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Research paper

Fire resistance evaluation for a steel hall transverse frame depending on the simplification degree of the computational model applied

Mariusz Maślak[1], Michał Pazdanowski[2], Maciej Suchodoła[3], Piotr Wozniczka[4]

Abstract: It is presented in detail how the selection of a structural model describing the behaviour of a steel hall transverse frame when subject to fire exposure in a more or less complex way may affect the fire resistance evaluation for such a frame. In the examples compiled in this paper the same typical one-aisle and single-story steel hall is subjected to simulated fire action, each time following the same fire development scenario. A resultant fire resistance is identified individually in each case, using various computational models, on an appropriate static equilibrium path obtained numerically. The resulting estimates vary, not only in the quantitative sense, but also in terms of their qualitative interpretation. It is shown that the greater the simplification of the model used, the more overstated the estimated fire resistance is in relation to its real value. Such an overestimation seems to be dangerous to the user, as it gives him an illusory but formally unjustified sense of the guaranteed safety level.

Keywords: steel hall, fire resistance, structural model, static equilibrium path, safety evaluation

[1]Prof., PhD., Eng., Cracow University of Technology, Faculty of Civil Engineering, Warszawska 24, 31-155 Cracow, Poland, e-mail: mmaslak@pk.edu.pl, ORCID: 0000-0003-3592-429X
[2]PhD., Eng., Cracow University of Technology, Faculty of Civil Engineering, Warszawska 24, 31-155 Cracow, Poland, e-mail: michal@l5.pk.edu.pl, ORCID: 0000-0003-1344-1342
[3]PhD., Eng., Cracow University of Technology, Faculty of Civil Engineering, Warszawska 24, 31-155 Cracow, Poland, e-mail: maciej.suchodola@pk.edu.pl, ORCID: 0000-0003-4972-597X
[4]PhD., Eng., Cracow University of Technology, Faculty of Civil Engineering, Warszawska 24, 31-155 Cracow, Poland, e-mail: pwozniczka@pk.edu.pl, ORCID: 0000-0002-5471-9526

1. Introduction

Fire resistance of a steel hall transverse frame is usually interpreted as the time during which such a frame would be able to safely resist the loads applied to it, including the internal forces generated thermally due to the direct or indirect fire influence. In order to obtain a reliable estimate of this resistance one should conduct a precise analysis on the 3D model of the bearing structure of considered hall or, if the available computer resources exclude this option, determine this value approximately, after detailed analysis of only a single frame formally isolated from the whole structural system. The basic goal of this article is to show that the sought resistance, even though it has been calculated based on the analysis of the same hall subjected to the same fire development scenario initially assumed, may vary significantly in value depending on the model selected to represent the considered structure. In general, the fire resistance forecast for a steel frame is determined numerically based on a bar model of all structural components, subject to the assumption that the temperature of these components is evened out not only in the respective cross-sections but also along their whole lengths. In this analysis we intend to verify how the selection of a particular static scheme affects the resultant fire resistance. In order to do that in our study a selected typical one-bay single-story steel hall frame is subjected to a simulated fire action. However, its structural response is modelled each time in a different, more or less complex, way. In each of the considered cases the final fire resistance, specified for the same frame modelled in a different way, is identified on the equilibrium path corresponding to that model. Thus it is measured by the time elapsing between fire initiation and the moment when the displacements authoritative for the performed analysis begin to rapidly increase. The calculations have been conducted using SAFIR [1] computer code.

2. Frame considered in the examples and fire development scenario

A single transverse frame of a typical steel hall, having the geometry, dimensions and cross sections of columns and beams as depicted in Fig. 1 has been selected to perform the comparative analysis. All frame components are assumed to be made of S235 steel. It is assumed, that in two neighbouring bays directly adjacent to the considered frame

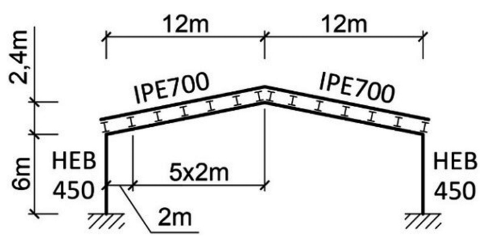


Fig. 1. Dimensions of the frame analyzed in the example. Support conditions and bar bracing modes differ based on the assumptions of the model presented in the text

a fully developed fire has been initiated, such that during each moment of its action on the frame the temperature of the fire plume is equilibrated in the whole fire compartment and increases in time following the conventional scenario of the standard fire [2].

It has also been assumed that both columns and beams of the frame have been evenly heated around the whole perimeter of the cross section (i.e. on all four sides). In the authors' opinion, when a lightweight sheathing made of sandwich plates or wall trays common in the modern steel halls is used, consideration of a three sides heating scheme as an alternative to the model proposed in calculations presented here, does not find formal justification. Due to the various ratios of the heated perimeter to the surface area of thermally uninsulated frame components, those components heat up at different speeds [3]. Uneven distribution of steel temperature in the cross-sections of beam and columns during each moment of a fire has been accounted for, however under assumption, that this distribution remained constant along the whole length of respective structural component. The characteristics of particular thermal actions related to selected moments in time are listed in the Table 1 [4], where minimum and maximum values of steel temperature determined for a given cross section are presented.

