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EXPERIMENTAL DIAGNOSTICS AND STATIC ASSESSMENT OF FIRE DAMAGED STEEL STRUCTURE

In the paper are presented the results of diagnostics, tests of materials, numerical modelling and static analysis of the current state of the bearing steel structure of the hall after a fire and design of the actions for the repair of the damaged steel structural members.

Key words: steel structure, diagnostics, material tests, modelling, fire, analysis, resistance of structure, renewal.

Подано результати діагностичного випробування матеріалів, числове моделювання і структурний аналіз поточного стану сталевих конструкцій залу після пожежі і запропоновано заходи щодо ремонту пошкоджених елементів сталевої конструкції.

Ключові слова: сталеві конструкції, діагностика, випробування матеріалів, моделювання, пожежа, аналіз, міцність конструкції, реконструкція.

Introduction

The aim of the paper is to characterize the diagnostics, a comprehensive analysis and design of measures required for renewal of fire-damaged steel roof tie-beams, based on a detailed static analysis of the bearing steel building structure. A fire with a temperature over 1000 °C and with the duration to about 20 minutes overran throughout the all height of the object, and reached the level of the roof structure at the height of 23,70 m.

The basic prerequisite for a realistic static assessment of the building damaged by fire is the knowledge of the current state of physical properties of materials of the individual structural members and the knowledge of their static interaction in the bearing system. The mentioned current parameters must be implemented into the theoretical computational model in order to achieve an optimal accordance between the real behaviour of the structure and the theoretical response obtained by the static computational model.

A study of the resistance of the steel bearing structure in the course of fire in order to simulate the temperature effect of fire is presented too. According to STN EN 1993-1-2 [6] the resistance of the structure was assessed and the time at which the structure has lost its resistance was determined.

Mechanical and brittle failure properties of investigated steels

The most important material characteristic of steels in the connection of their fire-resistance is the temperature dependence of yield stress and ultimate stress. In the standard [6] minimum values of the individual reduction factors (chapter 6 – equation (2), (3) and (4)) are introduced. These factors characterize the ratio of a given quantity at the studied temperature due to the value of this quantity at room temperature.

Brittle failures characteristics are verified by the Charpy V-notch toughness test, i.e. impact in bending on the test bars with the V-notch is performed. In the test, the influence of impact loading and stress concentration around the notch and the mode of the failure are highlighted.

Current state of the bearing steel structure after fire

The present hall building was built and put into use in 1965 (Fig. 1). Bearing system of the hall create columns of welded steel I-1100-sections with transverse stiffeners, situated on one side of the outer plane of the building and reinforced concrete columns mounted on the other side of the building. The bearing structures of the roofing create steel truss tie-beams. The main girder of the crane runway is made of a welded steel I-1200-section [2].

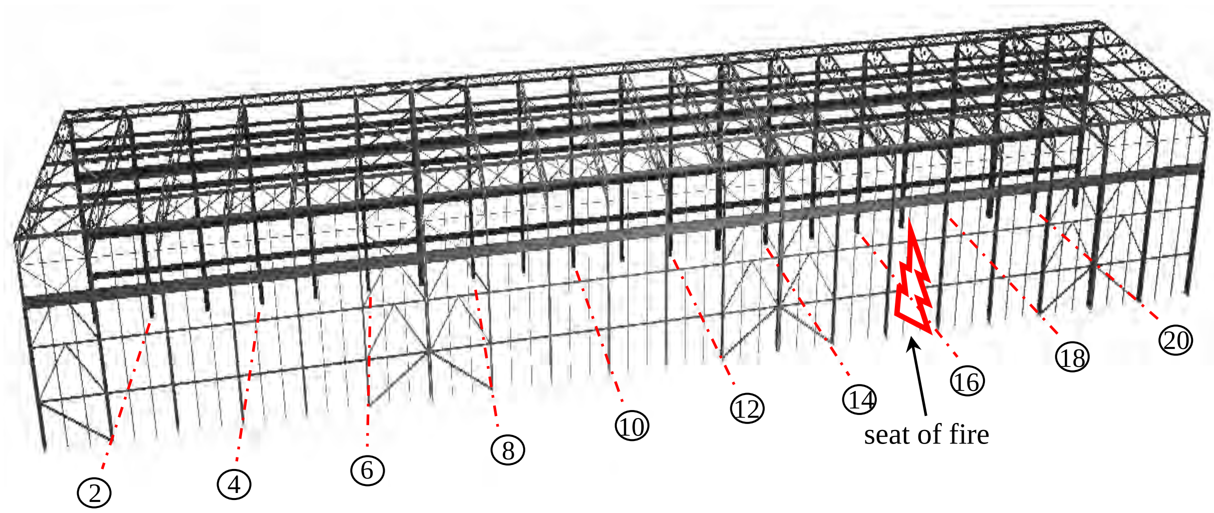


Fig. 1. Model of the hall with marked axes of the individual transversal bonds and denotation of the seat of fire

The subject of diagnostics, investigation and static assessment was focused on:

– Detection of the current technical condition of the bearing steel structure and the assessment of resistance of its structural members after the fire;

– Diagnosing of the degree of damage and design of such arrangements which enable a safe and reliable using of the building.

