Ensuring Advancement in Sandwich Construction Through Innovation and Exploitation

REPORT

No.: D3.3 – part 6

Stabilisation of frameless structures

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Task: 3.3

Object: Basics of transferring horizontal loads in frameless structures made of sandwich panels

This report includes 54 pages.

Date of issue: 19.07.2011

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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	
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Drafting History	
Draft Version 1.1	27.06.2011
Draft Version 1.2	11.07.2011
Draft Version 1.3	
Draft Version 1.4	
Final	19.07.2011

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

D3.3 – part 6: Stabilisation of frameless structures	Due date: Month 35 Completed: Month 34

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1 Introduction

The common application of sandwich panels is enclosure of buildings. The panels are mounted on a substructure and they transfer transverse loads, e.g. snow and wind, to the substructure. The panels are subjected to bending moments and transverse forces only. A new application is to use sandwich panels with flat or lightly profiled faces for frameless buildings. In smaller buildings – such as cooling chambers, climatic chambers and clean rooms – the panels are applied 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 transfer normal forces arising from the superimposed load from overlying roof or ceiling panels. Deliverable D3.3 – part 4 [3] deals with the design of axially loaded sandwich panels.

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 have also to transfer the horizontal load. For this purpose the high in-plane shear stiffness and capacity of sandwich panels is used.

In the report at hand basics of the transfer of horizontal loads in frameless buildings made of sandwich panels are presented. It is assumed that the buildings consist of vertically spanning wall panels, which are connected to the foundation. To connect panel and foundation often angles are fixed to the foundation. The panels are fixed to the angles, e.g. by self-drilling screws. The roof panels overlay on the wall panels. To connect wall and roof panels also angles and mechanical fasteners (e.g. self-drilling screws, blind rivets) are used. To connect the longitudinal joints of the panels self-drilling screws or blind rivets can be used. 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 the roof panels is mandatory. Transfer of horizontal loads via the roof is not possible, if the joints are not connected. The design procedures presented in the report at hand cover predominantly static loadings, e.g. wind loads; fatigue loading is not covered. Also the effects of openings are not taken into account.

The design procedures presented in D3.3 – part 4 [3] are based on different investigations and design methods for shear loaded sandwich panels, e.g. sandwich panels used for shear diaphragms in conventional constructions.

For shear loaded walls made of sandwich panels ETAG 21 [10] gives a test procedure for design by testing. The test procedure and the basics of the design are also briefly introduced in the report at hand.

2 Load bearing behaviour of in-plane shear loaded sandwich panels

In EN 14509 [4] and also in other standards no design methods for sandwich panels loaded by in-plane shear forces are given. But several investigations on this topic are available, e.g. [13], [14], [15], [16]. These investigations mainly deal with sandwich panels mounted on a substructure, e.g. shear diaphragms made of sandwich panels.

In all of these investigations it has been shown, that sandwich panels have a very high stiffness and a very high 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. Because of that for design purposes 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 frameless structure is loaded by horizontal wind loads, the fastenings have to be designed for this load. Particular attention has to be paid to the connections between wall and roof panels. At this connections shear forces are introduced into rectangular adjacent panels. In conventional buildings this shear forces are introduced in the substructure as normal force. Also the connection between wall and foundation and the connections of longitudinal joints have to be considered.

In comparison to the in-plane shear stiffness the bending stiffness of sandwich panels is very small. Thus for design purposes it can be assumed that transferring horizontal loads leads only to forces acting in the plane of the panels. Besides in the directly loaded walls, no transverse forces and bending moments arise due to horizontal wind loads.

3 Transfer of horizontal wind loads in frameless structures

Generally there are two possibilities for wind loads to act on a frameless building. The wind direction can be parallel or orthogonal to the span of the roof panels. Depending on that the mechanism of load transfer through the roof of the building is different.

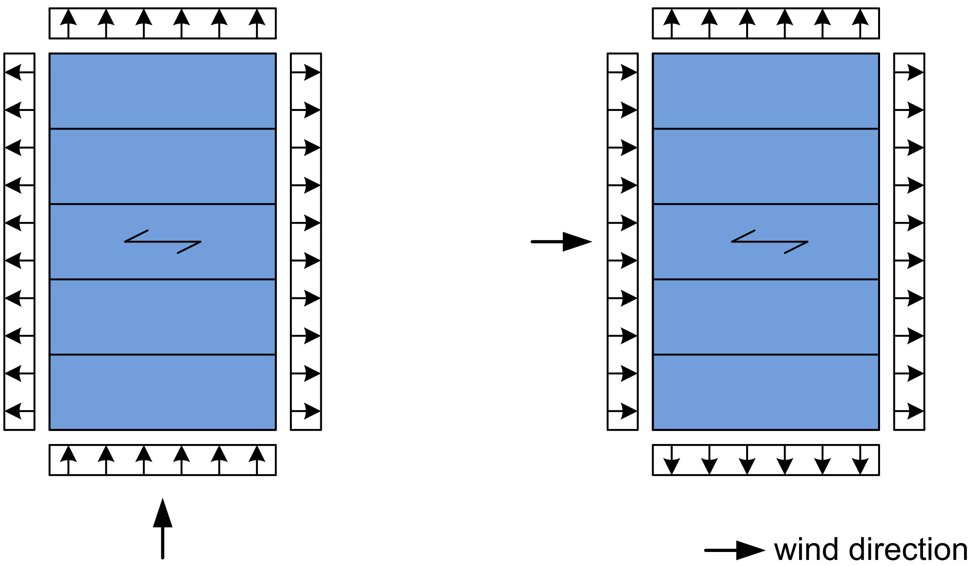

Fig. 3.1: Wind loads

The horizontal wind load acts directly on the wall panels of a building. The wall panels are usually single span elements with one support at the foundation and the other support at the roof. So half of the horizontal wind load is introduced into the roof. The second part is directly transferred to the foundation. Via the roof the load is introduced into the walls and finally into the foundation (Fig. 3.2). Depending on the relation of direction of load and span of roof panels, a circumferential shear force occurs at the connections between wall and roof. So also in the walls being orthogonal to the direction of load shear forces may occur.

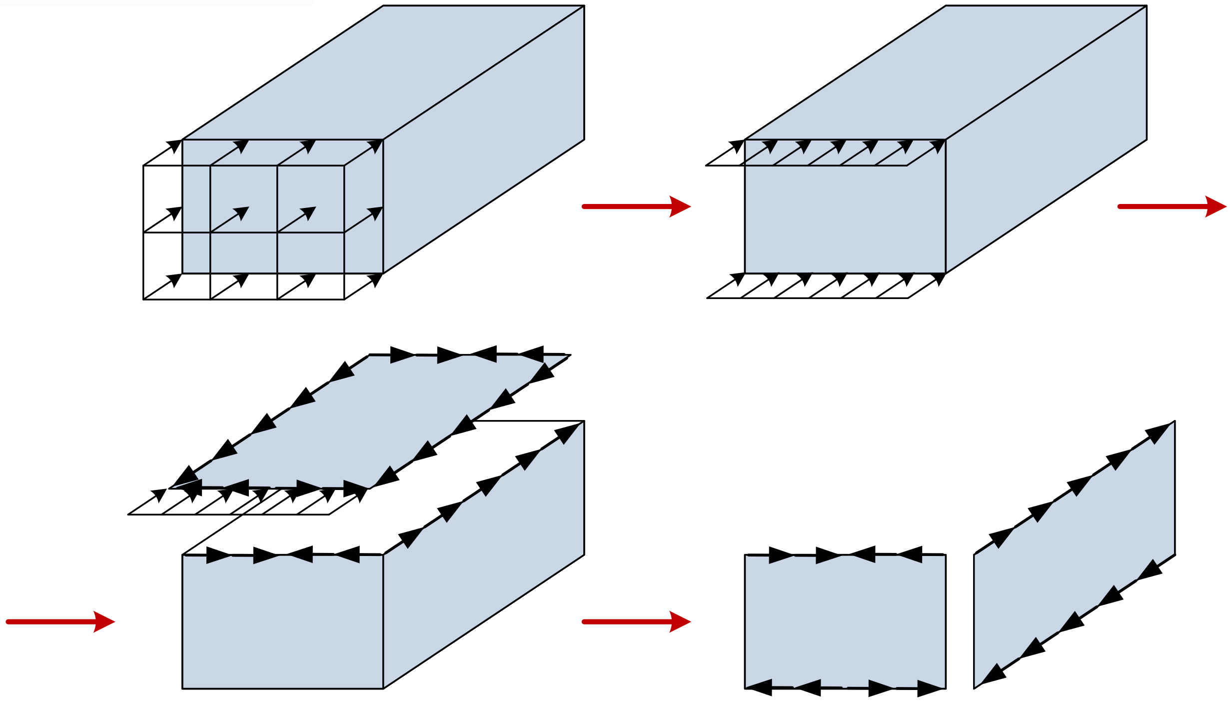

Fig. 3.2: Transfer of horizontal loads in frameless buildings

Transferring horizontal loads is only possible if the longitudinal joints of the roof panels are connected. Connecting the longitudinal joints of wall panels is not is not necessarily required. But if they are connected, there is a wide increase of stiffness and load bearing capacity.

4 Connections of frameless structures

4.1 Preliminary remark

For the transfer of horizontal loads in frameless structures the connections are decisive. In Fig. 4.1 a summary of the connections, which have to be taken into account, is given. In the following sections each kind of connection is briefly introduced.

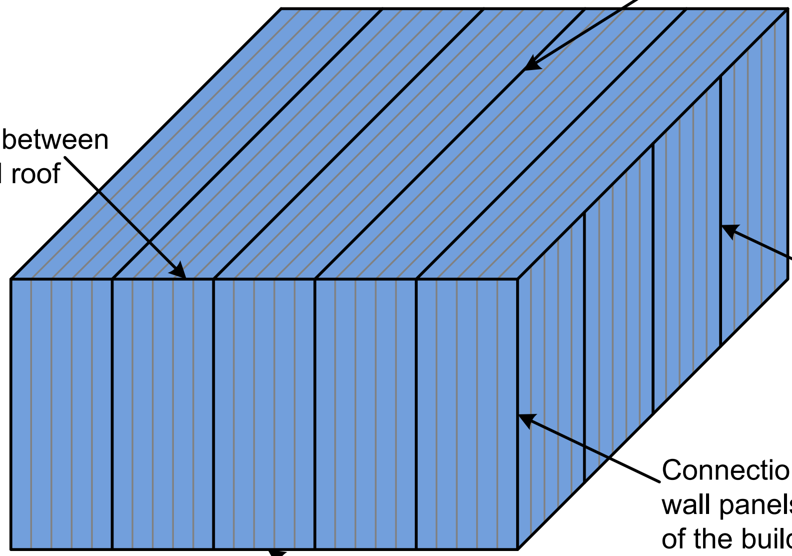

Fig. 4.1: Connections of a frameless building

The forces the fastenings have to be designed for depend on the stiffness of the connections. So to design a frameless structure the stiffness of the fastenings has to be known. It is only necessary to know the stiffness for working loads. Usually working loads are in the linear elastic part of the load-extension curve of a fastening [8]. So it is sufficient to determine the stiffness of the linear part of the curve.

4.2 Connections between wall and roof

Horizontal loads acting on the building are transferred through the roof into the wall panels. In Fig. 4.2 some possible variants of connections between wall and roof are presented.

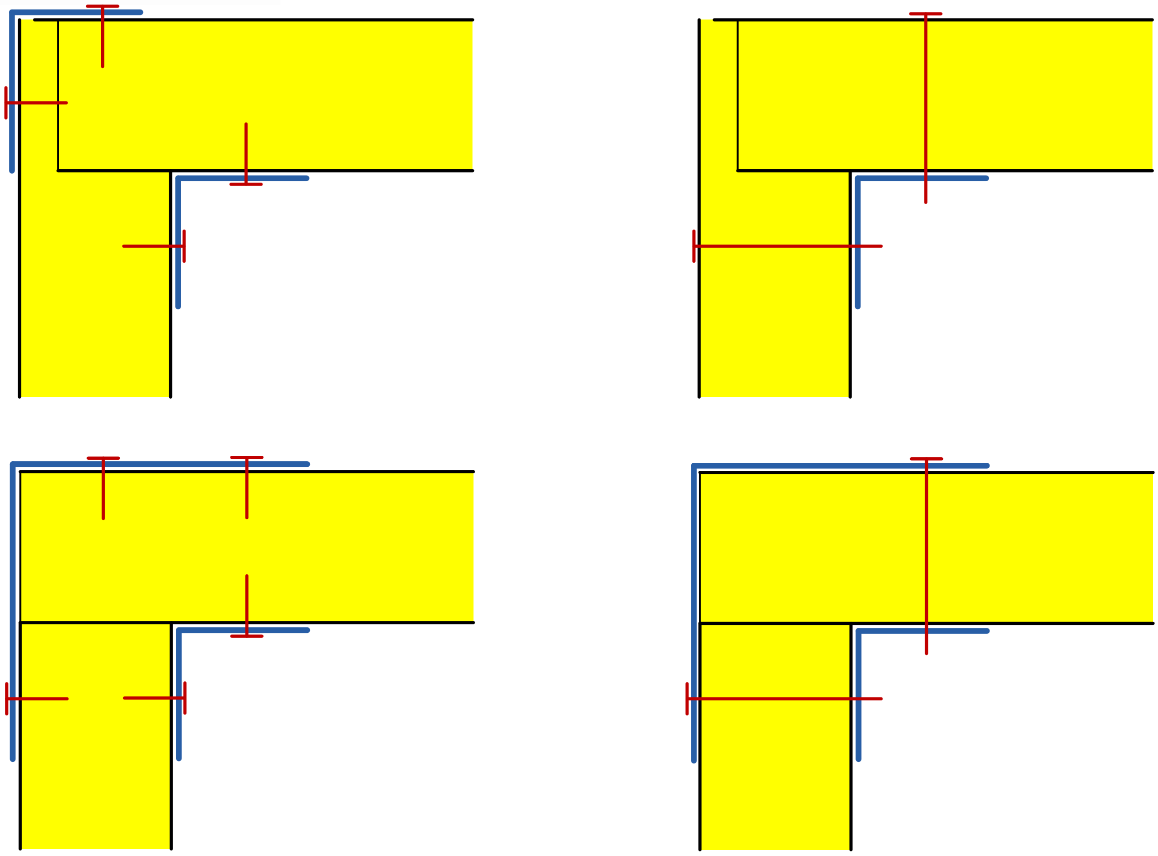

Fig. 4.2: **Connections between wall and roof**

At these connections in-plane shear forces are introduced from the roof into rectangular adjacent wall panels. So wall and roof panels have to be connected by steel or aluminium angles, which are mechanically fastened to the face sheets of the panels. For these fastenings usually self-drilling screws and sometimes also blind rivets are used. To get a sufficiently stiff connection with a sufficient load-bearing capacity it is recommended to connect both face sheets. Furthermore the angles have to be comparatively stiff, i.e. relatively thick sheets have to be used.

The angels are screwed to comparatively thin face sheets. For these "thick-to-thin" fastenings usually different kinds of special screw fasteners, sometimes with special washers, are available, e.g. the fasteners shown in the following figure.

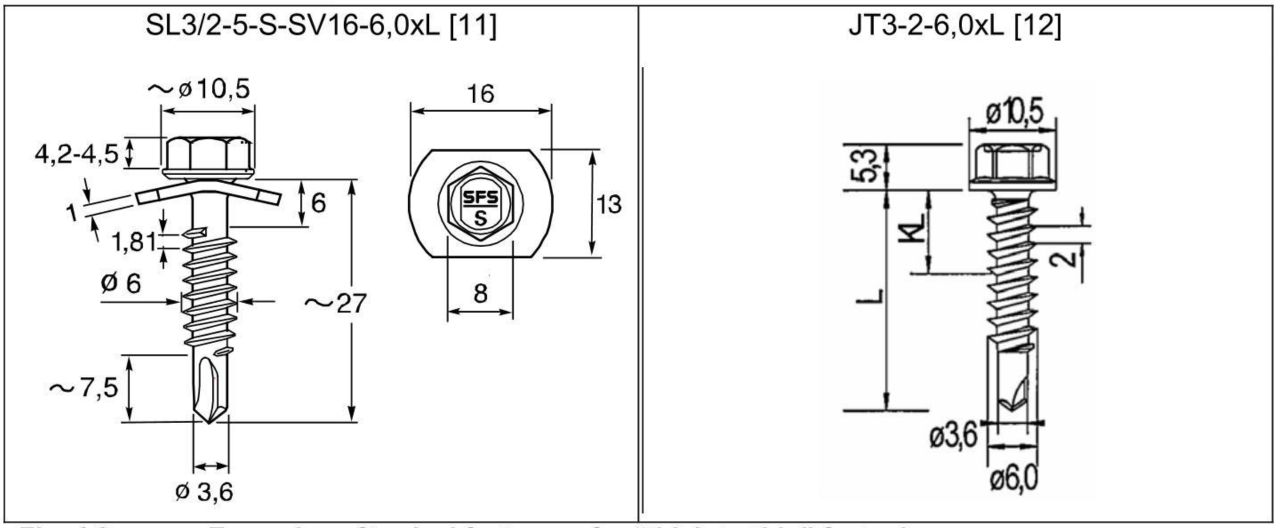

Fig. 4.3: Examples of typical fasteners for “thick-to-thin” fastenings

To get an approximation of the stiffness, which can be achieved by these fastenings exemplary the two fasteners given in Fig. 4.3 have been tested. A detailed documentation of the tests can be found in Deliverable D3.2 – part 2 [1]. In the following figures the load-extension curves determined in the tests are given. Also the stiffness k_v of the linear part of the curve is shown in the diagrams.

