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PERFORMANCE-BASED ANALYSIS OF OLDER-TYPE LARGE-SPACE HALL IN FIRE

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The paper presents the example of performance-based analysis for the existing large-space steel structure raised in 1980s. Hall is used as a paper products warehouse. Advanced mechanical simulations are performed using Safir software. Factors that impact the final fire resistance of the structure are discussed. Local and global imperfections and possible ways of structure modelling are taken into account. For selected cases advanced fire scenario that considers both localised fire and possibility of further ignition of stored goods is prepared using Fire Dynamics Simulation software. The results obtained indicate that added imperfections have little impact on the fire resistance of the structure and older-type steel hall roof without any fire protection could survive 30 minutes of fire. Main failure modes and values of structure's deflections are also presented. Finally, performed simulations show that even for large-space structure the flashover is possible in some special cases.

Keywords: large-space hall, steel structure, fire, performance-based analysis

1. Introduction

Fire resistance of large-space steel buildings is among current interest of many researchers. Theoretical background and analytical equations that allow to estimate fire temperature distribution in case of localised fires for considered type of buildings are discussed in e.g. [2]. Computer modelling of localised fires in relation to results of full scale experiments is described in e.g. [12], while forecast of fire spreading is given in [8]. Finally, issues related to the mechanical analysis of a whole structures or substructures under the fire action are presented in [13], [10] and [6].
However, in most cases of the previously mentioned papers, some assumptions that require further investigation are made. Firstly, in typical performance-based analysis of surrounding equipment or

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stored goods is not considered. Secondly, advanced fire structural analysis usually deals with contemporary modern structures while a lot of older steel halls are still in use. Finally, in the recent works it is hard to find information about local and global imperfections that were set in FEM models of steel structures, except cases of simple separated members.

In this situation the performance-based analysis for real existing large-space steel structure that was raised in 1980s is presented and special attention is focused on:

- possible ways of structure modelling including different types of substructures and different boundary conditions,
- the impact of the local and global imperfections on the load bearing capacity of the structure in fire. If it is possible the failure modes are also recognized,
- estimating the fire resistance for an older-type structure,
- using the computational fluid dynamics model that considers both localised fire and possibility of further ignition of storaged goods.

2. STRUCTURE MODELLING

2.1. DESCRIPTION OF THE HALL

The considered steel hall is a one-storey building with basement under a part of the structure, used for paper industry. Object has been modified a few times since its erection. The building is 240m long and 72m wide and it is divided into three independent fire zones (Fig. 1). For one of the zones (no. 3, 72m by 72m and 15m height) type of occupancy was changed and it is now used as a cellulosic products warehouse. The storage height is very low and it is equal to 1m, however this still results in a very high nearly uniformly distributed fire load density of 2000 MJ/m^2. The described fire zone is equipped with 4 main gates (4m by 4m each) and with 27 roof glazings (3m by 6m each). The building doesn't have any sprinklers or fire ventilation. According to the local law prescriptive rules based on ISO curve are preferred. However, in special cases e.g. for older structures it is allowed to use performance based analysis. In such situation the consent of the fire brigade authorities is needed. Mandatory regulations are not clear and they are often the subject of discussion but for described hall it is assumed that the roof structure should resist 30 minutes of fire (R30) while the columns (classified as main structural elements) need to satisfy the requirement of class R120.

Structure of the hall consists of independent repeated segments with in plane dimensions 24m by 24m. Each segment is in fact a separated spatial truss. Top and bottom chords are made of hot-rolled channel sections. Vertical and diagonal bars are made of angle sections or double channel sections, folded in this way to obtain a shape similar to modern RHS sections. Steel used for top and bottom chords (18G2A) is close to nowadays steel grade S355 while for the rest of the structure steel (St3SX) is like modern grade S235. Main columns are designed as a box girders and their bases are fixed. In this situation the hall doesn't need any additional bracings. The building's envelope is made of sandwich panels. As a result of specific and flexible connection roof sheeting could be considered as independent from the main structure. Full description of the structure could be found in [5].