– In order to determine the degree of damage of the steel bearing structure after the fire the following non-destructive investigations “in situ” and destructive tests “in situ” and “in laboratory” were carried out:

– Detection of actual mechanical properties of the material of the bearing steel structure, hence samples of the selected members of the steel structure were detracted (from the columns, roof truss tie-beams and roof bracings as well as from girders of the crane runway);

– Magnetic-defect check of welds;

– Thickness measurement of cross-sections of selected members of the steel structure;

– Geodetic measurements of the distorted shape of the steel roof truss tie-beams and their deflections as well as measurement of the rail offset of the crane runway.

Some selected results from a comprehensive analysis of the mechanical properties of materials of members of the steel bearing structure exposed to the effects of temperature at fire are presented hereinafter.

Steel specimens taken were then tested in accredited laboratories and testing labs.

To compare the current mechanical properties of materials of steel structure affected and unaffected by fire samples were taken:

– from the area directly affected by the fire (in fireplace) – between the15-16 axes;

– from the area indirectly affected by the fire (in the vicinity of an outbreak) – between the 13-14-15 axes;

– from the area unaffected by the fire – between the 1 to 13 axes.

On samples were carried out:

– Static tensile test for the determination of yield stress f_y, tensile strength f_u, ductility A_5 and contraction Z;

– Determination of tensile modulus of elasticity E (for the selected samples);

– Charpy V-notch toughness test (for the selected samples).

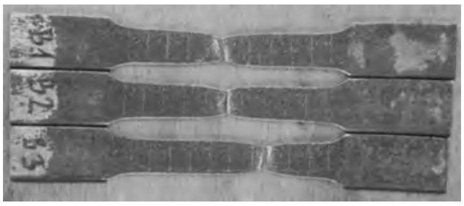
Totally 49 tensile specimens were tested (3 testing specimens for each sample).

Tensile tests were performed on a ZWICK 1387 – Hegewald & Peschke, Mess-und Prüftechnik GmbH (inspect retrofit), Nossen tensile machine under standardized conditions. For Young’s modulus E measuring a special ZWICK extensometer of the 7852-type with a high accuracy was used.

For a controlling the test procedure and for a calculation of results the Labmaster software was used. This software numerically and graphically evaluates all relevant variables, namely the yield stress value $f_{y0,2}$, the upper yield stress value f_{yU}, lower yield stress value f_{yL} and ultimate strength f_u (Fig. 2).

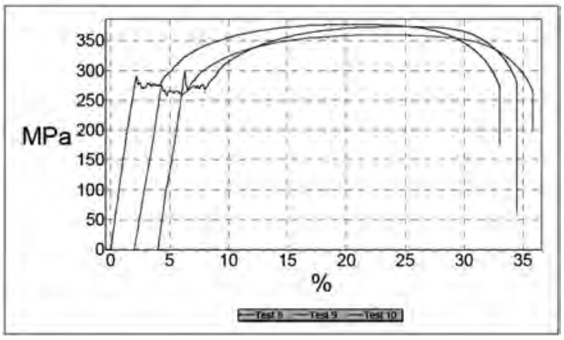
Tensile diagrams correspond to the low carbon structural steel used for welded structures. The fracture appearance corresponds to the mechanical properties evaluated. No anomalies in the behaviour of tested samples were detected.

a



b

name	Spec. No.	Rp0,2 [MPa]	ReH [MPa]	ReL [MPa]	Rm [MPa]
Test 8	B-1	289	291	269	377
Test 9	B-2	265	276	259	360
Test 10	B-3	280	300	267	374



name	Spec. No.	Rp0,2 [MPa]	ReH [MPa]	ReL [MPa]	Rm [MPa]
Test 1	1-1	242	254	241	417
Test 2	1-2	238	243	228	415
Test 3	1-3	231	259	230	428

