Preliminary European Recommendations for the Design of Sandwich Panels with Openings
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PRELIMINARY EUROPEAN RECOMMENDATIONS
FOR THE DESIGN OF SANDWICH PANELS WITH OPENINGS

Preface

This report includes current information about the influence of openings on the behaviour and resistance of sandwich panels. With this report, it is intended to complete the directions given in the European product standard EN 14509, which studies the sole complete sandwich panels and does not give any guidance for the design or cutting of openings. The report has been drafted for its use in design but also for the use in forming and placing openings in practice.

This report introduces technical information such as calculation models and experimental arrangements concerning the influence of the openings as well as useful practical directions based on the experience and guidance from companies. Background information covering rules, expressions and knowledge from practice is given in note boxes.

This report introduces aspects on the mechanical design of sandwich panels. Most of the information concerns openings in flat-faced sandwich panels exposed to short-term loading, i.e., openings in wall panels. However, the information can be also applicable to the design of roof panels. The report does not consider subjects such as thermal insulation and leakages and air and water tightness, which may be also equally important items when making a design of sandwich panels with openings. The information given in the report is based on research and experience of metal sheet faced sandwich panels. In some countries there may be national regulations, which are different compared to the recommendations given in this report. The national regulations shall be respected.

This report has been prepared by the European Joint Committee on Sandwich Constructions, consisting of the Technical Working Group ECCS TWG 7.9 and of CIB Commission W56. The following persons cooperated on the drafting of the report:

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This report includes the results created in the EC funded research project EASIE. It includes still several items necessitating more information. However, the Joint Committee wants to publish this state-of-the-art report believing that it will support designers, manufacturers, installers and authorities to understand the risks caused by openings and further, to see the possibilities to verify and if needed, to reinforce the panels with openings and further, show ways to develop more optimal systems for different purposes and cases. Based on practical experience and on future technical and scientific sources of information, this state-of-the-art report will be updated and shall be published later as European Recommendation for sandwich panels.

Compared to the first edition, in the present second edition, editorial corrections were made.
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Annex A2  Allowable size and position of a small opening
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Annex C   Example on the verification of the resistance of a panel with openings and additional frame structures
1 Introduction

1.1 General

Self-supporting sandwich panels are known building components used to cover external and internal walls and roofs. Most facades of buildings include openings due to doors, windows, HVAC (heating, ventilation and air conditioning) lines and other technical inlets. Thus, also the covering components in facades are often penetrated and cut with openings of different sizes and geometries.

The European product standard EN 14509 gives guidance for manufacturing and designing complete factory-made sandwich panels. It does not include any information about the influence of the openings and detailing in the static resistance behaviour of panels. A partial reason may be the fact that many of the openings are made on a building site, thus being outside of control of the phase of manufacturing. This report includes current information about the influence of the openings on the behaviour and resistance of sandwich panels. The objective of this report is to complete the directions given in the European product standard EN 14509. Thus, the report may give useful information for the design of sandwich panels but also for cutting, forming and placing of openings in practice.

This report introduces technical information such as calculation models and experimental arrangements being based on research reports and reviewed publications, and also useful practical directions based on the experience and guidance from companies in practice. Useful background information and limitations covering rules, expressions and knowledge from practice is presented in note boxes. This report mostly covers mechanical design aspects, only. Most of the information concerns openings in flat metal sheet faced sandwich panels exposed to short-term loading, i.e., openings in wall panels, and further, openings in panels having a PU- or EPS-foam or a mineral wool core layer. However, with special consideration of the long-term actions and the point loads, the information may be also applicable to the design of roof and ceiling panels. Building physical aspects such as thermal insulation and air and water tightness are outside of the scope of the report.

This report includes several points necessitating more information. The open points at the moment of preparing this report concern the design of the openings in panels having a strongly profiled face, influence of eccentric openings to the mid-line of the panels, design of reinforcements around and outside the openings and further the influence of long-term loads. The use of these instructions shall be limited to panels loaded by static short-term loads (wind, short-term thermal loads, no axial loads). Based on the results of the new research projects and further experience from practice, the report shall be updated later.

1.1.1 State of the art

Windows, doors and different technical inlets in typical walls of buildings obviously necessitate openings in the wall panels. The openings reduce the structural cross-section of the faces and the core of the sandwich panels, and thus, may have influences on the load-bearing capacity of the panels.

The "official" state of the art concerning the design of walls made of self-supporting sandwich panels is a "replacement", which means strengthening the panels with an additional
support in the area of the opening. According to this principle, all applied loads on the window and door openings shall be transferred to the spaced structural supports, e.g. framework, by longitudinal beams and cross beams (Fig. 1). The replacing concept always results in additional structural components and is a substantial effort in design and construction. According to the present technical information, additional supports are not always needed, which is also often in agreement with requirements of the current visual and architectural appearance.

Note:
In Nordic countries, especially in Finland, since the late 1980s it has been a common practice to transfer loads from panels with openings to adjacent panels. The design has been based on instructions given by the panel manufacturer. This procedure has been successfully used and significantly reduced the need for additional supporting structures.

The first scientific document on the influence of openings is probably the analysis presented by T. Höglund in IABSE Colloquium in 1986. Based on test results of PU-foam cored sandwich panels he introduced a model for the torsional stiffness of a flat faced cross-section of a sandwich panel. He derived design expressions from the distribution of the loads to the adjacent panels close to the panel with openings. The analysis is based on the compatibility of deflections at the longitudinal joints between the panels, assuming that the deflections caused by bending, shear and torsion are affine.

Based upon experimental and numerical studies, Toma and Courage (1994) presented models to consider the effect of an opening into the wrinkling strength of the face and the shear resistance of the core. In the expression for the wrinkling strength the effect of the stress concentration in the corner of the opening is included. They also introduced a model to calculate the increased deflection of a one-span flat faced sandwich panel with an opening. The model applies for openings located symmetrically and centrically to the mid-point of the sandwich panels.
Heselius (2004) performed numerical studies on the effects of the bond strength between the face and core, and the out-of-flatness on the wrinkling failure. In addition, he studied the development of the wrinkling failure in the edge of openings. The result of the work is the combination of the reduction factor $k_2$ and the model developed by Toma and Courage to describe the wrinkling failure of a face close to an opening also taking into account the tensile strength of core and bond.

Böttcher (2005) and Lange & Böttcher (2006) studied deformations and stresses of one-span and multi-span sandwich panels with an opening placed in different locations using a software based on finite beam elements. The model was also used in the evaluation of the load distributions to adjacent sandwich panels. The results of the analysis were verified experimentally. In addition, an experimental method was developed to measure the shear stiffness and resistance of the longitudinal joint.

Berner (2006) has studied and developed reinforcement structures placed around the openings and in the longitudinal joints between the panels. The work has resulted in principal structural solutions and procedures to evaluate the strength and resistance of the reinforcements.

The EC funded research project EASIE has created new knowledge about the load transfer to adjacent panels and about the influence of the openings in sandwich panels with profiled faces (Rädel & Lange 2011). Furthermore, the experimental and computational research has resulted in models to analyse the influence of window frames installed in the openings. The new information covers Section 3.1.4, part of Section 3.2.3.1 and Annex C.

The report introduces new possibilities to make the design of sandwich panels with openings. The following aims are promoted in the report.

- Sandwich panels have an adequate load bearing capacity and an allowable span also with openings.
- Additional supports shall be installed only in those cases in which the supports are really needed.
- Careful analyses shall be made for stresses covering all load cases as well as for the strength of the cross-sections in the area of openings.

1.1.2 Layouts and types of openings

Typical layouts and alternatives of the openings depend on the intended use of the opening and may differ in the following ways.

- Size of the opening (Fig. 2a)
- Location of the opening (Fig. 2b)
- Direction of the span of the panels (Fig. 2c and d)
- Geometrical form of the opening (Fig. 2e)

Based on various arrangements shown in Fig 2, it is obvious, that the structural behaviour and resistance of the panels are varying in different cases which have to be taken into account for the structural design. In the following chapters, three possibilities and their feasibility to design sandwich panels with openings are presented in principle. The first possibility covers the evaluation of the remaining load bearing capacity of the panels with open-
ings. In the second case, the possibilities to transfer the load or a part of the load to adjacent panels are studied. The third possibility is to carry the loads using additional beams and frames.

Fig. 2: Openings in sandwich wall panels may vary a) in size and b) in location. Openings may be cut in panels c) installed in vertical direction or d) in horizontal direction. e) Openings may have very different geometrical forms.

1.2 Symbols and notations

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>overall width of the panel</td>
</tr>
<tr>
<td>BS</td>
<td>bending stiffness</td>
</tr>
<tr>
<td>D</td>
<td>overall depth of the panel</td>
</tr>
<tr>
<td>E</td>
<td>modulus of elasticity</td>
</tr>
<tr>
<td>GC</td>
<td>shear modulus of the core</td>
</tr>
<tr>
<td>GF</td>
<td>shear modulus of the face</td>
</tr>
<tr>
<td>I</td>
<td>moment of inertia</td>
</tr>
<tr>
<td>L</td>
<td>span</td>
</tr>
<tr>
<td>M</td>
<td>bending moment</td>
</tr>
</tbody>
</table>
Mo bending moment in the area of an opening
M_{Rd} bending resistance of the panel without openings
M_T torsional moment
S shear stiffness
V shear force
V_S torsional stiffness
b width of the opening inside a panel
d depth of face profile or stiffeners
e_C distance between the centroids of the faces
e_o eccentricity of the mid-line of the opening to the mid-line of the cross-section
f_{CV} shear strength of the core
f_{Ct} tensile strength of the core
f_2, f_3 parameters used in adjustment of the test results
k parameter, correction factor
k_C reduction factor of shear stress
k_F reduction factor of wrinkling stress
k_2 reduction factor due to low cross-panel tensile strength
k_{F, eccentricity} reduction factor due to the eccentricity of the opening in a sandwich panel
q load, shear load in the longitudinal joint
t thickness of face sheet
\tau local shear stress

Subscripts
F face
C core
M material
R resistance
T torsion
d design
w wrinkling
c centroid, compression
t tension
### 1.3 Definitions

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>additional support</td>
<td>support of sandwich panel, which is needed additionally because of the reduced resistance due to the opening</td>
</tr>
<tr>
<td>core</td>
<td>layer of material, having thermal insulating properties, which is bonded between two metal faces</td>
</tr>
<tr>
<td>cut-out</td>
<td>an activity and a series of operations to make an opening in a sandwich panel</td>
</tr>
<tr>
<td>face layer</td>
<td>flat, lightly profiled or profiled thin metal sheet firmly bonded to the core</td>
</tr>
<tr>
<td>fastening</td>
<td>a point of connection between the sandwich panel and its supporting framework or a point of connection in a face of a sandwich panel</td>
</tr>
<tr>
<td>nominal thickness</td>
<td>thickness given in the product information table</td>
</tr>
<tr>
<td>opening</td>
<td>opening may have a shape of a rectangular, circular or non-regular hole, which perforates the face and core of one or more sandwich panels</td>
</tr>
<tr>
<td>reinforcement</td>
<td>stiffening frame around the opening or around the sandwich panel</td>
</tr>
<tr>
<td>remaining cross-section</td>
<td>gross cross-section minus the cross-section of the opening</td>
</tr>
<tr>
<td>replacement</td>
<td>resistance of the sandwich panel with opening is neglected and the loads are carried through additional structures</td>
</tr>
<tr>
<td>sandwich panel</td>
<td>building product consisting of two metal faces positioned on either side of a core and firmly bonded to each other so as to act compositely under load</td>
</tr>
<tr>
<td>small opening</td>
<td>opening that is located in one panel only and that fulfils the requirement: minimum distance to the longitudinal edges is smaller than smallest of 100 mm or 0.1B (B is overall width of the panel)</td>
</tr>
<tr>
<td>steel core thickness</td>
<td>thickness of the face layer all coatings, both metallic and organic, removed. In case of zinc-coated steel faces, the steel core thickness is the nominal thickness of the zinc-coated steel minus the thickness of zinc layers.</td>
</tr>
<tr>
<td>sub-structure</td>
<td>structure into which the fasteners of the sandwich panel are fixed</td>
</tr>
</tbody>
</table>
1.4  Design aspects

In many cases, the openings shall be located in places, in which the sandwich panel has a resistance reserve of a variable degree. The reserve may be caused by several reasons;

- opening is not located in a most stressed part of the panel,
- the panel is not located in the corner or on the top floor of the building, where the loads are typically at the highest and
- the depth of the panel has been determined by another requirement such as the thermal insulation power.

