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THE NECESSITY OF ACCOUNTING FOR SECONDARY EFFECTS IN FIRE RESISTANCE CALCULATIONS OF BUILDINGS AND STRUCTURES

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Abstract. Conventional fire resistance calculations evaluate individual structural elements (beams, columns, walls, floor slabs) based on their material properties. Current analysis methods, including standard fire resistance tests, fail to account for actual structural interactions in building systems.

Building structures inherently function as interconnected systems through various connections (pinned, hinged, continuous, or multi-span configurations). Consequently, either the complete failure of a load-bearing element during fire exposure or even just the alteration of its stiffness properties can trigger significant force redistribution. This may lead to structural behavior changes ranging from localized damage to potential progressive collapse. Furthermore, the performance of structural connections during and after fire exposure critically influences the global stability of the building's load-bearing system.

The standard fire resistance calculation criteria are based on two key requirements: firstly, a building's fire resistance duration must surpass the regulatory-mandated timeframe determined by its specific occupancy classification, and secondly, the overall fire resistance depends directly on the fire resistance ratings of the building's principal structural components.

All buildings must comply with fundamental functional requirements specified in regulatory documents, including provisions for safe evacuation routes, control of internal fire spread pathways, prevention of external fire propagation, and ensuring fire service access to firefighting equipment.

It should be emphasized that building codes establish minimum necessary standards for protecting occupant and public safety, focusing primarily on health and life protection rather than structural preservation - though damage limitation measures, while not explicitly required, are not precluded by these regulations.

Building codes are not intended to mitigate financial losses from fire incidents, a crucial consideration for designing building fire protection systems when regulatory requirements alone prove insufficient to satisfy client specifications.

The paramount requirement for building structures during internal fire propagation scenarios is as follows:

A building must be designed and constructed to maintain structural stability for a code-specified duration during fire exposure.

The requirements for maintaining a building's stability during a reasonably sufficient time period traditionally correspond to the structure's survival time in standard fire resistance tests.

Consequently, in addition to mechanical effects, one must consider changes in both thermal and mechanical properties of steel resulting from structural heating during a fire.

Keywords: fire resistance; heat capacity; thermal conductivity; fire

PROBLEM STATEMENT

Generally, the main load-bearing elements of buildings and structures include steel, reinforced concrete (RC), stone, and timber components. These structural types are analyzed separately in structural design. Accordingly, fire protection requirements for each material (such as concrete, steel, timber, stone, aluminum, etc.) are specified in the fire safety sections of the respective design codes [1–6]. These basic construction materials differ in their physical and mechanical properties and respond differently when exposed to high temperatures during a fire [7–21].

The strength and rigidity of steel and reinforced concrete structural elements decrease with increasing temperature, and the decrease in these characteristics is particularly significant in the temperature range between 400 and 700°C.

For example, concrete is a heterogeneous material whose fire-resistant properties are ensured by the corresponding properties of fillers and cement paste. Concrete has low thermal conductivity (50 times lower than steel), so it heats up very slowly during a fire. It is precisely because of its low thermal conductivity that reinforced concrete structures have good fire resistance. However, the specific heat capacity of concrete varies depending on the moisture content (in percent) by weight of the concrete structure (Fig. 4).

It is believed that concrete heated above 500°C loses its strength and rigidity, while concrete heated to temperatures below 500°C retains its characteristics as at normal temperatures.

In addition to mechanical loads, changes in the thermal and mechanical properties of steel, concrete, and reinforcement under fire exposure must also be considered.

Hot-rolled carbon steel begins to lose strength at temperatures above 300°C, with a progressive reduction up to around 800°C. Beyond this point, its residual strength declines more gradually until it reaches its melting point, approximately 1500°C. This behavior is typical of all hot-rolled steels. Cold-formed steel, including reinforcing bars, loses strength more rapidly above 300°C. In addition to reductions in strength and stiffness, both types of steel also exhibit creep at temperatures exceeding 450°C. The specific heat capacity of carbon steel increases sharply at around 730°C due to the phase transformation of ferrite–pearlite into austenite (see Fig. 1).

Therefore, when evaluating the performance of steel structures under fire conditions, it is essential to account not only for mechanical loads but also for changes in the thermal and mechanical properties of the material.

Fire resistance calculations: particularly those carried out in accordance with Eurocodes [1–6] are typically based on one of four fire exposure scenarios: standard fire, hydrocarbon fire, external fire, or smoldering fire. Each scenario is defined by its own temperature–time curve. However, actual fire conditions can be either more or less severe than the standard fire curve, depending on the specific characteristics of the space where the fire occurs, such as geometry, ventilation, and fire load.

In this context, the fire resistance of building structures is evaluated using three performance criteria:

- Load-bearing capacity (R) – the ability to sustain mechanical loads during fire exposure;
- Integrity (E) – the ability to prevent the passage of flames and hot gases;
- Thermal insulation (I) – the ability to limit temperature rise on the unexposed side.

The following discussion will focus primarily on reinforced concrete and steel structures. According to fire safety standards such as EN 1992-1-2 and EN 1993-1-2, fire resistance may be assessed using several different methods, including:

According to EN 1992-1-2 (for reinforced concrete structures), fire resistance can be assessed using:

1) Tabulated data;
2) Simplified calculation models, such as the 500°C isothermal method and the zonal method;
3) Advanced (refined) calculation models.

