UDC 624.014 DOI:10.31650/2707-3068-2024-28-55-62

DESIGN OF INDUSTRIAL HALLS WITH A STEEL STRUCTURE DUE TO A FIRE DESIGN SITUATION

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Abstract: The article presents the behaviour of a structure subjected to fire - fire design situation. A portal industrial hall with a span of 18.0 m, a length of 42.0 m and a column height of 7.0 m was considered. The spacing of frames - the main transverse systems - was assumed equal to 6.0 m. A case was considered in which the elements of the structure are unprotected from the influence of high temperature. The load carrying capacity, critical temperature and time required to reach the critical temperature were analysed for column cross sections. The calculations were performed using Autodesk's Robot Structural Analysis 2024 computer software.

In the case of HEB-type profiles, the time required to reach the critical temperature at which the cross-section of the element loses its compressive capacity is significantly greater compared to that of IPE-type profiles. This is (as above) a recommendation for the use of solid elements, with low wall slenderness, for structures significantly exposed to fire and high temperatures

Keywords: industrial hall, stability of portal frame, accidental design situation, fire design situation, FEM analysis

1. Introduction

The presented paper is a continuation of the paper Design of industrial halls with a steel structure due to an accidental design situation - the impact of a vehicle on a frame column [1]. The paper presents the issue of checking the load capacity of cross sections and checking the stability of selected elements of the structure exposed to fire. Calculations were carried out on the example of a steel hall structure as in the paper [1], Fig. 1.

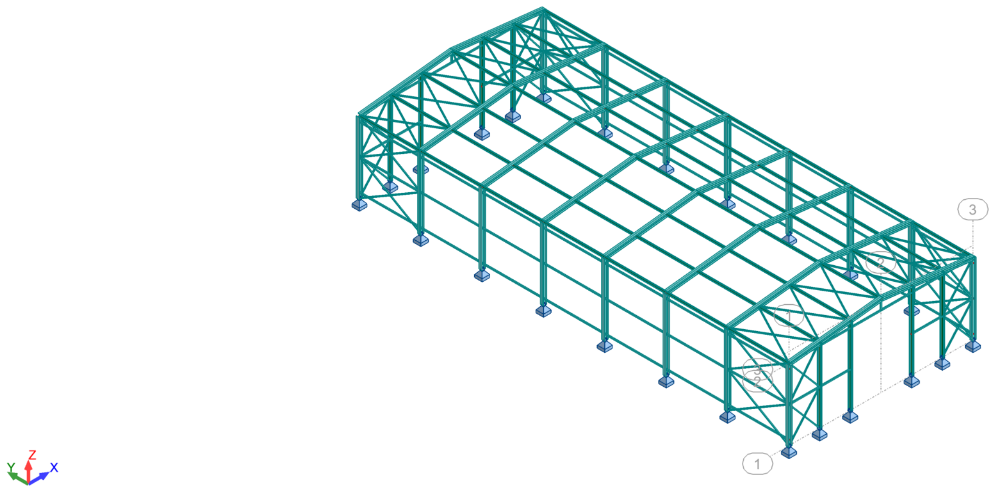

Fig. 1. Structure of the industrial hall - 3D view

2. Structural Fire design procedure

A structural fire design analysis should take into account the following steps as relevant:

- selection of the relevant design fire scenarios,
- determination of the corresponding design fires,

- calculation of temperature evolution within the structural members,
- calculation of the mechanical behaviour of the structure exposed to fire.

To identify the accidental design situation, the relevant design fire scenarios and the associated design fires should be determined. For each design fire scenario, a design fire, in a fire compartment, should be estimated.

The calculations adopted the Standard temperature-time curve in accordance with the Eurocode EN 1991-1-2. The temperature of the gas Θ_g in the fire compartment after time t, was determined according to the formula:

$$\Theta_g = 20 + 345log_{10}(8 \cdot t + 1)$$

Θ_g – the gas temperature in the fire compartment
t – the time

The critical temperature $\theta_{a,cr}$ of carbon steel elements was calculated according to formula:

$$\theta_{a,cr} = 39{,}19 \cdot ln\left[\frac{1}{0{,}9674 \cdot \mu_0^{3{,}833}} - 1\right] + 482$$

μ_0 – degree of utilization

3. Computational analysis

For the computational analysis, spatial models of the steel structure of the industrial hall were developed. Columns and rafters were modelled using beam-type bar elements with 6 degrees of freedom at the node. Vertical and horizontal bracing rods were given attributes to allow them to carry only longitudinal forces (compression/tension).