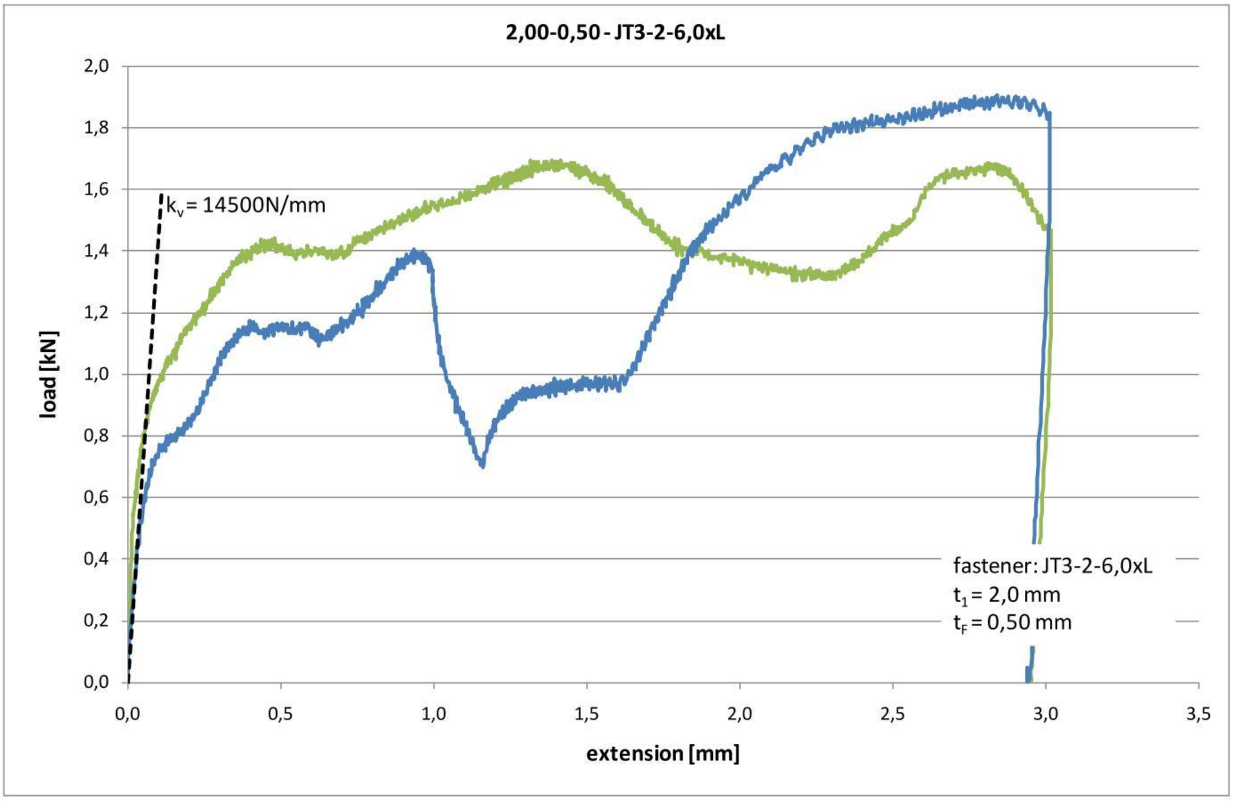

Fig. 4.4: Load-extension relation

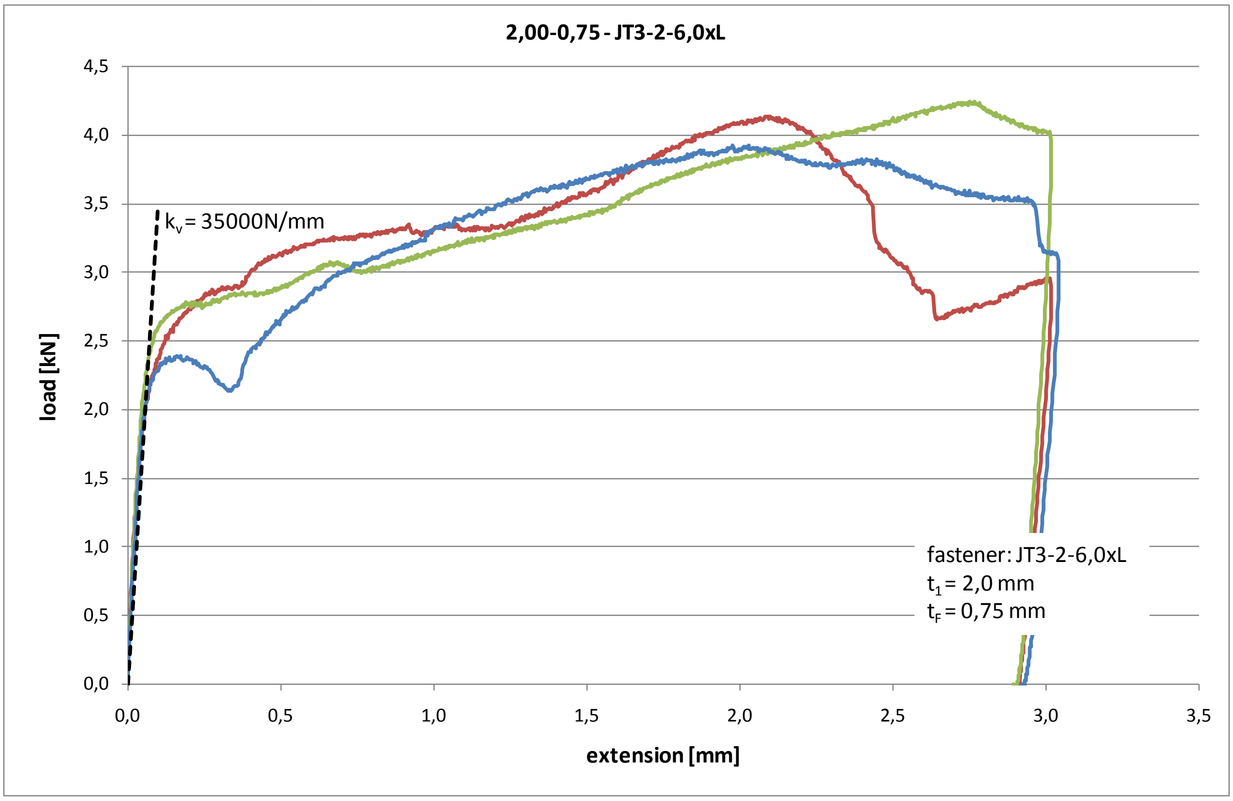

Fig. 4.5: **Load-extension relation**

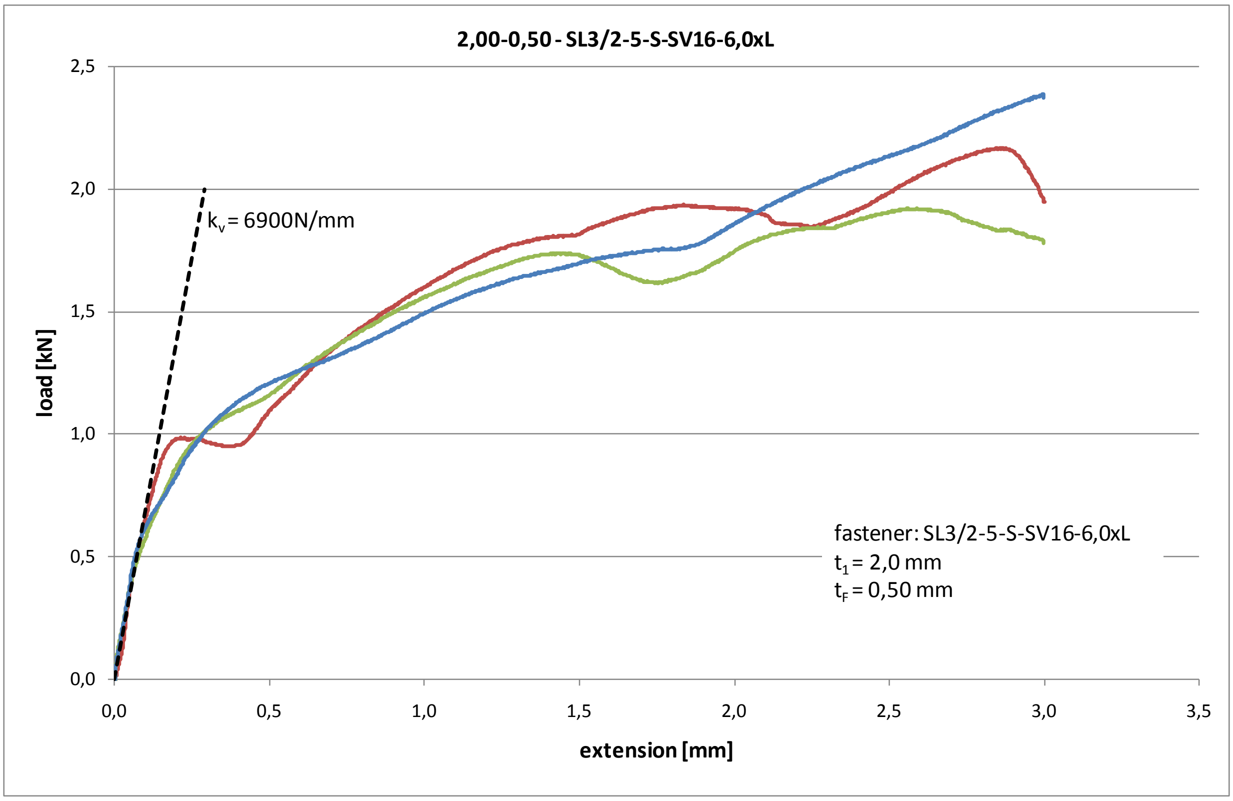

Fig. 4.6: **Load-extension relation**

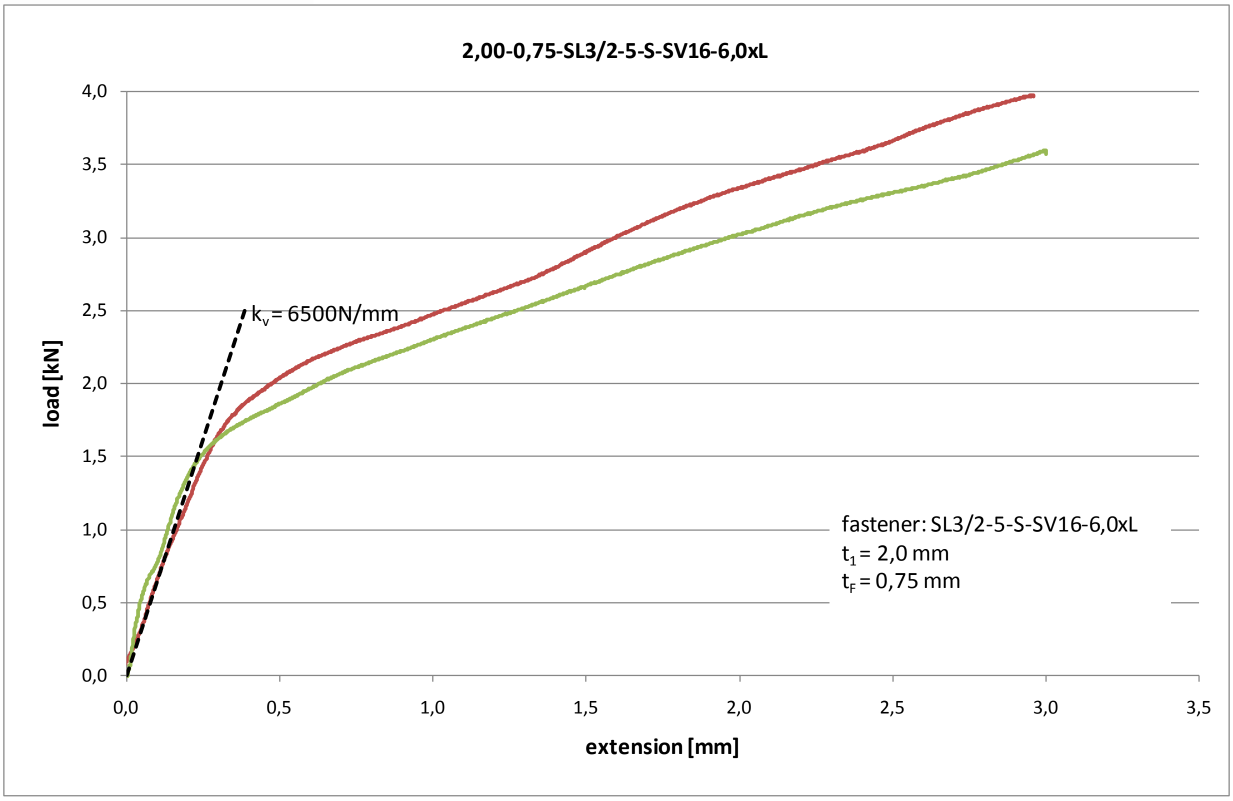

Fig. 4.7: Load-extension relation

fastener	thickness of angle [mm]	thickness of face sheet [mm]	stiffness k_v of fastening [kN/mm]
JT3-2-6,0xL	2,0	0,50	14,5
SL3/2-5-S-SV16-6,0xL	2,0	0,50	6,9
JT3-2-6,0xL	2,0	0,75	35,0
SL3/2-5-S-SV16-6,0xL	2,0	0,75	6,5

Tab. 4.1 Stiffness determined in the tests

Obviously stiffness and load-bearing capacity of a fastening strongly depend on the type of fastener. So no general values can be given and the values have to be determined by testing for each single case. Therefore small-scale tests on fastenings can be performed, e.g. according to the ECCS-recommendations for connections with mechanical fasteners in steel sheeting and sections [8].

For determination of the stiffness it should be noted that the load is transferred from the roof panel to the 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 should be relatively stiff, i.e. they should be made of comparatively thick sheets. Thus the angle can be considered as rigid. In the tests steel angles with thickness t = 2,0 mm have been used together with panels with thickness of face sheets t_F = 0,50 mm to 0,75 mm. In all tests no deformation of the angle occurred. All deformation was caused by hole elongation in the thin face sheet and by inclination of the fastener. So only the flexibility of the fastenings and not the deformation of the angle have to be taken into account. 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.

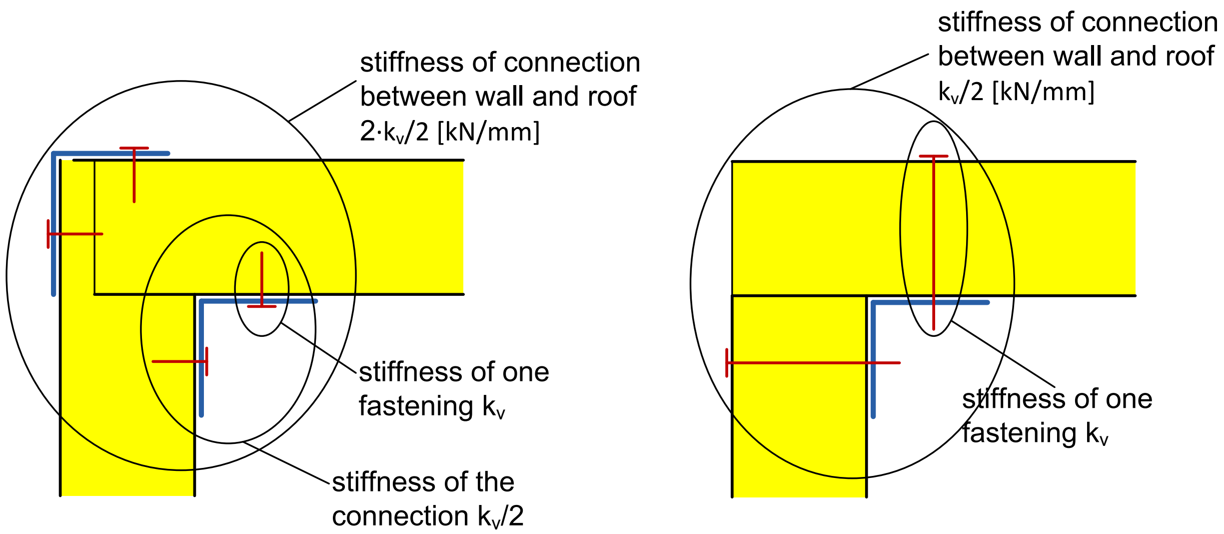

Fig. 4.8: Stiffness of connection between wall and roof

For the exemplary tested fasteners the stiffness of one fastening is between 6,5 kN/mm and 35 kN/mm. So if e.g. the internal and the external face are connected by one fastener each (Fig. 4.8 left) the stiffness of the connection is between 6,5 kN/mm and 35 kN/mm.

If the panels are connected as shown in Fig. 4.8 right, the stiffness of one fastening is between 2,5 kN/mm and 5,5 kN/mm (stiffness calculated according to [2], thickness of face sheets 0,4 mm to 0,75 mm, thickness of steel angle 1,5 mm to 3,0 mm, nominal diameter of fastener 5,5 to 8,0). The stiffness of one connection consisting of two fasteners has the stiffness 1,25 kN/mm to 2,75 kN/mm.

Thus, there is a wide range of possible stiffness for the considered connections. The stiffness is influenced by many parameters, e.g. type of fastener and washer, thickness of the face sheets, configuration and distance of fastenings. Because of that determination of stiffness has to be done with care. Furthermore the stiffness has to be determined for each single case.

4.3 Connection between wall and foundation

From the wall panels forces are transferred to the foundation. In Fig. 4.9 some possible versions of this connection are shown. The wall panels can either be directly connected with the foundation or they are connected with 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.

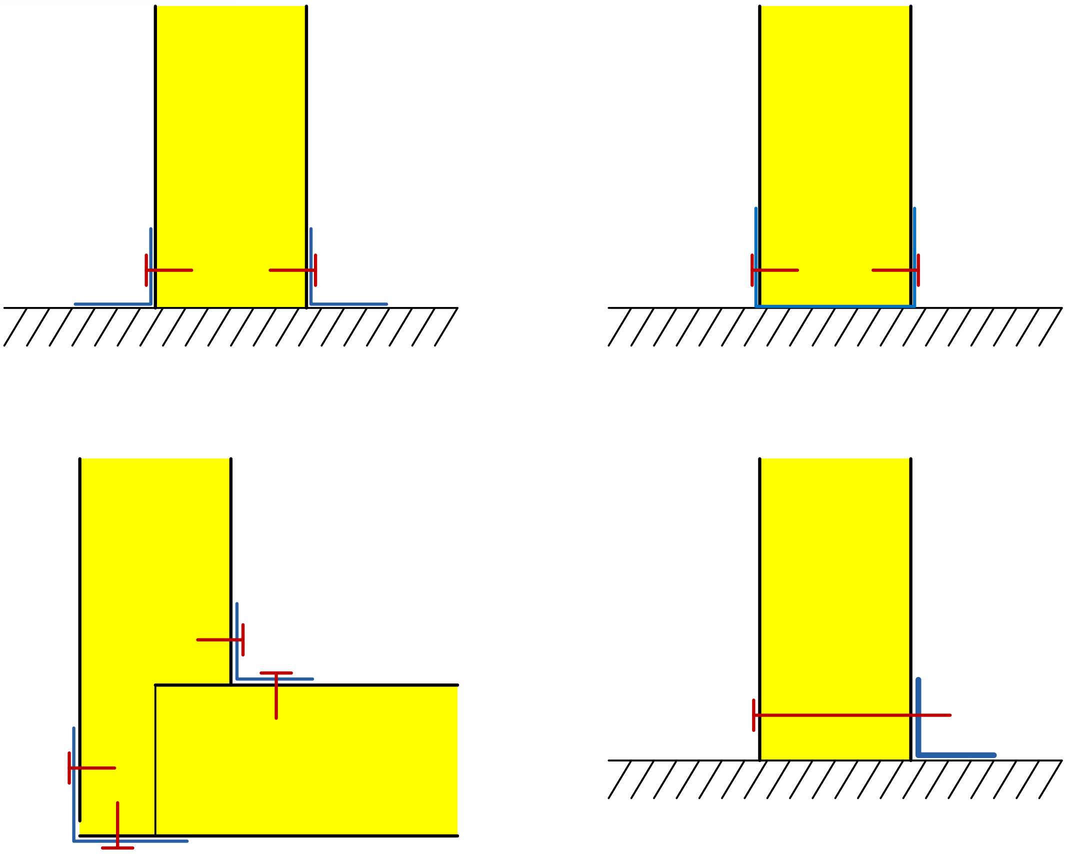

Fig. 4.9: Connection of wall panel and foundation

If the panel is connected to the foundation by two angles (Fig. 4.9, top), usually comparatively thick sheets are connected to the thin face sheets of the panel. So the same kind of fasteners as already introduced in the section above can be used.

In [13] a similar kind of connection was tested. Here self-tapping screws (nominal diameter 6,3) were used to connect L- and T-shaped steel profiles to the transverse edges of sandwich panels acting as shear diaphragms (in conventional constructions). The shear forces were introduced in the panels through these profiles and the stiffness of the fastening between section and sandwich panel was recalculated. The panels had steel face sheets with thickness 0,63 mm. Walls consisting of three or five panels have been tested. For these kind of fastening from the shear diaphragm tests a stiffness of approximately k_v = 3,3 kN/mm was recalculated. Because the stiffness of the diaphragm depends only on the stiffness of the fastenings the same stiffness of the fastening is determined independently of the dimensions of the wall.

For the kind of fastenings shown in Fig. 4.9 bottom right for usual applications the stiffness is between 2,5 /mm and 5,5 kN/mm [2], [17].

So also for these connections stiffness and load-bearing capacity have to be determined for each single case.

4.4 Connection between wall panels at corner of building

The connections at the corners of a building are not necessary for the transfer of loads. They mainly contribute to the water tightness of the building. So these connections are neglected in the following.

4.5 Longitudinal joints of roof panels

Usually flat and lightly profiled panels are only connected via a key and slot system at their longitudinal joints (Fig. 4.10). There are not any mechanical connections of the joints, what is sufficient if the panels are mounted on a substructure.

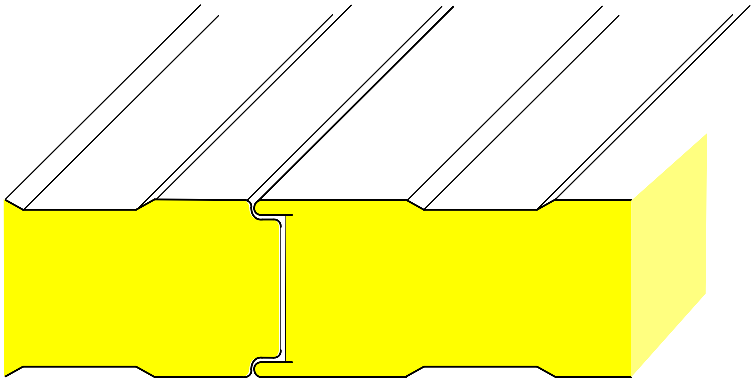

Fig. 4.10: Longitudinal joint

In frameless buildings horizontal loads are transferred through the roof or ceiling to the walls. Because of that unlike to panels mounted on a substructure a connection of the longitudinal joints is mandatory for roof panels. To increase load-bearing capacity and stiffness of the building it is recommended to construct the connections of the joints comparatively stiff.

To connect the longitudinal joints of flat or lightly profiled panels there are different possibilities. Mechanical fasteners as self-drilling screws or blind rivets can be used. In [14] different kinds of connections have been tested. Different kinds of fasteners – e.g. self-drilling screws and blind rivets -, glued connections and combinations of glue and mechanical fasteners have been used. Furthermore the arrangement of the fastenings has been varied, e.g. screwing through one or through two faces. Load-bearing capacity and stiffness have been determined for each tested variant of connection and compared to each other. As result of this comparison, [14] recommends mechanical fasteners, which connect both face sheets, to get strong and stiff joints. It is possible to use one screw, which passes through the thickness of the panel or to use two screws one mounted from the internal and one from the external side (Fig. 4.11). For the investigations presented in [15] blind rivets were used to connect the external face sheets of panels at longitudinal joints. With this kind of fastening, very stiff connections were constructed.

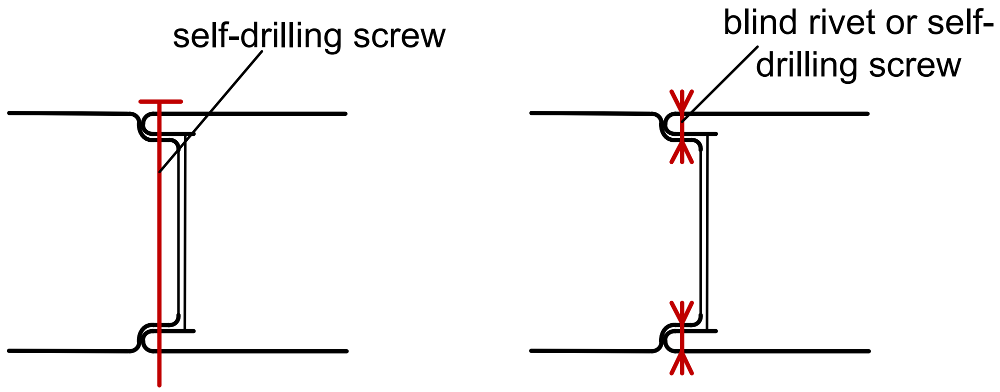

Fig. 4.11: Fastenings of longitudinal joints

The stiffness and the loads-bearing capacity of a fastening depend on both - on the type of fastener and on the geometry of the joint. The stiffness as well as the load-bearing capacity increase with an increasing number of sheets the fastener passes through. E.g. one blind rivet passes at least through two sheets. If panels with edge folded sheeting are used, one rivet can pass through four sheets (Fig. 4.12). If the geometry of the joint constrains an inclination of the fastener (e.g. panels with edge folded sheeting are used), very stiff fastenings can be achieved.

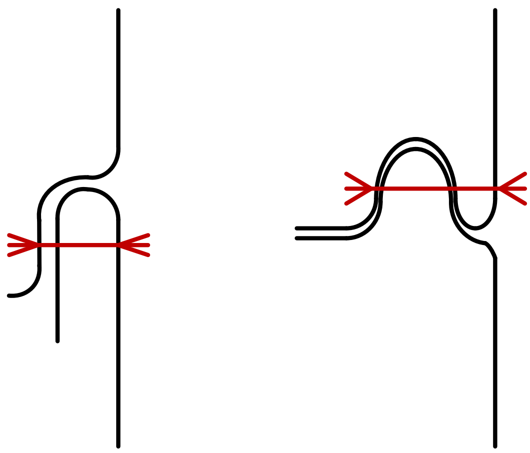

Fig. 4.12: Fastenings at longitudinal joint

Because there are many different possibilities of fasteners and geometries of joints, a general value for stiffness and load-bearing capacity cannot be given. Both values have to be determined for each single case. Therefore small-scale tests can be done. In the test the specific geometry of the joint and the specific fastener must be considered.

