DESIGN GUIDELINE

No.: D3.4 – part 1

Frameless buildings made of sandwich panels

Publisher: Saskia Käpplein
Thomas Misiek
Karlsruher Institut für Technologie (KIT)
Versuchsanstalt für Stahl, Holz und Steine

Task: 3.5

Objects: Design of axially and in-plane shear loaded sandwich panels in frameless buildings

This report includes 58 pages.

Date of issue: 27.10.2011

Versuchsanstalt für Stahl, Holz und Steine, Karlsruher Institut für Technologie (KIT) Tel.: +49 (0)721 608 42215
Abt. Stahl- und Leichtmetallbau, D-76128 Karlsruhe, Deutschland Fax: +49 (0)721 608 44078

Project co-funded under the European Commission Seventh Research and Technology Development Framework Programme (2007-2013)
Theme 4 NMP-Nanotechnologies, Materials and new Production Technologies

Prepared by

Saskia Käpplein, Thomas Misiek, Karlsruher Institut für Technologie (KIT), Versuchsanstalt für Stahl, Holz und Steine

Drafting History

Draft Version 1.1	12.10.2011
Draft Version 1.2	
Draft Version 1.3	
Draft Version 1.4	
Final	27.10.2011

Dissemination Level

PU	Public	X
PP	Restricted to the other programme participants (including the Commission Services)	
RE	Restricted to a group specified by the Consortium (including the Commission Services)	
CO	Confidential, only for members of the Consortium (including the Commission Services)	

Verification and approval

Coordinator	
Industrial Project Leader	
Management Committee	
Industrial Committee	

Deliverable

D3.4 – part 1: Design Guideline – Frameless buildings made of sandwich panels	Due date: Month 35 Completed: Month 37

Table of contents

Symbols and notations

A_C	cross section area of core
A_F	cross section area of face sheet
B	width of panel
B_S	flexural rigidity of panel
B/n	distance between connections at transverse edge
D	thickness of panel, for panels with flat or lightly profiled faces also distance between centroids of faces
E_C	elastic modulus of core material
E_F	elastic modulus of face sheets
EI_F	bending stiffness of face sheet
GA	shear rigidity of panel
G_C	shear modulus of the core
L	length of panel
L/n	distance between connections at longitudinal edge
M	bending moment
M^N	end moment due to eccentric axial load
N	normal force, axial load
N_{cr}	elastic buckling load of a sandwich panel
Q	variable load
V	transverse force

e	distance between centroids of faces
e_0	initial deflection
e*	distance between centroidal axis and face, loaded by axial fore (eccentricity)
f_{Cv}	shear strength of the core
t_F	thickness of face sheet
g	self-weight load
q_{e0}	equivalent load (initial deflection)
q_{WT}	equivalent load (temperature difference)
k	relationship between deflection due to shear and deflection due to bending, stiffness of a connection [kN/mm]
k_C	stiffness of a connection [N/mm]
k_v	stiffness of a fastening [kN/mm]
k_J	stiffness of connections of joints [kN/mm^2]
k_W	stiffness of connections between wall and foundation [kN/mm^2]
k_{Wt}	stiffness of connections between wall and roof (transverse edge) [kN/mm^2]
k_{Wl}	stiffness of connections between wall and roof (longitudinal edge) [kN/mm^2]

w deflection, wind load
w_b deflection due to bending
w_v deflection due to transverse force

α amplification factor, imperfection factor
α_F thermal expansion coefficient of faces
γ_F load factor
γ_M material safety factor
$\sigma_{cr,c}$ elastic buckling load (crippling)
$\sigma_{cr,w}$ elastic buckling load (wrinkling)
σ_c crippling stress
σ_F normal stress in the face sheet
σ_w wrinkling stress
λ_c slenderness of face (crippling)
λ_w slenderness of face (wrinkling)
X_c reduction factor (crippling)
X_w reduction factor (wrinkling)
τ_C shear stress in the core
φ_{St} sandwich creep coefficient
φ_t creep coefficient
Ψ combination coefficient

M^I, V^I, w^I moment, transverse force, deflection calculated by 1st Order Theory
M^{II}, V^{II}, w^{II} moment, transverse force, deflection calculated by 2nd Order Theory

Subscripts

G self-weight load, permanent load
Q variable load
S snow load
W wind load
T temperature load
d design
k characteristic
st short-term
lt long-term

1 Introduction

Sandwich panels are traditionally used as covering and isolating components, thus being secondary structural components of the building. The sandwich panels are mounted on a substructure and they transfer transverse loads as wind and snow to the substructure. The panels are subjected to bending moments and transverse forces only. A new application is to apply sandwich panels with flat or lightly profiled faces in smaller buildings – such as cooling chambers, climatic chambers and clean rooms – without any load transferring substructure (Fig. 1.1).

Fig. 1.1: Buildings made of sandwich panels but without substructure

In this new type of application in addition to space enclosure, the sandwich panels have to transfer loads and to stabilise the building. The wall panels have to transfer normal forces arising from the superimposed load from overlying roof or ceiling panels. So the wall panels have to be designed for axial loads or a combination of axial and transverse loads. Furthermore horizontal wind loads have to be transferred to the foundation and the building has to be stabilised. Because of the lack of a substructure the sandwich panels also have to transfer the horizontal loads. For this purpose the high in-plane shear stiffness and capacity of the sandwich panels is used.

Within the framework of work package 3 of the EASIE project, design methods for frameless structures made of sandwich panels have been developed. The investigations are documented in Deliverable D3.3 [2], [3], [4]. In the guideline at hand calculation procedures and design methods, which can be used for the design of frameless structures, are introduced. Calculation examples are given in Deliverable D3.5 [5].

2 Frameless buildings

Frameless buildings are made of sandwich panels only; they do not have a load-bearing substructure. The buildings consist of vertically spanning wall panels and roof panels, which are supported by the wall panels.

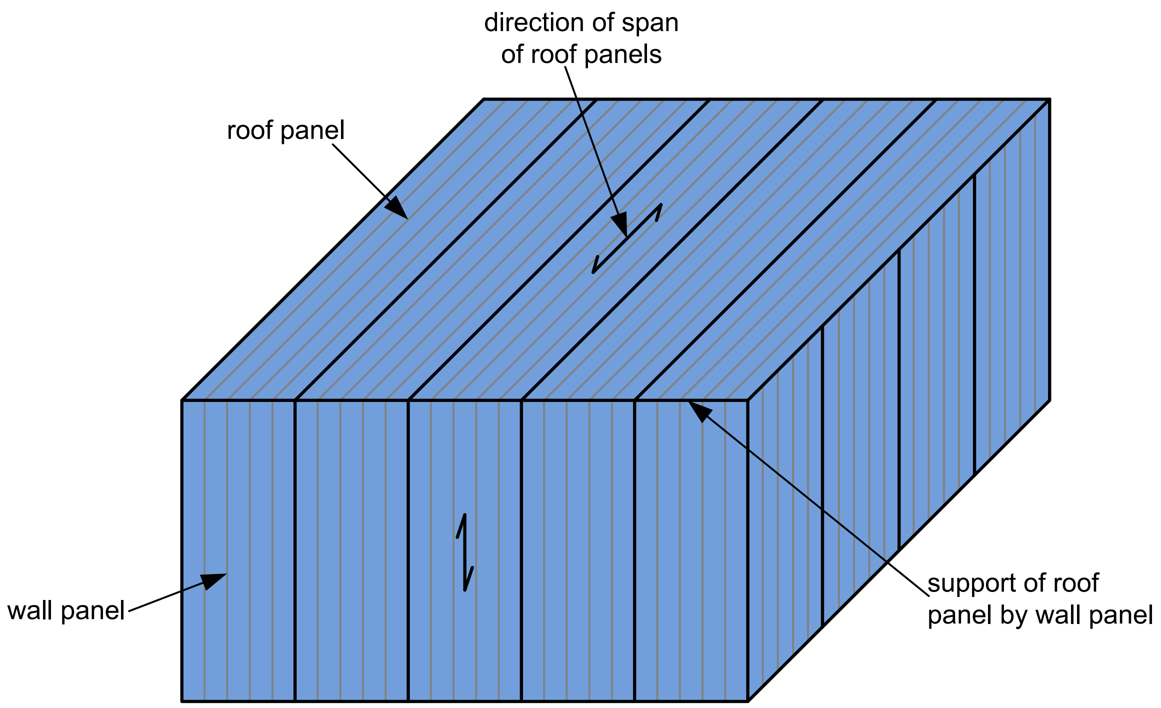

Fig. 2.1: Frameless building

At the connection between wall and roof normal forces as well as in-plane shear forces are introduced from the roof into rectangular adjacent wall panels. Normal forces are usually introduced by contact, whereas in-plane shear forces are introduced through mechanical fasteners. Wall and roof panels are connected by steel or aluminium angles, which are mechanically fastened to the face sheets of the panels (Fig. 2.2). For these fastenings usually self-drilling screws or blind rivets are used. To get a sufficiently stiff and strong connection it is recommended to connect both face sheets. Furthermore the angles should be comparatively stiff, i.e. they should be made of relatively thick sheets.

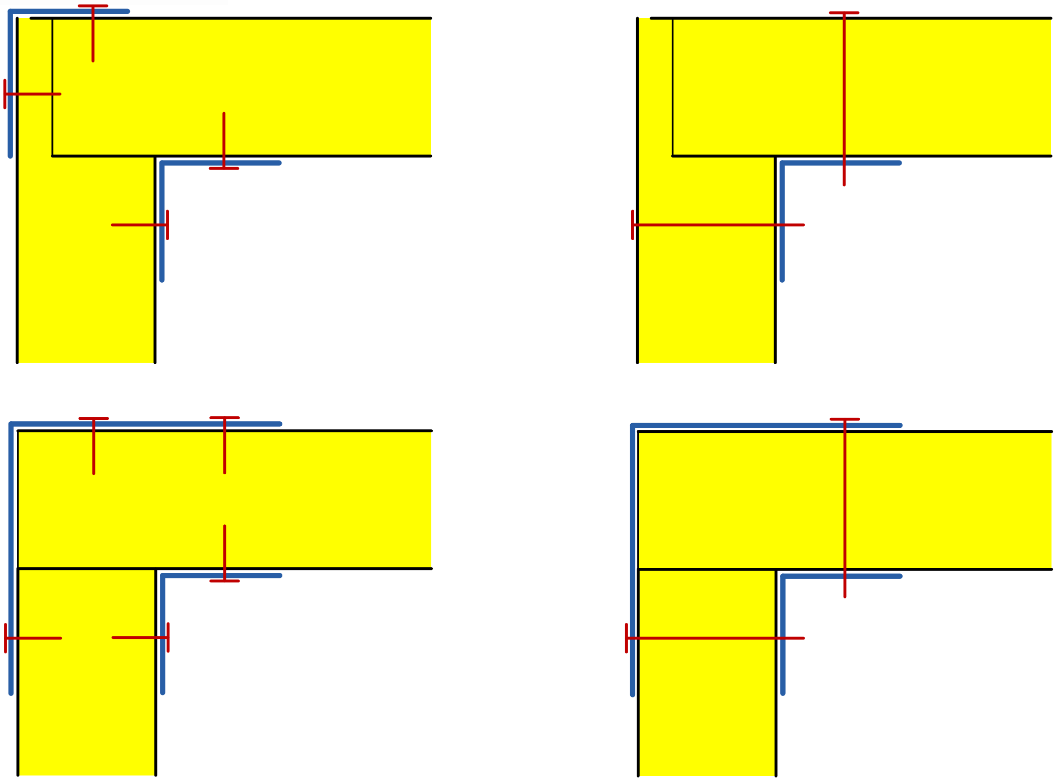

Fig. 2.2: Possible versions of connections between wall and roof

There are also some manufacturer-specific details available. E.g. wall and roof are connected by PVC-angles injected with polyurethane, with a came-lock system. In the guideline at hand only the standard solutions are considered.

The wall panels are either directly fixed to the foundation or they are connected to an additional panel, which builds the ground floor of the building. In this case the connection is very similar to the connection between wall and roof. The panels are fixed to comparatively stiff profiles by mechanical fasteners. Also for these connections fixing of both face sheets is recommended. In Fig. 2.3 some possible versions of the connection between wall and foundation are shown.

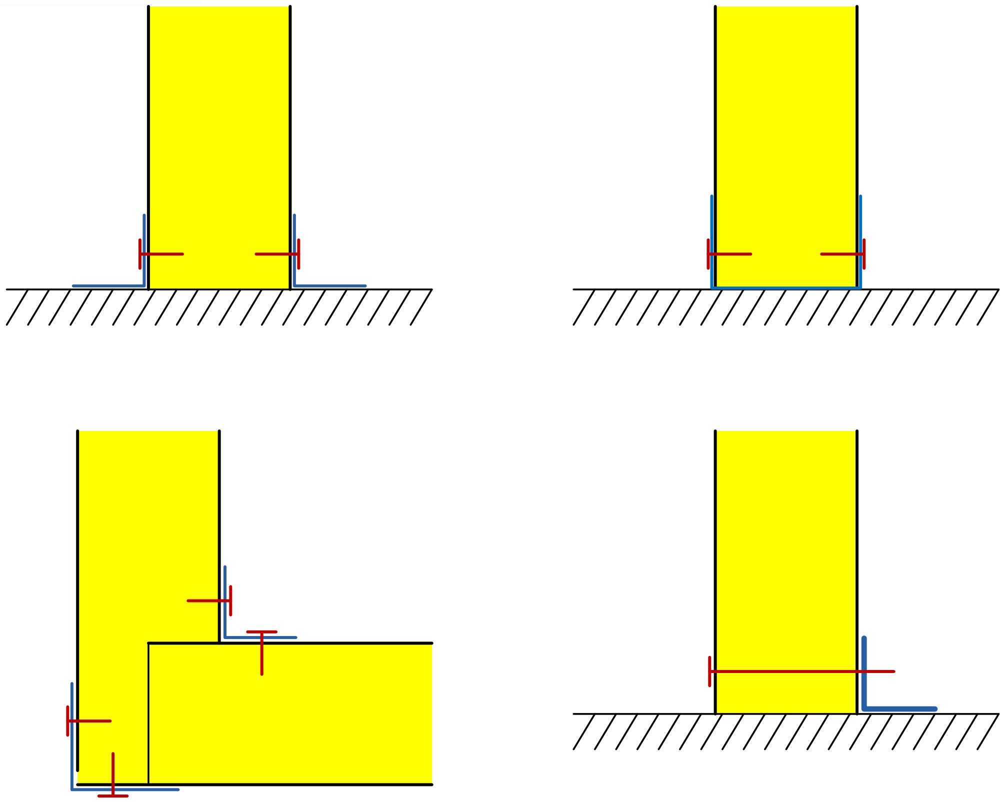

Fig. 2.3: **Possible versions of connections between wall and foundation**

A connection of the longitudinal joints of wall panels is not necessary, but it increases the stiffness and load-bearing capacity. Connecting the joints of roof panels, however, is mandatory. Transfer of horizontal loads via the roof is not possible, if the joints are not connected. To connect the joints mechanical fasteners as self-drilling screws or blind rivets can be used. To get strong and stiff joints both face sheets should be connected. It is possible to use one screw, which passes through the whole thickness of the panel or to use a pair of screws or blind rivets, one mounted from the internal and one from the external side.

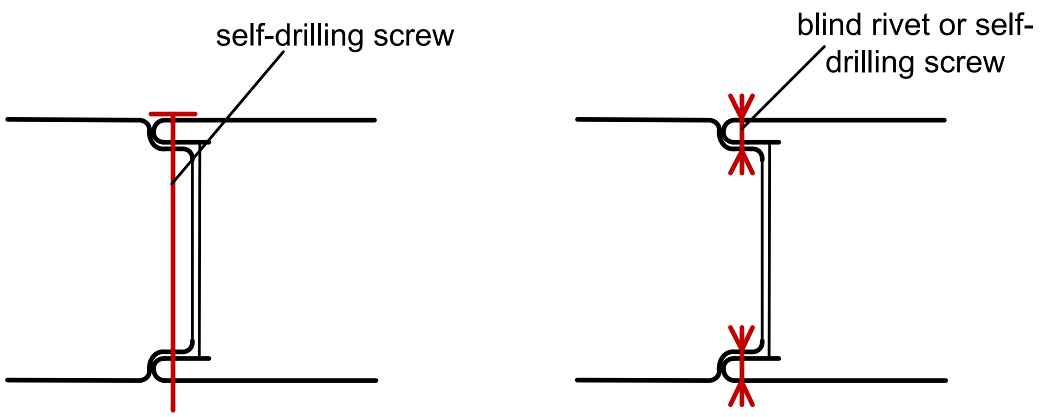

Fig. 2.4: **Connections of longitudinal joints**

The connections between wall panels at the corners of a building are not necessary for load transfer. They mainly contribute to the water tightness of the building.

3 Load bearing behaviour of sandwich panels

3.1 Transverse and axial load

3.1.1 General

In sandwich panels with flat or lightly profiled faces, bending moments and axial forces are transferred by normal stresses in the face sheets. A normal force is distributed to both face sheets in relation to the thickness of the faces.

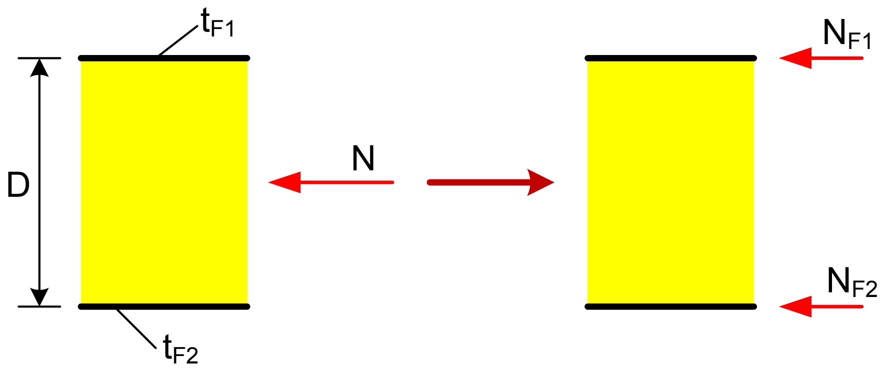

Fig. 3.1: Load bearing behaviour of panels subjected to an axial force

The normal forces acting on the face sheets are

$$N_{F1} = N \cdot \frac{A_{F1}}{A_{F1} + A_{F2}} \tag{3.1}$$

$$N_{F2} = N \cdot \frac{A_{F2}}{A_{F1} + A_{F2}} \tag{3.2}$$

with

$A_F = B \cdot t_F$ cross sectional area of a face sheet

t_F thickness of a face sheet

B width of panel

From the normal force the following normal stress results in the face sheets.

$$\sigma_{F1} = \sigma_{F2} = \frac{N}{A_{F1} + A_{F2}} \tag{3.3}$$

A bending moment M is transferred by a force couple in both face sheets.

