

ASSESSMENT OF THE FIRE RESISTANCE OF A FLANGED JOINT OF A STEEL TRUSS

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Abstract. The article considers the specific features of the numerical assessment of the fire resistance of a spatial joint connecting tubular elements with a ring flange and radial stiffening ribs. The relevance of the study is due to the fact that, in steel structures under fire conditions, the limit state is often governed not by the overall strength of the main cross-section, but by the local behavior of joints, where a complex interaction occurs between the base metal, welds, bolts, contact zones, and stiffeners. It is precisely the joints that are the most sensitive to the reduction in the physical and mechanical properties of the material when heated, since stresses are concentrated in them, the force transfer pattern changes, and local failure mechanisms manifest themselves more rapidly. The aim of the study is to determine the influence of the fire-protective layer on the temperature regime and load-bearing capacity of the investigated joint under standard fire exposure in accordance with the R60 fire resistance requirement.

The study was carried out in two successive stages. At the first stage, a fire analysis of the joint without fire protection was performed using the following initial parameters: the standard fire curve, convective heat transfer coefficient $\alpha_c = 25 \text{ W/m}^2\text{K}$, steel surface emissivity $\varepsilon_m = 0.80$, and fire environment emissivity $\varepsilon_f = 1.00$. The analysis showed intensive heating of the joint components to temperatures of approximately 935.86-942.88°C. However, after the introduction of contour fire protection with the following parameters: layer thickness $d_p = 30 \text{ mm}$, thermal conductivity $\lambda_p = 0.2 \text{ W/(m}\cdot\text{K)}$, specific heat capacity $c_p = 1700 \text{ J/(kg}\cdot\text{K)}$,



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and density $\rho_p = 800 \text{ kg/m}^3$, the temperature decreased to the range of 221.43-380.97°C.

Keywords: fire resistance; steel structures; fire protection; finite element method.

PROBLEM STATEMENT

For steel structures, the issue of fire resistance is traditionally associated with the reduction in the physical and mechanical properties of the material when heated, which

leads to a decrease in the strength, stiffness, and load-bearing capacity of the elements.

In most cases, when analyzing the behavior of structures under fire conditions, the main attention is paid to bar elements – beams, columns, braces, and posts – whereas joints are often considered in a simplified manner or are even assumed to automatically provide the required level of reliability. However, it is precisely the joints that, in many cases, are the most vulnerable parts of the structure, since a complex stress-strain state is formed in them, local forces are concentrated there, contact interactions arise between individual parts, and the load is transferred through a system of plates, bolts, welds, and ribs.

For tubular elements with flange joints, this problem is especially relevant. Such joints are characterized by spatial behavior, complex geometry, local zones of stress concentration, and a significant dependence of the load-bearing capacity on the performance of individual components. Under fire conditions, heating occurs unevenly, which leads to a non-uniform temperature field in the tubes, flanges, stiffening ribs, bolts, and welds. As a result, within the same joint, individual elements may operate under different temperature conditions, and the overall limit state may be governed not by the base metal, but by local failure of the welds, loss of bolt performance, or excessive deformations in the contact zones.

Design practice shows that the use of generalized engineering approaches for assessing the fire resistance of joints does not always provide a reliable result. Simplified methods based on the average temperature of an element or on checking only individual cross-sections do not allow the actual force transfer pattern in the joint, the specific features of local heating, and the mutual influence of all structural components to be fully taken into account. This is especially important for flange joints, where, under elevated temperatures, the properties of the base metal, weld metal, the contact stiffness of the joint, the behavior of bolts in tension and shear, as well as the deformation pattern of the strengthening elements, change simultaneously.

Another important issue is that, when

assessing the fire resistance of a joint, it is necessary to analyze not only the thermal action itself, but also its consequences for the mechanical behavior of the connection. That is, the problem is of a complex nature and includes two interrelated stages: determining the temperature field in the joint elements for a specified fire scenario, and then verifying the load-bearing capacity of the heated structure while taking into account the reduction in the physical and mechanical properties of the material when heated. Only such an approach makes it possible to determine which particular component is decisive in the formation of the limit state and whether the joint ensures the required standard fire resistance rating.

