Steel and Composite Structures, Vol. 8, No. 3 (2008) 231-242
DOI: http://dx.doi.org/10.12989/scs.2008.8.3.231

Design of corrugated sheets exposed to fire

Zdeněk Sokol*, František Wald and Petra Kallerová

Faculty of Civil Engineering, Czech Technical University, Thákurova 7, 166 29 Prague, Czech Republic

(Received October 2, 2007, Accepted June 3, 2008)

Abstract. This paper presents results of fire tests on corrugated sheets used as load bearing structure of roofs of industrial buildings. Additional tests of bolted sheet connections to the supporting structure at ambient and elevated temperatures are described. Three connection types were tested and their resistance, stiffness and deformation capacity was evaluated. Finite element simulations of the corrugated sheet based on the experimental observations are briefly described and design models are presented.

Keywords : corrugated sheet; fire resistance; bolted connection; experiments.

1. Introduction

Corrugated sheets are designed for roofs of span up to 9 metres. Their fire resistance is usually evaluated by experiments. The load bearing criterion R, the integrity criterion E and thermal insulation criterion I are observed for roof structures.

The mechanical load during the test corresponds to accidental design situation and the temperature follows nominal temperature curve. The section factor of corrugated sheets A_m / V (m^{-1}) exceeds 1,000 m^{-1}, therefore the temperature of the steel sheet θ_a can be approximately taken equal to the gas temperature θ_g (Lawson *et al*. 1993). The change of the mechanical properties of steel of thin-walled elements at elevated temperature can be found in literature (Outinen and Mäkeläinen 2002) and also in European standard (EN 1993-1-2).

The reliability of the roof structure at fire is highly influenced by connection of the sheets to the supporting structure. The connection is loaded by forces induced by thermal expansion and contraction and its resistance is influenced by the temperature.

2. Experiments

An experimental programme was carried out in the last few years in fire test laboratories PAVUS (Veselí nad Lužnicí, Czech Republic) and FIRES (Batizovce, Slovak Republic) in cooperation with the Czech company Kovové profily.

The test programme was focussed on experiments where the corrugated sheets were used as the load bearing structure and the thermal insulation of the roof was made from mineral wool or polystyrene, see Table 1 (Hůzl 2002).

†Lecturer, Corresponding author, E-mail: sokol@fsv.cvut.cz

Table 1 Summary of test results with corrugated sheets exposed to fire

Test	Span L, mm	Load q, kN/m^2	Deflection		Fire resistance
			d_1, mm	d_2, mm	
C1	6000	0,83	735	805	R 14, E 14, I 12
C2	6000	0,72	380	390	R 21, E 21, I 21
C3	6000	0,74	709	829	R 20, E 20, I 20
S1	6000	1,00	196	326	R 28, E 27, I 27
C4	6000	0,75	610	591	R 60, E 60, I 60
F1*	4800	0,98	-	442	R 22, E 30, I 30
F2**	4800	0,90	364	469	R 57, E 60, I 60
F3	4800	0,79	429	397	R 42, E 45, I 45
F4	4800	0,79	489	418	R 23, E 30, I 30

Corrugated sheet TR 150/280/0,75 was used for all tests, except
* TR 200/375/0,88, 4 bolts in rib,
** TR 160/250/0,75.

One experiment was carried out on a simply supported beam (S1). For the other tests, a beam with a cantilever end simulating a two-span continuous beam was used, see Fig. 1. The span of the beam was limited by the size of the furnace, see Table 1, the width of the specimens was 3 metres. Only the main span of the beam was heated during the test while the cantilever end and the supports were located outside the furnace. The maximal deflection of the cantilever end was limited to 20 mm to avoid collapse of the structure if a plastic hinge was to form at the cantilever support.

The gas temperature in the furnace θ_g followed the ISO834 standard temperature curve given by

$$\theta_g = 20 + 345\ \log(8t + 1) \tag{1}$$

where t is time in minutes.

The structure was supported on a rigid frame made from two channels U 200. Two self-tapping bolts E-VS BOHR 5-5, 5 × 38 in every rib of the sheet were used to connect the sheets to the frame. The sheets were connected along the longitudinal joints by self-drilling bolts 4.8 × 20 mm spaced at 500 mm.