In [14] also panels with reinforced edges have been investigated. For reinforcement cold-formed profiles have been used. These lead to a slight increase of stiffness and load-bearing capacity. But panels with reinforced joints have a more expensive cost of production. As a further disadvantage the reinforcing profiles can act as thermal bridges.

In [14] and [15] also some connections with glue or a combination of glue and mechanical fasteners have been tested. But in both publications this kind of connection is not recommended.

4.6 Longitudinal joints of wall panels

For load transfer it is not necessary to connect the longitudinal joints of wall panels. If the panels are connected, load-bearing capacity and stiffness increase. The connections can be constructed as given in the previous section for roof panels.

4.7 Gluing of core materials

In some application glued connections between the core materials of adjacent panels are common practise. E.g. at the connection between wall and roof the cores are often glued by a polyurethane foam glue. Because the face sheets are much stiffer than the core material in-plane shear forces are transferred by the faces. So gluing the cores has no influence on load-bearing behaviour and capacity of shear loaded sandwich panels.

5 Design by numerical calculation

If frameless structures are subjected to horizontal loads, walls and roof are loaded by in-plane shear forces. When loaded by in-plane shear forces not the panels but the fastenings are decisive. So the fastenings have to be designed for the horizontal load.

The design values of the load have to be determined using load factors γ_F. The load factors γ_F are given in national specifications, e.g. in EN 1990 [5] and the related national annex. For wind loads γ_F is usually 1,5.

$$w_d = \gamma_F \cdot w_k \tag{5.1}$$

Using the design loads the forces of the different fastenings have to be determined. This can be done by numerical calculation. For some simple cases the forces of the fastenings can also be determined analytically. The design forces of the fastenings have to be compared to the load-bearing capacity. The design value of the load-bearing capacity is determined by dividing the characteristic value by the material factor γ_M.

$$V_{Rd} = \frac{V_{Rk}}{\gamma_M} \tag{5.2}$$

The characteristic value V_{Rk} is usually determined by testing. The characteristic values are usually given in European technical approvals (ETA) or in national approvals. The material factor γ_M is given by national specifications. According to EN 1993-1-3 [6] γ_M = 1,25, according to different approvals γ_M = 1,33 has to be used. For some kinds of fastenings there are also methods available to determine the resistance value by calculation, e.g. EN 1993-1-3 and [2].

Because the panels can be 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 corresponding displacements can be determined by a numerical analysis.

In a FE-analysis each panel is modelled as a rigid body. E.g. shell elements with a comparatively high thickness and a high elastic modulus can be used. For the examples presented in the report at hand shell elements with a thickness of 20 mm and an elastic modulus of 200.000 N/mm^2 have been used.

In the FE-model 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 has to be determined as shown above.

In the connections of the longitudinal joints and in the connections between wall and foundation forces in longitudinal as well as in transverse direction of the panel are transferred. To represent these connections two springs are used in the FE-model – one spring acts in longitudinal the other one in transverse direction. The resulting shear force of a fastening has to be calculated by vectorial addition of both forces.

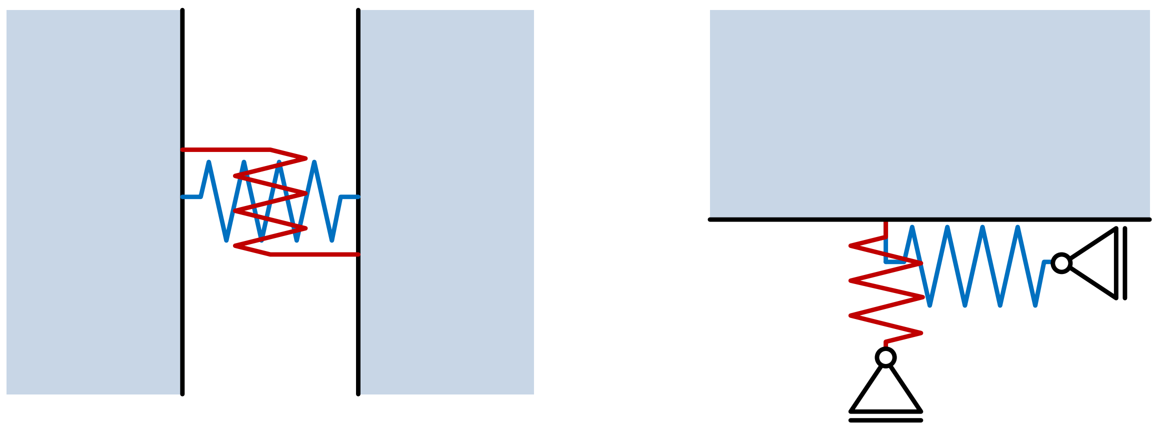

Fig. 5.1: Fastenings in the FE- model

Exemplarily the FE-models of a roof and a wall are presented in the following figures. In the figures only the panels, the supports and the load are shown. But there are also longitudinal springs between the panels and the supports and between adjacent panels.

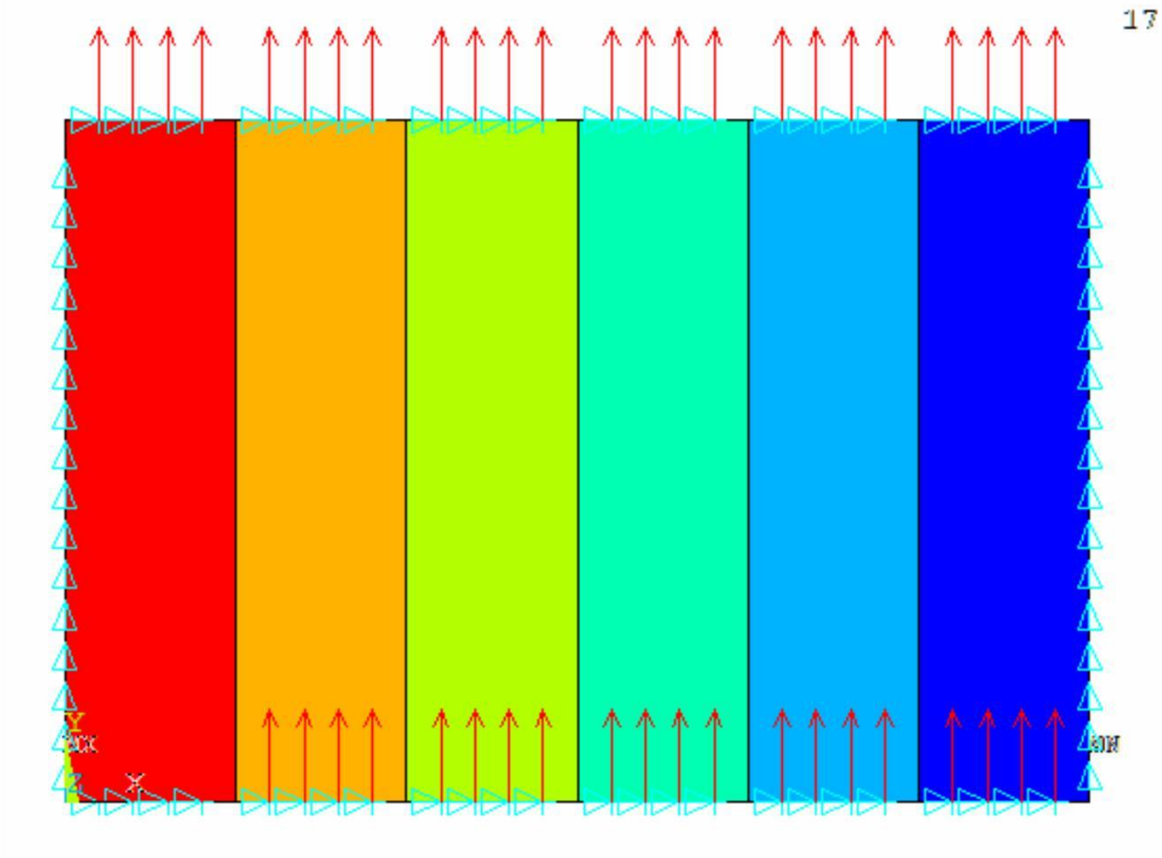

Fig. 5.2: FE-model of a roof

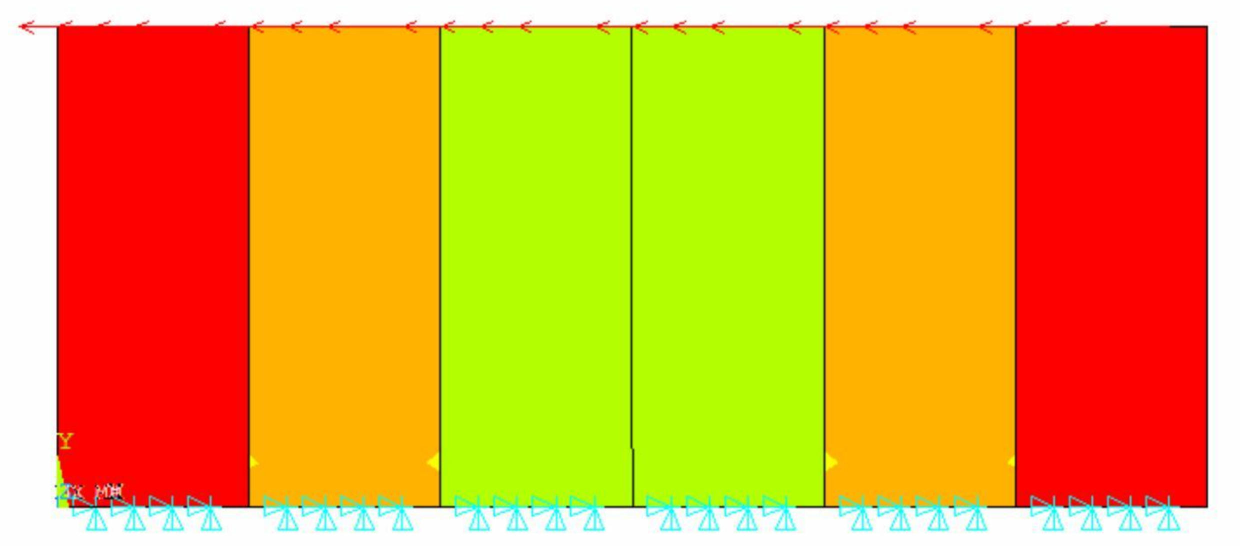

Fig. 5.3: FE-model of a wall

In addition to a numerical calculation for some simple applications the forces of the connections can also be determined by analytical calculation methods.

6 Roof panels – load in transverse direction

6.1 General load-bearing behaviour

Fig. 6.1 shows the general load bearing behaviour of a roof loaded by horizontal wind loads acting in orthogonal direction of the span of the roof panels. The outer (directly loaded) panel transfers a part of the wind load to the wall panels, which support this roof panel. 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. So the longitudinal joints are subjected to tension or compression loads.

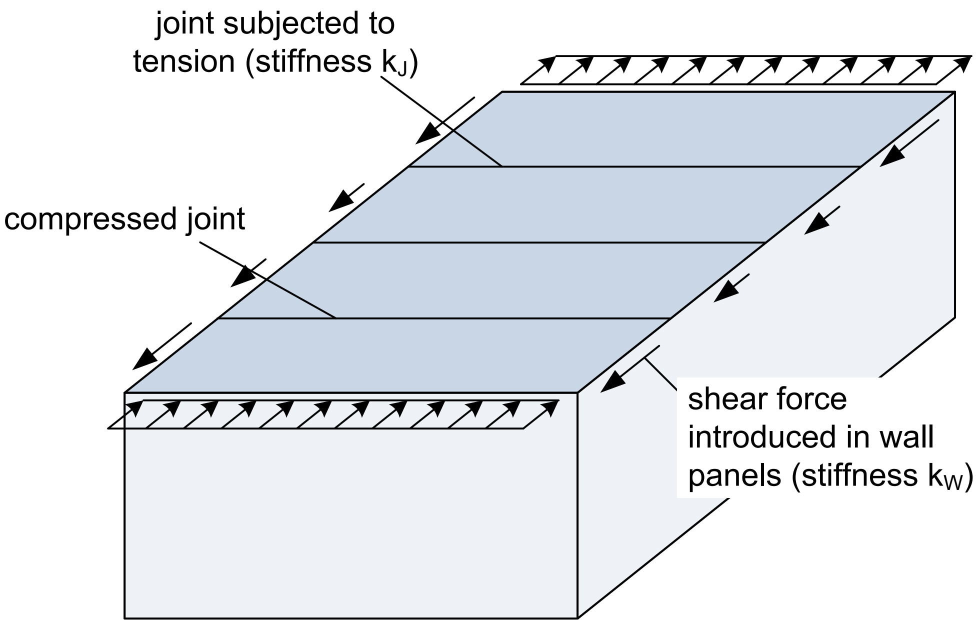

Fig. 6.1: General load bearing behaviour of a roof

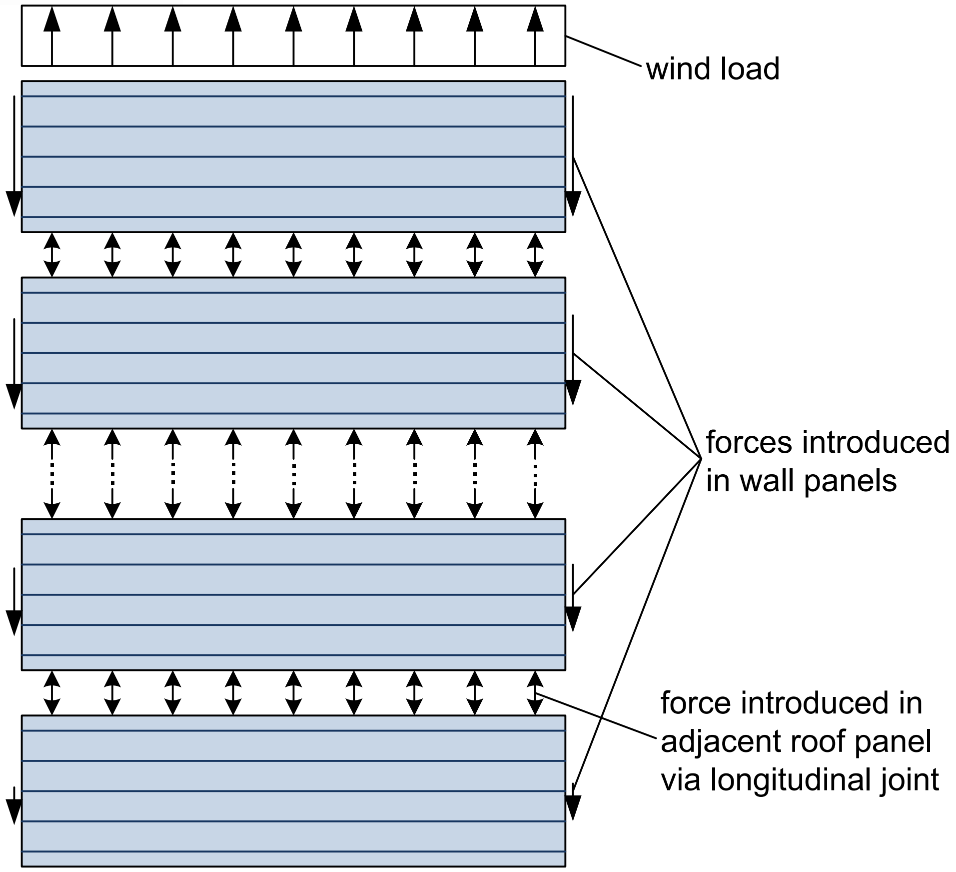

Fig. 6.2: **Transfer of horizontal loads through the roof**

The general load-bearing behaviour is shown by the roof given in Fig. 6.3. The roof consists of six panels with the length 4000 mm and the width 1000 mm. At the transverse edges the roof panels are connected to the wall with four connections. The stiffness of a fastening is 5 kN/mm. A connection consists of 4 fastenings. So the stiffness of a connection is 5 kN/mm. There are 16 connections at each longitudinal joint. Both face sheets are connected by a fastening with stiffness k_v = 5 kN/mm. So the stiffness of a connection is 10 kN/mm. Between the longitudinal edge of the outer panel and the wall there 16 connections with stiffness 5 kN/mm.

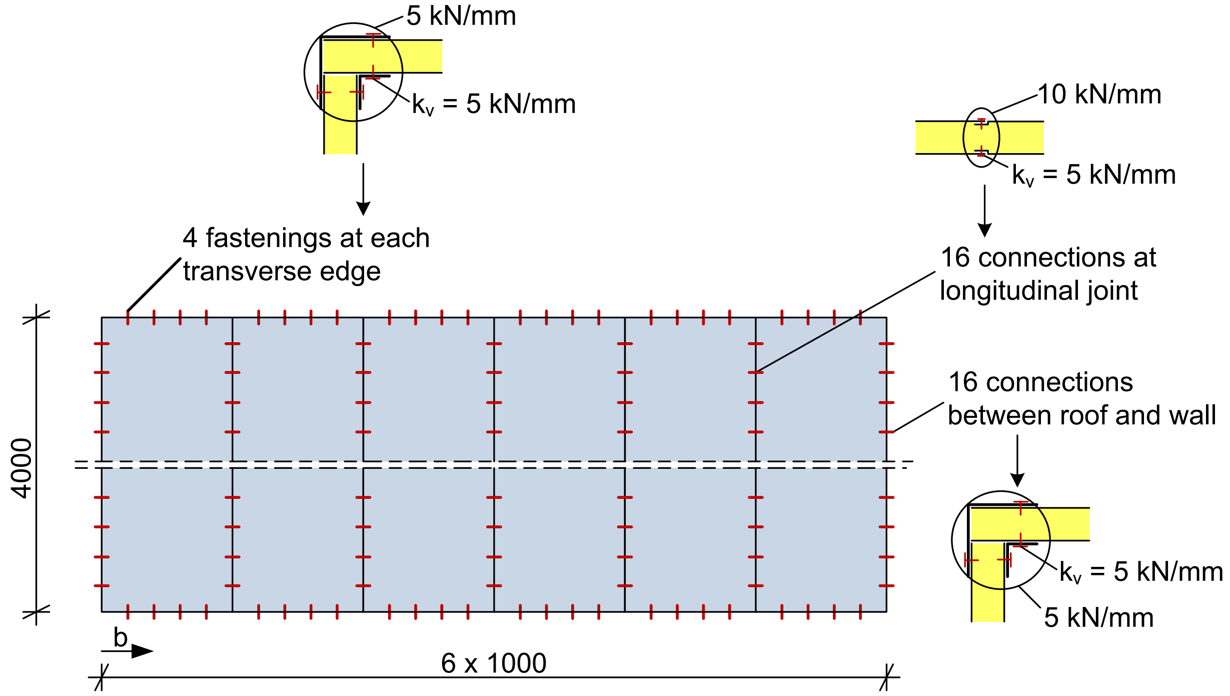

Fig. 6.3: Roof of FE-calculation

In the FE-model the connections at the joints are replaced by two longitudinal springs with stiffness 10 kN/mm, one spring in longitudinal and the other in transverse direction of the panels. The connections between wall and roof are replaced by one spring with stiffness 5 kN/mm. These springs act in the plane of the connected wall panels, i.e. at the longitudinal edges in longitudinal direction of the roof panel and at the transverse edge in transverse direction of the roof panel.

A wind suction load of 2 kN/m is assumed. To show the mechanisms of load transfer more clearly in the following example only the wind suction load acts on the roof, but not the wind pressure load at the opposite side. The FE-Model is shown in Fig. 6.4.

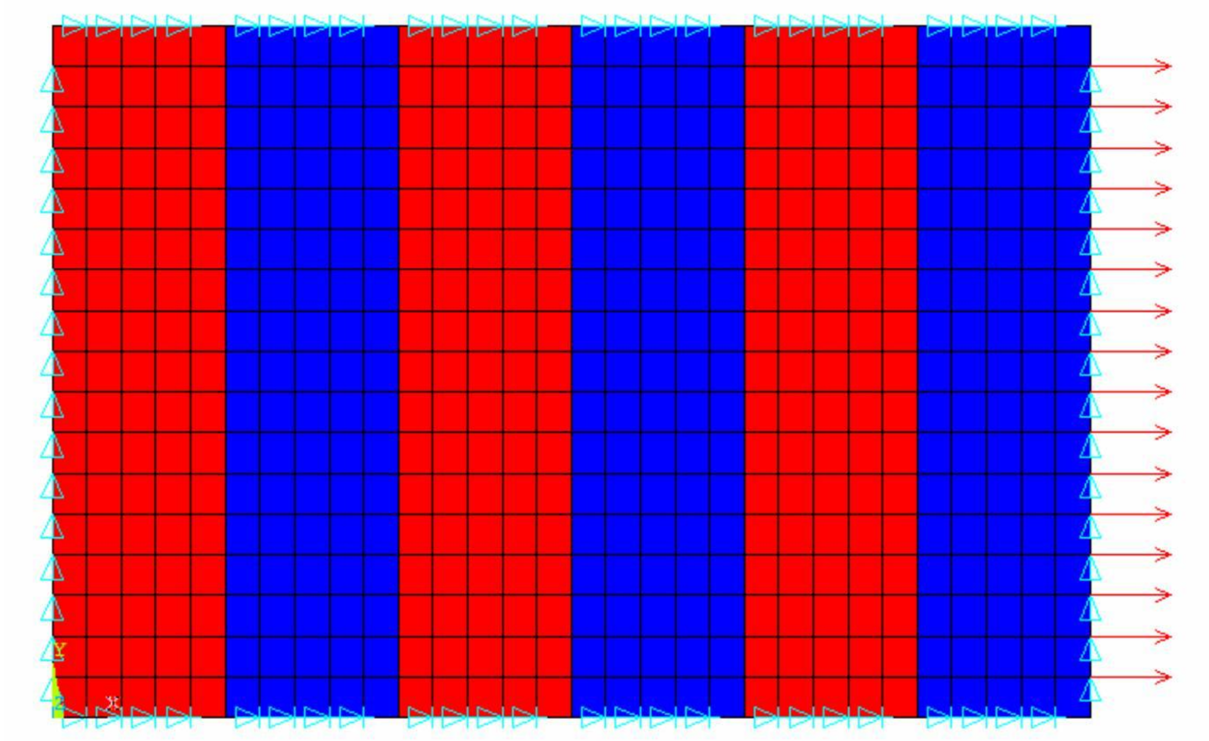

Fig. 6.4: Model of a roof with load in transverse direction

The horizontal load leads to a transverse displacement of the panels as shown in Fig. 6.5.