For the investigated flange joint of tubular elements, the task is further complicated by the need to assess the effectiveness of fire protection. In the absence of a protective layer, the steel elements of the joint can quickly heat up to high temperatures, which causes a sharp reduction in their load-bearing capacity. At the same time, the application of a fire-protective material changes the intensity of heat transfer, reduces the rate of steel heating, and potentially makes it possible to ensure the required fire resistance class. However, the quantitative assessment of such an influence requires a special numerical analysis, since the effectiveness of fire protection depends not only on its presence, but also on the layer thickness, thermal conductivity, heat capacity, density, and application pattern.

The scientific and practical problem lies in establishing the patterns of change in the temperature state and load-bearing capacity of the flange joint of tubular elements under standard fire exposure, as well as in determining the influence of the fire-protective layer on satisfying the R60 fire resistance requirement. Solving this problem makes it possible not only to assess the serviceability of a specific joint, but also to formulate an approach to substantiating the fire-protection parameters for similar steel connections in which the limit state is governed by the local behavior of individual components.

ANALYSIS OF PREVIOUS RESEARCH

In studies devoted to joints and connections, it has been shown that the joint zone is one of the most complex regions for analysis, since it combines local thermal effects, a complex stress-strain state, contact interaction between individual components, and nonlinear effects. In particular, the study on 3D modeling of steel structure joints emphasizes the expediency of combining frame bar analysis with detailed joint modeling in a BIM environment and in specialized computational software packages. It is shown that such an integrated approach makes it possible not only to reproduce the geometry of the connection, but also to take into account the actual behavior of elements within the joint zone under thermal exposure, as well as to use the results to refine the structural solution, reduce steel consumption, and rationalize load-bearing capacity reserves [1].

In the study of a fire-protected steel floor beam, a step-by-step calculation procedure in the LIRA-SAPR software package is presented, taking into account the nonlinear dependence of thermal conductivity, heat capacity, and convective heat transfer on temperature. The obtained results showed that accounting for material nonlinearity significantly affects both the temperature distribution and changes in stiffness characteristics. The presented comparison of calculations with linear and nonlinear properties demonstrated that the error in temperature determination may reach 26%, while the error in determining the modulus of elasticity may reach 65.9%, which directly affects the reliability of the mechanical verification [2].

Some studies prove that fire resistance assessment cannot be limited only to checking the strength of an isolated element. Of particular importance is the consideration of secondary effects associated with changes in the stiffness of heated elements, thermal deformations, and the subsequent redistribution of internal forces within the structural system [3, 5].

Alongside studies of steel elements and joints, an important line of research is represented by works that consider different fire temperature regimes and the limits of applicability of simplified methods. Thus, for

the hydrocarbon fire regime, it has been shown that it is characterized by significantly higher intensity, rapid temperature rise to about 1100 °C in the first minutes, and therefore requires the use of advanced calculation methods. These studies emphasize that tabulated data and simplified approaches may be used only for the standard temperature regime, whereas more severe scenarios require full thermal modeling and subsequent verification of residual load-bearing capacity [4, 8].

The further development of this topic is associated with the formation of the general theory of structural fire engineering, within which the behavior of a steel structure under fire conditions is considered as the result of the interaction of thermal and mechanical processes. These studies have shown that under the effect of high temperature, not only are the elements heated, but there is also a reduction in the physical and mechanical properties of the material when heated, a change in the stiffness of the system, the development of additional deformations, and a redistribution of internal forces. This approach has become the basis for the transition from simplified code-based checks to a deeper engineering analysis focused on the actual behavior of the structure in a fire scenario [9-14].

In studies devoted to heat transfer in steel elements, particular attention has been paid to determining the temperature field as the basic stage of fire analysis. It has been shown that the accuracy of temperature assessment in a structure directly affects the reliability of the subsequent mechanical analysis. It has been established that for exposed steel elements, the decisive role is played by the total heat transfer, which includes convective and radiative components, whereas in the presence of a protective layer, its thermal conductivity, heat capacity, density, and application pattern become of significant importance [15-17, 20].

A significant part of previous studies is devoted to the behavior of steel joints at elevated temperatures. These works have shown that under fire conditions a joint changes not only its stiffness, but also the very mechanism of force transfer between its constituent elements. It has been found that

when heated, the role of local plate deformations increases, the behavior of bolts changes, the condition of welds has a significant influence, and failure often has a local component-based character. As a result, it is advisable to consider the joint not as a conventionally homogeneous element, but as a multicomponent system in which each component responds differently to increasing temperature. It is precisely this approach that has formed the basis for the development of component-based and spring-component models of connections in fire [18-19, 21-23, 25].