Static calculations were performed using first-order linear analysis.

The computational analysis was carried out on a flat frame isolated from the model. Consideration was given to frames whose columns and rafters are made of HEA, HEB and IPE profiles. Frames restrained in the foundation and articulated in the foundation were considered, giving a total of 6 computational models.

Table 1. Types of computational models of the framework adopted for the analysis

Calculation model name	The method of connecting the column to the foundation	Column – type of profile	Beam – type of profile
Model 1 - Frame 1	pinned	HEA	HEA
Model 2 - Frame 2	rigid	HEA	HEA
Model 3 - Frame 3	pinned	HEB	HEB
Model 4 - Frame 4	rigid	HEB	HEB
Model 5 - Frame 5	pinned	IPE	IPE
Model 6 - Frame 6	rigid	IPE	IPE

The analysis was carried out in two steps:

1. checking the stability of structural elements in a design situation of permanent,
2. checking the stability of structural elements in an exceptional design situation - action of fire, in accordance with the recommendations of EN 1993-1-2, the load on structural elements in an exceptional design situation was reduced by 35%.

3.1. Checking the stability of structural elements in a permanent design situation

In the first stage of calculations, the overall stability of the steel frames of the hall structure was checked. Optimization of the sections of the structure was carried out. As a criterion, the minimum mass of individual elements while maintaining their stability was adopted.

The parameters and coefficients adopted for checking the stability of the bars, as well as the method of analysis, are presented in the paper [1]. The results of the calculations in the form of the type and size of the adopted profiles are presented below.

Table 2. Results of calculations in the form of values of size and type of used steel profiles, frame weight and painting area

Calculation model name	Column	Beam	Frame weight [kg]	Painting area [m^2]
Frame 1 pinned	HEA 340 Ratio: 0,95	HEA 320 Ratio: 0,88	3239	56,95
Frame 2 rigid	HEA 300 Ratio: 0,92	HEA 300 Ratio: 0,91	2836	55,25
Frame 3 pinned	HEB 320 Ratio: 0,88	HEB 280 Ratio: 0,97	3644	54,13
Frame 4 rigid	HEB 280 Ratio: 0,84	HEB 280 Ratio: 0,83	3308	52,03
Frame 5 pinned	IPE 500 Ratio: 0,89	IPE 450 Ratio: 0,88	2676	53,50
Frame 6 rigid	IPE 500 Ratio: 0,78	IPE 450 Ratio: 0,80	2676	53,50

3.2. Checking the stability of structural elements in an accidental design situation – fire load

The analysis of the load carrying capacity of the structure under fire was carried out using the Fine Element Method and computer software Autodesk Structural Analysis 2024. Calculations were performed using the calculation procedures recommended by EN 1993-1-2.

3.2.1. Structural fire design according to EN 1993-1-2

To determine the fire resistance the following design methods are permitted:

- simplified calculation models;
- advanced calculation models;
- testing.

Simple calculation models are simplified design methods for individual members, which are based on conservative assumptions. The resistance of a steel member under fire conditions is preserved after time t if the following condition is met:

$$E_{fi,d} \leq R_{fi,d,t}$$

$E_{fi,d}$ - the design effect of actions for the fire design situation, according to EN 1991-1-2

$R_{fi,d,t}$ - the corresponding design resistance of the steel member, for the fire design situation, at time t.

The design resistance $R_{fi,d,t}$ at fire duration t is determined by assuming a normally uniform temperature distribution in the section and using a modification of the design resistance at normal temperature according to EN 1993-1-1, which involves assuming the mechanical properties of steel at elevated temperatures

Resistance - tension members

The resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform temperature θ_a should be determined from:

$$N_{fi,\theta,Rd} = k_{y,\theta} \cdot N_{Rd} \cdot [\gamma_{M0}/\gamma_{M,fi}]$$

$k_{y,\theta}$ – the reduction factor for the yield strength of steel at temperature θ_a reached at time t,

N_{Rd} – the design resistance of the cross-section for normal temperature design, according to EN 1993-1-1.