According to EN 1993-1-2 (for steel structures), the following methods are available:

1) Calculated resistance method, accounting for both spatially **homogeneous** and **non-uniform** temperature distributions;
2) Critical temperature method;
3) Advanced (refined) calculation models.

The most commonly accepted method for demonstrating compliance with building codes and fire resistance regulations relies on tabulated data derived from standardized fire resistance tests.

The Eurocodes offer a comprehensive spectrum of calculation approaches, ranging from guidelines based on standard fire resistance ratings and tabulated data to advanced calculation methods that incorporate real fire scenarios and assess the overall behavior of the building during fire exposure.

Simplified calculation methods, combined with an assessment of the relevant load under fire limit conditions, rely on indicators of material property degradation due to elevated temperatures. Fire resistance is then determined using reduction factors that correspond to the calculated thermal exposure, and these are compared against the loads applied to the structure during a fire. Verification can be performed with respect to both fire resistance criteria and structural temperature limits.

Refined calculation methods typically employ complex finite element models.

The tabulated data for reinforced concrete structures: specifying minimum dimensions and minimum concrete cover thickness (protective layer) are based on the need to: limit the temperature rise of protected surfaces, and ensure structural stability for a sufficiently long period. These requirements are met by providing adequate concrete thickness, which restricts the average temperature rise of embedded surfaces to **140°C,** and by ensuring a sufficient protective layer thickness to limit the temperature rise of the reinforcement to: **550°C** (for conventional reinforcement), or **450°C** (for prestressed reinforcement elements).

Simplified calculation methods for reinforced concrete structures are provided along with their corresponding strength reduction values. **EN 1992-1-2** includes two approaches: the 500°C isothermal method, and the zonal method (Annex B).

The Eurocode-compliant calculation procedure consists of the **following stages:**

1) Determination of design fire characteristics (selection of a suitable fire model scenario and appropriate fire model in accordance with **EN 1991-1-2);**
2) Thermal analysis of temperature distribution in the structure (calculation of temperature rise in structural elements according to **EN 1992-1-2 and EN 1993-1-2);**
3) Assessment of structural response to fire (mechanical analysis of the structure's behavior under fire exposure according to **EN 1992-1-2 and EN 1993-1-2).**

Currently, there has been a shift in design priorities: calculation-based design now **takes precedence over** test-based approaches and serves as an alternative computational method.

As demonstrated, fire resistance calculations for building structures must account for numerous input parameters and material-specific properties.

The Eurocodes specify that the primary objective of these calculations is to ensure structural elements maintain adequate load-bearing capacity under elevated temperatures for a required duration.

The element must resist the applied load until the ultimate limit state of load-bearing capacity is reached during a fire. This design principle is similar to the principles of designing structural elements at normal temperatures. Thus, it is first necessary to know the temperature of the structural element (or its distribution within the element).

The temperature distribution across the cross-section differs significantly for steel and reinforced concrete elements. While for steel elements, due to the homogeneity of the material and the speed of temperature propagation, it is possible to assume a uniform temperature across the cross-section, for reinforced concrete elements, the change in the temperature field across the cross-section is determined by solving a nonlinear non-stationary heat conduction equation.

The strength test of a reinforced concrete cross-section, performed during the assessment of fire resistance, differs from the usual strength test in that here:

- a special (emergency) limit state is considered, and therefore, in addition to fire exposure, only standard constant and continuous loads are taken into account

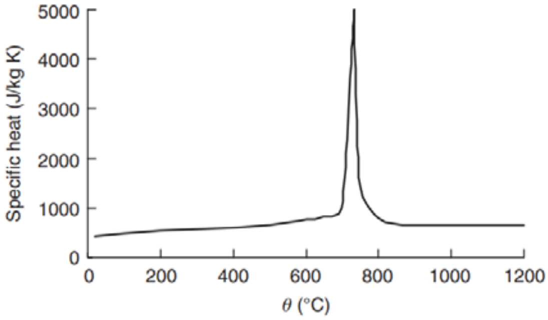


Fig. 1. Specific heat of carbon steel [8].
Рис.1. Питома теплоємність вуглецевої сталі [8].

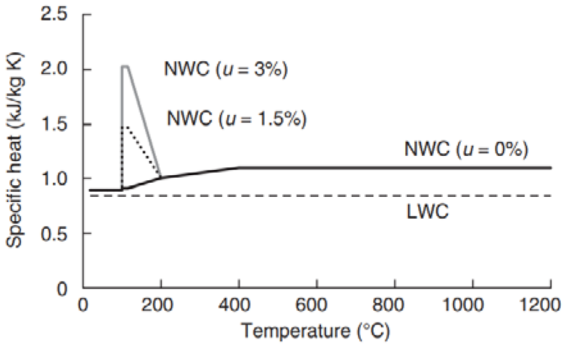


Fig. 2. Specific heat of concretes [8].
Рис. 2. Питома теплоємність бетонів [8].

