Missouri University of Science and Technology
Scholars' Mine

International Specialty Conference on Cold-Formed Steel Structures

(1996) - 13th International Specialty Conference on Cold-Formed Steel Structures

Oct 17th, 12:00 AM

Design Models of Continuous Sandwich Panels

Paavo Hassinen

Lassi Martikainen

Thirteenth International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri U.S.A., October 17-18, 1996

DESIGN MODELS OF CONTINUOUS SANDWICH PANELS

Paavo Hassinen [1] & Lassi Martikainen [2]

Abstract

Static behaviour and failure modes of single-span lightweight sandwich panels are well known in general and several design guidelines and recommendations have been published concerning the determination of the resistance values for the design. At intermediate supports continuous sandwich panels are loaded by high bending moments and transverse support reactions simultaneously, which leads to nonlinear interactions between the stresses caused by the bending moment and support reaction. The interaction failure modes have not been introduced on acceptable level in the current recommendations. The paper presents experimental and analytical results concerning the static behaviour and strength of the intermediate support area and further, makes proposals of new calculation models for the serviceability limit state design and reports important findings for the ultimate limit state design of continuous sandwich panels.

1. Introduction

Sandwich panels with steel sheet faces and a plastic foam or mineral wool core are used to cover walls and roofs of buildings but also to build up ceilings and to depart spaces inside buildings. Sandwich panels are industrially produced building components, which from the static point of view can in most cases be classified to be beam structures. Continuous multi-span sandwich beams are used to span roofs from the ridge to the eaves and walls from the eaves to the foundations to escape transverse joints between the panels, which may be risks for water- and air-tightness of the structure. In the design additional criteria have to be set up for the intermediate support area to take into account the combinations of high bending moments, shear forces and support reactions.

In flat steel sheet faced sandwich panels the global bending moments cause axial compressive and tensile stresses in the faces. The shear forces yield in shear stresses mainly in the core layer. At intermediate supports, the support reactions cause transverse loads to the sandwich structure, which results in local bending stresses in the face layer placed against the support structure, and further, local compressive and shear stresses in the core layer. The axial compressive stresses in the face caused by the global bending moments increase the local stresses in the face and core because of the geometrically nonlinear interaction. In addition to that, also the non-elastic stress-strain behaviour of the face and core materials influence on the local and global resistance of the sandwich structure. The first failure in the face or in the core layer caused by the combination of bending moment, shear force and support reaction establish the criterion for the serviceability limit state. The remaining bending, shear and support reaction resistances of the cross-section after the first failure effect on the ultimate limit load.

Two different loading cases have to be separated for the analysis and design of multi-span sandwich panels. In this work the cases are named positive and negative support reactions and they indicate the direction of the support reaction at the intermediate support. Also the sign of the global bending moment changes according to the direction of the support reaction. Positive support reactions cause compressive contact

[1] Laboratory manager, Helsinki University of Technology, Laboratory of Structural Mechanics
[2] Research scientist, Helsinki University of Technology, Laboratory of Structural Mechanics

stresses between the support structure and the sandwich panel. In this case the face placed against the support structure is loaded by compressive axial stresses. Negative support reactions cause tensile stresses in the fasteners between the support structure and the sandwich panel, in which case the outer face is loaded by axial compressive stresses but also by transverse loads caused by the heads of the fasteners.

According to the assumptions made for the analysis, sandwich panels are divided into thin faced and thick faced sandwich panels. The faces of thin faced sandwich panels are flat or lightly profiled and the flexural stiffness of the faces has no influence on the global bending moment and shear force distributions. The faces of thick faced panels are usually strongly profiled and their flexural stiffness has to be taken into account in the analysis of global stress distributions in the face and core layers. When investigating the local behaviour at the supports, even the small flexural stiffness of the face layer has effects on the stress distributions and strength. In this paper the faces are assumed to have negligible flexural stiffness in the analysis of global stress resultants but the finite stiffness is taken into account in evaluations of the local stresses and resistances at the intermediate supports.

2. Failure modes at intermediate supports

In the previous works of the authors, derivations of analytical expressions have been presented for the design of sandwich panels at intermediate supports */Hassinen & Martikainen 1994, 1995/*. Those formulations have resulted in a set of expressions, which cover the three failure modes for the serviceability limit state design; shear failure of the core (Eq. 1), crushing failure of the core (Eq. 2) and bending and buckling failure of the face (Eq. 3), (Fig 1).

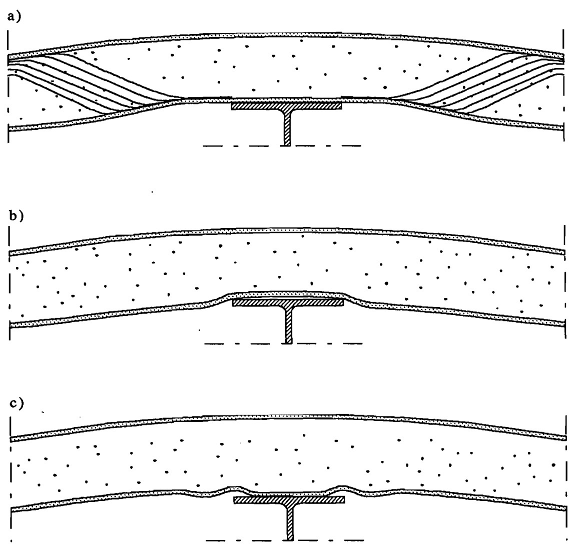

Fig. 1. Failure modes of continuous sandwich panels at intermediate supports; a) shear failure, b) core crushing failure and c) face bending and buckling failure.