Regardless of the reason for the resistance reserve, the degree of the utilization of the strength of the face and core shall be taken into account when evaluating the resistance of the panel with openings.
2. Flow chart for design

In the structural design of sandwich panels with openings, the possibilities and steps of design may be useful as described in Fig. 3.

![Flow chart for design](image_url)

Fig. 3a: Decision paths of the design of sandwich panels with openings.
<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
</tr>
</thead>
</table>
| 0       | Evaluation of the panel resistance without openings  
- use the standardized methods given in EN 14509 and in the complementary documents |
| 1       | If the opening is small, evaluate the resistance of the sandwich panel with a small opening  
- use the expressions to evaluate the resistance of the remaining cross-section of the panel to carry the loads by itself |
| 2       | If the opening, being not that small, is located in a sandwich panel or if the resistance of the panel with a small opening is not adequate, evaluate and verify experimentally the possibilities to transfer loads to adjacent panels and the resistance of the adjacent panels to carry the additional loads  
- use the analytical and/or experimental methods to evaluate the resistance of the longitudinal joints and the resistance of the adjacent panels to carry the additional load |
| 3       | If the opening, being not that small, is located in one panel or if the resistance of the panel with a small opening is not adequate, and it is not possible to transfer loads to adjacent panels, evaluate the applicability of the reinforcement structure placed around the opening and in longitudinal joints of the panel  
- use the analytical and/or experimental methods to evaluate the resistance and compatibility of the reinforcement structure to carry partially or totally the loads exposed to the area of the opening or to the whole sandwich panel |
| 4       | If the opening, being not that small, is located in one panel or if the resistance of the panel with a small opening is not adequate, and it is not possible to transfer loads to adjacent panels or to place a reinforcement structure around the opening or in the longitudinal joints of the panel, or if the opening extends to more than one panel, design a substructure outside the panel to carry the total loads of the panel according to relevant standards, or modify the location or the size of the opening  
- use the methods given in relevant standards to design the substructure to carry the whole load exposed to the sandwich panel |
| 5       | Installation and erection  
- apply the directions with care in the system in question |

Thermal insulation power, influence of thermal bridges and air and water tightness shall be verified not covered in the report

Fig. 3b: Instructions given in the state-of-the-art report.
3 Resistance of panels with openings
3.1 Resistance of panels with small openings
3.1.1 Principles

In this report, the term “small opening” is used to describe openings, the width of which is smaller than the width of the sandwich panel. A small opening shall neither cut nor meet the longitudinal joints of the panel. The minimum distance from the edge of the small opening to the longitudinal edge of the panel is the smallest (100 mm, 0.1 x B) in which B is the overall width of the panel.

The remaining cross-section of a sandwich panel with small openings can be able to carry the loads exposed to a sandwich panel, if the size of the opening is small or the opening is located in a cross-section with a resistance reserve. The remaining cross section is described by the difference of the overall width (B) and the width of the opening (b) (Fig 4). The bending moment and the shear force in the area of the opening shall be carried by the remaining cross-section weakened by the opening.

![Fig. 4: Remaining cross-section in the area of an opening in a sandwich panel.](image)

It has to be pointed out that a simple calculation based on the pure remaining cross section leads to an unsafe side, because of the substantial stress concentrations in the corners of the opening. Thus, the design cannot be based on the pure remaining cross-section, but the stress concentrations shall be considered for the calculations.

The remaining cross-section of a sandwich panel (B-b) is classified to be able to carry the loads exposed to a sandwich panel with an opening, if the verifications of the wrinkling strength of the face and the shear strength of the core are fulfilled as given in Section 3.1.2.

The influence of the opening on the bending and shear rigidity shall be taken into account.

Information about the effects of the openings on the wrinkling stress and shear resistance of sandwich panels with strongly profiled faces is based on a few experimental results. If an opening is located between the profiles of the face, the influence on the compression resistance of the face seems to be minor (Fig 5a). The opening strongly reduces the compression resistance of the face, if it cuts a face profile (Fig 5b). Cutting of the face profile results in challenges with regard to tighten the corners of the opening. Thus, cutting of a profile is not recommended.
3.1.2 Openings placed symmetrically to the mid-line

Verification of the existing stress $\sigma_{Fcd}$ against the wrinkling stress $\sigma_w$ of a flat or slightly profiled face with openings placed symmetrically along the mid-line of the sandwich panel shall be made using the expression 1a and 1b. The first expression (1a) is the usual verification of the full cross-section in the point of the highest bending moment of the panel. The second expression (1b) describes the verification in the point of the highest bending moment in the area of the opening. The expression (1b) is based on the research work of Courage & Toma (1994) and it has been verified later by Heselius (2004), Lange & Böttcher (2006).

\[
\begin{align*}
\sigma_{Fcd} &= \frac{M_{d_{max}}}{e_C B t_{Fd}} \leq k_2 \frac{\sigma_w}{\gamma_M} \\
\sigma_{Fcd} &= \frac{M_{ad}(x)}{e_C B t_{Fd}} \leq k_3 k_F \frac{\sigma_w}{\gamma_M}
\end{align*}
\]

\( (1a,b) \)
where

\[
k_F = \begin{cases} 
(1 - \beta)^2 & \text{if } 0 \leq \beta \leq 0.4 \\
0.6(1 - \beta) & \text{if } 0.4 < \beta \leq 0.8 
\end{cases}
\]  

(2a,b)

\[
k_2 = 0.61 \left( \frac{f_{Ct}}{f_{Ct0}} \right) + 0.39 \leq 1.0
\]  

(3)

\[
\beta = b/B
\]  

(4)

In the expressions, \( f_{Ct} \) is the characteristic cross-panel tensile strength of the core scaled to \( f_{Ct0} = 0.10 \text{ MPa} \). \( M_{d,max} \) represents the highest design value of the bending moment of the sandwich panel and \( M_{od}(x) \) is the highest design value of the bending moment in the area of the opening.

Factor \( k_2 \) is defined in Section A.5.5.5 of EN 14509 considering a low tensile strength of bond and core. Cutting of the openings in the faces of the panel with low bonding strength is a sensitive task and can further reduce the strength of bond and core and thus, causes a clear risk of delamination.

Fig. 7: Failure mode at an opening in a slightly profiled face of a sandwich panel.

Verification of the shear resistance of a sandwich panel with flat or slightly profiled faces with openings placed symmetrically along the mid-line of the sandwich panel shall be made using the expressions 5a and 5b. The first expression (5a) is the common verification of the full cross section in the point of the highest shear force of the panel. The second expression (5b) is the verification in the point of the highest shear force in the area of the opening. The factor \( k_C \) is based on the work of Courage & Toma (1994).

\[
\tau_{Cd} = \begin{cases} 
\frac{V_{d,max}}{e_c B} \leq \frac{f_{Ct}}{\gamma_M} & \text{if } V_{od}(x) \leq k_C \frac{f_{Ct}}{\gamma_M} \\
\frac{V_{od}(x)}{e_c B} \leq k_C \frac{f_{Ct}}{\gamma_M} & \text{if } V_{d,max} \leq \gamma_M \frac{f_{Ct}}{e_c B} 
\end{cases}
\]  

(5a,b)

in which
\( k_c = (1 - \beta) \) if \( 0 \leq \beta \leq 0.8 \) \hspace{1cm} (6)

In the expression, \( V_{d,\max} \) represents the highest design value of the shear force of the sandwich panel and \( V_{od}(x) \) is the highest design value of the shear force in the area of the opening.

The expressions apply to round and rectangular openings, where the rectangular ones are the most serious in practice. The expressions have not been verified to consider the interaction at mid-supports of the continuous multi-span panels. The influence of the openings on the support reaction resistance was unknown at the moment of writing this report.

Table 1: Numerical values of the reduction factors \( k_F \) and \( k_C \)

<table>
<thead>
<tr>
<th>( \beta = b/B )</th>
<th>( k_F )</th>
<th>( k_C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,0</td>
<td>1,0</td>
<td>1,0</td>
</tr>
<tr>
<td>0,1</td>
<td>0,81</td>
<td>0,9</td>
</tr>
<tr>
<td>0,2</td>
<td>0,64</td>
<td>0,8</td>
</tr>
<tr>
<td>0,3</td>
<td>0,49</td>
<td>0,7</td>
</tr>
<tr>
<td>0,4</td>
<td>0,36</td>
<td>0,6</td>
</tr>
<tr>
<td>0,5</td>
<td>0,3</td>
<td>0,5</td>
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<td>0,6</td>
<td>0,24</td>
<td>0,4</td>
</tr>
<tr>
<td>0,7</td>
<td>0,18</td>
<td>0,3</td>
</tr>
<tr>
<td>0,8</td>
<td>0,12</td>
<td>0,2</td>
</tr>
</tbody>
</table>

3.1.3 Openings placed non-symmetrically to the mid-line of the panel

If the opening is located non-symmetrically to the mid-line of the sandwich panel with flat or slightly profiled faces, the reduction factor \( k_{F,\text{ecc}} \) introduced in 7a and 7b shall be used instead of the factor \( k_F \) in expressions 2a, 2b.

The reduction factor (7a,b) is based on the model describing the elastic distribution of the normal stress for a face in a cross-section including an opening (Lange & Böttcher 2006) (Fig. 8 and Exp. 13). According to new experimental information, the factor \( k_{F,\text{ecc}} \) results in a larger reduction of the resistance compared to the measured reduction in the tests, and thus, leads to a design on the safe side. A reason for this may be that the highest stress is located in the longitudinal edge, which is stiffened by cold-formed joint profiling in contrast to the straight cut edges of the opening inside the panel.

\[
k_{F,\text{ecc}} = \begin{cases} 
\frac{(1 - \beta)^2}{1 + \frac{(1 - \beta)B y_s y_e}{I_{zo}}} & \text{if } 0 \leq \beta \leq 0.4 \\
\frac{0.6(1 - \beta)}{1 + \frac{(1 - \beta)B y_s y_e}{I_{zo}}} & \text{if } 0.4 < \beta \leq 0.8
\end{cases} \quad (7a,b)
\]

\[
y_s = \frac{b}{B - b} \epsilon_o \quad (8)
\]
\begin{align*}
y_e &= y_s + 0.5 \, B \\
I_{\omega} &= \frac{1}{12} \left( b_1^3 + b_2^3 \right) + b_1 \left( 0.5 \, B - y_s - 0.5 \, b_1 \right)^2 + b_2 \left( 0.5 \, B + y_s - 0.5 \, b_2 \right)^2 \\
b_1 &= 0.5 \, B - 0.5 \, b + e_o \\
b_2 &= 0.5 \, B - 0.5 \, b - e_o
\end{align*}

where \( e_o \) is the eccentricity of the mid-line of the opening to the mid-line of the cross-section. The expressions apply, if \( b_1 \geq b_2 \).