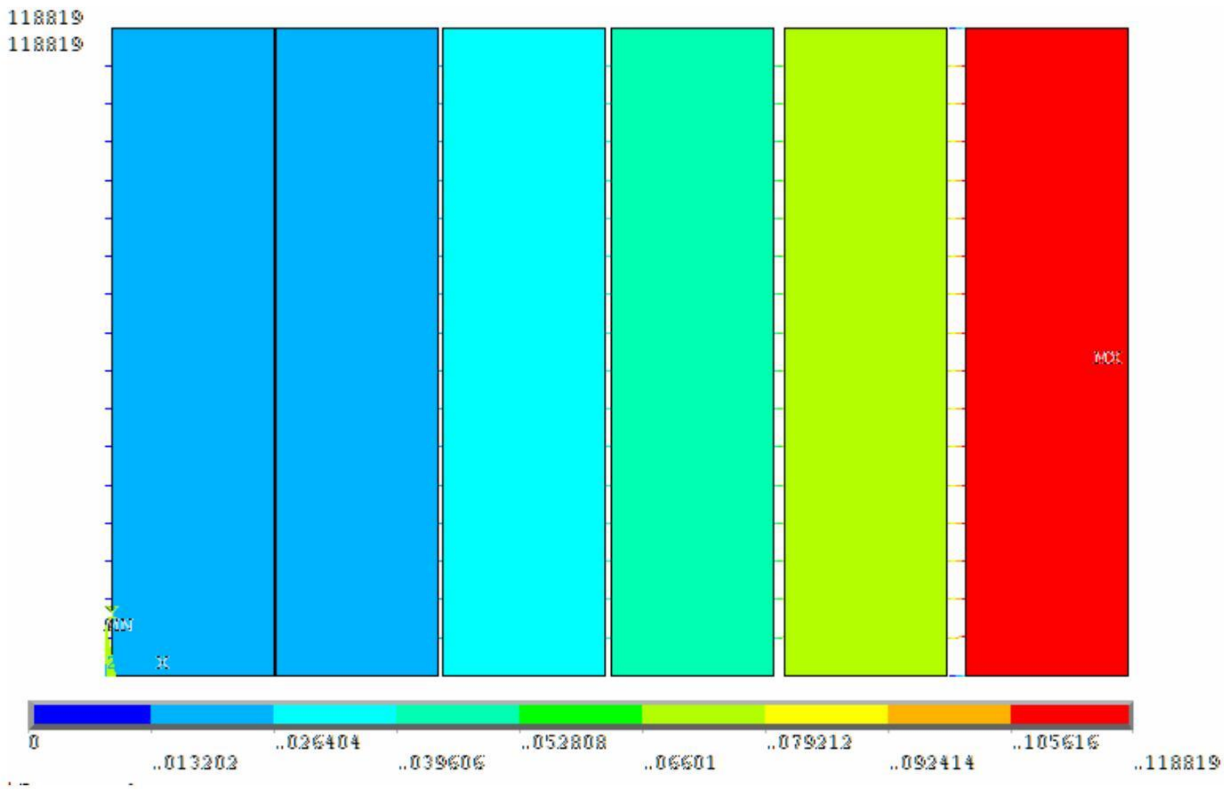

Fig. 6.5: Displacement of panels

The displacement of the panels causes forces in the connections between wall and roof and in the connection of the longitudinal joints. Only forces in direction of the load occur. So there are only forces introduced in the walls being parallel to the direction of load.

The force introduced in a wall panel decreases with increasing distance of the panel from the line of load application.

In the following figure the forces resulting in the connections are presented. At the transverse edge of a panel all connections are loaded by the same force. Also all connections of a longitudinal joint transfer the same force. In longitudinal direction of the panels only very small and therefore negligible forces occur.

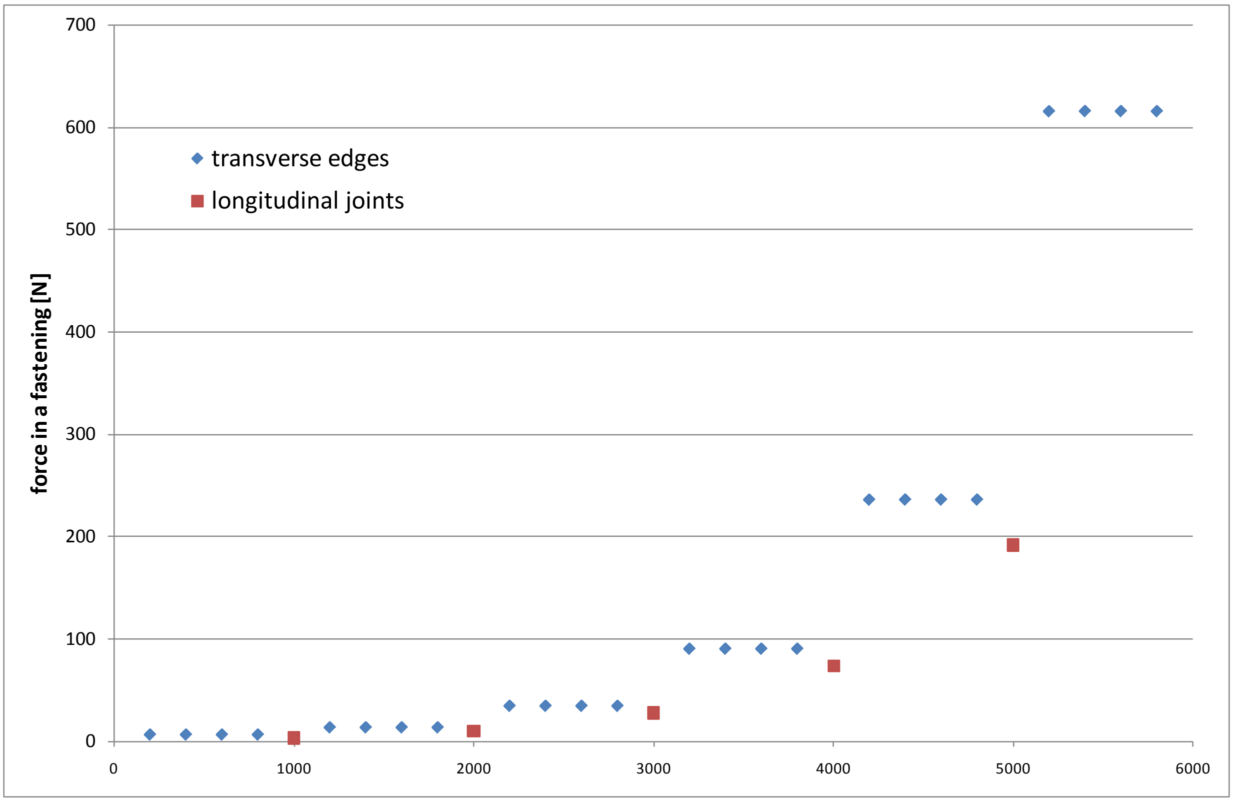

Fig. 6.6: Forces of fastenings

The distribution of the force to the wall and to the adjacent roof panel depends on the stiffness of the connections. With increasing stiffness of the connection of the joints the part of the force transferred to the adjacent roof panel increases. If a rigid connection of the longitudinal joints is assumed, all roof panels act together as one single rigid element. Thus there is a continuous distribution of the horizontal load to all panels of a wall.

If we consider the extreme example – roof panels without connections at the longitudinal joints – the outer wall panels have to transfer the whole horizontal load, which is introduced in the outer roof panel. So only the outer panels of a wall are loaded by in-plane shear forces. Because of that it is important to connect the roof panels at the longitudinal joints. These connections should be comparatively stiff. Stiff connections of the longitudinal joints have the advantage of a more evenly distribution of the forces to the wall panels.

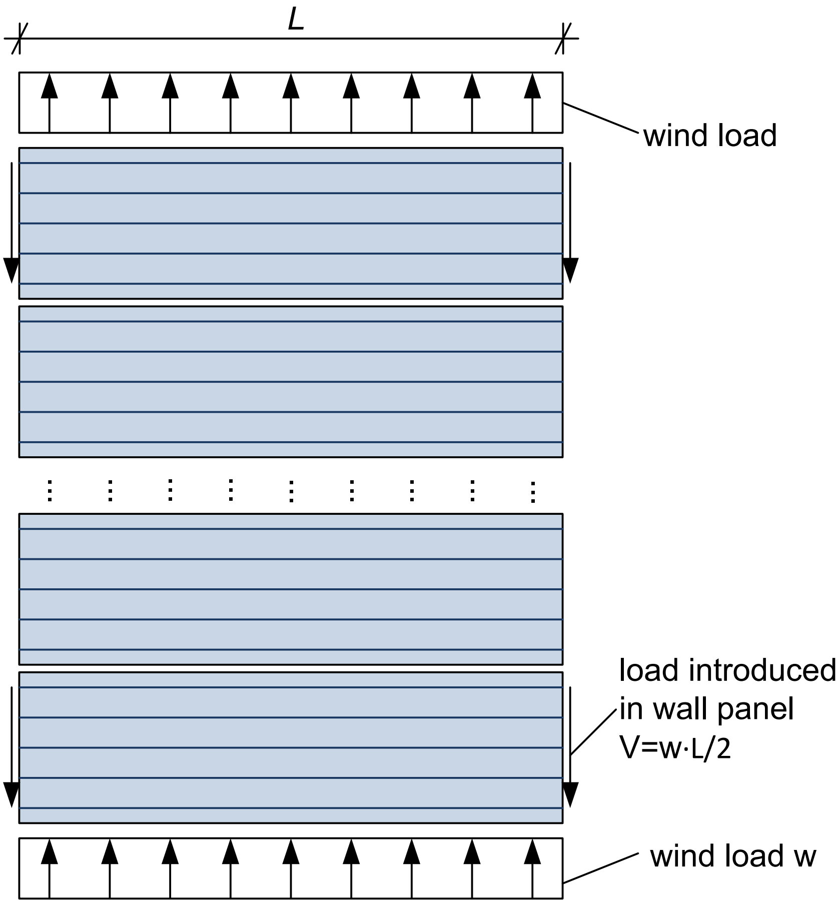

Fig. 6.7: Transfer of loads through the roof (no connections at longitudinal joints)

A wind pressure load causes compression forces in the longitudinal joints. Depending on the geometry of the joint compression forces can (partly) by transferred by contact. In this case the stiffness and the load-bearing capacity of the joint increase. If the stiffness of the joints increases, the forces are more evenly distribution to the wall panels and the force introduced in one wall panel decreases. So it is on the safe side to neglect a transfer of forces by contact.

6.2 Analytical determination of forces and displacements

For simple applications the distribution of forces to the connections can also be determined by analytical calculation. To simplify the calculation procedure the forces acting in the connections are smeared over the width B or over the length L of the panel. So also the stiffness of the longitudinal springs has to be converted to a stiffness per unit length [N/mm^2]. For the connection between wall and roof the stiffness is

$$k_W = \frac{n \cdot k}{B} \tag{6.1}$$

n number of connections at a transverse edge k stiffness of one connection

B width of a panel

and for the connections of a longitudinal joint

$$k_J = \frac{n \cdot k}{L} \tag{6.2}$$

n number of connections at a longitudinal edge

L length of panels

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. Only forces, which are parallel to the direction of load act.

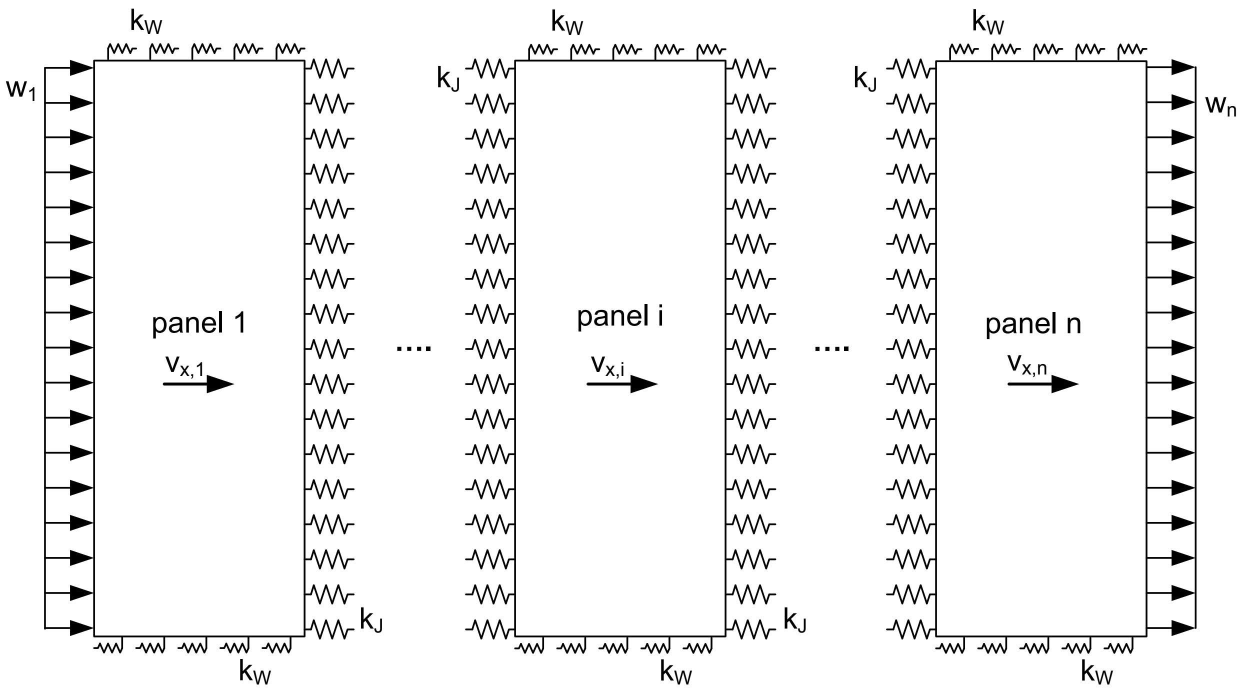

Fig. 6.8: Model of a roof for analytical calculation

For each panel i a displacement $v_{x,i}$ in transverse direction occurs. With the stiffness of the connections the following forces can be determined.

At the connection between wall and roof:

$$F_{W,i} = v_{x,i} \cdot k_W \cdot B \tag{6.3}$$

At the longitudinal joint between panel i and i-1:

$$F_{J,i,l} = \left(v_{x,i-1} - v_{x,i}\right) \cdot k_J \cdot L \tag{6.4}$$

At the longitudinal joint between panel i and i+1:

$$F_{J,i,r} = \left(v_{x,i+1} - v_{x,i}\right) \cdot k_J \cdot L \tag{6.5}$$

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

$$F_{1,l} = w_1 \cdot L \tag{6.6}$$

and at panel n

$$F_{n,r} = w_n \cdot L \tag{6.7}$$

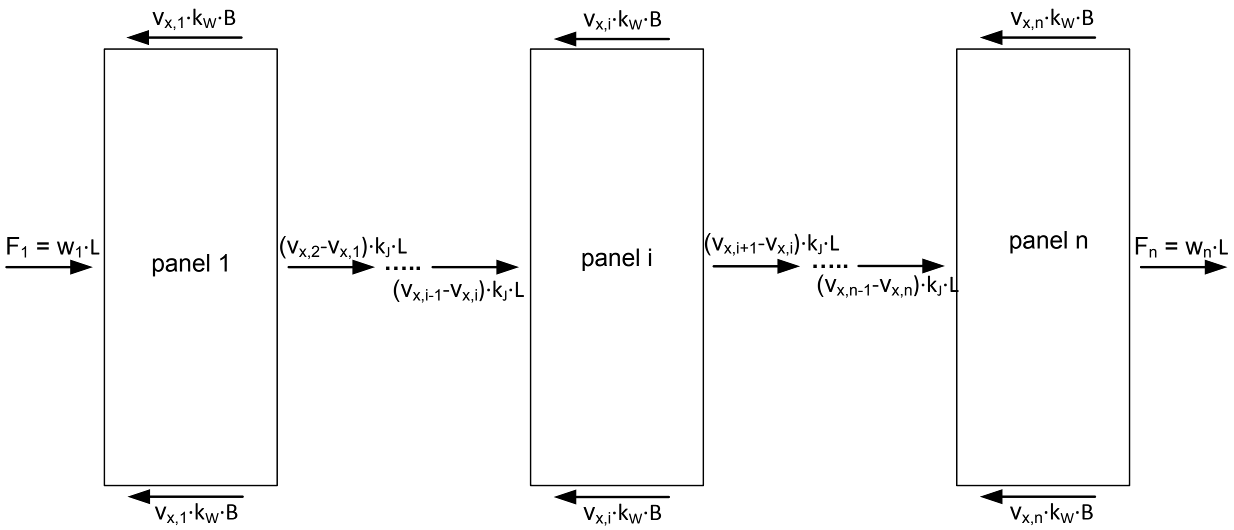

Fig. 6.9: Forces acting on the panels of a roof

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

$$\left(v_{x,i-1} - v_{x,i}\right) \cdot k_J^l \cdot L + \left(v_{x,i+1} - v_{x,i}\right) \cdot k_J^r \cdot L - 2 \cdot v_{x,i} \cdot k_W \cdot L + F_i = 0 \tag{6.8}$$

$$A_i \cdot v_{x,i-1} + B_i \cdot v_{x,i} + C_i \cdot v_{x,.i+1} + F_i = 0 \tag{6.9}$$

$$A_i = k_J^l \cdot L \tag{6.10}$$

$$B_i = -\left(k_J^l \cdot L + k_J^r \cdot L + 2 \cdot k_W \cdot B\right) \tag{6.11}$$

$$C_i = k_J^r \cdot L \tag{6.12}$$

F_i external load according to (6.6) and (6.7)

For n panels we get n equations with n unknown displacements $v_{x,i}$. Solving the equation system results in the displacement v_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:

$$V = \frac{v_{x,i} \cdot k_W \cdot B}{n} \tag{6.13}$$

n number of fastenings at a transverse edge

Connection at longitudinal joints:

$$V = \frac{\Delta v_{x,i} \cdot k_J \cdot L}{n} \tag{6.14}$$

n number of connections at a joint

$$\Delta v_{x,i} = \begin{cases} v_{x,i-1} - v_{x,i} \\ v_{x,i+1} - v_{x,i} \end{cases} \tag{6.15}$$

7 Roof panels – load in longitudinal direction

7.1 General load-bearing behaviour

Fig. 7.1 shows the general load-bearing behaviour of a roof subjected to horizontal wind loads acting in direction of the span of the roof panels. The wind load is transferred to the walls, which are parallel to the direction of the load. So in the longitudinal joints of the roof panels forces in longitudinal direction have to be transferred. If the longitudinal joints of the roof panels are not connected, transfer of horizontal loads via the roof is not possible. Forces are also introduced in the walls, which are orthogonal to the direction of load. So there is a circumferential shear force, as known from shear diaphragms made of trapezoidal sheeting.

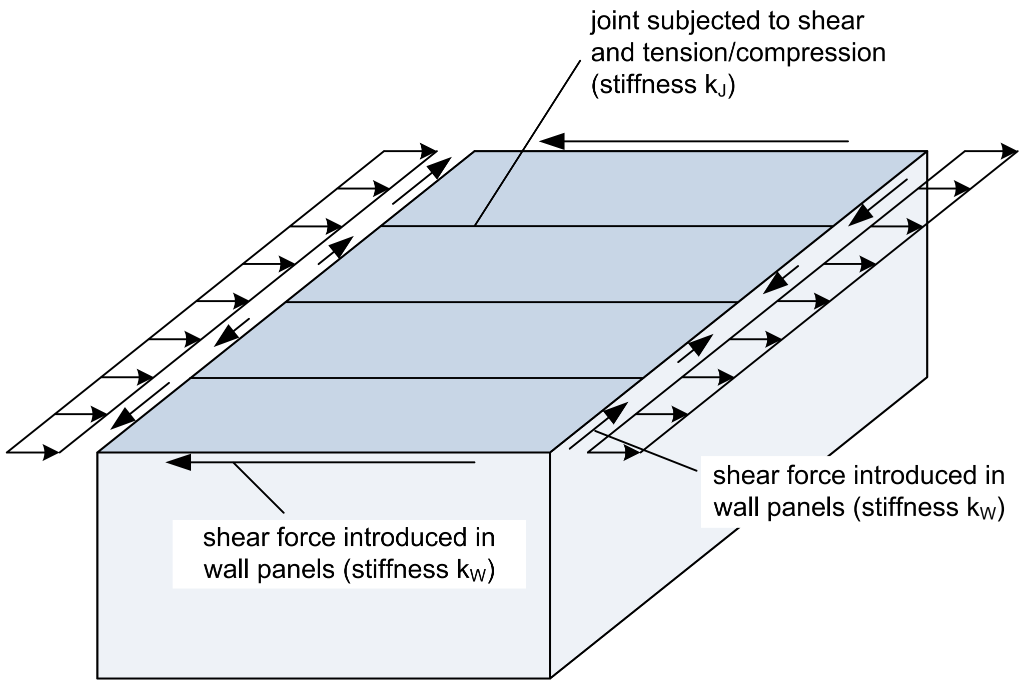

Fig. 7.1: General load-bearing behaviour

The general load-bearing behaviour is shown by the roof introduced in section 6. The FE-model is given in the following figure. For wind suction and compression load 2 kN/m are assumed.