$$\tau_C \leq f_{Cv} \tag{1}$$

$$\sigma_{Cc} \leq f_{Cc} \tag{2}$$

$$\sigma_{f2} = \sigma_{S2} + \sigma_{f2R} \leq f_y \qquad \text{and} \qquad \sigma_{S2} \leq f_{fc} \tag{3a,b}$$

In the equations, the left sides of the expressions represent the design stresses, which are the stress values caused by the characteristic loads multiplied by the partial load factors. The right sides of the expressions represent the strengths and resistances being the corresponding characteristic values divided by the material safety factors. In this paper the design philosophy and safety factors are not studied and, therefore, the safety factors are not either included in the given expressions.

The shear failure (Eq. 1) is assumed to be a separate failure mode without any interactions with bending and support reaction resistances. The shear failure of the most core materials is a brittle-type failure mode leading to total collapse of the sandwich panel. The experimental observations made in this research show, that the core layers made of structural rock wool do not fail in totally brittle way in shear, but have also some plastic-type shear resistance. This fact may change some of the basic assumptions in the design of multi-span sandwich panels. However, this observation may not be valid to all types of wool cored sandwich panels. The shear failure has not been studied in greater detail in this work.

In the conventional design the first plastic strains in the structure define the criterion to the serviceability limit state load. To accomplish an accurate design, the values of the both sides of the design equations (Eqs. 1-3) have to be evaluated as exactly as possible. In this work models to evaluate the stresses (left side) at intermediate supports are investigated in Chapters 3.1 and 4. The models for the strengths and resistances (right side) of the two last failure modes (Eqs. 1b,c) are given in Chapters 3.2 and 4.2.

The first failure mode of a multi-span sandwich panel does not always determine the load-bearing capacity of the structure, but the panel is able to carry more load until the final collapse. If the failure mode is not a brittle shear failure, generally a plastic hinge with zero bending resistance is assumed to turn into the structure at the intermediate support, and the continuous sandwich panel is assumed to change to a series of simply supported beams. However, the failed cross-section at the intermediate support has some remaining plastic bending resistance, which has effects on the bending moments and shear forces in the span and, thus, also on the ultimate limit load.

The influence of the remaining bending resistance on the load-bearing capacity of a sandwich panel can be described by the plastic postbuckling strength of the compressed face layer at the intermediate support. Based on the equilibrium of the loads an example has been calculated to show the additional load which could be carried through by a sandwich panel (Fig. 2). The plastic compression strength of 30 MPa represents the level of remaining compression strength values observed in the experiments with full-scale two-span sandwich panels */Martikainen & Hassinen 1996/.*

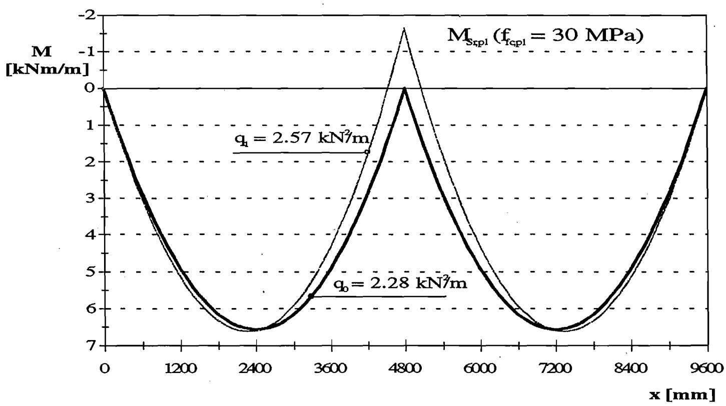

Fig. 2. Example depicting the additional load carried through by a two-span sandwich panel with a remaining bending resistance at the central support. Depth of the sandwich panel is 100 mm, thickness of the faces 0.55 mm, compression strength of the face in the span 120 MPa and the remaining plastic compression strength at the central support 30 MPa.

3. Positive support reaction

3.1 Interaction between bending moment and support reaction

Stress resultants at an intermediate support of a continuous thin-faced sandwich beam consist of a bending moment, shear force and support reaction (Fig. 3a). At the support the lower face can be described by a beam-column model, which is loaded by a compressive axial force N_S and a support pressure q(x) and stabilized by the load p(x) caused by the core layer (Fig. 3b). The shear force is carried by the core layer alone. If the support beam is symmetric and its cross-section does not distort when loaded by the support pressure, the support reaction can be assumed to consist of two line loads, R/2, located at the edges of the support beam (Fig. 3c). Based on the above assumptions the design equations (2) and (3a) lead finally in the following two expressions giving the serviceability limit state design criteria for the face and core layers */Hassinen & Martikainen 1995/.*

$$\alpha \frac{\sigma_{S2}}{\sigma_{w,2}} + \frac{R}{R_R} \frac{1}{\sqrt{1 - \frac{\sigma_{S2}}{\sigma_{w,2}}}} \leq 1 \quad \text{and} \quad \delta \frac{R}{R_R} \frac{1}{\sqrt{1 - \frac{\sigma_{S2}}{\sigma_{w,2}}}} \leq 1 \qquad (4, 5)$$

$$\text{where } \alpha = \frac{\sigma_{w,2}}{f_y} \quad \text{and} \quad \delta = \frac{R_R}{R_C} \qquad (6a,b)$$

Parameters R_R and R_C represent the support reaction resistances determined by the face and core layers and $\sigma_{w,2}$ the ideal wrinkling stress of a thin face layer based on the two-parameter foundation model. σ_{S2} and R are the axial compressive stress in the lower face and the support reaction. The numerical values of R_R , R_C and $\sigma_{w,2}$ depend on the values of the two foundation coefficients, k_W and k_l representing the transverse compression and shear stiffness of the core layer.