![Diagram showing coordinates and eccentricity](image)

\begin{equation}
\sigma_{FMd} + \sigma_{FNd} \leq \sigma_w \begin{cases} 
(1 - \beta)^2 & \text{if } 0 \leq \beta \leq 0.4 \\
0.6(1 - \beta) & \text{if } 0.4 < \beta \leq 0.8
\end{cases} \tag{13}
\end{equation}

Fig. 8: Coordinates defining location and eccentricity of the opening. The expression (13) is a background information and was used for the derivation of the parameter \( k_{F,ecc} \).

If the opening is located non-symmetrically to the mid-line of the sandwich panel with flat or slightly profiled faces, the reduction factor \( k_C \) (Exp. 6) describing the influence of the opening to the shear resistance, shall be modified.

Notes:
The expressions 1 - 6 have been verified with regard to EPS- and PU-foam as well as mineral wool cored sandwich panels (Courage & Toma 1994, Heselius 2004, Böttcher 2005).

The expressions 7 – 12 are based on the elastic distribution of the normal stress in a face of a sandwich panel (Lange & Böttcher 2006). A preliminary verification of experimental results regarding PU-cored sandwich panels shows that the models result in rather con-
The design models of openings have been validated to uniformly distributed loads. A local load located close to a corner of the opening such as a frame of a window can cause further reductions of the resistance of the panel with opening.

More information is also needed on the influence of the cut ends of the mw-lamellas or other pre-formed core materials.

3.1.4 Openings in panels with profiled faces

The load-bearing resistance of a sandwich panel with openings in the profiled faces strongly depends on the location of the opening. If the opening is located between the profiles in the inner flat part of the face and leaves a distance of more than 10 mm to the webs of the profile, the influence of the opening is minor. The verification of the bending moment resistance of the remaining cross-section can be carried out using the expressions (14) (Rädel & Lange 2011).

![Diagram of opening in a sandwich panel with profiled faces](image)

**Fig. 9a:** Opening in a flat part between the profiles in a face of a sandwich panel.

\[
M_{od}(x) \leq \left(1 - 0.5 \frac{b}{B}\right) M_{rd} \quad x_1 < x < x_2 \tag{14}
\]

In which \(b < \) width of the flat part of the profile minus 20 mm. \(M_{od}(x)\) describes the maximum appearing bending moment in the area of the opening between \(x_1\) and \(x_2\). It covers the moment due to mechanical loading and the moment due to temperature difference between the faces (according to EN 14509, table E.10.2).
Fig. 9b: Opening cuts of a profile in a face of a sandwich panel.

If the opening cuts a profile, the resistance of the panel is clearly reduced. The verification of the bending resistance of the remaining cross-section shall be done using the expression (15) (Rädel & Lange 2011).

\[ M_{od}(x) \leq \frac{k-a}{k} M_{Rd} \quad x_1 < x < x_2 \]  
(15)

In the expressions (14) and (15), \( M_{Rd} \) is the bending resistance of the sandwich panel with profiled faces without openings. \( k \) is the number of profiles in width direction of the sandwich panel. \( a \) is the number of profiles, which have been cut by openings. The expressions (14) and (15) can be used in the design of PU-foam and mineral wool cored sandwich panels.

The expressions (14) and (15) are restricted to panels with an overall depth less than 150 mm and a depth of the profile of more than 30 mm.

3.2 Transfer of loads to adjacent panels

3.2.1 Principles

Openings in a sandwich panel reduce the cross-section and the bending and shear stiffness of the panel. Due to the difference of the stiffness, the loads or a part of the loads exposed directly to the panel with openings will be transferred via longitudinal joints to adjacent panels (case A in Fig. 10). A transfer of the load of the complete cut panel to the adjacent panels causes the most severe case (case B in Fig. 10). Whereas in case “A” the panel can have sufficient capacity to carry the load, case “B” relies on the transfer of the load to the other panels. In both cases, the neighbouring panels will receive additional loads due to the compatibility of the deflections in the longitudinal joints (Fig. 11). These loads are transferred through longitudinal joints and therefore, the assessment of the strength and stiffness of the joints is important. Furthermore, the load transfer will result in an eccentric line load to the neighbouring panel without an opening activating its torsional rigidity and causing
additional shear stresses and additional normal stresses in the faces due to the torsional moment.

Fig. 10: Small opening and a full-width opening in the wall covered with sandwich panels

Fig. 11: Line loads in the longitudinal joints between the neighbouring sandwich panels with different bending and shear stiffnesses due to the opening.

Notes:
As a rule of thumb, the load exposed directly to the panel with an opening can be transferred to the two adjacent panels on the basis of the 70-30-rule. This rule states that 70% of the load has to be carried by the first neighbouring panel and 30% by the second one. This rule of thumb applies under the assumption that the bonding strength of the face to the core is high enough to avoid a delamination when transferring the load from panel to panel over the longitudinal joint.

The information shall be only used in the preliminary design, and is further restricted to be used in the cases in which the cross-panel tensile strength of the core and the bond is higher than 0.1 N/mm².

If the rule is used in the final design, experimental verification is absolutely needed.

In symmetric cases, it may be correct to assume, that the joints at each side of the panel
will transfer 50% of the panel’s load to the adjacent panels. Since all panels usually have the same width and the panel with the opening has neighbouring panels on both sides, this assumption leads to a load amplification factor of 1.35 for the first and 1.15 for the second panel.

3.2.2 Modelling of the static system of panels

The intensity of the load, which can be transferred through the longitudinal joints to the adjacent panels, depends on bending, shear and torsional rigidity of the complete panels and in addition, on the shear rigidity of the longitudinal joint. On the basis of the geometry and the stiffness parameters of the panels, a numerical model can be generated for the evaluation of internal forces and deflections using software based 3D beam elements. As an example, a strut model of a three-panel-system is shown in Fig. 12.

Fig. 12: Three-dimensional beam model describing a three-panel-system, in which the panel in the middle has an opening.

The three-dimensional beam model shown in Fig. 12 consists of
- beam elements representing complete sandwich panels (No 1),
- beam elements representing partial elements along the opening (No 2 and 3),
- rigid load distribution of struts (No 4),
- transfer struts representing the stiffness perpendicular to the main span (No 5),
- transfer struts with reduced size due to the opening (No 6 + 7) and
- transfer struts representing the joint stiffness (No 8 + 9).

Additional panels may be included in the model. The main goal of this model is the assessment of the loads in the struts representing the joints. The derivation of the required cross-section values of the beam elements shall be made experimentally or by calculations such as shown in the following sections and in Annex C.
Alternatively, the approximate intensity of the shear loads in the longitudinal joint may be evaluated using an analytical approach introduced in refs (Höglund 1986) and (Davies 2001). The approach is based on the assumption with regard to the compatibility of the deflections caused by bending and torsional moments, and the shear forces in the mid-point in the longitudinal joint, and on the knowledge about the typically uniform distribution of the shear load in the joint. Thus, the approach applies for a system with openings, which are distributed over the span of the panel such as regular lines of windows in a wall (Annex B).

3.2.3 Cross-sectional values of sandwich panels
3.2.3.1 Torsional stiffness and stresses

A numerical value of the torsional stiffness of a sandwich panel is needed for analysing the complete wall structure that consists of sandwich panels with and without openings. Stamm & Witte have derived the expressions (16) and (17) to evaluate the torsional stiffness and the shear stress of the core caused by the torsional moment $M_T$ of a flat faced sandwich panel (Stamm & Witte 1974).

$$V_S = G_F I_T = 4 \cdot G_F \cdot e^c \cdot B \cdot \frac{t_1 \cdot t_2}{t_1 + t_2} \left(1 - \frac{\tanh(0.5 \lambda \cdot B)}{0.5 \lambda \cdot B}\right)$$

$$\tau_{CT} = \frac{\lambda \sinh(\lambda y)}{\cosh(0.5 \lambda B) - \sinh(0.5 \lambda B)} \frac{M_T}{2e_c B} \quad (17)$$

In expressions

$$\lambda = \sqrt{\frac{G_C}{G_F} \frac{t_1 + t_2}{d_C t_1 t_2}} \quad \text{and} \quad d_C = e_c - 0.5(t_1 + t_2) \quad (18a,b)$$

$I_T$ is the torsional rigidity, $G_F$ the shear modulus of the face and $t_1$ and $t_2$ the design thicknesses of the external and internal faces.

Höglund gives the semi empirical expressions (19) and (20) for the torsional stiffness and the highest shear stress of the core caused by the torsional moment (Höglund 1986). The expressions (20) and (21) have been derived for flat faced sandwich panels.

$$V_S = 4 \left(\frac{2 B}{3}\right) \frac{e^c}{3} \frac{2 B}{3 G_F t + 3 e_c G_C B}$$

$$\tau_{CT} = \frac{3 M_T}{4 \left(\frac{2 B}{3}\right) e_c B}$$

$$\tau_{CT} = \frac{3 M_T}{4 \left(\frac{2 B}{3}\right) e_c B}$$

(20)
An important finding in the EASI E project is the distribution of the shear stresses in the core, and the normal stresses in the faces due to eccentric loads in the longitudinal joints, caused by the transfer of loads to adjacent panels. Based on the proposal of Rädel & Lange 2011, the additional shear stress $\tau_{CT}$ caused by transferring the loads, can be simply evaluated by subdividing the shear force caused by the eccentric load to a width of $B_C = 250$ mm on the support of the flat faced sandwich panel (21).

$$\tau_{CTd} = \frac{V_{jd}}{e_c B_C}$$

(21)

The maximum value of the additional shear force $V_{jd}$ (22) caused by a uniform or a non-uniform distribution of the shear load $q_j(x)$ in the longitudinal joint shall be applied in the calculations.

$$V_{jd} = \frac{1}{L} \int_0^L q_{jd}(x)(L-x)dx$$

(22)

Experimental and computational work has shown, that the eccentric load caused by transferring the load through the longitudinal joints, also results in normal stresses in the faces of the sandwich panel. The highest normal stress can be evaluated using the expression (23) (Rädel & Lange 2011).

$$\sigma_{Fjd} = \frac{k_\sigma M_{jd}}{e_c Bt}$$

(23)

In this expression, the bending moment $M_{jd}$ is caused by a uniform or a non-uniform distribution of the shear load $q_j(x)$ in the longitudinal joint (24). The maximum value of $M_{jd}$ shall be used in the calculations.

$$M_{jd} = \max(V_{jd} x - \int_0^x q_{jd}(x')(x-x')dx')$$

(24)

If the shear load in the joint is uniformly distributed $q_j(x) = q_{j0}$, $V_{jd} = q_{j0}d L/2$ and $M_{jd} = q_{j0}d L^2/8$. If the shear load has a distribution of a sinusoidal curve $q_{jd}(x) = q_{j0d} \sin\left(\frac{\pi x}{L}\right)$, the shear force and the bending moment are $V_{jd} = q_{j0d} L/\pi$ and $M_{jd} = q_{j0d} L^2/\pi^2$.

When drafting this report, the application of the expressions (21) and (23) is recommended.