Compression members with Class 1, Class 2 or Class 3 cross-sections

The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a Class 1, Class 2 or Class 3 cross-section with a uniform temperature θ_a should be determined from:

$$N_{b,fi,t,Rd} = \chi_{fi} \cdot A \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}$$

χ_{fi} – the reduction factor for flexural buckling in the fire design situation,

$k_{y,\theta}$ – the reduction factor for the yield strength of steel at temperature θ_a reached at time t,

The buckling length L_{fi} of a column for the fire design situation should generally be determined as for normal temperature design.

Beams with Class 1 or Class 2 cross-sections

The design moment resistance $M_{fi,\theta,Rd}$ of a Class 1 or Class 2 cross-section with a uniform temperature θ_a should be determined from:

$$M_{fi,\theta,Rd} = k_{y,\theta} \cdot M_{Rd} \cdot [\gamma_{M0}/\gamma_{M,fi}]$$

M_{Rd} – the plastic moment resistance of the gross cross-section $M_{pl,Rd}$ for normal temperature design, according to EN 1993-1-1 or the reduced moment resistance for normal temperature design, allowing for the effects of shear if necessary, according to EN 1993-1-1,

$k_{y,\theta}$ – the reduction factor for the yield strength of steel at temperature θ_a.

The design lateral torsional buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained member with a Class 1 or Class 2 cross-section should be determined from:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} \cdot W_{pl} \cdot k_{y,\theta,com} \cdot f_y/\gamma_{M,fi}$$

$\chi_{LT,fi}$ – the reduction factor for lateral-torsional buckling in the fire design situation,

$k_{y,\theta,com}$ – the reduction factor from section 3 for the yield strength of steel at the maximum temperature ill the compression t1ange $\theta_{a,com}$ reached at time t.

Beams with Class 3 cross-sections

The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 3 cross-section with a uniform temperature should be determined from:

$$M_{fi,t,Rd} = k_{y,\theta} \cdot M_{Rd} \cdot [\gamma_{M0}/\gamma_{M,fi}]$$

M_{Rd} – the elastic moment resistance of the gross cross-section for normal temperature design, according to EN 1993-1-1 or the reduced moment resistance allowing for the effects of shear if necessary according to EN 1993-1-1

$k_{y,\theta}$ – the reduction factor for the yield strength of steel at temperature θ_a.

The design buckling resistance moment k1l1.fi.t.Rd at time of a laterally unrestrained beam with a Class 3 cross-section should be determined from:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} \cdot W_{el} \cdot k_{y,\theta,com} \cdot f_y/\gamma_{M,fi}$$

Members with Class 1, 2 or 3 cross-sections, subject to combined bending and axial compression

The design buckling resistance $R_{fi,t,Rd}$ at time t of a member subject to combined bending and axial compression should be verified by satisfying expressions:

– member with a Class 1 or Class 2 cross-section,

$$\frac{N_{fi,Ed}}{\chi_{min,fi} \cdot A \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_y \cdot M_{y,fi,Ed}}{W_{pl,y} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_z \cdot M_{z,fi,Ed}}{W_{pl,z} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} \leq 1{,}00$$

$$\frac{N_{fi,Ed}}{\chi_{z,fi} \cdot A \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_{LT} \cdot M_{y,fi,Ed}}{\chi_{LT,fi} \cdot W_{pl,y} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_z \cdot M_{z,fi,Ed}}{W_{pl,z} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} \leq 1{,}00$$

– member with a Class 3 cross-section.

$$\frac{N_{fi,Ed}}{\chi_{min,fi} \cdot A \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_y \cdot M_{y,fi,Ed}}{W_{el,y} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_z \cdot M_{z,fi,Ed}}{W_{el,z} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} \leq 1{,}00$$

$$\frac{N_{fi,Ed}}{\chi_{z,fi} \cdot A \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_{LT} \cdot M_{y,fi,Ed}}{\chi_{LT,fi} \cdot W_{el,y} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} + \frac{k_z \cdot M_{z,fi,Ed}}{W_{el,z} \cdot k_{y,\theta} \cdot f_y/\gamma_{M,fi}} \leq 1{,}00$$

The values of coefficients, parameters and recommendations and requirements for the above cases are detailed in Chapter 4 of EN 1993-1-2.