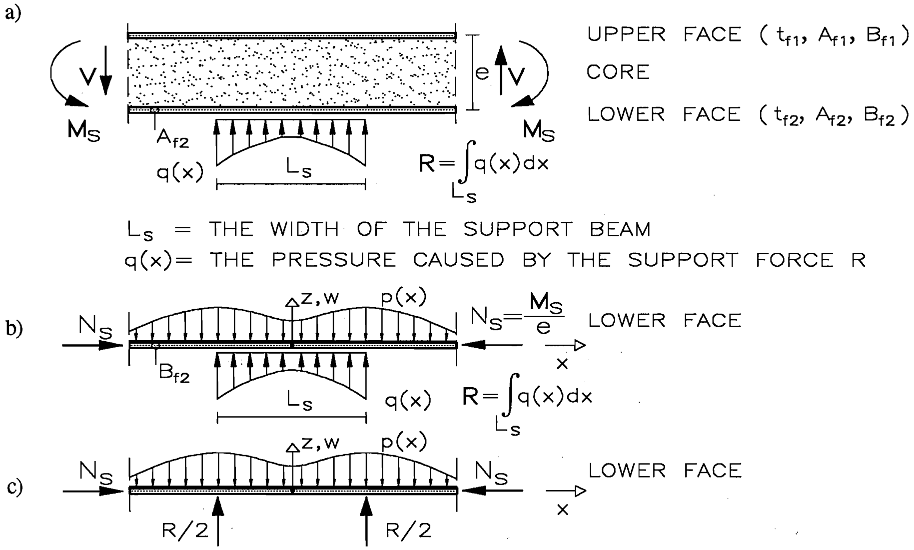

Fig. 3. Modeling the intermediate support area for the serviceability limit state design: a) Stress resultants at symmetric intermediate support, b) beam-column model describing the behaviour of the lower face and the core and c) model used in derivation of design equations.

In practice there are many difficulties in determining the numerical values of the foundation coefficients. Therefore, several ways have been studied to simplify the equations (4, 5). Comparisons with experimental results show the design expressions (7, 8) to yield in a reasonable agreement */Martikainen & Hassinen 1996/* (Fig. 4).

$$\alpha \frac{\sigma_{S2}}{f_{fc}} + \frac{R}{R_r} \frac{1}{\sqrt{1-\frac{\sigma_{S2}}{f_{fc}}}} \leq 1 \quad \text{and} \quad \frac{R}{R_r} \frac{1}{\sqrt{1-\frac{\sigma_{S2}}{f_{fc}}}} \leq 1 \qquad (7),(8)$$

Support reaction resistance R_r in equations (7, 8) is calculated using the model given in ECCS- and CIB-Recommendations */ECCS 1991, ECCS&CIB 1993/*:

$$R_r = f_{Cc}\,(L_S + \eta\; e)\, b \qquad (9)$$

The model represents the local support reaction resistance of the core layer but it indirectly takes into account also the influence of the flexural stiffness of the lower face by increasing the compressed width of the core with the depth of the panel. In the recommendations the distribution factor η has the value of η=*0.5* for the support reaction resistance at the intermediate supports. The distribution factor can also be determined experimentally case by case for a specific sandwich panel product.

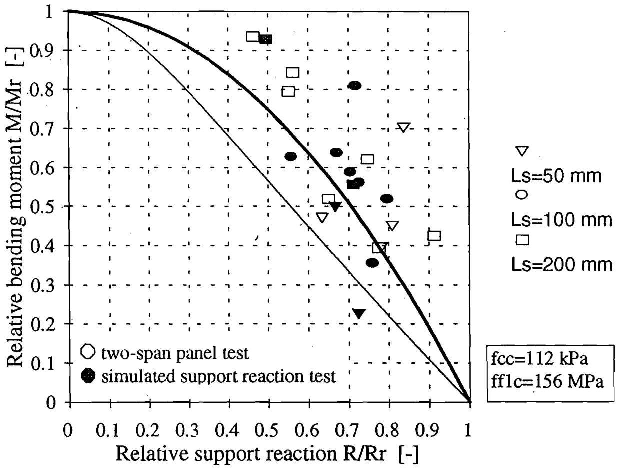

Fig. 4. Design curves for positive support reaction loading case at intermediate support. Curve drawn with thin line depicts the equation (7) and the curve drawn with bold line the equation (8).