The value of the parameter $k_\sigma$ depends on the geometry and on the stiffness of the sandwich panels. Values of $k_\sigma$ in function of the shear modulus of the core and on the bending stiffness, and the span length of the panel can be chosen in Fig. 13 in which the sinusoidal distribution of the shear load is assumed in a longitudinal edge of the panel with a total width of 1 m.
Fig. 13: Parameter $k_\sigma$ in the expression for the normal stress of the face (Rädel & Lange 2011).
3.2.3.2 Bending and shear stiffness to the direction of the span

In definitions for the stiffness values, the coordinate axes x, y and z, follow the longitudinal, width and depth directions of the panel. The bending and shear stiffness to the y-y axis are

\[
B_{xy} = \int E(z) \frac{z^2}{x} dz = \frac{E_{F1} A_{F1x} E_{F2} A_{F2y}}{E_{F1} A_{F1x} + E_{F2} A_{F2y}} e_{cx}^2
\]

(25)

\[
S_{yz} = B e_{cy} G_C
\]

(26)

In expression (25) \(E_{F1}\) and \(E_{F2}\) are the moduli of the flat faces in the direction of the span (x-direction). The shear modulus in expression (26) represents the shear modulus in the xz-plane, \(G_C = G_{Cxz}\), i.e., in the span-depth plane of the panel.

![Coordinates for the definitions for the stiffness values.](image)

Fig. 14: Coordinates for the definitions for the stiffness values.

3.2.3.3 Bending and shear stiffness perpendicular to the span

Bending and shear stiffness to the x-x axis is

\[
B_{yx} = \int E(z) \frac{z^2}{y} dy = \frac{E_{F1} A_{F1y} E_{F2} A_{F2y}}{E_{F1} A_{F1y} + E_{F2} A_{F2y}} e_{cy}^2
\]

(27)

\[
S_{xz} = B e_{cy} G_C
\]

(28)

In the expressions, \(e_{cy}\) is the distance between the centroids of the external and internal face in the transverse direction to the span. The expressions only apply for the flat faced panels. Even the slight profiles significantly reduce the transverse bending stiffness of the face. The moduli \(E_{F1}\) and \(E_{F2}\) describe the stiffness of the faces in the width direction to the span. The shear modulus represents the shear modulus in the yz-plane \(G_C = G_{Cyz}\) in which y and z represent the coordinates in the width and depth directions of the panel. \(G_{Cyz}\) can be different compared to the shear modulus in the span-depth plane of the panel due to the anisotropy of the core material. The shear modulus shall be measured using the shear test arrangements introduced in A3, A4 and A5 of EN 14509, the face of the test specimen, however, shall be completely flat without any profiling.

3.2.4 Resistance and stiffness of the longitudinal joints

The longitudinal joints of the panels are loaded using shear forces, if the load is transferred through the joints between the panels. Fig. 15 shows a typical crack pattern caused by over-
loading of the joint. The panel on the left is that with a reduced stiffness. It is on the panel on the right side of the picture. The load transfer produces a line load to the joint, which has to be transmitted by the tongues and grooves of the metal sheet faces and the core. The ultimate load is determined through a combined failure mode of the core material and the delamination of the faces.

![Cracks in the longitudinal joint due to load transfer.](image)

**Fig. 15:** Cracks in the longitudinal joint due to load transfer.

Usually, modern sandwich panels have a joint geometry that allows a sufficient load transfer. However, experimental information is needed in order to verify the shear resistance and shear stiffness of the longitudinal joints of the panels. Fig. 16 shows test arrangements to measure the shear resistance and stiffness of the longitudinal joint. Panel No. 1 is a panel that was cut into two halves in longitudinal direction. Both halves are restrained through steel profile no. 3, a pair of load bearing structures, e.g. U-sections with a foamed rubber for a soft load introduction. The load bearing structures are fixed to a stiff structure (no. 4) through threaded rods (no 3). The specimen is fixed in lateral direction using threaded rods (no. 7) in order to avoid the decomposition of the specimen, which also corresponds to boundary conditions in wall panels in practice. The rod (no. 7) is preloaded with a load, which provides a contact for the end plates (no 7), and in the joints of the specimen. The rod (no. 7) shall not be pre-stressed. A normal panel (no. 2) is fixed between the two halves of the panels. The load is introduced into two loading beams (no. 5) through a spreader beam (no. 6). The typical length of the panels in the test rig is about 800 mm.

With this test setup, the load deflection curve obtained in the test allows the derivation of the stiffness and the ultimate load of the longitudinal joint (Fig 17). The load-deflection curves and the ultimate loads can be different in the two joints of the specimen, because of the different geometry. However, the test set up only shows the ultimate load of the weakest joint. The stiffness and the resistance of the joint shall be used in further calculations.
Fig. 16: Test set-up for the assessment of the shear stiffness and resistance of the longitudinal joint. \( w_1 - w_8 \) show the points to measure the displacements in order to derive the shear stiffness of the longitudinal joints.

Fig. 17: Example of load–deflection curves for the shear test of the longitudinal joint.

The thickness of the panel used in the tests shall represent the small, the large and a medium thickness of the sandwich panel product. The specimen is loaded with a monotonously increasing static load up to the failure. The test series consists of at least three similar tests. Measurements and observations concerning the deformations and the tightness can give useful further information for the verification of the serviceability limit state. Repeated loading with an intensity of about \( \frac{1}{2} \) of the ultimate load can result in further interesting information for the verification of the joint. Testing of fastenings with repeated loading history is introduced in the Preliminary European Recommendation for the Testing and Design...
of Fastenings for Sandwich Panels (ECCS 2009 and CIB 2009), the information of which shall be used in testing the properties of the longitudinal joints.

3.2.5 Simplified test and design procedure

A simplified procedure to test the shear resistance of the longitudinal joint and thus, the ability to transfer loads from the panel with openings to the adjacent panel, is based on two simply supported panels placed side by side on the supports. The panels are fixed to each other in the joints on the supports. One of the panels is loaded up to failure, whereas there are no loads on the second panel. The ultimate load of the two-panel system \( M_{u,2} \) is compared to the ultimate load of a single panel \( M_{u,1} \). The increase of the load carrying capacity in the term of the bending moment is marked as \( M_{\text{joint}} = M_{u,2} - M_{u,1} \) (or shear force capacity \( V_{\text{joint}} = V_{u,2} - V_{u,1} \)) and shall be used as the load that can to be transferred to adjacent panels in the design in practice. The characteristic value and the corresponding design value of the increase of the bending moment capacity \( M_{\text{joint},d} \) (or the shear force capacity \( V_{\text{joint},d} \)) is determined on the basis of at least three tests as a minimum. In the following, the load increase capacity will be treated as increase in bending moment capacity:

1. The bending moment resistance of a panel with an opening \( M_{\text{Rod}} \) is determined according to chapter 3.1

\[
M_{\text{Rod}} = k_2 k_F \frac{\sigma_w}{\gamma_M} e_c B t \quad \text{or} \quad M_{\text{Rod}} = k_2 k_{F,ec} \frac{\sigma_w}{\gamma_M} e_c B t
\]  

(29a,b)

2. If the bending moment resistance of the panel with opening \( M_{\text{Rod}} \) is not adequate, the additionally required bending moment resistance \( \Delta M = M_{\text{Rd,required}} - M_{\text{Rod}} \) and the corresponding load, designated as additional load capacity \( \Delta q \) is determined on the basis of the usual design procedure of that specific sandwich panel as \( E_{\text{ULS,d}} \leq R_d \) (see EN 14509, Annex E)

3. It is assumed that a half of the bending moment is distributed to each side of the panel with an opening

4. If the additionally required bending moment resistance \( \Delta M \) is less than \( 2 \times M_{\text{joint},d} \) no additional supporting frame structure is needed

5. The panel to which half of the load corresponding to half of the bending moment \( \Delta M \) shall be transferred through the longitudinal joint, shall be designed for the load, which is the actual bending moment plus the transferred bending moment \( \Delta M/2 \)

6. If the additionally required bending moment resistance \( \Delta M \) is higher than \( 2 \times M_{\text{joint}} \), an additional supporting frame structure shall be made according to the directions given in chapter 3.3

The same principles shall be followed, if the resistance to the shear force is dominant in the design.

If the load shall be transferred to two neighbouring panels instead of the one described above, a system of three panels connected to each other shall be used and connected to each other during the test. The first panel is loaded up again to the ultimate limit state whereas there are no loads on the second and third panel. The transferred load to the third panel is described with a bending moment \( M_{\text{joint}2} \) which is determined as
3.2.6 Verification

Verification of the resistance to transfer the load through longitudinal joints shall follow the usual practice of design and shall be made at serviceability and ultimate limit states with relevant load and material safety factors. Additional criteria for the serviceability limit state verification are the deformation and tightness of the longitudinal joint. Additional criteria for the ultimate limit state verification is the ultimate shear load to be carried through the joint.

Ultimate limit state verification includes the verification of the bending and shear resistance of the panel with openings and further, the resistance of the adjacent panels exposed to increased loads due load transfer over the longitudinal joints and finally, the shear resistance of the longitudinal joints. The core of the panels is loaded by a combined shear stress (31) caused by the shear force (32) and the torsional moment (21).

\[
\tau_{Cvd} + \tau_{Ctd} \leq \frac{f_{Cv}}{f_M} \quad \text{(31)}
\]

in which

\[
\tau_{Cvd} = \frac{V_d}{e_c B} \quad \text{and} \quad \tau_{Ctd} = \frac{V_{jd}}{e_c B_c} \quad \text{(32a, b)}
\]

3.3 Reinforcing of panels

3.3.1 General

Additional frames shall be constructed, if the load carrying resistance of the longitudinal joints cannot be used, the load carrying resistance of the longitudinal joints is not high enough or if the resistance of the neighbouring panels is not high enough for additional loads. The additional frame shall be designed to carry through the load of the sandwich panel with openings. To be on a safe side, the additional frame should be designed to carry through all the loads exposed to the panel with an opening. An additional frame shall be placed in the longitudinal joints of the panels or on the side of the internal or external face of the panel.
3.3.2 Reinforcement of opening with a particular framework

The area of an opening can be strengthened through installing a particular frame around the opening. The principal possibilities of the design of the framework can be characterized with the following remarks.

- All openings, independent on their size and location, can be strengthened through structural components which are not additional components but belong to the window and door frame.
- Also with large openings up to the width of an entire panel, useful allowable spans can be achieved using reinforcement around the opening. The useful spans shall approximately correspond to the spans of the panels, which have not been weakened by openings.
- The longitudinal joints and thus, the adjacent sandwich panels, shall not be stressed with additional loads transferred from the panel with openings.
- The reinforcement shall be associated with relevant static calculations to be recorded in the official design documents. The static calculations may be assisted by experimental results, however, without complex series of tests.
- The design of the mechanical and adhesive joints shall be made using relevant experimental methods and calculations. The resistance of the joints shall correspond to actual loads in the joints of the system.

An example solution can consist of the following components. In principle, the reinforcement can consists of a frame out of four (two at the lower and two at the upper side) cross beams placed on the level of the faces in the transverse edges of the opening, and two special side beams in the area of the longitudinal edge of the opening (Fig. 18). In the example, the frame shall transfer the loads by itself without any resistance of the remaining cross section of the panel.

Fig. 18: A frame construction installed around the opening can consist of special aluminium profiles connected with a plastic web profile to avoid thermal bridges (APK profiles).

External pressure loads cause bending moments which results in compressive stresses ($\sigma_{F1,c}$) in the upper face, and tensile stresses ($\sigma_{F2,t}$) in the lower face of the panel. The stresses shall be transmitted into the cross beams, respectively. This can take place in principle through
mechanical connections or through adhesive joints. In both cases, a secure load transfer has to be guaranteed.

The cross beams, which may consist of two separate angle profiles placed on the levels of the external and internal faces, transmit the loads, which in principle are horizontal support reactions of the respective crossbars, into the upper chord \( (F_u) \) and the lower chord \( (F_l) \) of the longitudinal beams. In addition, it must be ensured that the vertical shear forces \( V \) can be transmitted from the corners of the framework into the longitudinal beams.