Table 1. Steel temperature in the beam and columns of the considered frame after several fire exposure times

Fire duration [s]	Temperature min/max [°C]		
	Fire plume Θ_g	Beam (IPE700) Θ_a^{beam}	Columns (HEB450) Θ_a^{column}
180	502	104/135	77/128
360	603	240/300	179/286
540	663	380/452	292/434
720	705	499/567	402/516
900	738	591/647	498/633

These data are complemented here by a sample distribution of steel temperature in the beam cross section obtained after 660 seconds of a standard fire exposure, depicted in Fig. 2. One may easily observe there, that relatively thin and slender web is heated significantly faster than the usually thicker flanges. The quantitative differences in steel temperature observed in those parts of cross section at that moment of fire are undoubtedly significant. However, the most important for the fire resistance of the considered structure is the fact that the distribution of temperature is symmetrical with respect to both vertical and horizontal axis of the cross section. Thus additional internal forces amplifying bending are not induced in the frame. Let us note also the fact, that in evenly heated I shaped steel structural members usually the zones adjacent to the transition area between flange and web are the slowest heated, as the material mass in those zones is the biggest.

In the analysis conducted by the authors it is also assumed, that both the left and right hand side end plate beam-to-column joints remain completely rigid during the whole duration of a fire. Of course the assumption of this type constitutes a significant simplification

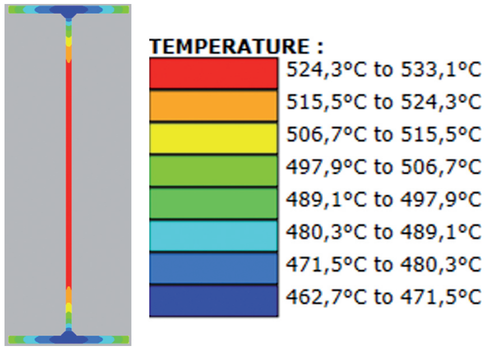


Fig. 2. Distribution of the steel temperature in the uninsulated frame beam, determined after 660 seconds of fire exposure conforming to the standard fire development scenario

of the real phenomenon, and usually yields an overestimated, and thus overly optimistic assessment of the fire resistance sought [5]. It is commonly known that joint rigidity decreases with increasing temperature affecting such a joint when subjected to fire conditions. The results of such overestimating are analyzed by the authors in many works in both qualitative and quantitative aspects, however, in this paper this phenomenon is devoid of significant cognitive importance, as it is dealt here with an analysis of a more general nature, allowing for a comparative evaluation of whether, and if yes then how and to what extent, the level of simplification adopted by the person conducting the evaluation in the computational model applied in practice affects the final value of the considered resistance derived via calculations, instead of accurate determination of fire resistance for a specific hall.

Detailed calculations have been performed using special 3-node bar elements. Each element of this type had 7 degrees of freedom in the end nodes (3 translational, 3 rotational and 1 additional to account for warping) and also 1 degree of freedom in the middle node (to account for nonlinear phenomena in axial deformation). The evolution of steel properties when subject to fire exposure has been assumed according to the EN1993-1-2 [6] standard recommendations. The structure has been treated as subjected to dynamic loads to avoid any potential instabilities during iterative calculations. The solution was found using the Newmark's method.

3. Juxtaposition and description of the considered structural models

The transverse frame described above and subjected to numerically simulated fire exposure of intensity increasing in time is in the following analysis represented by several structural models of varying complexity. These are as follows [7]:

– Models denoted by symbols A1 and A2 – understood as fully 2D models. In those models only the deformations occurring in frame plane are accounted for. In such case usually flexural in-plane buckling of beam or column, i.e. global instability of an element about the "strong" axis of its cross section, determines frame bearing capacity. Model A1 represents a frame with fully rigid supports while model A2 represents a frame with pinned supports (Fig. 3).

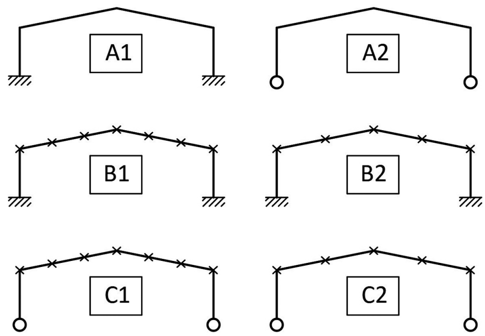


Fig. 3. Structural models of the analyzed frame defined for groups A, B and C

- Model B1 – frame with rigid supports. Deformation in the plane perpendicular to frame plane, including both classical flexural out-of-plane buckling as well as lateral-torsional buckling of beam or column is possible. Thus the global instability of an element about the "weak" axis of its cross section may determine its bearing capacity. Such a set of transverse roof bracings has been assumed, for which every other purlin constitutes a support stabilizing the beam at its support point against buckling out of the frame plane (Fig. 3).
- Model B2 – analogous to model B1, but in this case, due to different placement of transverse roof bracings only every third purlin stabilizes the beam in the frame plane (Fig. 3).
- Model C1 – analogous to the model B1, but for a frame with pinned supports.
- Model C2 – analogous to the model B2, but for a frame with pinned supports.
- Model C3 – analogous to the model C1, but for a frame made of S355 steel.
- Model D1 – analogous to the model C1, but taking into account the offset of purlins bracing the beam with respect to the longitudinal axis of the beam (Fig. 4). This offset has been assumed at 35 cm as approximately half of the beam height (a rather conservative assumption).