3.3. Results of analysis

The results of the computational analyses performed, in the form of:
- the critical temperature value of the element,
- the degree of strain on the cross-section of the element,
- the temperature of the element after time t of fire exposure,
- the time required to reach the critical temperature.

These results relate to:
- elements made of S235 grade steel, yield strength fy = 235 N/mm2,
- the required fire resistance time of the element of 30 minutes (R30),
- calculations were performed with elements unprotected against the effects of temperature,
- standard fire curve, Figure 2.

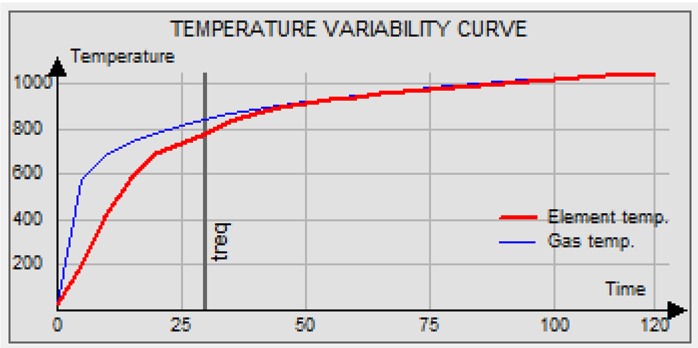

Fig. 2. Example Standard temperature-time curve, marked required fire resistance time of 30 minutes, profile temperature after 30 minutes and fire/gas temperature

Table 3. Values of internal forces and the other parameters calculated in Fire Design Situation

Calculation model name	Column profile	Section factor	$N_{c,Rd}$ $N_{c,fi,t,Rd}$	Temperature $Q_{a,max}$, °C	Temperature $Q_{a,cr}$, °C	Time $t_{fi,max}$, min.
Frame 1	HEA340	134	3125 541	757	635	17
Frame 2	HEA300	152	2655 422	770	560	15
Frame 3	HEB320	110	3783 715	736	603	20
Frame 4	HEB280	123	3078 564	744	575	18
Frame 5	IPE500	151	2726 366	790	350	9
Frame 6	IPE500	151	2726 366	790	350	9

$Q_{a,max}$ - temperature of the element after time t=30 min (time assumed for calculations corresponding to fire resistance class R30), $Q_{a,cr}$ – critical element temperature, $t_{fi,max}$ – fire resistance time of the cross-section, $N_{c,Rd}$ – resistance of cross-section in normal temperature, $N_{c,fi,t,Rd}$ – resistance of cross-section in temperature $Q_{a,max}$.

2.4. Conclusions

Based on the results of the above calculations, the following conclusions were drawn:

1. The compressive capacity of the cross-section of the column, under fire conditions, is several times lower than the capacity at normal temperature. Therefore, it seems mandatory to use protection of elements against temperature - Contour encasement of uniform thickness or Hollow encasement of uniform thickness.