Material tests were performed at 20°C. The yield limit of the steel was f_y = 374 MPa, the ultimate strength f_u = 461 MPa and the ductility = 18.9%.

The mechanical load represented snow load and services (air conditioning, *etc.*) and was introduced

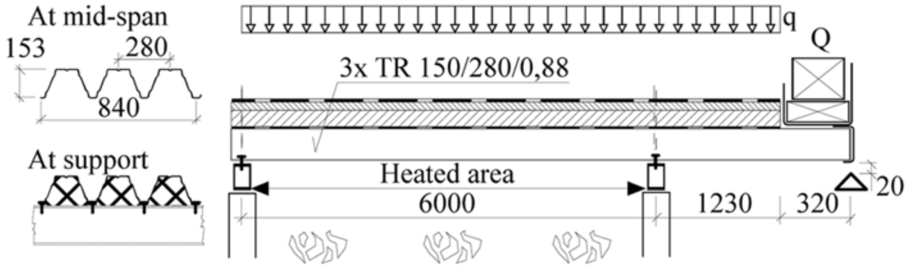

Fig. 1 Test geometry of the fire resistance test C3 (Hůzl 2002)

by lightweight concrete blocks and steel plates, see Fig. 2.

The fire resistance criterion R was evaluated from the experiment using the following conditions which should be satisfied simultaneously (EN 1963-1, 2000). The maximal deflection (mm) should not exceed the limit

$$\delta_{lim} = \frac{L}{400h} \tag{2}$$

where h is the height of the section, and the rate of deformation (*mm/min*) should no exceed

$$\frac{d\delta}{dt} = \frac{L^2}{9000h} \tag{3}$$

after the deflection of $L / 30$ is reached.

The deformations were measured at mid span of the sheet. Two measurements were taken, the

Fig. 2 Fire resistance test C3, load introduced by lightweight concrete blocks

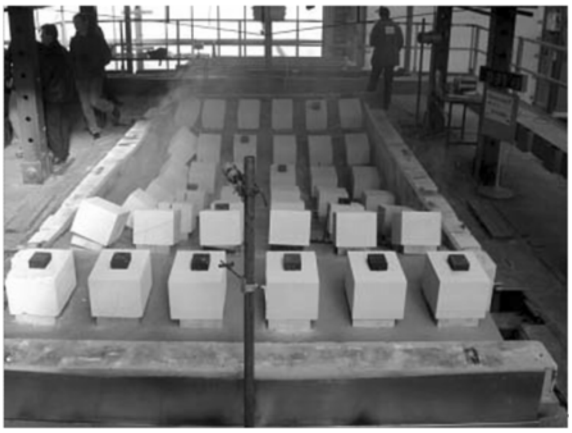

Fig. 3 Fire resistance test C3, the specimen after the test

transducers were located 500 mm from the longitudinal edges of the sheet.

The experimental results are summarized in Table 1 and plotted on Fig. 7. All the experiments were stopped when the resistance criteria listed above were exceeded. No collapse of the structure was observed. Local buckling on the bottom surface of the sheet was observed as a result of clamped support, see Fig. 4. Plastic strains and deformation of the cross section were observed at mid-span of the sheets, see Fig. 5.

Temperature of the connections was measured during test C3, see Fig. 7. The thermocouples were attached to the sheet close to the bolts. The ribs at the end support were filled with mineral wool to achieve thermal insulation of the bolts (measured by thermocouples C3TB1 and C3TB2). No thermal insulation was applied at the cantilever support (thermocouples C3TB3 a C3TB4). The temperature at time $t = 15$ min reached 135°C at the end support (insulated bolts) and 352°C (non-insulated bolts).

The connections were placed outside the furnace and were not exposed to the fire. They were heated only by the heat conduction of the steel sheet. Therefore, these tests may not reflect practical situations because it is expected that temperature of the connections of a real structure exposed to fire will be higher than those obtained during the laboratory tests.