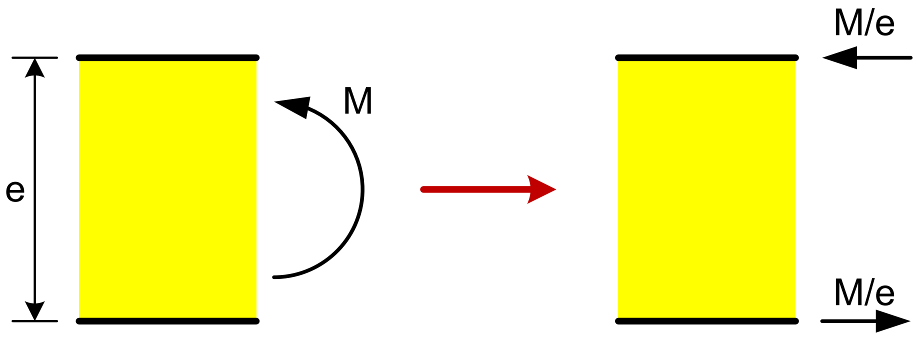

Fig. 3.2: Load bearing behaviour of panels subjected a bending moment

The stress in the face sheets of a sandwich panel loaded by a bending moment M is

$$\sigma_F = \frac{M}{e \cdot A_F} \tag{3.4}$$

with

e distance between centroids of the faces (for panels with flat or lightly profiled faces equal to thickness D of the panel)

The load bearing capacity of a sandwich panel subjected to bending moments and axial forces is mostly restricted by reaching the ultimate stress in the compressed face sheet (Fig. 3.3). Failure by yielding of the face sheet subjected to tension occurs very rarely. The face sheet represents a plate which is elastically supported by the core material. The stability failure of the compressed face sheet is termed wrinkling; the ultimate compression stress is termed wrinkling stress σ_w. Usually the wrinkling stress of a panel is determined by bending tests according to EN 14509 [6].

Fig. 3.3: Wrinkling of the compressed face sheet

The withstanding of transverse forces V is done by shear stresses in the core of the panel.

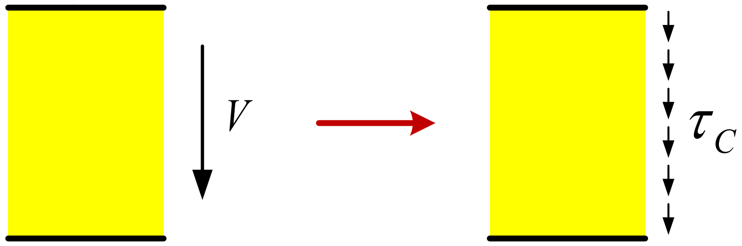

Fig. 3.4: **Load bearing behaviour of panels subjected to a transverse force**

The shear stress in the core is calculated by

$$\tau_C = \frac{V}{A_C} \tag{3.5}$$

with

$A_C = B \cdot e$ cross sectional area of the core

When reaching the shear strength f_{Cv} of the core material shear failure occurs (Fig. 3.5). Also the shear strength of the core material is determined by testing according to EN 14509.

Fig. 3.5: **Shear failure of the core**

Because of the relatively soft core layer for sandwich panels deflections caused by transverse forces have to be considered. So the deflection of a sandwich panel consists of the bending part w_b and the shear part w_v.

$$w = w_b + w_v = \int \frac{M\overline{M} \cdot dx}{B_S} + \int \frac{V\overline{V} \cdot dx}{GA} \tag{3.6}$$

with

bending stiffness:

$$B_S = E_F \cdot \frac{A_{F1} \cdot A_{F2}}{A_{F1} + A_{F2}} \cdot e^2 \tag{3.7}$$

shear stiffness:

$$GA = G_C \cdot A_C \tag{3.8}$$

E_F elastic modulus of the face sheets

G_C shear modulus of the core

3.1.2 Long-term behaviour

Both, organic core materials, such as polyurethane or expanded polystyrene, and mineral wool show creep effects under long-term loads, e.g. dead-weight load and snow. If a constant load acts on a panel over a long period of time, the shear strain γ increases with constant shear stress. The shear deformation of the core material and thus the shear part w_v of the deflection increase. For sandwich panels the time-dependent shear strain $\gamma(t)$ is usually described by a creep function $\varphi(t)$.

$$\gamma(t) = \gamma(0) \cdot \left(1 + \varphi(t)\right) \tag{3.9}$$

Therefore the time-dependent deflection of a sandwich panel with constant transverse loading is

$$w(t) = \int \frac{M\overline{M} \cdot dx}{B_S} + \int \frac{V\overline{V} \cdot dx}{GA} \cdot \left(1 + \varphi(t)\right) \tag{3.10}$$

For design purposes not the complete creep-function must be known. Usually only two creep coefficients φ_t are used. The creep coefficient φ_{2000} (at time t = 2000 h) is used to consider snow loads; the creep coefficient $\varphi_{100.000}$ (at time t = 100.000 h) is used to consider permanent loads (self- weight).

The creep coefficients are determined by long-term tests according to EN 14509. In (bending-) creep tests on panels subjected to a constant transverse dead load the deflection of the panel is measured for a period of 1000 hours and the creep function $\varphi(t)$ is recalculated. Based on the experimentally determined creep function, the creep coefficients φ_{2000} and $\varphi_{100.000}$ are determined by extrapolation. According to EN 14509 the creep coefficient φ_{2000} determined in the test has to be increased by 20%. So it is taken into account that a part of the creep deformation caused by snow loads during winter time is not recovered during summer time.

For transverse loaded single-span sandwich panels creeping of the core material results in an increase of deflections only, whereas bending moment and transverse force are not influenced. If a panel is additionally loaded by an axial load, an increase of deflection causes also an increase of bending moment and transverse force. Therefore, creep effects have not only to be considered in the design of serviceability limit state (deformation limit) but also in the design of ultimate limit state (load-bearing capacity), i.e. creeping must be taken into account for the determination of moment and transverse force.

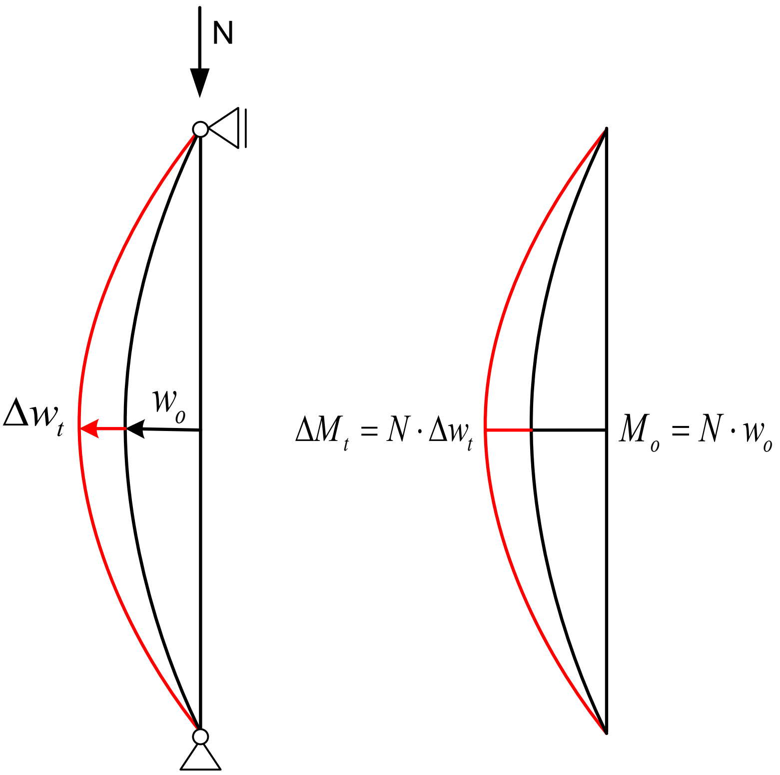

Fig. 3.6: Creeping of axially loaded panels

3.1.3 Effects of 2nd order theory

If (slender) building components are loaded by axial compression loads, effects of 2nd order theory have to be taken into account, i.e. deformations are considered in determination of bending moment and transverse force. If stress resultants are determined according to 1st order theory deformations are neglected. So if effects of 2nd order theory are considered, also geometrical imperfections such as initial deflections must be taken into account. Furthermore deflections, which are caused by temperature differences between inside and outside, cause additional bending moments and transverse forces.

Because of effects of 2nd order theory stresses do not increase proportionally to the axial load. Due to the axial force an increase of deflection also results in an increase of moment and transverse force.

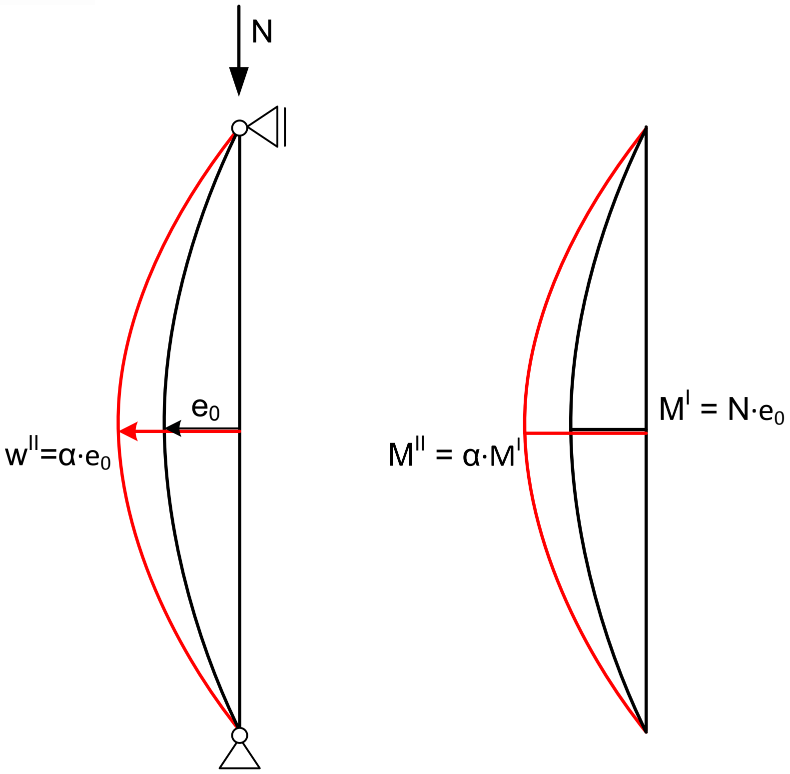

Fig. 3.7: **2nd order effects for a panel with initial deflection and axial load**

The increase of the stress resultants bending moment M and transverse force V as well as the amplification of the deflection w can approximately be considered by the amplification factor α.

$$M^{II} = \alpha \cdot M^{I} \tag{3.11}$$

$$V^{II} = \alpha \cdot V^{I} \tag{3.12}$$

$$w^{II} = \alpha \cdot w^{I} \tag{3.13}$$

$$N^{II} = N^{I} \tag{3.14}$$

$$\alpha = \frac{1}{1 - \frac{N}{N_{cr}}} \tag{3.15}$$

with

M^I, V^I, w^I moment, transverse force and deflection according to 1st order theory

M^{II}, V^{II}, w^{II} moment, transverse force and deflection according to 2nd order theory

For determination of the amplification factor the elastic buckling load N_{cr} of the sandwich panel has to be determined. The elastic buckling load of a sandwich panel loaded by a centric axial load consists of the part N_{ki} considering the bending rigidity of the face sheets and the part GA considering the shear rigidity of the core. The part N_{ki} corresponds to the elastic buckling load of both face sheets.

$$N_{cr} = \frac{N_{ki}}{1 + \frac{N_{ki}}{GA}} \qquad (3.16)$$

with

elastic buckling load due to bending rigidity

$$N_{ki} = \frac{\pi^2 \cdot B_S}{L^2} \qquad (3.17)$$

elastic buckling load due to shear rigidity

$$GA = G_C \cdot A_C \qquad (3.18)$$

3.2 Area of load application of axially loaded panels

Axial forces may not only cause wrinkling failure in mid-span, but also a local failure at the load application area, where the normal force is introduced into the panel, e.g. at the connection between wall and roof or between wall and foundation. For the common load application details (cf. Fig. 2.3) the axial force is introduced into the face of the wall panel by contact. The failure mode of the load application area is usually crippling of the face at the loaded cut edge (Fig. 3.8). This stability failure mode is strongly related to crippling of the compressed face in mid-span. The ultimate stress of the compressed face sheet at the free edge is termed as crippling stress σ_c.

Fig. 3.8: Crippling of the face at load application area

3.3 In-plane shear load

Sandwich panels have a very high stiffness and load bearing capacity, when loaded by in-plane shear forces. Both, stiffness and load bearing capacity are very much higher than the corresponding values of the fastenings. So if sandwich panels are designed for in-plane shear loads the shear deformation of the panels can be neglected. Only the flexibility of the fastenings has to be considered. Also for the load-bearing capacity only the fastenings are decisive. Thus, if a sandwich panel is loaded by in-plane shear loads, the fastenings have to be designed for this load.

4 Loads on frameless structures

4.1 Characteristic loads

Self-weight (G) and snow (S) loads

Roof panels are loaded by self-weight and snow loads. Snow loads can be determined according to EN 1991-1-3 [11] and the related national annex or according to national standards.

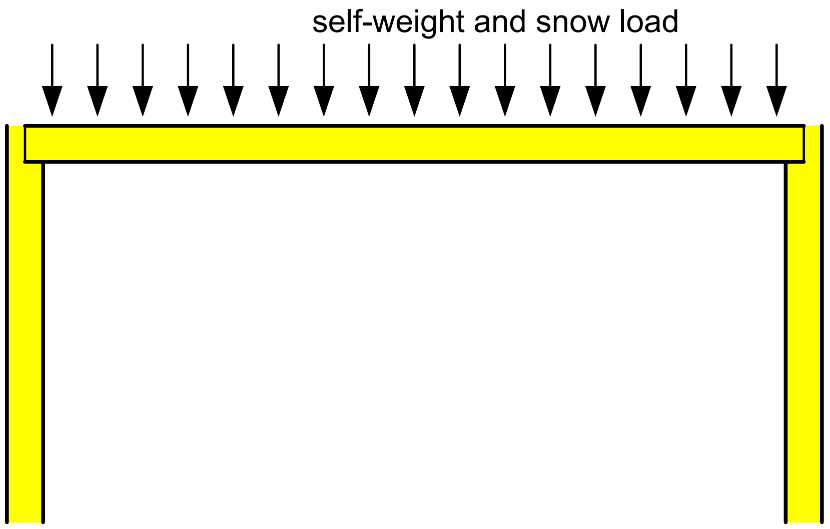

Fig. 4.1: Self-weight and snow loads

Wind loads (W)

Roof as well as wall panels are loaded by wind loads. They wind load is applied to the panels as transverse load. Horizontal wind loads are transferred from the directly loaded walls into the roof, where they cause in-plane shear forces. From the roof they are transferred to the walls. In these walls also in-plane shear forces occur. Finally they are introduced into the foundation. The characteristic wind loads can be determined according to EN 1991-1-4 [12] and the related national annex or according to national standards.

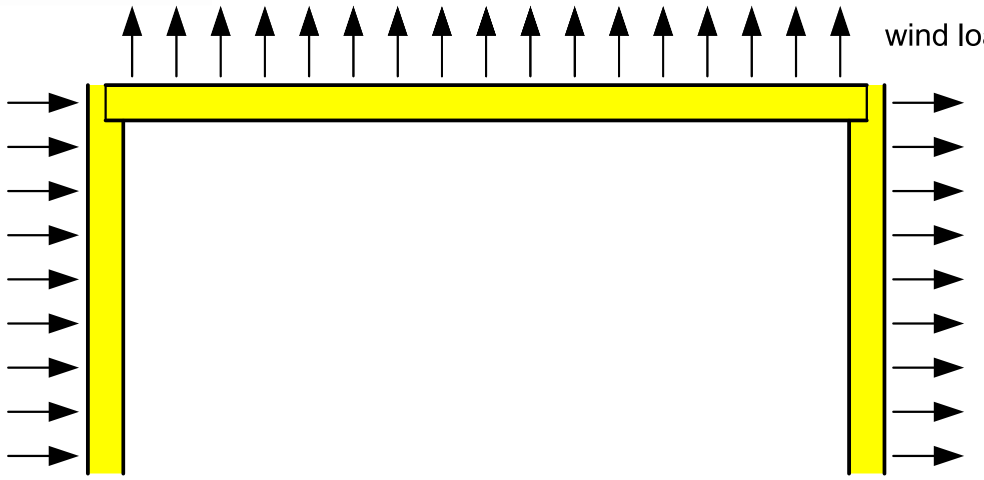

Fig. 4.2: Wind loads

Temperature differences between inside and outside

Temperature differences between inside and outside cause deflections as well in wall as in roof panels. For a single span panel the deflection caused by a temperature difference ΔT is

$$w_T = \Delta T \cdot \frac{\alpha_F \cdot L^2}{8 \cdot e} \tag{4.1}$$

with

α_F coefficient of thermal expansion of faces

For single span panels in conventional applications, i.e. for panels without axial load, this deflection does not lead to additional stress resultants. But if a panel is also loaded by an axial force a deflection causes additional bending moments and transverse forces.

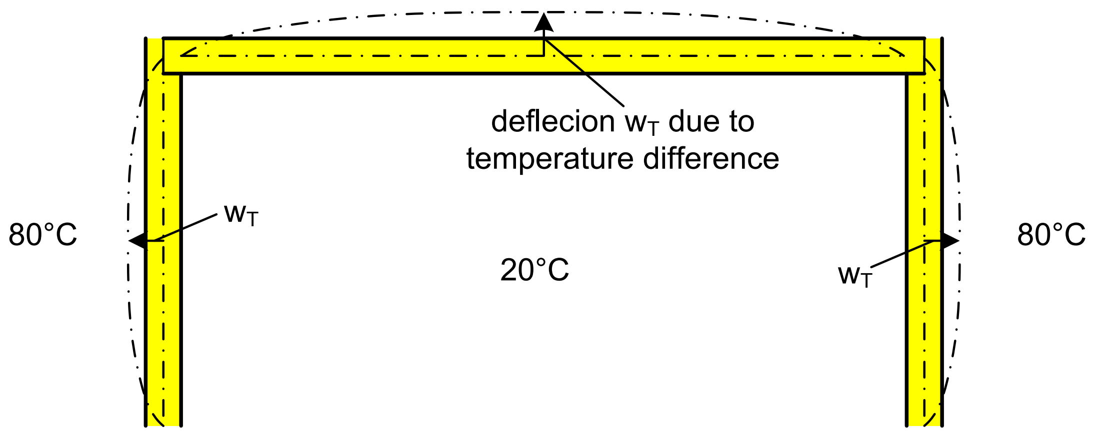

Fig. 4.3: Temperature differences

Note:

If multi-span panels are subjected to temperature differences, moments and transverse forces occur, even if there are not any axial forces. In EN 14509 calculation formulae are given for these cases.

