CZASOPISMO INŻYNIERII LĄDOWEJ, ŚRODOWISKA I ARCHITEKTURY
JOURNAL OF CIVIL ENGINEERING, ENVIRONMENT AND ARCHITECTURE
JCEEA, t. XXXV, z. 65 (2/18), kwiecień-czerwiec 2018, s. 103-114, DOI:10.7862/rb.2018.28

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TEMPERATURE DISTRIBUTION IN A STEEL BEAM-TO-COLUMN JOINT WHEN EXPOSED TO FIRE. PART 2: FLANGE-PLATES AND WEB-CLEATS JOINT

In the second part of this paper the temperature distribution is analysed for a thermally uninsulated steel beam-to-column flange-plates and web-cleats joint after 15 minutes of its exposure to a fully developed fire. Two types of such a joint are considered separately, firstly the pure steel connection with a beam and a column evenly heated on all four sides and then the analogous one, but with a massive reinforced concrete floor slab lying on the upper beam flange. In the latter case the joint beam is heated only on three sides. In addition, in each of the analysed joint the beams of two sizes are analysed independently for comparative purposes. Those that are made of the bigger I-section have a more slender web, while the smaller ones are more stocky. However, the smaller I-section heats faster than the bigger one because the section factor calculated for it has a greater value. In general, it can be concluded that in all the joints considered by the authors the steel temperature turned out to be much lower than that measured outside these joints. Moreover, a significant difference is observed in the temperature values identified in the beam web and in the beam flanges. Finally, the temperature distribution obtained from a numerical simulation and identified in the selected cross-sections of the joint beam in the case of a joint with adjacent floor slab is referred to the analogous distribution recommended for use in such circumstances in the standard EN 1993-1-2.

Keywords: beam-to-column joint, flange-plates joint, web-cleats joint, temperature distribution, section factor, numerical simulation

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1. Introduction

In the first part of this paper the temperature distribution in a steel end-plate beam-to-column joint was analysed in detail, identified by a numerical analysis after 15 minutes of its simulated exposure to a fully developed fire. Two types of such a joint were considered separately, the first which was pure steel with heating the beam on all four sides and the second one, with a massive reinforced concrete floor slab lying directly on the upper beam flange, in which this beam was heated only on three sides. It has been pointed out that in fire conditions the temperature observed in individual joint components is always much lower than that recorded outside the joint. This diversity results mainly from the large accumulation of the steel sheets added in this zone that increase the effective thickness of the steel which has to be heated.

In this part of the paper, a different steel beam-to-column joint type is chosen for consideration, the one with flange plates and web cleats. Similarly as before, both a purely steel connection and a connection with an adjacent floor slab are analysed. Additionally, in each of the considered joint type it is assumed that the beams are made of the sections of two sizes, first from the IPE 330 steel section which is relatively stocky and then from the IPE 500 steel section, significantly more slender. The primary goal of the authors is now both a qualitative and a quantitative verification of the correctness of the temperature distribution recommended for use for such a joint in the standard EN 1993-1-2 [1].

2. The case of a pure steel joint

Let us start the detailed analysis of the temperature distribution observed in the joint under consideration after 15 minutes of its exposure to a fully developed fire from simulating the response of the numerical model of such a pure steel joint to a direct fire influence. This model is prepared using the ANSYS environment [2] and it is presented in detail in Fig. 1. As one can see the joint beam made of the IPE steel section (first of the IPE 330 and then of the IPE 500) is modelled as connected to the column made of the HEB wide-flange steel section (precisely of the HEB 180 when the beam is made of the IPE 330 and of the HEB 220 in the case when the beam is made of the IPE 500). This connection is realized by the web cleats composed of two steel angles of size L80x80x8 in the first case and L80x80x10 in the second case, set oppositely one to the other, with three rows of bolts M16. Such web cleats are attached to the column flange also by three bolts of the same size for each angle. The stiffness of the joint is ensured mainly by the flange plates of thickness 12mm in the first case when the section IPE 330 is considered and of thickness 16 mm in the second case when the section IPE 500 is assumed to the analysis, covering the beam from the bottom and from the top and attached to flanges of such the beam by four rows of bolts of size M20 with two bolts in each row. Moreover, two double-sided horizontal ribs of thickness 10 mm (in the first case) or 16 mm (in the second case) are used in the levels defined by the upper and by the bottom beam flanges to stiffen the web of the column. There is neither a diagonal rib nor a wedge added at the bottom of the joint between the beam and the column flange.

