text{rec}, i} ), ( t_{\text{con}} ), ( D_r ) [mm]</th>
<th>4,00</th>
<th>5,00</th>
<th>6,00</th>
<th>8,00</th>
<th>10,00</th>
<th>12,00</th>
<th>14,00</th>
<th>16,00</th>
<th>18,00</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0,88</td>
<td>0,98</td>
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</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>( \sigma_{\text{ld}} + t_{\text{u}} ) [mm]</th>
<th>4,00</th>
<th>5,00</th>
<th>6,00</th>
<th>8,00</th>
<th>10,00</th>
<th>12,00</th>
<th>14,00</th>
<th>16,00</th>
<th>18,00</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0,88</td>
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<td>3,10</td>
<td>3,10</td>
<td>3,10</td>
<td>3,10</td>
</tr>
</tbody>
</table>

**Drilling capacity:** \( \Sigma(\sigma_{\text{ld}} + t_{\text{u}}) \leq 12,0 \text{ mm} \)

**Timber supporting structure:**

No performance determined

---

**Fig. 5.1:** Example of a table from an ETA - steel supporting structure

- **Materials:**
  - **Fastener:** stainless steel (1.4301) – EN 10088
  - stainless steel (1.4401) – EN 10088
  - **Washer:** stainless steel (1.4301) – EN 10088
  - **Component I:** S320GD or S350GD – EN 10346
  - **Component II:** C235, C275 or C355 – EN 10025-1

---

**Self drilling screw**

**Fastener Type 5,5 x L**

with sealing washer \( \geq 0,16 \text{ mm} \)

**Annex xx**
# 5 RESISTANCE DETERMINED BY TESTING

**Fig. 5.2: Example of a table from an ETA - steel supporting structure**

<table>
<thead>
<tr>
<th>( d_{sh} ) [mm]</th>
<th>( \sigma_0 ) [MPa]</th>
<th>( \sigma_0 ) [MPa]</th>
<th>( \sigma_0 ) [MPa]</th>
<th>( \sigma_0 ) [MPa]</th>
<th>( \sigma_0 ) [MPa]</th>
<th>( \sigma_0 ) [MPa]</th>
<th>( \sigma_0 ) [MPa]</th>
<th>( \sigma_0 ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.40</td>
<td>0.98</td>
<td>0.98</td>
<td>1.00</td>
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<td>0.98</td>
<td>0.98</td>
<td>0.88</td>
<td>0.88</td>
</tr>
<tr>
<td>0.50</td>
<td>0.98</td>
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<tr>
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<td>0.98</td>
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<td>3.10</td>
<td>3.10</td>
<td>3.10</td>
<td>3.10</td>
</tr>
</tbody>
</table>

**Materials:**
- Fastener: stainless steel (1.4301) – EN 10088
  - stainless steel (1.4401) – EN 10088
- Washer: stainless steel (1.4301) – EN 10088
- Component I: S320GD or 8350GD – EN 10346
- Component II: Ø236, Ø275 or Ø365 – CN 10025-1

**Pre-drilling diameter:** see table below

**Timber supporting structures:**
- No performance determined

---

**Self tapping screw**

<table>
<thead>
<tr>
<th>( M_{s, req} ) [Nmm]</th>
<th>Fastener Type 6.3 x L</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>with sealing washer ( \geq ) Ø19 mm</td>
</tr>
</tbody>
</table>

---

Annex xx
5.2.2 Sandwich panels on timber supporting structures

Design of fasteners for sandwich panels on timber supporting structures may be based on tabulated values, but is often amended by design procedures given in EN 1995-1-1, see chapter 6 of this recommendation.

In the latter case, input parameters such as withdrawal strength $f_{ax,k}$ and characteristic yield moment of fastener $M_{y,Rk}$ are given in the approval.

Tables contain of evaluated design equations, providing resistance values for different screw-in lengths for fixed materials and modification factors. Applicability of design tables depends on material properties (steel grade for example) of component I and modification factors of at least the factor given. Tables issued for structural timber according EN 14081 may also be used for glued laminated timber and glued solid timber according to EN 14080.

Nominal sheet thicknesses and screw-in length are input values for determining the resistance values: Shear resistance is given depending on the screw-in length and nominal thickness $t_{nom,F2}$ of the internal face. Tensile resistance is given depending on screw-in length and nominal thickness $t_{nom,F1}$ of the external face. Care should be taken as screw-in length is sometimes given as effective screw-in length $l_{eff}$ or screw-in length including drill point $l_g = l_{eff} + l_b$. Applied modification factor $k_{mod}$ is usually given. Beside division by a safety factor, no further calculation procedures are necessary.

Allowable head deflection is given depending on distance $D_F$ between top of supporting structure and top of external face at point of fastening. Fig. 5.3 and 5.4 show examples of European Technical Approvals for fasteners for fastening sandwich panels on timber supporting structures.
### Table: Example of a table from an ETA - timber supporting structure

<table>
<thead>
<tr>
<th>max ( a ) [mm]</th>
<th>( N_{ax} ) [kN]</th>
<th>( V_{ax} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>0.50</td>
<td>0.55</td>
</tr>
<tr>
<td>60</td>
<td>0.55</td>
<td>0.60</td>
</tr>
<tr>
<td>80</td>
<td>0.60</td>
<td>0.65</td>
</tr>
</tbody>
</table>

#### Notes:
- The values indicated above depend on the screw depth 4, which shall apply for: 4 = 0.60 and the timber strength class C24 (4).
- Drilling capacity: 24 ± 2.0 mm
- Timber supporting structure: Component 1: structural timber - EN 4001
- Component 2: stainless steel - EN 10088

#### Materials:
- Fastener: stainless steel 1.4017 - EN 10088
- Washer: EN 10088

---

**Fig. 5.3:** Example of a table from an ETA - timber supporting structure
### RESISTANCE DETERMINED BY TESTING

**Materials:**
- Fastener: stainless steel (1.4301) – EN 10088
- Stainless steel (1.4401) – EN 10088
- Washer: stainless steel (1.4301) – EN 10088
- Component I: S280GD, S320GD or S350GD – EN 10346
- Component II: structural timber – EN 14001

**Pre-drilling diameter** 4.5 mm

Timber supporting structures:
- performance determined with
  
  \[ M_{p,\text{req}} = 10,744 \text{ Nm} \]
  
  \[ f_{\text{m},k} = 11,080 \text{ N/mm}^2 \]

<table>
<thead>
<tr>
<th>( h_{\text{act},1,1} ), ( f_{\text{m},k} ), ( D_0 ) [mm]</th>
<th>45</th>
<th>48</th>
<th>51</th>
<th>54</th>
<th>57</th>
<th>60</th>
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<td>2.31</td>
<td>2.31</td>
<td>2.31</td>
<td></td>
</tr>
</tbody>
</table>

The values indicated above, depending on the screw depth \( h_0 \), shall apply for \( k_{\text{act}} = 0.90 \) and the timber strength class C24 (\( \rho_x = 350 \text{ kg/m}^3 \)). For other values of \( k_{\text{act}} \) and timber strength classes see section 4.5.2.

Pre-drill diameter is \( d_{\text{pd}} = 4 \text{ mm} \)

**Self tapping screw**

- Fastener type 6.5 x L
- with sealing washer \( \varnothing 16 \text{ mm} \)

**Annex xx**

Fig. 5.4: Example of a table from an ETA - timber supporting structure
5.3 Design based on an approval for sandwich panels or EN 14509-2

Fastenings of sandwich panels are not yet covered by current version of EN 14509, therefore resistance values depending on geometry and material properties of the panels (for example pull-through resistance of hidden fastenings) cannot be specified in the CE-mark or Declaration of Performance (DoP), but need to be specified elsewhere. This is often done in a national approval. With publication of EN 14509-2, situation will change and resistance values will be specified in the CE-mark and DoP. Nevertheless, in some cases, only a technical documentation published by the panel manufacturer exists. The manufacturer has to give information about the boundaries and parameters applied during the tests and to confirm that the evaluation of the test corresponds to the state of the art. The designer has to reconfirm themselves that the technical documentation resembles the state of the art and that the parameters of the structure lay within the boundaries described in the documentation. Especially it must be checked if the values provided are characteristic values or design values of resistance or even allowable values according to a deterministic safety concept.

The designer may take the pull-through resistance provided by the approval for the sandwich panel (see Figs. 5.5. and 5.6 as examples). The pull-out resistance may be taken from the approval of the fastener (see chapter 5.2) or be determined by calculation (see chapter 6.2). In the first mentioned case, where tables give tensile resistance depending on both nominal thickness of the external face and nominal thickness of the supporting structure, the pull-out resistance may be estimated as the tensile resistance assuming maximum thickness of the external face.

The smaller value of both pull-through-resistance and pull-out resistance is the tensile resistance of the fastening.
5 RESISTANCE DETERMINED BY TESTING

Hidden fastenings

<table>
<thead>
<tr>
<th>Type of fastening</th>
<th>Nominal thickness of the core (mm)</th>
<th>Support</th>
<th>( N_{up} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sealing washer</td>
<td>60</td>
<td>End support</td>
<td>3.78</td>
</tr>
<tr>
<td></td>
<td>140</td>
<td>Internal support</td>
<td>3.78</td>
</tr>
<tr>
<td>Load spreader plate</td>
<td>60</td>
<td>End support</td>
<td>2.13</td>
</tr>
<tr>
<td></td>
<td>140</td>
<td>Internal support</td>
<td>4.14</td>
</tr>
</tbody>
</table>

The values given above apply for the introduction of the tensile forces into the fastener (pull-through resistance). Introduction of forces into the supporting structure has to be checked, too.

Sandwich panels according to EN 14509 with steel faces and polyurethane core

Sandwich wall panels – concealed fixings
with sealing washer ≥ Ø 19 mm

Annex xx

Fig. 5.5: Example of an annex of a national approval for a sandwich panel
Fig. 5.6: Example of a CE-mark (preliminary draft)
6 RESISTANCE DETERMINED BY CALCULATION
6 RESISTANCE DETERMINED BY CALCULATION

6.1 Application range

Although the equations given in this chapter allow for calculation of characteristic values of resistance, they are also based on tests. These equations were derived by evaluating large numbers of tests. As these tests cover only a limited application range, application of the equations is limited to this specific range, usually expressed by material properties of the sandwich panel, the supporting structure or fastener and washer, but also spacing and edge distance. For applications outside the specified scope, design shall be based on tests unless otherwise specified in the following.

6.2 Pull-through resistance

6.2.1 Background

First investigations on pull-through resistance of fastenings for sandwich panels were done by Hassinen (2005), restricted to sandwich panels with flat or lightly profiled faces and a mineral wool core. The design approach given here for direct fastening of sandwich panels with flat or lightly profiled faces was published by Hassinen & Misiek (2009). This approach was amended for crest-fastening without saddle washers of panels with profiled faces by Misiek & Ummenhofer (2012). Latest research by Ungermann & Lübke (2012) led to more sophisticated approaches for panels with flat or lightly profiled faces, but generally confirmed the approaches given below.

6.2.2 Design procedures for sandwich panels with steel faces

The characteristic value of the pull-through resistance of fastenings of sandwich panels with flat or lightly profiled faces can be calculated using

\[
N_{Rk,J} = F_{C,k} + F_{F,k} = 2.21 \cdot \sqrt{E_{Cc} \cdot f_{Cc} \cdot d_w^2} + 0.65 \cdot t_{F1} \cdot f_{u,F1} \cdot d_w
\]

Eq. (6.1)

with
- \(E_{Cc}\) compression modulus of the core
- \(f_{Cc}\) compression strength of the core
- \(d_w\) diameter of the washer
- \(t_{F1}\) design core sheet thickness of the external face
- \(f_{u,F1}\) tensile strength of the external face
The application range of Eq. (6.1) is limited:
- direct fastening;
- external faces made of steel;
- mineral wool, polystyrene, polyurethane or polyisocyanurate core material;
- panel thickness \( d_C \geq 40 \text{ mm} \);
- \( 2.0 \text{ N/mm}^2 \leq E_{Cc} \leq 8.0 \text{ N/mm}^2 \) compression modulus of the core. Values \( E_{Cc} > 8.0 \text{ N/mm}^2 \) should not be taken into account;
- \( 0.1 \text{ N/mm}^2 \leq f_{Cc} \leq 0.3 \text{ N/mm}^2 \) compression strength of the core. Values \( f_{Cc} > 0.3 \text{ N/mm}^2 \) should not be taken into account;
- \( 0.40 \text{ mm} \leq t_{F1} \leq 0.80 \text{ mm} \) design core sheet thickness of the face (steel). Values \( t_{F1} > 0.80 \text{ mm} \) should not be taken into account;
- \( 360 \text{ N/mm}^2 \leq f_{u,F1} \leq 520 \text{ N/mm}^2 \) tensile strength of the face (steel). Values \( f_{u,F1} > 520 \text{ N/mm}^2 \) should not be taken into account;
- self-tapping screws and self-drilling screws made of steel or stainless steel;
- diameter of the screws \( 5.5 \text{ mm} \leq d \leq 8.0 \text{ mm} \);
- diameter of the head \( \geq 8.0 \text{ mm} \);
- washers made of austenitic stainless steel
- diameter of the washer \( d_W \geq 11 \text{ mm} \);
- thickness of the metallic part the washer \( t_w \geq 1.0 \text{ mm} \), thickness of the cured-on elastomer seal \( \geq 2.0 \text{ mm} \)
- pull-through failure of the screw head through the washer itself as this could be observed for \( d_W \geq 29 \text{ mm} \) has to be excluded;

The simulated central support test according to EN 14509 uses the design resistance \( N_{R,k,I} \) according to EN14509. Therefore the use of \( N_{R,k,I} \) according to Eq. (6.1) might lead to a reduced wrinkling stress at the central support.

This approach does not cover reduction of resistance due to repeated loading as it can be neglected for smaller diameters of washers. As a simplified approach, resistance should be reduced to 75% in cases where first summand gives more than 20% of the total resistance calculated (i.e if \( F_{C,k} > 0.25 F_{F,k} \)).

For crest-fastening of panels with profiled faces, the equation

\[
N_{Rr,I} = \left( 2.21 \cdot \sqrt{E_{Cc} \cdot f_{Cc} \cdot d_W^2} + 0.65 \cdot t_{F1} \cdot f_{u,F1} \cdot d_W \right) \left( 0.55 + 0.45 \cdot e^{-\frac{d_w}{\pi R}} \right)
\]

Eq. (6.2)
applies with

\( b_1 \) width of a rib, see Fig. 6.1

and the application range as given with Eq. (6.1), but only polyurethane or polyisocyanurate core material with auto-adhesive bond.

Fig. 6.1: Definitions of geometry according to EN 14509

6.3 Pull-out resistance

6.3.1 Background

As there is no difference in failure modes and resistance, design procedures for calculation of pull-out resistance given in EN 1993-1-3 (2006) and EN 1999-1-4 (2007) do also apply for fastenings of sandwich panels. Latest design procedure for supporting structures made of steel or aluminium was developed by Hettmann (2008). The procedure is related to the one used for metric screws and nuts, but taking into account the local deformation behaviour of the supporting structure, usually also consisting of thin sheets. Compared to the aforementioned equations, this approach leads to both higher and more realistic results, but requires much more effort.

For supporting structures made of timber, EN 1995-1-1 applies. The design procedures given there require pull-out parameters determined by tests, but may be also estimated by design equations.