Geometrical imperfections

If building components are axially loaded also geometrical imperfections have to be considered when calculating moments and transverse forces. As geometrical imperfection an initial deflection e_0 is applied. The range of this imperfection can be selected according to the maximum allowable longitudinal bowing following EN 14509.

$$e_0 = \frac{1}{500} \cdot L \leq 10mm \tag{4.2}$$

Together with the axial force the initial deflection causes bending moments and transverse forces.

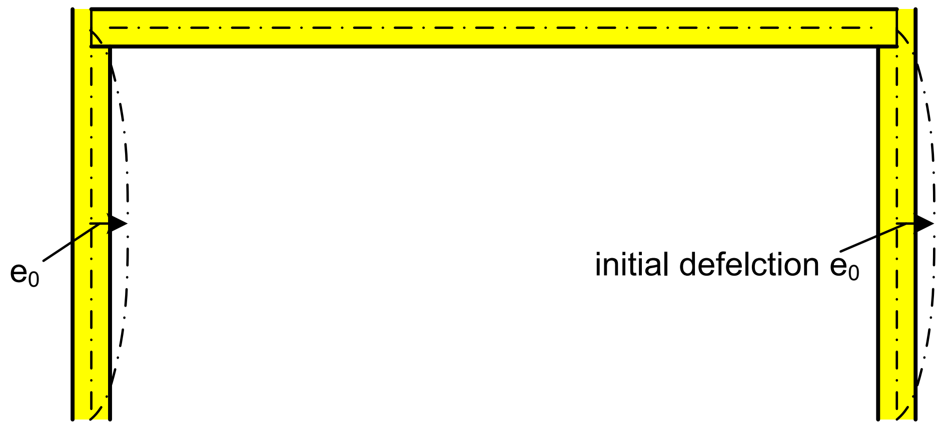

Fig. 4.4: Initial deflection of wall panels

4.2 Design loads

Design loads are determined by multiplying the characteristic load by the corresponding load factor γ_F and by the corresponding combination coefficient Ψ. They are given in EN 1990 [9] or national standarts. Load factors and combination coefficients for the design of sandwich panels are also given in EN 14509. A summary of the design loads is given in the following table.

	permanent action (self-weight) G_d	dominant variable action Q_{d1}	other variable actions Q_{di}
ultimate limit state	$\gamma_G \cdot G_k$	$\gamma_{Q1} \cdot Q_{k1}$	$\gamma_{Qi} \cdot \psi_{0i} \cdot Q_{ki}$
serviceability limit state (displacements)	G_k	$\psi_{11} \cdot Q_{k1}$	$\psi_{0i} \cdot \psi_{1i} \cdot Q_{ki}$

Tab. 4.1: Design loads according to EN 14509, table E.4 and E.5

The loads factors γ_F are also given in EN 14509. They have to be used if there are not any values available in national standards.

	ultimate limit state	serviceability limit state (displacements)
permanent action G	1,35 (1,00)	1,00
variable action	1,50	1,00
temperature actions ΔT	1,50	1,00
creep effects	1,00	1,00

Tab. 4.2: Load factors according to EN 14509, table E.8

Also combination coefficients Ψ are given in EN 14509. As an alternative the combination coefficients according to EN 1990 or according to national standards may be used.

	snow	wind	temperature
Ψ_0	0,6	0,6	0,6 / 1,0[a)]
Ψ_1	0,75 / 1,0[b)]	0,75 / 1,0[b)]	1,0

[a)] Ψ_0 = 1,0 is used if the winter temperature T_1 = 0°C is combined with snow

[b)] Ψ_1 = 0,75 for snow and wind is used if the combination includes the action effects of two or more variable actions and coefficient ψ_1 = 1,0 for snow and wind is used if there is, in the combination, only a single action effect representing the variable actions and it is caused by either the sole snow load or the sole wind load, acting alone.

Tab. 4.3: Combination coefficients according to EN 14509, table E.6

5 Resistance values

5.1 Sandwich panels

The characteristic resistance values and some additional calculation requirements are usually given on the CE-mark of the sandwich panels. For design of the panels, which are considered in the report at hand, the following resistance values and additional calculation requirements are needed.

- Wrinkling stress of a compressed face sheet σ_w
- Yield strength of face sheet f_y
- Shear strength of core f_{Cv}
- Shear strength after long-term loading $f_{Cv,t}$
- Design thickness of the faces t_F
- Shear modulus of the core material G_C
- Elastic modulus of the core material E_C
- Creep coefficients φ_{2000} for t = 2000 h (snow) and $\varphi_{100.000}$ t = 100000 h (permanent load)

- Crippling stress at load application area σ_c (cf. section 8.3)

The design resistance values are determined by dividing the characteristic values by an appropriate material factor γ_M.
Wrinkling stress of compressed face sheet:

$$\sigma_{w,d} = \frac{\sigma_{w,k}}{\gamma_M} \tag{5.1}$$

Yielding strength of a face sheet:

$$f_{y,d} = \frac{f_{y,k}}{\gamma_M} \tag{5.2}$$

Shear strength of core:

$$f_{Cv,d} = \frac{f_{Cv,k}}{\gamma_M} \tag{5.3}$$

Shear strength after long-term loading

$$f_{Cv,t,d} = \frac{f_{Cv,t,k}}{\gamma_M} \tag{5.4}$$

Crippling stress at load application area:

$$\sigma_{c,d} = \frac{\sigma_{c,k}}{\gamma_M} \tag{5.5}$$

For sandwich panels the material factors γ_M represent the variability of the mechanical properties of the panel. They are determined by the results of initial type testing and factory production control.

5.2 Fastenings

In addition to the panels the fastenings have to be designed. Usually the load-bearing capacity of the fastenings is determined by testing. The characteristic values are given in European technical approvals (ETA) or in national approvals. For some kinds of fastenings the characteristic values can also be determined by calculation. E.g. the characteristic values of fastenings of metal sheeting can be calculated according to EN 1993-1-3 [13]. For direct fixings of sandwich panels calculation formulae are given in Deliverable D3.3 – part 3 [1].
The design value of the load-bearing capacity of a fastening is determined by dividing the characteristic value by the material factor γ_M.

$$V_{Rd} = \frac{V_{Rk}}{\gamma_M} \tag{5.6}$$

The material factor γ_M is given by national specifications. According to EN 1993-1-3 γ_M = 1,25, according to different approvals and EN 14509 γ_M = 1,33 has to be used.

6 Design of roof panels

6.1 Static system and loads

The roof panels are supported by the wall panels. They introduce self-weight and snow loads as normal forces into the walls. At external walls the roof panels are usually supported only by the internal face of the wall panels. For design of the roof panels this support can be regarded as hinged. The hinged support is located at the internal face sheet of the wall panel.

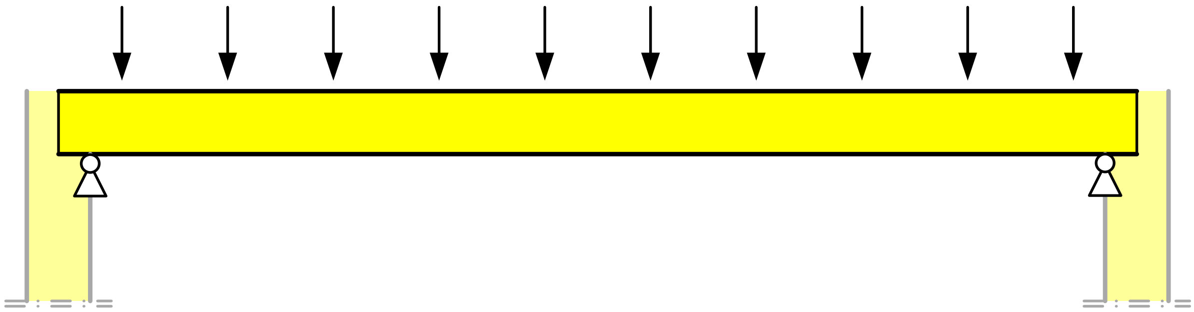

Fig. 6.1: Static system of roof panels

If there are also internal walls, which support roof panels, the roof panels are multi-span beams. The interior walls also represent hinged supports.

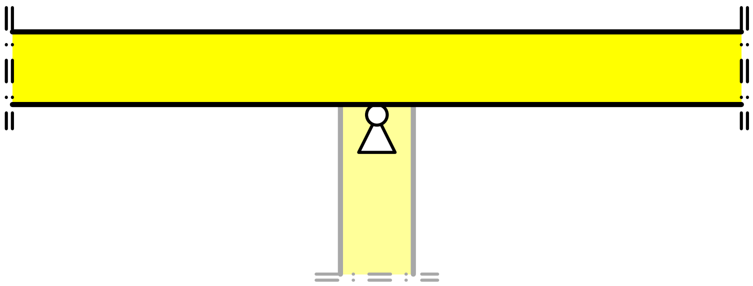

Fig. 6.2: Support of a roof panel by an interior wall

The roof of a frameless building is loaded by self-weight, snow and wind loads. Also temperature differences between inside and outside may be available. As stress resultants there are bending moments and transverse forces.

6.2 Design procedures

6.2.1 Preliminary remark

Roof panels are loaded by transverse loads only. As stress resultants bending moments and transverse forces act. So the conventional design procedures according to EN 14509 are sufficient to design roof panels of frameless structures. For self-weight and snow loads creeping of the core material has to be considered.

In the following exemplarily the design formulae for single span panels are summarised. Formulae for two- and three-span panels can be found in EN 14509, Table E.10.

6.2.2 Deflections

According to EN 14509 the deflection of sandwich panels has to be limited. The deflection is calculated for serviceability loads (cf. section 4).

The deflection of a single-span panel loaded by a uniform transverse load q is

$$w = w_b + w_v = \frac{5}{384} \cdot \frac{q \cdot L^4}{B_S} + \frac{1}{8} \cdot \frac{q \cdot L^2}{GA} \tag{6.1}$$

Also temperature differences ΔT between inside and outside cause a deflection

$$w_{\Delta T} = \frac{\alpha_F \cdot \Delta T}{e} \cdot \frac{L^2}{8} \tag{6.2}$$

α_F thermal expansion coefficient of faces

Due to creep effects of the core material the shear part w_v of the deflection increases for long-term loads. The additional deflection caused by creep effects is determined by creep coefficients φ_t.

$$w_t = w_{S,b} + w_{S,v}\left(1 + \varphi_{2000}\right) + w_{G,b} + w_{G,v}\left(1 + \varphi_{100.000}\right) + w_{\Delta T} \tag{6.3}$$

w_S deflection due to snow load (long-term load, t = 2000 h)

w_G deflection due to permanent load (long-term load, t = 100.000 h)

$w_{\Delta T}$ deflection due to temperature difference

φ_{2000} creep coefficient t = 2000 h

φ_{100000} creep coefficient t = 100000 h

Note:

Temperature differences, which are caused by climatic effects, can be considered as short-term loads [7]. E.g. in cooling rooms, where temperature differences are not caused by climatic effects, they have to be considered as long-term loads.

In EN 14509 for roofs and ceilings the following deflection limits are given.

Short-term loading

$$w \leq \frac{L}{200} \tag{6.4}$$

Long-term loading (including creep effects)

$$w_t \le \frac{L}{100} \tag{6.5}$$

Note:

The short-term deflection is the initial deflection, i.e. the deflection at t=0; no creep effects are considered. It includes deflections due to short-term loads (e.g. wind) and also deflections due to long-term loads (e.g. snow and self-weight). The long-term deflection consists of the short-term deflection and the additional deflection caused by creeping.

6.3 Stresses in face sheets and core

Also the stresses in the face sheets and in the core of a sandwich panels have to be limited. This is done for the ultimate limit state (cf. section 4).

A bending moment M causes the following normal stresses in the face sheets.

$$\sigma_{F,d} = \frac{M_d}{e \cdot A_F} \tag{6.6}$$

The stress in the compressed face is compared to the wrinkling stress.

$$\sigma_{F,d} \le \sigma_{w,d} \tag{6.7}$$

The stress in the face subjected to tension is compared to the yield strength.

$$\sigma_{F,d} \le f_{y,d} \tag{6.8}$$

A transverse force V causes shear stresses in the core.

$$\tau_{C,d} = \frac{V_d}{A_C} \tag{6.9}$$

The shear stress is divided into two parts – the short-term shear stress $\tau_{C,st}$ is caused by short-term loads and the long-term shear stress $\tau_{C,lt}$ by long-term loads (snow, self-weight).

$$\tau_{C,st,d} = \frac{V_{st,d}}{A_C} \tag{6.10}$$

$$\tau_{C,lt,d} = \frac{V_{lt,d}}{A_C} \tag{6.11}$$

V_{st} transverse force due to short-term loads

V_{lt} transverse force due to long-term loads

The shear stresses are compared to the shear strength of the core material.

$$\frac{\tau_{C,st,d}}{f_{Cv,d}} + \frac{\tau_{C,lt,d}}{f_{Cv,t,d}} \le 1 \tag{6.12}$$

Note:

For determination of moments, transverse forces and deflections of panels with three and four supports (and also for panels with profiled faces) calculation formulae are given EN 14509, Annex E. In these systems creeping influences not only the deflection but also bending moment and transverse force. Therefore the stresses in the face sheets and in the core are also time-dependent. To consider this the shear modulus G_C can be reduced to the notional time-dependent shear modulus G_{Ct}.

$$G_{Ct} = \frac{G_C}{1+\varphi_t} \tag{6.13}$$

7 Design of wall panels for axial and transverse load

7.1 Preliminary remark

In EN 14509 [6] no design methods for axially loaded sandwich panels are given. Within the framework of work package 3 of the EASIE project design procedures for sandwich panels with axial load or a combination of axial and transverse load have been developed. The design model is based upon the existing model for sandwich panels subjected to transverse loads according to EN 14509. The existing model is extended in a way that consideration of axial forces and influences of 2nd order theory is possible. Also the behaviour due to long-term loads (creeping of the core material) is considered in this model. To design sandwich panels for axial load no additional tests are necessary.

7.2 Static systems

The wall panels of frameless buildings are usually single span elements. They have one support at the connection between wall and foundation and the other one at the connection between wall and roof. For design purposes both supports can be regarded as hinged. The wall panels are loaded by transverse loads as wind and equivalent line loads resulting from geometrical imperfections (initial deflections) and deflections due to temperature differences. Furthermore they are loaded by the axial force, which is introduced from the roof. The axial force is usually introduced in one face sheet only. Because of this eccentricity an end moment occurs. Depending on the kind of connection there is an end-moment on both ends of the wall panel or only at the upper end, where the wall is connected to the roof.

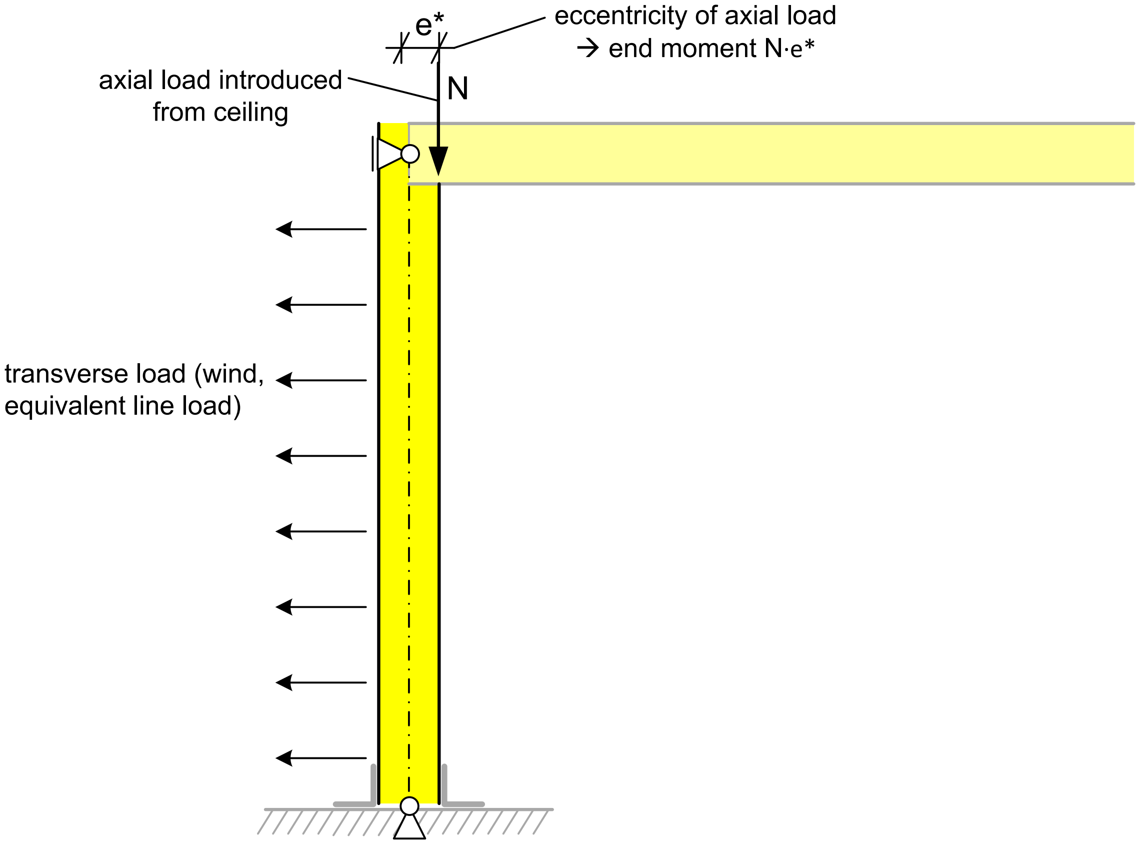

Fig. 7.1: Static system of wall panels

At the lower end the reaction forces are directly transferred to the foundation. At the upper end the horizontal reaction forces are introduced into the roof. So the roof panels are loaded by in-plane shear forces. Through the roof the in-plane shear forces are transferred to the walls, which are parallel to the direction of load, and finally they are introduced in the foundation (cf. section 9).