Fig. 4 Fire resistance test F2, the specimen after the test

Fig. 5 Fire resistance test F2, the specimen after the test

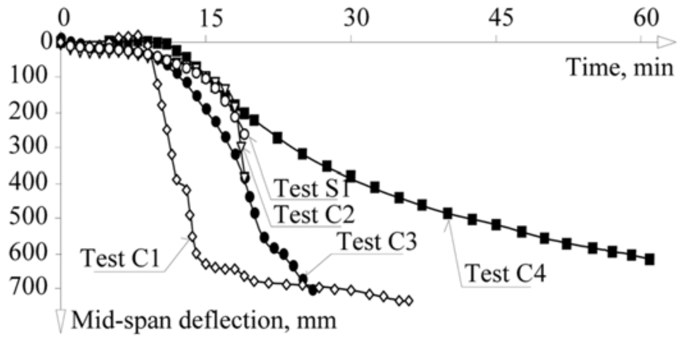

Fig. 6 Mid-span deflections of specimens C1 - C4 and S1

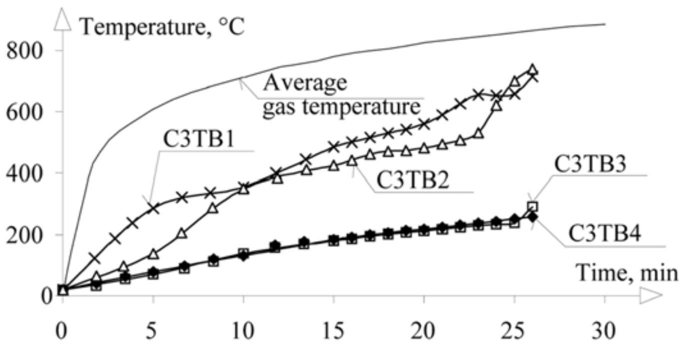

Fig. 7 Measured temperature in the connections of test C3 (Hůzl 2002)

3. Behaviour of the connections

Separate elevated temperature tests were carried out on the same types of connections used in the fire tests. Fig. 8 shows the test set up. The connection of the sheet to the support was subjected to experimental study at the laboratory of Czech Technical University in Prague. The tests were performed on steel sheets thickness 0,75 mm bolted to 10 mm steel plate. Three connection types were used for the tests:

- bolt E-VS BOHR 5-5,5 × 38 with sealed washer ∅ 19 mm,
- bolt E-VS BOHR 5-5,5 × 38 with steel washer ∅ 29 mm,
- bolt SD8-H15-5,5×25 without washer.

The tests were carried at constant temperatures 20, 200, 300, 400, 500, 600 and 700°C, see Fig. 9, 10 and 11. The resistance, stiffness, deformation capacity and collapse mode of the connection were observed.

The resistance of the connection with sealed washer was limited by bearing resistance of the sheet. The sealant burnt at temperatures higher than 200°C and the flexible washer did not influence the bearing resistance and the stiffness of the connection. For the typical collapse mode of the connection see Fig. 13.

When steel washers were used, the stiffness of the connection increased and the resistance was almost