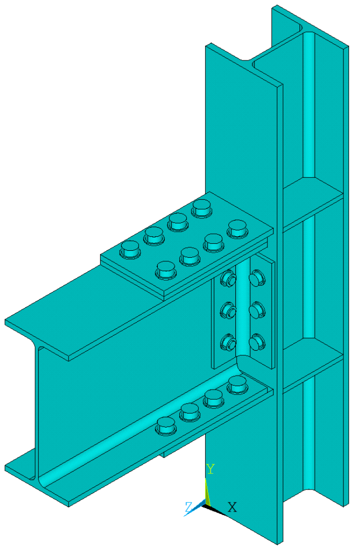


Fig. 1. Scheme of a numerical model of the pure steel joint considered in the example

The joint model presented above was evenly heated on all sides for 15 minutes by a simulated fire developing in accordance with a conventional standard scenario [3]. In Fig. 2a the temperature distribution is shown in detail identified in such a joint after this time of a fire exposure in the case when the IPE 330 steel section was assumed as a frame beam while in Fig. 2b the analogous temperature distribution but this time that related to the joint with the IPE 500 steel section used for this beam. As one can see, at any time of a fire the steel temperature observed at the joint itself is always significantly lower than that measured outside the joint. Let us note that in the first of the cases presented above the difference between the hottest and the least hot points of the joint turned out to be almost 200 degrees Celsius, however, in the second case it exceeded 250 degrees Celsius.