7.3 Loads on wall panels

Axial loads

Dead weight loads and snow loads are introduced from the roof into the wall panels as axial loads.

N_G	dead weight load	(7.1)
N_S	snow load	(7.2)

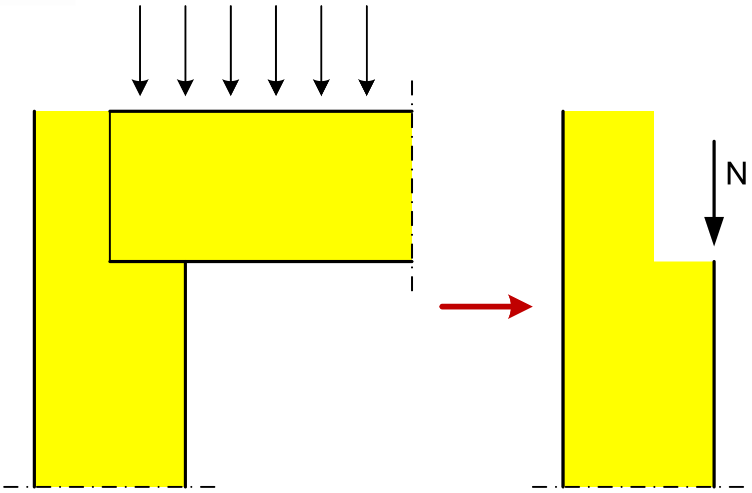

Fig. 7.2: **Axial load**

End moments

If the axial load is introduced in one face sheet only, additional end-moments occur.

$M_G^N = N_G \cdot e^*$ dead weight load (7.3)

$M_S^N = N_S \cdot e^*$ snow load (7.4)

with

e* distance of loaded face from centroid of the wall panel

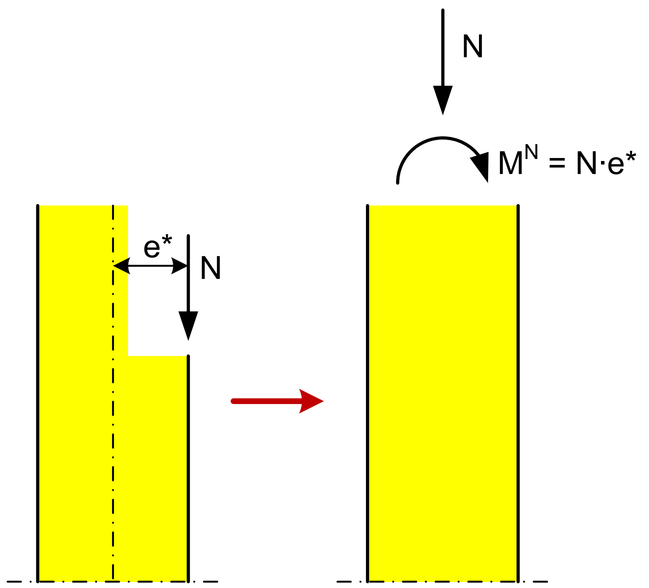

Fig. 7.3: **End moment**

Transverse loads

Wind is applied to the panel as a distributed transverse load.

w wind load (7.5)

As geometrical imperfection an initial deflection e_0 is assumed. The initial deflection and the axial load cause additional bending moments and transverse forces. Therefore the initial deflection e_0 is considered by equivalent line loads q_{e0}.

$$q_{e0,G} = N_G \cdot 8 \cdot \frac{e_0}{L^2} \quad \text{dead weight load} \tag{7.6}$$

$$q_{e0,S} = N_S \cdot 8 \cdot \frac{e_0}{L^2} \quad \text{snow load} \tag{7.7}$$

Temperature differences between inside and outside cause a deflection w_T of the panel.

$$w_T = \Delta T \cdot \frac{\alpha_F \cdot L^2}{e \cdot 8} \tag{7.8}$$

Together with the axial force the deflection w_T causes bending moments and transverse forces. The deflection is considered by equivalent line loads q_{WT}.

$$q_{wT,G} = N_G \cdot 8 \cdot \frac{w_T}{L^2} = N_G \cdot \Delta T \cdot \frac{\alpha_F}{e} \quad \text{dead weight load} \tag{7.9}$$

$$q_{wT,S} = N_S \cdot 8 \cdot \frac{w_T}{L^2} = N_S \cdot \Delta T \cdot \frac{\alpha_F}{e} \quad \text{snow load} \tag{7.10}$$

In the following figure the loads usually acting on a wall panel of a frameless structure are summarised. The both most common static systems are shown. In system (a) at the upper end the axial loads are introduced into one face sheet and at the lower end they are introduced into both face sheets. In system (b) at both ends the axial loads are introduced into one face sheet only.

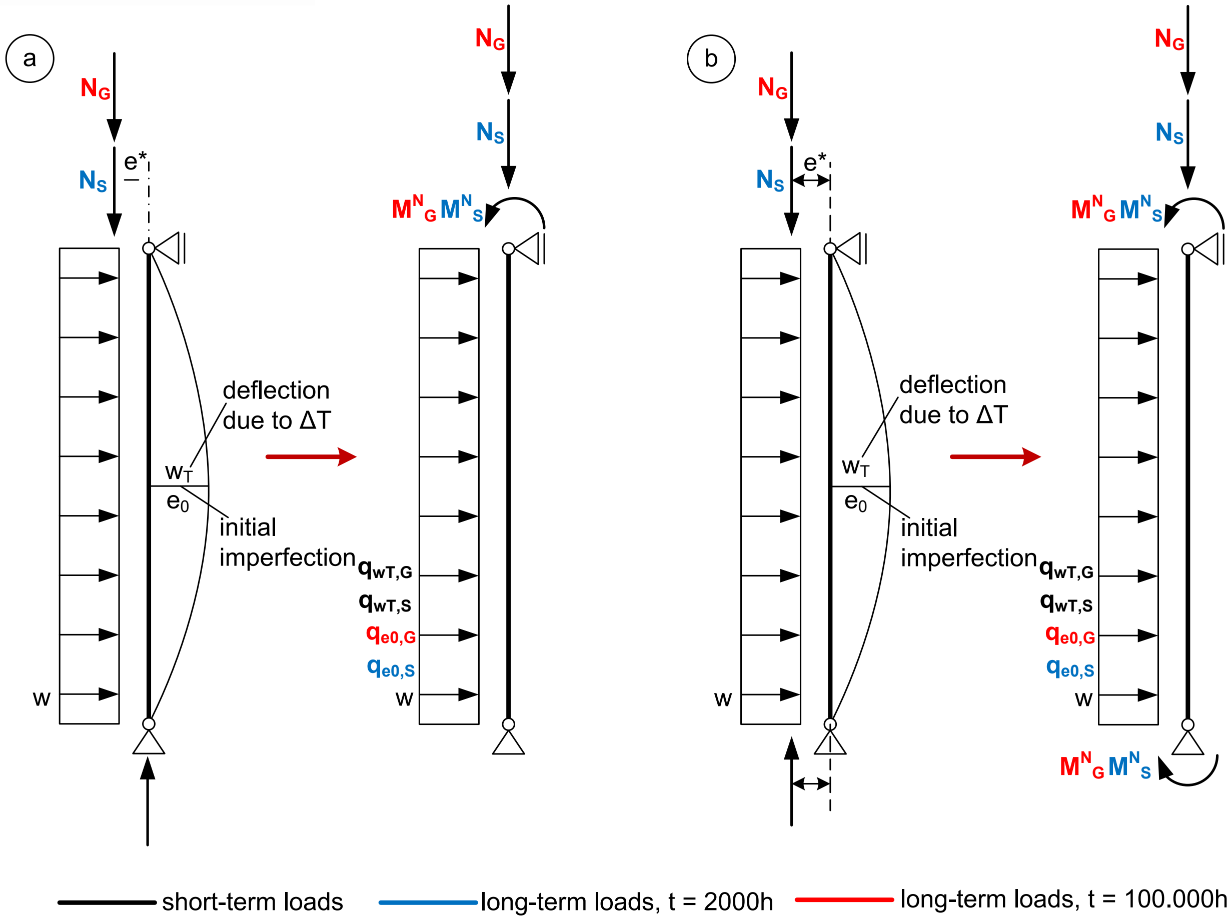

Fig. 7.4: Loads on wall panels

7.4 Deflections

7.4.1 1st order theory without consideration of creep effects

The deflections according to 1st order theory without consideration of creep effects are calculated by the following formulae.

$$w^I = w_W^I + w_{\Delta T}^I + w_S^I + w_G^I \tag{7.11}$$

with

- Deflection due to wind load (W)

$$w_W^I = w_{W,b}^I(w) + w_{W,v}^I(w) \tag{7.12}$$

- Deflection due to temperature difference (ΔT)

$$w_{\Delta T}^I = w_{\Delta T,b}^I(q_{wT,S}) + w_{\Delta T,v}^I(q_{wT,S}) + w_{\Delta T,b}^I(q_{wT,G}) + w_{\Delta T,v}^I(q_{wT,G}) \tag{7.13}$$

- Deflections due to snow load (S)

$$w_S^I = w_{S,b}^I(q_{e0,S}) + w_{S,v}^I(q_{e0,S}) + w_{S,b}^I(M_S^N) + w_{S,v}^I(M_S^N) \tag{7.14}$$

- Deflection due to self-weight load (G)

$$w_G^I = w_{G,b}^I(q_{e0,G}) + w_{G,v}^I(q_{e0,G}) + w_{G,b}^I(M_G^N) + w_{G,v}^I(M_G^N) \tag{7.15}$$

The single components of the deflections are calculated by the following formulae:

- Wind loads (W)

$$w^{I}_{W,b}(w) = \frac{5}{384} \cdot \frac{w \cdot L^4}{B_S} \tag{7.16}$$

$$w^{I}_{W,v}(w) = \frac{1}{8} \cdot \frac{w \cdot L^2}{GA} \tag{7.17}$$

- Snow load and temperature difference

$$w^{I}_{\Delta T,b}(q_{wT,S}) = \frac{5}{384} \cdot \frac{q_{wT,S} \cdot L^4}{B_S} = \frac{5}{48} \cdot \frac{N_S \cdot w_T \cdot L^2}{B_S} \tag{7.18}$$

$$w^{I}_{\Delta T,v}(q_{wT,S}) = \frac{1}{8} \cdot \frac{q_{wT,S} \cdot L^2}{GA} = \frac{N_S \cdot w_T}{GA} \tag{7.19}$$

- Self-weight load and temperature difference

$$w^{I}_{\Delta T,b}(q_{wT,G}) = \frac{5}{384} \cdot \frac{q_{wT,G} \cdot L^4}{B_S} == \frac{5}{48} \cdot \frac{N_G \cdot w_T \cdot L^2}{B_S} \tag{7.20}$$

$$w^{I}_{\Delta T,v}(q_{wT,G}) = \frac{1}{8} \cdot \frac{q_{wT,G} \cdot L^2}{GA} = \frac{N_G \cdot w_T}{GA} \tag{7.21}$$

- Snow load and initial deflection

$$w^{I}_{S,b}(q_{e0,S}) = \frac{5}{384} \cdot \frac{q_{e0,S} \cdot L^4}{B_S} = \frac{5}{48} \cdot \frac{N_S \cdot e_0 \cdot L^2}{B_S} \tag{7.22}$$

$$w^{I}_{S,v}(q_{e0,S}) = \frac{1}{8} \cdot \frac{q_{e0,S} \cdot L^2}{GA} = \frac{N_S \cdot e_0}{GA} \tag{7.23}$$

- End moment due to snow load

$$w^{I}_{S,b}(M^{N}_{S}) = \frac{1}{16} \cdot \frac{M^{N}_{S} \cdot L^2}{B_S} = \frac{1}{16} \cdot \frac{N_S \cdot e^* \cdot L^2}{B_S} \quad \text{system (a)} \tag{7.24}$$

$$w^{I}_{S,b}(M^{N}_{S}) = \frac{1}{8} \cdot \frac{M^{N}_{S} \cdot L^2}{B_S} = \frac{1}{8} \cdot \frac{N_S \cdot e^* \cdot L^2}{B_S} \quad \text{system (b)} \tag{7.25}$$

$$w^{I}_{S,v}(M^{N}_{S}) = 0 \quad \text{system (a) and (b)} \tag{7.26}$$

- Self-weight load and initial deflection

$$w^{I}_{G,b}(q_{e0,G}) = \frac{5}{384} \cdot \frac{q_{e0,G} \cdot L^4}{B_S} = \frac{5}{48} \cdot \frac{N_G \cdot e_0 \cdot L^2}{B_S} \tag{7.27}$$

$$w^{I}_{G,v}(q_{e0,G}) = \frac{1}{8} \cdot \frac{q_{e0,G} \cdot L^2}{GA} = \frac{N_G \cdot e_0}{GA} \tag{7.28}$$

- End moment due to self-weight load

$$w^I_{G,b}(M^N_G) = \frac{1}{16} \cdot \frac{M^N_G \cdot L^2}{B_S} = \frac{1}{16} \cdot \frac{N_G \cdot e^* \cdot L^2}{B_S}$$ system (a) (7.29)

$$w^I_{G,b}(M^N_G) = \frac{1}{8} \cdot \frac{M^N_G \cdot L^2}{B_S} = \frac{1}{8} \cdot \frac{N_G \cdot e^* \cdot L^2}{B_S}$$ system (b) (7.30)

$$w^I_{G,v}(M^N_G) = 0$$ system (a) and (b) (7.31)

7.4.2 Effects of 2nd order theory

The effects of 2nd order theory are considered by the amplification factor α.

$$\alpha = \frac{1}{1 - \dfrac{N}{N_{cr}}} \tag{7.32}$$

with

Elastic buckling load

$$N_{cr} = \frac{N_{ki}}{1 + \dfrac{N_{ki}}{GA}} \tag{7.33}$$

Bending part of elastic buckling load

$$N_{ki} = \frac{\pi^2 \cdot B_S}{L^2} \tag{7.34}$$

Shear part of elastic buckling load

$$GA = G_C \cdot A_C \tag{7.35}$$

7.4.3 Creep effects

Creeping of the core material is considered by creep coefficients φ_t. For snow loads the creep coefficient φ_{2000} (t = 2000 h) and for permanents actions (self-weight load) $\varphi_{100.000}$ (t = 100.000 h) is used. Creeping causes an increase of the shear deflection w_v only. The bending deflection w_b is not influenced by long-term effects.

7.4.4 Deflection according to 2nd order theory with consideration of creep effects

To considerate creep effects the shear deflections are multiplied by the corresponding creep coefficients. The effects of 2nd order theory are considered by multiplying the total deflection (including creep effects) by the amplification factor.

$$w^{II}_t = \left(w^I_W + w^I_{\Delta T} + w^I_{S,b} + w^I_{S,v} \cdot (1 + \varphi_{2000}) + w^I_{G,b} + w^I_{G,v} \cdot (1 + \varphi_{100.000})\right) \cdot \alpha \tag{7.36}$$

7.5 Moments and transverse forces

7.5.1 1st order theory without consideration of creep effects

In the following the stress resultants according to 1st order theory are given. Creep effects are not considered.

$$M^{I} = M_{W}^{I} + M_{\Delta T}^{I} + M_{S}^{I} + M_{G}^{I} \tag{7.37}$$

$$V^{I} = V_{W}^{I} + V_{\Delta T}^{I} + V_{S}^{I} + V_{G}^{I} \tag{7.38}$$

with

- Moments and transverse forces due to wind load (W)

$$M_{W}^{I} = M_{W}^{I}(w) \tag{7.39}$$

$$V_{W}^{I} = V_{W}^{I}(w) \tag{7.40}$$

- Moments and transverse forces due to temperature difference (ΔT)

$$M_{\Delta T}^{I} = M_{\Delta T}^{I}(q_{wT,S}) + M_{\Delta T}^{I}(q_{wT,G}) \tag{7.41}$$

$$V_{\Delta T}^{I} = V_{\Delta T}^{I}(q_{wT,S}) + V_{\Delta T}^{I}(q_{wT,G}) \tag{7.42}$$

- Moments and transverse forces due to snow loads (S)

$$M_{S}^{I} = M_{S}^{I}(q_{e0,S}) + M_{S}^{I}(M_{S}^{N}) \tag{7.43}$$

$$V_{S}^{I} = V_{S}^{I}(q_{e0,S}) + V_{S}^{I}(M_{S}^{N}) \tag{7.44}$$

- Moments and transverse forces due to self-weight load (G)

$$M_{G}^{I} = M_{G}^{I}(q_{e0,G}) + M_{G}^{I}(M_{G}^{N}) \tag{7.45}$$

$$V_{G}^{I} = V_{G}^{I}(q_{e0,G}) + V_{G}^{I}(M_{G}^{N}) \tag{7.46}$$

The single components of the bending moments and transverse forces are calculated by the following formulae. The distribution of the moments and transverse forces is given in Fig. 7.5. In the following the values in mid-span are given.