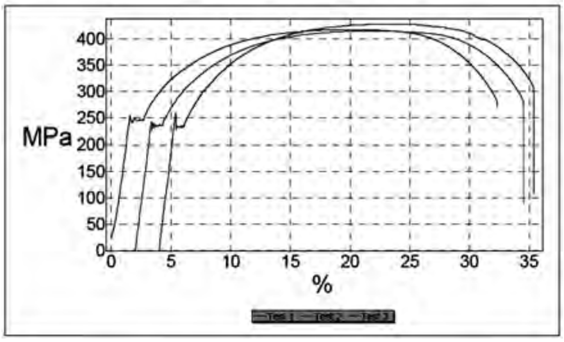


Fig. 2. Flat testing specimens (a), cylindrical testing specimens (b) and stress-strain diagrams ($R_{p0,2} \rightarrow f_{y0,2}$, $R_{eH} \rightarrow f_{yU}$, $R_{eL} \rightarrow f_{yL}$, $R_m \rightarrow f_u$)

Based on a visual inspection, diagnostics and investigation of the steel structure of the hall the following facts can be introduced:

– Plastic deformations of the top chords of truss tie-beams in the 15 and 16 axes and their spatial displacement were identified as is shown in Fig. 3;

– In the bottom chords of truss tie-beams in the 15 and 16 axes local plastic deformations of the cross-sections near supports (at the steel columns) were occurred;

– Web buckling between stiffeners of the column in the 16 axis was identified;

– The main girder of the crane track is not distorted (deflection of the rail axis obtained by measurement is compensated through the necessary rectification);

– Vertical longitudinal roof bracings situated between the truss tie-beams in the 15-16 axes are deformed;

– Side horizontal roof bracings in the roof plane are not deformed.

a

b

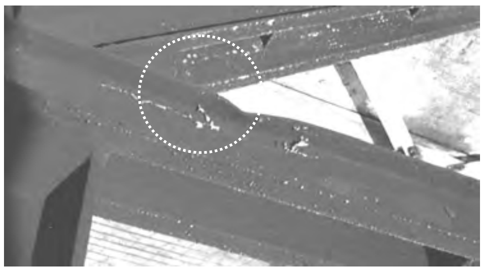


Fig. 3. a) the deformed buckled top chord of the truss tie-beam in the 16 axis (over the outbreak of fire)
b) the deformed bottom chord of the truss tie-beam in the 16 axis (over the outbreak of fire)

Based on the tests carried out on taken specimens (tensile tests, Charpy V-notch toughness tests), welds inspections, measurement of thickness of cross-sections and geodetic measurements the following results were obtained:

– Results of the tensile tests of the material (steel) samples taken from the columns in the zone of the fire (in the 15 and 16 axes), in the places near the fire (in the 13 and 14 axes) and in the zones far-off the fire (in the 6-axis) did not confirmed changes of the mechanical properties of the steel.

– Results of the tensile tests of the material (steel) samples taken from the roof horizontal bracings in the zone of the fire (between the 15 and 16 axes) and in the places near the fire (between the 14 and 15 axes) confirmed approximately 10 % average decrease of the physical properties compared to the properties of the specimens selected from the zones far-off the fire (between the 7 and 8 axes).

– Results of the tensile tests of the material (steel) samples taken from the transversal stiffeners of the beam of the crane track in the zone of the fire (between the 15 and 16 axes), confirmed approximately 10% average decrease of the physical properties compared to the properties of the specimens selected from the zones far-off of the fire (between the 10-11 and 11-12 axes).

– Results of Charpy V-notch toughness tests of the steel samples taken from the column in the zone of the fire (the 16 axis) did not confirm a decrease of the toughness.

– Results of Charpy V-notch toughness tests of the steel samples taken from the roof horizontal bracing in the zone of the fire (between 15 and 16 axes) did not confirm a decrease of the toughness.

– Magnetic-defect control of welds of the bottom and trop chord of the truss tie-beam in the 15 and 16 axes and of the column in the 16 axis as well as of the beam of the crane track between the 15-16 axes did not confirm the failures of the welds.

– Measuring of a metal thickness of the selected member cross-sections did not confirm significant deviations from the values introduced in the project documentation.

– Continuous geodetic measurements of the selected points in the top and bottom chords of roof trusses in the 15 and 16 axes confirmed the progressive increase of the displacements (deformations) in the horizontal transversal direction during the period of the four days. In the place of the buckled chord (truss in the 16 axis) the increment of the displacement in the horizontal direction of approximately 35 mm occurred. In the middle of the bottom chord of the truss in the axis 15 the increment of the displacement in the horizontal direction of approximately 70 mm was measured.

– The main girder of the crane track is not distorted (deflection of the rail axis obtained by measurement is compensated through the necessary rectification).