The longitudinal beams of the reinforcing frame carry through the whole bending moment and the shear force. However, the construction of the longitudinal beams as a homogeneous solid cross section, e.g. as a steel or aluminium full-profile, is not possible because of the thermal bridges between external and internal faces. Therefore, the upper and lower chords of the side members shall be thermally separated.

At the same time, the longitudinal beams shall be connected with “shear-stiff” joints or connection elements in order to achieve a sufficient bending and shear capacity. The essential requirement is to reach the load bearing capacity of the non-weakened panel, if comparable spans have to be attained. In order to fulfil these requirements, special aluminium profiles with plastic webs, have been developed for this purpose. The profiles are special composite sections.

In principle, the upper and lower aluminium chords of these profiles can be freely selected. Thus, also special edge profiles could be developed to correspond to the shapes of the tongue and groove edges of the adjacent sandwich panels. In particular, the transmission of shear forces through the plastic webs shall be studied experimentally under short and long-term loads at different temperatures even up to +90 °C.

Important matters in design of reinforcement are the compatibility of the longitudinal beams with full-width adjacent sandwich panel and careful details in order to avoid additional secondary stresses and stress concentrations in the system. Another important point is the ratio of the stiffness of the panel and the reinforcement. Indentations of a very stiff reinforcement in the faces have to be avoided.

Fig. 19 shows details of a reinforcement solution in practice.

Another way to strengthen the area of an opening is to strengthen the sandwich panel itself. The solution differs from that described above.
Fig. 19: An example depicting the reinforcement placed in the joints around the opening. a) an opening inside a panel, b) an opening in two panels installed in vertical direction and c) an opening in two panels installed in horizontal direction.
Associated static calculations can be based on comprehensible engineer-level verifications (Fig. 20). The design shall follow the structural analysis based on the characteristic values of the resistance of the special longitudinal beams. Thus, in principle all possible variations of the openings can be designed and introduced for official approvals.

In principle, modelling of the structural system can be based upon well-known definitions for possible single beams, e.g. the beams, which simulate the panel in the area outside of the opening with the flexural rigidity “B,” according to the panel itself (calculated e.g. according to table E 10.1, EN 14509), multiplied by the factor b/B (b= the supposed width of single beam, B = overall width of the panel). In principle, the cross beams and the longitudinal beams are also sandwich beams due to the plastic web, which is necessary to avoid thermal bridges. The cross-sectional values of these beams needed for the modelling of the structural system, should be also calculated on the basis of the sandwich theory, e.g. analogous to table E 10.2, EN 14509 or on the basis of simple one span bending moment tests with separate beams. These tests are normally required in any case in order to find out the ultimate failure loads for the final design of these beams.

Fig. 20: Beam model describing the static system of a sandwich panel with a reinforcing frame around the full-width opening.

3.3.3 Reinforcement in the area of longitudinal joints

In principle, reinforcement may consist of special beams, which are installed in the area of the longitudinal joints of the sandwich panels. These beams could have the same cross section, e.g. special thermally separated aluminium profiles with plastic webs, such as longitudinal beams in the framework (see Section 3.3.1). Additional loads due to the openings could be transmitted directly to the supports on the supporting structure. In principle, the area of the openings in the sandwich panel could then remain stress-free. The experimental verification of the longitudinal beams based on ETAG guidelines is briefly described in tables 2 and 3.
Fig. 21a: Cross-section of a reinforcement based on a longitudinal perforated steel beam and plywood plate placed in the longitudinal joint of the panel inside the plane of the wall.

Fig. 21b: Reinforcement based on two perforated steel profiles and a thermal barrier jointed together with rivets and placed in the longitudinal joint between two sandwich panels. The thermal barrier may be made of a plastic or of a band having a certain resistance to the fire.
### Table 2: Determination of design parameters of beams placed in the longitudinal joints of the panels (ETAG 002).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Test procedures</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_k$ [N/m]</td>
<td>Longitudinal shear strength at a normal (20°C), low (-20°C) and high temperature (façades: +70°C / roofs +80°C)</td>
<td>See ETAG 002, Fig. 3 for test set-up</td>
</tr>
<tr>
<td>$C$ [N/mm²]</td>
<td>Longitudinal shear stiffness at a normal (20°C), low (-20°C) and high temperature (façades: +70°C / roofs +80°C)</td>
<td>See ETAG 002, Fig. 3 for test set-up</td>
</tr>
<tr>
<td>Reduction factor $A_T$ [-]</td>
<td>$A_T = \frac{T_{k,aged}}{T_k}$ Factor considering the ageing due to the permanent shear loading and high temperature. $A_T = 1/A_2$ Requirement: $A_T \geq 0.6$.</td>
<td>Ageing using the method 3 according to ETAG002 (100 hrs at +70°C/+80°C with a shear load of $1/3 T_k$) followed by a shear test. See ETAG 002, Fig. 3 for test set-up</td>
</tr>
</tbody>
</table>

### Table 3: Determination of parameters for quality control and for identification of the product

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Test procedures</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q_{11}$ [N/m]</td>
<td>Transverse tensile strength at a normal (20°C), low (-20°C) and high temperature (façades: +70°C / roofs +80°C)</td>
<td>See ETAG 002, Fig. 1 for test set-up</td>
</tr>
<tr>
<td>Reduction factor $A_{Q_{11}}$ [-]</td>
<td>$A_{Q_{11}} = \frac{Q_{k,aged}}{Q_k}$ Factor to take into account the ageing due to the permanent tensile loading and high temperature. Requirement: $A_{Q_{11}} \geq 0.6$.</td>
<td>Ageing using the method 3 according to ETAG002 (100 hrs at +70°C/+80°C with a tensile load of 10 N/mm) followed by a tensile test. See ETAG 002, Fig. 1 for test set-up</td>
</tr>
<tr>
<td>Density of the plastic material</td>
<td>See ETAG 002, chapter 5.2 for further parameters</td>
<td>See ETAG 002, chapter 5.2 for testing standards</td>
</tr>
<tr>
<td>Tensile strength of the plastic material (web) and the metallic material (flanges)</td>
<td>Fibre content of the plastic material (if applicable)</td>
<td></td>
</tr>
</tbody>
</table>

$^*$This parameter may also be required for design purposes (depending on the type and geometry of the profile)
3.3.4 Reinforcement outside from the plane of the panel

Sandwich panels weakened with openings may be strengthened by additional beams and frames placed outside the plane of the panels. These additional frames and beams shall be designed to carry the total loads exposed to the sandwich panels according to the relevant standards. Special attention shall be paid to the compatibility between the additional frame and the neighbouring sandwich panels without openings and reinforcements.

3.3.5 Tests of reinforcement structures

The load-bearing capacity of the reinforcements installed around the openings and of the special longitudinal beams placed in longitudinal joints shall be verified experimentally. Especially the flow of the shear force through the plastic webs and through the adhesive joints requires experimental studies under short and long-term loads. The necessary tests of the longitudinal beams have been described in table 2 and 3. Using the test results, a structural design can be made covering all load cases using comprehensive engineer-like calculations based on beam models. Adhesive joints and possibly also the mechanical joints can require special target-purpose testing.

A typical test programme includes usual identifications of all members used in the test specimens. It shall be based upon certificate documents and on measurements of essential geometrical and mechanical properties.
4 Additional considerations

4.1 Identification of the test specimens

The tests introduced in Section 3.3.2 give experimental information about the shear properties of the beams placed in the longitudinal edges of the sandwich panels. In order to analyse and to adjust the test results to the nominal values of the properties of the constituent elements of the product, information about the properties of the core and faces of the specimens is required. Additional tests described in this section are intended to determine these properties. They can be classified as follows:

- external and internal face layers
  - metal core thickness ($t_{obs}$), yield stress ($R_{p0.2}$, $R_{eff}$) and ultimate tensile strength ($R_m$), in all cases
- core layer
  - cross panel tensile strength ($f_{Ct}$), in all cases
  - cross panel compressive strength ($f_{Cc}$), in all cases
  - shear strength ($f_{Cv}$), in all cases
- dimensions of the test specimen, in all cases

The relevant tests on the core layer and on the sandwich panel itself are introduced in detail in EN 14509 and the tests on the metal sheet face layers in EN ISO 6892-1.

In addition to the properties of the sandwich panel specimens, information about additional reinforcement beams and reinforcement sections described in Chapter 3.3 is needed. The identification tests of the beams can be classified as follows:

- metal core thickness ($t_{obs}$), yield stress ($R_{p0.2}$, $R_{eff}$) and ultimate tensile strength ($R_m$), in all cases
- core layer thickness ($t_w$) and mechanical properties (table 3)
- dimensions of the beams
- dimensions and mechanical properties of mechanical and adhesive fastenings

4.2 Adjustment of the test results

In general, the thickness and strength of the face of the sandwich panel in a test do not correspond exactly with the nominal thickness and strength used for the design. Therefore, the results of the tests have to be modified in order to adjust the results to the values used for the design.

Since the core of the sandwich panel has an important influence on the shear resistance of the longitudinal joints, the results shall also be adjusted to the nominal strength of the core.

The shear test results of the longitudinal joint shall be adjusted using the expression:

$$R_{adj,d} = \min(f_2, f_3) R_{obs,d}$$  \hspace{1cm} (33)

in which
where the factors $f_2$ and $f_3$ consider the influence of the cross-panel tensile strength and shear strength of the core. The smallest of these factors shall be considered in the analysis.

In the above expressions:

- $R_{\text{obs},i}$ = result of test number $i$
- $R_{\text{adj},i}$ = result of test $i$ modified to correspond to the nominal values of the face and the core used in the design
- $f_{\text{Ctk}}$ = characteristic value of the cross-panel tensile strength of the core used in design
- $f_{\text{Ct},\text{obs}}$ = mean value of the cross-panel tensile strength of the core measured in the test specimens
- $f_{\text{Cvk}}$ = characteristic value of the shear strength of the core used in design
- $f_{\text{Cv},\text{obs}}$ = mean value of the shear strength of the core measured in the test specimens
5 Considerations for practice

5.1 General

In practice, openings in panels have been made since the start of the use of panels. Therefore, much experience has been acquired during several decades when using sandwich panels and cutting openings in them.

In general, openings are made on the installation site and only occasionally during the production itself. The reason for this lies in the fact that tolerances are quite loose in civil engineering, and the opening in a panel shall be adapted to the actual position in a façade, roof or partition wall.

The position of an opening is highly arbitrary, depending on the sandwich panel and on the composition of the sandwich panels in a wall. An important role also plays the panel tolerance and consideration of the pressing force when installing the panel against the joints of the already-installed panels. For these reasons, the required width of an opening is only defined just before installing the panel in which phase the opening shall be performed.

In general and in practice, a minimum distance of the cut edge from the edge of a panel is allowed for panel openings, which should amount to more than 10% of the width of the entire panel B (b_{\text{min}} and h_{\text{min}} > 10 \% B).

For cases where b_{\text{min}} and h_{\text{min}} amount to less than 10% of the width of panel B, a solution in practice is shown in Fig. 22. The sandwich panel is cut at the opening and the gap between the two adjoining panels for a module width of one or several panels. A portion of the difference of the larger opening for the installation of windows, for example, is covered by the endings or a panel part is inserted in the window opening as an inserted piece. If this cannot be done, the opening shall be performed after installing a panel. Since nobody can count on the loads of this part of the panel cut-out being taken over (the potential contact with lamella in case of panels with mineral wool, EPS, etc.).