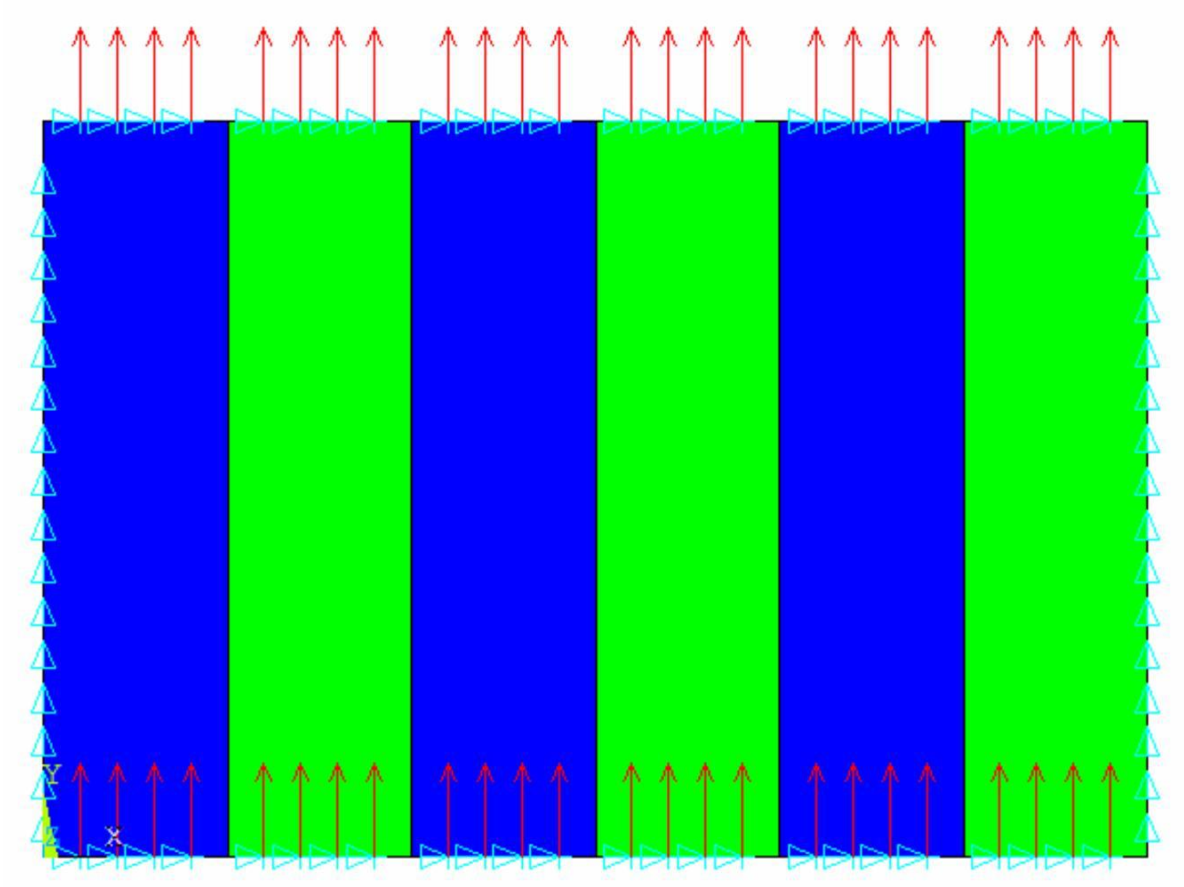

Fig. 7.2: Model of a roof

In contrast to common shear diaphragms consisting of trapezoidal sheeting or sandwich panels mounted on a substructure, in addition to a displacement in longitudinal direction of the panels a rotation occurs (Fig. 7.3).

Because of this rotation in the longitudinal joints not only forces in longitudinal but also in transverse direction of the panels occur.

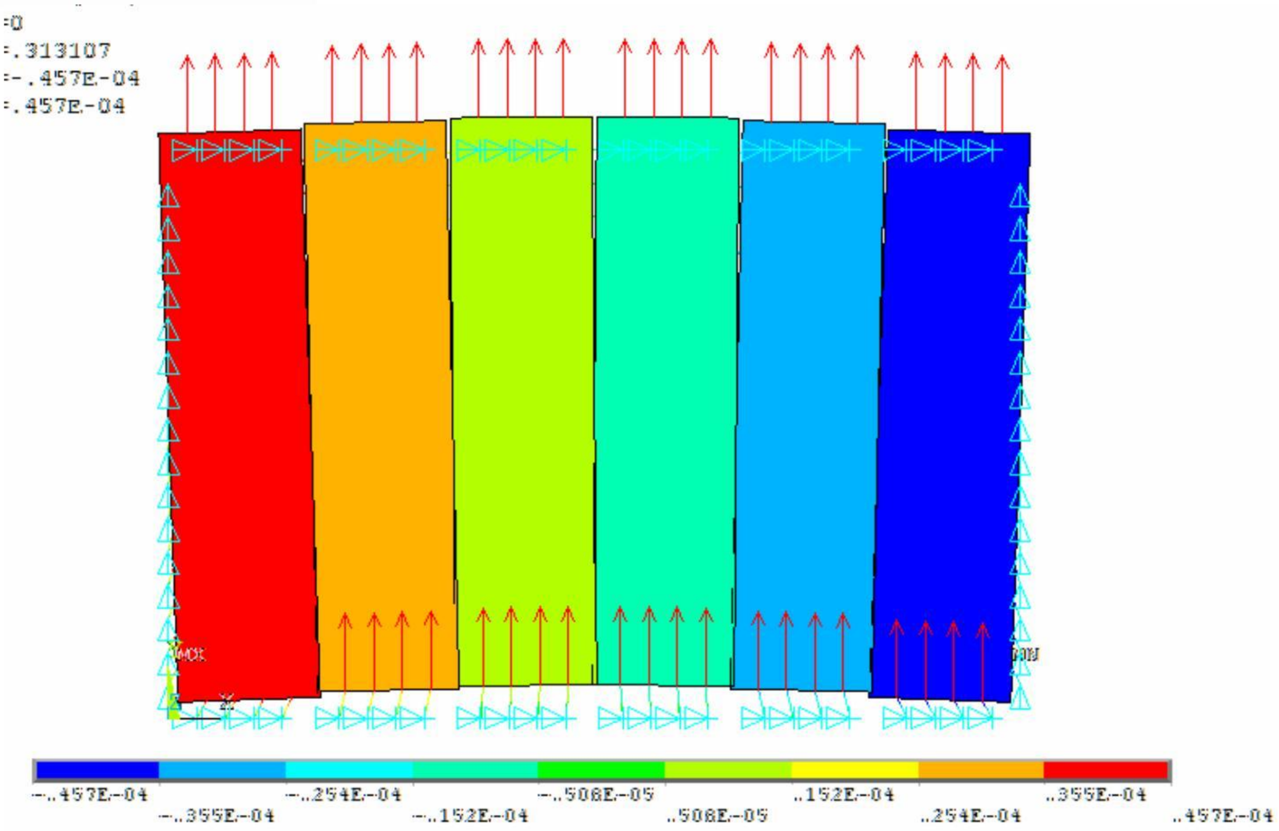

Fig. 7.3: Rotation of panels

For the roof given above the forces of the connections are given in the following figures. In Fig. 7.4 the forces transferred to the walls, which are orthogonal to the direction of the load (circumferential shear force), are given. The forces at a transverse edge of a panel are distributed constant to the connections of this edge.

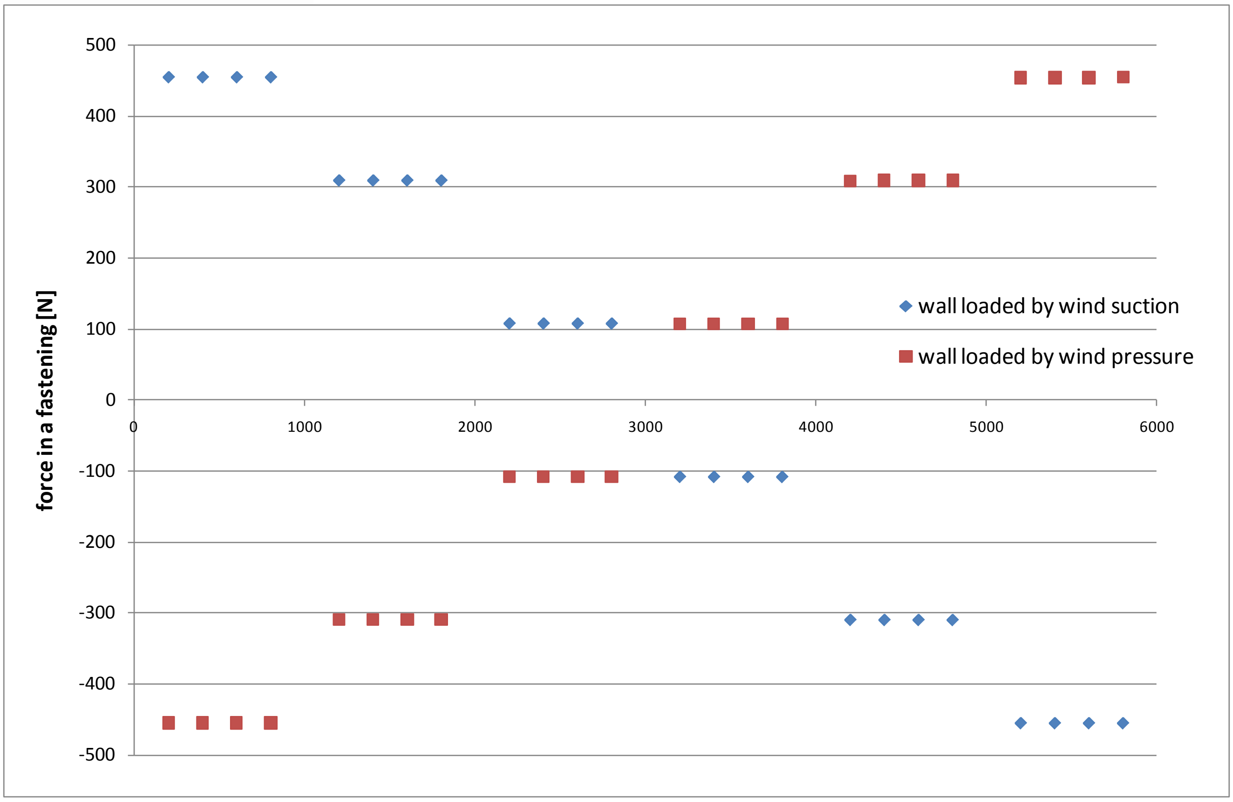

Fig. 7.4: Forces introduced in wall panels

In Fig. 7.5 the distribution of the forces in the longitudinal joints is given. Only the force acting in transverse direction is given in the diagram. Obviously the force is distributed linear over the joint. In the centre of the joint the force is approximately zero. So the centre of rotation is located in the centre of the panel.

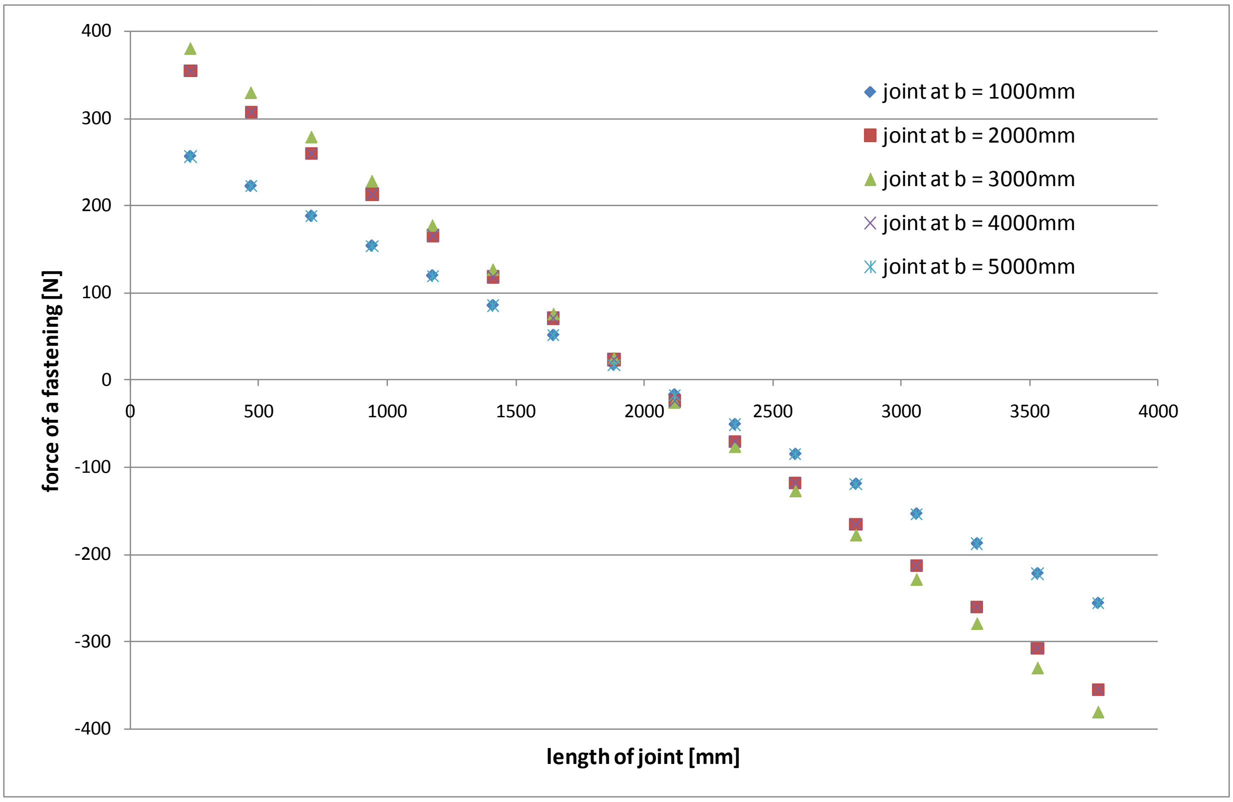

Fig. 7.5: Forces of fastenings at longitudinal joints (transverse direction)

The forces acting in longitudinal direction are uniformly distributed to the connections of a joint. Also all connections, which connect the longitudinal edges of the outer panels to the walls, are loaded by the same force.

The forces acting in direction of the span are independent of the stiffness of the connections, whereas the forces, which act in transverse direction of the panels, depend on the stiffness. With increasing stiffness of the connections between wall and roof, the forces, which are introduced in the wall at the transverse edges, increase. They decrease with increasing stiffness of the connections of the longitudinal joints, whereas the forces at the longitudinal joints increase. If for the longitudinal joints connections without flexibility are assumed - i.e. all roof panels act as one rigid element -, no forces are introduced in the wall panels being orthogonal to the load.

7.2 Analytical determination of forces and displacement

Also in this case an analytical determination of the forces of connections and the displacements of the panels is possible. To simplify the calculation procedure the smeared stiffness k_W and k_J as described above is used.

The forces, which are introduced in the walls being parallel to the direction of load, can be determined from equalisation of forces in longitudinal direction.

$$V_1^l = V_n^r = \frac{n_{SW}}{2} \cdot w \cdot B \tag{7.1}$$

with

n_{SW} number of sandwich panels

B width of a panel

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

$$w = w_S + w_C \tag{7.2}$$

w_S wind suction load

w_C wind compression load

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

$$V_i^l - w \cdot B - V_i^r = 0 \tag{7.3}$$

$$V_i^l = V_{i-1}^r \tag{7.4}$$

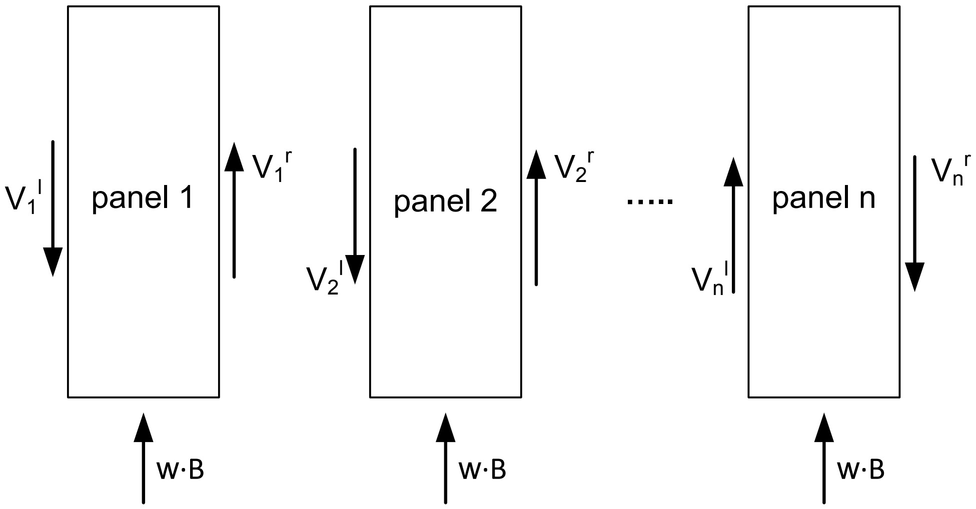

Fig. 7.6: Forces acting in longitudinal direction of the panels

The rotation φ_i of a panel leads to the following displacements at the edges. The displacements are described using a local coordinate system, which has its point of origin at the centre of rotation, i.e. at the centre of the panel. The direction of the x-axis is the transverse direction of the panel; the direction of the y-axis is parallel to the span of the panel.

Transverse displacement of transverse edge:

$$v_{x,i} = \varphi_i \cdot \frac{L}{2} \tag{7.5}$$

Longitudinal displacement of transverse edge:

$$v_{y,i} = \varphi_i \cdot x \tag{7.6}$$

Transverse displacement of longitudinal edge:

$$v_{x,i} = \varphi_i \cdot y \tag{7.7}$$

Longitudinal displacement of longitudinal edge:

$$v_{y,i} = \varphi_i \cdot \frac{B}{2} \tag{7.8}$$

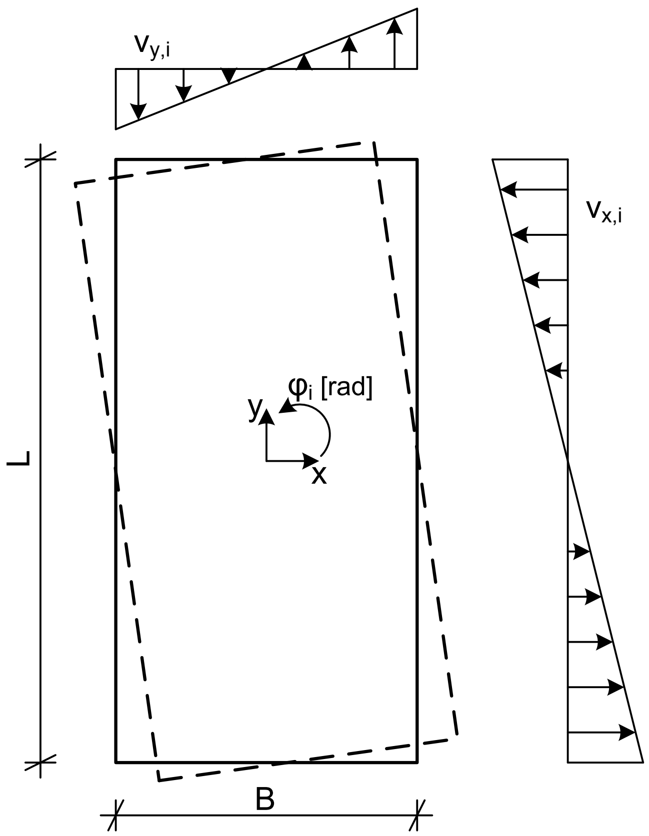

Fig. 7.7: Displacements caused by rotation φ_i

In Fig. 7.8 the transverse forces caused by the rotation φ_i of a panel are given. They 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.

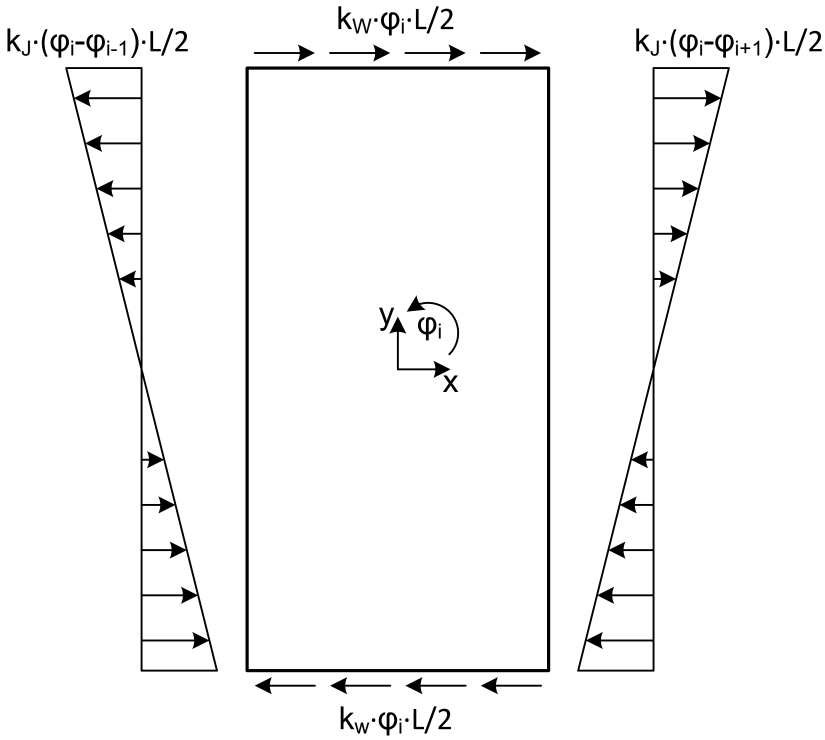

Fig. 7.8: Transverse forces caused by rotation φ_i

In Fig. 7.9 the forces resulting from the smeared forces given in Fig. 7.8 are shown. In addition the external wind load and the longitudinal forces in the joints, which can be determined from equilibrium of forces, are given.