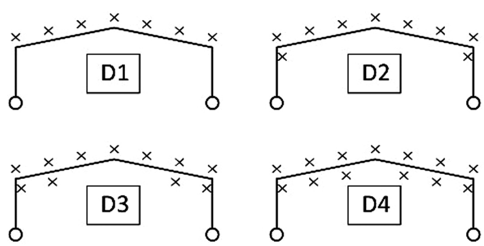


Fig. 4. Models of the considered frame related to group D

– Models D2, D3 and D4 – analogous to the model D1, but taking into account application of anti-torsion braces in selected cross-sections of the frame (different in each case), stabilizing beam bottom flange in the frame plane (Fig. 4). The size of this offset, determined with respect to the beam longitudinal axis has been assumed as equal to 35 cm.
– Model E1 – model of a single transverse frame with pinned supports. Action of („hot") purlins heated in fire is simulated here by bar elements of real length and flexural stiffness (reduced due to elevated steel temperature), supported without taking into account the offset accounted for in analogous models belonging to group D. External supports of those purlins, located along the axes of adjacent transverse frames are fully articulated, thus the possibility of support translation in any direction is excluded. Only rotations are possible. In the considerations presented here it has been assumed, that purlins have been made of HEA 140 section (Fig. 5).

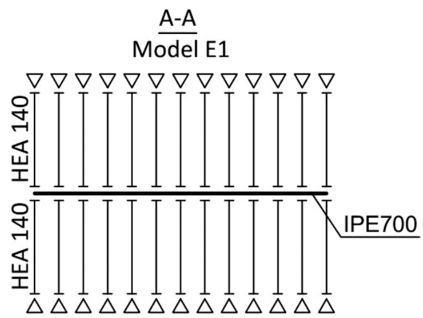


Fig. 5. View from above of transverse frame static scheme analysed as model E1

– Model E2 – analogous to the model E1, but all the external supports of purlins are modelled as elastically flexible in the direction determined by the longitudinal axis of respective purlin (Fig. 6). The flexibility parameter $k = 1730$ kN/m is assumed to be the same for all purlins and does not change during fire action (value of this parameter

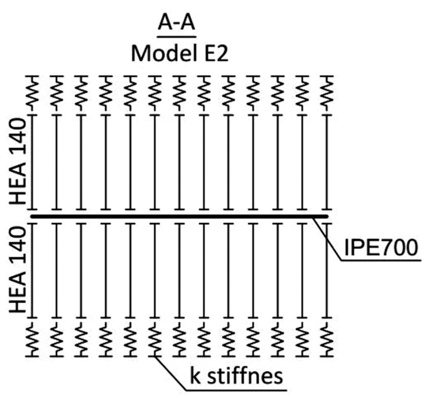


Fig. 6. View from above of transverse frame static scheme analysed as models E3 and E4

has been transferred to this model directly from models E3 and E4, described below). This flexibility is interpreted here as a measure of rigidity of a potential transverse bracing, bracing adjacent beams of the frame. Thus the possible effect of pushing of the frame beams adjacent to the one under consideration (at low purlin deflections) and subsequently pulling of the same beams (when the deflections of purlins resting on these beams become sufficiently large) is accounted for. Purlin cross sections are identical as in the model E1. An assumption of the invariant value of parameter k, remaining constant during the whole period of fire exposure, means that the thermal action is accounted for here only through the reduction of flexural stiffness of the individual purlins. This is equivalent to simplified treatment of the modelled bracing as the so called "cold bracing", i.e. bracing completely unaffected by the fire action, thus fitted with bars perfectly thermally insulated.

– Models E3 and E4 – analogous to the model E2, but this time the extreme purlins are pinned inflexibly, and the remaining purlins are supported as in model E2 (Fig. 7). This way the stabilizing influence of wall bracing is accounted for. The models differ only in the assumed stiffness of the supports (in model E3 it is identical as in model E2, while in model E4 it is halved). The reduction in assumed support stiffness models purlin weakening induced by the increased steel temperature. The value of parameter k applied in these models has been assumed as to approximately correspond to the real flexibility of the transverse roof slope bracing against potential buckling in the bracing plane at the mid span of the analysed frame. For calculations it has been assumed, that the frames of the hall, located 6 meters apart, are transversally braced at the axes of girders by a truss of the X type made of angle iron L50x5. In addition, it has been assumed that the columns of this bracing are located 2 meters apart (i.e. their axes coincide with purlin axes). It has been also assumed, that the external horizontal loads applied at the nodes of the bracing are effectively transferred only by its cross braces (this means in turn, that the bracing is modeled as a Pratt truss). Similarly to the E2 model, when determining

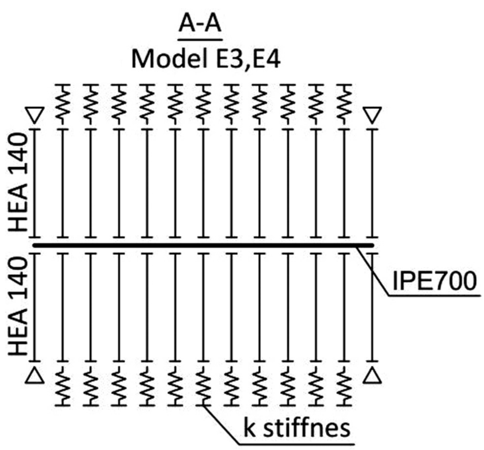


Fig. 7. Dimensions of the frame analyzed in the example. Support conditions and bar bracing modes differ based on the assumptions of the model presented in the text

the value of the factor k, the scenario that did not take into account any impact of the fire temperature on the bracing has been treated as the authoritative.