- Wind load (W)

$$M_{W}^{I}(w) = \frac{w \cdot L^2}{8} \tag{7.47}$$

$$V_{W}^{I}(w) = \frac{w \cdot L}{2} \tag{7.48}$$

- Snow load and temperature difference

$$M_{\Delta T}^{I}(q_{wT,S}) = \frac{q_{wT,S} \cdot L^2}{8} = N_S \cdot w_T \tag{7.49}$$

$$V_{\Delta T}^{I}(q_{wT,S}) = \frac{q_{wT,S} \cdot L}{2} = \frac{N_S \cdot 4 \cdot w_T}{L} \tag{7.50}$$

- Self-weight load and temperature difference

$$M_{\Delta T}^{I}(q_{wT,G}) = \frac{q_{wT,G} \cdot L^2}{8} = N_G \cdot w_T \tag{7.51}$$

$$V_{\Delta T}^{I}(q_{wT,G}) = \frac{q_{wT,G} \cdot L}{2} = \frac{N_G \cdot 4 \cdot w_T}{L} \tag{7.52}$$

- Snow load and initial deflection

$$M_{S}^{I}(q_{e0,S}) = \frac{q_{e0,S} \cdot L^2}{8} = N_S \cdot e_0 \tag{7.53}$$

$$V_{S}^{I}(q_{e0,S}) = \frac{q_{e0,S} \cdot L}{2} = \frac{N_S \cdot 4 \cdot e_0}{L} \tag{7.54}$$

- End moment caused by snow load

$$M_{S}^{I}(M_{S}^{N}) = \frac{M_{S}^{N}}{2} = \frac{N_S \cdot e^*}{2} \quad \text{system (a)} \tag{7.55}$$

$$M_{S}^{I}(M_{S}^{N}) = N_S \cdot e^* \quad \text{system (b)} \tag{7.56}$$

$$V_{S}^{I}(M_{S}^{N}) = \frac{M_{S}^{N}}{L} = \frac{N_S \cdot e^*}{L} \quad \text{system (a)} \tag{7.57}$$

$$V_{S}^{I}(M_{S}^{N}) = 0 \quad \text{system (b)} \tag{7.58}$$

- Self-weight load and initial deflection

$$M_{G}^{I}(q_{e0,G}) = \frac{q_{e0,G} \cdot L^2}{8} = N_G \cdot e_0 \tag{7.59}$$

$$V_{G}^{I}(q_{e0,G}) = \frac{q_{e0,G} \cdot L}{2} = \frac{N_G \cdot 4 \cdot e_0}{L} \tag{7.60}$$

- End moment caused by self-weight load

$$M_{G}^{I}(M_{G}^{N}) = \frac{M_{G}^{N}}{2} = \frac{N_G \cdot e^*}{2} \quad \text{system (a)} \tag{7.61}$$

$$M_{G}^{I}(M_{G}^{N}) = N_G \cdot e^* \quad \text{system (b)} \tag{7.62}$$

$$V_{G}^{I}(M_{G}^{N}) = \frac{M_{G}^{N}}{L} = \frac{N_G \cdot e^*}{L} \quad \text{system (a)} \tag{7.63}$$

$$V_{G}^{I}(M_{G}^{N}) = 0 \quad \text{system (b)} \tag{7.64}$$

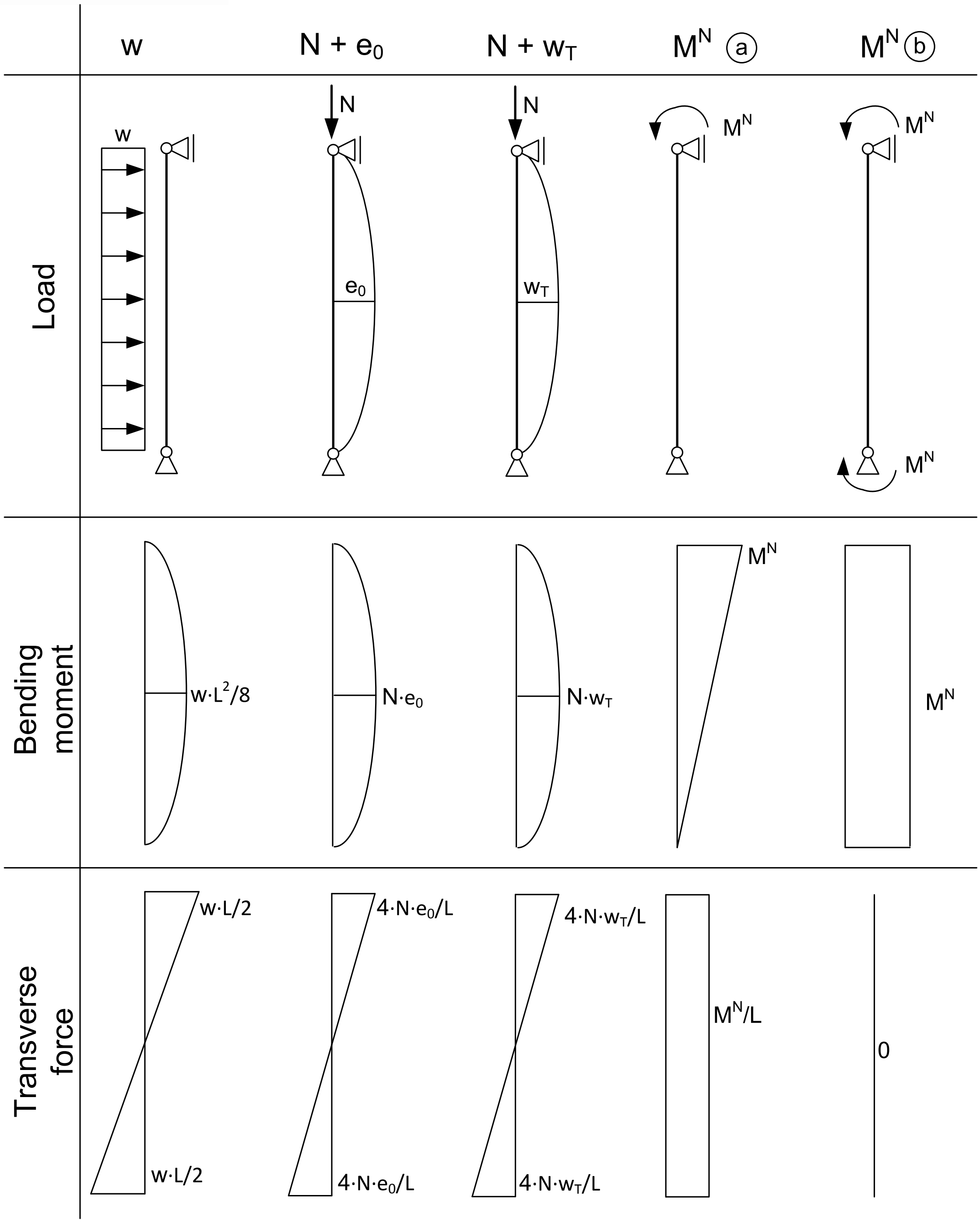

Fig. 7.5: **Distribution of moment and transverse force (1st order theory, without creeping)**

7.5.2 Effects of 2nd order theory

The effects of 2nd order theory are considered by the amplification factor α according to formulae (7.32) to (7.35).

7.5.3 Creep effects

Creeping of the core material is considered by "sandwich creep coefficients" φ_{St}. For snow loads the coefficient φ_{S2000} and for permanents actions (self-weight load) $\varphi_{S100.000}$ is used. The coefficient φ_{St} describes the increase of the deformation of the sandwich part of the cross-section and not only the increase of the shear deformation. So these coefficients can be used to determine the increase of bending moment and transverse force.

$$\varphi_{St} = \frac{k}{1+k} \cdot \varphi_t \tag{7.65}$$

$$k = \frac{w_v}{w_b} = \frac{B_S}{GA} \cdot \frac{\int V\overline{V}dx}{\int M\overline{M}dx} \tag{7.66}$$

The “sandwich factor” k corresponds to the relationship between deflection due to shear and deflection due to bending. Unlike the creep coefficient φ_t the “sandwich creep coefficient” φ_{St} is not a material parameter. It also considers the long-term loads acting on the panel. Therefore it has to be calculated for each single load case.

For the typical systems and loads of axially loaded sandwich panels the sandwich factor k is calculated by the following formulae.

System (a):

$$k = \frac{B_S}{GA \cdot L^2} \cdot \frac{48 \cdot e_0}{3 \cdot e^* + 5 \cdot e_0} \tag{7.67}$$

System (b):

$$k = \frac{B_S}{GA \cdot L^2} \cdot \frac{48 \cdot e_0}{6 \cdot e^* + 5 \cdot e_0} \tag{7.68}$$

e_0 initial deflection

e^* distance between centroidal axis and face loaded by axial force N

7.5.4 Stress resultants according to 2nd order theory with consideration of creep effects

To consider creep effects the stress resultants are multiplied by the corresponding sandwich creep coefficients φ_{St}. The effects of 2nd order theory are considered by the amplification factor α.

- Moment:

$$M_t^{II} = \left(M_W^I + M_{\Delta T}^I + M_S^I \cdot (1+\varphi_{S2000}) + M_G^I \cdot (1+\varphi_{S100.000})\right) \cdot \alpha \tag{7.69}$$

- Transverse force:

$$V_t^{II} = \left(V_W^I + V_{\Delta T}^I + V_S^I \cdot (1+\varphi_{S2000}) + V_G^I \cdot (1+\varphi_{S100.000})\right) \cdot \alpha \tag{7.70}$$

For design purposes the transverse force is divided into two parts – the first part V_{st}^{II} is caused by short-term loads (wind, temperature difference), the second part V_{lt}^{II} is caused by long-term loads (snow, self-weight).

$$V_{st}^{II} = \left(V_W^I + V_{\Delta T}^I\right) \cdot \alpha \tag{7.71}$$

$$V_{lt}^{II} = \left(V_S^I \cdot (1+\varphi_{S2000}) + V_G^I \cdot (1+\varphi_{S100.000})\right) \cdot \alpha \tag{7.72}$$

- Normal force:

$$N^{II} = N^{I} = N_S + N_G \tag{7.73}$$

Note:

In some applications temperature differences are not caused by climatic effects, e.g. in cooling chambers. In these cases bending moments, transverse forces and deflections caused by temperature difference must be considered as long-term loads.

$$w_t^{II} = \begin{pmatrix} w_W^I + w_{\Delta T,b}^I + w_{S,b}^I + \left(w_{S,v}^I + w_{\Delta T,v}^I(q_{wT,S})\right)\cdot\left(1+\varphi_{2000}\right) + \\ + w_{G,b}^I + \left(w_{G,v}^I + w_{\Delta T,v}^I(q_{wT,G})\right)\cdot\left(1+\varphi_{100.000}\right) \end{pmatrix}\cdot\alpha \tag{7.74}$$

$$M_t^{II} = \left(M_W^I + \left(M_S^I + M_{\Delta T}^I(q_{wT,S})\right)\cdot\left(1+\varphi_{S2000}\right) + \left(M_G^I + M_{\Delta T}^I(q_{wT,G})\right)\cdot\left(1+\varphi_{S100.000}\right)\right)\cdot\alpha \tag{7.75}$$

$$V_t^{II} = \left(V_W^I + \left(V_S^I + V_{\Delta T}^I(q_{wT,S})\right)\cdot\left(1+\varphi_{S2000}\right) + \left(V_G^I + V_{\Delta T}^I(q_{wT,G})\right)\cdot\left(1+\varphi_{S100.000}\right)\right)\cdot\alpha \tag{7.76}$$

$$V_{st}^{II} = V_W^I \cdot \alpha \tag{7.77}$$

$$V_{lt}^{II} = \left(\left(V_S^I + V_{\Delta T}^I(q_{wT,S})\right)\cdot\left(1+\varphi_{S2000}\right) + \left(V_G^I + V_{\Delta T}^I(q_{wT,G})\right)\cdot\left(1+\varphi_{S100.000}\right)\right)\cdot\alpha \tag{7.78}$$

7.6 Stresses in faces and core

If the moment according to 2nd order theory is known, the normal stresses in the face sheets can be calculated.

$$\sigma_{F1} = \frac{N}{A_{F1} + A_{F2}} - \frac{M_t^{II}}{D \cdot A_{F1}} \tag{7.79}$$

$$\sigma_{F2} = \frac{N}{A_{F1} + A_{F2}} + \frac{M_t^{II}}{D \cdot A_{F2}} \tag{7.80}$$

The transverse force causes shear stresses in the core. As the transverse force the shear stress is divided in a short-term part $\tau_{C,st}$ and a long-term part $\tau_{C,lt}$.

$$\tau_{C,st} = \frac{V_{st}^{II}}{A_C} \tag{7.81}$$

$$\tau_{C,lt} = \frac{V_{lt}^{II}}{A_C} \tag{7.82}$$

7.7 Design calculations

To design a sandwich panel according to EN 14509 design calculations for the serviceability limit state (deflections) and for the ultimate limit state have to be done. For a single span panel with axial and transverse load the following design calculations have to be provided.

Serviceability limit state

For the serviceability limit state the deflection is limited.

$$w \leq w_{ult} \tag{7.83}$$

According to EN 14509 for wall panels

$$w_{ult} = \frac{L}{100} \tag{7.84}$$

can be used, if there are not any other values from national standards.

Ultimate limit state

For the ultimate limit state the stresses in the face sheets and in the core are limited to the resistance values (cf. section 5.1).

- Wrinkling of a face sheet subjected to compression

$$\sigma_{F,d} \leq \sigma_{w,d} \tag{7.85}$$

- Yielding of a face sheet subjected to tension

$$\sigma_{F,d} \leq f_{y,d} \tag{7.86}$$

- Shear failure of the core

$$\frac{\tau_{C,st,d}}{f_{Cv,d}} + \frac{\tau_{C,lt,d}}{f_{Cv,t,d}} \leq 1 \tag{7.87}$$

8 Load application areas of axially loaded sandwich panels

8.1 Introduction

In section 7 the global design of axially loaded wall panels was introduced. Additionally the load application area, e.g. at the connection between wall and roof or at the lower end of the wall, has to be designed. At the load application area the axial load is introduced into the face of the panels by contact (Fig. 8.1).

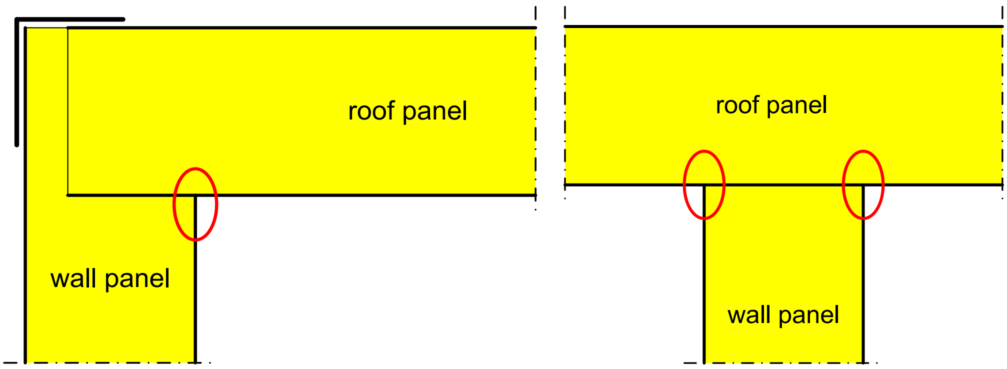

Fig. 8.1: Examples of load application areas

The failure mode of the load application area is crippling of the face at the free cut edge (Fig. 3.8). That is a stability failure mode, which is related to wrinkling of a face in mid-span. The ultimate compression stress of the free edge is termed crippling stress σ_c. Because of the relation between wrinkling stress and crippling stress the resistance value for the design of the load application area is determined based on the wrinkling stress of a face in mid-span. So to be able to design the load application area of an axially loaded panel no additional tests have to be performed.

8.2 Loads

The cut edge of the face is loaded by normal forces, which are introduced into the wall panel. At the connection between wall and roof forces are introduced from the roof into the wall. At the lower end of the wall forces are introduced from the wall into the foundation. Usually self-weight loads and snow have to be considered in calculation of the introduced normal compression stress σ_d.

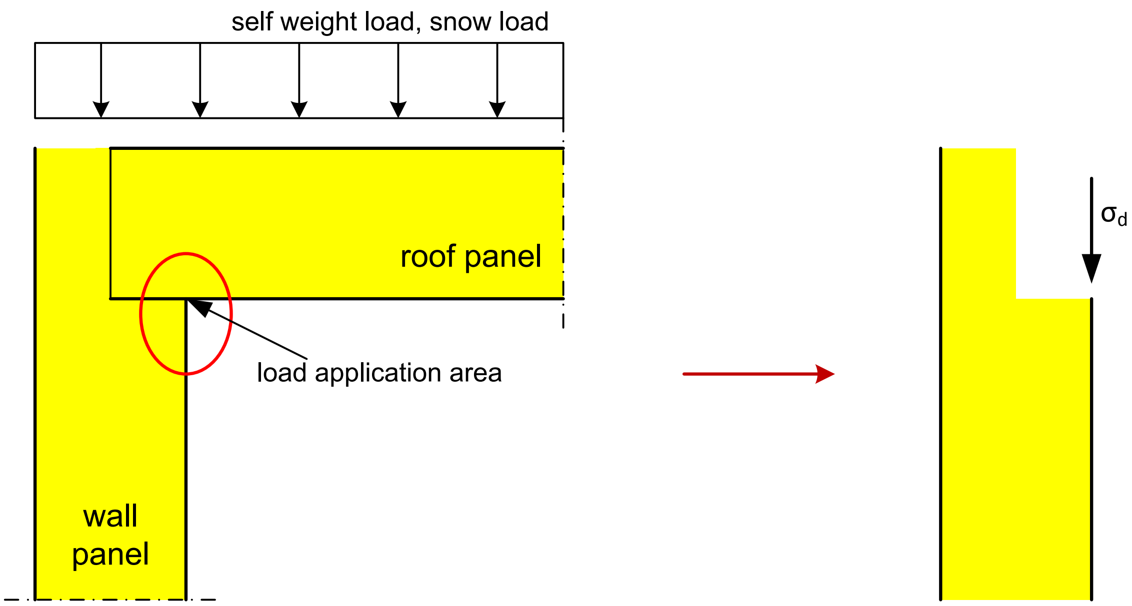

Fig. 8.2: Normal stress introduced into the wall panel

8.3 Crippling stress

The resistance value for the design of the load application area is the crippling stress σ_c. Because it is related to the wrinkling stress, the basis of the determination of the crippling stress is the wrinkling stress σ_w in mid-span.

Based on the wrinkling stress the imperfection factor α of the considered panel is calculated. The imperfection factor depends on imperfections resulting from the production process as well as on the quality of the bond between core and face. As a minimum value α = 0,21 is used.

$$\alpha = \frac{1 + \chi_w \cdot \lambda_w^2 \cdot (\chi_w - 1) - \chi_w}{\chi_w \cdot (\lambda_w - \lambda_0)} \geq 0{,}21 \tag{8.1}$$

with

reduction factor for wrinkling stress:

$$\chi_w = \frac{\sigma_w}{f_{y,F}} \tag{8.2}$$

elastic buckling load (wrinkling):

$$\sigma_{cr,w} = \frac{3}{A_F} \cdot \sqrt[3]{\frac{2}{9} \cdot EI_F \cdot G_C \cdot E_C} \tag{8.3}$$

slenderness of the face (wrinkling):

$$\lambda_w = \sqrt{\frac{f_{y,F}}{\sigma_{cr,w}}} \tag{8.4}$$

EIF bending stiffness of the face sheet

$\lambda_0 = 0{,}7$

With the imperfection factor α the crippling stress σ_c* of the face at the free edge is determined. This value only considers imperfections, which are available at the free edge as well as at mid-span. Further imperfections of the free edge, e.g. contact imperfections, are not considered.

$$\sigma_c^* = \chi_c \cdot f_{y,F} \tag{8.5}$$

slenderness of the face (crippling):

$$\lambda_c = \sqrt{2} \cdot \lambda_w \tag{8.6}$$

reduction factor for crippling stress:

$$\chi_c = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}} \leq 1 \tag{8.7}$$

$$\phi = \frac{1}{2} \cdot \left(1 + \alpha \cdot (\lambda_c - \lambda_0) + \lambda_c^2\right) \tag{8.8}$$

To consider further imperfections of the free edge, e.g. uneven cut edges, which can cause contact imperfections or small cracks between core and face, an additional reduction of the crippling stress has to be taken into account. From the stress σ_c* the characteristic value of the crippling stress is calculated with

$$\sigma_{c,k} = 0{,}54 \cdot \sigma_c^* \tag{8.9}$$

8.4 Design calculations

To design the load application area the crippling stress has to be compared to the introduced normal compression stress σ_d.

$$\sigma_d \le \sigma_{c,d} \tag{8.10}$$

9 Transfer of horizontal wind loads and stabilisation of the building

9.1 Introduction

In comparison to the bending stiffness the in-plane shear stiffness of sandwich panels is very high. Thus, if a frameless building is subjected to horizontal wind loads, walls and roof are loaded by in-plane shear forces only. Besides in the directly loaded walls, no bending moments and transverse forces arise due to a horizontal load.