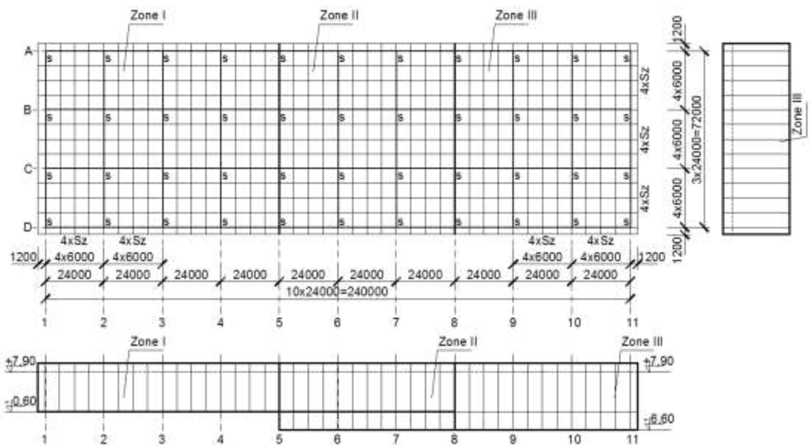


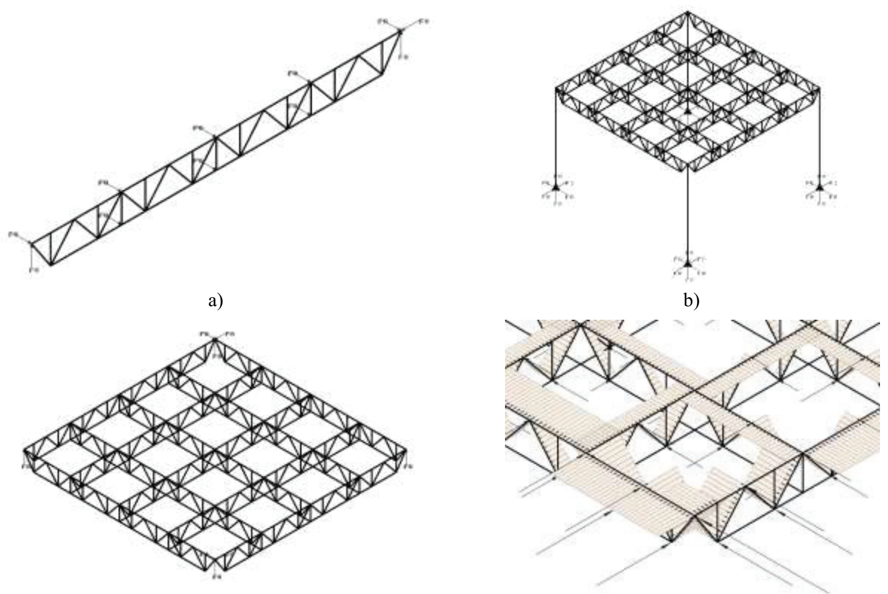
Fig. 1. Layout of the hall

2.2. COMPARISON OF DIFFERENT WAYS OF STRUCTURE MODELLING

For the hall described different ways of structure modelling are analysed. In this stage for comparison reasons values of temperature are defined according to ISO curve. Safir software [4] is used for calculation purposes. All simulations are performed using special beam elements previously validated against experimental results (in e.g. [7]) and temperature across the sections is not uniform. The following possibilities are considered:

- 2D models of selected trusses (marked as S1),
- 3D models of separate typical roof segment (marked as S2),
- 3D models of separate typical roof segment with fire protected columns (marked as S3).