Fig. 22: The principle of performing the panel installation for very critical openings (less than 1/10 B).
5.2 Cut-out Performance

In general, cut-outs in sandwich panels are performed on site just before installation. The cut-out is performed using tools capable of performing a “cold cut-out”. The recommended tools are circular saws with a blade of hard metal and small teeth, drilling devices or electronic jig saws. These tools do not cause excessive vibrations and do not damage the protective layer on the sheet metal. Never use an angle grinder or a torch.

Due to the jagging effects, cutting corners shall be performed with an appropriate radius, which is, why the first stage of the cut-out consists of drilling through the panel, i.e. cutting both metal sheets with a hole sawn at all four corners of a rectangular opening. When this operation is finished, a line cut of the steel sheets is performed using a circular or jig saw, for which it is recommended to install a plastic or wooden crossbar before cutting, as well as to fasten it along the cutting line using clamps. This prevents vibrations from the circular saw from being transferred to the contact between the sheet metal and the panel core. When the sheet metal is cut on one side, the panel is turned over so that the cut may be performed on the other side as well. When cutting of both sheets is finished, then the core of the panel is cut with a reciprocating saw (EPS) or an insulating knife (mineral wool).

In practice, in order to prevent the jagging effects at the corners of the opening, the following restrictions are observed or the following minimum radius are recommended, as shown in Figs. 23 a, 23 b and 23 c.

![Fig. 23: Possibilities in shaping the corners of openings; a) Making and forming the corners of an opening by drilling and by a line cut in tangential direction, b) Creating the corner of an opening by drilling a hole in the intersection of the lines of the opening and c) Creating the corner of an opening with a hole having a radius larger than the minimum prescribed.](image-url)

In practice, the most frequently used principle is to place the mid-point of the drilled hole into the intersection of both cut-out axes (Fig. 23 b). If flashing covers an extensive strip around the opening and an appropriate seal is provided between the flashing and the sand-
wich panel, cut-outs with larger radii in the corners are also performed (using a hole saw). Cutting of the insulating core of the panel is only performed in the geometry of the required size of the opening (Fig. 23 c).

5.3 Handling up to the site of installation
5.3.1 Handling panels with cut-out openings before installation

After creating the openings, it is very important to handle the machined panel with openings in an appropriate way, in order to prevent the handling to cause any additional deformations or damages to the panel. Large openings and openings with a minimum distance to the edges of the sandwich panel are exceptionally sensitive to deformations, reduced load capacity or damages. The critical moment is lifting of the panel from the flat position to a suspended position or to the installation position. During these critical operations auxiliary accessories are used, as shown in the Fig. 24. The panel is stiffened and supported on both sides across the opening using a crossbar and clamps. The clamps and crossbar are removed when the panel is in vertical position, i.e. just before the installation into its final position.

It can be even easier to use vacuum grips, which can grip the panel over a larger area (Fig. 25).

It is also possible to use other clamping methods and accessories to guarantee that the cut-out panel is not deformed, i.e. that the panel does not acquire permanent deformations which would reduce the usefulness of the panel.

![Fig. 24: Utilization of accessories for critical movement of panels with opening cut-outs.](image-url)
5.3.2 Making cut-outs in sandwich panels after installation

Although it is more demanding to create a cut-out after the sandwich panel has been installed into a roof or a façade, when the panel has been placed at its position and height, this is the best option for panels with large openings. The procedure for creating the cut-out is similar to that described in Section 5.2. The only difference lies in the fact that the panel is lifted and installed first. Then, a cut-out is performed on both sides or using a jig saw with a cutting tool larger than the panel thickness so that the cut is only performed from one side of the panel.

Note:
Based on experience from practice, an opening with a maximum width of 10% of the total panel width can be made in every fifth panel without further analysis.
6 Conclusions

This report is a state of the art report on the influence of openings regarding the behavior and resistance of sandwich panels. It introduces the currently known calculation models, test arrangements and gives practical directions for the design of the sole sandwich panels with openings, design of reinforcements and design of the complete sandwich panel walls and roofs, which have openings of different sizes and shapes placed in different locations to the span and width of the panels. This report is aimed at the completion of guidelines given in the European product standard EN 14509.

Several points with missing or limited information have been found during the drafting process of the report. The most essential are

- influence of the opening in sandwich panels with strongly profiled faces
- user-friendly calculation models to describe the transfer of the loads in transverse direction through the longitudinal joints
- design rules and calculation models for additional frames
- effects of very stiff window frames in the strength of the faces
- effects of the difference of the temperature between the faces
- influence of the long-term loads in roof and ceiling panels
- openings in panels made of other core materials than reported earlier
- opening on or close to the intermediate support of a continuous multi-span panel

The report will be updated on the basis of new information and results and it shall be published later in the series of European Recommendations for sandwich panels. To collect more experience and opinions of users, however, the report has been already distributed to the public in its current form.
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Annexes

Annex A1. Example on the verification of the resistance of a panel with a small opening

Annex A2. Allowable size and position of a small opening

Annex B. Example on the load transfer through longitudinal joints

Annex C. Example on the verification of the resistance of a panel with openings and additional frame structures
A1 Example on the verification of the resistance of a sandwich panel with a small opening
A centric and an eccentric opening in a one-span wall panel with flat steel sheet faces

An opening centric to the mid-line of the panel

\[ L := 4800 \text{ mm} \]  
span width

\[ D := 150 \text{ mm} \]  
total depth

\[ B := 1200 \text{ mm} \]  
overall width of the panel

\[ t_1 := 0.6 \text{ mm} \]  
nominal and design thickness of the external face

\[ t_2 := 0.5 \text{ mm} \]  
nominal and design thickness of the internal face

\[ e_C := D - 0.5 \left( t_1 + t_2 \right) = 149.45 \text{ mm} \]  
distance between the centroids of faces

\[ b := 400 \text{ mm} \]  
width of the opening

\[ L_o := 500 \text{ mm} \]  
length of the opening

\[ \beta := \frac{b}{B} = 0.333 \]  
relative width of the opening

\[ x_1 := 1000 \text{ mm} \]  
location of the opening in direction of the span

\[ x_2 := x_1 + L_o = 1500 \text{ mm} \]  

\[ k_F := \left( 1 - \beta \right)^2 = 0.444 \]  
reduction factor of wrinkling stress

\[ k_C := \left( 1 - \beta \right) = 0.667 \]  
reduction factor of shear stress

\[ \sigma_{w1} := 150 \frac{N}{\text{mm}^2} \]  
characteristic wrinkling strength of the external face

\[ \sigma_{w2} := 130 \frac{N}{\text{mm}^2} \]  
characteristic wrinkling strength of the internal face

\[ f_{CvK} := 0.1 \frac{N}{\text{mm}^2} \]  
characteristic shear and cross-panel tensile strength of the core

\[ f_{CtK} := 0.08 \frac{N}{\text{mm}^2} \]  
characteristic shear and cross-panel tensile strength of the core

\[ q_{wp} := 1 \frac{kN}{\text{m}^2} \]  
characteristic wind pressure loads exposed to the panel

\[ q_{ws} := -1 \frac{kN}{\text{m}^2} \]  
characteristic wind suction loads exposed to the panel

\[ \gamma_F := 1.5 \]  
load factor

\[ \gamma_M := 1.25 \]  
material safety factor

\[ M_{1,d}(x) := \frac{\gamma_F B q_{wp}}{2} \left( L - x \right) \]  
bending moment caused by wind pressure load

\[ M_{2,d}(x) := \frac{\gamma_F B q_{ws}}{2} \left( L - x \right) \]  
bending moment caused by wind suction load

\[ V_d(x) := \frac{\gamma_F B \max\left( q_{wp}, q_{ws} \right)}{2} \left( L - 2 x \right) \]  
shear force caused by wind load
Verification of the wrinkling and shear stress, symmetric opening

Reduction factor due to a low cross-panel tensile strength

\[ f_{Ct0} := 0.1 \quad \frac{N}{mm^2} \quad k_2 := 0.61 \cdot \frac{f_{Ctk}}{f_{Ct0}} + 0.39 = 0.878 \]

Compressive stress of the full-width external face at mid-span, pressure load

\[ \sigma_{F1.c.d} := \frac{M_{1,d}(0.5-L)}{e_c B t_{1d}} = 51.618 \quad \frac{N}{mm^2} < k_2 \cdot \frac{\sigma_{w1}}{\gamma_M} = 105.36 \quad \frac{N}{mm^2} \]

Compressive stress of the external face in the critical section of the opening, wind pressure load

\[ \sigma_{F1.o.d} := \frac{M_{1,d}(x_2)}{e_c B t_{1d}} = 44.359 \quad \frac{N}{mm^2} < k_2 \cdot k_F \cdot \frac{\sigma_{w1}}{\gamma_M} = 46.827 \quad \frac{N}{mm^2} \]

Compressive stress of the full-width internal face at mid-span, wind suction load

\[ \sigma_{F2.c.d} := \frac{M_{2,d}(0.5-L)}{e_c B t_{2d}} = 62.839 \quad \frac{N}{mm^2} < k_2 \cdot \frac{\sigma_{w2}}{\gamma_M} = 91.312 \quad \frac{N}{mm^2} \]

Compressive stress of the internal face in the critical section of the opening, suction load

\[ \sigma_{F2.o.d} := \frac{M_{2,d}(x_2)}{e_c B t_{2d}} = 54.002 \quad \frac{N}{mm^2} > k_2 \cdot k_F \cdot \frac{\sigma_{w2}}{\gamma_M} = 40.583 \quad \frac{N}{mm^2} \]

Shear stress of the full-width panel at a support

\[ \tau_{C.d} := \frac{V_d(0)}{e_c B} = 0.024 \quad \frac{N}{mm^2} < \frac{f_{Cvk}}{\gamma_M} = 0.08 \quad \frac{N}{mm^2} \]

Shear stress of the panel in the critical section of the opening

\[ \tau_{C.o.d} := \frac{V_d(x_1)}{e_c B} = 0.014 \quad \frac{N}{mm^2} < k_F \cdot \frac{f_{Cvk}}{\gamma_M} = 0.053 \quad \frac{N}{mm^2} \]

An opening located eccentrically to the mid-line of the panel

\[ e_o := 200-mm \quad \text{eccentricity of the opening to the mid-line of the panel} \]

\[ y_S := \frac{b}{B-b} e_o = 100-mm \quad \text{location of the mass center of the face in the area of the opening to the mid-line on the panel} \]

\[ y_e := y_S + 0.5B = 700-mm \quad \text{distance to the extreme edge from the line of the mass center of the panel in the area of the opening} \]

\[ b_1 := 0.5B - 0.5b + e_o = 600-mm \quad \text{remaining widths of the panels on two opposite sides of the opening} \]

\[ b_2 := 0.5B - 0.5b - e_o = 200-mm \]

\[ I_{zo} := \frac{1}{12} \left( b_1^3 + b_2^3 \right) + b_1 \left( 0.5B - y_S - 0.5b_1 \right)^2 + b_2 \left( 0.5B + y_S - 0.5b_2 \right)^2 = 1.147 \times 10^8 \quad mm^3 \quad \text{second moment of inertia of the face in the area of the opening} \]

\[ k_{F,2} := \frac{(1 - \beta)^2}{(1 - \beta) \cdot B \cdot y_S \cdot y_e} = 0.299 \quad k_F = 0.444 \quad \text{reduction factor of the eccentric opening to the wrinkling stress, for comparing the value of the reduction factor of the centric opening} \]
Verification of the wrinkling and shear stress, eccentric opening

compressive stress of the full-width external face at mid-span, pressure load

\[ \sigma_{F1c.d} := \frac{M_{1,d}(0.5L)}{e_{C^1}Bt_{1d}} = 51.618 \text{ N/mm}^2 < k_2 \frac{\sigma_{w1}}{\gamma_M} = 105.36 \text{ N/mm}^2 \]

compressive stress of the external face in the critical section of the opening, wind pressure load

\[ \sigma_{F1c.o.d} := \frac{M_{1,d}(x_2)}{e_{C^1}Bt_{1d}} = 44.359 \text{ N/mm}^2 > k_2 k_{F.2} \frac{\sigma_{w1}}{\gamma_M} = 31.462 \text{ N/mm}^2 \]

compressive stress of the full-width internal face at mid-span, wind suction load

\[ \sigma_{F2c.d} := \frac{M_{2,d}(0.5L)}{e_{C^1}Bt_{2d}} = 62.839 \text{ N/mm}^2 < k_2 \frac{\sigma_{w2}}{\gamma_M} = 91.312 \text{ N/mm}^2 \]

compressive stress of the internal face in the critical section of the opening, suction load

\[ \sigma_{F2c.o.d} := \frac{M_{2,d}(x_2)}{e_{C^1}Bt_{2d}} = 54.002 \text{ N/mm}^2 > k_2 k_{F.2} \frac{\sigma_{w2}}{\gamma_M} = 27.267 \text{ N/mm}^2 \]