6.3.2 Design procedures for steel and aluminium supporting structures

The characteristic value of pull-out resistance is given by

\[
N_{R_{k,II}} = 0.80 \times \min \left[ \frac{F_{As,SG}}{R(V_{MG,SG})}, \frac{F_{As, MG}}{R(V_{MG,SG})} \right]
\]

Eq. (6.3)
6 RESISTANCE DETERMINED BY CALCULATION

with

\( F_{As,SG} \) Load-bearing capacity of the thread of the screw

\( F_{As,MG} \) Load-bearing capacity of the inside thread in the supporting structure

\( R(V_{MG,SG}) \) Reduction divisor for combined failure in both threads taking into account failure of both threads.

The load-bearing capacity of the thread of the screw is calculated using Eq. (6.4).

\[
F_{As,SG} = 1.2 \cdot \alpha_{s,SG} \cdot f_{u,SG} \left[ 0.15 \text{mm} + \left( d - d_{pd} \right) \cdot \tan \left( \frac{\alpha}{2} \right) \right] \cdot d_{pd} \cdot \frac{t_{Il} \cdot \pi}{P}
\]

Eq. (6.4)

with

\( \alpha_{s,SG} \) ratio of shear to tensile strength (0.70 for austenitic stainless steels and 0.61 for low-alloy carbon steels)

\( f_{u,SG} \) tensile strength of the material of the screw (930 N/mm² for austenitic stainless steels and 1190 N/mm² for low-alloy carbon steels)

\( t_{Il} \) design core sheet thickness of the supporting structure.

and dimensions \( d \), \( d_{pd} \), \( P \) and angle \( \alpha \) according to Fig. 6.2.

\[ d_{pd} = \max \{ d_{dp}, d_1 \} \]

Eq. (6.5)

applies. The load-bearing capacity of the inside thread in the supporting structure is calculated using Eq. (6.6).
6 RESISTANCE DETERMINED BY CALCULATION

\[ F_{As, MG} = \begin{cases} 
1.1 \cdot F^{*}_{As, MG} & t_{II} < 0.60\text{mm} \\
\frac{a \cdot t_{II}^b}{1.1} & \text{for } 0.60\text{mm} \leq t_{II} \leq \max\{t^*; 4.90\text{mm}\} \\
1.1 \cdot F^{*}_{As, MG} & t_{II} > \max\{t^*; 4.90\text{mm}\}
\end{cases} \]

Eq. (6.6)

with

\[ t^* = \begin{cases} 
0.9 \cdot d_{pd} & d_{pd} \leq 4\text{mm} \\
d_{pd} & \text{for } 4\text{mm} < d_{pd} \leq 5\text{mm} \\
1.1 \cdot d_{pd} & d_{pd} > 5\text{mm}
\end{cases} \]

Eq. (6.7)

\[ F^{*}_{As, MG} = 0.60 \cdot f_{u, II} \cdot \frac{d}{2} \cdot \left[ \frac{\pi \cdot t_{II}^2 + t_{II} \cdot \varphi_{EA}}{P} \right] \]

\[ + \left( P - 0.15\text{mm} \right) \left( \frac{2\pi}{P} \cdot t_{II} - 2\pi + \varphi_e \right) \]

\[ + \varphi_{EA} \left( P - 0.15\text{mm} - \frac{P}{4\pi} \cdot \varphi_{EA} \right) \]

\[ \text{for } t_{II} \leq 0.60\text{mm} \]

\[ \text{for } t_{II} \geq \max\{t^*; 4.90\text{mm}\} \]

Eq. (6.8)

\[ a = \frac{0.95 \cdot F^{*}_{As, MG} (t_{II} = 0.60\text{mm})}{(0.60\text{mm})^b} \]

Eq. (6.9)

the exponent

\[ b = \ln \left( \frac{F^{*}_{As, MG} (t_{II} = t^*)}{0.95 \cdot F^{*}_{As, MG} (t_{II} = 0.60\text{mm})} \right) \]

\[ \ln \left( \frac{t^*}{0.60\text{mm}} \right) \]

Eq. (6.10)

\[ \varphi_{EA} = (d - d_{pd}) \cdot \tan \left( \frac{\alpha}{2} \right) \cdot \frac{\pi}{P} \]

Eq. (6.11)
Combined failure in both threads is taken into account with

\[ V_{MG,SG} = \frac{F_{As, MG}}{F_{As, SG}} \]

Eq. (6.13)

and

\[ R(V_{MG, SG}) = \begin{cases} 1 + 0.3 \cdot e^{1.95(1 - V_{MG, SG})} & \text{for } V_{MG, SG} \leq 1 \\ 1 + 0.3 \cdot e^{2.5(1 - V_{MG, SG})} & \text{for } V_{MG, SG} > 1 \end{cases} \]

Eq. (6.14)

EN 1993-1-3 gives an alternative equation for self-tapping screws in supporting structures made of steel:

\[ N_{Rs, II} = \begin{cases} 0.45 \cdot d \cdot t_{II} \cdot f_{u, II} & \text{if } t_{II} < P \\ 0.65 \cdot d \cdot t_{II} \cdot f_{u, II} & \text{if } t_{II} \geq P \end{cases} \]

Eq. (6.15)

The application range of Eq. (6.15) is limited:
- self-tapping screws made of steel or stainless steel;
- diameter of the screws 3.0 mm ≤ d ≤ 8.0 mm;
- t_{II} ≥ 0.9 mm
- f_{u, II} > 550 N/mm² should not be taken into account in design;

EN 1999-1-4 gives an alternative equation for self-tapping screws or self-drilling screws in supporting structures made of steel or aluminium:

\[ N_{Rs, II} = 0.95 \cdot f_{u, II} \cdot \sqrt[d]{t_{II}^3} \]

Eq. (6.16)
The application range of Eq. (6.16) is limited:

- self-tapping screws and self-drilling screws made of steel or stainless steel;
- diameter of the screws \(6.25 \leq d \leq 6.5\) mm;
- \(t_{II} > 6\) mm and \(f_{u,II} > 250\) N/mm² for aluminium should not be taken into account in design;
- \(t_{II} > 5\) mm and \(f_{u,II} > 400\) N/mm² for steel should not be taken into account in design;
- the diameter of the drilling hole should be in accordance with the recommendations of table 6.1

<table>
<thead>
<tr>
<th>(t_{\text{nom,II}}) [mm]</th>
<th>(0.75 \leq 1.5)</th>
<th>(1.5 \leq 3.0)</th>
<th>(3.0 \leq 5.0)</th>
<th>(5.0 \leq 7.0)</th>
<th>(&gt; 7.0)</th>
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<tbody>
<tr>
<td>(d_0) [mm]</td>
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<td>4.5</td>
<td>5.0</td>
<td>5.3</td>
<td>5.5</td>
</tr>
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</table>

6.3.3 Design procedures for timber supporting structures

EN 1995-1-1 applies for fastenings of sandwich panels for timber supporting structures. Corrigendum A1 published in 2010 shall be taken into account, because major issue of this corrigendum were axially loaded screw fasteners. The provisions presented here are limited to fastening to timber (solid timber, sawn, planed or in pole form, glued laminated timber). For fastening to wood-based structural products (e.g. LVL) or wood-based panels, see EN 1995-1-1.

Load-bearing capacity is calculated according to EN 1995-1-1, Eq. (8.40a):

\[
F_{ax,Rk} = \frac{n_{ef} \cdot f_{ax,k} \cdot d \cdot l_{ef}}{1.2 \cdot \cos^2 \alpha + \sin^2 \alpha \left( \frac{\rho_k}{\rho_a} \right)^{0.8}}
\]

Eq. (6.17)

with

- \(n_{ef}\) effective number of fasteners according to EN 1995-1-1, 8.2.3, Eq. (8.17)
- \(f_{ax,k}\) characteristic withdrawal strength perpendicular to the direction of grain
- \(d\) external thread diameter of the screw, usually the nominal diameter
- \(l_{ef}\) effective length, penetration length of the thread, \(l_{ef} = l_g - l_b\); \(l_{ef} \geq 5 \cdot d\)
- \(l_g\) screw-in length - part of thread into component II including length of drill point
- \(l_b\) length of unthreaded part of the drill point
- \(\alpha\) angle between the force and the direction of grain
- \(\rho_k\) characteristic density
- \(\rho_a\) associated density related to \(f_{ax,k}\), usually \(\rho_a = 350\) kg/m³ for timber of
strength grade C24 as reference density

EN 1995-1-1 requires $l_{ef} \geq 5 \cdot d$ which is often reduced to $l_{ef} \geq 4 \cdot d$ for fastenings of sheeting profiles or sandwich panels. If this is done, this has to be supported by test results: The characteristic withdrawal strength $f_{ax,k}$ has to be determined with this lower effective length. For spacings and edge distances see table 6.2

Table 6.2: Minimum spacings and edge distances [mm]

<table>
<thead>
<tr>
<th>spacings</th>
<th>distance to end grain</th>
<th>distance to edge parallel to grain</th>
</tr>
</thead>
<tbody>
<tr>
<td>direction of grain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d$ [mm]</td>
<td>$a_1$</td>
<td>$a_2$</td>
</tr>
<tr>
<td>4.8</td>
<td>34</td>
<td>24</td>
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<td>5.5</td>
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<td>33</td>
</tr>
<tr>
<td>8.0</td>
<td>56</td>
<td>40</td>
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</tbody>
</table>

Designations according to EN 1995-1-1, see also Figs. 6.4 to 6.7.

For fasteners with threads usually applied for fastening of sandwich panels to timber supporting structures, the characteristic withdrawal strength perpendicular to the direction of grain can be calculated with

$$f_{ax,k} = 70 \cdot 10^{-6} \cdot \rho_k^2$$

Eq. (6.18)

with

- $f_{ax,k}$ characteristic withdrawal strength perpendicular to the direction of grain in N/mm²
- $\rho_k$ characteristic density in kg/m³

For most fastenings of sandwich panels, Eq. (6.17) can be simplified to
for \( n = n_{\text{ef}} = 1.0 \). For the calculation of the characteristic load-bearing capacity, the modification factor for duration of load and moisture content (see EN 1995-1-1, table 3.1) has to be taken into account:

\[
N_{R_k,II} = F_{ax,R_k} \cdot k_{\text{mod}}
\]

Eq. (6.20)

with

\( k_{\text{mod}} \) modification factor for load duration class and moisture content described by service class

Actions are assigned to one of the load-duration classes given in table 6.3. Examples of load-duration assignment are given in table 6.3, too. Since climatic loads (snow, wind) vary between countries, the assignment of load-duration classes may be specified in the National annex to EN 1995-1-1.

<table>
<thead>
<tr>
<th>Load duration class</th>
<th>Order of accumulated duration of characteristic load</th>
<th>Examples of loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>permanent</td>
<td>more than 10 years</td>
<td>self-weight</td>
</tr>
<tr>
<td>long-term</td>
<td>6 months - 10 years</td>
<td>storage, temperature</td>
</tr>
<tr>
<td>medium-term</td>
<td>1 week - 6 months</td>
<td>imposed floor load, snow</td>
</tr>
<tr>
<td>short-term</td>
<td>less than one week</td>
<td>snow, wind</td>
</tr>
<tr>
<td>instantaneous</td>
<td></td>
<td>wind, accidental load</td>
</tr>
</tbody>
</table>
Table 6.4: Service class

<table>
<thead>
<tr>
<th>Service class</th>
<th>Average moisture content</th>
<th>Environmental conditions</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12%</td>
<td>temperature 20°C and relative humidity of 65% exceeded for just a few weeks per year</td>
<td>components and structures in heated buildings</td>
</tr>
<tr>
<td>2</td>
<td>20%</td>
<td>temperature 20°C and relative humidity of 85% exceeded for just a few weeks per year</td>
<td>covered open components and structures (exception: areas with condensation water)</td>
</tr>
<tr>
<td>3</td>
<td>&gt; 20%</td>
<td>climate conditions leading to higher moisture contents than in service class 2</td>
<td>components and structures exposed to weather</td>
</tr>
</tbody>
</table>

For fastenings of sandwich panels supported by solid timber or glued laminated timber in service class 1 or 2 and dominating wind loads, \( k_{mod} = 1.0 \) applies in most cases. For fastenings of sandwich panels supported by solid timber or glued laminated timber in service class 1 or 2 and dominating temperature loads, \( k_{mod} = 0.70 \) applies in most cases. If a load combination consists of actions belonging to different load-duration classes a value of \( k_{mod} \) should be chosen which corresponds to the action with the shortest duration, see EN 1995-1-1. E.g. for a combination of dead load and a short-term load, a value of \( k_{mod} \) corresponding to the short-term load should be used.

6.4 Shear resistance

6.4.1 Background
As the internal face of the sandwich panel is usually thinner than the supporting structure, failure of the internal face dominates load-bearing capacity of the fastening. This is also expressed by the available design equations which base on the properties of the internal face, only, but require minimum resistance of the supporting structure. For steel supporting structures this minimum resistance is defined by
parameters of the application range. Up to now, no comparable application range is available for panels on aluminium supporting structures. For timber supporting structures, an additional verification of the capacity of the supporting structure has to be done.

Equations for calculating the shear resistance of fastenings were developed within the framework of the EASIE project and published in the reports by Käpplein & Misiek (2011). A summary can be found in Käpplein & Ummenhofer (2011).

### 6.4.2 Design procedures for sandwich panels with steel faces

The characteristic shear resistance represented by the bearing strength of the internal face can be estimated using

\[
V_{Rk,I} = 4.2 \cdot \sqrt{t_{F2}^2 \cdot d_1 \cdot f_{u,F2}}
\]

Eq. (6.21)

with

- \(t_{F2}\) design core sheet thickness of the internal face of the sandwich panel
- \(d_1\) internal thread diameter of the threaded part of the fastener
- \(f_{u,F2}\) tensile strength of the internal face

The application range is limited:

- direct or hidden fastening, no indirect fastening;
- internal faces made of steel;
- mineral wool, polystyrene, polyurethane or polyisocyanurate core material;
- panel thickness \(d_c \geq 40\) mm;
- self-tapping screws and self-drilling screws made of steel or stainless steel;
- diameter of the screws \(5.5\) mm \(\geq d \geq 8.0\) mm;
- \(0.40\) mm \(\leq t_{F2} \leq 1.00\) mm design core sheet thickness of the internal face (steel). Values \(t_{F2} > 1.0\) mm should not be taken into account;
- \(t_{II} \geq 1.46\) mm core thickness of the supporting structure (steel or aluminium);
- spacing and edge distances according to Fig. 6.3.
Within the application range, no further verification of the characteristic shear resistance $V_{Rk,II}$ of the supporting structure (component II) has to be done. For interaction verification for the supporting structure, $V_{Rk,II}$ might be conservatively estimated by evaluating Eq. (6.21) for $t_{II} = t_{F2} = 1.46$ mm.

### 6.4.3 Design procedures for sandwich panels with aluminium faces

Up to now, no design formulas are available for sandwich panels with aluminium faces. Determination of resistance of fastenings of sandwich with aluminium faces has to be based on tests.

### 6.4.4 Additional design procedures for timber supporting structures

EN 1995-1-1 applies for fastenings of sandwich panels for timber supporting structures. The provisions presented here are limited to fastening to timber (solid timber, sawn, planed or in pole form, glued laminated timber). For fastening to wood-based structural products (e.g. LVL) or wood-based panels, see EN 1995-1-1.