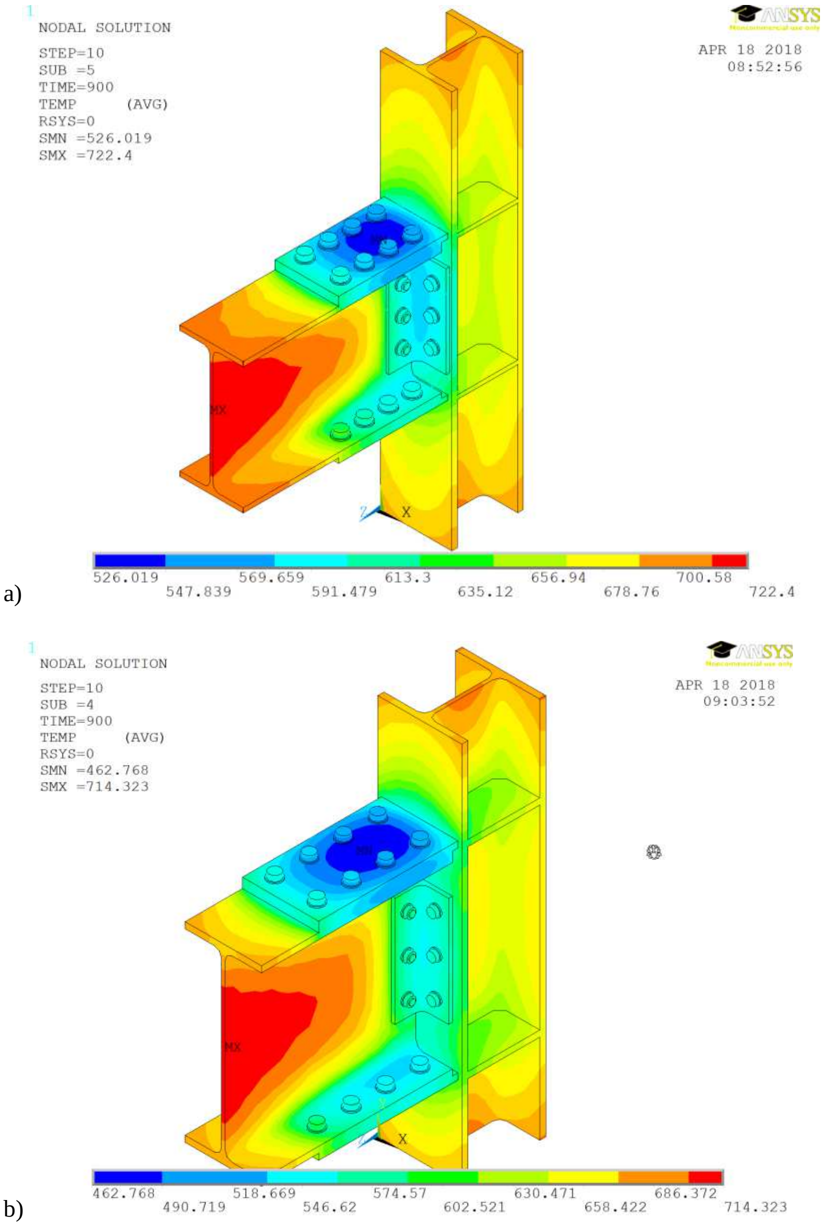


Fig. 2. Temperature distributions obtained for the evenly heated pure steel joint considered in the example after 15 minutes of its exposure to a fully developed standard fire, including: a) the case when the beam was made of a IPE 330 steel section, b) the case when the beam was made of the IPE 500 steel section

Details of the temperature values identified having finished the heating process in the two selected cross-sections of the considered beam, i.e. in the first section denoted by the symbol A-A which is situated in the joint itself, in place where the effective thickness of the heated steel sheet is simply the sum of the thickness of the beam web and of the thickness of two flanges belonging to the angles composed the web cleat, and in the second section denoted by the symbol B-B and localised outside the joint, are gathered in Fig. 3.

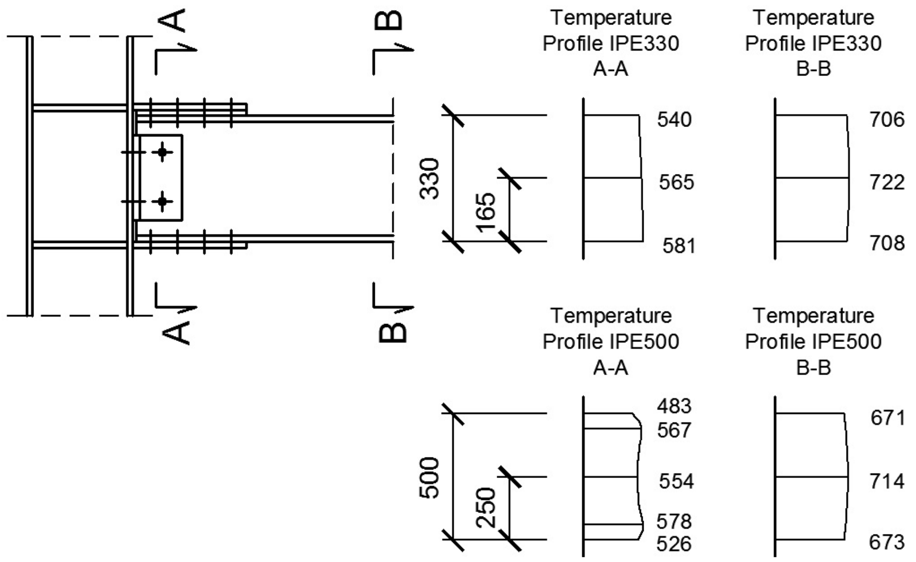


Fig. 3. Details of the steel temperature values as well as of the steel temperature distributions identified after 15 minutes of a direct fire exposure in the selected cross-sections of a beam being a part of the considered pure steel joint

Lack of symmetry in both temperature distributions presented in Fig. 3 and related to the section A-A results from the use of a horizontal steel spacer in the upper part of the joint that on the one hand facilitates the joint assembly but on the other hand increases the effective thickness of the heated beam flange. It is essential that the smaller I-section heats up much faster than the bigger one despite the fact that the latter section has more slender web. It is a result of the fact that the value of the so-called fire exposure coefficient, which is commonly named the section factor, for a smaller I-beam section ($(U/A) = 201\ \mathrm{m}^{-1}$) is significantly higher than in the case of the bigger I-beam ($(U/A) = 151\ \mathrm{m}^{-1}$). It is also worth noting that the temperature distribution in the web of beams which are respectively high is usually nonlinear while the analogous distribution observed in the beams which are not so high remains in the same conditions almost exactly linear. Let us also note that in the B-B cross-section situated outside the joint the most hot area turned out to be the web of the beam in the place furthest from the flanges. In the section A-A the same zone was, however, much less heated due to the angles forming the web cleats and adjoining to the beam web. Therefore, the most hot in this section turned out to be some insulated web zones, those to which no additional elements are added.