Some studies demonstrate that the temperature state of a joint may differ significantly from the temperature state of the adjacent bar elements. This is especially important for unprotected or partially protected connections, in which individual components have different massiveness, different heating conditions, and different heating rates. Under such conditions, the use of an average temperature for the entire joint does not allow its serviceability to be assessed reliably. That is why contemporary works emphasize the need to take into account the actual temperature distribution within the joint, since local temperature differences directly affect the load-bearing capacity of individual components and the formation of the limit state [24, 26].

An important direction is the study of connections in which the main attention is paid not only to strength, but also to deformability, robustness, and the ability to maintain serviceability under a complex stress state in fire conditions. It has been established that even those joints which at normal temperature are designed mainly for shear or local force transfer may, in fire, be subjected to additional tensile, compressive, or bending actions caused by thermal deformations and interaction with the entire structural system. This means that the fire verification of a joint should take into account not only the traditional strength criteria, but also the ability of the connection to deform without brittle failure, to retain rotational capacity, and to allow redistribution of forces between components [27-28].

One publication is devoted to the study of

the fire resistance of steel floor beams during the reconstruction of the UN office building in Ukraine. Available bibliographic data indicate that the authors analyze the increase in the fire resistance rating of load-bearing beams, compare fire-protection options, and, after the thermal analysis stage, proceed to the strength calculation. The study also takes into account the influence of an air gap between the protection boards and the structure, which is important for more accurate modeling of heat transfer. This source is valuable for your paper because it demonstrates an example of applied fire analysis of steel elements in a real reconstruction project and confirms the expediency of combining thermal and static analysis [29].

Another paper describes in detail an algorithm for analyzing a steel floor beam under high-temperature effects using the LIRA-SAPR software package. The author solves the problem of transient heat conduction, obtains the temperature distribution across the cross-section, and then uses these results to assess the stress-strain state of the beam. Importantly, the study takes into account the nonlinear variation of the thermophysical properties of steel and fire-protective material, and also presents a comparison of numerical results with a full-scale experiment. For your study, this paper is useful primarily as an example of a consistent transition from the thermal problem to the mechanical verification of a structure under fire conditions [30].

The issue of the fire resistance of steel structures in contemporary research is considered as a complex problem that combines regulatory support, thermal modeling, assessment of the stress-strain state, and verification of the load-bearing capacity of elements and joints during fire exposure. The basis of this approach is formed by regulatory documents that define the principles of structural design based on limit states, the procedure for considering fire action, the requirements for steel elements under fire conditions, as well as the rules for the design of connections. It is precisely in these sources that the approach is established according to which fire design must take into account not only the

temperature regime of the environment, but also the actual preservation of the load-bearing capacity of the structure for a specified period of time [31-32].

PRINCIPAL RESEARCH

The aim of the study is to assess the influence of the fire-protective layer on the load-bearing capacity of the flange joint of tubular elements under fire conditions in accordance with the R60 requirement.

To achieve this aim, the following tasks were set:

- to develop a calculation model of the joint;
- to define fire exposure according to the standard fire curve;
- to perform the analysis of the joint

without fire protection;

- to perform the analysis of the same joint with contour fire protection;
- to compare the temperature fields and integral performance indicators;
- to identify the critical elements of the joint for each stage of the analysis.

The investigated joint is a connection between two tubular elements of the lower chord of the truss, joined by means of a ring flange, radial stiffening ribs, bolts, and welds. The transfer of forces from one tubular element to the other is carried out through the flanged system with ribs. In the model, an axial load is applied along the longitudinal axis of the elements (Fig. 1).