Load-bearing capacity is calculated according to EN 1995-1-1, Eq. (8.9):

$$F_{V,Rk} = \min \left\{ \frac{0.4 \cdot f_{h,k} \cdot t_1 \cdot d_{ef}}{1.15 \cdot \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,k} \cdot d_{ef} + \frac{F_{ax,Rk}}{4}}} \right\}$$

Eq. (6.22)

with

- $f_{h,k}$ characteristic embedment strength of timber member
- $t_1$ is the smaller of the thickness of the timber side member or the penetration depth
- $d_{ef}$ effective diameter: 1.1-times the thread root diameter, for which the simplified
assumption of $0.7 \cdot d$ can be made. For screws with partially smooth shank diameter and a penetration depth $> 4 \cdot d_{ef}$ is taken as the shank diameter $M_{y,Rk}$ characteristic yield moment of fastener

$F_{ax,k}$ characteristic axial withdrawal capacity of the fastener acc. to Eq. (6.17) or Eq. (6.19)

Eq. (6.22) applies for $t_{F1} \leq 0.5 \cdot d$, neglecting the effect of clamping of the fastener in the sandwich panel. Clamping of the fasteners and assuming a plastic hinge in the fastener as given by EN 1995-1-1, Eq. (8.10) should not be taken into account because the thin faces do not prevent deformation by elongation of holes and do not allow evolution of a plastic hinge.

Eq. (6.22) assumes fasteners perpendicular to the direction of grain. Nails in end grain should not be considered capable of transmitting lateral forces. Spacing and edge distances according to Figs. 6.4 to 6.7 and table 6.5 apply.
6 RESISTANCE DETERMINED BY CALCULATION

Fig. 6.6: Edge distances in the direction of force - unloaded edge

Fig. 6.7: Edge distances perpendicular to the direction of force
Table 6.5: **Minimum spacings and edge distances [mm]**

<table>
<thead>
<tr>
<th>direction of grain</th>
<th>in direction of force</th>
<th>perpendicular to the direction of force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>spacings</td>
<td>distance to loaded edge</td>
</tr>
<tr>
<td></td>
<td></td>
<td>distance to unloaded edge</td>
</tr>
<tr>
<td>d [mm]</td>
<td>(a_1)</td>
<td>(a_{3,1})</td>
</tr>
<tr>
<td></td>
<td>(a_2)</td>
<td>(a_{4,1})</td>
</tr>
<tr>
<td></td>
<td>(a_{3,c})</td>
<td>(a_{4,c})</td>
</tr>
<tr>
<td>4.8</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>5.5</td>
<td>28</td>
<td>17</td>
</tr>
<tr>
<td>6.0</td>
<td>30</td>
<td>18</td>
</tr>
<tr>
<td>6.3</td>
<td>32</td>
<td>19</td>
</tr>
<tr>
<td>6.5</td>
<td>33</td>
<td>20</td>
</tr>
<tr>
<td>8.0</td>
<td>40</td>
<td>24</td>
</tr>
</tbody>
</table>

Based on the assumption of predrilled holes, including fastenings with self-drilling screws. Designations according to EN 1995-1-1, see also Figs. 6.4 to 6.7. See EN 1995-1-1 for detailed determination of spacings and edge distances or for applications not covered here.

EN 1995-1-1 requires a minimum effective length of \(l_{ef} \geq 5 \cdot d\), but if confirmed by tests, a smaller value of \(l_{ef} \geq 4 \cdot d\) may be used, see chapter 5 of this recommendation.

The characteristic embedment strength for fastenings with predrilled holes (including fastenings with self-drilling screws) is calculated with

\[
f_{h,k} = 0.082 \cdot (1 - 0.01 \cdot d_{ef}) \cdot \rho_k
\]

Eq. (6.23)

with
- \(d_{ef}\): effective diameter: 1.1-times the thread root diameter, for which the simplified assumption of 0.7\( \cdot d\) can be made. For screws with partially smooth shank diameter and a penetration depth > 4\( \cdot d\) \(d_{ef}\) is taken as the shank diameter
- \(\rho_k\): characteristic density

For calculating the embedment strength, \(d\) can be used instead of \(d_{ef}\), causing no significant difference in value. If not available, the characteristic yield moment of a fastener is calculated with
6 RESISTANCE DETERMINED BY CALCULATION

\[ M_{y,Rk} = 0.3 \cdot f_{u,SG} \cdot d_{ef}^{2.6} \]

Eq. (6.24)

with

\( f_{u,SGk} \) characteristic tensile strength, in N/mm². A value of \( f_{u,SG} = 500 \) N/mm² should be assumed using this equation.

\( d_{ef} \) effective diameter: 1.1-times the thread root diameter, for which the simplified assumption of \( 0.7 \cdot d \) can be made. For screws with partially smooth shank diameter and a penetration depth \( > 4 \cdot d \) \( d_{ef} \) is taken as the shank diameter.

The summand \( F_{ax,k}/4 \) describes the effects of catenary action and has to be limited to

\[ \frac{F_{ax,Rk}}{4} \leq 1.15 \cdot \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,k} \cdot d_{ef}} \]

Eq. (6.25)

The increase in stressing of component I (pull-through resistance of the external face of the sandwich panel) has to be taken into account. The characteristic value of shear resistance of the timber supporting structure (component II) is

\[ V_{Rk,II} = F_{V,Rk} \cdot k_{mod} \]

Eq. (6.26)

with

\( k_{mod} \) modification factor for load duration class and moisture content described by service class, see chapter 6.2.3.

For component I, an additional verification using Eq. (6.21) has to be done, taking into account the resistance of the internal face. The smaller value applies.

When a force in a connection acts at an angle to the grain the possibility of splitting caused by the tension force component \( V_{Ed} \cdot \sin \alpha \) perpendicular to the grain shall be taken into account. Splitting resistance shall be checked against the shear forces on either side of the connection using the splitting capacity.
6 RESISTANCE DETERMINED BY CALCULATION

\[ F_{90,Rk} = 14 \frac{N}{mm^{0.7}} \cdot h \cdot \sqrt{\frac{b_e}{1 - \frac{b_e}{b}}} \]

Eq. (6.27)

with

- \( h \) is the timber member height
- \( b_e \) is the loaded edge distance to the center of the most distant fastener
- \( b \) is the timber member width (width of support)

The characteristic value of splitting resistance of the timber supporting structure (component II) is

\[ V_{90,Rk,II} = F_{90,Rk} \cdot k_{mod} \]

Eq. (6.28)

with

- \( k_{mod} \) modification factor for load duration class and moisture content described by service class, see chapter 6.2.3.

6.5 Head deflection

6.5.1 Background

Allowable value of head deflection depends on material of the fastener, length of cantilever (thickness of the panel) and rotational spring at point of clamping into the supporting structure (thickness of the supporting structure). Calculation of head deflection \( u(D,T) \) is described in Annex A. As tests are comparatively time-consuming and expensive, but demanded head deflection is usually much lower than allowable value of head deflection determined in tests, simple and conservative equations were developed in Misiek et al. (2011) and Misiek et al. (2013).

6.5.2 Design equations for steel, aluminium and timber supporting structures

The design equations for austenitic stainless steel fasteners can be written as

\[ \max u = \max \left\{ 0.3mm \cdot \frac{D_F}{t_{III}}, 0.07 \cdot D_F \right\} \]

Eq. (6.29)
and for carbon steel fastener can be written as

\[
\max u = \begin{cases} 
0.51 \ln mm^2 \cdot \frac{D_F}{t_H^2} \\
0.023 \cdot D_F 
\end{cases}
\]

Eq. (6.30)

These equations were originally developed for steel supporting structures, but give conservative but not uneconomical values for aluminium supporting structures. For timber supporting structures, the minimum value given with each equation is a conservative approximation.
7 EXECUTION AND MAINTENANCE

7.1 General

This chapter gives information about basic requirements on execution and maintenance. Additional information can be found in EN 1090-4 (2016), EN 1090-5 (2016), IFBS (2014), RAGE (2014a) and RAGE (2014b).

Fasteners according to European Standards, European Technical Assessments (ETA), European Technical Approvals (ETA) or national technical approvals shall be used in fastening the sandwich panels to the supporting structure or in the connections between the adjacent sandwich panels.

The type of fastener with designation of the relevant European Standard, ETA or approval (which states the dimensions, materials and mechanical properties of the fasteners as well as characteristic values of the resistance of the connections) shall be specified.

7.2 Layout drawings

Layout drawings shall be prepared for the execution as a part of the execution specification as defined in EN 1090-4 and EN 1090-5. With respect to fastening, the following details shall be included:
- fasteners with type designation and material
- type and material of washer and/or saddle washer
- arrangement and separation distances (spacings, end and edge distances)
- predrill-diameter and pre-drilling depth for self-tapping screws, concrete screws and spikes
- minimum effective length for timber supporting structures
- special assembly instructions depending on the type of fastening

7.3 Product and procedure testing

The performance of fasteners will depend on the site methodology that may be determined by procedure testing. Procedure tests may be used to demonstrate that the required connections can be performed under site conditions.

7.4 Installation personnel

Installation may only be undertaken by companies that possess the necessary specialist knowledge and experience and can demonstrate they employ sufficient
qualified personnel. The following aspects should be considered:

a) ability to produce correct hole size for self-tapping screws and rivets;
b) ability to correctly adjust power screwdrivers with the correct tightening torque/depth location;
c) ability to drive a self-drilling screw perpendicular to the connected surface and set sealing washers to correct compression within the limits recommended by the washer manufacturer;
d) ability to form an adequate structural connection and to recognize an inadequate one.

7.5 Installation

7.5.1 Alignment of sandwich panels
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 recommended dimensions (e.g. distances between fasteners, edge distances) as defined in the layout drawings and the as-built spacings between the supporting elements, they are able to define the center line of the supporting structure or the other target points. The fixing problems may come from the fact that fasteners hit a wrong location in the supporting structure, either too close to the welding or the edge. The correct alignment of the sandwich panel would help in many cases.

7.5.2 Fasteners and tools
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. During installation, the provisions given in the approvals and the fabricator’s instructions regarding suitable sheet thicknesses, materials, clamping thicknesses and tools to be used shall be fulfilled. Power tools for fixing screws shall possess an adjustable depth and/or torque control that shall be set in accordance with the equipment manufacturer’s recommendations. If power screwdrivers are used, the drilling and driving speeds (revolutions per minute) shall be in accordance with the fastener manufacturer’s recommendations.

7.5.3 Position and inclination of fasteners
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 supporting beam has tight tolerances for example. With I-shaped beams, the fasteners shall be arranged in a straight line or in an alternating fixing pattern positioned on
both sides of the beam’s flanges, depending on the requirements defined in the design. The pre-drill diameter of the self-tapping screws shall comply with the value given in the technical approval.

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

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. Further information about the mounting of regular sandwich panels can be found in the ECCS publication no. 62 (1990), IFBS (2014), RAGE (2014a) and RAGE (2014b).

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 variances 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.

### 7.5.4 Tightening of fasteners

When attaching sandwich panels directly to supporting structure, the fasteners have to be positioned such that there is no gap at the point of contact between component I and component II.

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 external 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. 7.1). Larger deformations shall be avoided. The depth gauge, of a power screwdriver, shall be adjusted to compress the elastomeric washer within these limits.

If screws are fastened in the crest of a sandwich panel with profiled face, care shall be taken to avoid dents in the sheet at the penetration point.
7.6 Maintenance
The building owner is advised to have roofs and façades serviced at regular intervals by qualified personnel with the necessary professional competence and experience and sufficient knowledge in order to avoid damage.
REFERENCES

Materials and Durability


General design


ECCS TC7, (2008), *The Testing of Connections with Mechanical Fasteners in Steel Sheeting and Sections*, ECCS publication no. 124, Brussels.

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ECCS TC7 TWG7.9 and CIB W056, (under preparation), *European Recommendations for the Design of Sandwich Panels with Point or Line Loads*. ECCS publication/CIB publication, Brussels/Rotterdam.

EOTA, (2010), *Fastening Screws for Sandwich Panels*. CUAP 06.02/12, Common Understanding of Assessment Procedure for European Technical Approval according to Article 9.2 of the Construction Products Directive, Brussels.


Misiek Th, (2010), *Connections of Sandwich Panels / Spojevi sandwich panela*, Presentation held at the EASIE Workshop in Zagreb.


**Structural analysis**


EN 14509, (2013), *Self-supporting double skin metal faced insulating panels - Factory made products - Specifications*. Published by CEN (Comité Européen de Normalisation), Brussels.
Pull-out resistance


Pull-through resistance


Shear resistance


Execution and maintenance

ECCS TC7 TWG 7.4, (1990), Preliminary European Recommendations for Sandwich Panels – Part II Good Practice. ECCS publication no. 62, Brussels.

EN 1090-4, (2018). Execution of steel structures and aluminium structures – Part 4: Technical requirements for cold-formed structural steel elements and cold-formed structures for roof, ceiling, floor and wall applications. Published by CEN (Comité Européen de Normalisation), Brussels.

EN 1090-5, (2017). Execution of steel structures and aluminium structures – Part 5: Technical requirements for cold-formed structural steel elements and cold-formed structures for roof, ceiling, floor and wall applications. Published by CEN (Comité Européen de Normalisation), Brussels.


IFBS Installation, (2014), Guideline for the planning and installation of roof, wall and deck constructions made from profiled metal sheeting - IFBS - Technical Rules for Lightweight Metal Construction. IFBS International Association for Metal Building Envelopes, Krefeld.


ANNEX A: HEAD DEFLECTION

A.1 Mechanisms which cause head deflections

Both shear deformation (Fig. A.1) and bending rotation (Fig. A.2) at supports results in a relative displacement of both faces, related to a deflection of the head of the fastener relative to the point of fixation at the supporting structure.

![Shear deformation at support](image1)

**Fig. A.1: Shear deformation at support**

The shear strain $\gamma$ due to elastic shear deformation in the core is

\[
\gamma = \frac{\tau}{G_C}
\]

Eq. (A.1)

![Bending rotation at support](image2)

**Fig. A.2: Bending rotation at support**

The bending rotation at the support is

\[
\gamma_1 = \frac{dw}{dx}
\]

Eq. (A.2)

The displacement $u$ of the head of the fasteners can be calculated on the basis of the superposition of displacements $u(\gamma)$ and $u(\gamma_1)$ as shown in Fig. A.3.
Fig. A.3: Displacements at the end of sandwich panel

\[ u = u(\gamma_f) - u(\gamma) = \gamma_1 \cdot D_F - \gamma \cdot e \]

Eq. (A.3)

with distance \( D_F \) between top of supporting structure and top of external face at point of fastening.

The calculated deflection \( u \) shall be less than the maximum allowable value of head deflection \( \text{max } u \) as determined in chapter 6.5.

Analytical calculation methods to determine \( \gamma \) and \( \gamma_1 \) for different mechanical systems and loading conditions are given in the following chapters. For an approximate calculation for single or multi-span panels with flat or profiled faces see chapter A.4. See Naujoks & Misiek (2015) for calculation of head deflection using truss systems.
A.2 Analytical calculation procedures for single span panels

A.2.1 General parameters

The parameters and symbols which are used in calculating head deflection \( u \) comply with EN 14509, but are given here for easier use.