– Model F1 – analogous to the model E1 but this time instead of single transverse frame a system of two neighbouring frames is considered (Fig. 8). Thus in this model a fire limited to three adjacent bays is considered (in comparison to all the previous models where only two bays of the hall were affected by the fire).

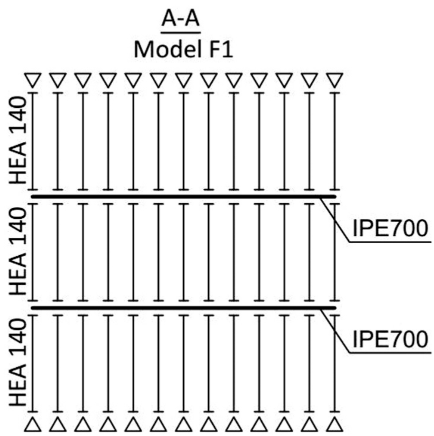


Fig. 8. Two frame system analysed in detail in model F1

– Model F2 – analogous to the model E3, but instead of a single transverse frame a system of two neighboring frames is considered (Fig. 9).

In each of considered models a nonlinear stress-strain relationship in structural steel affected by direct fire action and appropriate reduction of yield limit induced by increasing ambient temperature during fire are accounted for.

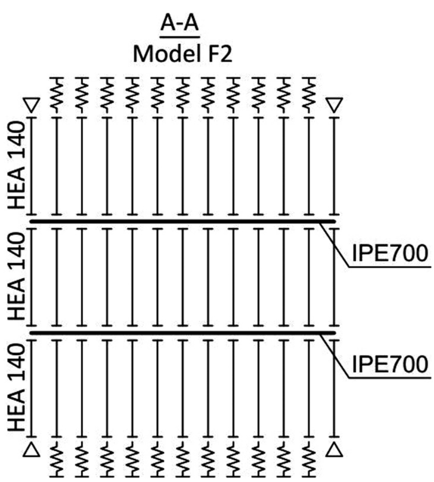


Fig. 9. Two frame system analyzed in detail in model F2

4. Comparative analysis of the obtained results

In each of the considered cases during the initial phase of fire exposure, due to the thermal expansion of steel the columns increasing in length push the beam in an upward direction and at the same time the expanding beam pushes the columns outwards. However, with passing time the beam gradually weakened by the increasing temperature sags more and more. This sag effectively eliminates the initial push up of the beam. As a result the columns initially pushed outwards now, at the relatively large beam sag increasing with increasing fire intensity, are pulled inwards. The change in the direction of displacements occurs in general quite abruptly, thus allowing for an unambiguous determination of fire exposure time related to the fire resistance of the considered frame.

The detailed analysis of equilibrium paths depicted in Fig. 10, 11 leads to the following estimates of the fire resistance $\tau_{fi,R}$ [s] sought: for the model A1 – $\tau_{fi,R}$ = 1088 s, for the model B1 – $\tau_{fi,R}$ = 997 s, for the model B2 – $\tau_{fi,R}$ = 899 s. The quantitative differences identified in this juxtaposition and measured in absolute numbers seem to be rather small, however in the percentage terms they appear to be significant. It has to be underlined as well, that the flexural in plane buckling of the column or beam seems to be the destruction mechanism in the model A1, while in general flexural out of plane buckling or more

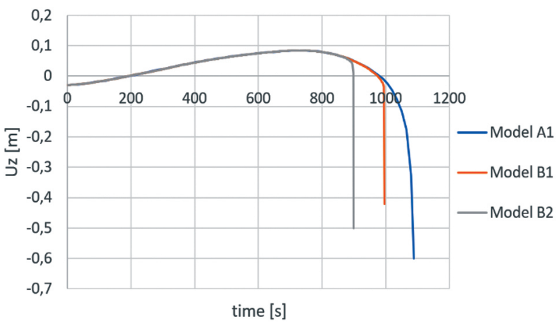


Fig. 10. Static equilibrium paths obtained at the ridge for vertical displacements of the frame with inflexible supports (A and B group models)

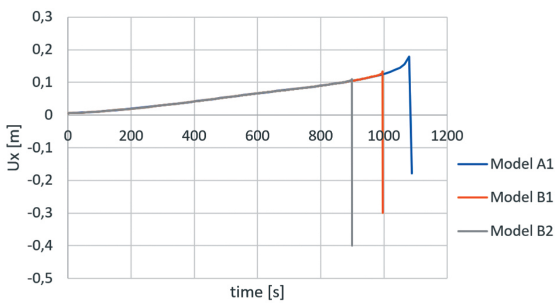


Fig. 11. Dimensions of the frame analyzed in the example. Support conditions and bar bracing modes differ based on the assumptions of the model presented in the text

probably lateral torsional buckling seem to be the destruction mechanisms in models B1 and B2.