3. The joint with a reinforced concrete floor slab lying directly on the upper beam flange

The second part of the analysis is devoted to recognition of the shape of the steel temperature distribution identified after 15 minutes of even heating under fully developed fire conditions in the steel beam-to-column joint with the floor slab lying directly on the upper beam flange. The thickness of such a floor slab was assumed in the considered example to be equal to 12 cm. The numerical model of the joint of this type with a geometry and dimensions fully analogous to the previous one described in the first part of this paper, which is prepared also in the ANSYS environment, is presented in detail in the Fig. 4. The upper horizontal rib stiffening the column web is invisible in this figure because it is fully covered with concrete.

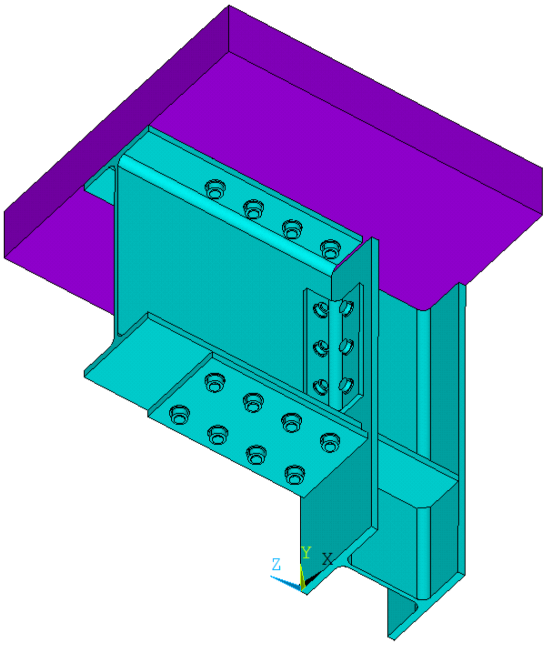


Fig. 4. Scheme of a numerically modelled steel beam-to-column joint with a reinforced concrete floor slab lying directly on the upper flange of the beam

As one can see, the beam is now heated under fire conditions only on three sides. The map of the temperature distribution identified in this joint after 15 minutes of its exposure to a fully developed fire is shown in detail in Fig. 5 for the case when the beam was made of the IPE 330 steel section.