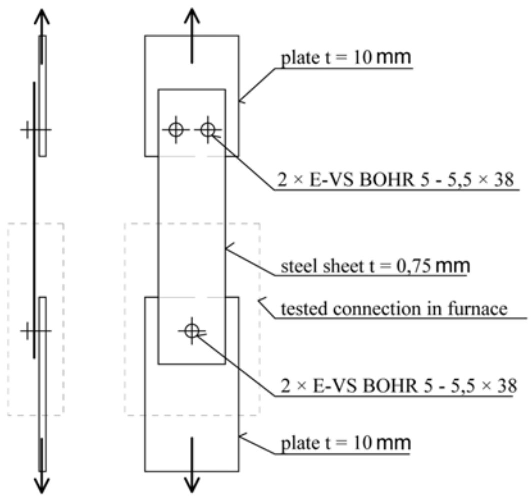

Fig. 8 Test set-up of bolted connection at elevated temperature

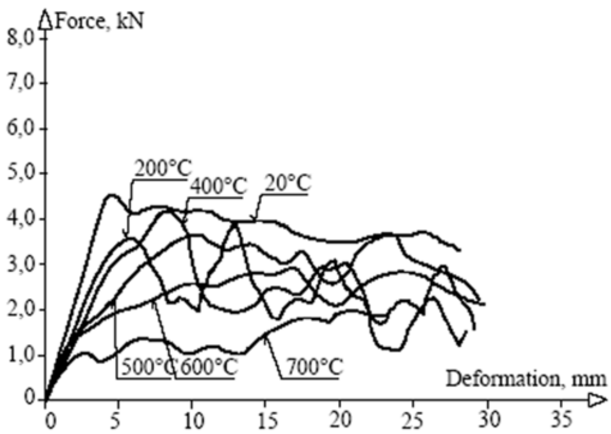

Fig. 9 Connection behaviour at elevated temperature, bolts E-VS BOHR 5-5,5 × 38 with sealed washer ý 19 mm (Sokol 2005)

doubled compare to the connection with sealed washers, see Fig. 10. The benefit of using steel washers is related to the collapse mode, see Fig. 14, where the thin sheet was deformed and pushed in front of the washer. However, at temperatures exceeding 500°C shear failure of the bolt was observed. However, this failure mode is accompanied by reduced deformation capacity and therefore should be avoided. This can be achieved by increasing the number of bolts.

4. FEM modelling

The general finite element code ANSYS was used for numerical simulations of the fire tests. Plastic

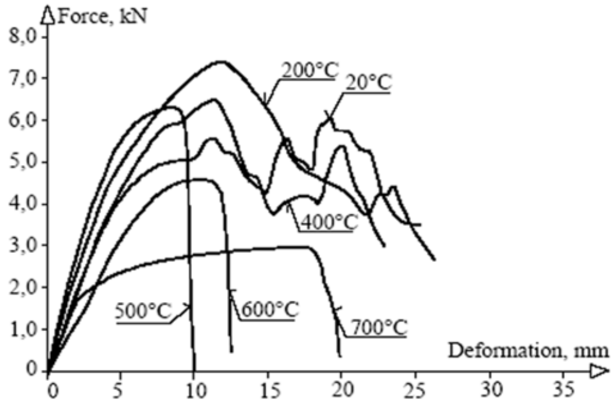

Fig. 10 Connection behaviour at elevated temperature, bolts E-VS BOHR 5-5,5×38 with steel washer Ø 29 mm (Sokol 2005)

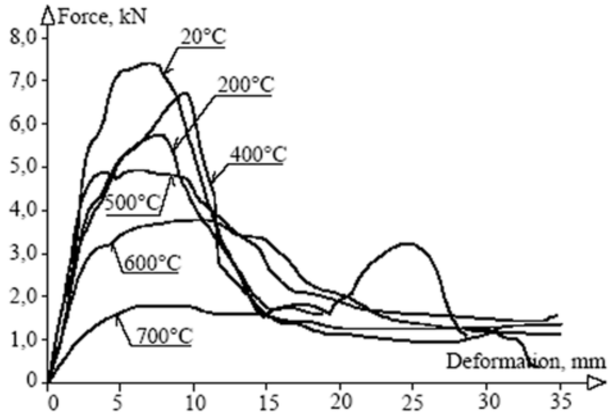

Fig. 11 Connection behaviour at elevated temperature, bolts SD8-H15-5,5×25 without washer (Sokol and Kallerová 2006)