For S1 models out-of-plane buckling is allowed and two further cases are taken into account: external truss without any horizontal restraint (models S1.1) and external truss with additional spring element modeled to imitate the influence of the adjacent structure (models marked as S1.2). Stiffness of the spring elements is evaluated for the temperature of 20°C and remains constant during the whole fire. This assumption is due to the limitations of the program used. For S2 category models horizontal displacements are possible for two supports. Another two supports are blocked to avoid the rotation of the whole structure. For S3 models both fire protected columns and spring elements are added. Stiffness is calculated in the same way as for the S1 case. However, as there are no bracings and the building is relatively high the value of this stiffness is low (K_z=1370kN/m).



a) b)

c) d)

Fig. 2. Types of analysed structural models: a) S1i0; b) S3i0 c) S2i0 d) S2ih with equivalent forces

This fact, together with quite computationally expensive model, is the reason why complete 3D model of the structure is not considered in this paper. All connections between diagonals, verticals and chords are defined as hinge connections. Point loads specified according to EN1991-1-2 are placed right in the nodes of the trusses. Reduction factor η_{fi} (in the meaning of clause 2.4.2 of EN 1993-1-2) is equal to 0,5. Views of described models with appropriate boundary conditions are presented in the Fig. 2a to Fig. 2c. In the second stage of analysis local and global imperfections are added to the previously listed structural models. Local horizontal imperfections are marked as "ih", local vertical imperfections are marked as "iv" and global imperfections are marked as "ig". There are no remarks in EN 1993-1-2 about the values of imperfections that should be used in the case of advanced mechanical analysis for structures subjected to fire action. In this situation these imperfections are calculated using EN 1993-1-1 and then replaced by equivalent forces. For channel and angle sections buckling curves "c" and "b" are selected. Therefore, for plastic analysis the values of initial bow imperfections are equal to 1/150 and 1/200 respectively. In e.g. equivalent forces added for S2ih model are presented in Fig. 2d. Disadvantages and potential objections to the method that was used are well known, however application of equivalent forces is a relatively simple way of taking into account imperfections and the discussion about the correctness of standard statements is beyond the scope of this paper.

Nearly all cross sections are classified as Class 1 according to EN 1993-1-2 and it is decided to use Safir's STEELEC3EN material type. An exception that requires some explanation is a top chord of side truss that is made of C300. Under pure compression it is evaluated as Class 2 and more advanced material definition should be adopted. However, the c/t ratio is equal to 22,86 and this value is just beyond the limit of Class 1 that is equal to 23,21. Therefore, as the reduction of the properties is not significant, it is decided that for this case it is not necessary to use advanced STEELSL material [3]. The dynamic analysis is used to avoid stopping calculations due to local failures that do not affect the safety of a whole structure. Minimum value of time step is set for 0,01s and initial time step is 1s. In the Table 1 analysed structural models are listed. Fire resistance, description of the type and location of failure mode and maximum deflections when calculations are stopped for each case are given. Moreover, critical temperatures of the members for which failure occurs are shown.

2.3. RESULTS OF MECHANICAL ANALYSIS

According to the data set out in the Table 1 individual models differ from each other in both location where the failure occurs and resistance time. The highest fire resistance is achieved by the simplest model S1i0 (t=784s), which does not take into account neither perpendicular forces that arise during a fire nor the influence of the adjacent structure. It should be mentioned that it is also the only analysed model for which critical member is diagonal bar of the external truss. For nearly all other cases, except S2iv, failure mode is associated with the top chord of the external truss. However, two further possibilities can be distinguished here: buckling of the C300 section in the middle of the span or destruction near the box girder column. The last one happens mainly for the most advanced models classified in S3 category. For previously mentioned model S2iv the failure mode is the buckling of a chord of internal truss immediately followed by buckling of another top chords in adjacent internal trusses. Control "hand calculations" performed according to EN 1993-1-2 indicate that this should be the weakest point of the structure with fire resistance of 660s. Observed failure modes are shown in Fig. 3.