- \( S \) shear stiffness of the core
- \( B \) overall width of the panel
- \( L \) span
- \( e \) distance between centroids of faces
- \( d_c \) depth of core
- \( B_{F1}, B_{F2} \) bending stiffness of the profiled external and internal face
- \( B_s \) bending stiffness of the sandwich
- \( \alpha_{F1}, \alpha_{F2} \) coefficients of thermal expansion of faces 1 and 2

\[
S = A_c \cdot G_c = \frac{e^2}{d_c} \cdot B \cdot G_c
\]

Eq. (A.4)

\[
\alpha = \frac{B_s}{B_{F1} + B_{F2}}
\]

Eq. (A.5)

\[
\beta = \frac{B_s \cdot L^2}{S \cdot L^2} = \frac{B_s}{A_c \cdot G_c \cdot L^2} = \frac{B_s \cdot d_c}{B \cdot e^2 \cdot G_c \cdot L^2}
\]

Eq. (A.6)

\[
\lambda = \frac{1 + \alpha}{\alpha \cdot \beta}
\]

Eq. (A.7)
### A.2.2 Sandwich panels with flat or lightly profiled faces

**Table A|1:** Simply supported panel with uniformly distributed load \( q_0 \)

\[
V = \frac{q_0 \cdot L}{2} \quad \text{Eq. (A.8a)}
\]

\[
\gamma = \frac{V}{S} \quad \text{Eq. (A.8b)}
\]

\[
\gamma_1 = \frac{q_0 \cdot L^3}{24 \cdot B_S} + \frac{q_0 \cdot L}{2 \cdot S} \quad \text{Eq. (A.8c)}
\]

**Table A|2:** Simply supported panel with a line load parallel to the supports

\[
\varepsilon = \frac{a}{L} \quad \text{Eq. (A.9a)}
\]

\[
V_A = F \cdot \frac{L - a}{L} = F \cdot (1 - \varepsilon) \quad \text{Eq. (A.9b)}
\]

\[
\gamma_A = \frac{V_A}{S} \quad \text{Eq. (A.9c)}
\]

\[
\gamma_B = \frac{V_B}{S} = \frac{F \cdot \varepsilon}{S} = \frac{V_A}{S} \cdot \frac{\varepsilon}{1 - \varepsilon} \quad \text{Eq. (A.9d)}
\]

\[
\gamma_{A,3} = \frac{F \cdot L^2}{6 \cdot B_S} \left[ 2 \cdot \varepsilon - 3 \cdot \varepsilon^2 + \varepsilon^3 \right] + \frac{F}{S} \cdot (1 - \varepsilon) \quad \text{Eq. (A.9e)}
\]

\[
\gamma_{B,3} = \frac{F \cdot L^2}{6 \cdot B_S} \left[ \varepsilon^3 - \varepsilon \right] - \frac{F}{S} \cdot \varepsilon \quad \text{Eq. (A.9f)}
\]
ANNEX A: HEAD DEFLECTION

Table A|3: Simply supported panel with a temperature difference between the faces

\[ \Delta T = |T_{\text{internal}} - T_{\text{external}}| \]

\[ \gamma = 0 \quad \text{Eq. (A.10a)} \]

\[ \gamma_1 = \frac{\alpha_{F_2} \cdot T_2 - \alpha_{F_1} \cdot T_1}{2 \cdot e} \cdot L \quad \text{Eq. (A.10b)} \]

A.2.3 Sandwich panels with profiled faces

Table A|4: Simply supported panel with uniformly distributed load \( q_0 \)

\[ \gamma = \frac{q_0 \cdot L^3 \cdot \beta}{(B_s + B_p)} \left[ 1 - \frac{1}{2} \cdot \frac{1}{\lambda} \cdot \tanh \frac{\lambda}{2} \right] \quad \text{Eq. (A.11a)} \]

\[ \gamma_1 = \frac{q_0 \cdot L^3}{(B_s + B_p)} \left[ 1 + \frac{1}{24} \cdot \frac{1}{\alpha \cdot \lambda^2} - \frac{1}{\alpha \cdot \lambda^3} \cdot \tanh \frac{\lambda}{2} \right] \quad \text{Eq. (A.11b)} \]
Table A|5: Simply supported panel with a line load parallel to the supports

\[ \varepsilon = \frac{a}{L} \]

Eq. (A.12a)

\[ \gamma_A = \frac{F \cdot L^2 \cdot \beta}{(B_S + B_D)} \left[ 1 - \varepsilon - \frac{\sinh (\lambda \cdot (1 - \varepsilon))}{\sinh \lambda} \right] \]

Eq. (A.12b)

\[ \gamma_B = \frac{F \cdot L^2 \cdot \beta}{(B_S + B_D)} \left[ -\varepsilon + \frac{\sinh (\lambda \cdot \varepsilon)}{\sinh \lambda} \right] \]

Eq. (A.12c)

\[ \gamma_{A,1} = \frac{F \cdot L^2}{(B_S + B_D)} \left[ \frac{1}{6} \left[ 2 \cdot \varepsilon - 3 \cdot \varepsilon^2 + \varepsilon^3 \right] + \frac{1}{\alpha^2} \left[ 1 - \varepsilon - \frac{\sinh (\lambda \cdot (1 - \varepsilon))}{\sinh \lambda} \right] \right] \]

Eq. (A.12d)

\[ \gamma_{B,1} = \frac{F \cdot L^2}{(B_S + B_D)} \left[ \frac{1}{6} \left[ \varepsilon^3 - \varepsilon \right] + \frac{1}{\alpha^2} \left[ -\varepsilon + \frac{\sinh (\lambda \cdot \varepsilon)}{\sinh \lambda} \right] \right] \]

Eq. (A.12e)

Table A|6: Simply supported panel with a temperature difference between the faces

\[ \Delta T = |T_{\text{internal}} - T_{\text{external}}| \]

\[ \theta = \frac{\alpha_{F2} \cdot T_2 - \alpha_{F1} \cdot T_1}{e} \]

Eq. (A.13a)

\[ \gamma = -\frac{\theta \cdot L}{\lambda} \cdot \tanh \frac{\lambda}{2} \]

Eq. (A.13b)

\[ \gamma_1 = \frac{\theta \cdot L}{1 + \alpha} \left[ \frac{1}{2} - \frac{1}{\lambda} \cdot \tanh \frac{\lambda}{2} \right] \]

Eq. (A.13c)

A.3 Analytical calculation procedures for multi-span panels

Head deflection for multi-span panels is determined by the superposition of the simply supported system and the statically indeterminate forces as shown in Fig.
A.4 below (method of consistent deformation). An example can be found in Annex C.

Fig. A.4: Method of consistent deformation for multi-span panels

A.4 Approximate calculation for single or multi-span panels with flat or profiled faces

For most applications, calculation of head deflection using approximate calculation procedures is sufficient and leading to conservative results.

\[ u(q_0) = \frac{q_0 \cdot L^3 \cdot e}{24 \cdot (B_s + B_p)} \]

Eq. (A.14)

\[ u(\Delta T) = \frac{\theta \cdot L \cdot e}{2} \]

Eq. (A.15)

The limits of application of the above formulae are given by
ANNEX A: HEAD DEFLECTION

\[ L_1 = L_2 = L_3 = \cdots = L \]

Eq. (A.16)

\[ L \cdot \sqrt{\frac{S}{B_3}} > 2.5 \]

Eq. (A.17)

see also Fig. A.5.

Fig. A.5: Application limits for approximate calculations
ANNEX B: TRANSVERSE COMPRESSION DEFORMATION

B.1 Calculation of deformation

The verifications shall be done using the formula

\[
W_S = \begin{cases} 
\frac{F_{Ed}}{E_{Cc}} \cdot \frac{1}{n_k \cdot k} \cdot \ln \left( \frac{L_S + n_k \cdot k \cdot e}{L_S} \right), & k \neq 0.00 \\
\frac{F_{Ed}}{E_{Cc}} \cdot \frac{e}{L_S}, & k = 0.00 
\end{cases}
\]

Eq. (B.1)

with

- \( F_{Ed} \) support reaction force
- \( E_{Cc} \) compression modulus of the core
- \( L_S \) width of the support
- \( k \) distribution parameter according to EN 14509
- \( n_k \) distribution factor, see figure B.1

![Fig. B.1: Load distribution](image)

The value of transverse compression deformation is usually limited to 1 mm. See RAGE (2014a) for further requirements and additional verifications.

B.2 Detailing requirements

To ensure tightness, screws with an additional thread under the head and washers with \( d_w \geq 19 \text{ mm} \) and thickness of the cured-on elastomer seal \( \geq 3.0 \text{ mm} \) shall be used.
ANNEX C: DESIGN EXAMPLES

C.1 Background information

Both the following examples are based on the examples given in Berner (1998), Davies et al. (2001) and Naujoks & Misiek (2015). These examples are virtually identical, with the exception of some small differences in loads. Compared to the aforementioned references, additional actions and loads are covered here.

C2 Wall panel

C.2.1 System

Wall panel spanning vertical, lying over two spans with L = 4000 mm. The panel has lightly profiled faces and a nominal thickness of 60 mm.

External face is made from structural steel S320GD+Z275 ($f_{u,F1} = 390 \text{ N/mm}^2$) according to EN 10346, nominal thickness is $t_{nom,F1} = 0.60 \text{ mm}$ with special tolerance according to EN 10143. Internal face is made from structural steel S320GD+Z275 ($f_{u,F2} = 390 \text{ N/mm}^2$) according to EN 10346, nominal thickness is $t_{nom,F2} = 0.50 \text{ mm}$ with special tolerance according to EN 10143. The core material is polyurethane foam with shear modulus $G_C = 3.1 \text{ N/mm}^2$.

The panel will be fastened by hidden fastenings, using a load spreader plate placed in the longitudinal joints between the panels. There is one load spreader plate per panel of 1 m width, allowing fastening with up to three fasteners. In this
case two self-drilling screws 6.3 x L per load spreader plate are used at base and intermediate supports (supports No. 1 and No. 2) and one self-drilling screw 6.3 x L is used at top support (support No. 3). Both load spreader plates and fasteners are made of stainless steel.

Supporting structure is a cold-formed C-section made from structural steel S390GD+Z275 ($f_{u,II} = 420$ N/mm$^2$) according to EN 10346, nominal thickness is $t_{nom,II} = 2.00$ mm with normal tolerance according to EN 10143. Shear forces from self-weight loading are transferred only by the base support (support No. 3), see Fig. C.1.

C.2.2 Actions and loads

Self-weight

A value of

$$g_x = 0.107 \frac{kN}{m^2}$$

is assumed throughout this example.

Wind suction

The value

$$w_c = c_{pe,10} \cdot q_p = -0.50 \cdot 0.50 \frac{kN}{m^2} = -0.25 \frac{kN}{m^2}$$

as given in the aforementioned references is based on pressure coefficient $c_{p,10}$ which applies for the design of the sandwich panel. For design of fastenings, $c_{pe}$ is calculated for each support, with size of the loaded area (see EN 1991-1-4, chapter 7.2.1) simply defined from mid of span to mid of span.

End supports (supports No. 1 and 3):

$$c_{pe} = c_{pe,1} - \left( c_{pe,1} - c_{pe,10} \right) \cdot \log_{10} A = -0.7 - (-0.7 + 0.5) \cdot \log_{10} 2 \frac{m^2}{m} = -0.64$$

$$w_c = c_{pe,10} \cdot q_p = -0.64 \cdot 0.50 \frac{kN}{m^2} = -0.32 \frac{kN}{m^2}$$

Intermediate support (support No. 2)

$$c_{pe} = c_{pe,1} - \left( c_{pe,1} - c_{pe,10} \right) \cdot \log_{10} A = -0.7 - (-0.7 + 0.5) \cdot \log_{10} 4 \frac{m^2}{m} = -0.58$$

$$w_c = c_{pe,10} \cdot q_p = -0.58 \cdot 0.50 \frac{kN}{m^2} = -0.29 \frac{kN}{m^2}$$

3 Load not given in original examples
Temperature
For assumed colour group 2, the following values $T_1/T_2$ and $\Delta T$ apply for design:
Winter temperature $-20^\circ\text{C}/+20^\circ\text{C}$ $\Delta T = 40\text{K}$
Summer temperature $+65^\circ\text{C}/+25^\circ\text{C}$ $\Delta T = -40\text{K}$
Depending on national regulations, higher values for external temperature $T_1$ apply for ultimate limit state design. For fatigue limit state (head deflection), a temperature difference of $\Delta T = T_2 - T_1 = -70 \text{ K}$ applies for design.

C.2.3 Characteristic values of support reactions
All shear forces are transferred via the base support No. 1.

$$G_k = g_k \cdot 2 \cdot L = 0.107 \frac{kN}{m^2} \cdot 2 \cdot 4m = 0.854 \frac{kN}{m}$$

Transverse support reactions are given in the following table. Support reactions for wind loads were determined by software. For detailed information on calculation of support reactions under temperature loading see references.

<table>
<thead>
<tr>
<th>Support</th>
<th>$W_k$</th>
<th>$T_{k,\text{Summer}}$</th>
<th>$T_{k,\text{Winter}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.505</td>
<td>0.483</td>
<td>-0.483</td>
</tr>
<tr>
<td>2</td>
<td>-1.442</td>
<td>-0.966</td>
<td>0.966</td>
</tr>
<tr>
<td>3</td>
<td>-0.505</td>
<td>0.483</td>
<td>-0.483</td>
</tr>
</tbody>
</table>
C.2.4 Design values of support reactions

For shear force

\[ V_{Ed,1} = \gamma_F \cdot G_k = 1.35 \cdot 0.854 \frac{kN}{m} = 1.15 \frac{kN}{m} \]

applies. Both the combinations

\[ N_{Ed} = \gamma_F \cdot W_k + \gamma_F \cdot \psi_{0F} \cdot T_k \]

and

\[ N_{Ed} = \gamma_F \cdot T_k + \gamma_F \cdot \psi_{0W} \cdot W_k \]

need to be checked for maximum tensile forces at all three supports.

In this case, first combination leads to higher tensile forces:

\[ N_{Ed,1} = \gamma_F \cdot W_k + \gamma_F \cdot \psi_{0F} \cdot T_{k,Summer} = 1.5 \cdot -0.505 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -0.483 \frac{kN}{m} = -1.19 \frac{kN}{m^2} \]

\[ N_{Ed,2} = \gamma_F \cdot W_k + \gamma_F \cdot \psi_{0F} \cdot T_{k,Winter} = 1.5 \cdot -1.442 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -0.966 \frac{kN}{m} = -3.03 \frac{kN}{m} \]

\[ N_{Ed,3} = \gamma_F \cdot W_k + \gamma_F \cdot \psi_{0F} \cdot T_{k,Summer} = 1.5 \cdot -0.505 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -0.483 \frac{kN}{m} = -1.19 \frac{kN}{m^2} \]

Because of symmetry both of mechanical system and load, \( N_{Ed,1} = N_{Ed,3} \).
C.2.5 Head deflection

Analytical calculation of head deflection for multi-span panels is based on method of consistent deformations. In a first step, head deflection at end supports is calculated for the single span panel with \( L_{\text{eqv}} = 2 \cdot L = 800 \, \text{cm} \) and a temperature difference of \( \Delta T = T_2 - T_1 = -70 \, \text{K} \).

\[
\theta = \frac{\alpha_{r,F_2} \cdot T_2 - \alpha_{r,F_1} \cdot T_1}{e} = \frac{\alpha_r \cdot (T_2 - T_1)}{e} = \frac{1.2 \cdot 10^{-5} \cdot \frac{1}{K} \cdot -70 \, \text{K}}{5.866 \, \text{cm}} = -1.43 \cdot 10^{-4} \cdot \frac{1}{\text{cm}}
\]

Shear strain due to elastic shear deformation in the core is

\[
\gamma_T = 0
\]

The bending rotation at the support is

\[
\gamma_{IT} = \frac{\theta \cdot L_{\text{eqv}}}{2} = \frac{-1.43 \cdot 10^{-4} \cdot \frac{1}{\text{cm}} \cdot 800 \, \text{cm}}{2} = -5.73 \cdot 10^{-2}
\]

In a second step, head deflection caused by the force \( F \) is calculated. The force \( F \) is defined by the corresponding deflection which is equal to the one caused by the temperature difference:

\[
w_T = \frac{\theta \cdot L_{\text{eqv}}^2}{8} = \frac{-1.43 \cdot 10^{-4} \cdot \frac{1}{\text{cm}} \cdot (800 \, \text{cm})^2}{8} = -11.5 \, \text{cm}
\]