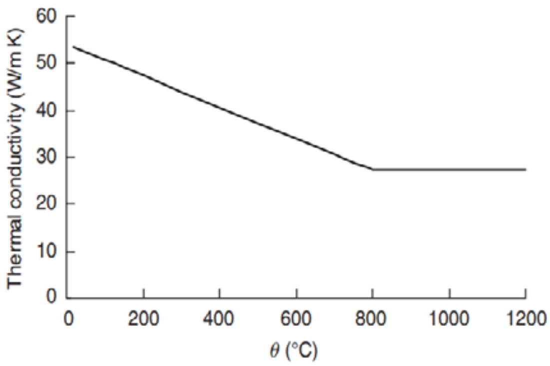


Fig. 3. Thermal conductivity of carbon steel [8].
Рис. 3. Теплопровідність вуглецевої сталі [8].

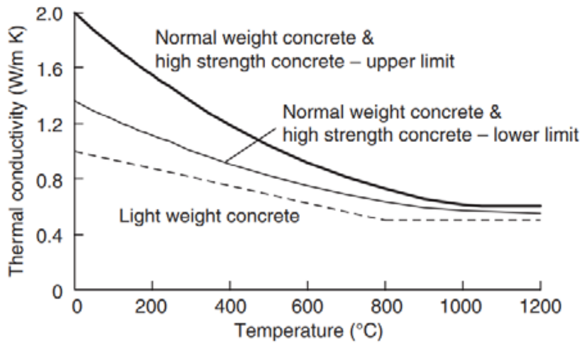


Fig. 4. Thermal conductivity of concretes [8].
Рис. 4. Теплопровідність бетонів [8].

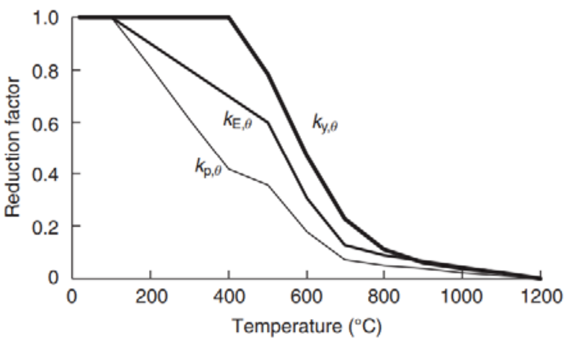


Fig. 5. Strength reduction factors for carbon steel [8].
Рис. 5. Коефіцієнти зниження міцності вуглецевої сталі [8].

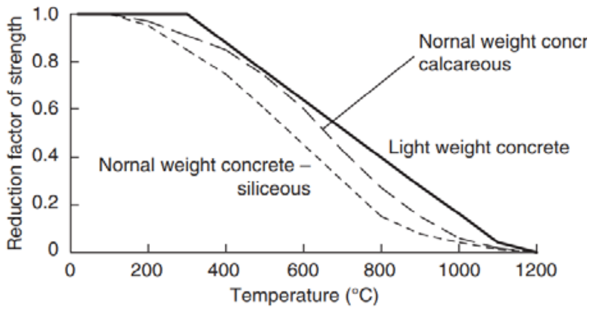


Fig. 6. Reduction in strength for normal- and lightweight concretes [8].
Рис. 6. Зниження міцності для звичайних та легких бетонів [8].

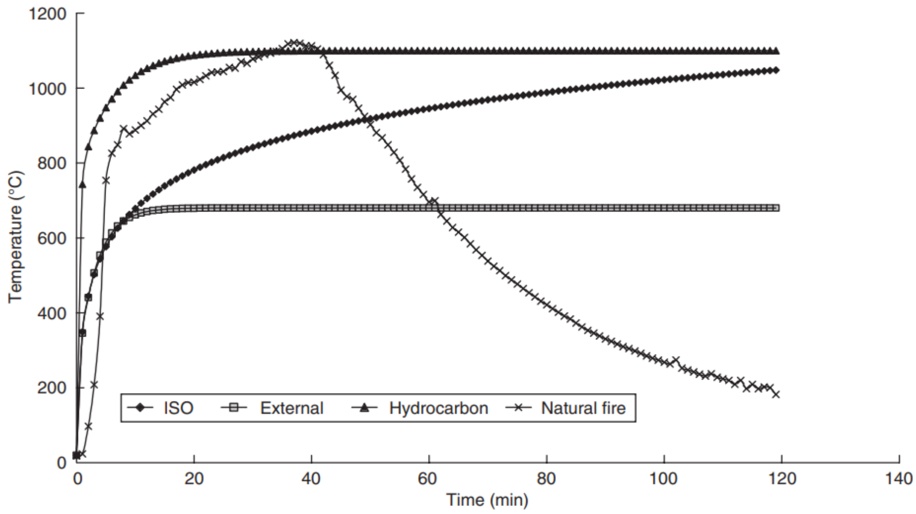


Fig. 7. Nominal fire curves – comparison with results from natural fire test [8].
Рис. 7. Номінальні криві пожежі – порівняння з результатами випробувань на стандартну пожежу [8].

- the change in the physical and mechanical parameters of materials (concrete and reinforcement) caused by heating is taken into account, and since the temperature field is uneven, these characteristics are determined for each calculation point of the cross section independently.

When calculating RCS, only the values of constant and long-term loads at time $t = 0$ are taken into account. In this case, the characteristic values of material strength are used.

The design strengths of concrete and reinforcement, as well as their elasticity modules, are taken into account based on the values of these quantities for the specified classes of concrete and reinforcement, but are adjusted to take into account the heating temperature.