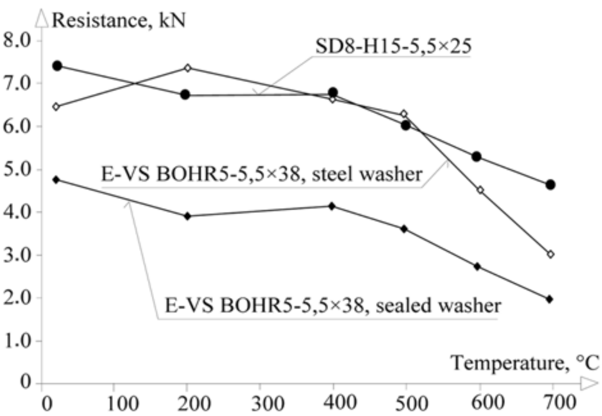

Fig. 12 Resistance of connections at elevated temperatures

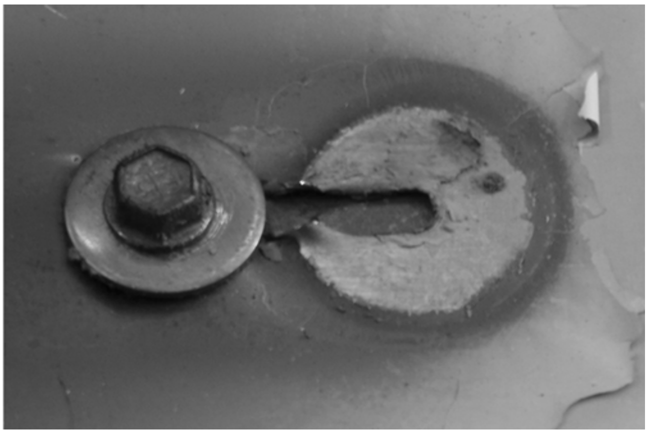

Fig. 13 The collapse mode of bolted connection at 400ºC, bolt E-VS BOHR 5-5,5×38 with sealed washer Ø19 mm

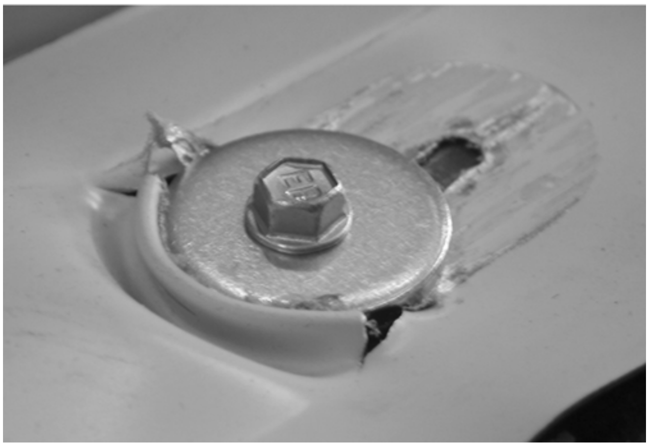

Fig. 14 The collapse mode of bolted connection at 200ºC, bolt E-VS BOHR 5-5,5×38 with steel washer Ø29 mm

Fig. 15 The collapse mode of bolted connection at 200ºC, bolt SD8-H15-5,5×25 without washer

beam element with three degrees of freedom at each node (BEAM 23 in the ANSYS element library) were used for the trapezoidal sheet, see Fig. 16.

Temperature dependent multi-linear isotropic material stress strain relationship ($\sigma - \varepsilon$) based on the rules in EN 1993-1-2 was introduced, see Fig. 17. The influence of the connections (stiffness and resistance) was modelled by non-linear translational spring elements NONLIN 39. The force-deformation relationship of the bolted connection corresponds to the experiments on individual connections described in section 3. Contact element representing the auxiliary support at the cantilever end was used to limit its deformation.

Non-linear analysis with large strains and large deformations was performed. The load was introduced in two steps, the first representing the mechanical and the second the thermal load (Sokol and Vácha 2002).

The numerical model was used to perform a numerical study to examine the influence of various parameters on the beam behaviour in fire.

Stiffness of the supports has significant influence on the overall response. When restrained supports are assumed, the thermal expansion of the beam is prevented which leads to large deflection of the structure. At higher temperatures membrane effect develops which contributes to the resistance of the structure after the bending moment resistance is significantly reduced by the temperature.

In reality, the supports do not need to be rigid but certain stiffness is required. The behaviour is quite different in this case: deformation of the supports allows for the thermal expansion and does not lead to large deflection at the early phase of the fire. Large deformation is observed at higher temperature when