Hence the force \( F \) will eliminate the deformation at mid of span of the equivalent single span panel. For the deflection under a line load \( F \)

\[
w_F = \frac{F \cdot L_{\text{eqv}}}{4} \cdot \left[ \frac{L_{\text{eqv}}^2}{12 \cdot B_S} + \frac{1}{S} \right]
\]

applies and therefore

\[
F(w_F = w_T) = \frac{4 \cdot w_T}{L_{\text{eqv}} \cdot \left[ \frac{L_{\text{eqv}}^2}{12 \cdot B_S} + \frac{1}{S} \right]} = \frac{4 \cdot 11.5 \, \text{cm}}{800 \, \text{cm} \cdot \left( \frac{(800 \, \text{cm})^2}{12 \cdot 1879579 \, \text{KNcm}^{-2}} + \frac{1}{181.8 \, \text{KNm}^{-1}} \right)} = 1.69 \, \text{kN/m}
\]

The corresponding values of shear strain due to elastic shear deformation and bending rotation at the support are

\[
\gamma_F = \frac{F}{2 \cdot B_S} = \frac{1.69 \, \text{kN}}{2 \cdot 181.8 \, \text{KNm}^{-1}} = 4.65 \cdot 10^{-3}
\]

and
ANNEX C: DESIGN EXAMPLES

\[
\gamma_{1F} = F \cdot \left[ \frac{L_{\text{eqv}}^2}{16 \cdot B_S} + \frac{1}{2 \cdot S} \right] = 1.69 \text{kN/m} \left[ \frac{(800 \text{cm})^2}{16 \cdot 1879579 \text{kNcm}^2/m} + \frac{1}{2 \cdot 181.8 \text{kN/cm}} \right] = 4.06 \cdot 10^{-2}
\]

The angles \( \gamma_1 \) resulting from shear deformation of the core at end support and \( \gamma \) resulting from bending rotation have to be subtracted from each other. Head deflection is finally calculated:

\[
u = \gamma_1 \cdot D_F - \gamma \cdot e = (\gamma_{1F} + \gamma_{F}) \cdot e - (\gamma_T + \gamma_{Fe}) \cdot e = (-5.73 \cdot 10^{-2} + 4.06 \cdot 10^{-2}) \cdot 6.0 \text{cm} - \left(0 + 4.65 \cdot 10^{-3}\right) \cdot 5.866 \text{cm} = -0.127 \text{cm}
\]

with \( D_F \approx d \) for wall panels with flat or lightly profiled faces.

C.2.6 Resistance and Verifications

Pull-through resistance

There is one load spreader plate per panel of 1 m width. The characteristic values of pull-through resistance were taken from the national approval of the sandwich panel, see Fig. C.4.
Fig. C.4: Annex of the national approval with pull-through resistance
Design values of pull-through resistance for each support

\[
N_{rd,1,3} = \frac{N_{Rk,1,3}}{\gamma_M} = \frac{2.13kN}{1.33} = 1.60kN
\]

\[
N_{rd,1,2} = \frac{N_{Rk,1,2}}{\gamma_M} = \frac{4.14kN}{1.33} = 3.11kN
\]

\[
N_{rd,1,3} = \frac{N_{Rk,1,3}}{\gamma_M} = \frac{2.13kN}{1.33} = 1.60kN
\]

**Verifications**

\[
\frac{N_{ed,1}}{N_{rd,1,3}} = \frac{1.19kN}{1.60kN} = 0.74 < 1.0
\]

\[
\frac{N_{ed,2}}{N_{rd,1}} = \frac{3.03kN}{3.11kN} = 0.97 < 1.0
\]

\[
\frac{N_{ed,3}}{N_{rd,1,3}} = \frac{1.19kN}{1.60kN} = 0.74 < 1.0
\]

**Pull-out resistance**

There is one load spreader plate per panel of 1 m width, with two fasteners at base and intermediate support, one fastener at top support.

Supporting structure is a cold-formed C-section made from structural steel S390GD+Z275 \((f_u,II = 420 \text{ N/mm}^2)\) according to EN 10346, nominal thickness is \(t_{\text{nom},II} = 2.00 \text{ mm}\) with normal tolerance according to EN 10143. In case of normal tolerances, tolerances have to be considered in calculation of design core sheet thickness.

Design core sheet thickness of the supporting structure:

\[
t_{II} = t_{\text{nom},II} - t_{\text{zinc}} - 0.5 \cdot t_{\text{tol}} = 2.00\text{mm} - 0.04\text{mm} - 0.5 \cdot 0.16\text{mm} = 1.88\text{mm}
\]

Dimensions of self-drilling screws 6.3 x L made of stainless steel:

- \(d = 6.3 \text{ mm}\)
- \(d_1 = 4.8 \text{ mm}\)
- \(d_{dp} = 5.5 \text{ mm}\)
- \(P = 1.8 \text{ mm}\)
- \(\alpha = 60^\circ\)

\[
d_{pd} = \max\{d_{dp}, d_1\} = \max\{5.5\text{mm}, 4.8\text{mm}\} = 5.5\text{mm}
\]
Load-bearing capacity of the thread of the screw

\[ F_{A_s,SG} = 1.2 \cdot \alpha_{s,SG} \cdot f_{u,SG} \cdot \left[ 0.15mm + \left( d - d_{pd} \right) \cdot \tan\left( \frac{\alpha}{2} \right) \right] \cdot d_{pd} \cdot \frac{t_H \cdot \pi}{P} \]

\[ = 1.2 \cdot 0.70 \cdot 930 \cdot \frac{N}{mm^2} \cdot \left[ 0.15mm + (6.3mm - 5.5mm) \cdot \tan\left( \frac{60^\circ}{2} \right) \right] \cdot 5.5mm \cdot \frac{1.88mm \cdot \pi}{1.8mm} \]

\[ = 8626N \]

Load-bearing capacity of the inside thread in the supporting structure for \( d_{pd} > 5 \text{ mm} \)

\[ t^* = 1.1 \cdot d_{pd} = 1.1 \cdot 5.5mm = 6.05mm \]

\[ \varphi_{EA} = \left( d - d_{pd} \right) \cdot \tan\left( \frac{\alpha}{2} \right) \cdot \frac{\pi}{P} = (6.3mm - 5.5mm) \cdot \tan\left( \frac{60^\circ}{2} \right) \cdot \frac{\pi}{1.8mm} = 0.806 \]

\[ \varphi_c = 0.15mm \cdot \frac{2\pi}{P} = 0.15mm \cdot \frac{2\pi}{1.8mm} = 0.524 \]

\[ F^{*}_{A_s, MG} \text{ for } t_{II} = 0.60 \text{ mm} \]

\[ F^{*}_{A_s, MG} = 0.60 \cdot f_{u,II} \cdot \frac{d}{2} \left[ \frac{\pi}{P} \cdot t_{II} + t_H \cdot \varphi_{EA} \right] \]

\[ = 0.60 \cdot 390 \cdot \frac{N}{mm^2} \cdot \frac{6.3mm}{2} \cdot \frac{\pi}{1.8mm} \cdot (0.60mm)^2 + 0.60mm \cdot 0.806 \]

\[ = 820N \]

\[ F^{*}_{A_s, MG} \text{ for } t_{II} = t^* = 6.05 \text{ mm} \]

\[ F^{*}_{A_s, MG} = 0.60 \cdot f_{u,II} \cdot \frac{d}{2} \left[ \frac{(2\pi - \varphi_c)^2 \cdot P}{4\pi} \right] + (P - 0.15mm) \cdot \left( \frac{2\pi}{P} \cdot t_{II} - 2\pi + \varphi_c \right) \]

\[ + \varphi_{EA} \cdot \left( P - 0.15mm - \frac{P}{4\pi} \cdot \varphi_{EA} \right) \]

\[ = 0.60 \cdot 390 \cdot \frac{N}{mm^2} \cdot \frac{6.3mm}{2} \cdot \left[ \frac{(2\pi - 0.524)^2 \cdot 1.8mm}{4\pi} \right] + (1.8mm - 0.15mm) \cdot \left( \frac{2\pi}{1.8mm} \cdot 6.05mm - 2\pi + 0.524 \right) \]

\[ + 0.806 \cdot \left( 1.8mm - 0.15mm - \frac{1.8mm}{4\pi} \cdot 0.806 \right) \]

\[ = 23094N \]

\[ b = \frac{\ln\left( \frac{F^{*}_{A_s, MG} (t_{II} = t^*)}{0.95 \cdot F^{*}_{A_s, MG} (t_{II} = 0.60mm)} \right)}{\ln\left( \frac{t^*}{0.60mm} \right)} = \frac{\ln\left( \frac{23094N}{0.95 \cdot 820N} \right)}{\ln\left( \frac{6.05mm}{0.60mm} \right)} = 1.47 \]
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\[
a = \frac{0.95 \cdot F_{\text{As, MG}}^{*}(t_{ll} = 0.60 \text{mm})}{(0.60 \text{mm})^{0.47}} = \frac{0.95 \cdot 820N}{(0.60 \text{mm})^{0.47}} = 1647N
\]

\[
F_{\text{As, MG}} = a \cdot t_{ll}^{1.47} = 1647N \cdot (1.88 \text{mm})^{1.47} = 4158N
\]

Reduction divisor for combined failure in both threads

\[
V_{\text{MG, SG}} = \frac{F_{\text{As, MG}}}{F_{\text{As, SG}}} = \frac{4158N}{8626N} = 0.482
\]

\[
R(V_{\text{MG, SG}}) = 1 + 0.3 \cdot e^{\frac{1.95 \cdot (1 - \sqrt{V_{\text{MG, SG}}})}{0.482}} = 1 + 0.3 \cdot e^{\frac{1.95 \cdot (1 - \sqrt{0.482})}{0.482}} = 1.037
\]

Characteristic value of pull-out resistance

\[
N_{Rk,II} = 0.80 \cdot \min\left\{ \frac{F_{\text{As, MG}}}{F_{\text{As, SG}}} \right\} = 0.80 \cdot \min\left\{ \frac{8626N}{4158N} \right\} = 3208N
\]

For comparison, characteristic value of pull-out resistance is additionally calculated according to EN 1993-1-3 and EN 1999-1-4, respectively:

with \( t_{ll} \geq P \)

\[
N_{Rk,II} = 0.65 \cdot d \cdot t_{ll} \cdot f_{u,II} = 0.65 \cdot 6.3 \text{mm} \cdot 1.88 \text{mm} \cdot 390 \frac{N}{\text{mm}^2} = 3002N
\]

\[
N_{Rk,II} = 0.95 \cdot f_{u,II} \cdot \sqrt{d \cdot t_{ll}^{3}} = 0.95 \cdot 390 \frac{N}{\text{mm}^2} \cdot \sqrt{6.3 \text{mm} \cdot (1.88 \text{mm})^{3}} = 2397N
\]

Design value of pull-out resistance

\[
N_{Rd,II} = \frac{N_{Rk,II}}{\gamma_M} = \frac{3208N}{1.33} = 2,41kN
\]

Verifications, with two screws per a load spreader plate at base and intermediate support

\[
\frac{N_{Ed,1}}{N_{Rd,II}} = \frac{1.19kN}{2.41kN} = 0.49 < 1.0
\]

\[
\sum \frac{N_{Ed,2}}{2 \cdot 2.41kN} = 0.63 < 1.0
\]

\[
\sum \frac{N_{Ed,3}}{2 \cdot 2.41kN} = 0.25 < 1.0
\]

Shear resistance

At support No. 1 there are two screws per load spreader plate and panel of 1 m width.

Dimensions of self-drilling screws 6.3 x L made of stainless steel:

\( d = 6.3 \text{ mm} \)

\( d_1 = 4.8 \text{ mm} \)
Internal face is made from structural steel S320GD+Z275 \((f_{u,F2} = 390 \text{ N/mm}^2)\) according to EN 10346, nominal thickness is \(t_{\text{nom,F2}} = 0.50 \text{ mm}\) with special tolerance according to EN 10143.

Design core sheet thickness of the internal face:
\[t_{F2} = t_{\text{nom,F2}} - t_{\text{ome}} = 0.50 \text{ mm} - 0.04 \text{ mm} = 0.46 \text{ mm}\]

Characteristic value of shear resistance:
\[V_{\text{Rk,II}} = 4.2 \cdot \sqrt{t_{F2}^3} \cdot d_1 \cdot f_{u,F2} = 4.2 \cdot \sqrt{(0.46 \text{ mm})^3} \cdot 4.8 \text{ mm} \cdot 390 \frac{N}{\text{mm}^2} = 1120 \text{ N}\]

Design value of shear resistance
\[V_{\text{Rd,II}} = V_{\text{Rk,II}} = \frac{1120 \text{ N}}{1.33} = 841.7 \text{ N}\]

Verification, with two screws at support No. 1
\[\sum V_{\text{Rd}} = 1.15 \text{ kN} = 0.68 < 1.0\]

In this example, use of a support angle at base point to transfer self-weight loading would allow reduction of number of fasteners to one per meter, loaded only by tensile forces.

**Interaction**

Interaction verification has to be made for supporting structure only. While for \(N_{\text{Rd,II}}\), a specific value was derived in this example, there is no equation for explicitly calculating the shear resistance \(V_{\text{Rd,II}}\). \(V_{\text{Rd,II}}\) is conservatively estimated.

Characteristic value of shear resistance:
\[V_{\text{Rk,II}} = 4.2 \cdot \sqrt{t_{II}^3} \cdot d_1 \cdot f_{u,F2} = 4.2 \cdot \sqrt{(1.46 \text{ mm})^3} \cdot 4.8 \text{ mm} \cdot 420 \frac{N}{\text{mm}^2} = 6817 \text{ N}\]

Design value of shear resistance
\[V_{\text{Rd,II}} = V_{\text{Rk,II}} = \frac{6817 \text{ N}}{1.33} = 5.13 \text{ kN}\]

Verification, with three fasteners at support No. 1
\[\frac{N_{\text{Ed,II}}}{N_{\text{Rd,II}}} + \frac{V_{\text{Ed,II}}}{V_{\text{Rd,II}}} = \frac{1.19 \text{ kN}}{2 \cdot 2.41 \text{ kN}} + \frac{1.15 \text{ kN}}{2 \cdot 5.13 \text{ kN}} = 0.25 + 0.11 = 0.36 < 1.0\]
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**Allowable head deflection**

\[
\text{max } u = 0.3 \text{mm} \frac{d_c}{t_{\text{II}}} = 0.3 \text{mm} \cdot \frac{60 \text{mm}}{1.88 \text{mm}} = 9.57 \text{mm}
\]

**Verification**

\[
\frac{u}{\text{max } u} = \frac{0.127 \text{ cm}}{0.957 \text{ cm}} = 0.13 < 1.0
\]

**C.3 Roof panel**

**C.3.1 System**

Roof panel, lying over two spans with \( L = 3300 \text{ mm} \), with a slope of 10°. The panel has a profiled external face with pitch of profile \( p = 333 \text{ mm} \) and width of the crest \( b_1 = 35 \text{ mm} \). The internal face is lightly profiled. The nominal thickness is 60 mm, the thickness at crest is \( D = 98 \text{ mm} \).

![Fig. C.5: Mechanical system](image)

External face is made from structural steel S320GD+Z275 \( (f_{u,F1} = 390 \text{ N/mm}^2) \) according to EN 10346, nominal thickness is \( t_{\text{nom,F1}} = 0.60 \text{ mm} \) with special tolerance according to EN 10143. Internal face is made from structural steel S320GD+Z275 \( (f_{u,F2} = 390 \text{ N/mm}^2) \) according to EN 10346, nominal thickness is \( t_{\text{nom,F2}} = 0.50 \text{ mm} \) with special tolerance according to EN 10143.