In addition, the critical deformation values of reinforcement and concrete also depend on temperature, since the stress-strain relationship changes with temperature.

When calculating steel structures, only the values of constant and long-term loads and the characteristic values of material strength are taken into account.

In addition to the direct impact of fire temperature, secondary effects associated with thermal expansion should also be taken into account.

In addition to the direct impact of fire temperature, **secondary effects** associated with thermal expansion should also be taken into account.

After all, everything mentioned above concerned a case where an isolated element was considered without accounting for its interaction with other parts of the system. In particular, nothing restrained the thermal expansion of the element.

Next, we will use an example to demonstrate the role that limited deformation of an element can play (the example was kindly provided by D.Eng.Sc. A.V. Perelmuter).

The frame of a three-story building was analyzed using SCAD software. Column spacing is 6×9 m, with 4 m floor heights. The steel structural elements lack fire protection. The fire scenario is applied to the first floor section, which is highlighted in color (Fig. 8). Using the Assembly mode, temperature changes at specific time intervals during the fire were modeled for the section's elements.

The columns are fabricated from 35K1 I-beams (GOST 26020-83), the main beams from 46B1 I-beams (GOST 26020-83), and the secondary beams from No. 20 I-beams (GOST 8239-89).

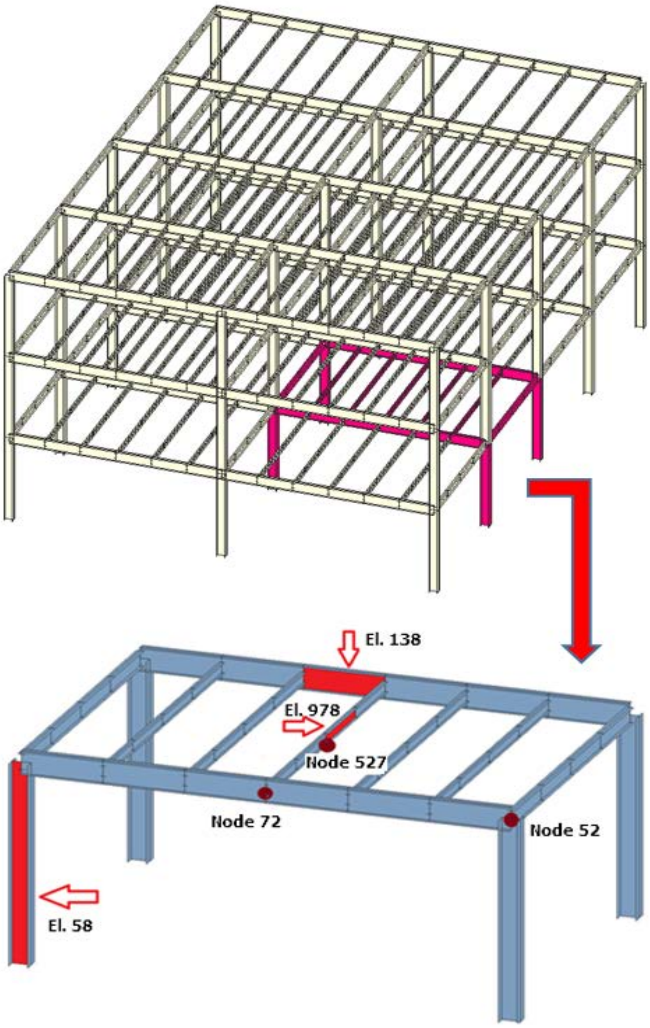


Fig. 8. General view of the calculation scheme.
Рис.8. Загальний вигляд розрахункової схеми.