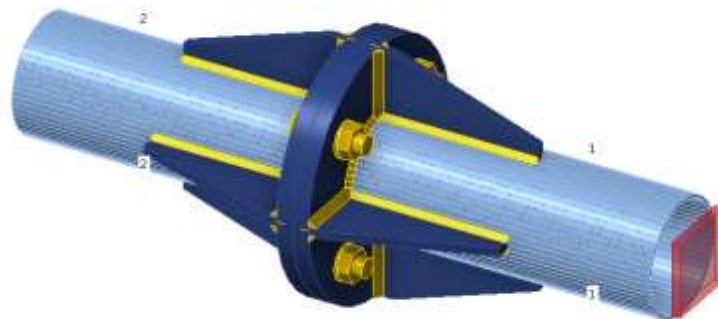


Fig. 1. Model of the flange connection of the lower chord of a steel truss

Рис. 1. Модель фланцевого з'єднання нижнього поясу сталеві ферми

To determine the temperature of the fire environment, the standard fire curve was used:

$$\theta_g(t) = 20 + 345 \log_{10}(8t + 1) \quad (1)$$

where $\theta_g(t)$ - gas environment temperature, °C;
 t - time, min.

Equation (1) describes the standard increase in temperature over time and serves as the basis for determining the thermal effect on the surface of the steel elements of the joint.

The total heat flux to the element surface is defined as the sum of the convective and radiative components:

$$q = q_c + q_r \quad (2)$$

$$q_c = \alpha_c(\theta_g - \theta_s) \quad (3)$$

$$q_r = \varepsilon_m \varepsilon_f \sigma [(\theta_g + 273)^4 - (\theta_s + 273)^4] \quad (4)$$

where α_c - convective heat transfer coefficient, W/m²K;

θ_s - steel surface temperature, °C;

ε_m - emissivity of the steel surface;

ε_f - emissivity of the fire environment;

σ - Stefan-Boltzmann constant.

Thus, we obtain:

$$q = \alpha_c(\theta_g - \theta_s) + \varepsilon_m \varepsilon_f \sigma [(\theta_g + 273)^4 - (\theta_s + 273)^4] \quad (5)$$

$$273)^4 - (\theta_s + 273)^4]$$

Equation (5) defines the instantaneous thermal effect on the steel element.

In the general matrix form, the thermal problem is written as follows:

$$[C_{th}]\dot{\theta} + [K_{th}]\theta = Q(t) \quad (6)$$

where $[C_{th}]$ - heat capacity matrix;
 $[K_{th}]$ - thermal conductivity and surface heat transfer matrix;
 $\{\theta\}$ - vector of nodal temperatures;
 $\{Q(t)\}$ - vector of heat fluxes.

For a simplified assessment of the heating of an unprotected steel element, the heat balance can be written as:

$$\rho_a c_a V \frac{d\theta_a}{dt} = A_m q \quad (7)$$

where ρ_a - density of steel;
 c_a - specific heat capacity of steel;
 V - volume of the steel element;
 A_m - heated surface area;
 θ_a - steel temperature.

Substituting (5) into (7), we obtain:

$$\rho_a c_a V \frac{d\theta_a}{dt} = A_m [\alpha_c (\theta_g - \theta_a) + \varepsilon_m \varepsilon_f \sigma ((\theta_g + 273)^4 - (\theta_a + 273)^4)] \quad (8)$$

Equation (8) shows that the rate of steel heating increases with increasing heated surface area and decreases with increasing heat capacity of the element.

In the presence of a fire-protective layer, the heat flux to the steel decreases. In a simplified engineering formulation, the thermal resistance of the protective layer can be expressed as:

$$R_p = \frac{d_p}{\lambda_p} \quad (9)$$

where d_p - thickness of the protective layer;
 λ_p - thermal conductivity coefficient of the protection material.

The thermal inertia of the protective layer is determined by the expression:

$$C_p = \rho_p c_p d_p \quad (10)$$

where ρ_p - density of the fire-protective material;
 c_p - its specific heat capacity.

The greater R_p and C_p , the more slowly the steel heats up.

After determining the temperature field, the mechanical verification of the joint is performed. In general form, the equilibrium equation is written as:

$$[K(\theta)]u = F \quad (11)$$

where $[K(\theta)]$ - temperature-dependent stiffness matrix;
 $\{u\}$ - displacement vector;
 $\{F\}$ - external load vector.

For nonlinear step-by-step analysis:

$$[K_t(\theta)]\{\Delta u\} = \{\Delta F\} \quad (12)$$

The temperature-induced degradation of mechanical properties is taken into account through reduction coefficients:

$$f_{y,\theta} = k_{y,\theta} f_y \quad (13)$$

$$E_\theta = k_{E,\theta} E \quad (14)$$

where f_y, E - yield strength and modulus of elasticity at normal temperature
 $f_{y,\theta}, E_\theta$ - corresponding properties at temperature θ ;
 $k_{y,\theta}, k_{E,\theta}$ - temperature reduction coefficients.