Because of the very high in-plane shear stiffness the deformation of in-plane shear loaded panels can be neglected; only the flexibility of the connections has to be considered. Also the load bearing capacity of in-plane shear loaded sandwich panels is very high. So only the fastenings and not the panels have to be designed for horizontal wind loads. In doing so, the forces of the different fastenings have to be determined. The design force of a fastening is compared to the design resistance value.

$$V_d \le V_{Rd} \tag{9.1}$$

The load of the fastenings depends on their stiffness. So to design a frameless building for the transfer of horizontal wind loads knowledge of the stiffness of the fastenings is mandatory. An easy method to determine the forces of the fastenings and the displacements of the panels are numerical calculations. For some simple applications it is also possible to determine forces and displacements by analytical calculations.

9.2 Basic principles of load transfer

Horizontal wind loads are introduced into the wall panels of a building as transverse load. From the directly loaded wall one part of the horizontal load is transferred directly to the foundation; the other part is transferred to the roof. Through the roof the load is introduced into the walls and finally into the foundation.

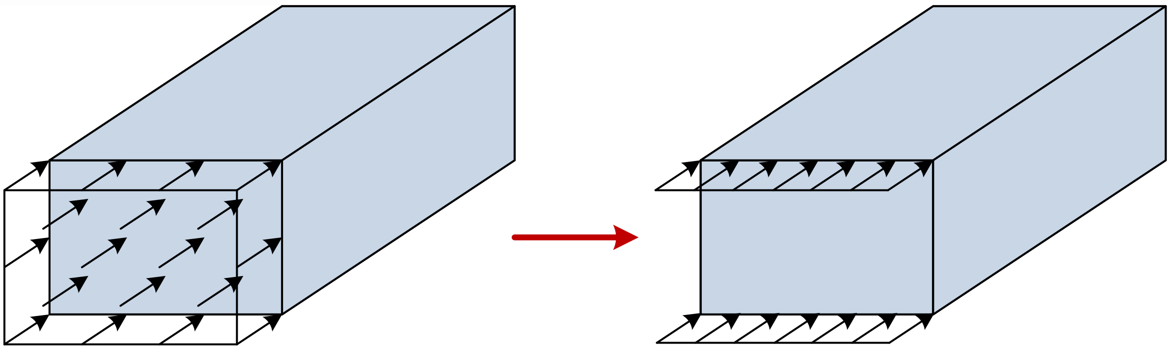

Fig. 9.1: Introduction of horizontal loads into the roof

Depending on the relation of direction of load and span of roof panels the load transfer through the roof is different.

Fig. 9.2 shows the general load transfer through a building, if the roof is loaded by horizontal wind loads acting in orthogonal direction of the span of the roof panels. The outer (directly loaded) roof panels transfer a part of the load to the wall panels, which support this roof panels. So in these wall panels in-plane shear forces occur. A second part is transferred to the adjacent roof panel via the longitudinal joint. The same applies for the following roof panels. In the longitudinal joints only forces in transverse direction are transferred. Depending on the direction of the wind load we have tension or compression loads in the joints of the roof panels.

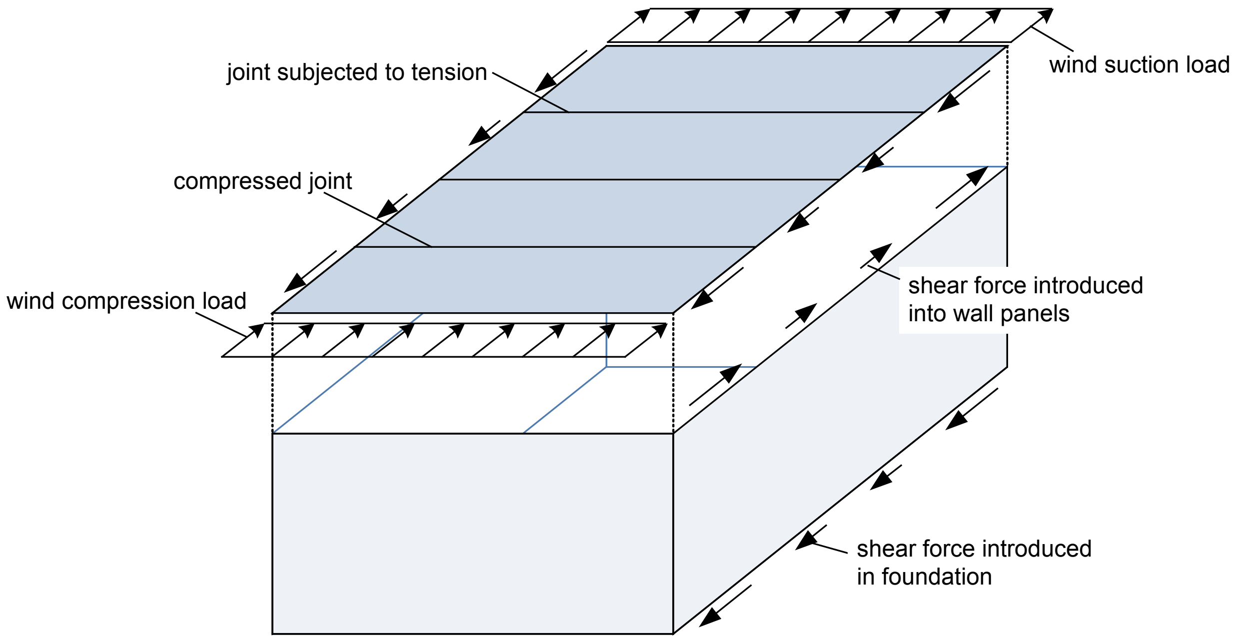

Fig. 9.2: Transfer of loads through the roof

Fig. 9.3 shows the general load-transfer through a building subjected to horizontal wind loads acting parallel to the span of the roof panels. The wind load is transferred through the roof to the walls, which are parallel to the direction of the load. In addition forces are introduced into the walls, which are orthogonal to the direction of load. A circumferential shear force occurs, which is comparable to the shear forces of shear diaphragms in conventional buildings with a substructure. In addition to a displacement in longitudinal direction a rotation of the roof panels occurs. So in the longitudinal joints of the roof panels forces in longitudinal as well as in transverse direction have to be transferred. Through the walls the forces are introduced into the foundation.

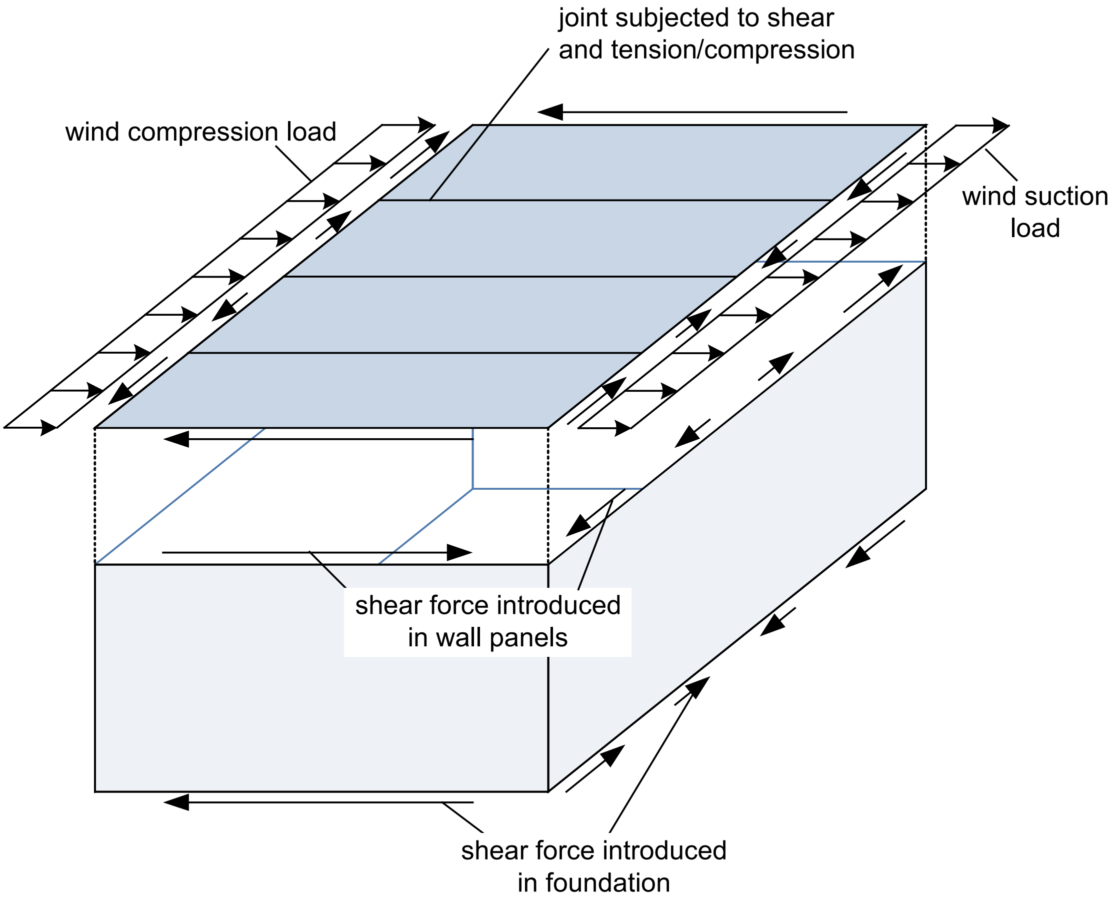

Fig. 9.3: Transfer of loads through the roof

In-plane shear forces resulting from horizontal loads are introduced from the roof into the upper end of the walls. So also in the wall panels in-plane shear forces occur, which are transferred to the foundation. If the longitudinal joints of wall panels are not connected, the panels are not influenced by adjacent panels; each panel acts as a single element. In this case the wall panel is loaded as a lever arm. A horizontal force and a moment have to be transferred by the connections between panel and foundation. In the connections forces in longitudinal and in transverse direction occur. The longitudinal forces are caused by the moment.

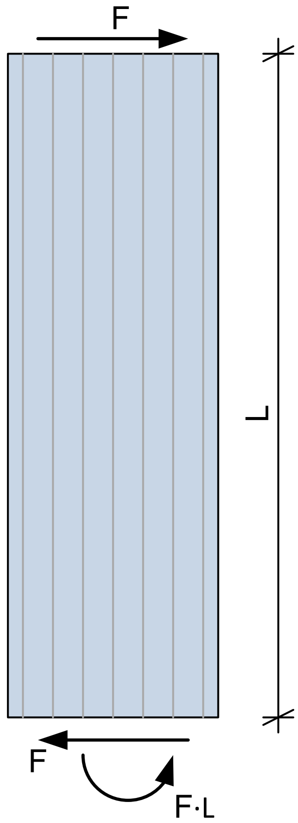

Fig. 9.4: Transfer of in-plane shear loads through a wall panel

If the longitudinal joints of wall panels are connected, the stiffness and also the load-bearing capacity increase. The panels do not act as independent single elements, they influence each other. Thus the load is not distributed uniformly to all connections. The connections between wall and foundation as well as the connections of the joints are loaded by transverse and by longitudinal forces.

9.3 Stiffness of connections

To determine the forces of the fastenings the stiffness of the different connections has to be known. The stiffness of a fastening depends on the kind of fastener as well as on the panel, e.g. on the thickness of the face sheets. Especially the geometry of the joints has a wide influence on the stiffness (and also on the load-bearing capacity) of the fastenings. So the stiffness of the fastenings has to be determined for each single case. This should be done by small scale tests, e.g. according to [8] and [9]. In addition to the stiffness also the load-bearing capacity of the fastening can be determined by these tests. For some kinds of fasteners there are also calculation procedures available to determine stiffness and load-bearing capacity, e.g. [1].

With the stiffness of the single fastenings the stiffness of the connection is determined. In doing so it has to be noted that e.g. at the connection between wall and roof the load is transferred from the roof panel to an angle and subsequently from the angle to the wall panel, i.e.

one connection consists of two fastenings, which are arranged in series. So the stiffness determined for one fastening has to be divided by two to get the stiffness of a connection consisting of two fastenings and an angle. The angles are assumed to be rigid. If there are several connections between the panels - e.g. internal as well as external face sheet are connected - the stiffness (and the load-bearing capacity) of these connections has to be added to get the resulting stiffness (and load-bearing capacity). In the following figure the determination of the stiffness of a connection is shown for some exemplary cases.

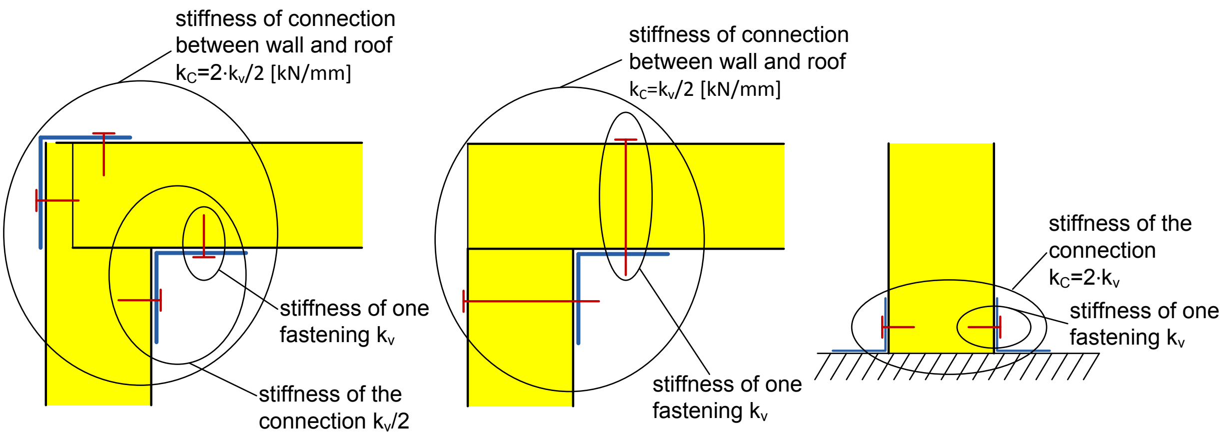

Fig. 9.5: Determination of stiffness of connections

9.4 Determination of forces by numerical calculation

Because the panels are assumed to be stiff and only the fastenings are flexible, a numerical determination of the forces, the fastenings have to be designed for, is relatively easy. In addition to the forces of the fastenings also the displacements of the panels can be determined by a numerical calculation. In the FE-model the panels are modelled as rigid bodies. E.g. shell elements with a comparatively high thickness and a high elastic modulus can be used. The connections are represented by longitudinal springs. The stiffness of the springs corresponds to the stiffness of the connections. If one connection consists of several fastenings, the stiffness is determined as shown above.

Through the connections of the longitudinal joints and through the connections between wall and foundation forces in longitudinal as well as in transverse direction of the panel are transferred. To represent these connections in the FE-model two springs should be used – one spring acts in longitudinal the other one in transverse direction (Fig. 9.6). The resulting shear force of a fastening is calculated by vectorial addition of the longitudinal force V_y and the transverse force V_x.

$$V = \sqrt{(V_x)^2 + (V_y)^2} \tag{9.2}$$

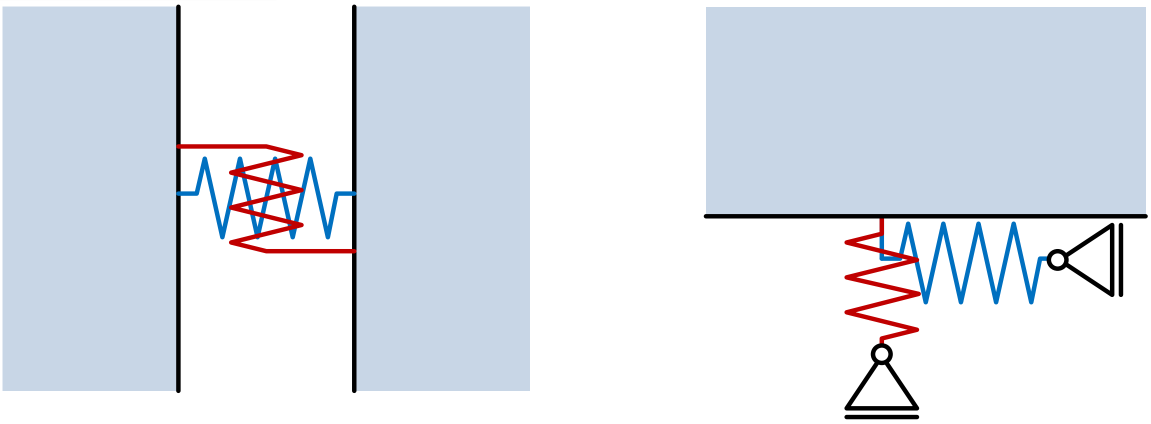

Fig. 9.6: **Connection of joint and connection to foundation in the FE- model**

Exemplarily the FE-models of a roof and a wall are presented in the following figures. In the figures the panels, the supports and the load are shown. The springs between adjacent panels and between panel and supports are not displayed.