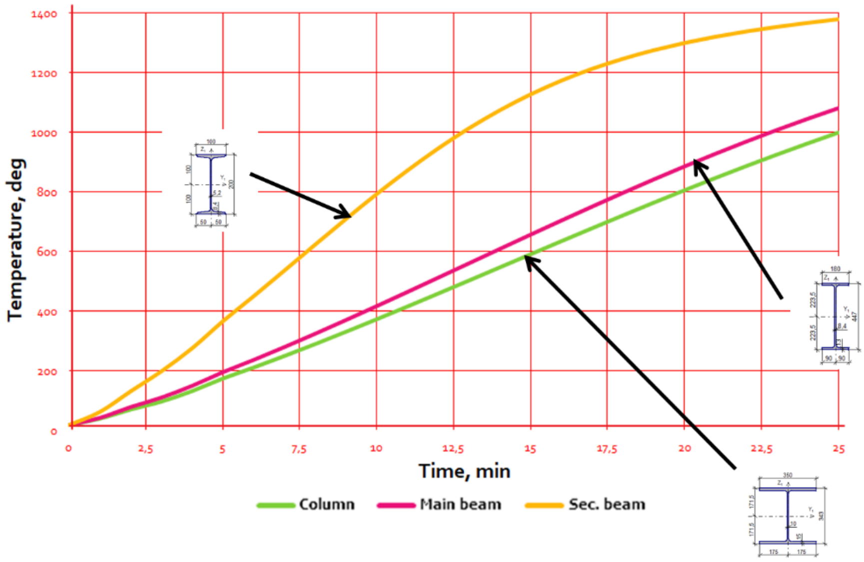
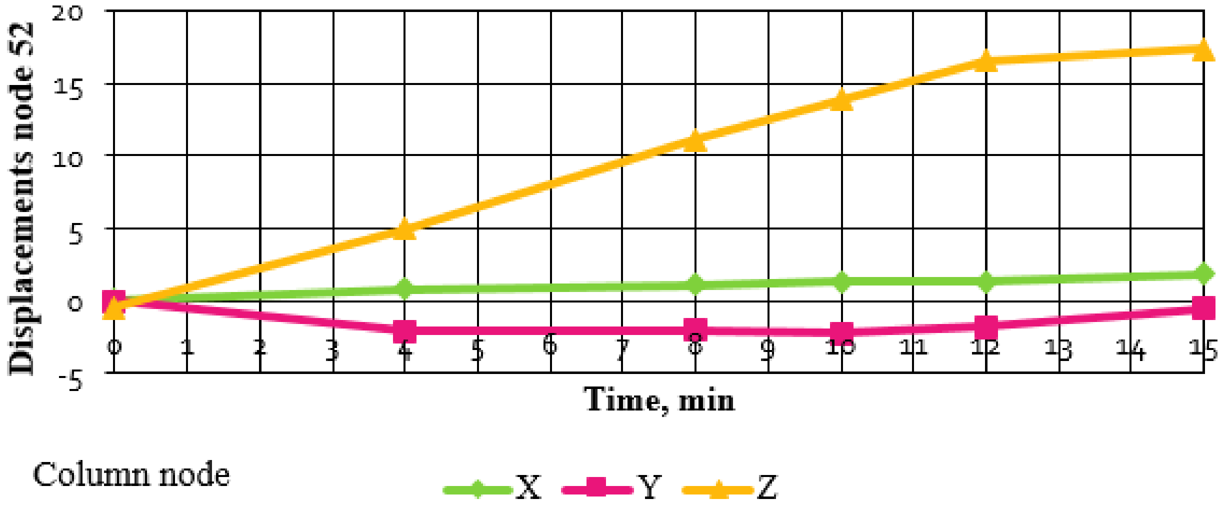


Fig. 9. Temperature change graph in the tracked elements.
Рис.9. Графік зміни температури у відстежуваних елементах.



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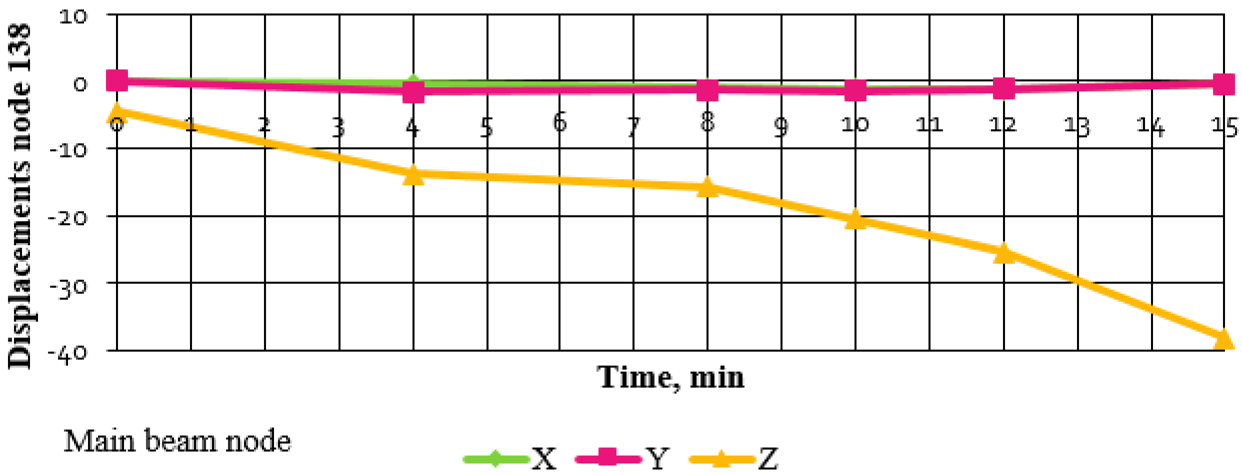

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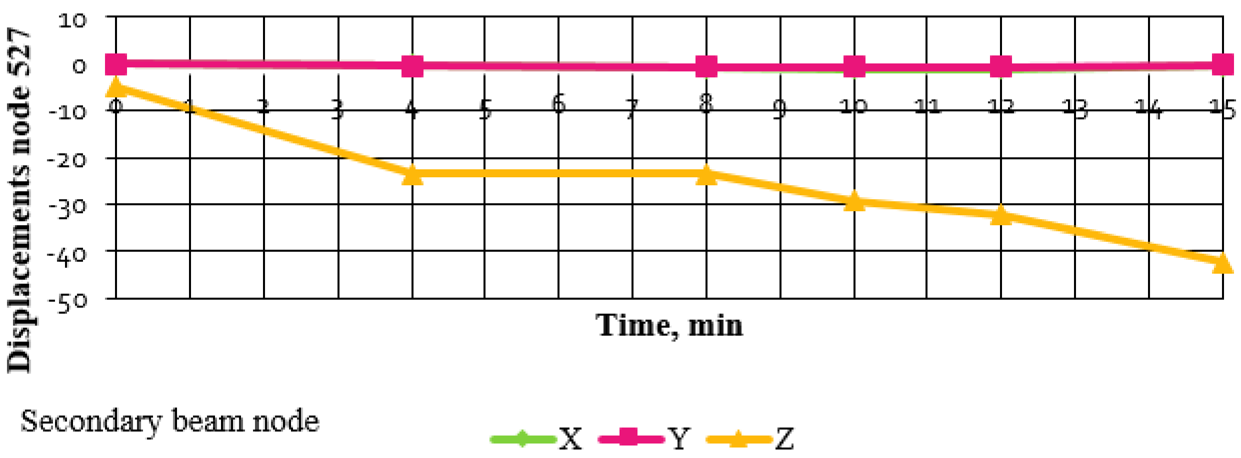