Verification of the shear stress of the core, wind pressure and suction loads

No expressions for eccentric openings available, here the expressions for symmetric openings are used

shear stress of the full-width panel at a support

\[ \tau_{C.d} := \frac{V_d(0)}{e_{C^1}B} = 0.024 \text{ N/mm}^2 < \frac{f_{Cv_k}}{\gamma_M} = 0.08 \text{ N/mm}^2 \]

shear stress in the critical section of the opening

\[ \tau_{C.o.d} := \frac{V_d(x_1)}{e_{C^1}B} = 0.014 \text{ N/mm}^2 < k_{C^1} \frac{f_{Cv_k}}{\gamma_M} = 0.053 \text{ N/mm}^2 \]
A2 Allowable size and position of a small opening

In a sandwich panel, a small opening can be made in the area, in which extra non-utilised resistance is available. In one-span panels, typically the ends of the panel are loaded by shear forces and the mid-span area by a bending moment. Extra resistance is available between the end and the mid-span of the panel.

In this example, an allowable size and position of an opening in a one-span simply supported sandwich panel is determined assuming that the bending moment resistance (wrinkling stress) and the shear resistance are fully utilized. The sandwich panel is a simply supported one-span wall panel, which is loaded using a uniformly distributed load. The analysis is based on the expressions 1a,b & 5a,b.

Figure A.1. Allowable size and position of an opening in a wall panel.

Figure A.2. Allowable size and position of an opening scaled to the width of a panel of 1200 mm.
B Example on the load transfer through longitudinal joints
Distribution of the shear loads in the joints and the loads of the panels

A wall is made of four horizontally installed sandwich panels. The elements Nos. 1, 3 and 4 of the wall are structural wall panels. The second element consists of a row of windows and window frames, which are not able to carry their loads to the supports. The load of the second element is transferred to the lower and upper wall panels through the longitudinal joints of the panels. The analysis is made to evaluate the distribution of the load of the second element to the first, third and fourth element, which are load-carrying sandwich panels.

Dimensions of the wall panel are the same as in Annex A

\[
\begin{align*}
L &:= 4800 \text{ mm} \\
D &:= 150 \text{ mm} \\
B &:= 1200 \text{ mm} \\
t_1 &:= 0.6 \text{ mm} \\
t_{1d} &:= 0.56 \text{ mm} \\
t_2 &:= 0.5 \text{ mm} \\
t_{2d} &:= 0.46 \text{ mm} \\
t_{12d} &:= 0.5 \left( t_{1d} + t_{2d} \right) = 0.51 \text{ mm} \\
e_C &:= D - 0.5 \left( t_1 + t_2 \right) = 149.45 \text{ mm} \\
E_s &:= 210000 \frac{\text{N}}{\text{mm}^2} \\
G_s &:= 81000 \frac{\text{N}}{\text{mm}^2} \\
G_C &:= 5 \times 10^3 \frac{\text{N}}{\text{mm}^2}
\end{align*}
\]

modulus of elasticity and shear modulus of steel

shear modulus of the core

For simplicity, the same order of the reduction of the bending, shear and torsional stiffness due to the openings is assumed to take place in the second element.

The remaining stiffness of the second wall panel replaced by a row of windows and window frames is assumed to be 5% of the stiffness of a wall panel in this example.

\[k_{\text{red.opening}} := 0.05\]

degree of the remaining stiffness compared to full stiffness

bending stiffness of the panel without and with openings

\[
\begin{align*}
B_{S1} &:= E_s \frac{t_{1d} t_{12d}}{t_{1d} + t_{2d}} B e_C^2 = 1.421 \times 10^3 \text{ kN} \cdot \text{m}^2 \\
B_{S2} &:= k_{\text{red.opening}} B_{S1} = 71.074 \text{ kN} \cdot \text{m}^2
\end{align*}
\]
shear stiffness of the panel without and with openings

\[ S_1 := G_C \cdot B \cdot e_C = 896.7 \text{kN} \]

\[ S_2 := k_{\text{red.opening}} S_1 = 44.835 \text{kN} \]

torsional stiffness of the panel without and with openings

\[ V_{s1} := \frac{8 \cdot e_C^2 \cdot B^3 \cdot 12d \cdot G_s \cdot G_C}{6 \cdot B^2 \cdot G_C + 27 \cdot e_C^2 \cdot 12d \cdot G_s} = 303.847 \text{kN} \cdot \text{m}^2 \]

\[ V_{s2} := k_{\text{red.opening}} V_{s1} = 15.192 \text{kN} \cdot \text{m}^2 \]

parameters in the numerical analysis

\[ w_{B1} := \frac{5}{384} \cdot L^4 \cdot B_{S1} \]

\[ w_{B2} := \frac{5}{384} \cdot L^4 \cdot B_{S2} \]

measure of the bending flexibility

\[ w_{S1} := \frac{L^2}{8} \cdot S_1 \]

\[ w_{S2} := \frac{L^2}{8} \cdot S_2 \]

measure of the shear flexibility

\[ w_{V1} := \frac{1}{32} \cdot B^2 \cdot L^2 \cdot V_{s1} \]

\[ w_{V2} := \frac{1}{32} \cdot B^2 \cdot L^2 \cdot V_{s2} \]

measure of the torsional flexibility

\[ w_j := 3. \text{mm}^2 \cdot \text{kN} \]

shear flexibility of the longitudinal joint

\[ q_w := 1 \frac{\text{kN}}{\text{m}^2} \]

wind load exposed to the panel

\[ w_{11} := w_{B1} + w_{S1} + w_{V1} = 11.487 \frac{\text{mm}^2}{\text{N}} \]

additional parameters for the numerical analysis

\[ w_{12} := w_{B2} + w_{S2} + w_{V2} = 229.732 \frac{\text{mm}^2}{\text{N}} \]

\[ w_{21} := -w_{B1} - w_{S1} + w_{V1} = -4.662 \frac{\text{mm}^2}{\text{N}} \]

\[ w_{22} := -w_{B2} - w_{S2} + w_{V2} = -93.242 \frac{\text{mm}^2}{\text{N}} \]

\[ w_{31} := w_{B1} + w_{S1} = 8.074 \frac{\text{mm}^2}{\text{N}} \]

\[ w_{32} := w_{B2} + w_{S2} = 161.487 \frac{\text{mm}^2}{\text{N}} \]

compatibility equations of the mid-span deflections of longitudinal joints

\[ \mathbf{f} := \begin{pmatrix} w_{11} + w_{12} + w_j & w_{22} & 0 \\ w_{22} & w_{11} + w_{12} + w_j & w_{21} \\ 0 & w_{21} & w_{11} + w_{12} + w_j \end{pmatrix} \]

\[ \mathbf{Q} := \mathbf{B} \cdot q_w \begin{pmatrix} -w_{31} - w_{32} \\ w_{31} - w_{32} \\ 0 \end{pmatrix} \]

\[ q := \mathbf{f}^{-1} \cdot \mathbf{Q} \]

\[ q = \begin{pmatrix} 0.549 \\ -0.553 \\ -0.112 \end{pmatrix} \text{kN} \quad \text{shear loads } q_1, q_2 \text{ and } q_3 \text{ in longitudinal joints} \]
Influence of openings on the behavior and resistance of sandwich panels

\[
q_{\text{wall}} := \begin{cases} 
B \cdot q_w + q_1 \\
B \cdot q_w - q_1 + q_2 \\
B \cdot q_w - q_2 + q_3 \\
B \cdot q_w - q_3 
\end{cases} = \begin{pmatrix} 
1.749 \\
0.098 \\
1.641 \\
1.312 
\end{pmatrix} \ \text{kN/m} \\
\text{total loads of the wall panels}
\]

\[
q_{\text{wall,rel}} := \begin{cases} 
1 + \frac{q_1}{B \cdot q_w} \\
1 - \frac{q_1}{B \cdot q_w} + \frac{q_2}{B \cdot q_w} \\
1 - \frac{q_2}{B \cdot q_w} + \frac{q_3}{B \cdot q_w} \\
1 - \frac{q_3}{B \cdot q_w} 
\end{cases} = \begin{pmatrix} 
1.458 \\
0.081 \\
1.367 \\
1.094 
\end{pmatrix} \\
\text{relative total loads of the wall panels compared to uniformly distributed initial wind load } q_w
\]

\[
m_{\text{t,wall}} := 0.5 \cdot B \cdot \begin{pmatrix} 
q_1 \\
q_1 + q_2 \\
q_2 + q_3 \\
q_3 
\end{pmatrix} = \begin{pmatrix} 
329.66 \\
-2.121 \\
-399.103 \\
-67.322 
\end{pmatrix} \ \text{N/m} \\
\text{torsional moments of the wall panels}
\]

References
Annex C: Example on the verification of the resistance of a panel with openings and additional frame structures

A possibility is presented to calculate the load bearing resistance and deflection of sandwich panel systems with ribbon windows and adapter profiles installed within the longitudinal joint. The ribbon window itself is not able to transfer the load to the substructure. Thus, the whole load shall be transferred to the adjacent panels. In figure C.1, an example of an adapter profile with and without sandwich panel is shown.

![Fig. C.1 Adapter profiles and beam model](image)

Beam 1 represents the complete adjacent sandwich panel in longitudinal direction with the related torsional rigidity $I_T$ and the bending stiffness $E_{L1}$. The cross-beams (2) represent the behavior of the panel in transverse direction ($E_{Q1}, G_{Q}$). The cross-beams have a distance of $l_0$. The vertical elements (3) represent the stiffness of the joint $k_F$ and the horizontal beam 4 represents the adapter profile with its bending stiffness $E_{Adapter}$.