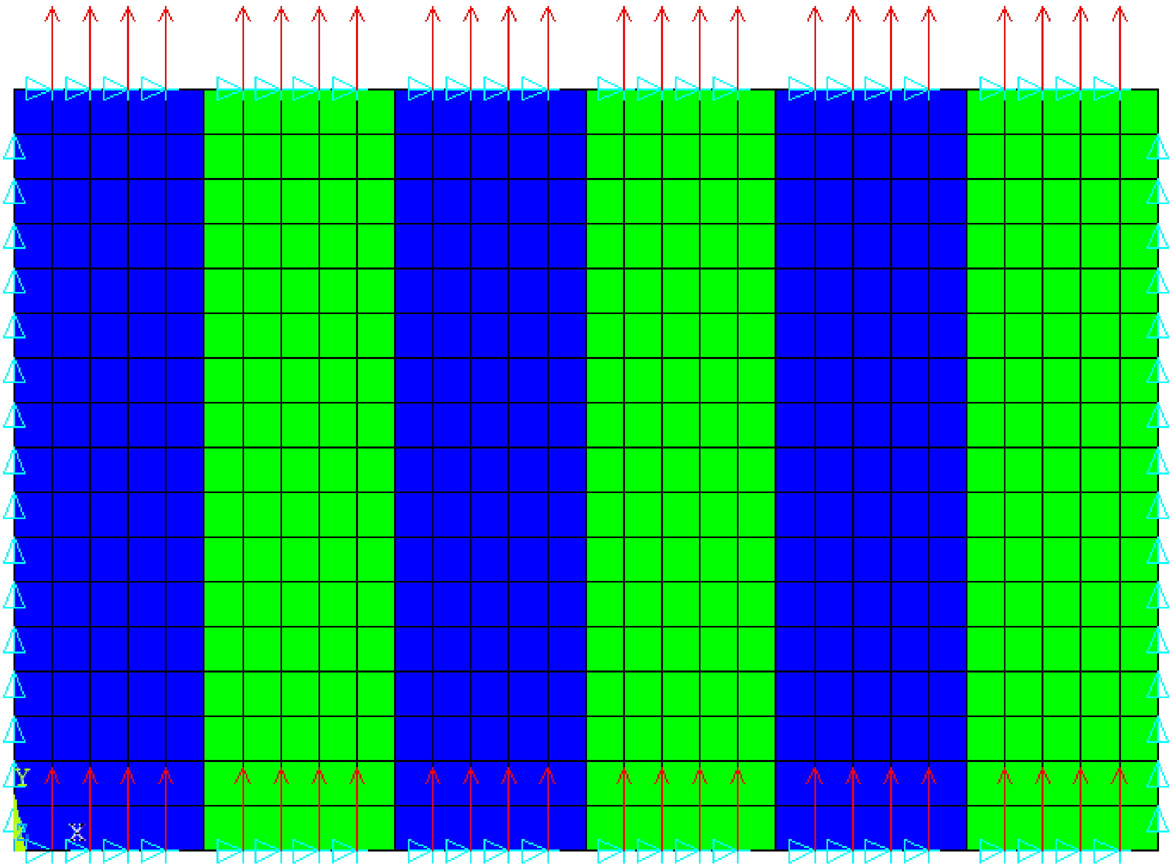

Fig. 9.7: **FE-model of a roof**

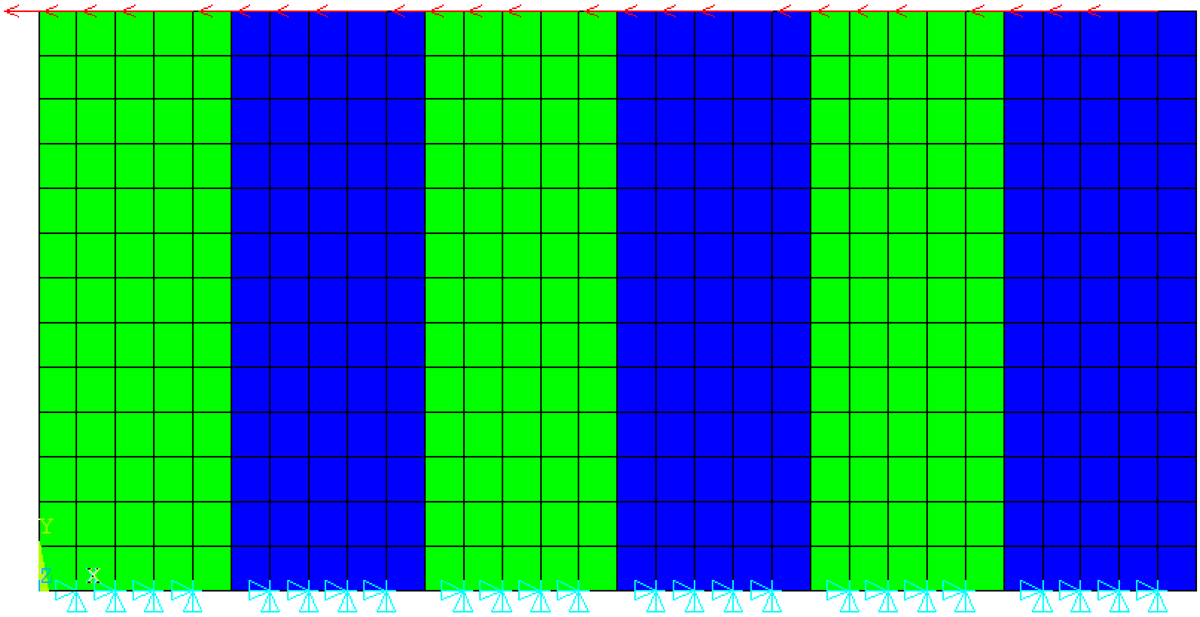

Fig. 9.8: FE-model of a wall

9.5 Analytical determination of forces

9.5.1 Basics

For simple applications the distribution of forces to the different fastenings and the displacement of the panels can also be determined analytically.

To simplify the calculation procedures the forces of the connections are smeared over the width B or over the length L of the panel. The stiffness of the longitudinal springs is transformed into a stiffness per unit length [N/mm^2]. For the connection between wall and roof the stiffness is

$$k_{Wt} = k_C \cdot \frac{n}{B} \quad \text{at the transverse edge of the panel} \quad (9.3)$$

$$k_{Wl} = k_C \cdot \frac{n}{L} \quad \text{at the longitudinal edge of the panel} \quad (9.4)$$

with

k_C stiffness of one connection

B/n distance between connections at a transverse edge

L/n distance between connections at a longitudinal edge

For the connections of longitudinal joints the stiffness is

$$k_J = k_C \cdot \frac{n}{L} \quad (9.5)$$

With

k_C stiffness of one connection

L/n distance between connections at a longitudinal joint

For the connections between wall and foundation the stiffness is

$$k_W = k_C \cdot \frac{n}{B} \tag{9.6}$$

with

k_C stiffness of one connection

B/n distance between connections at a longitudinal joint

9.5.2 Roofs with load in transverse direction of the panels

If the horizontal wind load acts in orthogonal direction to the span of the panel, only forces in transverse direction occur. For analytical determination of forces and displacements only the connections in transverse direction are considered. Fig. 9.9 shows the resulting model. The wind load is introduced in both outer panels of the roof. So we have a line load w_1 introduced in panel 1, and a line load w_n introduced in panel n. At the transverse edges of the panels there are longitudinal springs with stiffness k_{Wt}. Between adjacent panels there are longitudinal springs with stiffness k_J.

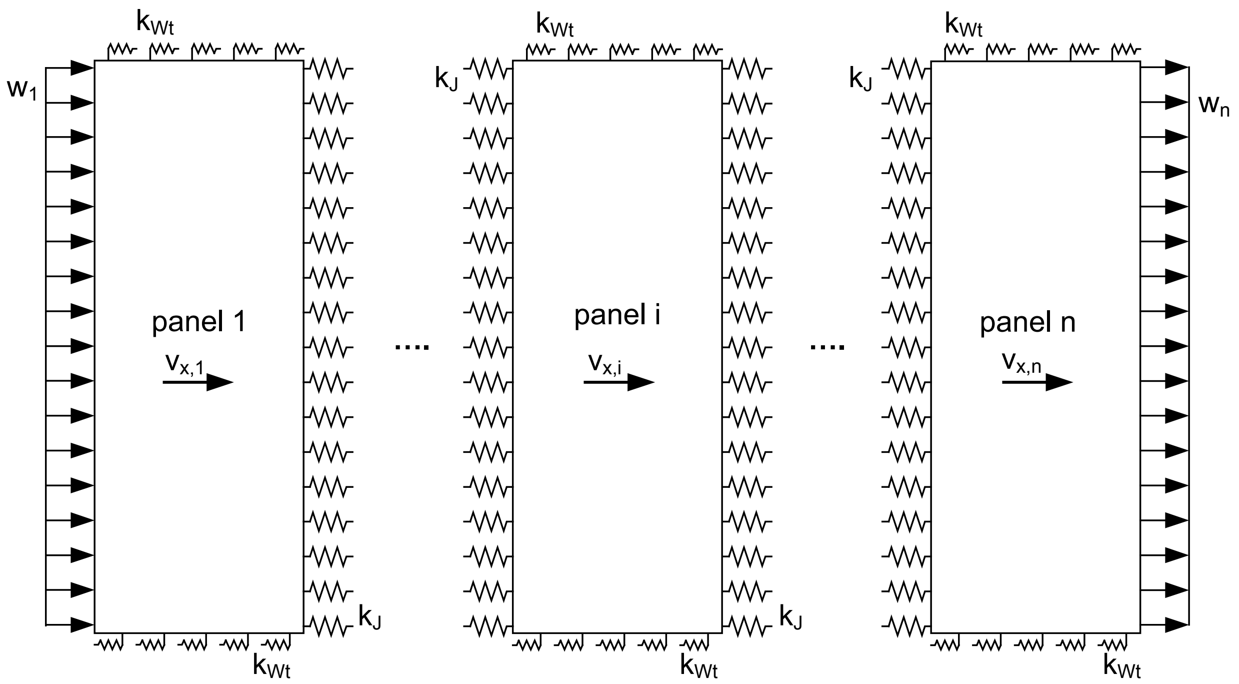

Fig. 9.9: **Model of a roof for analytical calculation**

At each panel i a displacement $v_{x,i}$ in transverse direction occurs. By multiplication of the displacement and the stiffness of the connections the forces acting on the panels are calculated (Fig. 9.10).

Connection between wall and roof:

$$F_{W,i} = v_{x,i} \cdot k_{Wt} \cdot B \tag{9.7}$$

Longitudinal joint between panels i and i-1:

$$F_{J,i,l} = \left(v_{x,i-1} - v_{x,i}\right) \cdot k_J \cdot L \quad (9.8)$$

Longitudinal joint between panels i and i+1:

$$F_{J,i,r} = \left(v_{x,i+1} - v_{x,i}\right) \cdot k_J \cdot L \quad (9.9)$$

At the outer panels the external force has to be considered, i.e. at panel 1

$$F_1 = w_1 \cdot L \quad (9.10)$$

and at panel n

$$F_n = w_n \cdot L \quad (9.11)$$

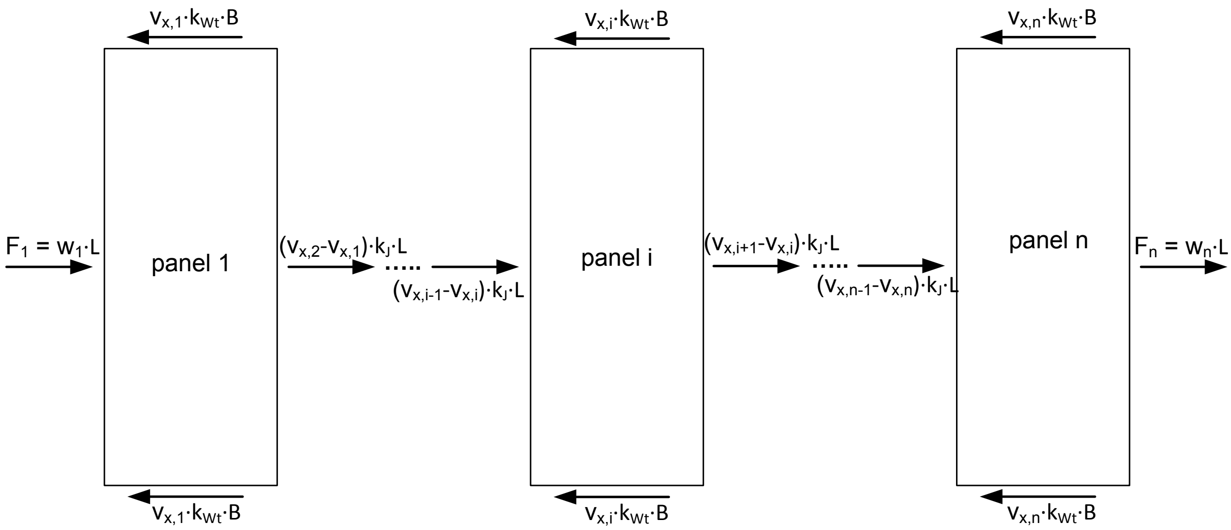

Fig. 9.10: Forces acting on the panels of a roof

Equilibrium of forces for panel i results in the following equation.

$$\left(v_{x,i-1} - v_{x,i}\right) \cdot k_J^l \cdot L + \left(v_{x,i+1} - v_{x,i}\right) \cdot k_J^r \cdot L - 2 \cdot v_{x,i} \cdot k_{Wt} \cdot L + F_i = 0 \quad (9.12)$$

$$A_i \cdot v_{x,i-1} + B_i \cdot v_{x,i} + C_i \cdot v_{x,.i+1} + F_i = 0 \quad (9.13)$$

with

$$A_i = k_J^l \cdot L \quad (9.14)$$

$$B_i = -\left(k_J^l \cdot L + k_J^r \cdot L + 2 \cdot k_{Wt} \cdot B\right) \quad (9.15)$$

$$C_i = k_J^r \cdot L \quad (9.16)$$

F_i external load according to (9.10) and (9.11)

This equation is set up for each panel. So for n panels we get a linear equation system consisting of n equations. Solving the equation system results in the displacement $v_{x,i}$ of each panel.

If the displacements are known the forces acting on a connection can be calculated by the following formulae.

Connection between wall and roof:

$$V = v_{x,i} \cdot k_{Wt} \cdot \frac{B}{n} \tag{9.17}$$

Connection of longitudinal joints:

$$V = \Delta v_{x,i} \cdot k_J \cdot \frac{L}{n} \tag{9.18}$$

$$\Delta v_{x,i} = \begin{cases} v_{x,i-1} - v_{x,i} \\ v_{x,i+1} - v_{x,i} \end{cases} \tag{9.19}$$

9.5.3 Roof panels with load in longitudinal direction

The wind loads acting on both ends of the roof (in general wind suction and wind compression load) are added to a resulting load w.

$$w = w_S + w_C \tag{9.20}$$

w_S wind suction load

w_C wind compression load

Fig. 9.11 shows the longitudinal forces, which act on the panels of a roof. These forces are determined by equilibrium of forces. The forces, which are introduced in the walls being parallel to the direction of load, are.

$$V_1^l = -V_n^r = \frac{n_{SW}}{2} \cdot w \cdot B \tag{9.21}$$

with

n_{SW} number of sandwich panels

B width of a panel

The longitudinal forces, which are transferred by the connections of a joint, are determined by equalisation of forces for each panel.

$$V_i^r = V_i^l - w \cdot B \tag{9.22}$$

$$V_i^l = V_{i-1}^r \tag{9.23}$$

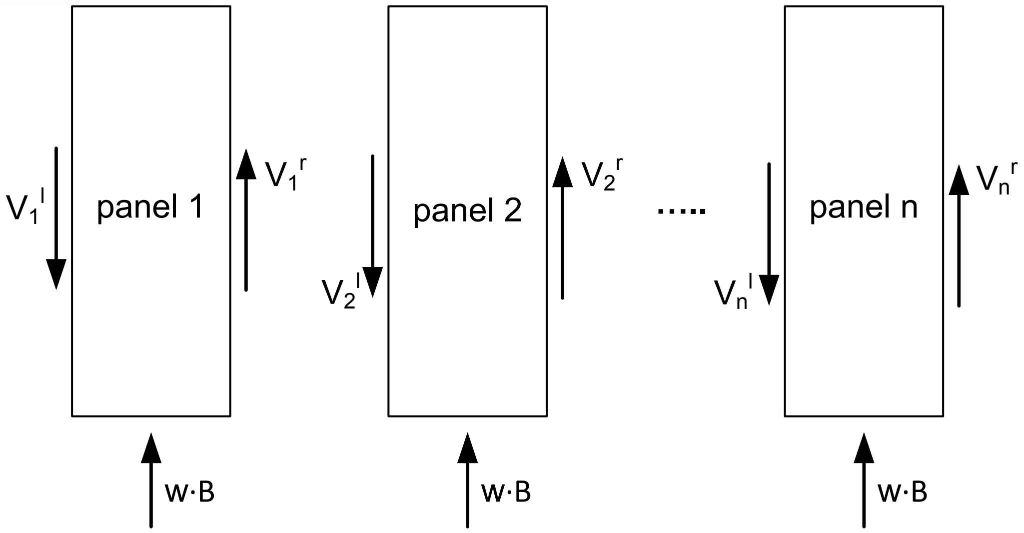

Fig. 9.11: Forces acting in longitudinal direction of the panels

In Fig. 9.12 the displacements und the transverse forces, which result from the rotation φ_i of a panel, are shown. The forces are determined by multiplying the displacement by the stiffness of the connection. For the connections at the longitudinal joints the difference of the displacements of adjacent panels has to be used.