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Fig. 10. Design displacement values, mm: *a* – node 52; *b*- node 138; *c* - node 527.
Рис. 10. Розрахункові значення переміщень, мм: *а* – вузол 52; *б* – вузол 138; *в* – вузол 527.

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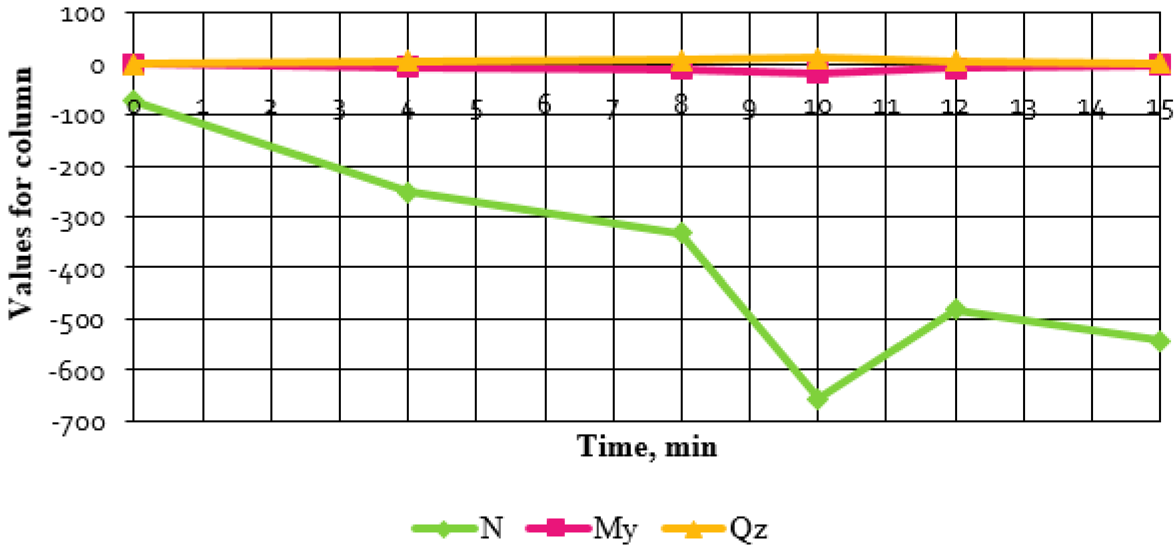

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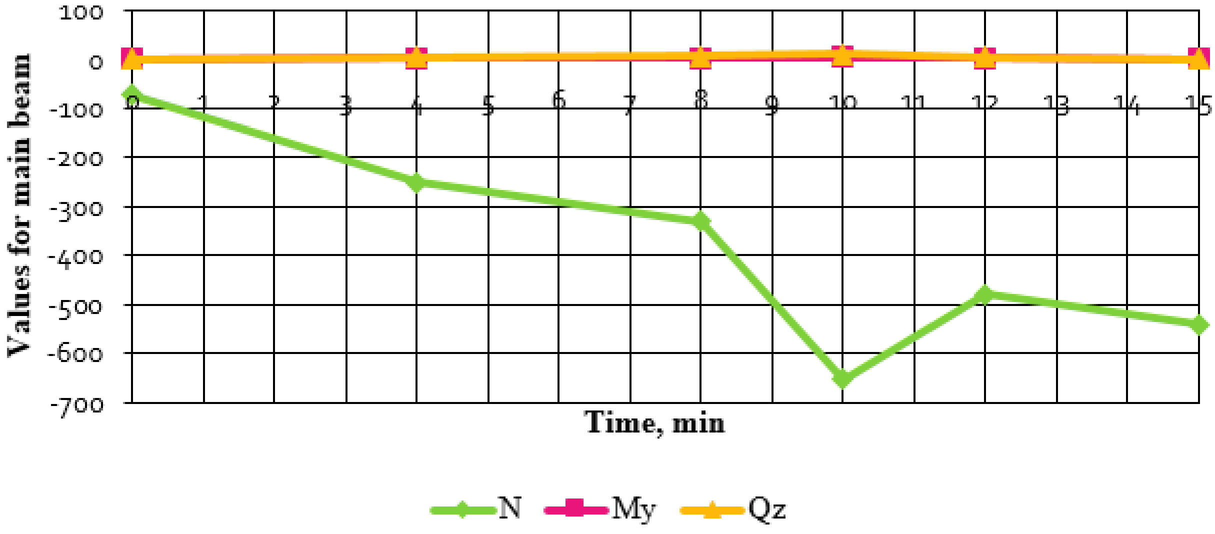