The results obtained for the frame with pinned supports (models C1 and C2), analogous to those previously presented in Fig. 10 are shown in detail in Fig. 12. They are as follows: for the model A2 – $\tau_{fi,R}$ = 1027 s, for the model C1 – $\tau_{fi,R}$ = 941 s, for the model C2 – $\tau_{fi,R}$ = 882 s. This figure is juxtaposed here with Fig. 13 to show the difference in the estimated fire resistance value in the case, when an identical frame is considered, but this time with a beam and columns made of S355 steel, characterized by significantly higher strength. The fire resistance obtained for this frame is therefore higher (in particular: for the model A2 – $\tau_{fi,R}$ = 1267 s and for the model C2 – $\tau_{fi,R}$ = 1068 s, respectively).

Next group of analysed cases refers to the frame with pinned supports but including offsets of various types (models D1, D2, D3 and D4). In the model D1 only an offset at the beam to purlin joint is taken into account, while in models D2, D3 and D4 an additional application of anti-torsion braces in various cross sections has been accounted for (Fig. 4). The static equilibrium paths obtained numerically under such boundary conditions

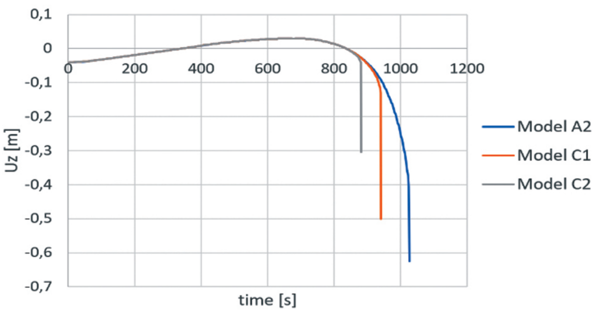


Fig. 12. Static equilibrium paths obtained at the ridge for vertical displacements of the frame with pinned supports (A2, C1 and C2 models) in the case of beam and columns made of S235 steel

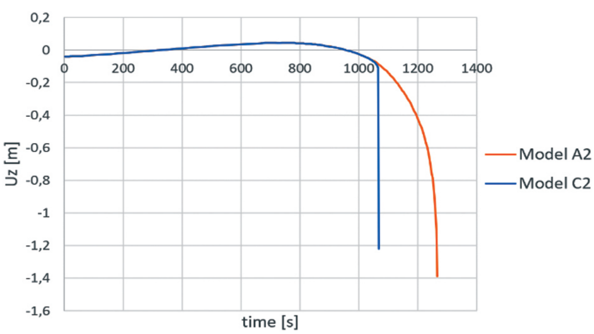


Fig. 13. Static equilibrium paths obtained at the ridge for vertical displacements of the frame with pinned supports (A2 and C2 models) in the case of beam and columns made of S355 steel

are juxtaposed in Fig. 14. The fire resistance obtained after application of D1 model ($\tau_{fi,R}$ = 900 s) proved to be significantly lower than obtained earlier for the model C1 ($\tau_{fi,R}$ = 941 s). This is the adverse result of an additional torsional moment acting on the beam. Application of appropriate anti-torsion braces efficiently negates this influence. In the model D2 ($\tau_{fi,R}$ = 910 s) this is visible to relatively low degree, as the braces have been applied only in the plane of columns. The stiffening phenomenon observed in models D3 ($\tau_{fi,R}$ = 986 s) and D4 ($\tau_{fi,R}$ = 990 s) equipped with significantly higher number of braces becomes so pronounced, however, that the fire resistance obtained in those cases is significantly higher than the one obtained for the model C1.

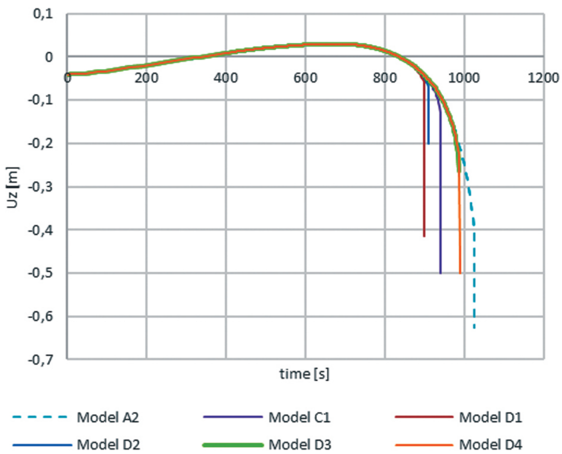


Fig. 14. Static equilibrium paths obtained at the ridge for vertical displacements of the frame with pinned supports after taking into account offsets of various types (models D1–D4 of Fig. 4)

The equilibrium paths obtained for the remaining models, of the group E (models E1, E2, E3 and E4), do not yield substantial differences in the resultant fire resistance, as the following results have been obtained here: for the model E1 – $\tau_{fi,R}$ = 1045 s, for the model E2 – $\tau_{fi,R}$ = 1035 s, for the model E3 – $\tau_{fi,R}$ = 1039 s, and finally for the model E4 – $\tau_{fi,R}$ = 1018 s. These estimates are quite close to analogous estimates obtained after application of model A2, for which $\tau_{fi,R}$ = 1027 s (Fig. 15). This means, that taking into account during calculations an additional action exerted on the frame subjected to direct fire action by “hot” purlins supported on it may be disregarded without substantially affecting the final result.