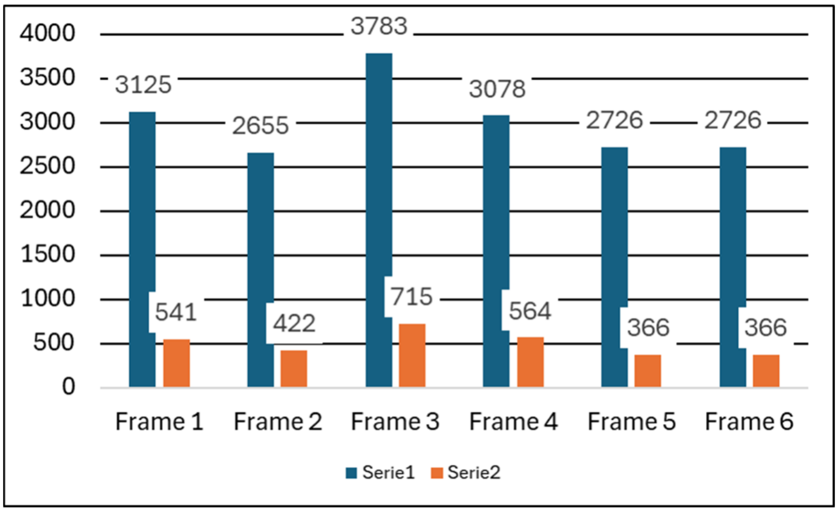

Fig. 3. Resistance of column cross-section under compression: Serie1 – normal temperature, Serie2 – fire conditions, after 30 minutes of high temperature, fire duration R30

2. The critical temperature of a section (the temperature at which it loses its compressive capacity), decreases as the slenderness of the section increases. This means that solid elements with thicker walls show greater resistance to temperature. Therefore, for structures exposed to high temperatures, it is recommended to use elements made of HEB profiles. IPE profiles are less favourable and not recommended.

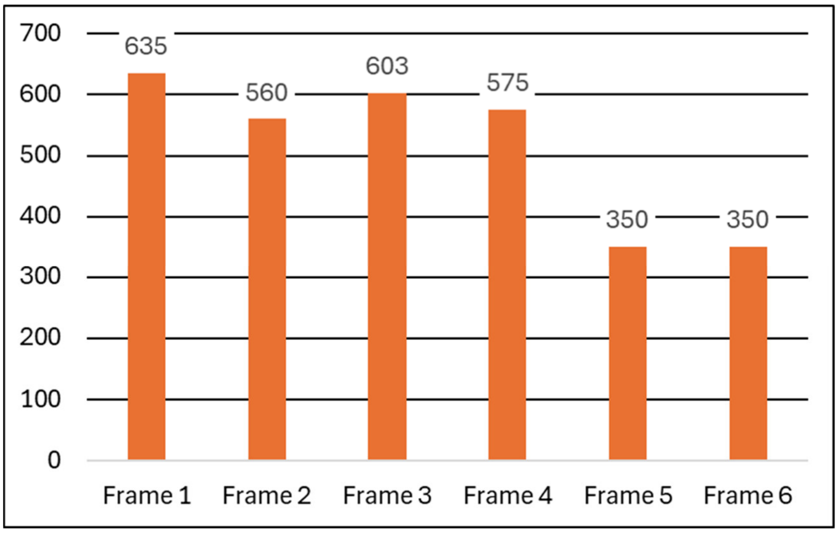

Fig. 4. The critical temperature value of the profile depending on its type - Frame 1 and 2 - HEA, Frame 3 and 4 - HEB, Frame 5 and 6 - IPE. Once the critical temperature value is reached, the section loses its load bearing capacity

3. In the case of HEB-type profiles, the time required to reach the critical temperature at which the cross-section of the element loses its compressive capacity is significantly greater compared to that of IPE-type profiles. This is (as above) a recommendation for the use of solid elements, with low wall slenderness, for structures significantly exposed to fire and high temperatures.