Similarly, for an individual component of the joint, the serviceability condition can be written as:

$$\eta_i = \frac{S_{Ed,i}}{R_{d,\theta,i}} \leq 1.0 \quad (15)$$

where $S_{Ed,i}$ - design force in the i -th

component;
 $R_{d,\theta,i}$ - its design load-bearing capacity at temperature θ ;
 η_i - utilization factor.

$$\eta_{w,1} = 1.002 > 1.0 \quad (18)$$

For the bolts:

$$\eta_{b,1} = 0.992 \approx 1.0 \quad (19)$$

For the plates:

$$\eta_{p,1} = 0.018 \quad (20)$$

Stage 1. Analysis of the joint without fire protection.

At the first stage, the following fire analysis parameters were adopted:

- Fire resistance class: R60;
- Fire curve type: Standard fire;
- $\alpha_c = 25\text{W/m}^2\text{K}$;
- $\varepsilon_m = 0.80$;
- $\varepsilon_f = 1.00$.

No protective layer was applied to the joint elements.

The analysis showed that by the end of the considered fire exposure, the temperatures of the joint elements had reached:

$$\theta_{min,1} = 935.86^\circ\text{C} \quad (16)$$

$$\theta_{max,1} = 942.88^\circ\text{C} \quad (17)$$

The obtained temperature range indicates almost complete heating of the steel joint to the temperature level of the fire environment. This means a significant reduction in the mechanical properties of all steel components.

For the welds, we have:

From (18-20), it can be seen that the load-bearing capacity of the joint without fire protection is governed not by the plates, but by the local force-transfer elements – primarily the welds and bolts.

At temperatures close to 940°C , the base metal, welds, and bolts undergo severe temperature-induced degradation. As a result:

- the yield strength of steel decreases;
- the modulus of elasticity drops sharply;
- local deformations increase in stress concentration zones;
- the redistribution of forces in the joint becomes unfavorable;
- the weakest links are the welds and bolted elements.

Thus, the first stage confirmed that without fire protection, the joint does not satisfy the required R60 fire resistance rating (Fig. 2).

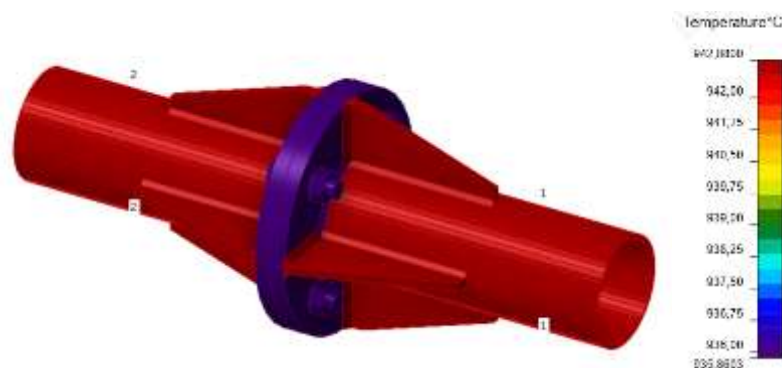


Fig. 2. Calculation result without applying the fire protection layer
Рис. 2. Результат розрахунку без введення вогнезахисного шару

Stage 2. Analysis of the joint with contour fire protection.

At the second stage, a fire-protective layer with the following parameters was introduced:

- Encasement: Contour;
- $c_p = 1700\text{J}/(\text{kg}\cdot\text{K})$;

- $d_p = 30\text{mm}$;
- $\lambda_p = 0.2\text{W}/(\text{m}\cdot\text{K})$;
- $\rho_p = 800\text{kg}/\text{m}^3$.