Expression (7) results in always lower values compared with the expression (8), which could indicate, that the strength of the face is dominant compared with the strength of the core layer. However, the comparison shows that the equation (8) represents better the experimental results in average (Fig. 4). On the other hand, the simplified formulae (7, 8) do not describe any more the real static behaviour of the face and core at intermediate supports. Based on these facts a set of three design equations covering all the three failure modes at intermediate supports are recommended for the practical design.

$$\tau_C \le f_{Cv} \tag{10}$$

$$\frac{R}{R_r}\frac{1}{\sqrt{1-\dfrac{\sigma_{S2}}{f_{fc}}}} \le 1 \tag{11}$$

$$\sigma_{S2} \le f_{fc} \tag{12}$$

In experiments, sandwich panels only with thin flat steel-sheet faces were investicated. If sandwich panels with lightly profiled face layers are installed on narrow support beams, the second model given in ECCS-Recommendations may be more suitable in evaluations of the support reaction resistances. In the model the face is assumed to be a beam, which is subjected to two line loads at the edges of the support beam and supported by an elastic Winkler's foundation modeling the core */ECCS 1991/*. With this addition, the expression of the support reaction resistance (9) could be extended to

$$R_r = \max\begin{cases} f_{Cc}\,(L_S + \eta\; e)\,b \\[2ex] \dfrac{4 f_{Cc}\, b}{\beta\left(1 + e^{-\lambda}(\cos\lambda + \sin\lambda)\right)} \end{cases} \tag{13}$$

where $\lambda = \beta\, L_S$ and $\beta = \sqrt[4]{\frac{E_C b}{4\, e\, B_{f2}}}$ (14), (15)

Validity of the expressions (11), (12) and (13) to sandwich panels with lightly profiled faces have to be verified experimentally.

If the support beam is an open cold-formed Z- or C-profile, the support pressure in not distributed symmetrically over the profile's upper flange, because one of the edges of the flange is supported by the web plate while the another being supported only by a flexible edge stiffener. In the calculations the support width L_S could possibly be replaced with a reduced width of the profile's upper flange, $L'_S = \eta' \, b_p$. To verify the model, experimental and analytical investigations are needed to check the local failure modes and to determine the resistances of sandwich panels supported by asymmetric open cold-formed profiles.

3.2 Remaining bending resistance

In sandwich panels the compressive force N_S caused by the global bending moment M_S is carried by the face layer alone. After the first failure at an intermediate support, a thin compressed face is still able to carry axial compressive forces, but the compression resistance is notably reduced compared with the compression strength corresponding the first failure at the intermediate support. The compression resistance decreases further with the axial and bending deformations of the face. Therefore, the remaining bending resistance after the first failure has to be evaluated on the basis of the corresponding plastic rotation at the intermediate support (Eq. 16). Distribution of stress components in the faces and core after the first failure is a rather complicated task because of the unclear multi-axial yielding and fracture criterion of the core materials. Therefore, the remaining bending resistance is usually evaluated in a macroscopic scale without any profound analyses of distributions of stresses and strains. Several procedures have been developed for the design of trapezoidal sheetings and purlins and they can be used also in determining the stress resultants of sandwich panels at the ultimate limit state at the supports and in the spans */Unger 1973, Luure & Crisinel 1993, Eurocode 1993/.* The methods developed for sheetings and purlins base on three point bending tests, which are used to simulate the stress resultants at the intermediate supports of continuous beams.

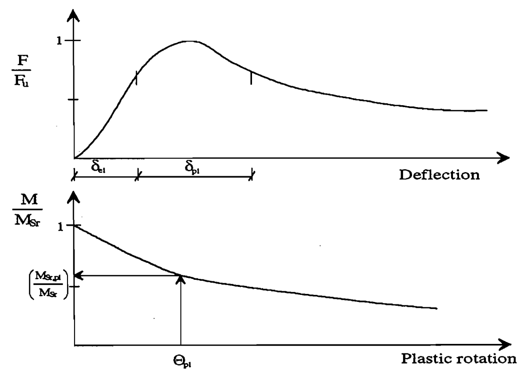

Fig. 5. Principles of the determination of remaining plastic bending resistance at an intermediate support.

$$M_{Sr,pl} = f(\theta_{pl}) = f_{fc,pl}(\theta_{pl})\, A_f\, e(\theta_{pl}) \tag{16}$$

where $\theta_{pl} = \theta - \theta_{el} = \theta - \left(\theta_q + \theta_{M_{Sr,pl}}\right)_{el}$ (17)

When applying Eqs. (16) and (17) to sandwich panels, the displacements caused both by the axial tensile and compressive deformations of the faces and by the shear deformations of the core layer have to be included in the elastic part of the rotation.