The shape of equilibrium paths pertaining to frame models belonging to group E significantly differs from the shape of path obtained earlier for model A2. As can be observed in Fig. 15, in these models the “hot” purlins, in spite of substantial weakening by fire, at sufficiently large vertical displacements of the beam efficiently resist the further rapid build-up of these displacements. This phenomenon obviously may not be identified after application of the model A2. However, in the analysed example this phenomenon revealed

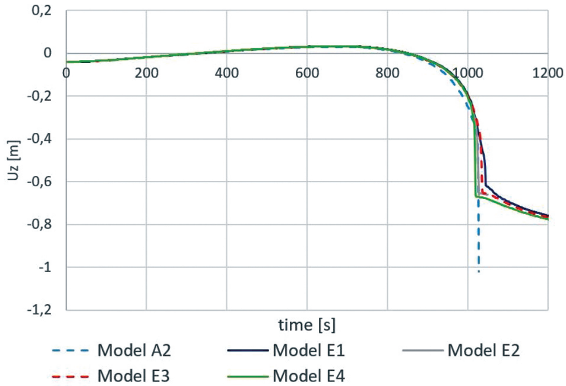


Fig. 15. Fire resistance estimates for a transverse frame exposed to fire actions obtained after application of models E1, E2, E3 and E4 juxtaposed with analogous estimate obtained for model A2. Static equilibrium paths pertain to the vertical displacement at ridge

itself only after the vertical displacements reached the magnitude of approximately 65 cm, so basically in the post-critical regime of the frame behaviour.

Purlin behaviour under fire conditions, identified after application of models belonging to the group E, is depicted in Fig. 16. It is clearly visible, that ever hotter purlins due to the restrained capability of thermal elongation during the initial phase of fire are compressed with increasing force, and tend to push the supports in outward direction. However, due to the increasing displacements induced by decreasing flexural rigidity this compression decreases gradually, finally to vanish completely. Then, at the decreasing bending resistance the tensile force in purlins becomes dominant. This means that at this stage the purlin begins to behave as a flexible tendon (the so called catenary effect). By juxtaposing Fig. 15 and Fig. 16 one may easily observe, that the rapid increase in the vertical displacements in the frame ridge is accompanied by a jump in the tensile force acting in purlin. Let us also

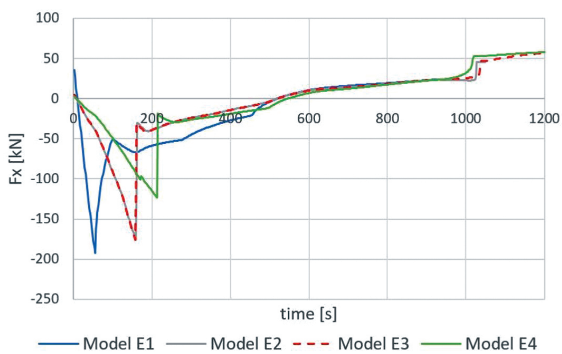


Fig. 16. Changes in the axial force in purlins affected by fire exposure, identified after application of models belonging to the group E. Negative values denote compression, while positive denote tension

note the fact, that the higher flexibility of supports assumed during modelling results in substantially lower maximum compressive force in the purlin, and also slower increases in its deflection.

Static equilibrium paths obtained for the models belonging to the group F and related to the vertical displacements at the ridge of considered frame are depicted in Fig. 17. The fire resistance estimated based on those models is equal to: for model F1 – $\tau_{fi,R}$ = 996 s and for model F2 – $\tau_{fi,R}$ = 1015 s, respectively. Thus those estimates are more conservative than those determined based on application of models belonging to the group E, and at the same time a little bit more optimistic, than the results obtained after application of model C1 ($\tau_{fi,R}$ = 941 s). Let us note, that in the example considered here, when model F1 is applied, the phenomenon of purlins supporting the sagging beam did not occur at all. The behaviour of purlins in the models belonging to group F is analogous to the behaviour observed on purlins described by the models belonging to group E (Fig. 18).

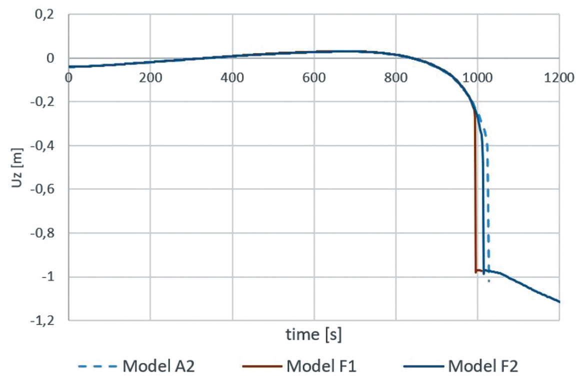


Fig. 17. Estimates of fire resistance for transverse frame exposed to fire action obtained after application of models F1 and F2 juxtaposed with analogous estimate obtained for model A2. Static equilibrium paths refer to the vertical displacement in ridge