The core material is polyurethane foam with shear modulus \( G_C = 3.02 \text{ N/mm}^2 \), compression modulus 3.5 N/mm² (not given in original examples) and compression strength 0.11 N/mm².

The panel will be fastened by crest fastenings, using washers \( d_w = 19 \text{ mm} \) with cured-on elastomer seals (no saddle washers). At base support (support No. 1), there are three fasteners per panel of 1 m width and per support, with each crest fastened. At intermediate and top support (supports No. 2 and No. 3), there are two fasteners per panel of 1 m width and per support, with two crests fastened and one crest unfastened per panel. Self-tapping screws 6.3 x L made of stainless steel are used.

Supporting structure is a hot-rolled section made from structural steel S235JR \( (f_{u,\text{II}} = 360 \text{ N/mm}^2) \) according to EN 10025-2, nominal thickness is \( t_{\text{nom,II}} = 5.00 \text{ mm} \).
Shear forces from self-weight loading are transferred only by support No. 1, see Fig. C.5.
Chapter C.4 deals with the identical system and sandwich panel, but fastened on a supporting structure made of timber using a different type of fastener.

C.3.2 Actions and loads

Self-weight
A value of
\[ g_k = 0.166 \frac{kN}{m^2} \]
is assumed throughout this example. It is split into two components, one acting perpendicular to the panel and the other one acting parallel to the panel:
\[ g_{k,p} = g_k \cdot \sin \alpha = 0.166 \frac{kN}{m^2} \cdot \sin 10^\circ = 0.029 \frac{kN}{m^2} \]
\[ g_{k,t} = g_k \cdot \cos \alpha = 0.166 \frac{kN}{m^2} \cdot \cos 10^\circ = 0.163 \frac{kN}{m^2} \]

Wind suction
Within the example, a panel spanning over zones G and H is assumed. The following pressure coefficients apply after interpolation between values for 5° and 15° slope:

<table>
<thead>
<tr>
<th>Zone</th>
<th>( c_{p,1} )</th>
<th>( c_{p,10} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone G</td>
<td>-1.75</td>
<td>-1.00</td>
</tr>
<tr>
<td>Zone H</td>
<td>-0.45</td>
<td>-0.75</td>
</tr>
</tbody>
</table>

Parameter \( e \) is assumed to be \( e = 6 \) m, therefore width of zone G is 0.6 m adjacent to support No. 1 (end support). For design of fastenings, \( c_{pe} \) is calculated for each support, with size of the loaded area (see EN 1991-1-4, chapter 7.2.1) simply defined from mid of span to mid of span.

Support No. 1, Zone G
\[ c_{pe} = c_{pe,1} = \left( c_{pe,1} - c_{pe,10} \right) \cdot \log_{10} A = -1.75 - (-1.75 + 1.00) \cdot \log_{10} 1.65 \frac{m^2}{m} = -1.59 \]
\[ w_e = c_{pe,1} \cdot q_p = -1.59 \cdot 0.50 \frac{kN}{m^2} = -0.79 \frac{kN}{m^2} \]
Support No. 1, Zone H

\[ c_{pe} = c_{pe,1} - \left( c_{pe,1} - c_{pe,10} \right) \cdot \log_{10} A = -0.75 - \left( -0.75 + 0.45 \right) \cdot \log_{10} 1.65 \frac{m^2}{m} = -0.68 \]

\[ w_e = c_{pe,10} \cdot q_p = -0.68 \cdot 0.50 \frac{kN}{m^2} = -0.34 \frac{kN}{m^2} \]

Support No. 2 (intermediate support), Zone H

\[ c_{pe} = c_{pe,1} - \left( c_{pe,1} - c_{pe,10} \right) \cdot \log_{10} A = -0.75 - \left( -0.75 + 0.45 \right) \cdot \log_{10} 3.3 \frac{m^2}{m} = -0.59 \]

\[ w_e = c_{pe,10} \cdot q_p = -0.59 \cdot 0.50 \frac{kN}{m^2} = -0.30 \frac{kN}{m^2} \]

For support N. 3, zone H see support No. 1, zone H.

\[ w_e = -0.34 \frac{kN}{m^2} \]

Snow loads are treated here because of shear forces from roof thrust. The value

\[ s_i = \mu \cdot s_k = 0.8 \cdot 0.94 \frac{kN}{m^2} = 0.75 \frac{kN}{m^2} \]

applies. It is split into two components, one acting perpendicular to the panel and the other one acting parallel to the panel:

\[ s_{\parallel} = s_i \cdot \cos \alpha \cdot \sin \alpha = 0.75 \frac{kN}{m^2} \cdot \cos 10^\circ \cdot \sin 10^\circ = 0.128 \frac{kN}{m^2} \]

\[ s_{\perp} = s_i \cdot \cos \alpha \cdot \cos \alpha = 0.75 \frac{kN}{m^2} \cdot \cos 10^\circ \cdot \cos 10^\circ = 0.727 \frac{kN}{m^2} \]

Temperature

For colour group 2, the following values \( T_1/T_2 \) and \( \Delta T \) apply for design::

Winter temperature 0°C/+20°C (for combinations including snow, not relevant for design of the fastenings)

Winter temperature -20°C/+20°C \( \Delta T = 40K \)

Summer temperature +65°C/+25°C \( \Delta T = -40K \)
Depending on national regulations, higher values for external temperature $T_1$ apply for ultimate limit state design. For fatigue limit state (head deflection), a temperature difference of $\Delta T = T_2 - T_1 = -70$ K applies for design.

C.3.3 Characteristic values of support reactions

**Self-weight**

For calculation of support reactions, the following parameters need to be calculated:

$$\beta = \frac{B_s}{S \cdot L^2} = \frac{2474198 \text{kNcm}^2}{197 \text{kN/m} \cdot (330 \text{cm})^2} = 0.115$$

$$\alpha = \frac{B_{F1} + B_{F2}}{B_s} = \frac{8.858 \text{cm}^4 \cdot 21000 \text{kN/cm}^2 + 0}{2474198 \text{kNcm}^2} = 0.0752$$

$$\lambda = \sqrt{\frac{1 + \alpha}{\alpha \cdot \beta}} = \sqrt{\frac{1 + 0.0752}{0.0752 \cdot 0.115}} = 11.1$$

$$\varepsilon_1 = \frac{5 \cdot (1 + \lambda) + 12 \cdot \beta \left( 1 - \frac{\text{cosh}(\lambda) - 1}{\lambda^2 \cdot \text{cosh}(\lambda)} \right)}{4 \cdot (1 + \alpha) + 12 \cdot \beta \left( 1 - \frac{\text{tanh}(\lambda)}{\lambda} \right)} = 1.211$$

$$\varepsilon_2 = \frac{1 - \varepsilon_1}{2} = 0.394$$

For background information on parameters see Davies et al. (2001). The characteristic values of support reaction are:

$$G_{k,\perp,1} = G_{k,\perp,3} = g_{k,\perp} \cdot \varepsilon_2 \cdot L = 0.163 \frac{kN}{m} \cdot 0.394 \cdot 3.3m = 0.212 \frac{kN}{m}$$

$$G_{k,\perp,2} = g_{k,\perp} \cdot \varepsilon_1 \cdot L = 0.163 \frac{kN}{m^2} \cdot 1.211 \cdot 3.3m = 0.651 \frac{kN}{m}$$

All shear forces are transferred via support No. 1.

$$G_{k,\parallel,1} = g_{k,\parallel} \cdot 2 \cdot L = 0.029 \frac{kN}{m^2} \cdot 2 \cdot L = 0.190 \frac{kN}{m}$$
**Snow**

For calculation of support reactions, the same parameters as for self-weight apply. The characteristic values of support reaction are

\[
S_{k_{\perp,1}} = S_{k_{\perp,2}} = S_{k_{\perp,3}} = s_{i,1} \cdot \varepsilon_2 \cdot L = 0.727 \frac{kN}{m} \cdot 0.394 \cdot 3.3m = 0.946 \frac{kN}{m}
\]

\[
S_{k_{\perp,2}} = s_{i,1} \cdot \varepsilon_1 \cdot L = 0.727 \frac{kN}{m} \cdot 1.211 \cdot 3.3m = 2.907 \frac{kN}{m}
\]

All shear forces are transferred via support No. 1.

\[
S_{k_{\perp,1}} = s_{i,4} \cdot 2 \cdot L = 0.128 \frac{kN}{m^2} \cdot 2 \cdot 3.3m = 0.845 \frac{kN}{m}
\]

**Wind suction**

Support reactions for wind loads were determined by software.

**Temperature**

For calculation of support reactions, the following parameters need to be calculated:

\[
\varepsilon_s = - \frac{3 \cdot (1 + \alpha) \left(1 - 2 \cdot \frac{\cosh(\lambda) - 1}{\lambda^2 \cdot \cosh(\lambda)}\right)}{1 + \alpha + 1 \cdot \beta \left(1 - \frac{\tanh(\lambda)}{\lambda}\right)} = 2.28
\]

\[
\varepsilon_b = -\frac{\varepsilon_s}{2} = -1.14
\]

\[
\theta = \frac{\alpha_{f2} \cdot T_2 - \alpha_{f1} \cdot T_1}{e} = \frac{1.2 \cdot 10^{-5} \frac{1}{K} \cdot 20^\circ C - 1.2 \cdot 10^{-5} \frac{1}{K} \cdot -20^\circ C}{6.529 cm} = 0.735 \cdot 10^{-4} \frac{kN}{cm}
\]

For parameters \(\alpha, \beta\) and \(\lambda\), see above. The characteristic values of support reaction are

\[
T_{k_{\perp,1}} = T_{k_{\perp,2}} = T_{k_{\perp,3}} = \varepsilon_6 \cdot \frac{\theta \cdot B_S}{L} = -1.14 \cdot \frac{0.735 \cdot 10^{-4} \frac{kN}{cm} \cdot 2474198 \frac{kNcm^2}{m}}{3300 cm} = -0.629 \frac{kN}{m}
\]

\[
T_{k_{\perp,3}} = \varepsilon_5 \cdot \frac{\theta \cdot B_S}{L} = 2.28 \cdot \frac{0.735 \cdot 10^{-4} \frac{kN}{cm} \cdot 2474198 \frac{kNcm^2}{m}}{3300 cm} = 1.259 \frac{kN}{m}
\]
Summary
Transverse support reactions are summarised in the following table.

<table>
<thead>
<tr>
<th>Support</th>
<th>( G_k )</th>
<th>( W_k )</th>
<th>( S_k )</th>
<th>( T_{k,\text{Summer}} )</th>
<th>( T_{k,\text{Winter}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.212</td>
<td>-0.689</td>
<td>0.946</td>
<td>0.629</td>
<td>-0.629</td>
</tr>
<tr>
<td>2</td>
<td>0.651</td>
<td>1.249</td>
<td>2.907</td>
<td>-1.259</td>
<td>1.259</td>
</tr>
<tr>
<td>3</td>
<td>0.212</td>
<td>-0.444</td>
<td>0.946</td>
<td>0.629</td>
<td>-0.629</td>
</tr>
</tbody>
</table>

C.3.4 Design values of support reactions
Different combinations need to be checked:

**A Maximum shear force at support No. 1:**
\[
V_{Ed,1} = \gamma_F \cdot G_k + \gamma_F \cdot S_k = 1.35 \cdot 0.190 \frac{kN}{m} + 1.5 \cdot 0.845 \frac{kN}{m} = 1.524 \frac{kN}{m}
\]
\[
N_{Ed,1} = 0.0 \frac{kN}{m}
\]

**B Maximum tensile force at support No. 1:**
\[
V_{Ed,1} = \gamma_F \cdot G_k = 1.00 \cdot 0.190 \frac{kN}{m} = 0.190 \frac{kN}{m}
\]
\[
N_{Ed,1} = \gamma_F \cdot G_k + \gamma_F \cdot W_k + \gamma_F \cdot \psi_{0T} \cdot T_{k,\text{Winter}}
\]
\[
= 1.00 \cdot 0.212 \frac{kN}{m} + 1.5 \cdot -0.689 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -0.629 \frac{kN}{m} = -1.388 \frac{kN}{m}
\]

**C Maximum tensile force at support No. 2:**
\[
N_{Ed,2} = \gamma_F \cdot G_k + \gamma_F \cdot T_{k,\text{Summer}} + \gamma_F \cdot \psi_{0W} \cdot W_k
\]
\[
= 1.00 \cdot 0.651 \frac{kN}{m} + 1.5 \cdot -1.259 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -1.249 \frac{kN}{m} = -2.362 \frac{kN}{m}
\]

**D Maximum tensile force at support No. 3:**
\[
N_{Ed,3} = \gamma_F \cdot G_k + \gamma_F \cdot T_{k,\text{Winter}} + \gamma_F \cdot \psi_{0W} \cdot W_k
\]
\[
= 1.00 \cdot 0.212 \frac{kN}{m} + 1.5 \cdot -0.629 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -0.444 \frac{kN}{m} = -1.131 \frac{kN}{m}
\]
C.3.5 Head deflection

Finally head deflection is checked. Analytical calculation of head deflection for multi-span panels is based on method of consistent deformations. In a first step, head deflection at end supports is calculated for the single span panel with $L_{eqv} = 2L = 660$ cm and a temperature difference of $\Delta T = T_2 - T_1 = -70$ K. As bending stiffness of the profiled face has to be considered, calculation is more complex. For background of stiffness values see references.

$$
\beta_{2L} = \frac{B_s}{S \cdot L_{eqv}^2} = \frac{2474198 \cdot kN \cdot cm^2}{197 \cdot kN \cdot m \cdot (660 \text{ cm})^2} = 0.0293
$$

$$
\alpha = \frac{B_{f1} + B_{f2}}{B_s} = \frac{8.858 \cdot cm^4 \cdot 21000 \cdot kN \cdot cm^2 + 0}{2474198 \cdot kN \cdot cm^2} = 0.0752
$$

$$
\lambda_{2L} = \sqrt{\frac{1 + \alpha}{\alpha \cdot \beta_{2L}}} = \sqrt{\frac{1 + 0.0752}{0.0752 \cdot 0.0293}} = 22.3
$$

$$
\theta = \frac{\alpha_{f,2} \cdot T_2 - \alpha_{T,1} \cdot T_1}{e} = \frac{\alpha_{f} \cdot (T_2 - T_1)}{e} = \frac{1.2 \cdot 10^{-5} \cdot \frac{1}{K} \cdot -70K}{6.529 \text{ cm}} = -1.29 \cdot 10^{-4} \frac{1}{\text{cm}}
$$

Shear strain due to elastic shear deformation in the core is

$$
\gamma'_T = -\frac{\theta \cdot L_{eqv}}{\lambda_{2L}} \cdot \tanh \left( \frac{\lambda_{2L}}{2} \right) = -\frac{1.294 \cdot 10^{-4} \cdot \frac{1}{\text{cm}} \cdot 660 \text{ cm}}{22.3} \cdot \tanh \left( \frac{22.3}{2} \right) = 3.81 \cdot 10^{-3}
$$