Local and global imperfections which are considered have little influence on the final fire resistance of the structure subjected to fire action. For S1 models type added imperfections reduce time needed for destruction by 8 to 10%. For S2 this reduction is equal to 6% and for the most advanced models category S3 it is up to 10%. However it should be noticed that total impact of boundary conditions

Table 1. Results - structure heated according to ISO curve

Model	Fire resistance [s]	Failure mode	Maximum deflection [mm]	Critical temperature [°C]
S1.1i0	784	Buckling of the diagonal member (2xC100)	220	585-596
S1.2i0	733	Buckling of the top chord in the middle of the span (external truss)	520	556-614
S1.1ih	723	Buckling of the top chord in the middle of the span (external truss)	390	555-612
S1.2ih	650	Buckling of the top chord in the middle of the span (external truss)	417	512-574
S2i0	733	Buckling of the top chord in the middle of the span (external truss)	395	556-614
S2ih	709	Buckling of the top chord in the middle of the span (external truss)	358	543-605
S2iv	691	Buckling of the top chord in the middle of the span (internal truss)	367	591-613
S3i0	709	Failure of the top chord close to the main column	355	543-605
S3ih	697	Buckling of the top chord in the middle of the span (internal truss)	350	537-598
S3iv	635	Failure of the top chord close to the main column	260	501-536
S3ihg	649	Failure of the top chord close to the main column	284	512-574

and imperfections considered together could lead to fire resistance decrease of 13,5%. This is the case when comparing S3iv to S2i0. All failures occur for values of temperatures in range of 500 to 615°C. For comparison purposes some additional information about outcome of "hand calculations" are necessary. The analysis performed according to EN 1993-1-2 for C300 section results in buckling factor χ_{fi} equal to 0,416 for 600°C and for C200 χ_{fi} is equal to 0,306 for 585°C. Therefore it was expected that fire resistance reductions due to the impact of imperfections would be higher. Finally, for all cases the dynamic approach was able to go over the buckling of L50x5 verticals which occurs at very early stage of simulation and the calculations were not interrupted by this local instability. Maximum deflections achieved during the analysis varied between 1/50 and 1/100 of the truss span length. The lowest fire resistance is evaluated for S3iv model which is chosen for further performance-based analysis.

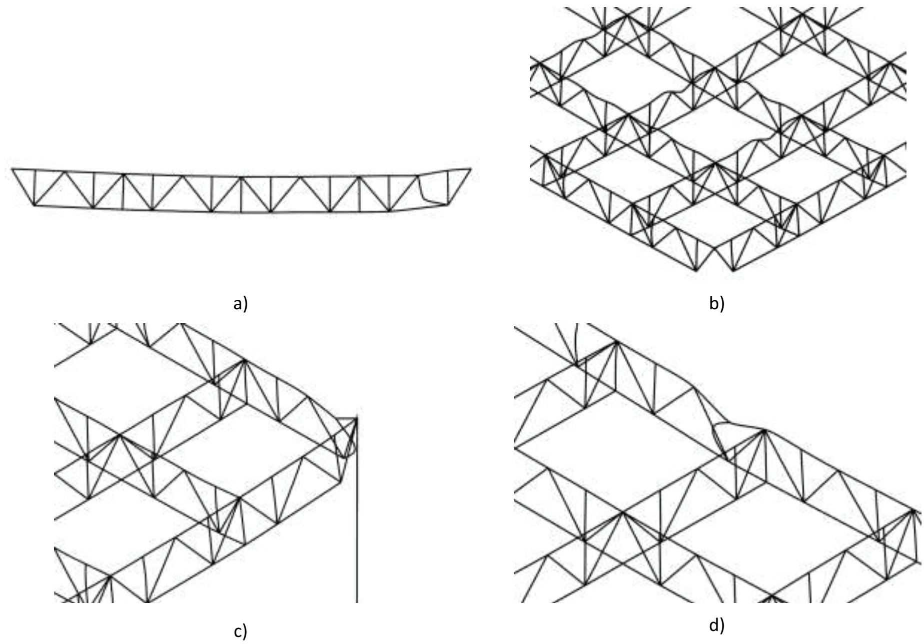


Fig. 3. Failure modes: a) S1.1i0 b) S2iv c) S3i0 d) S2ih

3. ADVANCED FIRE SCENARIO

2.1. FIRE DEVELOPMENT

The selection of the suitable fire scenario is the most critical part of a performance-based analysis. For considered structure localised fire could be the most appropriate. However even taking into account building's height it is not safe to omit the possibility of further ignition of stored goods. In this situation it is decided to use Fire Dynamics Simulator software [9] and check whether the flashover or further fire spread can occur. The applied fire modelling scheme and it's theoretical background are precisely described in [11].