Then, in accordance with (9):

$$R_p = \frac{0.03}{0.2} = 0.15 \text{ m}^2\text{K/W} \quad (21)$$

In accordance with (10):

$$C_p = \rho_p c_p d_p = 800 \cdot 1700 \cdot 0.03 = 40800 \text{ J/(m}^2\text{K)} \quad (22)$$

The values in (21) and (22) indicate that

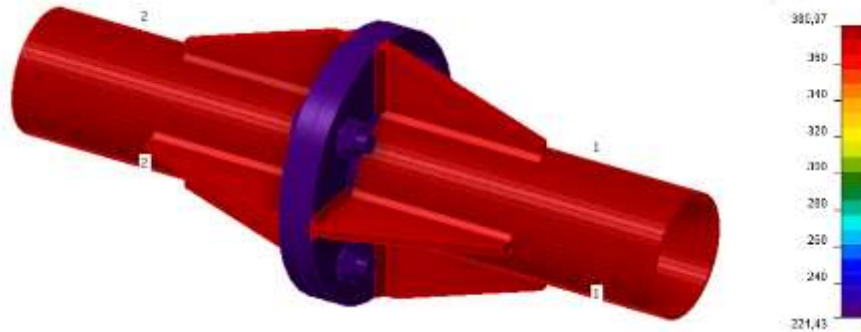


Fig. 3 Calculation result after applying the fire protection layer

Рис. 3 Результат розрахунку з введенням вогнезахисного шару

$$\theta_{min,2} = 221.43^\circ\text{C} \quad (23)$$

$$\theta_{max,2} = 380.97^\circ\text{C} \quad (24)$$

The maximum temperature reduction compared with the first stage is:

$$\Delta\theta_{max} = \theta_{max,1} - \theta_{max,2} = 942.88 - 380.97 = 561.91^\circ\text{C} \quad (25)$$

The relative reduction in the maximum temperature is:

$$\delta_\theta = \frac{942.88 - 380.97}{942.88} \cdot 100\% = 59.6\% \quad (26)$$

Thus, the applied fire-protective layer ensured a reduction in the maximum temperature by almost 60%.

After introducing the protection, the following utilization factors are obtained:

$$\eta_{p,2} = 0.000 \quad (27)$$

$$\eta_{b,2} = 0.127 \quad (28)$$

$$\eta_{w,2} = 0.477 \quad (29)$$

Let us compare the change in the utilization factors.

the protective layer has both sufficient thermal resistance and noticeable thermal inertia. This is precisely what physically explains the reduction in the heating rate of the steel elements.

After the introduction of fire protection, the following temperature range was obtained (Fig. 3):

For the bolts:

$$\frac{\eta_{b,1}}{\eta_{b,2}} = \frac{0.992}{0.127} \approx 7.81 \quad (30)$$

That is, the utilization level of the bolts decreased by approximately 7.8 times.

For the welds:

$$\frac{\eta_{w,1}}{\eta_{w,2}} = \frac{1.002}{0.477} \approx 2.10 \quad (31)$$

Therefore, for the welds, the utilization level decreased by more than two times.

The analysis shows that, in the investigated joint, it was precisely the introduction of fire protection that became the key factor in the transition from an unsatisfactory result to full compliance with the R60 requirement. This makes it possible to draw an important conclusion: for joints of this type, the effectiveness of fire protection should be assessed not only by the reduction in temperature, but also by the change in the utilization factors of the individual components, primarily the bolts and welds.

Taking into account (11-15), it is possible to trace a consistent cause-and-effect sequence.

At the first stage:

$$d_p = 0 \Rightarrow R_p \rightarrow 0 \quad (32)$$

which leads to a rapid increase in θ , and consequently to a reduction in $f_{y,\theta}$ and E_θ , that is:

$$\begin{aligned} \theta \uparrow &\Rightarrow k_{y,\theta} \downarrow, k_{E,\theta} \downarrow \Rightarrow \\ &\Rightarrow [K(\theta)] \downarrow, R_{d,\theta,i} \downarrow \end{aligned} \quad (33)$$

With the external load $\{F\}$ unchanged, this means an increase in the utilization factors:

$$R_{d,\theta,i} \downarrow \Rightarrow \eta_i = \frac{S_{Ed,i}}{R_{d,\theta,i}} \uparrow \quad (34)$$

At the second stage:

$$d_p > 0, \lambda_p, c_p \text{ i } \rho_p \Rightarrow R_p \uparrow, C_p \uparrow \quad (35)$$

and therefore:

$$\theta \downarrow \Rightarrow k_{y,\theta} \uparrow, k_{E,\theta} \uparrow \Rightarrow R_{d,\theta,i} \uparrow \Rightarrow \eta_i \downarrow \quad (36)$$

Equations (32-36) formally explain the obtained numerical result: fire protection does not change the external load, but it changes the temperature regime, and through it, the stiffness and load-bearing capacity of all components of the joint.