The bending rotation at the support is

$$
\gamma'_{1T} = \frac{\theta \cdot L_{eqv}}{1 + \alpha} \left[ \frac{1}{2} - \frac{1}{\lambda_{2L}} \cdot \tanh \left( \frac{\lambda_{2L}}{2} \right) \right]
$$

$$
= -1.294 \cdot 10^{-4} \frac{1}{\text{cm}} \cdot 660 \text{ cm} \cdot \left[ \frac{1}{2} - \frac{1}{22.3} \cdot \tanh \left( \frac{22.3}{2} \right) \right] = -3.59 \cdot 10^{-2}
$$
Deflection at mid of span caused by temperature difference is
\[
w_T = \frac{\theta \cdot L_{eq}^2}{8 \left[ 1 + \frac{8}{\lambda_{2L}} \left( 1 - \frac{1}{\cosh(\lambda_{2L}/2)} \right) \right]}
= -1.29 \cdot 10^{-4} \frac{1}{cm} \cdot \left( 660 cm \right)^2 \cdot \frac{1}{8 \left[ 1 + 0.0752 \left( 22.3 \right)^2 \left( 1 - \frac{1}{\cosh(22.3/2)} \right) \right]} = 6.41 cm
\]
Deflection at mid of span caused by a line load is
\[
w_F = \frac{F \cdot L_{eq}^3}{\left( B_s + B_D \right) \left[ \left( 1 - \frac{8}{\lambda_{2L}} \right) \left( 1 - \frac{1}{\cosh(\lambda_{2L}/2)} \right) \right]}
= \frac{24749197 kN cm^2}{m} + 21000 kN \cdot 8.858 cm^4 \cdot \frac{1}{660 cm} \cdot \frac{1}{4 \cdot 0.0752 \left( 22.3 \right)^2 \left( 1 - \frac{1}{\cosh(22.3/2)} \right)} \cdot \frac{1}{8 \cdot 0.0752 \left( 22.3 \right)^3 \left( 1 - \frac{1}{\cosh(22.3/2)} \right)}
= \frac{2.20 kN}{m}
\]
Shear strain due to elastic shear deformation in the core is
\[
\gamma_A = \frac{F \cdot L_{eq}^2 \cdot \beta_{2L}}{\left( B_s + B_D \right) \left( 1 - \varepsilon + \frac{\sinh(\lambda_{2L} \cdot (1 - \varepsilon))}{\sinh \lambda_{2L}} \right) \left( \frac{1}{16} - \frac{1}{\lambda_{2L}} \left( 1 - \frac{1}{2 \cdot \tanh(\lambda_{2L}/2)} \right) \right)}
= \frac{2.20 kN \cdot \left( 660 cm \right)^2}{24749198 kN cm^2 + 21000 kN \cdot 8.858 cm^2 \cdot \frac{0.0288 \cdot \left( 1 - 0.5 + \frac{\sinh(22.3 \cdot (1 - 0.5))}{\sinh 22.3} \right)}{m}}
= 5.19 \cdot 10^{-3}
\]
The bending rotation at the support is
\[
\gamma_{BF} = \frac{F \cdot L_{eq}^2 \cdot \alpha_{2L}}{\left( B_s + B_D \right) \left[ 1 + \frac{8}{\lambda_{2L}} \left( 1 - \frac{1}{\cosh(\lambda_{2L}/2)} \right) \right]}
= \frac{2.05 kN \cdot \left( 660 cm \right)^2}{24749194 kN cm^2 + 21000 kN \cdot 8.858 cm^2 \cdot \frac{0.0288 \cdot \left( 1 - 0.5 + \frac{\sinh(22.3 \cdot (1 - 0.5))}{\sinh 22.3} \right)}{m}}
= 2.74 \cdot 10^{-2}
The angles $\gamma_1$ resulting from shear deformation of the core at end support and $\gamma$ resulting from bending rotation have to be subtracted from each other. Head deflection is finally calculated using

$$u = \gamma_1 \cdot D_F - \gamma \cdot e = (\gamma_{\text{IF}} + \gamma_{\text{IF}}) \cdot D_F - (\gamma_1 + \gamma) \cdot e$$

$$= (-3.59 \cdot 10^{-2} + 2.74 \cdot 10^{-2}) \cdot 6.0 - (3.18 \cdot 10^{-3} + 5.19 \cdot 10^{-3}) \cdot 6.529 \cdot m = -0.110 \cdot m$$

with distance $D_F$ between supporting structure and head of fastener, i.e. $D_F = d$ because of fastening in the trough.

### C.3.6 Resistance and Verifications

**Pull-through resistance**

The panel will be fastened by crest fastenings without a saddle washer. Self-tapping screws 6.3 x L made of stainless steel and washers $d_W = 19$ mm with cured-on elastomer seals were used.

External face is made from structural steel S320GD+Z275 ($f_{\text{u,F1}} = 390$ N/mm²) according to EN 10346, nominal thickness is $t_{\text{nom,F2}} = 0.60$ mm with special tolerance according to EN 10143.

Design core sheet thickness of the external face:

$$t_{F1} = t_{\text{nom,F1}} - t_{\text{zinc}} = 0.60 \text{mm} - 0.04 \text{mm} = 0.56 \text{mm}$$

Characteristic value of pull-through resistance:

$$N_{\text{Rk,F1}} = (2.21 \cdot \sqrt{E_{\text{Cc}}} \cdot f_{\text{Cc}} \cdot d_w^2 + 0.65 \cdot t_{F1} \cdot f_{\text{u,F1}} \cdot d_w) \cdot \left(0.55 + 0.45 \cdot e^\left(\frac{8 \cdot d_w}{\pi \cdot b_t}\right)^2\right)$$

$$= (2.21 \cdot \sqrt{3.5 \cdot \frac{N}{\text{mm}^2} \cdot 0.11 \cdot \frac{N}{\text{mm}^2} \cdot (19 \text{mm})^2 + 0.65 \cdot 0.56 \text{mm} \cdot 390 \cdot \frac{N}{\text{mm}^2} \cdot 19 \text{mm}} \cdot \left(0.55 + 0.45 \cdot e^\left(\frac{8 \cdot 19 \text{mm}}{\pi \cdot 35 \text{mm}}\right)^2\right)$$

$$= 1968 \text{N}$$

Design value of pull-through resistance

$$N_{\text{Rd,F1}} = \frac{N_{\text{Rk,F1}}}{\gamma_M} = 1968 \text{N} \cdot \frac{1}{1.33} = 1.48 \text{kN}$$

Verification, with three fasteners per meter width at base support and two fasteners per meter width at intermediate and top supports

$$\sum N_{\text{Rd,F1}} \frac{N_{\text{Ed,F1}}}{3 \cdot 1.48 \text{kN}} = 1.388 \text{kN} = 0.31 < 1.0$$

$$\sum N_{\text{Rd,F1}} \frac{N_{\text{Ed,F2}}}{2 \cdot 1.48 \text{kN}} = 2.362 \text{kN} = 0.80 < 1.0$$
\[ \frac{N_{Ed,3}}{\sum N_{Ed,1}} = \frac{N_{Ed,3}^{D}}{\sum N_{Ed,1}} = \frac{1.131kN}{2 \cdot 1.48kN} = 0.38 < 1.0 \]

**Pull-out resistance**

The characteristic value of pull-out resistance is taken from the ETA of the fastener, see Fig. C.8. As there are only characteristic values of tensile resistance (minimum value of pull-through resistance and pull-out resistance) given, pull-out resistance will be used for maximum thickness \( t_{nom,F1} \) (in ETAs often designated as \( t_{N,F1} \)).
ANNEX C: DESIGN EXAMPLES

Fig. C.8: Annex of the ETA with pull-out resistance
Characteristic value of pull-out resistance

\[ N_{R,k,II} = 5.00 \text{kN} \]

Design value of pull-out resistance

\[ N_{Rd,II} = \frac{N_{R,k,II}}{\gamma_M} = \frac{5.00 \text{kN}}{1.33} = 3.76 \text{kN} \]

Verification, with three fasteners per meter width at base support and two fasteners per meter width at intermediate and top supports

\[
\begin{align*}
\sum N_{Ed,1} & = \sum N_{Ed,II} = \frac{1.388 \text{kN}}{3 \cdot 3.76 \text{kN}} = 0.12 < 1.0 \\
\sum N_{Ed,2} & = \sum N_{Ed,II} = \frac{2.362 \text{kN}}{2 \cdot 3.76 \text{kN}} = 0.31 < 1.0 \\
\sum N_{Ed,3} & = \sum N_{Ed,II} = \frac{1.131 \text{kN}}{2 \cdot 3.76 \text{kN}} = 0.15 < 1.0 
\end{align*}
\]

Shear resistance

The characteristic values of shear resistance were taken from the ETA of the fastener, see Fig. C.9. The ETA does not differentiate between \( V_{Rd} \), \( V_{Rd,I} \) and \( V_{Rd,II} \). Values depend from sheet thickness of the supporting structure \( t_{II} \) (in ETAs often designating the nominal thickness) and sheet thickness of the internal face \( t_{nom,F2} \) (in ETAs often designated as \( t_{n,F2} \)) and cover both sheets.

Supporting structure is made from structural steel S235JR \( (f_{u,II} = 360 \text{ N/mm}^2) \) with thickness \( t_{nom,II} = 5.0 \text{ mm} \). Internal face is made from structural steel S320GD+Z275 \( (f_{u,F2} = 390 \text{ N/mm}^2) \) according to EN 10346, nominal thickness is \( t_{nom,F2} = 0.50 \text{ mm} \).

Characteristic value of shear resistance:

\[ V_{Rk} = 0.98 \text{kN} \]

Design value of shear resistance

\[ V_{Rd} = \frac{V_{Rk}}{\gamma_M} = \frac{0.98 \text{kN}}{1.33} = 0.74 \text{kN} \]

Verification, with three fasteners at support No. 1

\[
\sum V_{Ed,1} = \sum V_{Ed} = \frac{1.524 \text{kN}}{3 \cdot 0.74 \text{kN}} = 0.69 < 1.0
\]
ANNEX C: DESIGN EXAMPLES

Fig. C.9: Annex of the ETA with shear resistance
**Interaction**

Interaction verification has to be made for supporting structure only. While for $N_{Rd,II}$, a specific value was derived in this example, the ETA does not differentiate between $V_{Rd}$, $V_{Rd,1}$ and $V_{Rd,II}$.

Verification, with three fasteners at support No. 1

$$\frac{N_{Ed,1}}{N_{Rd,II}} + \frac{V_{Ed,1}}{V_{Rd,II}} = \frac{N_{Ed,1}^{\Delta}}{N_{Rd,II}^{\Delta}} + \frac{V_{Ed,1}^{\Delta}}{V_{Rd}^{\Delta}} = \frac{1.388kN}{3 \cdot 3.76kN} + \frac{0.190kN}{3 \cdot 0.74kN} = 0.12 + 0.09 = 0.21 < 1.0$$

**Allowable head deflection**

The allowable head deflection is taken from the ETA of the fastener, see Fig. C.8.

$\max u = 18mm$

Verification

$$\frac{u}{\max u} = \frac{0.11cm}{1.80cm} = 0.06 < 1.0$$
Fig. C.10: Annex of the ETA with allowable head deflection
C.4 Roof panel

C.4.1 System
Roof panel, lying over two spans with L = 3300 mm, with a slope of 10°. The panel has a profiled external face with pitch of profile p = 333 mm and width of the crest b₁ = 35 mm. The internal face is lightly profiled. The nominal thickness is 60 mm, the thickness at crest is D = 98 mm.

![Fig. C.11: Mechanical system](image)

External face is made from structural steel S320GD+Z275 (f_{u,F1} = 390 N/mm²) according to EN 10346, nominal thickness is t_{nom,F1} = 0.60 mm with special tolerance according to EN 10143. Internal face is made from structural steel S320GD+Z275 (f_{u,F2} = 390 N/mm²) according to EN 10346, nominal thickness is t_{nom,F2} = 0.50 mm with special tolerance according to EN 10143.

The core material is polyurethane foam with shear modulus G_C = 3.02 N/mm², compression modulus 3.5 N/mm² (not given in original examples) and compression strength 0.11 N/mm².

The panel will be fastened by crest fastenings, using washers d_w = 16 mm with cured-on elastomer seals (no saddle washers). At base support (support No. 1), there are three fasteners per panel of 1 m width and per support, with each crest fastened. At intermediate and top support (supports No. 2 and No. 3), there are two fasteners per panel of 1 m width and per support, with two crests fastened and one crest unfastened per panel. Self-drilling screws 6.0 x 140 mm made of stainless steel are used.

Supporting structure is a purlin made of glued laminated timber GL 24h according to EN 14080 in service class 2, characteristic value of density is ρ_k = 380 kg/m³. Shear forces from self-weight loading are transferred only by support No. 1, see Fig. C.11.

Chapter C.3 deals with the identical system and sandwich panel, but fastened on a supporting structure made of hot-rolled sections made from structural steel using a different type of fastener.
C.4.2 Actions and loads
See chapter C.3.2.

C.4.3 Characteristic values of support reactions
See chapter C.3.3 and summary in the following.

Transverse support reactions are summarised in the following table.

<table>
<thead>
<tr>
<th>Support</th>
<th>G_k</th>
<th>W_k</th>
<th>S_k</th>
<th>T_k,Summer</th>
<th>T_k,Winter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.212</td>
<td>-0.689</td>
<td>0.946</td>
<td>0.629</td>
<td>-0.629</td>
</tr>
<tr>
<td>2</td>
<td>0.651</td>
<td>-1.249</td>
<td>2.907</td>
<td>-1.259</td>
<td>1.259</td>
</tr>
<tr>
<td>3</td>
<td>0.212</td>
<td>-0.444</td>
<td>0.946</td>
<td>0.629</td>
<td>-0.629</td>
</tr>
</tbody>
</table>

All shear forces are transferred via support No. 1:

\[ G_{k,1} = 0.190 \frac{kN}{m} \]

\[ S_{k,1} = 0.845 \frac{kN}{m} \]

C.4.4 Design values of support reactions
Timber supporting structures require consideration of service class and load duration class. Due to the different modification factors \( k_{\text{mod}} \) linked with the action with the shortest duration of a given combination, some more combinations have to be checked. See following table for modification factors \( k_{\text{mod}} \) applied in the example.

<table>
<thead>
<tr>
<th>loading</th>
<th>Load duration class</th>
<th>( k_{\text{mod}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>self-weight G_k</td>
<td>permanent</td>
<td>0.60</td>
</tr>
<tr>
<td>temperature T_k</td>
<td>long-term</td>
<td>0.70</td>
</tr>
<tr>
<td>snow S_k</td>
<td>medium-term</td>
<td>0.80</td>
</tr>
<tr>
<td>wind W_k</td>
<td>short-term to instantaneous</td>
<td>1.00</td>
</tr>
</tbody>
</table>

If a load combination consists of actions belonging to different load-duration classes a value of \( k_{\text{mod}} \) should be chosen which corresponds to the action with the shortest duration, see EN 1995-1-1. In this particular case for a combination of
wind action and temperature action, a value of $k_{\text{mod}}$ corresponding to the wind action should be used, while for a combination of temperature action only, the smaller value of $k_{\text{mod}}$ corresponding to the temperature action applies.

For component I (sandwich panel) only combinations A, C, E and G with maximum shear force or maximum tensile force need to be verified.