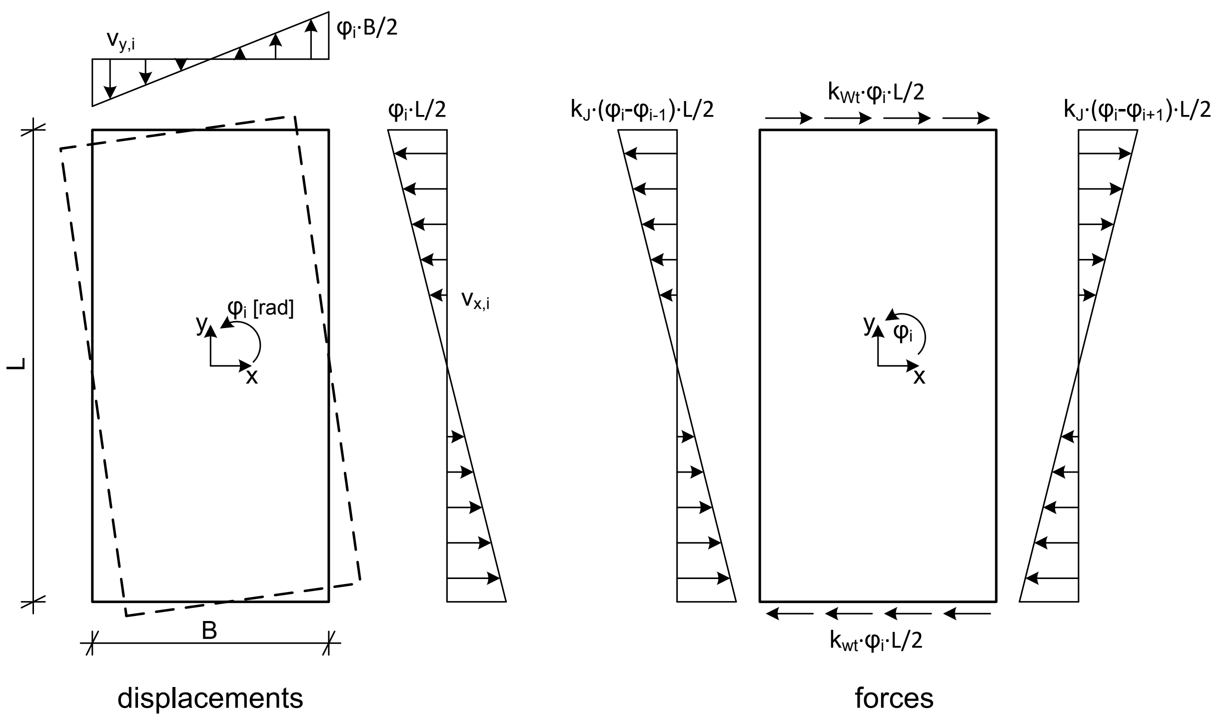

Fig. 9.12: Displacements and transverse forces caused by rotation φ_i

In Fig. 9.13 the forces resulting from the smeared forces given in Fig. 9.12 are shown. In addition the external wind load and the longitudinal forces, which can be determined from equilibrium of forces, are shown.

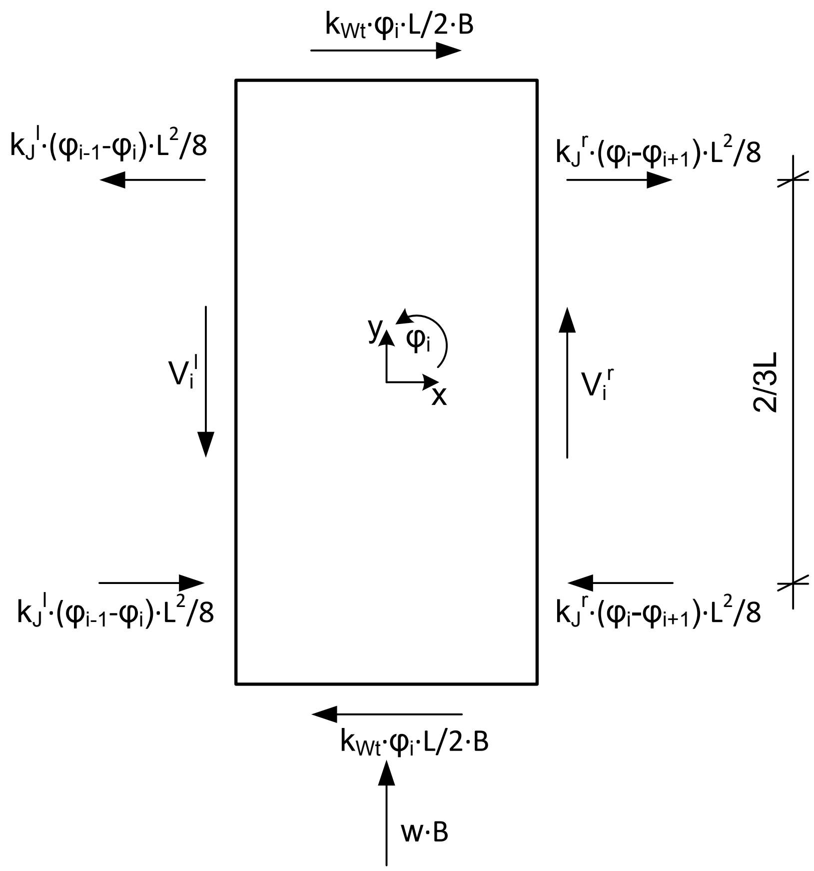

Fig. 9.13: Resulting forces of panel i

With this forces the equilibrium of moments (with reference to the centre of the panel) for panel i is

$$-k_J^l \cdot (\varphi_i - \varphi_{i-1}) \cdot \frac{L^2}{8} \cdot \frac{2}{3} L - k_J^r \cdot (\varphi_i - \varphi_{i+1}) \cdot \frac{L^2}{8} \cdot \frac{2}{3} L - k_{Wt} \cdot \varphi_i \cdot B \cdot \frac{L}{2} \cdot L + (V_i^l + V_i^r) \cdot \frac{B}{2} = 0 \quad (9.24)$$

$$A_i \cdot \varphi_{i-1} + B_i \cdot \varphi_i + C_i \cdot \varphi_{i+1} = D_i \quad (9.25)$$

with

$$A_i = -\frac{k_J^l \cdot L^3}{12} \quad (9.26)$$

$$B_i = \frac{k_J^l \cdot L^3}{12} + \frac{k_{Wt} \cdot B \cdot L^2}{2} + \frac{k_J^r \cdot L^3}{12} \quad (9.27)$$

$$C_i = -\frac{k_J^r \cdot L^3}{12} \quad (9.28)$$

$$D_i = (V_i^l + V_i^r) \cdot \frac{B}{2} \quad (9.29)$$

So we get a linear equation system consisting of n equations. The solution of the equation system is the n unknown rotations φ_i.

If the rotations are known the forces of the connections can be determined.

Connections at transverse edges:

$$V = k_{Wt} \cdot \varphi_i \cdot \frac{L}{2} \cdot \frac{B}{n} \tag{9.30}$$

B/n distance between connections at the transverse edge of a panel

Connections at longitudinal edges (connection between roof and wall):

$$V = \frac{V_1^l}{n} \tag{9.31}$$

or

$$V = \frac{V_n^r}{n} \tag{9.32}$$

n number of connections at the longitudinal edge of a panel

Connections at longitudinal joints:

Transverse direction (force of highest loaded connection at y = ±L/2):

$$V_x = k_J \cdot (\varphi_i - \varphi_{i-1}) \cdot \frac{L}{2} \cdot \frac{L}{n} \tag{9.33}$$

or

$$V_x = k_J \cdot (\varphi_i - \varphi_{i+1}) \cdot \frac{L}{2} \cdot \frac{L}{n} \tag{9.34}$$

L/n distance between connections of a joint

Longitudinal direction:

$$V_y = \frac{V_i^l}{n} \tag{9.35}$$

or

$$V_y = \frac{V_i^r}{n} \tag{9.36}$$

n number of connections of a joint

Resulting force:

$$V = \sqrt{(V_x)^2 + (V_y)^2} \tag{9.37}$$

Also the global displacement of the panels can be calculated analytically. In doing so, the longitudinal displacement of each panel relative to the adjacent panel has to be determined. Subsequently, the global displacement of a panel is determined by summation of the single relative displacements over the roof, starting at one of the outer panels. The highest global displacement occurs at the inner panel of a roof. In the following it is assumed that the summation starts at panel 1, which is located at the left edge of the roof.

The displacement in longitudinal direction of a panel consists of two parts. The first part results directly from the longitudinal forces in the joints or for the outer panels from the force of the connection between roof and wall (formulae (9.21) to (9.23)). The second part results from the rotation of the panel, which causes an additional displacement in longitudinal direction.

The displacement resulting from the longitudinal forces is calculated by the following formulae. It refers to the adjacent panel or for the outer panels to the wall.

Displacement $v_{y,1,v}$ of the outer panel of a roof:

$$v_{y,1,v} = \frac{V_1^l}{L \cdot k_{Wl}} \qquad (9.38)$$

Displacement $v_{y,i/i-1,v}$ of a panel i relative to panel i-1:

$$v_{y,i/i-1,v} = \frac{V_i^l}{L \cdot k_J} \qquad (9.39)$$

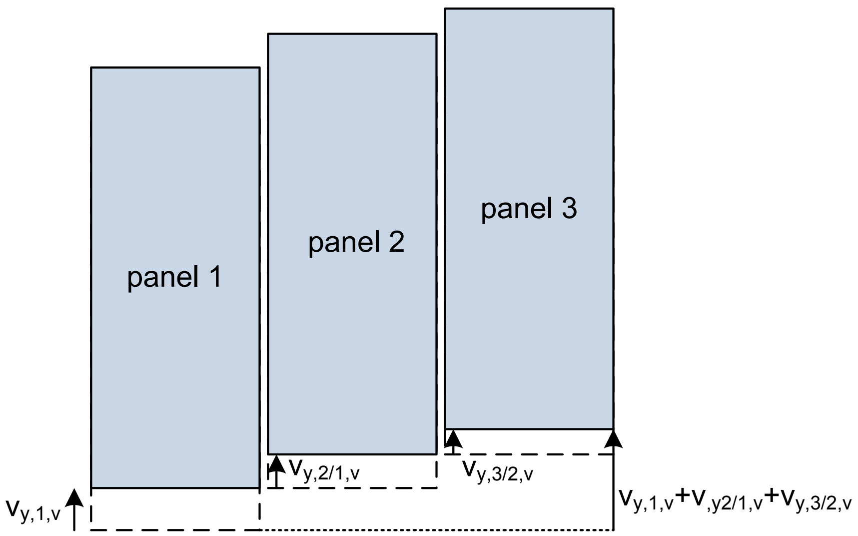

Fig. 9.14: Displacements of panels resulting from forces acting in longitudinal direction

The displacement caused by the rotation of the panel is

$$v_{y,i,\varphi} = B \cdot \varphi_i \qquad (9.40)$$

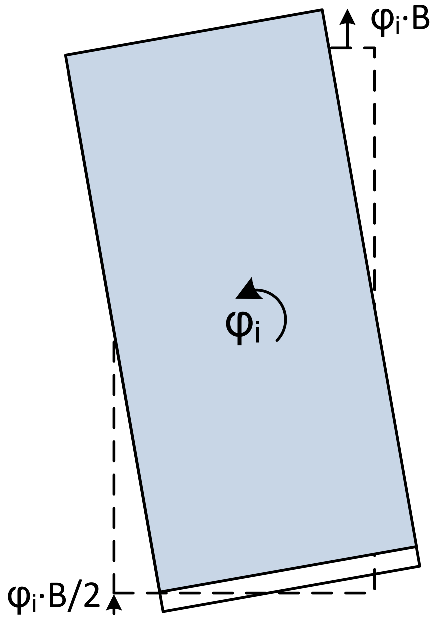

Fig. 9.15: Displacements of panels resulting from rotation

The global displacement of a panel j is determined by summation of the displacements $v_{y,i,v}$ and $v_{y,i,\varphi}$ over the panels, starting at panel 1.

$$v_{y,j} = \frac{V_1^l}{L \cdot k_{Wl}} + \sum_{i=2}^{j} \frac{V_i^l}{L \cdot k_J} + \sum_{i=1}^{j} B \cdot \varphi_i \tag{9.41}$$

9.5.4 Wall panels without connections of the joints

To transfer horizontal wind loads for wall panels a connection of the longitudinal joints is not necessarily required. In this case the panels are not influenced by adjacent panels; each panel acts as a single element. An analytical determination of forces and displacements is possible. If the longitudinal joints of wall panels are connected, also for simple configurations an analytical determination of forces and displacements is not reasonable. For all panels displacement and rotation as well as the centre of rotation are unknown. To determine them no set of linear equations is available. Therefore design should be done by numerical calculations.

If the joints are not connected the panel is loaded as a lever arm. A horizontal force and a moment have to be transferred by the connections between panel and foundation. So these connections are loaded by transverse forces and by a couple of longitudinal forces, which counteracts the moment resulting from the introduced force and the lever arm. Fig. 9.16 shows the forces for a panel with two connections to the foundation.

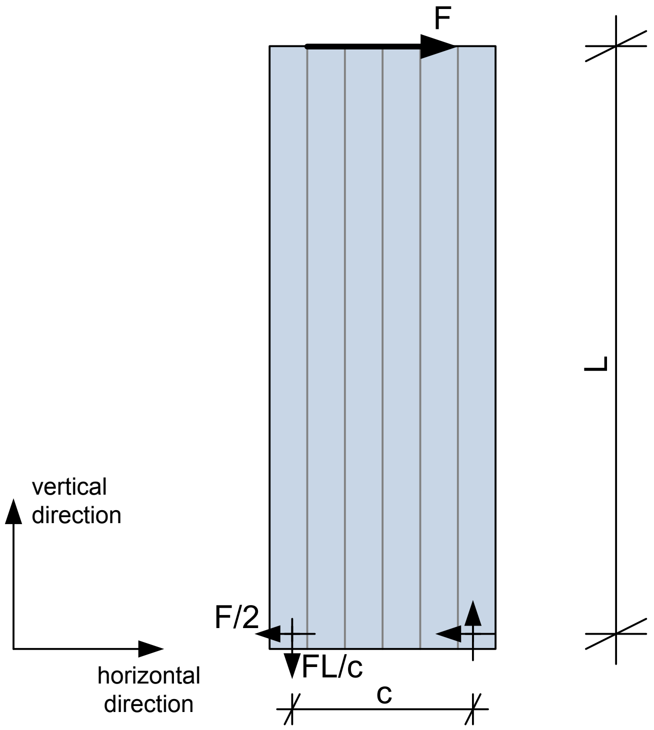

Fig. 9.16: **Forces at an in-plane shear loaded wall panel**

If there are more than two connections, the highest longitudinal force occurs at the outer connection. This force is determined by the following equation.

$$V_y = \frac{F \cdot L}{c + \frac{c_1^2}{c} + \frac{c_2^2}{c} + \ldots\ldots} \tag{9.42}$$

with

c_i distance between pair of connections

c distance between outer connections

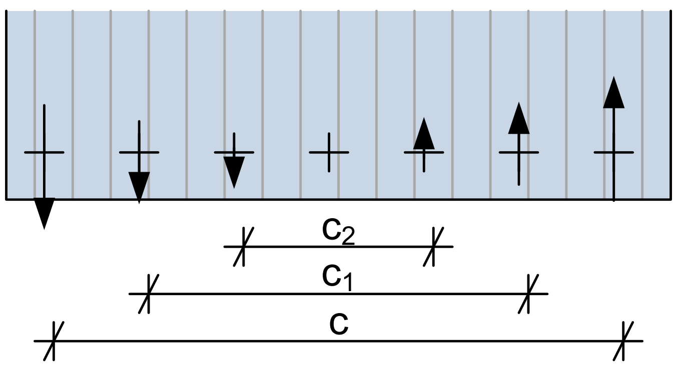

Fig. 9.17: **Longitudinal forces resulting from a moment**

The horizontal force is distributed constantly to the connections of the edge. So for one connection we get the following horizontal force

$$V_x = \frac{F}{n} \tag{9.43}$$

n number of connections at the edge

The resulting force of a connection is determined by vectorial addition

$$V = \sqrt{(V_x)^2 + (V_y)^2} \tag{9.44}$$

The horizontal force causes a displacement v_x of the panel. The moment leads to a rotation φ around the centre of the lower transverse edge. To determine the displacements smeared connections with a continuous stiffness k_W (formula (8.6)) are used.

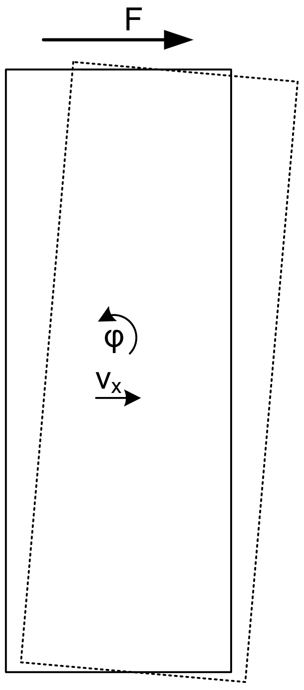

Fig. 9.18: Displacement and rotation of a wall panel

The rotation of the panel is

$$\varphi = \frac{12 \cdot F \cdot L}{B^3 \cdot k_W} \tag{9.45}$$

The displacement in transverse direction of the panel is

$$v_x = \frac{F}{B \cdot k_W} + \varphi \cdot L \tag{9.46}$$

This displacement occurs at the upper and of the panel.

10 Summary

Sandwich panels are traditionally used as covering and isolating components. A new application is to use sandwich panels in frameless structures, i.e. the panels are applied without any load transferring substructure. In this new type of application in addition to space enclosure, the sandwich panels have to transfer loads and to stabilise the building. The wall panels have to transfer normal forces arising from the superimposed load from overlying roof or ceiling panels. Furthermore horizontal wind loads have to be transferred to the foundation and the building has to be stabilised.
In the guideline at hand calculation procedures and design methods for sandwich panels of frameless structures are introduced. Calculation examples can be found in Deliverable D3.5 [5].

11 References

[1] D3.3 – part 3: Connections of sandwich panels, Deliverable of EASIE project, May 2011.

[2] D3.3 – part 4: Axially loaded sandwich panels, Deliverable of EASIE project, June 2011.

[3] D3.3 – part 5: Introduction of loads into axially loaded sandwich panels, Deliverable of EASIE project, October 2011.

[4] D3.3 – part 6: Stabilisation of frameless structures, Deliverable of EASIE project, July 2011.

[5] D3.5: Calculation examples – Design of frameless structures made of sandwich panels, Deliverable of EASIE project, October 2011.

[6] EN 14509:2006: Self-supporting double skin metal faced insulating panels –Factory made products –Specifications.

[7] European recommendations for sandwich panels. ECCS/CIB-Report – Publication 257, ECCS TWG 7.9 and CIB W056, 2000.

[8] The Testing of Connections with mechanical Fasteners in Steel Sheeting and Sections, ECCS publication No. 124, 2009.

[9] Preliminary European Recommendations for the Testing and Design of Fastenings for Sandwich Panels. ECCS publication No. 127, 2009.

[10] EN 1990:2002: Eurocode - Basis of structural design.

[11] EN 1991-1-3:2003: Eurocode 1 – Actions on structures – Part 1-3: General actions, Snow loads.

[12] EN 1991-1-4:2005: Eurocode 1 – Actions on structures – Part 1-4: General actions, Wind actions.

[13] EN 1993-1-3:2006: Eurocode 3: Design of steel structures - Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting

[14] Lange, J., Berner, K.: Sandwichelemente im Hochbau, Stahlbau-Kalender 2010, Ernst & Sohn, Berlin, 2010.