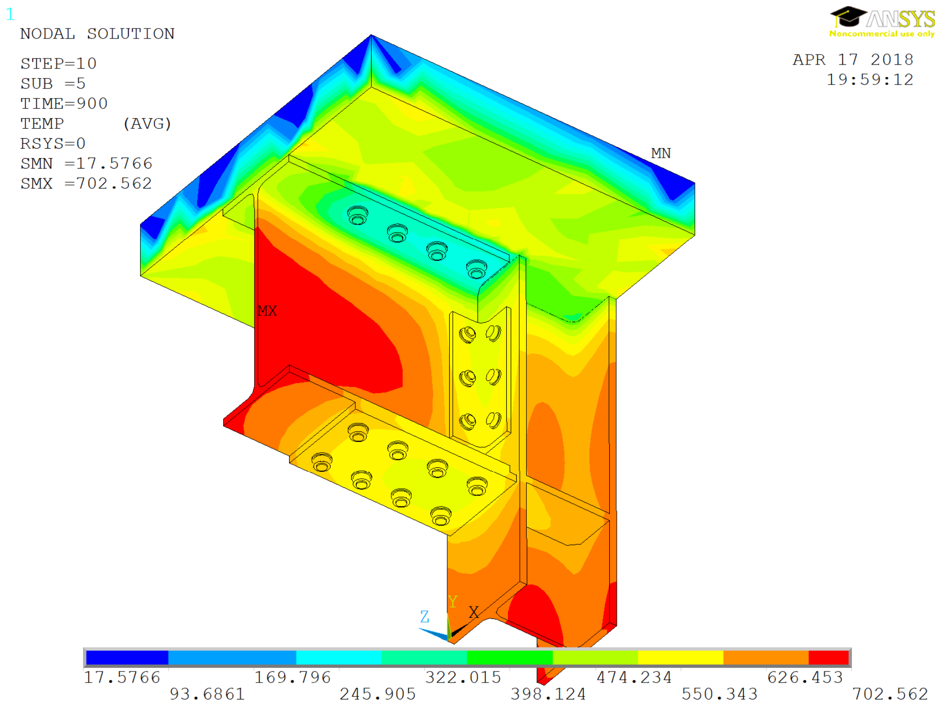


Fig. 5. Map of a steel temperature distribution obtained for a model of the joint considered in the example after 15 minutes of its simulated fire exposure (the case with the beam made of the IPE 330 steel section)

It is not a surprise now that the upper beam flange turned out to be definitely less hot than the neighbouring joint components. The heat reaching this flange is largely dissipated to a massive floor slab with a significant thermal capacity, which thus confirms good insulating properties of the concrete. This effect is well visible in the Fig. 6 in which the selected steel temperature distributions obtained for the joint considered in the example are presented in detail. Two cross-sections denoted in this figure by the symbols A-A and B-B, respectively, were chosen to conduct the suitable comparisons. As it was in the first part of the presented paper the first cross-section is now situated in the joint itself whereas the second one - outside the joint. Two pairs of the diagrams are stacked one above the other for comparative purposes. The first pair refers to a joint with a beam made of the IPE 330 steel section while the second one the analogous joint but that with a beam made of the IPE 500 steel section. In the bigger of these two beams in the section A-A the nonlinearity of the steel temperature distribution over the entire height of the beam web is already noticed which cannot be seen in the smaller beam. In this smaller beam, only a nonlinear effect of cooling of an upper part of the beam cross-section caused by the proximity of the reinforced concrete slab is visible.

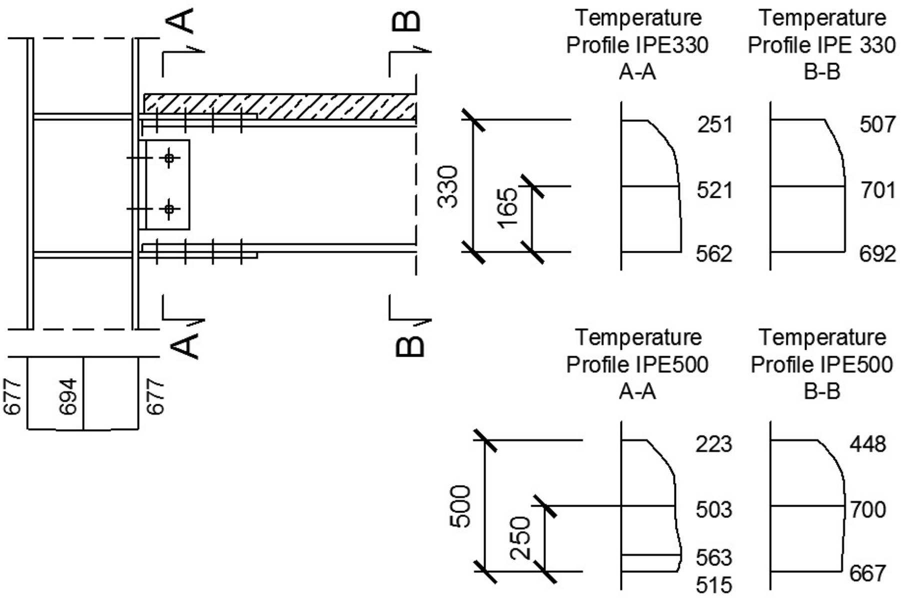
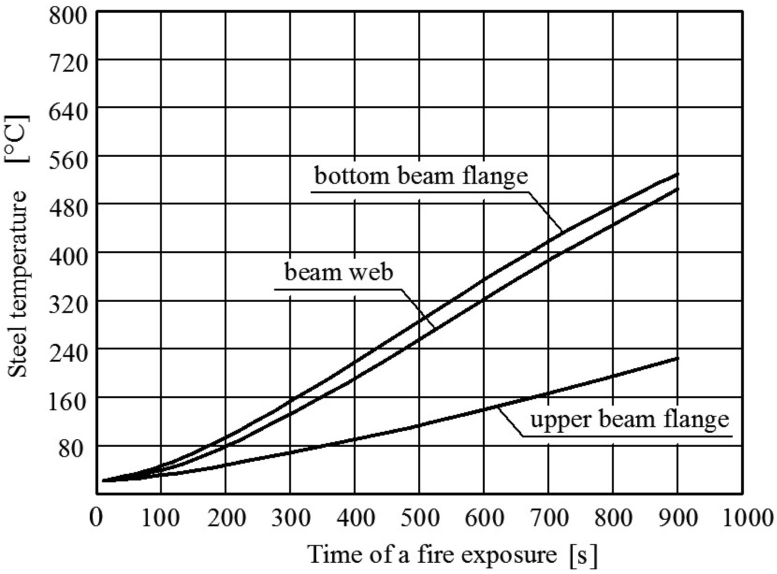