### A Maximum shear force at support No. 1:

$$ V_{Ed,1} = V_{Ed,1}^A = \gamma_F \cdot G_k + \gamma_F \cdot S_k = 1.35 \cdot 0.190 \frac{kN}{m} + 1.5 \cdot 0.845 \frac{kN}{m} = 1.524 \frac{kN}{m} $$

$$ N_{Ed,1} = N_{Ed,1}^A = 0.0 \frac{kN}{m} $$

### B Shear force at support No. 1 with smallest $k_{\text{mod}}$:

$$ V_{Ed,1} = V_{Ed,1}^B = \gamma_F \cdot G_k = 1.35 \cdot 0.190 \frac{kN}{m} = 0.257 \frac{kN}{m} $$

$$ N_{Ed,1} = N_{Ed,1}^B = 0.0 \frac{kN}{m} $$

### C Maximum tensile force at support No. 1:

$$ V_{Ed,1} = V_{Ed,1}^C = \gamma_F \cdot G_k = 1.00 \cdot 0.190 \frac{kN}{m} = 0.190 \frac{kN}{m} $$

$$ N_{Ed,1} = N_{Ed,1}^C = \gamma_F \cdot G_k + \gamma_F \cdot W_k + \gamma_F \cdot \psi_T \cdot T_{k,Winter} $$

$$ = 1.00 \cdot 0.212 \frac{kN}{m} + 1.5 \cdot -0.689 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -0.629 \frac{kN}{m} = -1.388 \frac{kN}{m} $$

### D Tensile force at support No. 1 with smallest $k_{\text{mod}}$:

$$ V_{Ed,1} = V_{Ed,1}^D = \gamma_F \cdot G_k = 1.00 \cdot 0.190 \frac{kN}{m} = 0.190 \frac{kN}{m} $$

$$ N_{Ed,1} = N_{Ed,1}^D = \gamma_F \cdot G_k + \gamma_F \cdot T_{k,Winter} $$

$$ = 1.00 \cdot 0.212 \frac{kN}{m} + 1.5 \cdot -0.629 \frac{kN}{m} = -0.732 \frac{kN}{m} $$

### E Maximum tensile force at support No. 2:

$$ N_{Ed,2} = N_{Ed,2}^E = \gamma_F \cdot G_k + \gamma_F \cdot T_{k,Summer} + \gamma_F \cdot \psi_{w} \cdot W_k $$

$$ = 1.00 \cdot 0.651 \frac{kN}{m} + 1.5 \cdot -1.259 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -1.249 \frac{kN}{m} = -2.362 \frac{kN}{m} $$
ANNEX C: DESIGN EXAMPLES

**F Tensile force at support No. 2 with smallest $k_{mod}$:**

\[ N_{Ed,2} = N_{Ed,2}^F = \gamma_F \cdot G_k + \gamma_F \cdot T_{k,Summer} \]
\[ = 1.00 \cdot 0.651 \frac{kN}{m} + 1.5 \cdot -1.259 \frac{kN}{m} = -1.238 \frac{kN}{m} \]

**G Maximum tensile force at support No. 3:**

\[ N_{Ed,3} = N_{Ed,3}^G = \gamma_F \cdot G_k + \gamma_F \cdot T_{k,Summer} + \gamma_F \cdot \psi_{0w} \cdot W_k \]
\[ = 1.00 \cdot 0.212 \frac{kN}{m} + 1.5 \cdot -0.629 \frac{kN}{m} + 1.5 \cdot 0.6 \cdot -0.444 \frac{kN}{m} = -1.131 \frac{kN}{m} \]

**H Tensile force at support No. 3 with smallest $k_{mod}$:**

\[ N_{Ed,3} = N_{Ed,3}^H = \gamma_F \cdot G_k + \gamma_F \cdot T_{k,Summer} \]
\[ = 1.00 \cdot 0.212 \frac{kN}{m} + 1.5 \cdot -0.629 \frac{kN}{m} = -0.732 \frac{kN}{m} \]

---

**Fig. C.12: Support reactions**

**C.4.5 Head deflection**

See chapter C.3.5.

**C.4.6 Resistance and Verifications**

**Pull-through resistance**

The panel will be fastened by crest fastenings without a saddle washer. Self-drilling screws 6.0 x 140 mm made of stainless steel and washers $d_w = 16$ mm with cured-on elastomer seals were used.

External face is made from structural steel S320GD+Z275 ($f_{u,F1} = 390$ N/mm²) according to EN 10346, nominal thickness is $t_{nom,F2} = 0.60$ mm with special tolerance according to EN 10143.

Design core sheet thickness of the external face:

\[ t_{F1} = t_{nom,F1} - t_{zinc} = 0.60\text{mm} - 0.04\text{mm} = 0.56\text{mm} \]

Characteristic value of pull-through resistance:
Design value of pull-through resistance

\[ N_{Rd,I} = \frac{N_{Rk,I}}{\gamma_M} = \frac{1747N}{1.33} = 1.31kN \]

Verification, with three fasteners per meter width at base support and two fasteners per meter width at intermediate and top supports

\[
\sum N_{Rd,I} = \frac{1,388kN}{3 \cdot 1.31kN} = 0.35 < 1.0
\]
\[
\sum N_{Rd,I} = \frac{2,362kN}{2 \cdot 1.31kN} = 0.90 < 1.0
\]
\[
\sum N_{Rd,I} = \frac{1,131kN}{2 \cdot 1.48kN} = 0.43 < 1.0
\]

Pull-out resistance

Supporting structure is a purlin made of glued laminated timber GL 24h according to EN 14080, characteristic value of density is \( \rho_k = 380 \text{ kg/m}^3 \).

The characteristic withdrawal strength perpendicular to the direction of grain is unknown but can be calculated.

\[ f_{ax,k} = 70 \cdot 10^{-6} \cdot \rho_k^2 = 70 \cdot 10^{-6} \cdot \left( \frac{380 \text{ kg}}{\text{m}^3} \right)^2 = 10.1 \frac{N}{\text{mm}^2} \]

Effective length

\[ l_g = L - D = 140\text{mm} - 98\text{mm} = 42\text{mm} \]
\[ l_{ef} = l_g - l_b = 42\text{mm} - 5\text{mm} = 37\text{mm} \geq 5 \cdot d = 30\text{mm} \]

Load-bearing capacity for one fastener

\[ F_{ax,Rk} = f_{ax,k} \cdot l_{ef} \cdot \left( \frac{\rho_k}{350 \frac{\text{kg}}{\text{m}^3}} \right)^{0.8} = 10.1 \frac{N}{\text{mm}^2} \cdot 6.0\text{mm} \cdot 37\text{mm} \cdot \left( \frac{380 \text{kg}}{\text{m}^3} \right)^{0.8} = 2.39kN \]

Characteristic value of pull-out resistance

\[ N_{Rk,II} = F_{ax,Rk} \cdot k_{mod} \]
Design value of pull-out resistance

\[ N_{rd,ll} = \frac{N_{Rk,II}}{\gamma_M} = \frac{F_{ax,Rk} \cdot k_{mod}}{\gamma_M} \]

Verification, with three fasteners per meter width at base support and two fasteners per meter width at intermediate and top supports

\[ C \text{ Maximum tensile force at support No. 1:} \]

\[ N_{Ed,1} = \sum N_{rd,II} = \sum F_{ax,Rk} \cdot k_{mod} = \frac{1.338kN}{3 \cdot 2.39kN \cdot 1.00} = 0.25 < 1.0 \]

\[ D \text{ Tensile force at support No. 1 with smallest } k_{mod}: \]

\[ N_{Ed,1} = \sum N_{rd,II} = \sum F_{ax,Rk} \cdot k_{mod} = \frac{0.732kN}{3 \cdot 2.39kN \cdot 0.70} = 0.19 < 1.0 \]

\[ D \text{ Maximum tensile force at support No. 2:} \]

\[ N_{Ed,2} = \sum N_{rd,II} = \sum F_{ax,Rk} \cdot k_{mod} = \frac{2.362kN}{2 \cdot 2.39kN \cdot 1.00} = 0.66 < 1.0 \]

\[ E \text{ Tensile force at support No. 2 with smallest } k_{mod}: \]

\[ N_{Ed,2} = \sum N_{rd,II} = \sum F_{ax,Rk} \cdot k_{mod} = \frac{1.238kN}{2 \cdot 2.39kN \cdot 0.70} = 0.49 < 1.0 \]

\[ G \text{ Maximum tensile force at support No. 3:} \]

\[ N_{Ed,3} = \sum N_{rd,II} = \sum F_{ax,Rk} \cdot k_{mod} = \frac{1.131kN}{2 \cdot 2.39kN \cdot 1.00} = 0.31 < 1.0 \]

\[ H \text{ Tensile force at support No. 3 with smallest } k_{mod}: \]

\[ N_{Ed,3} = \sum N_{rd,II} = \sum F_{ax,Rk} \cdot k_{mod} = \frac{0.732kN}{2 \cdot 2.39kN \cdot 1.00} = 0.29 < 1.0 \]

Shear resistance

At base support there are two screws per panel of 1 m width.

Dimensions of self-drilling screws 6.0 x L made of stainless steel:

\[ d = 6.0 \text{ mm} \]
\[ d_1 = 3.9 \text{ mm} \]

\[ d_{ef} = 1.1 \cdot d_1 = 1.1 \cdot 3.9 \text{ mm} = 4.3 \text{ mm} \]
Internal face is made from structural steel S320GD+Z275 ($f_{u,F2} = 390 \text{ N/mm}^2$) according to EN 10346, nominal thickness is $t_{\text{nom,F2}} = 0.50 \text{ mm}$ with special tolerance according to EN 10143.

Design core sheet thickness of the internal face:

$$t_{F2} = t_{\text{nom,F2}} - t_{\text{inc}} = 0.50 \text{ mm} - 0.04 \text{ mm} = 0.46 \text{ mm}$$

Characteristic value of shear resistance:

$$V_{Rk,t} = 4.2 \cdot \sqrt{t_{F2}^3 \cdot d_t \cdot f_{u,F2}} = 4.2 \cdot \sqrt{(0.46 \text{ mm})^3 \cdot 3.9 \text{ mm} \cdot 390 \frac{N}{\text{mm}^2}} = 1009 \text{ N}$$

Design value of shear resistance

$$V_{Rd} = V_{Rk,t} \frac{N}{\gamma_M} = \frac{1009 \text{ N}}{1.33} = 0.76 \text{ kN}$$

Verification, with three screws at base support

$$\sum V_{Rd} = \frac{1.524 \text{ kN}}{3 \cdot 0.76 \text{ kN}} = 0.67 < 1.0$$

Supporting structure is a purlin made of glued laminated timber GL 24h according to EN 14080, characteristic value of density is $\rho_k = 380 \text{ kg/m}^3$.

The characteristic embedment strength for fastenings with predrilled holes (including fastenings with self-drilling screws) is calculated with

$$f_{h,k} = 0.082 \cdot \left(1 - 0.01 \cdot d_{ef}\right) \cdot \rho_k = 0.082 \cdot \left(1 - 0.01 \cdot 4.3 \text{ mm}\right) \cdot 380 \frac{\text{ kg}}{\text{ m}^3} = 29.8 \frac{N}{\text{ mm}^2}$$

The characteristic yield moment of fastener is unknown but can be calculated.

$$M_{y,Rk} = 0.3 \cdot f_{u,SG} \cdot d_{ef}^2 = 0.3 \cdot 500 \frac{N}{\text{ mm}^2} \cdot (4.3 \text{ mm})^2 = 6654 \text{ Nmm}$$

Load-bearing capacity for one fastener

$$F_{V,Rk} = \min \left\{ \frac{0.4 \cdot f_{h,k} \cdot t_1 \cdot d_{ef}}{1.15 \cdot \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,k} \cdot d_{ef} + \frac{F_{ax,Rk}}{4}}} \right\}$$

$$= \min \left\{ \frac{0.4 \cdot 29.8 \frac{N}{\text{ mm}^2} \cdot 37 \text{ mm} \cdot 4.3 \text{ mm}}{1.15 \cdot \sqrt{2 \cdot 6654 \text{ Nmm} \cdot 29.8 \frac{N}{\text{ mm}^2} \cdot 4.3 \text{ mm} + 2.39 \text{ kN}^2}} \right\} = \min \left\{ 1.90 \text{ kN} \right\}$$

$$= \min \left\{ 1.90 \text{ kN} \right\}$$

The limitation

$$\frac{F_{ax,Rk}}{4} \leq 1.15 \cdot \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,k} \cdot d_{ef}}$$

is fulfilled.
The characteristic value of shear resistance of the timber supporting structure (component II) is

\[ V_{Rk,II} = F_{V,Rk} \cdot k_{mod} \]

Design value of shear resistance

\[ V_{Rd,II} = \frac{V_{Rk,II}}{\gamma_M} \]

For a combination with wind action being the action with shortest load duration

\[ V_{Rk,II} = F_{V,Rk} \cdot k_{mod} = 1.90kN \cdot 1.0 = 1.90kN \]

\[ V_{Rd,II} = \frac{1.90kN}{1.33} = 1.43kN \]

For a combination with temperature action being the action with shortest load duration

\[ V_{Rk,II} = F_{V,Rk} \cdot k_{mod} = 1.90kN \cdot 0.8 = 1.52kN \]

\[ V_{Rd,II} = \frac{1.52kN}{1.33} = 1.14kN \]

For a combination with snow action being the action with shortest load duration

\[ V_{Rk,II} = F_{V,Rk} \cdot k_{mod} = 1.90kN \cdot 0.7 = 1.33kN \]

\[ V_{Rd,II} = \frac{1.33kN}{1.33} = 1.00kN \]

For a combination with self-weight action being the action with shortest load duration

\[ V_{Rk,II} = F_{V,Rk} \cdot k_{mod} = 1.90kN \cdot 0.6 = 1.14kN \]

\[ V_{Rd,II} = \frac{1.14kN}{1.33} = 0.86kN \]

Verification, with three fasteners at base support

A  Maximum shear force at support No. 1:

\[ \sum V_{Ed,II} = \sum \frac{F_{V,Rk} \cdot k_{mod}^A}{\gamma_M} = \frac{1.524kN}{3} \cdot \frac{1.14kN \cdot 0.80}{1.33} = 0.74 < 1.0 \]

B  Shear force at support No. 1 with smallest \( k_{mod} \):

\[ \sum V_{Ed,II} = \sum \frac{F_{V,Rk} \cdot k_{mod}^B}{\gamma_M} = \frac{0.257kN}{3} \cdot \frac{1.14kN \cdot 0.60}{1.33} = 0.17 < 1.0 \]

Interaction
Interaction verification has to be made for supporting structure only.

Verification, with three fasteners at base support

**C Maximum tensile force at support No. 1:**

\[
\frac{N_{Ed,1}}{\sum N_{Rd,H}} + \frac{V_{Ed,1}}{\sum V_{Rd,H}} = \frac{N_{Ed,1}^C}{\sum \frac{F_{ax,Rk} \cdot k_{mod}^C}{\gamma_M}} + \frac{V_{Ed,1}^C}{\sum \frac{F_{V,Rk} \cdot k_{mod}^C}{\gamma_M}}
\]

\[
= \frac{1.338kN}{3 \cdot 2.39kN \cdot 1.00} + \frac{0.190kN}{3 \cdot 1.14kN \cdot 1.00} = 0.25 + 0.07 = 0.32 < 1.0
\]

**D Tensile force at support No. 1 with smallest \(k_{mod}\):**

\[
\frac{N_{Ed,1}}{\sum N_{Rd,H}} + \frac{V_{Ed,1}}{\sum V_{Rd,H}} = \frac{N_{Ed,1}^D}{\sum \frac{F_{ax,Rk} \cdot k_{mod}^D}{\gamma_M}} + \frac{V_{Ed,1}^D}{\sum \frac{F_{V,Rk} \cdot k_{mod}^D}{\gamma_M}}
\]

\[
= \frac{0.732kN}{3 \cdot 2.39kN \cdot 0.70} + \frac{0.190kN}{3 \cdot 1.14kN \cdot 0.70} = 0.19 + 0.10 = 0.30 < 1.0
\]

**Allowable head deflection**

The allowable head deflection is taken from the ETA of the fastener, see Fig. C.8.

\[\max u = 0.07 \cdot d_c = 0.07mm \cdot 60mm = 4.2mm\]

Verification

\[\frac{u}{\max u} = \frac{0.11cm}{0.42cm} = 0.26 < 1.0\]
ANNEX C: DESIGN EXAMPLES