An assumption is made that the first stage of the fire development is described by a well known t-square relationship. This proposition remains valid until the fire reaches some maximum power, which is specified according to [2]. The value of maximum power depends on the type of occupancy and fire-fighting facility. For a large-space steel structures without sprinklers that contain a large number of flammable materials Q_{max} could be taken as 25MW. For stored cellulosic materials increasing coefficient of heat release rate α is equal to 0,046890 kW/s^2 and the rate of heat release is adopted as equal to 500kW/m^2. As a result the maximum area of localised fire is 50m^2. The initial source of fire defined in this way is placed in two positions. In the first considered scenario the fire is exactly under the center of the structural segment (case marked as FC) and for the second case it is set under the external truss right in the middle of the span (marked as ST). The remaining surface of the warehouse is covered with stuff with physical and chemical properties close to materials based on cellulose. Simplified chemistry of burning goods is used ($C_1H_{1.7}O_{0.8}$). Ignition temperature is set as a surface property and its value is 250°C. In CFD simulations gates and roof glazing are also taken into account. It was checked that permanently opened gates lead to a worse scenario. For roof glazing different values of critical temperature are recommended in the literature. According to [1] temperatures for which first crack occur may vary from 360°C up to 600°C depending on the type of the material used to make the glazing. Due to this, quite high value of 500°C is taken into calculations. Mesh sizing 0,5x0,5x0,5m is used.

2.2. Results of fire modelling

During the first 30 minutes the fire development for both cases is very similar. However, afterwards higher values of temperature are reached for model with initial fire source placed under the external truss. It is a result of roof glazing behavior, or to be more precise, the result of a glazing location in relation to the fire source. For "FC" simulation opening is just above the fire and for "ST" it is placed in some distance from the initial burning area. Therefore for "ST" case glazing destruction occurs 8 minutes later than for the second model. This small change leads to two quite different scenarios of a fire development. For "ST" model after one hour of fire nearly whole surface of the warehouse is covered by flames. So if the structure in e.g. with fire protection is able to survive until this time the flashover is possible and localised fire limited to some chosen area is not safe assumption. For fire scenario with initial fire source placed in the centre of the analysed segment the fire area even after one hour is only twice as large as a maximum area of predefined localised fire. Charts showing values of temperature under the ceiling for both cases are presented in Fig. 4.

2.3. STRUCTURAL ANALYSIS BASED ON ADVANCED FIRE SCENARIO

Cases S3i0 and S3iv are calculated again using fire temperatures obtained with CFD simulations. As the connection between computational fluid dynamics and structural analysis is still being developed it is decided to use some simplifications. The structural segment is divided into 5 zones. For each of them the value of temperature is taken as the highest one from a few thermocouples. Next, selected values of temperatures are added to individual members that are in a given zone. Shadow factor is taken as equal to 1. For the initial fire source placed right in the centre of the analysed substructure temperatures exceed 500°C for only short period of time. Therefore construction is able to survive even one hour of fire and highest noted deflections are around 11cm. This is not the case for "ST" fire scenario. For S3i0+ST simulation fire resistance is 2149s while for the same model with added local vertical bow imperfections 2132s time is obtained. One can see that for an advanced fire scenario the impact of imperfections is even lower than for uniformly heated structure. The evaluated critical temperatures of the sections where failure occurs are nearly equal or higher than for the same models heated according to ISO curve. For S3i0+ST temperature reached for C300 section is 577°C to 630°C and for S3iv+ST it is 561°C to 611°C. These values taken from Table 1 are respectively 543°C to 605°C and 501°C to 536°C. Also failure modes are the same as for the proper cases described in Table 1.