The following stiffnesses and material properties can be applied:

Beam 1: \[ E = E_f = 210000 \text{ N/mm}^2 \] E-modulus of the face

\[ G = G_{C,L} \] G-modulus of the core in longitudinal direction

\[ I_{y,L} = \frac{B_{F1} \cdot t_1 \cdot B_{F2} \cdot t_2}{B_{F1} \cdot t_1 + B_{F2} \cdot t_2} \cdot e_C^2 \] Moment of inertia in longitudinal direction

\[ \lambda = \sqrt{\frac{G_{C,L}}{G_F}} \cdot \frac{t_1 + t_2}{e_C \cdot t_1 \cdot t_2} \] Coefficient for the torsional stiffness according to Stamm/Witte

\[ I_T = 4 \cdot e_C^2 \cdot B \cdot \frac{t_1 + t_2}{t_1 \cdot t_2} \left( 1 - \frac{\tanh \left( \frac{\lambda \cdot B}{2} \right)}{\lambda \cdot \frac{B}{2}} \right) \frac{G_F}{G_{C,L}} \] Torsional rigidity of the complete panel, normalized to $G_F$
\[ A_1 = B \cdot e_c \]  
Area of the core

Beam 2:  
\[ E = E_{V} \]  
E-modulus of the face in transverse direction

\[ G = G_{c, Q} \]  
G-modulus of the core in transverse direction

\[ I_{y, Q} = I_Q \cdot \frac{t_1 \cdot t_2}{t_1 + t_2} \cdot e_c^2 \]  
Moment of inertia in transverse direction

\[ l_Q = \text{distance of the joint elements} \]

\[ A_Q = l_Q \cdot e_c \]  
Area of the core in transverse direction

Beam 3:  
\[ E = E_{F} \]  
E-modulus of the face (definition)

\[ A = \frac{k_F \cdot l_Q \cdot l_F}{E_F} \]  
Back-calculated area for the stiffness of the joint

\[ k_F = \text{spring stiffness of the joint} \]

\[ l_F = \text{length of beam 3 in the model} \]

Beam 4:  
\[ EI_{y, \text{Adapter}} \]  
Bending stiffness of the adapter profile

\[ G = \infty \sim 1 \cdot 10^6 \]

In the model, the adapter profiles are supported by the substructure and loaded by half of the area load of the window. Depending on the stiffness of the adapter profile, the joint and the sandwich panel, the loads shall be divided on the adapter profile and the adjacent sandwich panel. The panel is loaded eccentrically resulting in additional stresses in the face and the core of the panel. These stresses shall be added to the stresses caused by bending moment and shear force.

The beam model described in Fig C.1 gives the following results for the particular beams:

- Longitudinal beam:  \( M_{y,L}, M_{T}, Q_{z,L} \)
- Transverse beam:  \( M_{y,Q} \) und \( Q_x \)
- Joint elements:  \( N \)
- Adapter profile:  \( M_y \) und \( Q_z \)

After determining the stress resultants and deflections using the beam model, the structural analyses for the different failure modes shall be implemented.

The following failure modes are possible:

1) Wrinkling of the face of the adjacent panel
2) Shear failure of the core of the adjacent panel
3) Joint failure
4) Failure of the adapter profile – This case will not be investigated in this example. The producer of the adapter profiles is responsible for the load bearing capacity of the profiles.
1) Wrinkling of the face:

The normal stress in the adjacent panel can be determined by adding the normal stress due to bending and the normal stress caused by eccentric load.

The following two components of normal stress in the face exist:

\[
\sigma_1 = \frac{M_y}{I_{y,l}} \cdot \left( -\frac{e_c}{2} \right) \quad \text{Existing normal stress due to bending (Independent of the window)}
\]

\[
\sigma_2 = k_\sigma \cdot M_{y,L} \cdot \frac{-e_c}{2 \cdot I_{y,L}} \quad \text{Normal stress due to eccentric loading}
\]

The complete normal stress is:

\[
\sigma = \frac{(M_y + k_\sigma M_{y,L})}{2 \cdot I_{y,L}} \cdot \frac{-e_c}{2 \cdot I_{y,L}} < \frac{\sigma_w}{\gamma_M}
\]

The factor \( k_\sigma \) can be taken from diagrams (see Fig. 13 in the main document).

The verification is made using:

\[
\gamma_F \cdot \frac{(M_y + k_\sigma M_{y,L})}{2 \cdot I_{y,L}} \cdot \frac{-e_c}{2 \cdot I_{y,L}} \leq \frac{\sigma_w}{\gamma_M}
\]

2) Shear failure of the core:

Also in the core, the stresses caused by the centric shear force shall be superimposed with the stresses from eccentric loading. For the eccentric load, an effective width of the panel of 25 cm (total panel width 100 to 120 cm) can be considered for conservative results.

The following two components of the shear stress in the core exist:

Shear stress due to the existing shear force in the panel:

\[
\tau_1 = \frac{Q}{A_L}
\]

Shear stress due to the eccentric load:

\[
\tau_2 = \frac{Q_{z,L}}{25 \, \text{cm} \cdot e_C}
\]

The confirmation according to the technical approval is:

\[
\gamma_F \cdot \left( \frac{Q}{A_L} + \frac{Q_{z,L}}{25 \, \text{cm} \cdot e_C} \right) \leq \frac{f_{C,yk}}{\gamma_M}
\]

3) Failure of the joint:

For the load transfer from the adapter profile to the adjacent panel, the load bearing capacity of the joint shall also be proven. The beam model provides the maximum load in the joint as normal force \( N \).
in the joint elements. The load bearing capacity of the joint $F_j$ shall be determined through tests (see Section 3.2.4 in the main document).

The normal force given by the beam model is:

$$ F_{j,\text{net}} = \frac{N}{I_Q} $$

The equation used in the verification is:

$$ \gamma_p \cdot \frac{N}{I_Q} \leq \frac{F_j}{\gamma_M} $$

Below, a numerical example for a ribbon window with adapter profiles is calculated.

### Table C.1 Input values of the calculated example

<table>
<thead>
<tr>
<th>Geometry:</th>
<th>Section properties:</th>
</tr>
</thead>
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<tr>
<td>$L$</td>
<td>$5000$ mm</td>
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<tr>
<td>$B$</td>
<td>$1000$ mm</td>
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<tr>
<td>$e_c$</td>
<td>$120$ mm</td>
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<tr>
<td>$t_{f2}$</td>
<td>$0.5$ mm</td>
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<tr>
<td>$t_{f1}$</td>
<td>$0.5$ mm</td>
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<tr>
<td>$l_Q$</td>
<td>$250$ mm</td>
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<tr>
<td>$l_F$</td>
<td>$100$ mm</td>
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<td></td>
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</tr>
</tbody>
</table>

Values of the approval:

| $f_{cv}$  | $0.09$ N/mm²         |
| $f_{\text{Fck}/\gamma_M}$ | $-113$ N/mm²         |
| Load:     | $w_D$ = $0.8$ kN/m² |

Test results for the joint:

| $k_F$  | $1.5$ kN/m/mm |
| $F_j/\gamma_M$ | $4.063$ kN/m |

The following input values can be used in the calculation using the beam model:

Beam 1: $E = E_F = 210000$ N/mm²

$$ G = G_{C,L} = 3.4 \text{ N/mm}^2 $$

$$ I_{y,L} = \frac{B_F \cdot t_1 \cdot B_F \cdot t_2 \cdot e_C^2}{B_F \cdot t_1 + B_F \cdot t_2} = 3600000 \text{ mm}^4 $$

$$ \lambda = \sqrt{\frac{G_{C,L}}{G_F} \cdot \frac{t_1 + t_2}{e_C \cdot t_1 \cdot t_2}} = 1,183 \times 10^{-3} \frac{1}{\text{mm}} $$
Influence of openings on the behavior and resistance of sandwich panels

\[ I_T = 4 \cdot e_C^2 \cdot B \cdot \frac{t_1 + t_2}{t_1 \cdot t_2} \left( 1 - \frac{\tanh\left( \frac{\lambda \cdot B}{2} \right)}{\frac{\lambda \cdot B}{2}} \right) \frac{G_F}{G_{C,L}} = 3.51 \cdot 10^{10} \text{ mm}^4 \]

\[ A_L = B \cdot e_C = 120000 \text{ mm}^2 \]

Beam 2:
\[ E = E_v = 76184 \text{ N/mm}^2 \]
\[ G = G_{C,Q} = 2.82 \text{ N/mm}^2 \]
\[ I_{y,Q} = l_Q \cdot \frac{t_1 \cdot t_2}{t_1 + t_2} \cdot e_C^2 = 900000 \text{ mm}^4 \]
\[ A_Q = l_Q \cdot e_C = 30000 \text{ mm}^2 \]

Beam 3:
\[ E = E_F = 210000 \text{ N/mm}^2 \]
\[ A = \frac{k_F \cdot l_Q \cdot L_F}{E_F} = 0.1785 \text{ mm}^2 \]

Beam 4:
\[ EI_{y,\text{adapter}} = 100 \text{ kNm}^2 \quad \text{(if the adapter profiles have a different stiffness, the lower stiffness shall be used)} \]
\[ G = \infty = 1 \cdot 10^6 \]
\[ \text{load: } q_{\text{adapter}} = 0.4 \text{ kN/m} \]

The calculations using the beam model are given through the following results for beam 1:

\[ \text{Static system} \]
Influence of openings on the behavior and resistance of sandwich panels

Deflection in mm

Bending moment in kNm

Shear force in kN

Torsional moment in kNm

In the joint elements, the following normal forces exist:
Influence of openings on the behavior and resistance of sandwich panels

With these results, the verification at the ultimate limit state shall be made.

1) Wrinkling of the face:

\[ \gamma_F \cdot \left( M + k_e M_{y,\ell} \right) \leq \frac{\sigma_{w}}{\gamma_M} \]

An interpolation between the diagrams (Fig 13) leads to a factor \( k \) of 1,33. For the load factor, the value of \( g_F = 1.50 \) is applied:

\[ 1,50 \cdot \left( \frac{0,8 kN/m \cdot 1 m \cdot (5 m)^2}{8} + 1,33 \cdot 1,04 kN/m \right) \cdot \frac{60 mm}{3600000 mm^4} \leq 113 \frac{N}{mm^2} \]

\[ 97,1 \frac{N}{mm^2} \leq 113 \frac{N}{mm^2} \]

The verification of the strength is accepted.

2) Shear failure of the core layer (\( \gamma_M = 1,2 \)):

\[ \gamma_F \cdot \left( \frac{Q}{A_L} + \frac{Q_{z,L}}{25 cm \cdot e_c} \right) \leq \frac{f_{C,vk}}{\gamma_M} \]

\[ 1,5 \cdot \left( \frac{0,5 \cdot 0,8 kN/m \cdot 5 m}{120000 mm^2} + \frac{0,69 kN}{25 cm \cdot 12 cm} \right) \leq \frac{0,09 N}{mm^2} \]

\[ 0,0595 \frac{N}{mm^2} \leq 0,075 \frac{N}{mm^2} \]

The verification of the shear strength is accepted.
3) Failure of the longitudinal joint:

\[
\gamma_F \cdot \frac{N}{l_0} \leq \frac{F_j}{\gamma_M}
\]

\[
1,5 \cdot \frac{0,091 \, kN}{0,25 \, m} \leq 4,06 \frac{kN}{m}
\]

\[
0,546 \frac{kN}{m} \leq 4,06 \frac{kN}{m}
\]

The verification of the shear resistance of the joint is accepted.
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### Extend of Involvement of Task Groups and Working Commissions

- Some of the Activities and Outcome of this Task Group or Working Commission may be of special importance to the respective Theme or Area.
- Activities and Outcome of this Task Group or Working Commission in principle always are of special importance to the respective Theme or Area.

### Abbreviations of Defined Themes and Areas

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### Symbols

- ▲: Activities and Outcome of this Task Group or Working Commission in principle always are of special importance to the respective Theme or Area.
- ▼: Some of the Activities and Outcome of this Task Group or Working Commission may be of special importance to the respective Theme or Area.

### Fields

- **TG**: Task Group
- **WT**: Working Commission
- **SC**: Sustainable Construction
- **IDDS**: Integrated Design and Delivery Solutions
- **C&S - RU**: Construction and Society - Resilient Urbanisation

06/06/2013