Static analysis

The structure was designed in 1964 according to then applicable standards. Determination of the load and the reliability assessment of the structure at an actual state was made according to the current standards STN 73 0035 [3] and STN 73 1401 [4], respectively.

From the original project documentation was not possible to determine the class of steel. Therefore results based on tests were used and the initial values of the current mechanical properties of the steel bearing structure material (Young's modulus and yield stress) in the computational model were defined.

3D spatial bar model of the steel structure of the hall (Fig. 1) in the IDA SCIA NEXIS program based on FEM was generated (model contains 2 423 nodes and 3 952 elements).

An assessment of trusses with a minimum load of the existing state was performed, hence only the effect of a dead load was considered.

In the analysis the following loads were considered:

1) Permanent load: the self-weight of the structure (directly generated in a computer program) and the vertical component of load from the roof cladding:

– lightweight porous concrete slabs: 1,574 kN.m^{-2};

– a weight of the insulation layers of the roof cladding: 0,42 kN.m^{-2};

2) Variable load: snow load (snow area No. II): s_o = 0,7 kN.m^{-2};

3) Variable load: wind load (wind area No. IV): w_o = 0,55 kN.m^{-2};

4) A load of the cranes effects: bridge cranes with lifting capacities of 50 t and 12,5 t.

Imperfections in the form of the deviation of the top chord of the truss tie-beam and an interrupting of the top chord of the truss tie-beam (formation of the plastic hinge in the place of deviation of the top chord) were modelled in the computational model of the steel structure of the hall.

In the analysis and reliability assessment a deviation with a size of 200 mm of the top chord of the truss tie-beam in the horizontal direction was considered (Fig. 4).

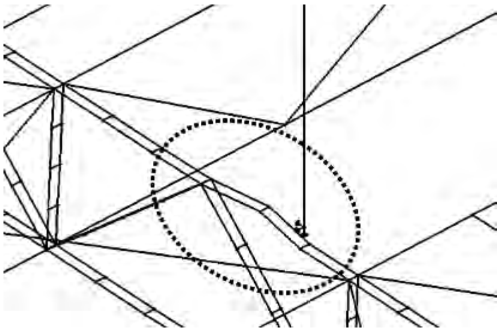


Fig. 4. Computational model with the introduction of imperfections

Results of the static analysis confirmed that the stresses of the top chords of the truss tie-beams (in the place of considered imperfections) are behind the elastic limit and chords are characterized by a plastic behaviour (Fig. 5).

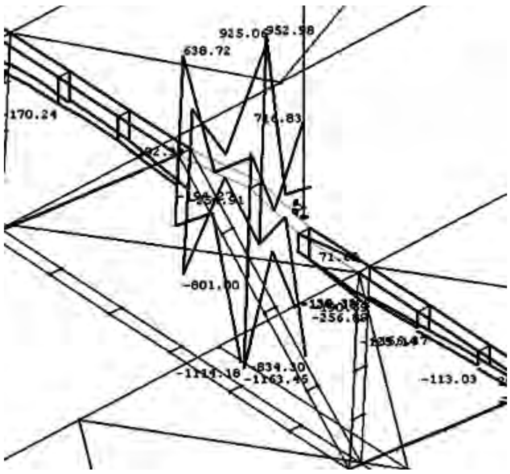


Fig. 5. The resultant distribution of stresses in the top chords of the truss tie-beams

The maximum vertical deflection obtained from the analysis of the truss tie-beam with imperfections loaded by its self-weight and by a weight of the roof cladding is 137 mm.

In order to investigate the behaviour of the diminished steel structure an interrupting of the top chords of the truss tie-beams were modelled too and the analysis of the diminished structure was performed (Fig. 6 and Fig. 7).

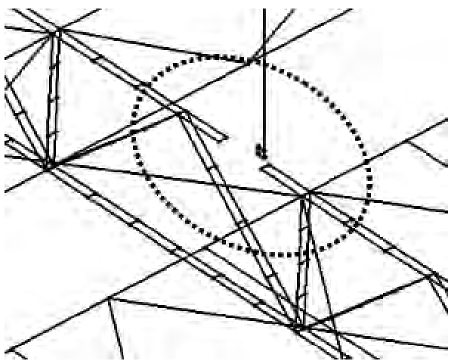


Fig. 6. Static computational model of the top chords of tie-beams with an interrupting