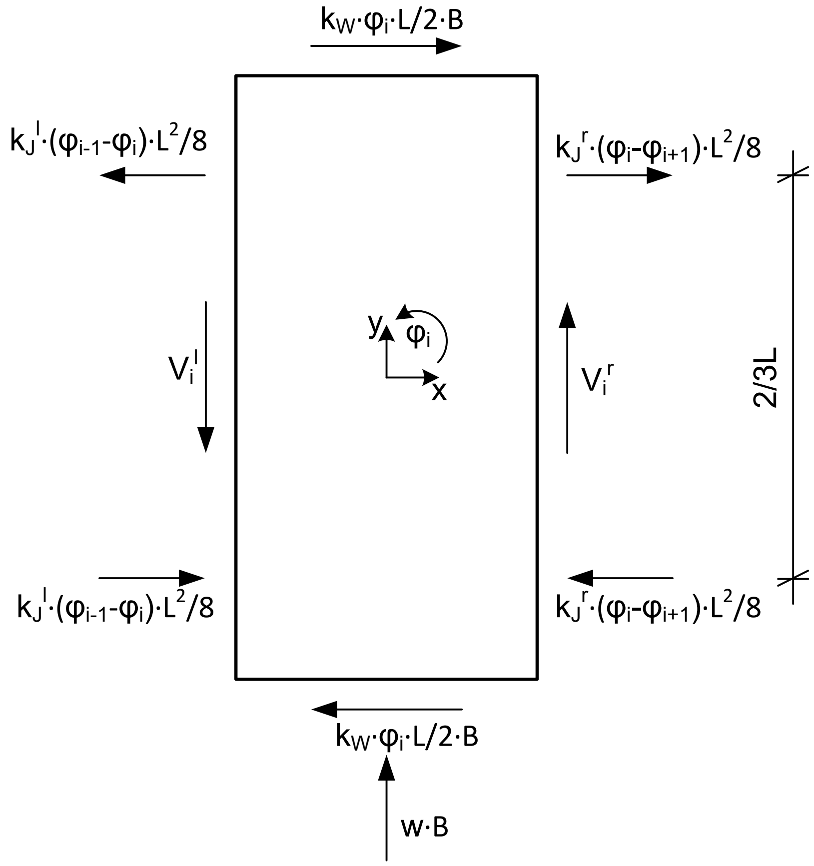

Fig. 7.9: **Resulting forces acting on panel i**

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

$$-k_J^l \cdot (\varphi_i - \varphi_{i-1}) \cdot \frac{L^2}{8} \cdot \frac{2}{3} L - k_J^r \cdot (\varphi_i - \varphi_{i+1}) \cdot \frac{L^2}{8} \cdot \frac{2}{3} L - k_W \cdot \varphi_i \cdot B \cdot \frac{L}{2} \cdot L + (V_i^l + V_i^r) \cdot \frac{B}{2} = 0 \tag{7.9}$$

$$A_i \cdot \varphi_{i-1} + B_i \cdot \varphi_i + C_i \cdot \varphi_{i+1} = D_i \tag{7.10}$$

$$A_i = -\frac{k_J^l \cdot L^3}{12} \tag{7.11}$$

$$B_i = \frac{k_J^l \cdot L^3}{12} + \frac{k_W \cdot B \cdot L^2}{2} + \frac{k_J^r \cdot L^3}{12} \tag{7.12}$$

$$C_i = -\frac{k_J^r \cdot L^3}{12} \tag{7.13}$$

$$D_i = (V_i^l + V_i^r) \cdot \frac{B}{2} \tag{7.14}$$

So for n panels we get n equations. Solving the equation system results in the n unknown rotations φ_i.

With the rotation and the stiffness of the connections the forces of the connections can be determined.

Connections at transverse edges:

$$V = \frac{k_W \cdot \varphi_i \cdot \frac{L}{2} \cdot B}{n} \tag{7.15}$$

n number of fasteners at the transverse edge of a panel

Connections at longitudinal joints:

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

$$V_x = k_J \cdot (\varphi_i - \varphi_{i-1}) \cdot \frac{L}{2} \cdot \frac{L}{n} \tag{7.16}$$

or

$$V_x = k_J \cdot (\varphi_i - \varphi_{i+1}) \cdot \frac{L}{2} \cdot \frac{L}{n} \tag{7.17}$$

L/n distance of fastenings

Longitudinal direction:

$$V_y = \frac{V_i^{l/r}}{n} \tag{7.18}$$

Resulting force:

$$V = \sqrt{V_x^2 + V_y^2} \tag{7.19}$$

If the rotation of each panel is known, also the global displacement of the panels can be calculated. In doing so, the longitudinal displacement of each panel relative to the adjacent panel has to be determined. The global displacement of a panel is determined by summation of the single 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 longitudinal edge and wall. The second part results from the rotation of the panel, which causes an additional displacement in longitudinal direction.

The forces acting in longitudinal direction of the panels result from equilibrium of forces (formulae (7.1) to (7.4)). With this forces and the stiffness of the connections the first part of the displacement of a panel can be calculated. The calculated displacement refers to the adjacent panel or for the outer panels to the wall.

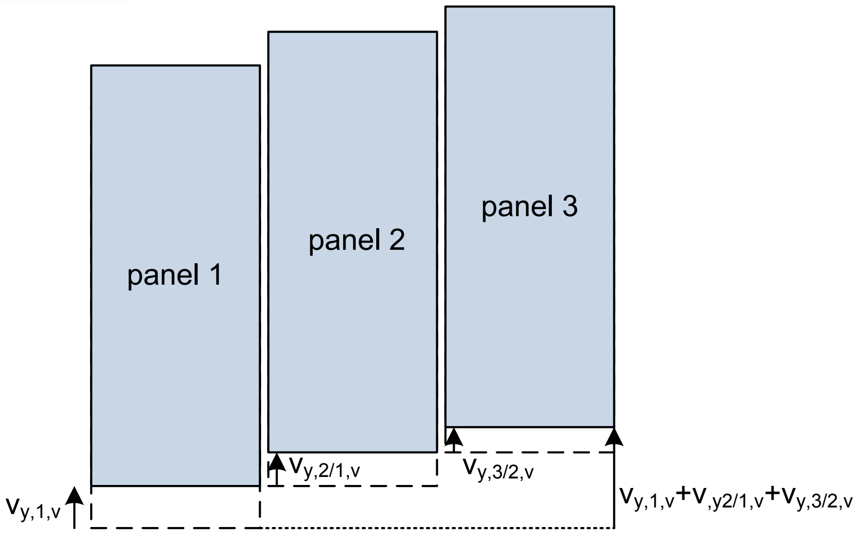

Fig. 7.10: Displacements of panels resulting from forces acting in longitudinal direction

The displacement $v_{y,1,v}$ of the outer panel of a roof is

$$v_{y,1,v} = \frac{V_1^l}{L \cdot k_W} \tag{7.20}$$

The displacement $v_{y,i/i\text{-}1,v}$ of a panel i relative to panel i-1 is

$$v_{y,i/i-1,v} = \frac{V_i^l}{L \cdot k_J} \tag{7.21}$$

The rotation of the panel causes the displacements shown in the following figure.

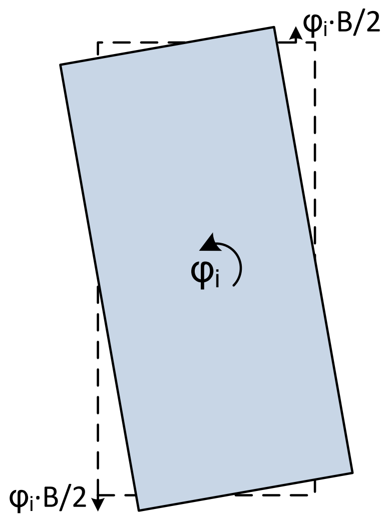

Fig. 7.11: Displacements of panels resulting from rotation

Because summation of displacements starts at the left edge of the roof, we first consider the left edge of the panel. There the rotation causes a force in opposite direction to the force resulting from equilibrium. So there must be an additional displacement in longitudinal direction to keep equilibrium of forces ($\varphi_i \cdot B/2$).

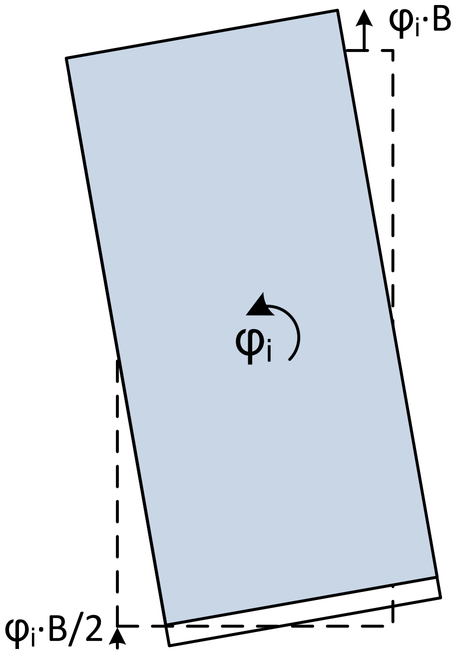

Fig. 7.12: Displacements of panels resulting from rotation

So at the opposite (right) edge of the panel we get the following longitudinal displacement, which results from rotation.

$$v_{y,i,\varphi} = 2 \cdot \varphi_i \cdot \frac{B}{2} = B \cdot \varphi_i \qquad (7.22)$$

The global displacement of a panel is determined by summation of the displacements $v_{y,i,v}$ and $v_{y,i,\varphi}$ over the panels, starting at panel 1. E.g. for a roof consisting of six panels the highest global displacement (right edge of panel 3) is calculated as follows.

$$v_{y,3} = \frac{V_1^l}{L \cdot k_W} + \frac{V_2^l}{L \cdot k_J} + \frac{V_3^l}{L \cdot k_J} + B \cdot \varphi_1 + B \cdot \varphi_2 + B \cdot \varphi_3 \qquad (7.23)$$

8 Walls without connections at the longitudinal joints

To transfer horizontal wind loads for wall panels a connection of the longitudinal joints is not necessarily required. In this case a panel is not influenced by adjacent panels; each panel acts as a single element. At the upper end horizontal in-plane shear forces are introduced from the roof in the wall panel. The wall panel is fixed to the foundation. So 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. 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.

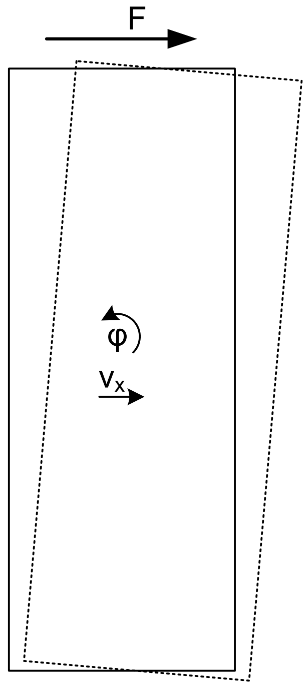

Fig. 8.1: Displacement and rotation of a wall panel

This kind of loading was also investigated in [16]. At one end sandwich panels were fixed to a substructure with two self-tapping screws. At the other end a force in transverse direction to the span of the panel was introduced. The fastenings introduce a transverse force into the substructure. In addition a couple of longitudinal forces occurs, which counteracts the moment resulting from introduced force and lever arm.

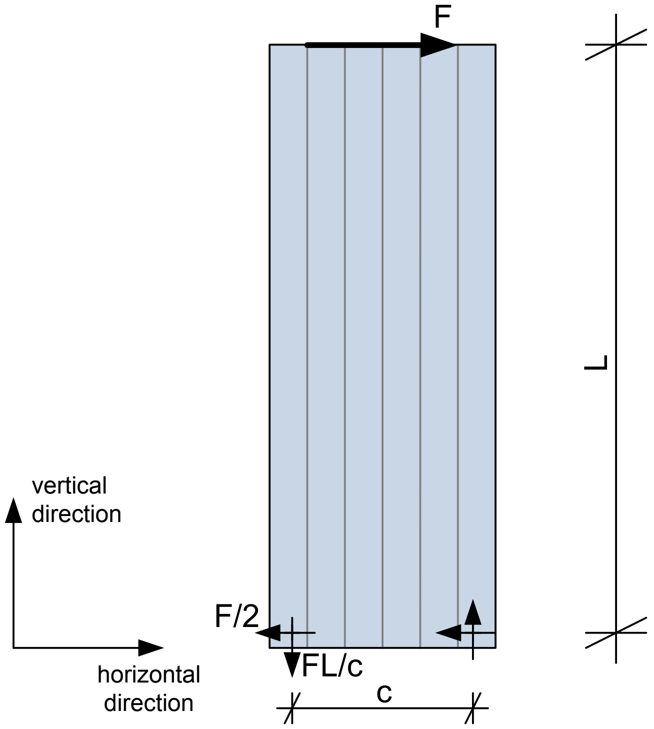

Fig. 8.2: **Forces at an in-plane shear loaded wall panel**

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

$$V_y = \frac{F \cdot L}{c + \frac{c_1^2}{c} + \frac{c_2^2}{c} + \ldots\ldots} \tag{8.1}$$

with

c_i distance between pair of connections

c distance between outer connections

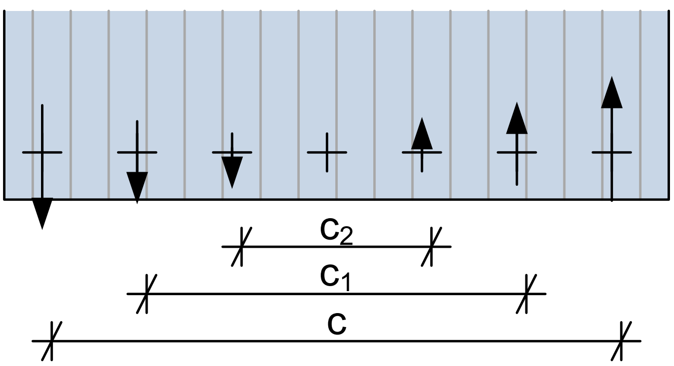

Fig. 8.3: **Longitudinal forces at a wall panel**

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

$$V_x = \frac{F}{n} \tag{8.2}$$

n number of connections at the transverse edge of a panel

The resulting force of a connection is determined by vectorial addition

$$V = \sqrt{(V_x)^2 + (V_y)^2} \tag{8.3}$$

Alternatively for simplification the forces in longitudinal direction can be determined for smeared connections. The stiffness k_v of the connections at the transverse edge is transformed to the continuous stiffness k_W in [kN/mm^2] (see formula (6.1)). A rotation of the panel causes a linearly distributed force f_y [kN/m] at the connections between wall and foundation. The moment resulting from the force f_y is

$$M = f_{y,\max} \cdot \frac{B}{4} \cdot \frac{2}{3} B = f_{y,\max} \cdot \frac{B^2}{6} \tag{8.4}$$

with

$f_{y,max}$ maximum value of f_y at the corners of the panel

The maximum value of the longitudinal force is determined by equalisation of moments.

$$F \cdot L = f_{y,\max} \cdot \frac{B^2}{6} \tag{8.5}$$

$$f_{y,\max} = \frac{6 \cdot F \cdot L}{B^2} \tag{8.6}$$

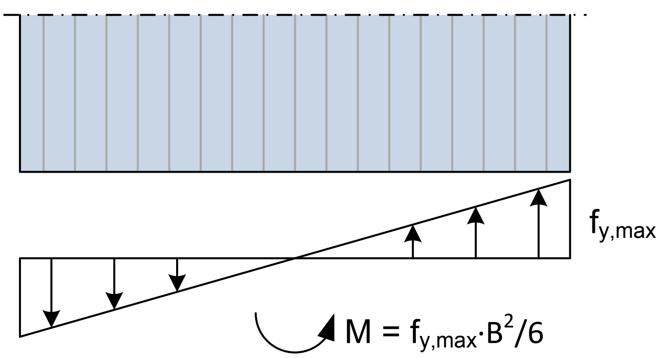

Fig. 8.4: Longitudinal forces at a wall panel (smeared fastenings)

The force of a connection is determined by integrating the distributed force f_y by the corresponding width, i.e. by the distance of the connections. So the longitudinal force of the most stressed connection is

$$V_y = \frac{6 \cdot F \cdot L}{B^2} \cdot \frac{B}{n} \tag{8.7}$$

B/n distance of connections

The rotation of the panel is determined using the condition that at the corners of the panel the vertical force $f_{y,max}$ must occur. The vertical displacement at the corner is

$$v_{y,\max} = \varphi \cdot \frac{B}{2} \tag{8.8}$$

The corresponding force $f_{y,max}$ is determined by multiplying the stiffness k_W of the fastening by the displacement.

$$f_{y,\max} = v_{y,\max} \cdot k_W = \varphi \cdot \frac{B}{2} \cdot k_W \tag{8.9}$$

By equalisation of (8.6) and (8.10) the rotation φ is determined.

$$f_{y,\max} = \varphi \cdot \frac{B}{2} \cdot k_W = \frac{6 \cdot F \cdot L}{B^2} \tag{8.10}$$

$$\varphi = \frac{12 \cdot F \cdot L}{B^3 \cdot k_W} \tag{8.11}$$

The transverse displacement of the panel consists of two parts, the displacement caused by the transverse force and the displacement caused by the rotation.
The displacement caused by the transverse force depends on the stiffness of the connection between panel and foundation.

$$v_{x,v} = \frac{F}{B \cdot k_W} \tag{8.12}$$

The rotation causes the following displacement, which occurs at the upper end of the panel.

$$v_{x,\varphi} = \varphi \cdot L \tag{8.13}$$

So the maximum transverse displacement of a panel is

$$v_x = \frac{F}{B \cdot k_W} + \varphi \cdot L \tag{8.14}$$

9 Walls with additional connections at the longitudinal joints

If the longitudinal joints of wall panels are connected, the stiffness and also the load-bearing capacity increase. In this case also for simple configurations an analytical determination of forces 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.
In the following the general load bearing behaviour is shown for the exemplary wall presented in Fig. 9.1. The wall consists of six panels with a length of 3000 mm. Each panel is connected to the foundation with for connections (stiffness 5 kN/m). There are 12 connections at each longitudinal joint (stiffness 10 kN/m). Each panel is loaded by a horizontal load of 1000 N.

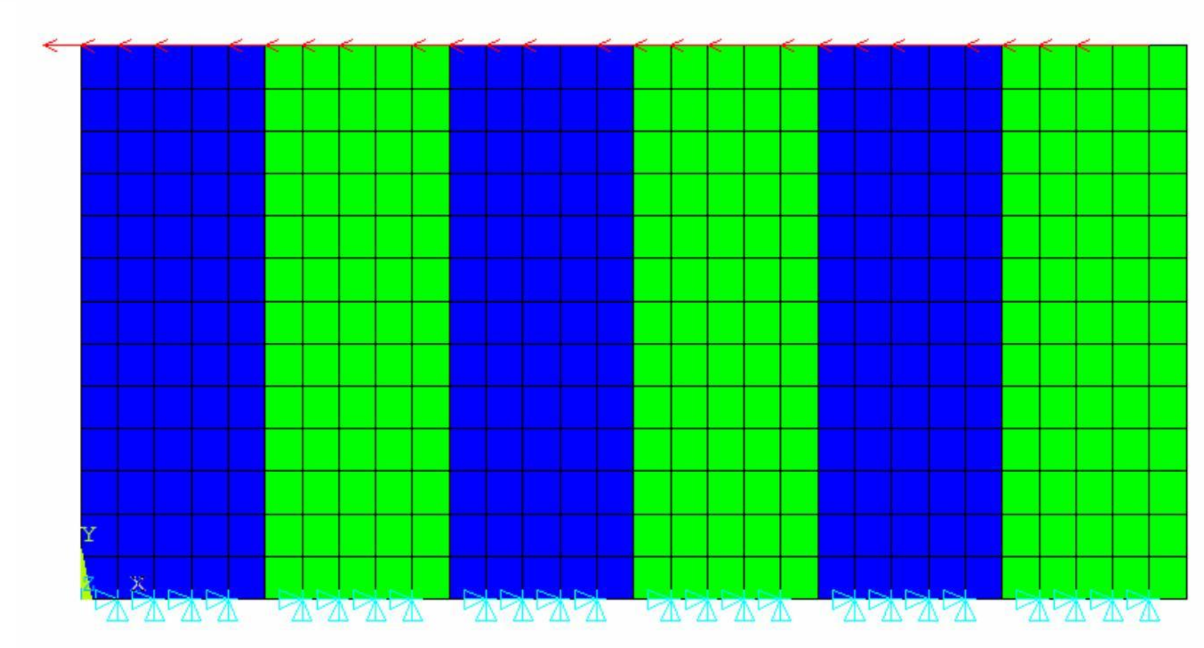

Fig. 9.1: Model of a wall for numerical calculation

A displacement in direction of the horizontal load and a rotation occur (Fig. 9.2).