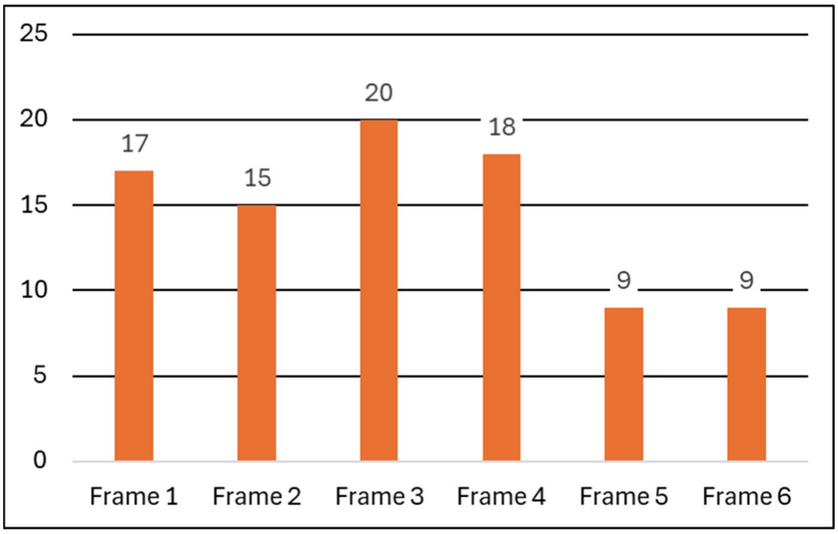

Fig. 5. The time required to reach the critical temperature of the profile depending on its type - Frame 1 and 2 - HEA, Frame 3 and 4 - HEB, Frame 5 and 6 - IPE. After this time, the section of the profile loses its load bearing capacity

3. Summary

In conclusion, it can be said that in the case of structures where there is an increased risk of fire, this fact should be taken into account at the stage of structural design. This is usually done using the appropriate design procedures contained in standards [2], [3], [4], and various guidelines and recommendations [5]. Thick-walled profiles with walls of low slenderness show greater resistance to high temperatures compared to slender profiles with high wall slenderness.

However, it should be noted that steel structures exposed to fire should be compulsorily protected against the effects of high temperatures by using appropriate insulating layers. Only in this way can the required safety of the structure be ensured during longer periods of fire exposure.

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ПРОЕКТУВАННЯ ПРОМИСЛОВИХ ЦЕХІВ ЗІ СТАЛЕВИМ КАРКАСОМ НА ВИНИКНЕННЯ ПОЖЕЖНОЇ РОЗРАХУНКОВОЇ СИТУАЦІЇ

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Анотація. У статті представлено поведінку конструкції під впливом пожежі – пожежна розрахункова ситуація. Розглядався портальний виробничий цех з прольотом 18,0 м, довжиною 42,0 м і висотою колони 7,0 м. Відстань між рамами - основними поперечними системами - прийняли рівним 6,0 м. Розглянуто випадок, коли елементи конструкції незахищені від впливу високої температури. Несучу здатність, критичну температуру та час, необхідний для досягнення критичної температури, аналізували для поперечних перерізів колон. Розрахунки проводилися за допомогою комп'ютерної програми Autodesk Robot Structural Analysis 2024.

За результатами наведених розрахунків зроблено наступні висновки:

1. Міцність на стиск поперечного перерізу колони в умовах пожежі в кілька разів нижча за міцність на стиск при дії звичайної температури. Тому видається обов'язковим використання захисту елементів від дії підвищених температур - контурної обшивки або порожнистої обшивки рівномірної товщини.

2. Критична температура (температура, при якій поперечний переріз елемента втрачає здатність до стиснення) зменшується зі збільшенням гнучкості. Це означає, що масивні елементи з більш товстими стінками виявляють більшу стійкість до температури. Тому для конструкцій, що піддаються впливу високих температур, рекомендується використовувати елементи з профілів HEB (широкополичні двотаври). Профілі IPE (нормальні двотаври) є менш сприятливими і не рекомендуються.

3. У випадку профілів типу HEB час, необхідний для досягнення критичної температури, за якої поперечний переріз елемента втрачає здатність до стиску, значно більший порівняно з профілями типу IPE. Це (як і вище зазначене) є рекомендацією щодо використання таких профілів елементів з низькою гнучкістю стінок для конструкцій, які піддаються впливу вогню та високих температур.

Підсумовуючи, можна сказати, що у випадку з конструкціями, де існує підвищений ризик пожежі, цей факт слід враховувати на етапі проектування конструкції. Товстостінні профілі зі стінками малої гнучкості виявляють більшу стійкість до високих температур у порівнянні з тонкими профілями з високою гнучкістю стінок.

Однак слід зазначити, що сталеві конструкції, що піддаються впливу вогню, повинні бути обов'язково захищені від впливу високих температур за допомогою відповідних ізоляційних шарів. Тільки таким чином можна забезпечити необхідну безпеку конструкції протягом тривалого періоду впливу вогню.

Ключові слова: виробничий цех, стійкість портальної рами, аварійна розрахункова ситуація, пожежна розрахункова ситуація, аналіз методом кінцевих елементів.