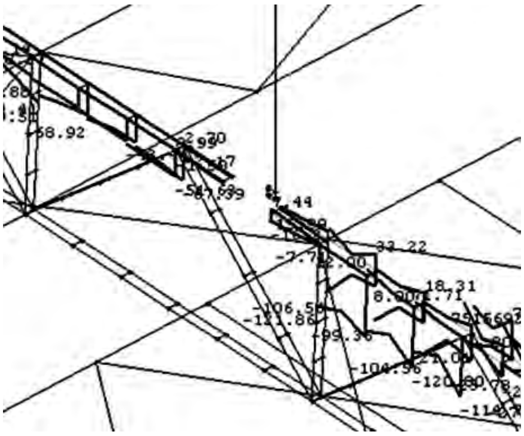


Fig. 7. The resultant distribution of stresses in the top chords of the truss tie-beams

Static analysis on the 3D model of the steel structure with real geometric imperfections was performed. In the model deformations of the top chords of the roof truss tie-beams were considered. Actual values of the physical properties of material (steel) after the fire obtained by the laboratory tests were used for the reliability assessment of the structure.

Geometrically nonlinear approach for a global analysis of the structure was used. In this approach the observed deformation was considered and its influence on the current structural behavior was investigated by the nonlinear analysis.

Results of the static analysis of the steel structure with the current properties and conclusions from the assessment of its reliability can be briefly summarized as follows:

– Current behaviour of the roof trusses in the 15 and 16 axes (an area of the fire) is behind the limits of the allowable stresses and deflections. Consequently, the roof trusses do not fulfil criteria of the reliability defined by the ultimate limit states as well as by the serviceability limit states.

– Columns fulfil conditions of reliability in the case, that the local damaged (buckled) web of the column in the 16 axis will be reinforced.

On the basis of the mentioned results characterizing the current technical state of the bearing steel structure of the hall after the fire, the following arrangements for an achievement the reliability of the structure and for an assurance of the safe operation are needed:

– Strengthening of a locally failed web of the column in the 16 axis by the addition of transversal stiffeners.

– Stabilization and disburdening of the roof truss tie-beams in the 15 and 16 axes by means of the placement of the temporary supporting columns under the purlines in the places between the 14-15, 15-16 and 16-17 axes. A temporary supporting system with the damping members may be situated on the bridge cranes.

– Removing of the roof porous concrete panels between the 14-15, 15-16 and 16-17 axes (replace them by a light-weight sandwich roof cladding).

– Strengthening of the top chords of the trusses.

Implementation of the renewal

The following conditions for the implementation of the strengthening of the top chords of the truss ties-beams were ordained:

– A temporary supporting system with the brace struts are situated on the bridge cranes of lifting capacities 20 t and 50 t.

– Support the purlines and activate supporting system as is shown in Fig. 8.

– A minimum resistance of each of the brace struts is 70 kN.

– In order to eliminate the influence of vibrations of cranes brace struts are situated on the elastic bases.

– The top chords of the truss tie-beams are in the places of deviations stabilized by brace struts situated in the joints of trusses (Fig. 8). Brace struts are connected to joints of trusses by wells.

– After an implementation of the mentioned measures the necessary strengthening of the top chords of the trusses can be performed (Fig. 9).

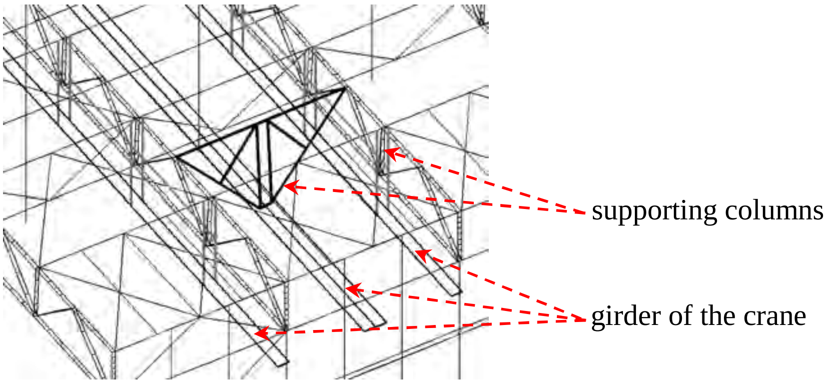


Fig. 8. Scheme of the purlines supporting by the brace struts situated on the girder of the bridge crane

Fig. 9. The strengthened top chord of the truss tie-beam (in the area over the fire)

Design of steel structures to the effects of fire

Fire resistance of buildings with a steel bearing structure can be enhanced in several ways: by fire protective coatings, by the application of combined steel-concrete members, by increasing of cross-sections or by using the fire-resistant steel (steel strength with increased resistance at high temperatures). Several authors investigated behaviour of steel structures directly during a fire [7], [8].