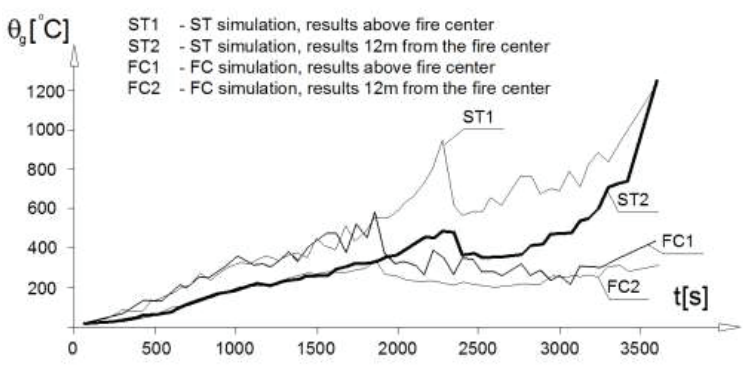


Fig. 4. Values of temperatures at selected points obtained for "ST" and "FC" fire scenarios

4. Conclusions

The performance-based analysis for a structure affected by fire action was presented. Different fire scenarios, types of substructures, boundary conditions and imperfections that affect the final fire resistance of the structure were taken into account. Further tests are carried out for this type of structures, however presented case shows that:

- the roof structure of older-type, which is without any fire protection, could resist 30 minutes of fire,
- added imperfections have little influence on fire resistance of the structure but the failure mode may be different depending on used imperfections and chosen boundary conditions.
- even in the case of a large space-structure the flashover is possible in some special cases. The assumption often made that the fire is limited to some chosen area may be wrong and can't be always considered as a safe one.

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ANALIZA WIELKOKUBATUROWEJ HALI STALOWEJ STARSZEGO TYPU W POŻARZE

Słowa kluczowe: hala wielkokubaturowa, konstrukcja stalowa, pożar, analiza oparta na właściwościach

PODSUMOWANIE:

Ocena odporności pożarowej wielkokubaturowych obiektów o konstrukcji stalowej jest przedmiotem bieżącego zainteresowania wielu badaczy. Jednocześnie zaawansowane metody obliczeniowe pozwalające na określenie nośności konstrukcji w warunkach pożaru charakteryzują się znacznym stopniem komplikacji i są ciągle na etapie rozwoju. W efekcie w analizie pożarowej obiektów wielkokubaturowych często przyjmowane są pewne założenia, które wymagają dalszego wyjaśnienia. Po pierwsze w typowych analizach nośności konstrukcji opartych na charakterystyce danej strefy pożarowej zakłada się zazwyczaj, że pożar lokalny jest ograniczony do pewnej maksymalnej mocy. Przyjmuje się przy tym, że zapłon zgromadzonego w okół początkowego źródła ognia paliwa nie jest możliwy. Po drugie zaawansowane metody oceny odporności pożarowej konstrukcji są zazwyczaj stosowane w odniesieniu do współczesnych obiektów wielkokubaturowych, podczas gdy nie uwzględnia się nadal użytkowanych obiektów starszego typu. Po trzecie w ostatnio publikowanych pracach, pomijając przypadki modelowania wydzielonych elementów, brak jest dostatecznych informacji o lokalnych i globalnych imperfekcjach uwzględnianych w analizach konstrukcji wykorzystujących metodę elementów skończonych.

W tej sytuacji w artykule przedstawiono opartą na charakterystyce strefy pożarowej analizę nośności rzeczywistej konstrukcji stalowej wzniesionej w latach osiemdziesiątych XX wieku. Szczególną uwagę zwrócono na:

- możliwe sposoby zamodelowania konstrukcji z uwzględnieniem różnych warunków brzegowych,
- wpływ lokalnych i globalnych imperfekcji na nośność konstrukcji w warunkach pożaru. Jeżeli było to możliwe wskazano modele zniszczenia danej podkonstrukcji,
- oszacowanie odporności pożarowej hali stalowej starszego typu,
- zastosowanie modelu bazującego na obliczeniowej mechanice płynów, pozwalającego na jednoczesne uwzględnienie pożaru lokalnego i możliwości dalszego zapłonu zgromadzongo paliwa.