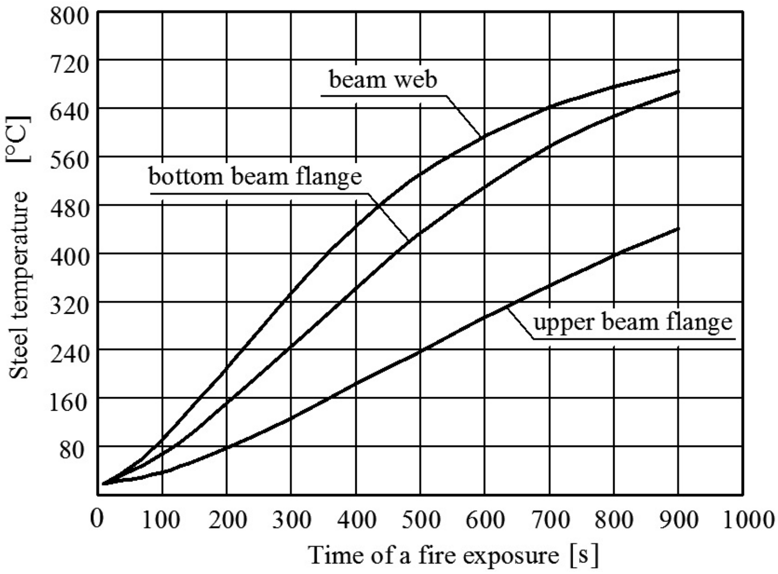
Fig. 6. Steel temperature distributions obtained in the selected beam cross-sections for the joint considered in the example after 15 minutes of a simulated fire exposure. The steel temperature distribution identified at the same time of the heating process in the cross-section related to the neighbouring column is presented on the left side of this figure

The conclusions resulting from Fig. 6 basically coincide with those formulated earlier after the presentation of Fig. 3. The smaller beam heats up faster than the bigger one and the difference between the temperature values identified in the same cross-sectional areas and at the same time points of the heating process in the sections A-A and B-B, respectively, reaches 250 degrees Celsius. In the case of the smaller of the two beams which are considered in this example, the most heated zone in the section A-A turns out to be the bottom flange, which is not true when the cross-section of the beam is higher and therefore more slender.

In addition, in Fig. 6 the temperature distribution relating to the selected cross-section of the neighbouring steel column when it is made of the HEB 180 wide-flange section is presented for comparative purposes. It is obvious that the web of such the column turned out to be more hot than the adjoining column flanges because they are significantly thicker.



a)



b)

Fig. 7. Development of a steel temperature rise observed in individual beam components being a part of the joint with concrete floor slab considered in the example, during the exposure of such a joint to a standard fully developed fire, including: a) relationships identified in the section A-A, b) relationships identified in the section B-B. The localisation of such both sections is marked in Fig. 6. It is assumed that the beam was made of the IPE 330 steel section

The diagrams presented in Figs. 7a and 7b seem to be very informative. It is shown in them how, in the case of a thermally unprotected steel beam-to-column joint considered in the example with the reinforced concrete floor slab lying on the upper beam flange and with the beam made of the IPE 330 steel section, together with the fire development increased the difference between the temperature values relevant for the beam web and for the upper and bottom beam flanges, appropriately identified in the section A-A (Fig. 7a) and in the section B-B (Fig. 7b).