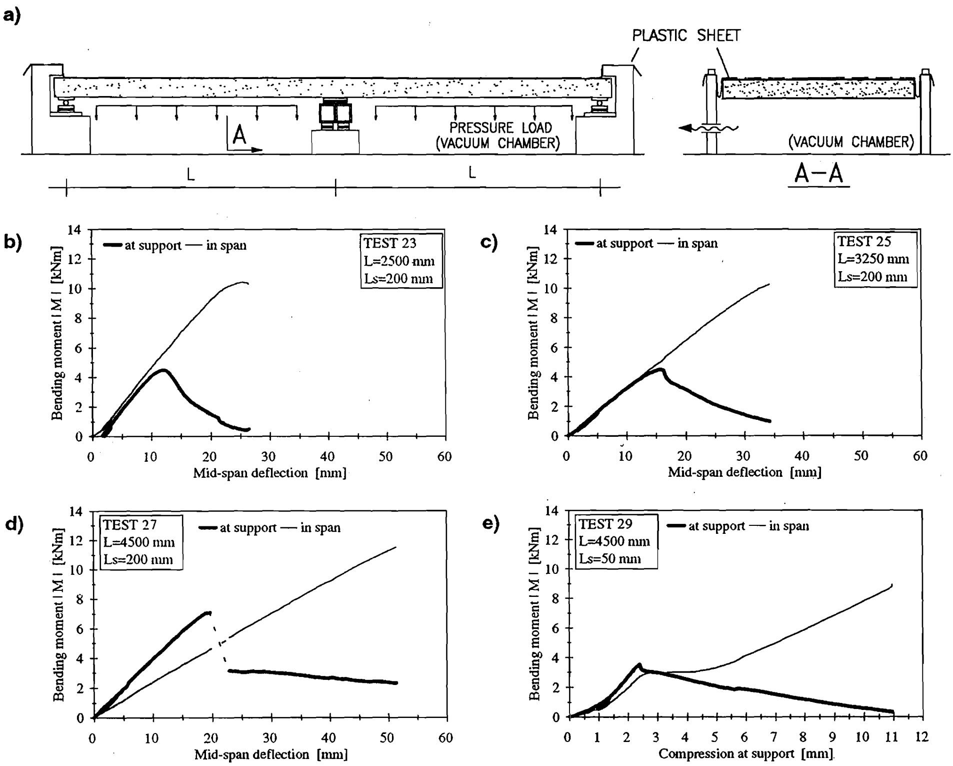

Fig. 6. Experimental bending moment - deflection curves of full-scale two-span sandwich panels loaded using the partial vacuum chamber. The first failure modes in the tests were the shear failure (Test 23), the core crushing failure (Test 25 and Test 29) or the face bending and buckling failure (Test 27).

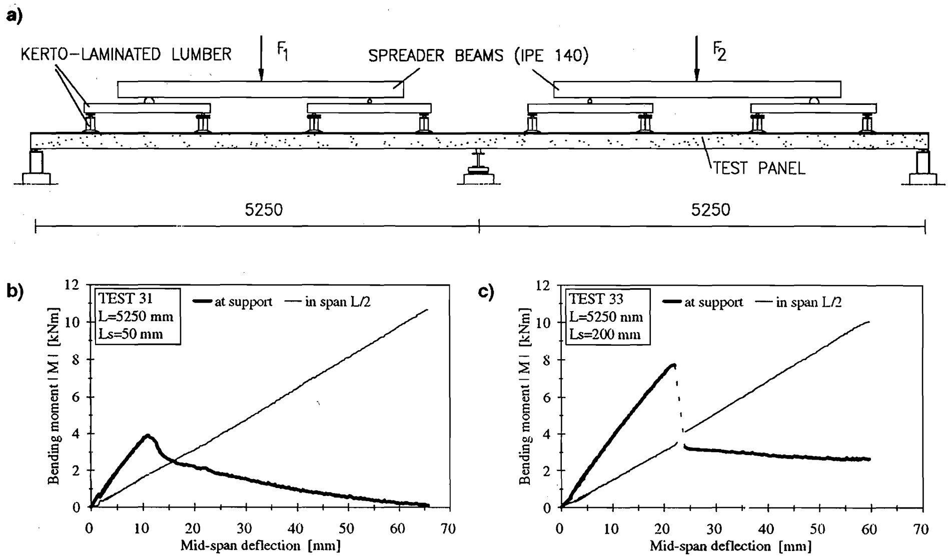

Fig. 7. Experimental bending moment - deflection curves of full-scale two-span sandwich panels loaded by four line loads in each span. The first failure modes in the tests are a core crushing failure (Test 31) and a face bending and buckling failure (Test 33).

Real load-deflection behaviours of continuous two-span sandwich panels can be seen in Figs. 6 and 7 */Martikainen & Hassinen 1996/*. The bending moment - deflection curves are linear up to the first failure at the central support. The first failure was the shear failure or the crushing failure of the core or the bending and buckling failure of the lower face depending on the span and the support width of the specimen. The first failure mode has large effects on the remaining bending resistance at the central support. The core shear and crushing failure modes result in bending resistances, which vanish with the displacements while the face buckling and bending failure modes yield in notable remaining bending resistances, which could be utilized in the design of multi-span sandwich structures.

4. Negative support reaction

4.1 Flexibility of fasteners

In the design models sandwich panels are assumed to be supported by immovable supports, which allow free rotations and axial deformations but no transverse displacements at the supports. The assumption is valid to sandwich panels, which are pressed against relatively rigid and narrow support beams. But if sandwich panels are loaded by wind suction loads or by thermal loads caused by the temperature differences between the face layers resulting in negative support reactions, the flexibility of the commonly used fasteners leads to transverse movements at the supports. That has large effects on the support reactions and the bending moment and shear force diagrams and, further, on the deflections of sandwich panels. The same is true in the case of positive support reactions, if the support beams are flexible enabling notable transverse displacements at the support lines of sandwich panels. The flexibility of the fasteners and support beams is an important design parameter especially in the loading cases, which include thermal loads. Namely, the flexibility of supports changes, and in the case of thermal loads, even reduces the support reactions and internal stress resultants but increases the deflections of sandwich panels.