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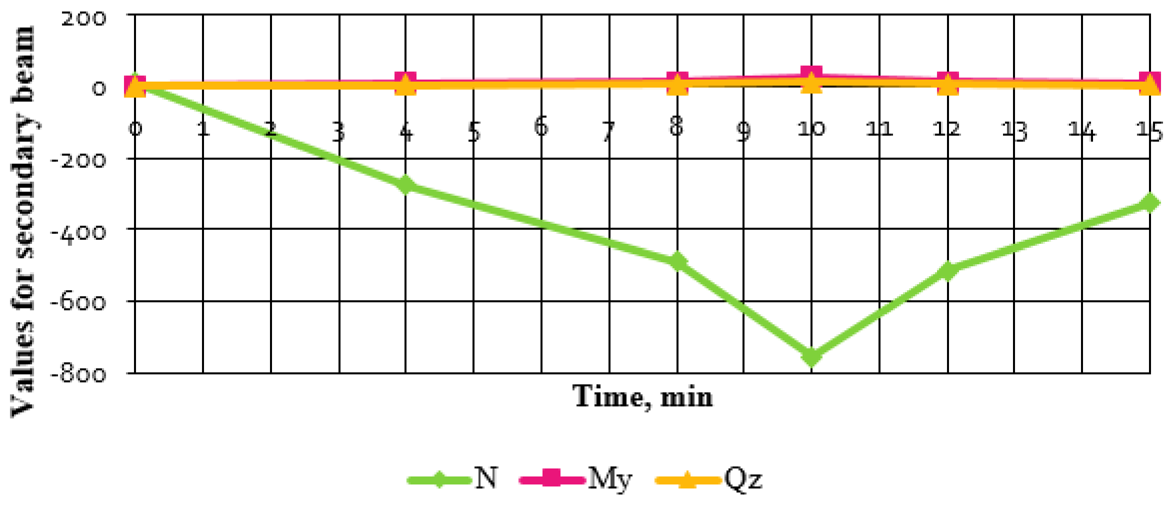

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Fig. 11. Design force values: *a* – column; *b* – main beam; *c* – secondary beam.
Рис. 11. Розрахункові значення зусиль**:** *а* – колона; *б* – головна балка; *в* – другорядна балка.

Table 1. Temperature change in selected elements and reduction factors
Табл. 1. Зміна температури у вибраних елементах та коефіцієнти зниження

t, min	Temperature			k_E			k_f		
	Col.	Main beam	Sec. beam	Col.	Main beam	Sec. beam	Col.	Main beam	Sec. beam
0	20	20	20	1,000	1,000	1,000	1,000	1,000	1,000
1	41	44	64	1,000	1,000	1,000	1,000	1,000	1,000
2	72	80	133	1,000	1,000	1,000	1,000	1,000	1,000
3	99	112	200	1,000	1,000	1,000	1,000	1,000	1,000
4	134	152	277	1,000	1,000	0,897	1,000	1,000	1,000
5	176	197	368	1,000	1,000	0,776	1,000	1,000	1,000
6	213	238	452	0,983	0,949	0,664	1,000	1,000	0,920
7	251	281	538	0,932	0,891	0,490	1,000	1,000	0,760
8	291	326	625	0,878	0,832	0,297	1,000	1,000	0,510
9	332	372	711	0,823	0,771	0,253	1,000	1,000	0,240
10	374	419	794	0,767	0,709	0,210	1,000	0,930	0,187
11	417	466	873	0,710	0,645	0,169	0,880	0,850	0,150
12	461	514	947	0,653	0,560	0,131	0,830	0,770	0,116
13	504	562	1014	0,588	0,421	0,096	0,780	0,603	0,085
14	548	610	1074	0,460	0,305	0,037	0,640	0,473	0,058
15	658	1128	0,333	0,280	0,037	0,522	0,344	0,033	0,043

Table 2. Results of strength tests
Табл. 2. Результати перевірок на міцність

Time, min	Column			Main beam			Secondary beam		
	Forces in element 58			Forces in element 138			Forces in element 978		
	N, kN	My, kNm	Qz, kN	N, kN	My, kNm	Qz, kN	N, kN	My, kNm	Qz, kN
0	-729,0	0,00	0,00	-727,0	0,00	0,00	85,7	2,3,4	12,5
4	-2512,9	-68,5	45,0	-2506,9	21,5	45,0	-2760,1	105,8	48,3
8	-3317,6	-104,5	66,6	-3309,6	28,7	66,6	-4892,3	125,2	56,2
10	-6559,5	-173,0	111,6	-6543,6	50,2	111,6	-7566,7	254,4	117,0
12	-4817,6	-80,9	50,7	-4805,6	20,3	50,7	-5148,4	136,4	61,5
15	-5418,1	-15,8	9,7	-5404,1	3,4	9,7	-3276,5	98,9	42,2

The columns are heated on all four sides, while both main and secondary beams are heated on three sides. The permanent floor load is G = 4 kN/m², and the live load is Q = 1 kN/m². The live load duration factor is taken as 0.25.