Fire resistance of the steel structures is determined according to EN 1993-1-2 [6] and related standards and appointments. Fire resistance is expressed by the time for which is guaranteed preservation of the stability and resistance of the structure. The intense heating of a steel structural member occur during the fire, as well as the decrease of its material strength characteristics, particularly decrease of the yield strength, ultimate strength and Young's modulus E. If the decrease of yield stress due to fire is high, which means that its value is less than the immediate working stress, so the structural member deform or break. The temperature at which deformation and/or break occurs, is the critical temperature. This temperature is achieved for the usual types of structural steels at approximately 550 °C, but may also be different depending on the dimensions of a structural member. At this temperature the steel maintains only about 60 % of the initial yield stress, compared with normal temperature.

For the determination of the carrying capacity of structures exposed to the effects of fire, are in [6] defined individual material characteristics ($X_{d,fi}$):

$$X_{d,fi} = \frac{k_\theta \cdot X_k}{\gamma_{M,fi}} \tag{1}$$

where X_k is the corresponding material characteristic at normal temperature, k_θ is the reduction factor applicable for this characteristic and material temperature at the fire and $\gamma_{M,fi}$ is the reliability factor for the given material characteristic and a fire situation.

For the individual strength characteristics at enhanced temperatures available are in [6] reduction factors that characterize decrease of the corresponding material characteristic (Fig. 10):

- reduction factor for the yield strength:

$$k_{y,\theta} = {f_{y,\theta}}/{f_y} \tag{2}$$

– reduction factor for limit of proportionality:

$$k_{p,\theta} = f_{p,\theta} \big/ f_p \tag{3}$$

– reduction factor for Young's modulus:

$$k_{E,\theta} = E_{a,\theta} \big/ E_a \tag{4}$$

The values of these factors are listed in Table 3.1 of the standard [6] and are valid for a low carbon structural steel.

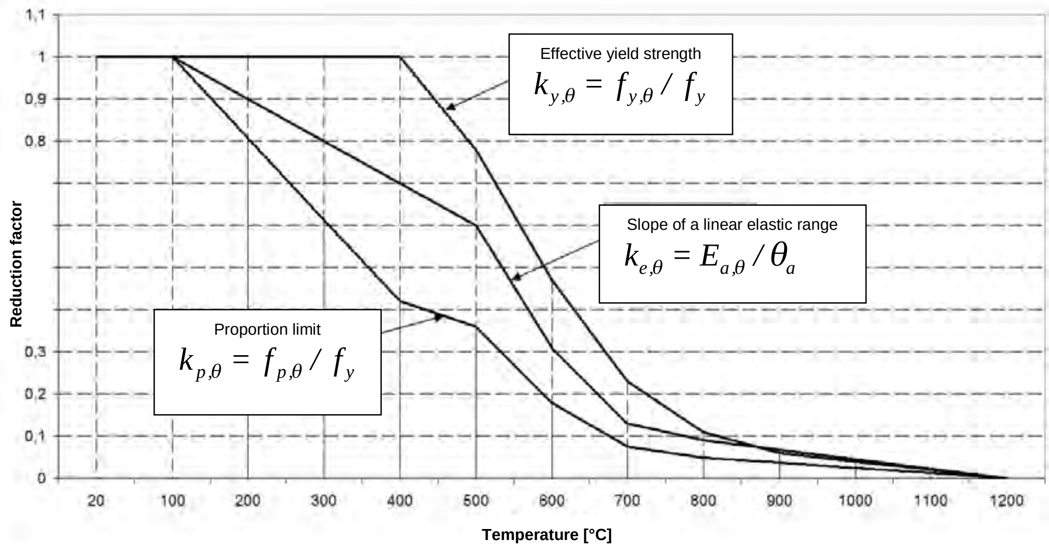


Fig. 10. Reduction factors of the material characteristics of the steel at enhanced temperatures

Study of resistance of steel structure at the time of fire

For the analysis of the structure at the time of the fire any of the following methods can be used:
– global analysis of the structure;
– analysis of the part of the structure;
– analysis of the member.

The assessment of fire resistance of steel structure in three areas can be implemented:
– the area of resistance;
– the temperature field;
– the time area.

In the area of resistance the structure after a certain period of a temperature exposure is assessed. In the field of temperature and time the temperature of the material after a certain period of a fire exposure with respect to the critical temperature of the material is assessed. The critical temperature of the material is calculated using the standard analytical formulas or by an iterative process.

In order to compare the behaviour and the resistance of the steel structure obtained by theoretical approaches with this determined on the basis of the real state at the fire the study of the resistance of steel structure at fire has been carried out.

In the global analysis of the structure the standard temperature curve ISO 834 describing the effect of the fire (Fig. 11) was considered. The temperature acting on the individual members of the structure from all sides was assumed and the fireproof coating was not considered.