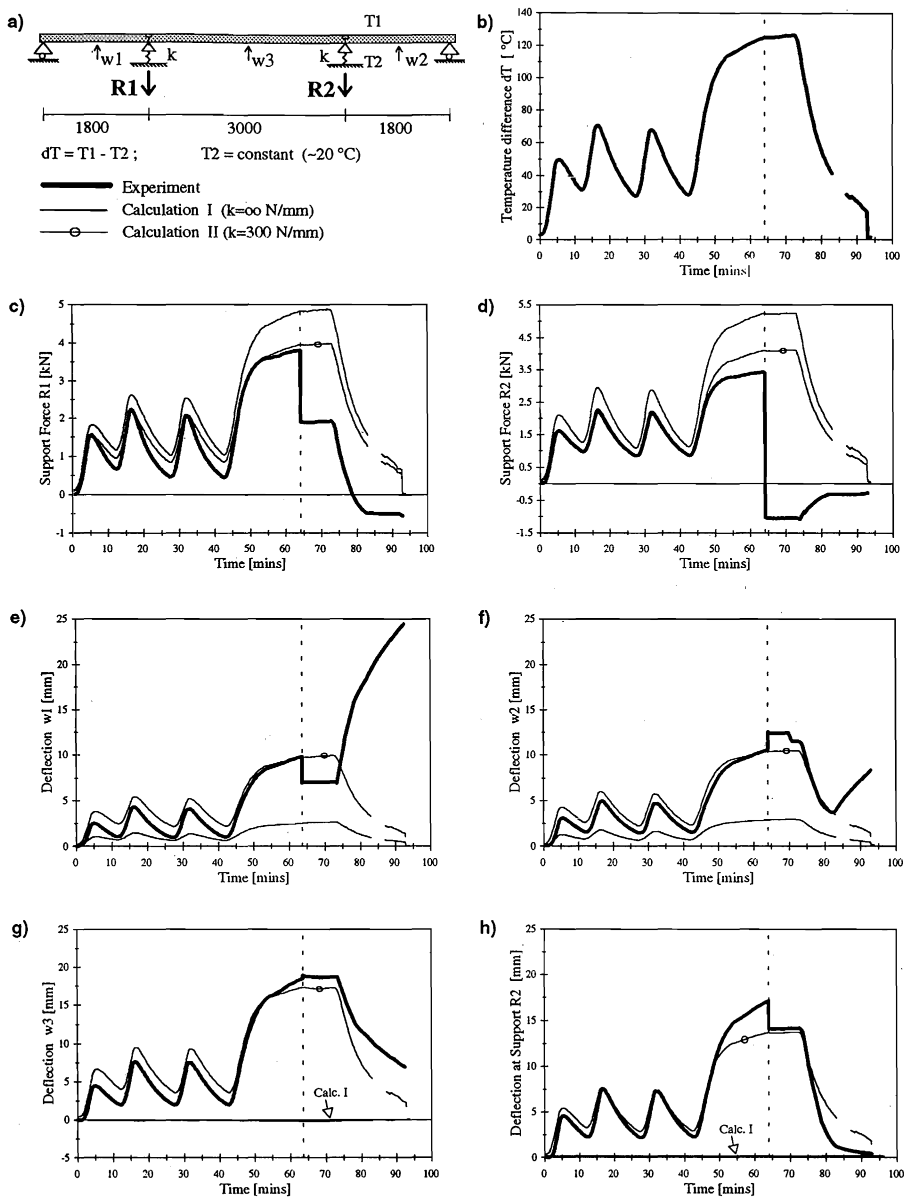

Fig. 8. Experimental and calculated support reactions and deflections of a three-span sandwich panel loaded by temperature differences between the face layers; a) static system, b) temperature loading history, c d) support reactions at the first and second intermediate supports, e f g) deflections in the first end span, central span and in the second end span and h) deflection at the second intermediate support.

In calculation models the flexibility of fasteners can be modeled by a displacement spring corresponding the tangential or secant tensile stiffness of the fastening system. The tensile stiffness depends very much on the fastening system. The special hidden fasteners placed in the longitudinal joints between two sandwich panels show a higher flexibility than the common screw fasteners drilled directly through the panels to the support structure. The flexibility of the support structure can also be modeled by a displacement spring constituting the second spring in the series, which models the total transverse stiffness of the supports lines.

Influence of the flexibility of the special hidden fasteners is illustrated in Fig. 8, which shows experimental and calculated support reactions and deflections of a three-span sandwich beam loaded by temperature differences between the face layers */Martikainen & Hassinen 1996/*. The faces of the test panels were made of steel sheet with a thickness of 0.48 mm and the core layer of a structural rock wool. Depth of the test panel was 100 mm and the width 1200 mm. The test panel was fixed with two hidden fasteners at the both intermediate supports. In the calculations spring constants, $k = \infty$ and $k = 300$ N/mm are used to estimate the flexibility of the fastening system at an intermediate support. The support beams have been immovable in the test.

Comparisons between the experimental and calculated results show large differences, if the spring constant, $k = \infty$, is used. The finite spring constant, $k = 300$ N/mm, modeling the tensile flexibility of the fasteners results in already a reasonable agreement between the experimental and calculated results. In reality, the tensile stiffness of the fastening system is not constant, but changes with the load. Therefore, the analysis with a constant stiffness results in approximate results. The use of spring stiffnesses, which depend on the tensile load in a fastener, complicates the analysis significantly.

4.2 Influence of fasteners on bending resistance

Two fastening systems are common in fixing the sandwich panels with the support structures. Wall panels are most often fastened with screws drilled directly through the panels. For roof panels, special hidden fasteners have been developed to guarantee the water- and air- tightness. The spotlike connection gives rise to initial imperfection in the cross-section of sandwich panel. The heads of the screws stress directly the thin face layer, which is loaded by compressive stresses in the longitudinal direction at the intermediate support. The special hidden fasteners load the joint between the two panels. The influence of the hidden fasteners on the resistance of sandwich panel depends strongly on the stiffness and strength of the structural details in the joint.