4. Verification of the correctness of the joint temperature distribution recommended for use in the standard EN 1993-1-2

In conventional fire safety assessment related to the steel frame load-bearing structures usually a simplifying but very conservative assumption is accepted for calculation that the steel temperature in all components of the considered joint at any given time of a fire can be treated as fully aligned and equal to the maximum temperature of such the steel measured outside this joint at the same time of this fire. For more precise estimation, however, in Annex D3 of the standard EN 1993-1-2 [1], in the case when the steel beam-to-column joint is covered by an adjacent reinforced concrete floor slab, it is recommended to use in this field the equivalent steel temperature distributions, differentiated depending on whether the height of the joint beam is greater or smaller than 400 mm. Considering that in the case of the first of beams analysed in this article, the one made of the IPE 330 steel section for which $h < 400$ mm, the steel temperature measured in the bottom beam flange in the section B-B, i.e. outside the joint, after 15 minutes of the standard fire exposure, was identified as being equal to 692 degrees Celsius (see Fig. 6) the equivalent temperature distribution corresponding to this situation and recommended in the standard [1] has the form shown in detail in the middle of the Fig. 8. Similarly, in the case of the second beam, the one made of the IPE 500 steel section for which $h > 400$ mm, the steel temperature measured after the same fire duration in the bottom beam flange was equal to 667 degrees Celsius (see Fig. 6), which gave an equivalent standard steel temperature distribution shown on the right side of the Fig. 8. Let us note that neither the temperature 692 degrees Celsius nor the temperature 667 degrees Celsius, both measured in section B-B in the bottom beam flange, were not the maximum temperature values identified in this section. Much hotter than the bottom beam flange in both cases turned out to be the web of the beam.

Comparison of the steel temperature distributions, those taken from the Fig. 8 with the corresponding distributions specified earlier for the A-A section of the considered steel beam-to-column joints covered by a reinforced concrete floor slab and obtained for the same beams and for the same fire exposure conditions but after the use of the more precise numerical analysis (they are presented in Fig. 6), allows to conclude that the recommendations proposed by the standard [1] are calibrated safely but they are still very conservative.

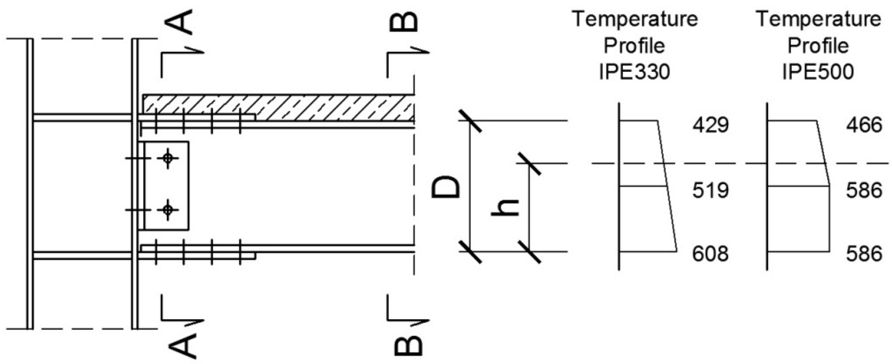


Fig. 8. Equivalent steel temperature distributions recommended in the standard EN 1993-1-2 [1] for beam-to-column joints with adjacent reinforced concrete floor slab considered in the example

5. Concluding remarks

The primary goal of the authors was to show in both parts of the presented paper that in a fire situation the steel temperature distribution in a steel beam – to-column joint is not homogeneous, regardless of whether it is a pure steel joint or one that is adjacent to a reinforced concrete floor slab. The fact of the heterogeneity of this type has a significant impact on the real course of the redistribution of internal forces in a fire, and thus on the guaranteed safety level [4, 5]. The uneven steel temperature distribution in the joint components determines both the effective load capacity of such a joint and its stiffness identified for the predicted scenario of a fire development. The situation is complicated by the fact that both of these parameters change with the development of a fire and they are therefore difficult to quantify.

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Przesłano do redakcji: 01.05.2018 r.
Przyjęto do druku: 15.06.2018 r.