In the global analysis of the structure the solution based on the temperature-time domain was considered. Consequently, the time at which the temperature of the material was assessed with respect to the critical temperature of the material was ascertained.

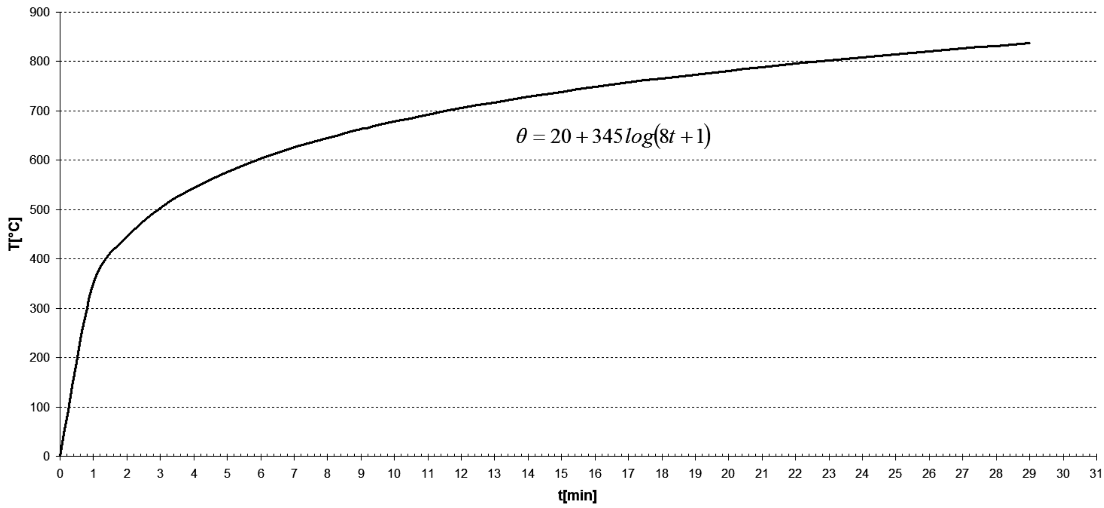


Fig. 11. Standard temperature curve ISO 834

Resistance of the bars of the truss tie-beam of the roof steel structure depending up the critical temperature of the material is shown in Fig. 12.

a) Temperature-time area: 16 minutes *b) Temperature-time area: 20 minutes*

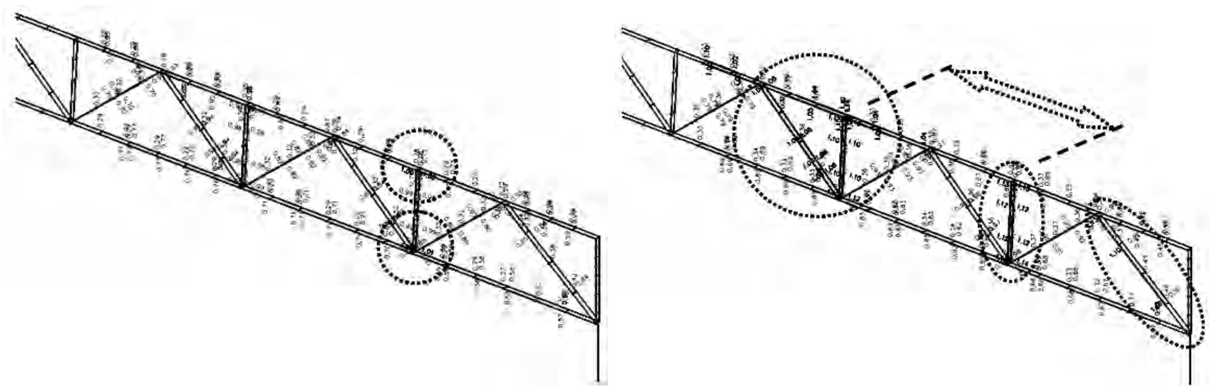


Fig. 12. Assessment of the individual members of the steel truss tie-beam in the temperature-time area with respect to the critical temperature of the material, with the marked members which have reached temperatures higher than the critical temperature, deformation zone is marked (Fig. 3a)

Resistance of the bars of the truss tie-beam of the roof steel structure monitored depending up the time influence of the fire is shown in Fig. 13 and Fig. 14. Resistance of the structure depending up the reached limit reduced material-mechanical properties of the steel was ascertained ($E_{red,f}$, Fig. 10).