Internal forces are monitored for the column (Element 58), at mid-span of the main beam (Element 138) and mid-span of the secondary beam (Element 978); these elements are indicated in Figure 8. Control Nodes 52, 72, and 527 are designated for displacement tracking. Figures 10 and 11 present the displacement time histories for the specified nodes and the internal force variation diagrams for the specified elements, respectively. Table 1 provides the temperature evolution of selected elements during 1-15 minutes of fire exposure along with corresponding reduction factors (k_E for linear elastic deformations and k_f for effective yield strength).

As evident from the calculation results (Table 2), the main beam fails the strength test between the 4th and 8th minutes, the secondary beam at the 4th minute, and the column between the 8th and 10th minutes.

In the examined case, the so-called secondary effects generated significant longitudinal forces in elements that were typically considered to experience only bending moments.

For the columns, the partial floor fire exposure proved particularly unfavorable. Had all floor columns been heated uniformly, the critical temperature assessment would have better matched the structure's predicted behavior.

All preceding calculations were performed under the assumption of strength verification according to design standards (while accounting for variations in steel's yield strength and elastic modulus). In other words, plastic behavior was essentially disregarded.

If plastic deformation is permitted, thermally-induced forces may decrease significantly.

Furthermore, several critical questions must be addressed when performing such calculations:

1) The building contains structural elements with varying fire resistance ratings. Should the structural analysis use the minimum or maximum rating as the governing criterion for the entire system?

2) The fire modeling approach remains ambiguous - particularly regarding the maximum fire spread area to consider when the facility lacks fire compartmentalization.

Similar secondary effects may also develop in reinforced concrete structures [7], particularly in joints where indirect fire effects significantly impact their performance characteristics. These effects can include increased support moments in continuous structures, thermal expansion-induced compressive forces, and eccentric loading resulting from large deflections or bending of flexible reinforced concrete members (columns, beams).

CONCLUSIONS

Based on extensive research into temperature effects on building frames, the following conclusions can be drawn:

1) Fire resistance calculations for isolated elements fail to fully represent the structural frame's behavior;
2) Secondary effects — including thermal expansion of individual frame elements and their interaction with unheated members — must be considered in addition to direct fire temperature impacts.

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НЕОБХІДНІСТЬ ВРАХУВАННЯ ВПЛИВІВ ВТОРИННИХ ЕФЕКТІВ ПРИ РОЗРАХУНКУ БУДІВЕЛЬ І СПОРУД НА ВОГНЕСТІЙКІСТЬ

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Анотація. Традиційний розрахунок на дію пожежі відноситься до експлуатаційних властивостей окремих елементів конструкцій (балки, колони, стіни, плити перекриття). В методиках аналізу (випробування вогнестійкості при стандартній пожежі) не розглядається взаємодія конструктивних елементів в реальних будівлях.

Однак, потрібно зазначити, що конструкції будівлі перебувають у певних взаємозв’язках та взаємодії одна з одною (защемлення, шарнір, одно- та багатопрольотна конструкція і т.д.). Тому виключення внаслідок пожежі із роботи якогось несучого елемента або навіть зміна його жорсткісних характеристик може призвести до значного перерозподілу зусиль у будівлі та подальшої її поведінки аж до можливого руйнування частини або всієї будівлі. Також робота стиків конструкцій в умовах пожежі або після неї є важливим фактором з точки зору збереження загальної стійкості несучої системи будівлі.

Загально прийнятими критеріями розрахунку на вогнестійкість є наступні вимоги: час вогнестійкості будівлі повинен перевищувати час, встановлений нормативними документами на основі оцінки приналежності будівлі до певної цільової групи.

Вогнестійкість будівлі залежить від меж вогнестійкості її основних конструктивних частин.

Всі будівлі повинні відповідати певним функціональним вимогам, що включає засоби евакуації, визначення шляхів розповсюдження пожежі в середині будівлі, визначення шляхів розповсюдження пожежі ззовні будівлі, визначення шляхів доступу пожежної служби до засобів гасіння пожежі, що викладено у нормативних документах.

Важливо зазначити, що будівельні норми і правила призначенні лише для забезпечення розумно необхідних стандартів із захисту здоров'я та безпеки людей, які знаходяться всередині і за межами будівлі. В них не розглядаються заходи з обмеження шкоди будівельним конструкціям, але така мета не виключається.

Будівельні норми і правила також не призначені для забезпечення мінімізації фінансових втрат, спричинених пожежею. Це має велике значення для проектування протипожежного захисту будівель і споруд, коли для виконання умов замовника не вистачає вимог нормативних документів.

Найбільш важлива вимога до будівельних конструкцій в умовах розповсюдження пожежі всередині будівлі полягає в наступному:

Будівля повинна бути запроектована і зведена таким чином, щоб у випадку пожежі її стійкість зберігалась протягом розумно достатнього періоду часу.

Вимоги до збереження стійкості будівлі протягом розумно достатнього періоду часу традиційно відносяться до часу, необхідного для «виживання» конструкції у ході стандартних випробувань на вогнестійкість.

Тому поряд з механічними впливами слід враховувати зміну теплотехнічних та механічних властивостей сталі під впливом нагрівання конструкції від пожежі.

Ключові слова: вогнестійкість; теплоємність; теплопровідність; пожежа.

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