Fig. 9 shows experimental stress distributions determined from the measured strains in the outer surfaces of the faces of three-point loaded sandwich panels */Martikainen & Hassinen 1996/*. The test panels were fixed with the loading beam with hidden fasteners placed in the longitudinal joints or with screws drilled through the panels. Based on the stress distributions and observations in the tests following remarks can be made.

<u>Special hidden fasteners</u>
Stress distributions in the compressed face are relatively uniform and constant over the whole compressed face. Small asymmetry in the connection system increases the longitudinal and transverse stresses in the opposite edges of the specimen. In the test the specimen curved strongly around the longitudinal axes, which phenomenon stiffened the compressed face because on the curved shape and the transverse tensile stress field in the face. The specimen failed by a sudden buckling-type failure mode of the compressed face. Only slight remaining bending resistance could be measured at the loading line after the buckling failure.

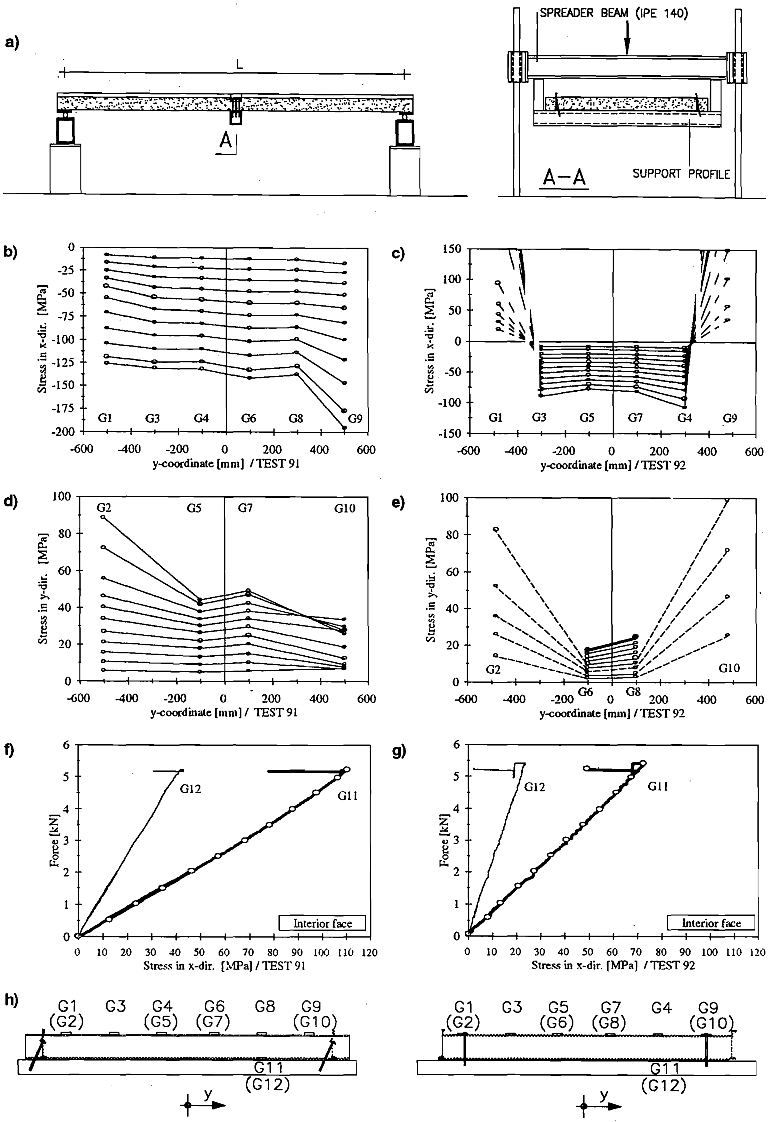

Fig. 9. Experimental results based on measured strains in three-point bending tests with two fastening systems; special hidden fasteners (Test 91) and screw fasteners (Test 92). a) Test arrangements, bc) stresses of the compressed face in the longitudinal and de) in the transverse direction at the mid-span of the specimen, fg) longitudinal tensile stresses (G11) and transverse compressive stresses (G12) in the lower face and h) cross-sections of the specimens and locations of the strain gauges. Markers on the curves in f) and g) show the load levels, on which the stress distributions are presented in figures b)-e).

Screw connectors
Connectors produce high stresses in the compressed face close to the connection points in longitudinal and transverse directions. Because of that, presumably only a part of the width of the compressed face carries through effectively the axial load. The specimen failed by a buckling failure mode of the compressed face. The failure started from the geometric imperfections caused by the screws. The tests with screw fasteners showed the location of the screws in the cross-section to have large effects on the imperfections and failure modes. A screw on a longitudinal stiffeners in the face caused higher imperfections compared with a screw, which was placed in a plane part between the stiffeners in the compressed face. The increased number of screws in a cross-section seemed not to increase the bending resistance of the sandwich panel. Remaining bending resistance is relatively low in sandwich panels fastened with screw connections.