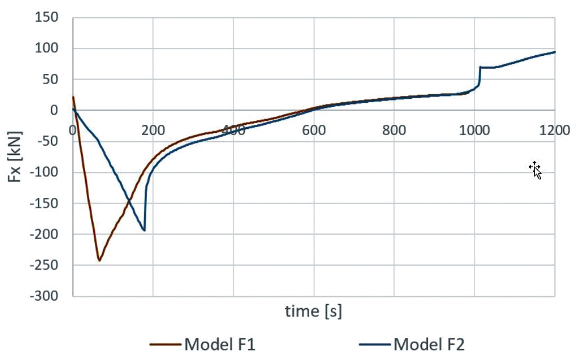
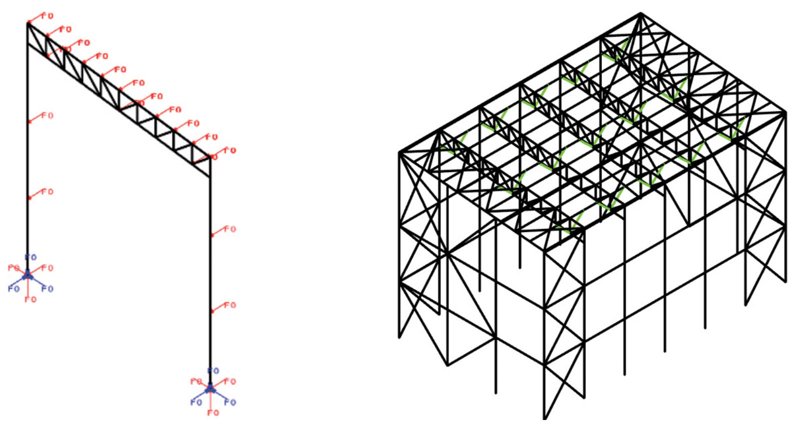


Fig. 18. Changes in axial force in purlins subjected to fire action, identified after application of models belonging to the group F. Negative values denote compression, while positive denote tension

5. Credibility of the model with single transverse frame

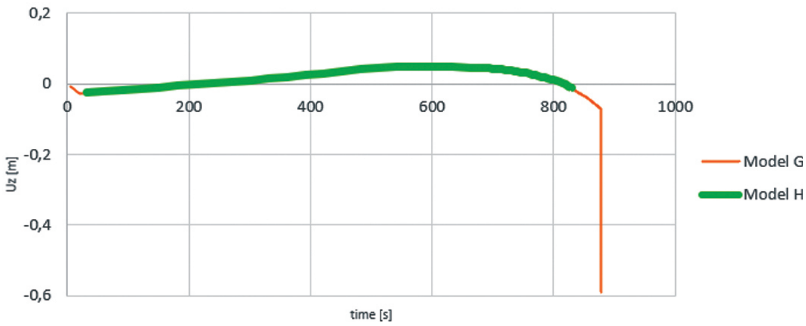
The analysis presented above seems to indicate unequivocally, that appropriate modelling of the boundary conditions has, in the case of application of simple computational model with single transverse frame, a direct and significant influence on the obtained value of fire resistance sought. The general rule seems to be, that the higher the degree of model simplification the more overestimated the final fire resistance is with respect to the real value. Thus the desire to simplify the calculations in this situation seems to be burdened by high risk, as the result obtained by the simulation may impart a completely unjustified sense of false security upon the hall user. However, the basic question remains, how credible the result of even the most accurate estimate may be to the user, if this result is supported only by a simple modelling with application of a single transverse frame.

The response, to be authoritative, requires at least a comparison of static equilibrium paths obtained for the same hall and determined with application of a full 3D model taking into account the spatial interactions between structural components and a reduced 2D model consisting of a single transverse frame formally isolated from the whole structure and with relatively carefully applied boundary conditions. The results of such comparison are depicted in Fig. 19 [7]. A single frame with truss girder depicted in Fig. 19a is in this case denoted as model G. The action of purlins and wall girts is in this model accounted for by application of appropriate supports. This simple model has been subsequently correlated with a full 3D model depicted in Fig. 19b and denoted as model H. The static equilibrium paths obtained for both models are juxtaposed in Fig. 19c. It is clearly visible, that the equilibrium path obtained in this juxtaposition for a complex model H is in general identical to the path obtained independently after analysis of a simple model G. There is a catch, however. The fire resistance obtained after application of the model G ($\tau_{fi,R}$ = 878 s) proved to be significantly higher, than the one obtained after application of the model H



(a) (b)

Fig. 19



(c)

Fig. 19. Verification of the credibility of results obtained after application of simple computational models, including: a) model of a single transverse frame (model G), b) 3D model taking into account the spatial interactions between structural components of the hall (model H), c) juxtaposition of the corresponding static equilibrium paths related to vertical displacements

($\tau_{fi,R}$ = 829 s), this in turn means that such result should be treated as an undoubtedly overestimated value with respect to the real value of fire resistance, overestimating the safety level actually warranted to the user.