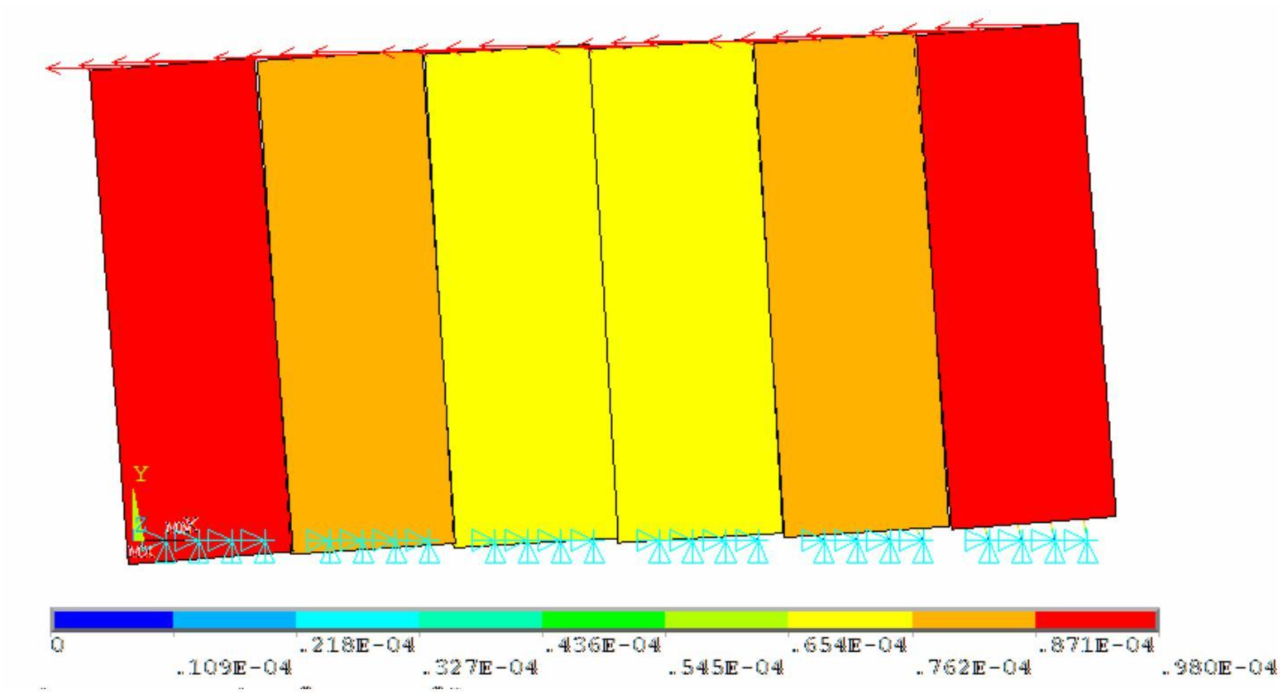

Fig. 9.2: Rotation of wall panels

Because the panels do not act as independent single elements, the centre of rotation of a panel is not located in the centre of its lower transverse edge. Thus the load is not distributed uniformly to all panels. In the following figure the forces of the connections between panels and foundation are given. The forces in transverse direction (i.e. in direction of the horizontal load) are constant for all connections of a panel. The highest transverse forces act at the inner panels of a wall. The distribution of forces acting in longitudinal direction is linear within one panel. The highest rotation occurs at the outer panels of a wall. For these panels also the distance of the centre of rotation to the centre of the panel has the highest value. So the highest forces in longitudinal direction occur at the outer panels.

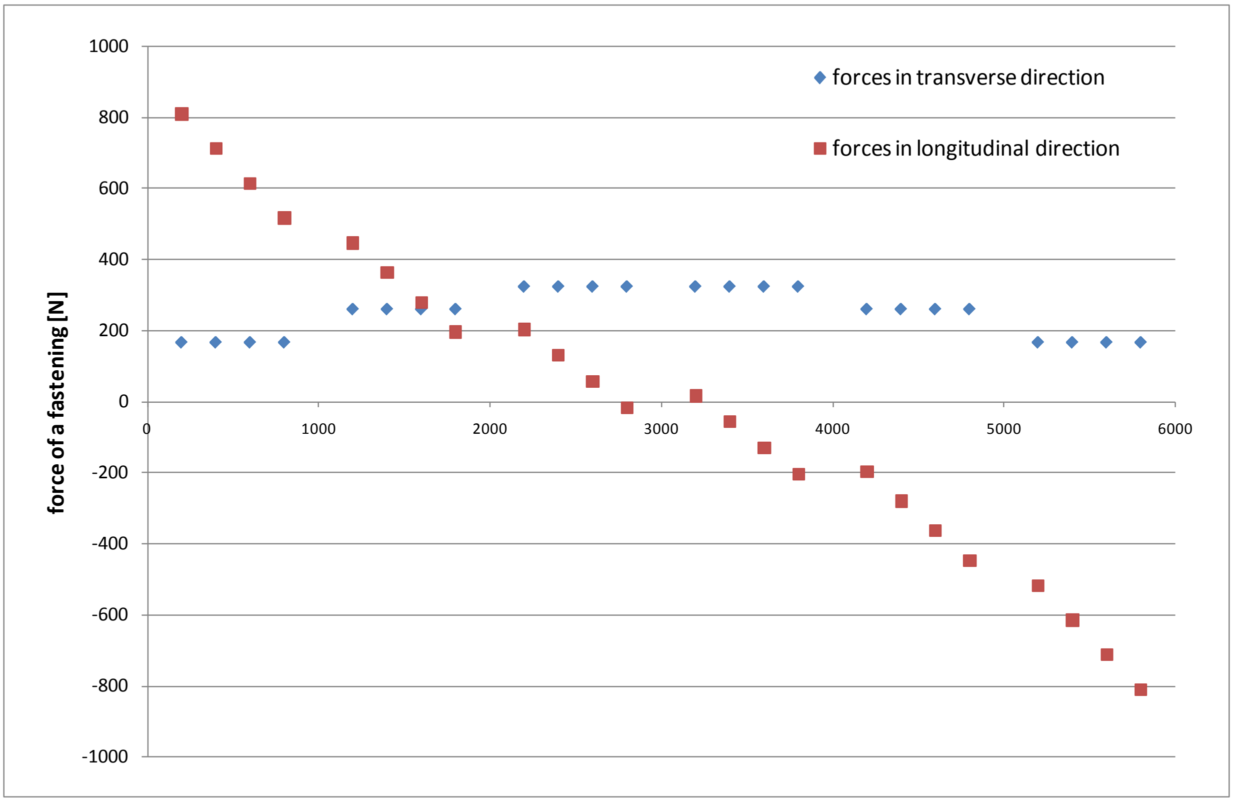

Fig. 9.3: Forces in the connections between wall and foundation

All connections of a longitudinal joint are loaded by the same longitudinal force. The highest longitudinal force occurs in the joints between the inner panels of a wall.

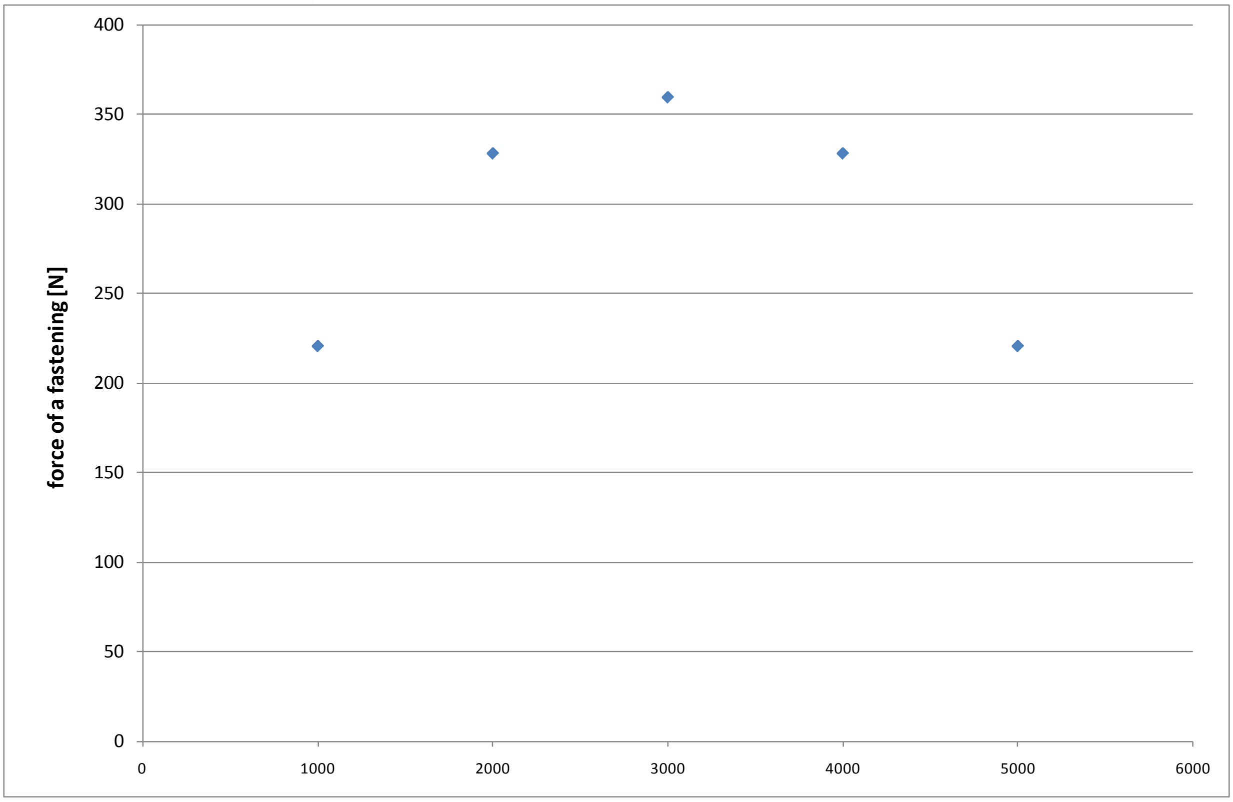

Fig. 9.4: Longitudinal forces of connections at joints

The forces acting in transverse direction are linearly distributed to the connections of a joint. The highest forces occur in the outer joints of a wall.

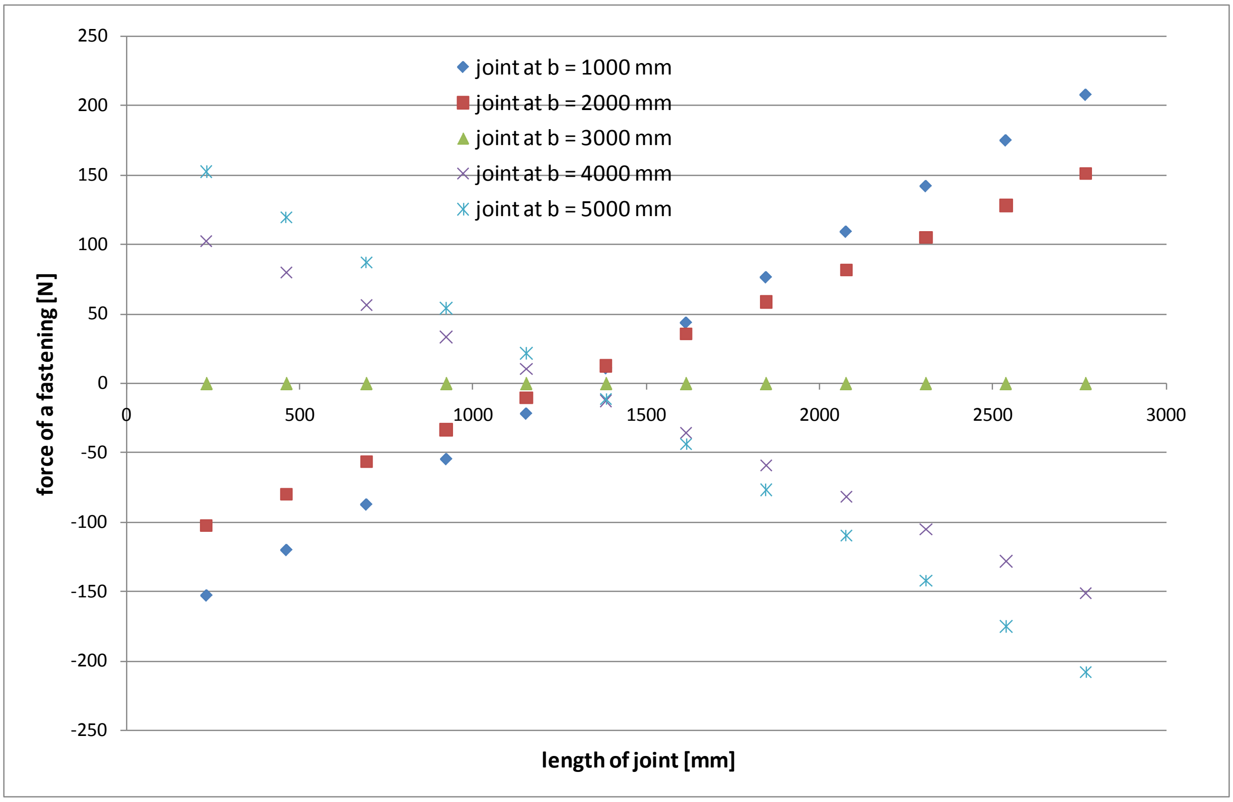

Fig. 9.5: Transverse forces at connections of joints

For both extreme examples – no connections and rigid connections at longitudinal joints – there is a constant distribution of the horizontal load to all panels of the wall. To demonstrate the influence of the stiffness of the connections of the joints, in the following the forces of the connections at the transverse edge are presented for different stiffness. Apart from the stiffness of the connections at longitudinal joints the wall introduced above was used for the calculations. In the following figure the horizontal forces acting at the connections between panel and foundation are shown for several stiffness's of the connections at the longitudinal joints.

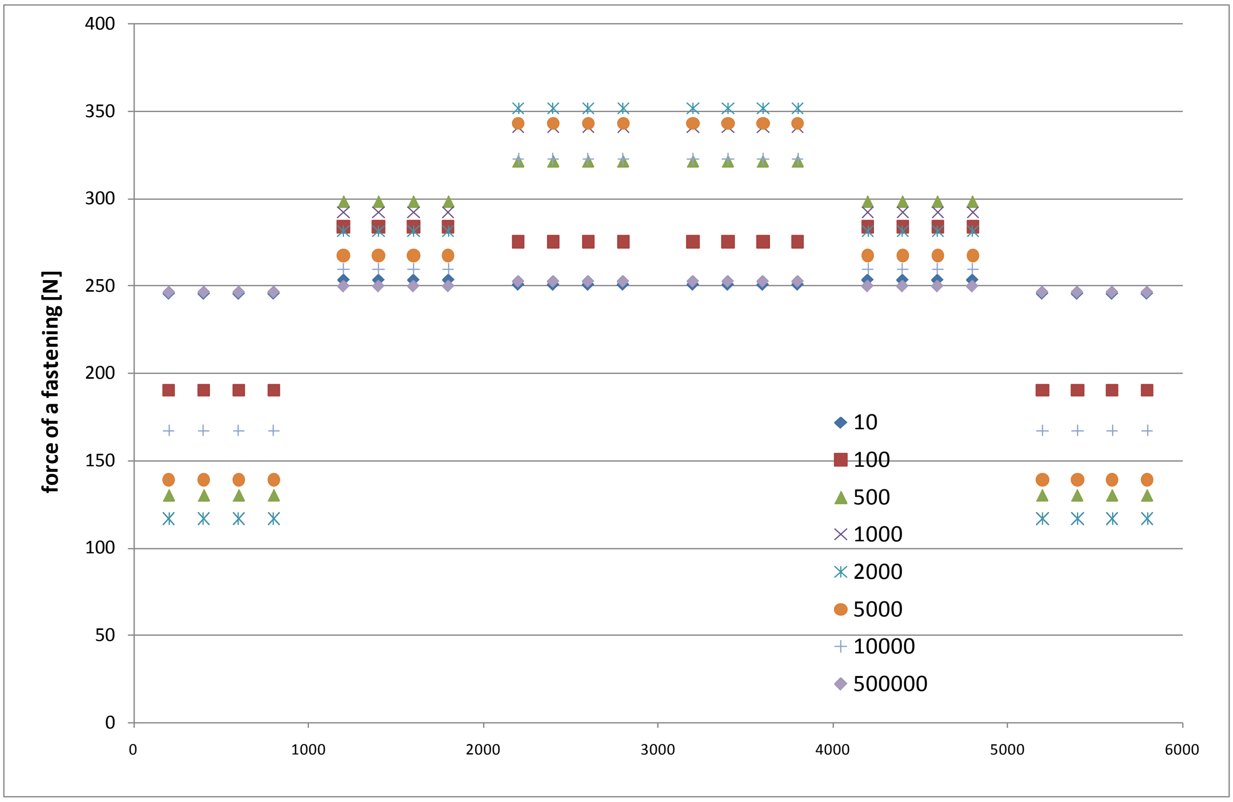

Fig. 9.6: Forces of connections between wall and foundation in dependence of stiffness

With increasing stiffness first the difference of the forces of inner and outer panels increases. For a further increase of the stiffness the difference decreases. For approximately rigid connections the quotient of forces of inner and outer panel approaches 1.

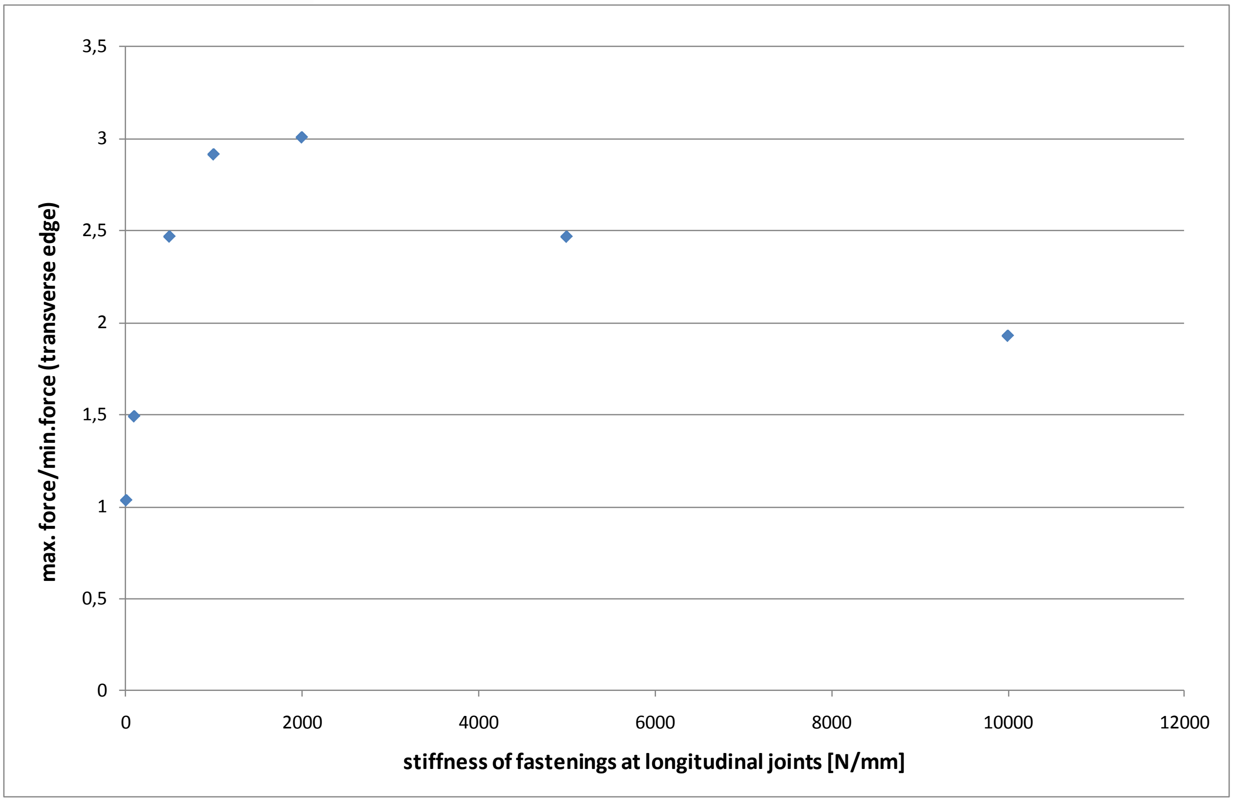

Fig. 9.7: Relation between stiffness of connections and forces

The forces acting in longitudinal direction of the wall panels decrease with increasing stiffness of the connections at the longitudinal joints. In the following figure the forces at the outer panel of the wall are shown for different stiffness.

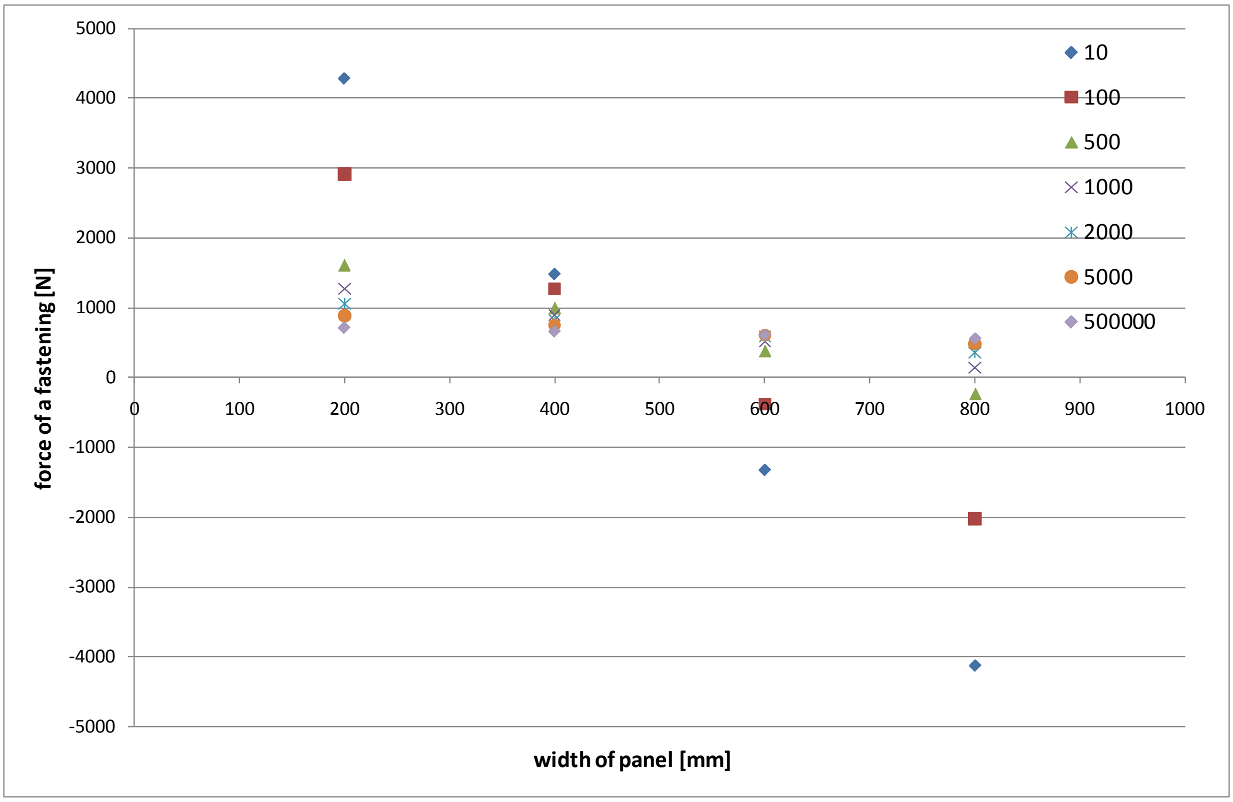

Fig. 9.8: **Relation between stiffness of connections and force**

Furthermore there are also cases, where the panels of a wall are not loaded by a constant force. If e.g. the horizontal load acts in longitudinal direction of the roof panels, only into the panels of the walls being parallel to the direction of load constant shear forces are introduced. At the walls being orthogonal to the direction of load the forces are linearly distributed with a change of direction of force in the centre of the wall.