5. Conclusions

The paper presents results of a research project on the static behaviour and strength of continuous multi-span sandwich panels. Test results are compared with results of analytical expressions derived in the previous contributions of the authors. Test specimens in the research have been lightweight sandwich panels with thin flat steel-sheet faces and a structural rock wool core layer, which has to be taken into account when applying the results to other sandwich panel products. On the basis of the results of analyses and tests the following conclusions can be drawn

- positive support reaction
  Three failure modes can be separated at the intermediate support of multi-span sandwich panels. A new design expression is added to take into account the interaction between the bending moment and support reaction and to replace the previous equation for core compression failure. The shear failure mode of the core and the compression failure mode of the face can be studied using the known design expressions.

- negative support reaction
  Flexibility of the fasteners has large influence on the bending moment and shear force distributions and on the deflections, in particular, if the sandwich panels are loaded by thermal loads. The flexibility can be taken into account by modeling the fasteners by displacement springs corresponding the tensile stiffness of the fastening systems.

  Fasteners cause geometric imperfections in the compressed face, which reduce the resistance of sandwich panels at intermediate supports down to 70...50% compared with the bending resistance in the span. Special hidden fasteners placed in the longitudinal joints seem to result in a smaller reduction compared with the screw connection systems.

- remaining bending resistance
  After the first failure, sandwich panels can have notable remaining bending resistance at intermediate supports. The remaining resistance depends strongly on the dominating failure mode at the support and is different in loading cases concerning the positive and negative support reaction. Therefore, a conservative value of zero remaining bending resistance is recommended to be used in the design if the existing failure modes can not be separated in each design case.

Acknowledgements

The contribution presents results of a research project, which was financially supported by Technology Development Centre and Paroc Oy Ab. The support is gratefully acknowledged.

References

European Convention for Constructional Steelwork (ECCS) 1991. Preliminary European Recommendations for Sandwich Panels, Part I Design. ECCS Publication No 66, 1991.

European Convention for Constructional Steelwork (ECCS) & International Council for Building Research, Studies and Documentation (CIB) 1993. Preliminary European Recommendations for Sandwich Panels with Additional Recommendations for Panels with Mineral Wool Core Material, CIB Report, Publication No 148, 1993.

Eurocode 3 1993. Design of Steel Structures, Part 1.3, Cold formed thin gauge members and sheeting. CEN Document TC250/SC3/N269E, Draft March 25, 1993.

Hassinen, P. & Martikainen, L. 1994. Analysis and design of continuous sandwich beams. Proc. of Twelfth International Specialty Conference on Cold-Formed Steel Structures held in St.Louis, Missouri October 18-19, 1994. pp. 523-538.

Hassinen, P. & Martikainen, L. 1995. Serviceability and ultimate limit states of continuous sandwich panels. Proc. of Third Int. Conf. on Sandwich Construction held in Southampton, UK, September 12-15, 1995. Volume 1, 10 p.

Luure, P. & Crisinel M. 1993. Redistribution of moments at the interior support for cold formed sheeting. Swiss Federal Institute of Technology Lausanne, Publication ICOM 288. 43 p.

Martikainen, L. & Hassinen, P. 1996. Load-bearing capacity of continuous sandwich panels. Helsinki University of Technology, Laboratory of Structural Mechanics, Research Report. 195 pp + app. (to be published in 1996).

Unger, B. 1973. Ein Beitrag zur Ermittlung der Traglast von querbelasteten Durchlaufträgern mit dünnwandigem Querschnitt, insbesondere von durchlaufenden Trapezblechen für Dach und Geschossdecken. Stahlbau 42(1973)1, pp. 20-24 (in German).

Appendix A - Notations

A_f	area of face layer per unit width
E_C	modulus of elasticity of the core
G_C	shear modulus of the core
F, F_u	load, ultimate load
$B_{f2} = E_f I_{f2}$	flexural rigidity of lower face per unit width
L	span of sandwich beam
L_S	width of the support of sandwich beam
M	bending moment
M_S	bending moment in sandwich part of cross-section
M_R	local bending moment of lower face caused by support reaction R
M_{Sr}	bending resistance in the span
$M_{Sr,pl}$	remaining bending resistance at intermediate support
N_S	compressive force of the lower face caused by the bending moment M_S
R	support reaction
R_R	support reaction capacity based on the strength of the lower face
R_C	support reaction capacity based on the strength of the core
R_r	support reaction capacity
b_p	width of flange of open cold-formed profile
e	Neper's constant, distance between centroids of upper and lower face
f_{Cc}	compressive strength of the core material
f_{Cv}	shear strength of the core material
f_{fc}	compressive strength of the face layer
$f_{fc,pl}$	remaining compressive strength of the face at intermediate support
f_y	yield stress of face material
k	spring constant
k_W	foundation coefficient of Winkler's foundation model
k_1	second foundation coefficient in two parameter foundation model
α	relation between wrinkling stress and yield stress
β	parameter
$\lambda = \beta L_S$	
δ	relation between support reaction capacities
δ_{el} , δ_{pl}	elastic and plastic parts of deflection
η, η'	distribution factors
σ_{Cc}	compressive stress in core layer
σ_{f2}	stress of lower face
σ_{f2R}	bending stress of lower face caused by support reaction
$\sigma_{w,2}$	wrinkling stress of face layer based on two parameter foundation model
σ_{S2}	axial compressive stress in the lower face caused by the moment M_S
τ_C	shear stress in core
θ	rotation
θ_{el}, θ_{pl}	elastic and plastic parts of rotation
θ_q , $\theta_{MSr,pl}$	elastic rotation caused by loads q and $M_{Sr,pl}$