Badania przeprowadzono w kilku etapach. W pierwszym kroku analizowano różne możliwości modelowania konstrukcji, dla celów porównawczych temperatury pożarowe określono za pomocą krzywej standardowej ISO. Do obliczeń wykorzystano oprogramowanie Safir. Rozważano następujące przypadki:

- modele 2D wybranych kratownic, z uwzględnieniem ewentualnego wyboczenia z płaszczyzny kratownicy,
- modele 3D wydzielonego typowego segmentu konstrukcji dachowej z możliwością swobodnego poziomego przesuwu wybranych podpór,
- modele 3D wydzielonego typowego segmentu konstrukcji dachowej z zabezpieczonymi przeciwogniowo słupami. Wpływ sąsiednich elementów konstrukcji budynku uwzględniono stosując elementy typu sprężyna, których sztywność (stałą przez cały czas trwania pożaru) określono dla temperatury 20°C. Ze względu na znaczną wysokość budynku i brak jakichkolwiek stężeń pionowych słupów wpływ sąsiednich elementów konstrukcji był niewielki. Podatność konstrukcji w połączeniu z długotrwałością i skomplikowaniem obliczeń jest przyczyną pominięcia w artykule przypadku kompletnego modelu 3D całego obiektu.

W drugim etapie analizy do wymienionych powyżej modeli dodano lokalne i globalne imperfekcje. Ponieważ w normie EN 1993-1-2 nie wyspecyfikowano odpowiednich wartości wspomnianych imperfekcji zostały one obliczone zgodnie

z normą EN 1993-1-1 a następnie zastąpione równoważnymi siłami zastępczymi. W symulacjach komputerowych wzięto pod uwagę lokalne pionowe i poziome imperfekcje łukowe oraz imperfekcje globalne.

W trzecim etapie analizy wybrane modele mechaniczne zostały przeliczone ponownie z uwzględnieniem zaawansowanego scenariusza pożarowego. Wartości temperatury gazów spalinowych zostały określone za pomocą programu Fire Dynamics Simulator a następnie zaimplementowane do modelu mechanicznego opracowanego w środowisku programu Safir. Założono przy tym, że początkowe źródło ognia znajduje się bezpośrednio pod środkiem rozpatrywanego niezależnego segmentu konstrukcji albo pod kratownicą skrajną. W modelu obliczeniowym uwzględniono pożar zlokalizowany oraz możliwość pełnego rozgorzenia ognia. Dla składowanego materiału określono temperaturę zapłonu oraz właściwości chemiczne i termiczne. Wzięto również pod uwagę właściwości przegród i świetlików dachowych.

W artykule zestawiono wartości temperatury krytycznej, wartości maksymalnych ugięć oraz możliwe schematy zniszczenia dla rozpatrywanej konstrukcji. Przeprowadzone symulacje komputerowe wskazują, że:

- konstrukcja dachowa starszego typu, pozbawiona jakiejkolwiek izolacji przeciwogniowej, może przetrwać 30 minut pożaru
- zadane imperfekcje mają mały wpływ na nośność konstrukcji w warunkach pożaru. Jednocześnie stwierdzono, że model zniszczenia może być różny w zależności od zadanych imperfekcji i uwzględnionych warunków brzegowych,
- nawet w przypadku konstrukcji wielkokubaturowej w pewnych warunkach może dojść do pełnego rozgorzenia ognia. Często przyjmowane założenie o ograniczeniu obszaru objętego pożarem do pewnej wybranej powierzchni może być błędne i nie powinno być w każdym przypadku traktowane jako w pełni bezpieczne.