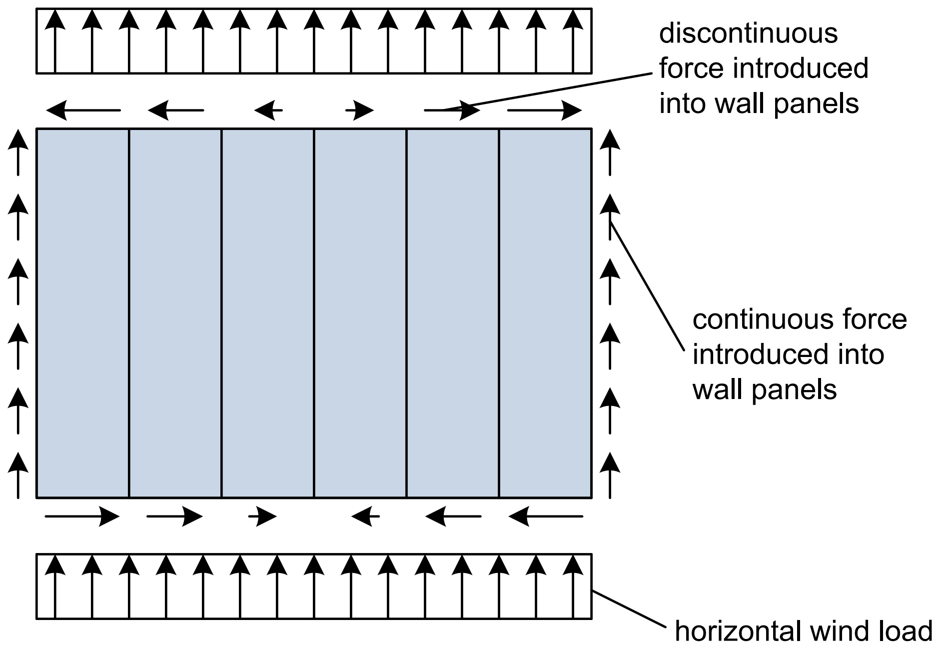

Fig. 9.9: **Forces introduces from roof into wall panels**

In the following the wall already introduced above is considered. The forces of the fastenings occurring for this kind of loading are presented. The forces introduced at the upper end of the panels are assumed to be 1760 N, 1200 N, 420 N, -420N, -1200 N, -1760 N (Fig. 9.10).

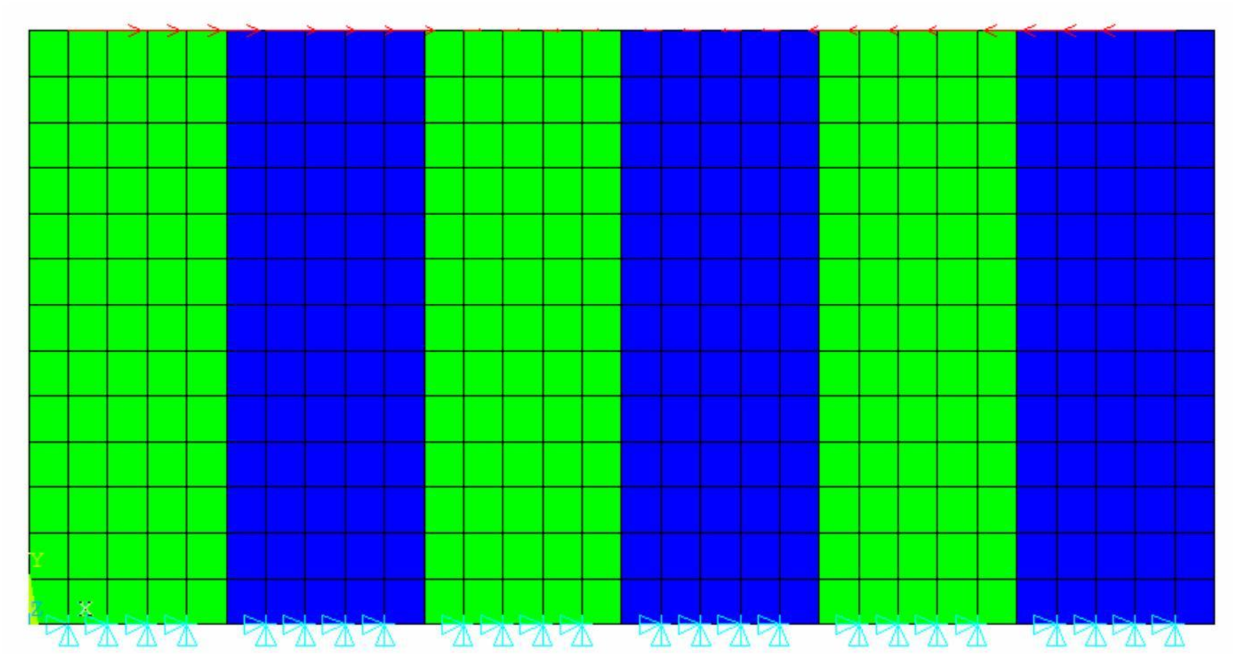

Fig. 9.10: FE-model of a wall

The load given above causes displacements as well as rotations as shown in Fig. 9.11.

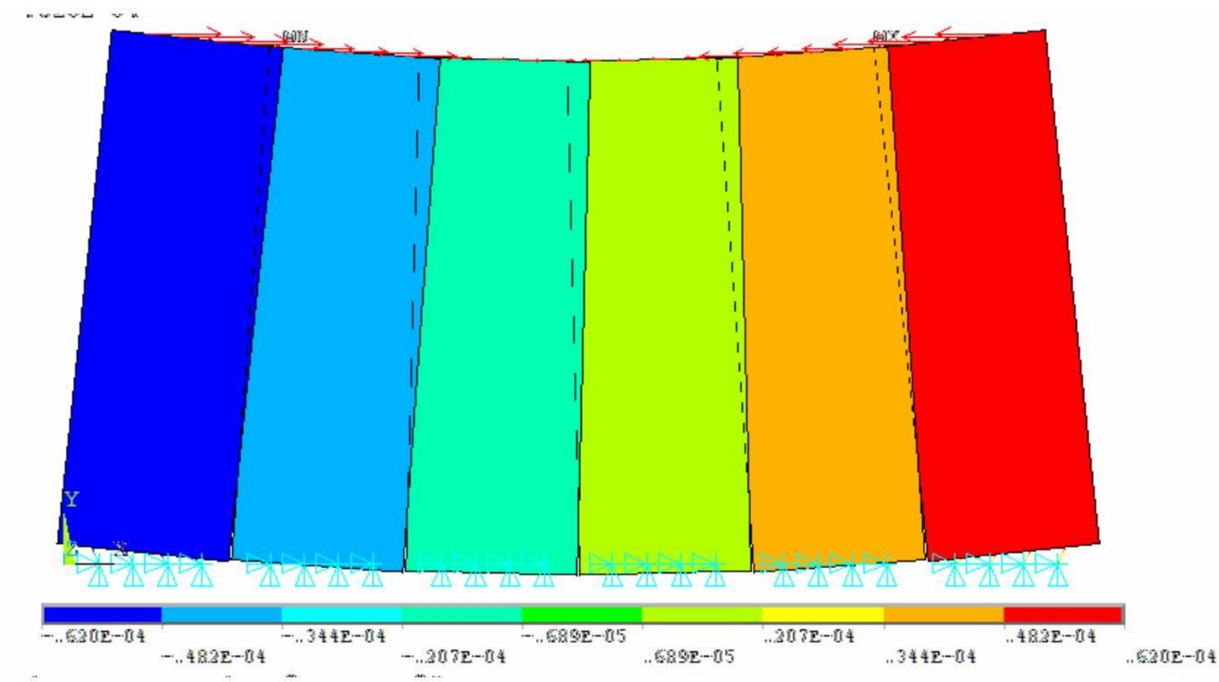

Fig. 9.11: Rotation of panels

In the following figure the forces in the connections between wall and foundation are shown. All connections of one panel are loaded by the same transverse force, whereas the longitudinal force is distributed linearly within one panel. The highest transverse force as well as the highest longitudinal force acts at the outer panels.

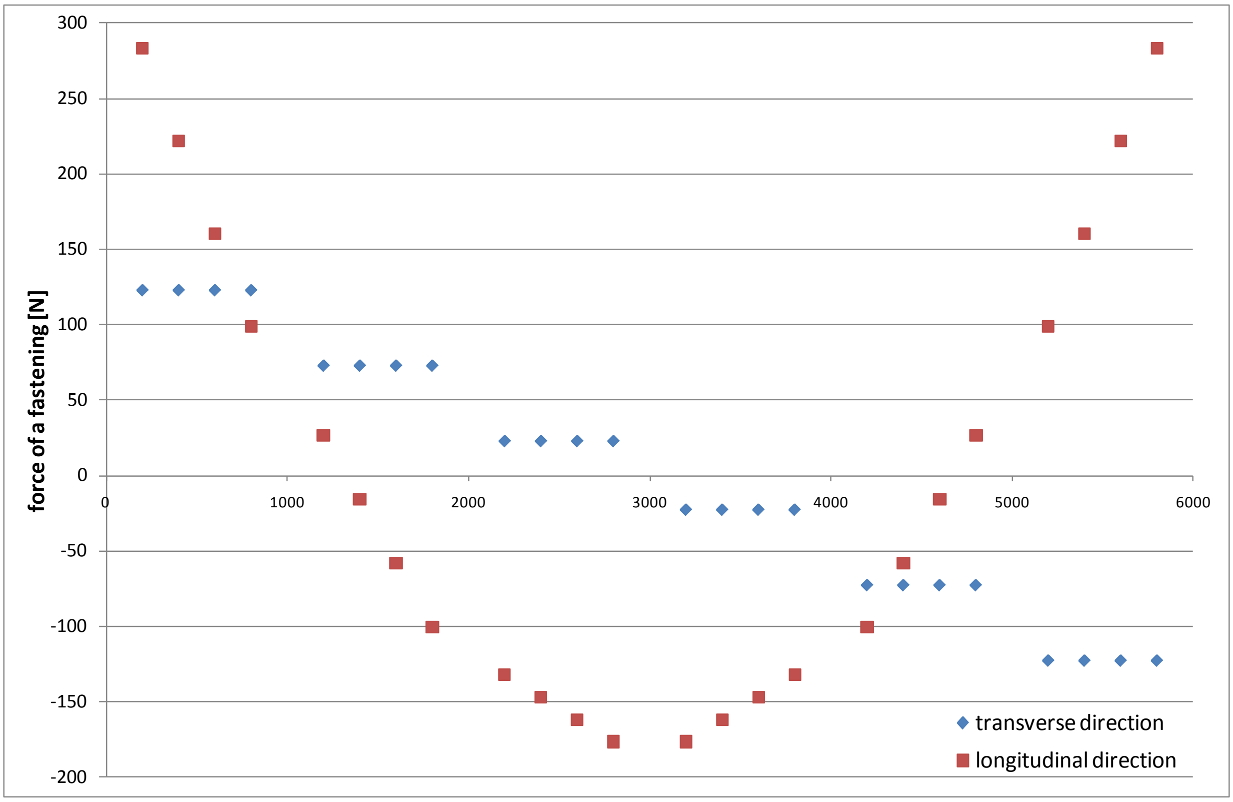

Fig. 9.12: **Forces at connections between wall and foundation**

Fig. 9.13 shows the distribution of transverse forces for the connections of the joints. The highest force occurs at the joint between the inner panels.

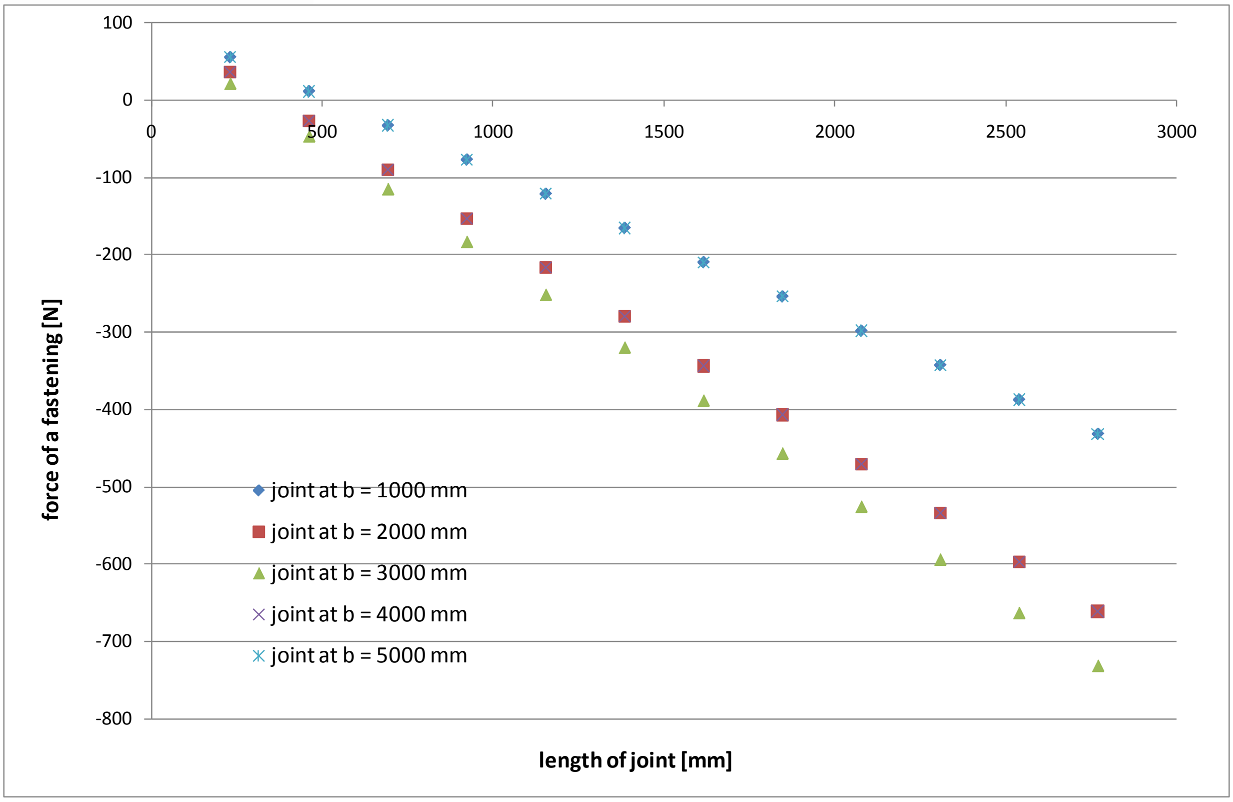

Fig. 9.13: Transverse forces at connections of joints

10 Design of shear loaded walls by testing

ETAG 21 [10] deals also with the design of shear loaded walls made of sandwich panels. A procedure for design by testing is given in Annex D of ETAG 21. If in practice the joints of the panels are connected at least two panels have to be tested in a test. If there are no connections at the joints, it is sufficient to test only one panel. The panels have to be fixed to the foundation as in practice. During the test the deformation v has to be recorded. The deformation v is the difference between the displacement at point A and B (Fig. 10.1).

First a vertical load F_v is applied to the tested wall. This load is maintained constant during the test. The horizontal load F is applied in three load cycles.

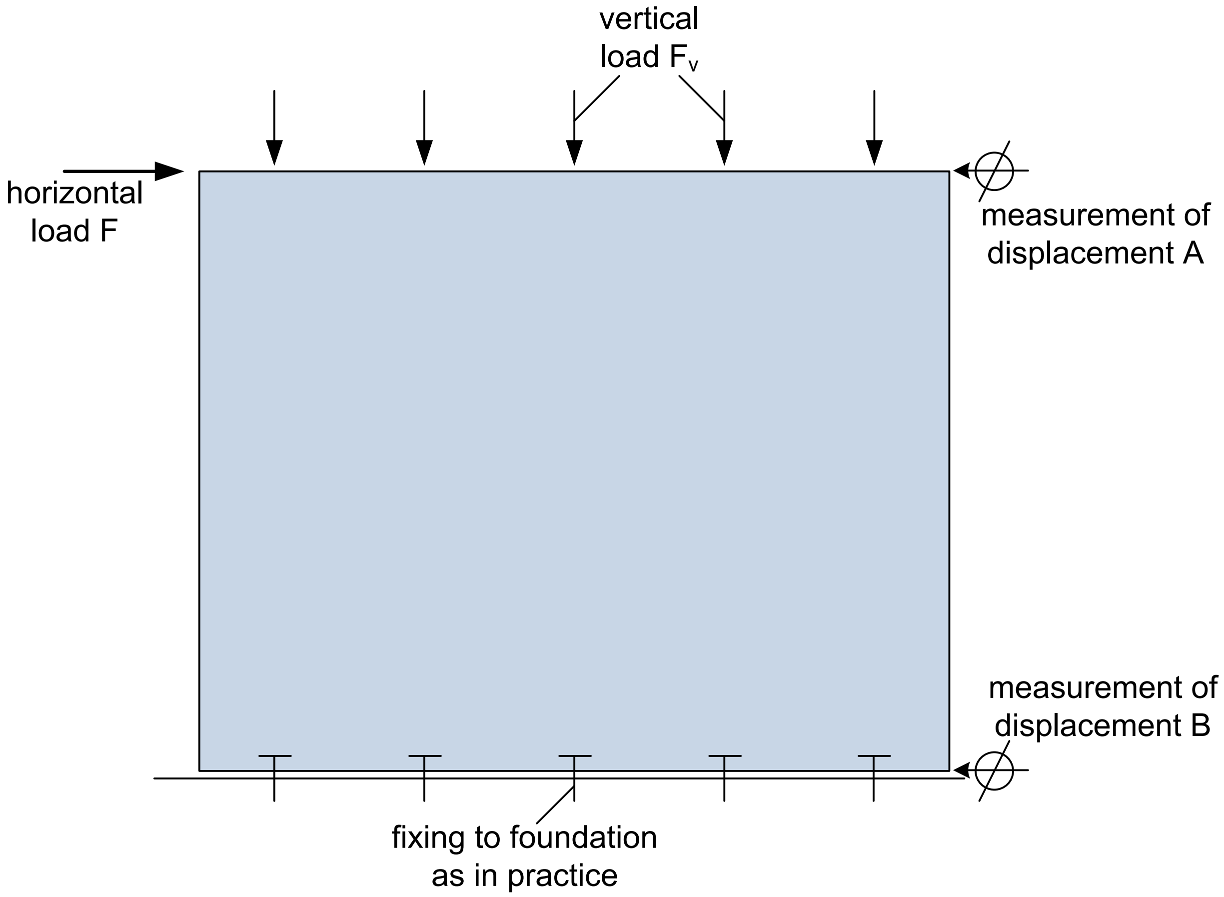

Fig. 10.1: Test set-up according to ETAG 21

- Stabilising load cycle

The horizontal load is increase to $0{,}1 \cdot F_{max,est}$ and maintained for 120 s. After that the horizontal load is removed and a recovery period of 600 s follows. $F_{max,est}$ is the estimated maximum horizontal load.

- Stiffness load cycle

The horizontal load is increased to $0{,}4 \cdot F_{max,est}$ and maintained for 300 s. The load is removed and a recovering period of 600 s follows.

- Strength test

The horizontal load is increased to $0{,}4 \cdot F_{max,est}$ and maintained for 300 s. The load is increased until F_{max} is reached. The maximum load F_{max} is reached, when the panel collapses or the deformation v exceeds 100 mm.

The tests are statistically evaluated according to EN 1990 [5].

The stiffness of the wall is calculated by the following formula.

$$R = 0{,}5 \cdot \left(\frac{F_4 - F_1}{v_{04} - v_{01}} + \frac{F_{24} - F_{21}}{v_{24} - v_{21}} \right) \quad (10.1)$$

with

$$F_1 = 0{,}1 \cdot F_{\max,est}$$

v_{01} displacement corresponding to F_1 determined in stiffness load cycle

$F_4 = 0{,}4 \cdot F_{\max,est}$

v_{04} displacement corresponding to F_4 determined in stiffness load cycle

$F_{21} = 0{,}1 \cdot F_{\max,est}$

v_{21} displacement corresponding to F_{21} determined in strength test

$F_{24} = 0{,}4 \cdot F_{\max,est}$

v_{24} displacement corresponding to F_{24} determined in strength test

The design strength is calculated by dividing the characteristic strength by a factor γ_{rs}.

$$\gamma_{rs} = 1{,}6 \cdot \gamma_M \qquad (10.2)$$

γ_M material factor of Eurocode

11 Summery

The common application of sandwich panels is enclosures of buildings. A new application is to use sandwich panels for frameless structures, i.e. the panels are applied without any load transferring substructure. In this new type of application the sandwich panels have to transfer loads and to stabilise the building. To transfer horizontal wind loads the very high in-plane shear stiffness and resistance of the panel is used. Because the in-plane shear stiffness of the panels is very much higher than the stiffness of the fastenings, only the fastenings are decisive for load-bearing behaviour and capacity. So the fastenings have to be designed for the forces resulting from transfer of horizontal loads.
In the report at hand basics of the transfer of horizontal loads in frameless buildings are presented. Basic principles for determination of the forces the fastenings have to be designed for are given.

12 References

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