a) Resistance: 9 minutes and 15 seconds *b) Resistance: 10 minutes*

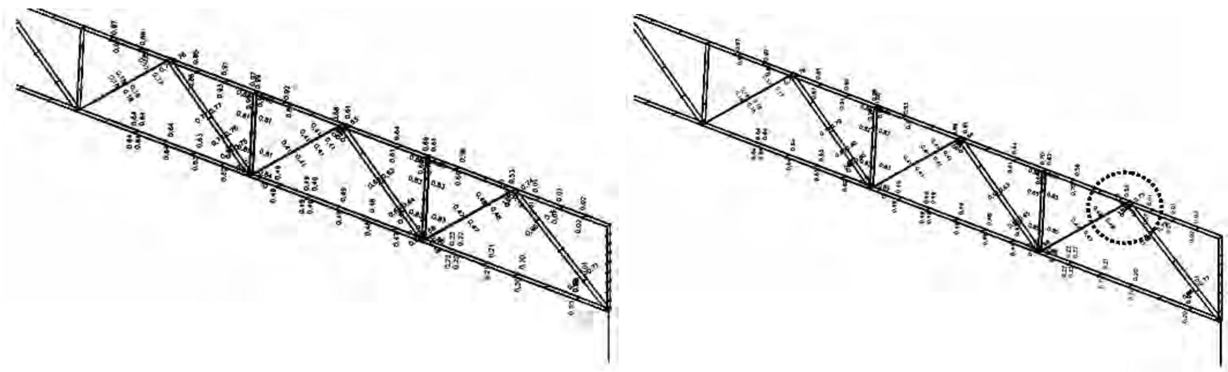


Fig. 13. Resistance of the individual members of the steel truss tie-beam depending on the time, where the members with the unsatisfactory resistance are marked, deformation zone is marked (Fig. 3a)

a) Resistance: 15 minutes *bd) Resistance: 20 minutes*

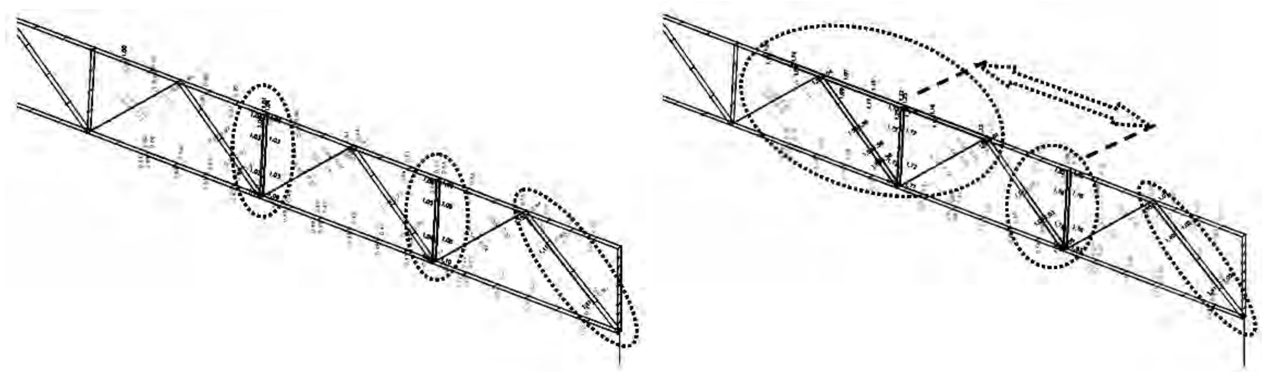


Fig. 14. Resistance of the individual members of the steel truss tie-beam depending on the time, where the members with the unsatisfactory resistance are marked, deformation zone is marked (Fig. 3a)

On the basis of the analyses, it can be alleged that the resultant time of the resistance of the steel truss tie-beam as well as results obtained by the solution in the temperature-time area are comparable with those determined on the basis of the real behaviour of the steel roof truss tie-beams at the fire, when the direct flame intensity affected on them was of approximately 10 minutes. Buckling of the top chord of the truss tie-beam (Fig. 3a, Fig. 14b and Fig. 12b) confirms the correctness of computational models and accuracy of numerical analyses used in this study.

Conclusion

An analysis and assessment of the reliability of the structure after fire belong to demanding engineering tasks. These are based on the survey and diagnostics of a current state of the structure affected by fire and on the experimental verification together with the numerical modelling of the current material, geometric and stiffness properties of the structure and its static analysis. Currently available sophisticated software products based on advanced numerical methods, allow complex modelling of properties of structures under the action of various physical fields, which exponentiate a realistic characterization of their behaviour under defined conditions. In the article we have tried to briefly describe the options for a comprehensive approach to modelling and analysis of steel structures damaged by the effects of fire.

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