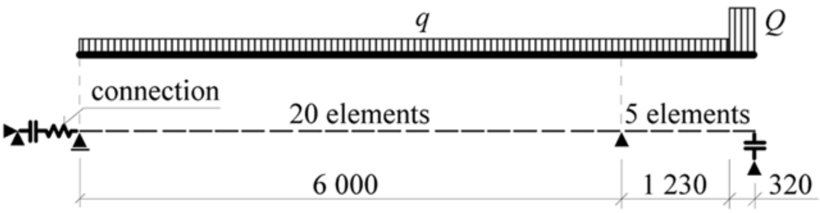

Fig. 16 The FEM model of the corrugated sheet

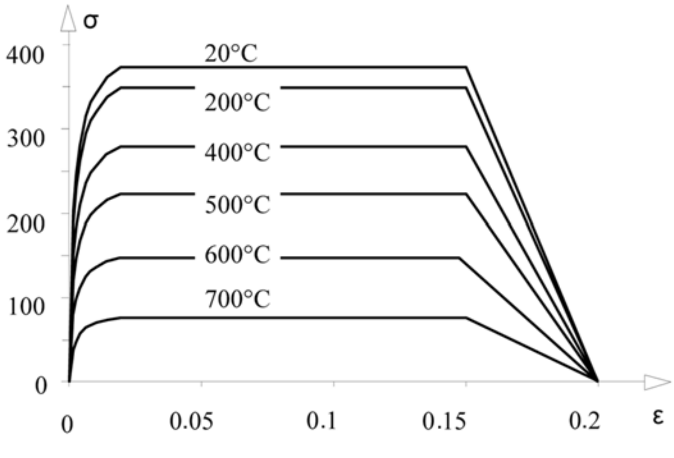

Fig. 17 Temperature dependent stress-strain diagram

the reduced bending moment resistance is equal to the applied bending moment. However, collapse of the structure is not observed because membrane effect will also contribute to the resistance, see Fig. 18. It can be seen that the membrane effect can be observed for a wide range of the stiffness of the support.

5. Design models

The design model of thin-walled elements at ambient temperature is based on effective cross section taking into account local buckling. The bending moment resistance is calculated using effective moment of inertia $I_{a,eff}$ (Rondal 2003). In addition, the resistance of the web to concentrated load at the supports needs to be checked.

The resistance check at elevated temperature is based on similar assumptions. The effective cross section can be derived for the steel temperature using reduced yield limit and modulus of elasticity (Ranby 1998). As a simplification, the effective cross section derived at ambient temperature can be used for check at elevated temperatures.

Assuming unrestrained supports of the corrugated sheet, the bending moment resistance is given by

$$M_{\theta,Rd} = W_{a,eff,\theta}\, k_{y,\theta} f_{y,p} \tag{4}$$

where $W_{a,eff,\theta}$ is effective section modulus at temperature q_a, $k_{y,\theta}$ is reduction factor of yield stress at temperature θ_a and $f_{y,p}$ is yield stress. Uniform temperature of the cross section is expected.

When restrain at the supports can be maintained during the fire, membrane effect will contribute to the resistance. As a simplification, it is assumed that the bending moment resistance can be neglected and the total load is transferred by membrane action of the sheets.

Axial force in the sheets needs to be calculated which depends on deflection of the trapezoidal sheet. The extension of the sheet consists of three parts:

- thermal extension of the material,
- slip in the bolted connections at the support and

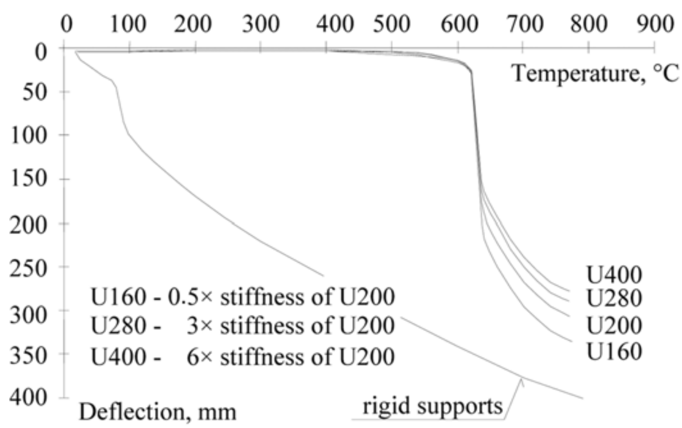

Fig. 19 Design model of corrugated sheet with restrained supports at elevated temperature

- extension caused by membrane force N.