6. Conclusions

The considerations presented here fit within the wider scope of research conducted by the authors and pertaining to the credibility of various computational models, differing in complexity, applied to estimate the fire resistance of steel hall bearing structures [8, 9]. In the juxtapositions presented here we intend to show, that application to that purpose of simple models based on the single transverse frame with appropriately modelled boundary conditions usually leads to overly optimistic estimates, more or less overestimating the level of safety warranted to the user. Of course, sufficiently careful modelling of support conditions and taking into account spatial nature of the potential deformations occurring in a frame exposed to fire action results in the fire resistance estimated on a model converging to the real value. The final result of the analysis is highly affected by the formal inclusion in the model of offsets induced by the way the purlins are supported on the girder – this fact in general is neglected in the analyses. The purlins induce an additional torsion in the girder, and this in turn in the case of girder rigidity decreased by the thermal action of fire may substantially accelerate its loss of stability.

In the computational models analysed above the influence of potential substitute geometrical imperfections has been disregarded. The authors' research seems to indicate, that formal inclusion of these imperfections does not significantly affect the final estimate of fire resistance. Additional inclusion of wall girts in the analysis proved to be similarly insignificant. The destruction mode is the determining factor, and this is obviously determined by the support conditions defined in the model.

The question of selecting the optimum computational model for the analysis of fire resistance of a bar bearing structure in a steel hall, and especially sufficiently precise modelling of support conditions seems to be especially important if the complex 3D models, taking into account the 3D interactions between structural components, are selected for analysis [10]. One has to remember, that the rigidity of the structural components decreases with the increasing ambient temperature. This is accompanied by the increasing pliability of the joints, even those between columns and beams. The joints, modelled as nominally rigid, under fire conditions with passing time become partially flexible, and this should be accounted for in the developed model. Research conducted so far seems to indicate as well, that the final fire resistance of a steel frame is highly affected by the behaviour of purlins stiffening the beam [11, 12]. Those purlins, due to smaller cross section get heated much faster, than the beam or columns of the transverse frame. Thus relatively early, at monotonous increase of temperature stop supporting the beam and thus its increasing deflections, often determining the bearing capacity of the analysed frame is no longer restrained in any way. Sufficiently precise modelling of the whole bracing system in the hall seems to be similarly important, as its influence changes during fire action.

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Ocena odporności ogniowej ramy poprzecznej hali stalowej w zależności od stopnia uproszczenia zastosowanego modelu obliczeniowego

Słowa kluczowe: hala stalowa, odporność ogniowa, model konstrukcji, ścieżka równowagi statycznej, ocean bezpieczeństwa

Streszczenie:

Pokazano jak dobór schematu statycznego pojedynczej ramy poprzecznej typowego ustroju nośnego hali stalowej determinuje uzyskaną z obliczeń dla tej ramy wartość poszukiwanej odporności ogniowej. Odporność tę interpretuje się z reguły jako czas przez który badana rama w warunkach ekspozycji pożarowej zachowuje zdolność do bezpiecznego przenoszenia przyłożonych do niej obciążeń. W celach porównawczych, dla tego samego scenariusza rozwoju pożaru, odpowiadającego standardowym warunkom nagrzewania, i dla tej samej ramy, rozpatrzono odpowiadające sobie rozmaite modele obliczeniowe o różnym stopniu złożoności. Reprezentatywną miarą odporności na oddziaływanie monotonicznie narastającej w czasie pożaru temperatury elementów stalowych było w każdym z rozpatrywanych przypadków wyczerpanie możliwości efektywnego przenoszenia obciążeń, identyfikowane na ścieżce równowagi statycznej. Zestawienie i porównanie uzyskanych wyników doprowadziło do konstatacji, że im większy stopień uproszczenia zastosowanego modelu obliczeniowego tym bardziej zawyżone otrzymane na jego podstawie oszacowanie poszukiwanej odporności, przeszacowujące realnie gwarantowany użytkownikowi poziom bezpieczeństwa. Wykazano, że znaczący wpływ na finalny wynik analizy może mieć formalne uwzględnienie w zastosowanym modelu obliczeniowym mimośrodów wynikających ze sposobu oparcia płatwi na ryglu, co na ogół nie jest dostrzegane. Generują one bowiem dodatkowy moment skręcający, który w sytuacji zmniejszonej wskutek oddziaływania temperatury pożarowej sztywności rygla może przyspieszać jego utratę stateczności. Szczególne znaczenie dla wynikowej odporności ogniowej analizowanej ramy ma również zachowanie się w pożarze płatwi dachowych usztywniających rygiel. Płatwie te, z uwagi na mniejszy przekrój, nagrzewają się bowiem znacznie szybciej niż rygiel i słupy ramy poprzecznej. Stosunkowo wcześnie zatem, przy monotonicznym wzroście temperatury, przestają efektywnie podtrzymywać ten rygiel, w efekcie czego narastanie jego ugięcia, często decydujące o nośności ramy, nie jest już w żaden sposób hamowane. W rozważanych modelach formalnych pominięto wpływ potencjalnie możliwych imperfekcji geometrycznych, zarówno tych o charakterze globalnym jak i tych lokalnych. W prowadzonej przez autorów analizie nie wydaje się on bowiem mieć istotnego znaczenia. Nie uwzględniono również faktu narastania wraz z rozwojem pożaru podatności węzłów.

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