CONCLUSIONS AND PERSPECTIVES FURTHER RESEARCH

1. It has been established that the flanged joint of tubular elements without fire protection does not satisfy the R60 fire resistance requirement. In this case, the joint elements are heated to temperatures of 935.86-942.88°C, while the critical components are the welds and bolts.
2. According to the results of the first stage of the analysis, the following values were obtained: Analysis = 17.2%, Plates = 1.8%, Bolts = 99.2%, Welds = 100.2%. This indicates the loss of the required serviceability of the joint as a result of the temperature-induced degradation of the

local force-transfer elements.

3. The introduction of contour fire protection with the parameters $d_p = 30$ mm, $\lambda_p = 0.2$ W/(m·K), $c_p = 1700$ J/(kg·K), $\rho_p = 800$ kg/m³ provided a significant reduction in temperature to 221.43-380.97°C. The maximum temperature decreased by 561.91°C, or approximately 59.6%.
4. After the introduction of fire protection, the joint fully satisfied the R60 requirement: Analysis = 100.0%, Plates = 0.0%, Bolts = 12.7%, Welds = 47.7%. Therefore, the applied fire-protective layer is sufficient to ensure the serviceability of the joint under the specified fire exposure.
5. It has been shown that for joints of this type, the governing components under fire conditions are not the plates, but the welds and bolted elements. Therefore, when assessing the fire resistance of joints, primary attention should be paid specifically to the local force-transfer components.
6. The proposed approach, which combines the thermal problem, the temperature-dependent mechanical formulation, and the comparison of cases without protection and with protection, is suitable for the practical substantiation of fire-protection parameters for complex steel joints.

ETHICAL DECLARATIONS

The authors have no relevant financial or non-financial interests to report.

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ОЦІНКА ВОГНЕСТІЙКОСТІ ФЛАНЦЕВОГО ВУЗЛА СТАЛЕВОЇ ФЕРМИ

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Анотація. У статті розглянуто особливості чисельної оцінки вогнестійкості просторового вузла стикування трубчастих елементів із кільцевим фланцем і радіальними ребрами жорсткості. Актуальність дослідження зумовлена тим, що у сталевих конструкціях в

умовах пожежі граничний стан часто визначається не загальною міцністю основного перерізу, а локальною роботою вузлів, де відбувається складна взаємодія основного металу, зварних швів, болтів, контактних зон і ребер жорсткості. Саме вузли є найбільш чутливими до зниження фізико-механічних властивостей матеріалу при нагріванні, оскільки в них концентруються напруження, змінюється схема передавання зусиль і швидше проявляються локальні механізми руйнування. Метою роботи є встановлення впливу вогнезахисного шару на температурний режим і несучу здатність досліджуваного вузла при дії стандартної пожежі відповідно до вимоги класу вогнестійкості R60.

Дослідження виконано у два послідовні етапи. На першому етапі проведено пожежний розрахунок вузла без вогнезахисту з використанням таких вихідних параметрів: стандартна пожежна крива, коефіцієнт конвективного теплообміну $\alpha_c = 25 \text{ W/m}^2\text{K}$, коефіцієнт випромінювальної здатності поверхні сталі $\epsilon_m = 0,80$ та коефіцієнт випромінювання пожежного середовища $\epsilon_f = 1,00$. Розрахунок показав інтенсивне нагрівання елементів вузла до температур приблизно $935,86\text{--}942,88^\circ\text{C}$. Однак після введення контурного вогнезахисту з такими параметрами: товщина шару $d_p = 30 \text{ мм}$, теплопровідність $\lambda_p = 0,2 \text{ Вт/(м}\cdot\text{К)}$, питома теплоємність $c_p = 1700 \text{ Дж/(кг}\cdot\text{К)}$ та густина $\rho_p = 800 \text{ кг/м}^3$, температура знизилася до діапазону $221,43\text{--}380,97^\circ\text{C}$.

Ключові слова: вогнестійкість; сталеві конструкції; вогнезахист; метод скінченних елементів.

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