The thermal expansion of the corrugated sheet is equal to

$$\Delta_p = \alpha L(\theta_a - 20) \quad (5)$$

where α is coefficient of thermal expansion of steel and L is span of the sheet. The influence of slip of the connection Δ_b at the support can be introduced by:

$$\Delta_b = \frac{N_H}{nK_{b,\theta}} \quad (6)$$

where n is number of bolts per unit width of the sheet, $K_{b,\theta}$ is stiffness per bolted connection in N/mm at temperature θ_a and N_H is the horizontal component of the axial force N, see Fig. 19.

The extension of the sheet caused by axial load N is given by

$$\Delta_N = \frac{NL}{E_\theta A} \quad (7)$$

The horizontal component of the axial force N_H, see Fig. 19, is obtained from the catenary equation

$$N_H = \frac{qL^2}{8\delta} \quad (8)$$

and the deflection of the trapezoidal sheet δ is

$$\delta = \frac{\sqrt{6L\Delta L}}{4} = \frac{\sqrt{6L(\Delta_p + 2\Delta_b + \Delta_H)}}{4} \quad (9)$$

The rotation α at the support, see Fig. 19, can be evaluated from

$$tg\ \alpha = \frac{4\delta}{L} \quad (10)$$

and the axial force N is equal to

$$N = \frac{N_H}{\cos\ \alpha} \quad (11)$$

Iterative procedure is necessary to solve the deflection, because the slip in the connection and extension of the sheet depend on the axial force N. An initial estimation of the force and several cycles of the calculation are necessary for accurate results.

The resistance of the bolted connections loaded by shear force N_H should be checked. Design model for the connection at elevated temperature is not available at present therefore the design resistance of the connection needs to be evaluated by testing.

6. Conclusions

The resistance of roof structure made from corrugated sheets can reach fire resistance R 60. It is achieved by low stress in the corrugated sheets at ambient temperature which is the reason of design to meet the SLS criteria (deflection).

Two design models are available depending on the support of the sheets. For unrestrained support, the model is based on bending resistance of the effective section. When thermal elongation / contraction is restrained, membrane model is used to predict the resistance.

The numerical study shows that even small stiffness of the supports is sufficient for the membrane effect to develop. The resistance and stiffness of the bolted connections and of the supporting structure should be checked in this case.

Acknowledgement

The research work was supported by grant MSM 6840770001 Reliability, optimization and durability of building materials and constructions.

References

EN 1363-1 (2000), Fire resistance tests. Part 1. General requirements. CEN, Brussels.

EN 1993-1-2 (2005), Eurocode 3. Design of steel structures. Part 1-2. General rules. Structural fire design. CEN, Brussels.

Hůzl J. (2002), Vodorovná nosná konstrukce, skládaný střešní plášt'. Protokol o zkoušce požární odolnosti Pr-02-02.014, 02.043. (Research report on fire test No. Pr-02-02.014, 02.043. In Czech). Veselí n. L., PAVUS.

Lawson R.M., Burgan B. and Newman G.M. (1993), *Building design using cold formed steel section, fire protection*, Publication No. P129, SCI, London.

Outinen, J. and Mäkeläinen, P. (2002), "High-temperature strength of structural steel and residual strength after cooling down", *Proceedings of Stability and Ductility of Steel Structures*, Akadémiai Kiadó, Budapest.

Ranby A. (1998), "Structural fire design of thin walled steel sections", *J. Const. Steel Res.*, **46**, 1-3.

Rondal J. (2003), Lesson 2. Design of Cold–Formed Steel Sections, CISM Course, CISM, Udine.

Sokol Z. and Vácha J. (2002), "Numerická simulace trapézového plechů za zvýšených teplot. Výzkumná zpráva. (Numerical simulation of corrugated sheets at elevated temperature, Research report. In Czech), Kovové profily, Praha.

Sokol Z. (2005), Experimenty s přípoji trapézových plechů, Část 2 Zkoušky za zvýšené teploty. Výzkumná zpráva (Experiments with connectionsf corrugated sheets. Part 2. Tests at elevated temperature. In Czech). Praha, ČVUT.

Sokol Z and Kallerová P. (2006), Experimenty s přípoji kazetové stěny. Zkoušky za zvýšené teploty. Výzkumná zpráva (Experiments with connections of thin-walled sheets. Tests at ambient and elevated temperatures. In Czech). Praha: ČVUT.