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Simple equations for the calculation of the temperature within the cross-section of slim floor beams under ISO Fire

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Abstract. The calculation of fire resistance for a composite structural element comprises the calculation of the temperature within its cross-section and of the load bearing capacity, considering the evolution of the steel and concrete mechanical properties, function of the temperature. The paper proposes a method to calculate the bending capacity under ISO fire, for Slim Floor systems using asymmetric steel beams, with a wider lower flange or a narrow upper flange welded onto a half hot-rolled profile. The temperatures in the cross-section are evaluated by means of empirical formulas determined through a parametrical analysis, performed with the special purpose non-linear finite element program SAFIR. Considering these formulas, the bending capacity may be calculated, using an analytical approach to determine the plastic bending moment, for different fire resistance demands. The results obtained with this simplified method are validated through numerical analysis.

Keywords: slim floor; fire design; ISO fire; simplified method.

1. Introduction

Slim Floor slabs are generally made of asymmetric steel beams with a wide lower flange to support prefabricated concrete elements, the gap between the steel beam and prefabricated elements being filled with concrete. The width of the bottom flange must guarantee a minimum support on both sides in accordance with the specific requirements of the slab manufacturer.

One obvious benefit of this solution is that it improves the fire resistance of the composite slab, owing to the fact that the steel beam, except for the lower flange, is insulated by the concrete.

There are several asymmetric beam models which may be considered for a Slim Floor. For example, as shown in Fig. 1, ArcelorMittal (2008) developed three types of steel beams: IFB (Integrated Floor Beams), in which a wider lower flange or a narrow upper flange is welded onto a half hot-rolled profile, and SFBs (Slim Floor Beams), in which a wider plate is welded under the lower flange of a hot-rolled profile.

Some of the first series of fire tests on isolated Slim Floor systems, made at ETH Zurich (Fontana and Borgogno 1996), showed that a fire resistance of 90 minutes may be achieved for this type of slabs, without considering any fire protection. A recent experimental study on isolated Slim Floor slabs using

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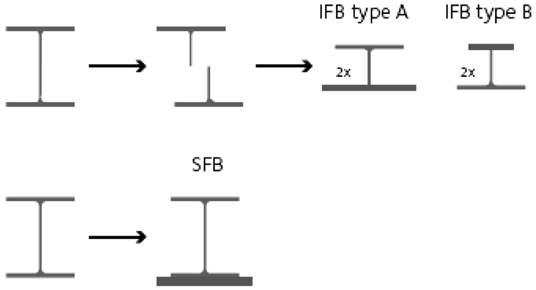


Fig. 1 Slim Floor beam models (ArcelorMittal 2008)

asymmetric beams, in which the protection of the lower flange exposed to fire and the use of supplementary reinforcement were considered, concluded that the lower flange reinforcement is the most effective way to improve the fire resistance (Kim *et al*. 2011).

Results of fire tests made during 1995-1996 (STC 1999), or more recently (Wald *et al*. 2005), (daSilva *et al*. 2005) on a full-scale steel concrete composite building constructed at the Building Research Establishment Laboratory Cardington, U. K., showed that the fire performance of composite steel framed buildings with composite floors is much better than indicated by standard fire resistance tests on composite slabs or composite beams as isolated structural elements. One reason for the excellent fire behaviour of the composite building in Cardington full-scale tests is the tensile membrane action which develops in the composite steel-concrete slabs (Wang 1996). In the above mentioned large scale tests, 'classic' solutions were used for the composite slabs, in which the concrete slab was placed above the steel beams.

A large scale fire test on a compartment of a composite building using asymmetric Slim Floor slabs was carried out at the same laboratory in Cardington, providing for the first time a useful insight into the behaviour of this slab system in its entirety (Bailey 2003). The experimental investigation showed that the beam-to-column connections, not assumed to transfer moment in normal design, were beneficial to the survival of the beams and of the system as a whole.

In order to investigate the behaviour of Slim Floor slabs under fire at structural level, two full-scale composite steel frames with Slim Floor slab construction were conducted more recently by Dong (2009). In one test the beam-to-column connections were protected, while in the second test both connections and steel columns were protected. The experimental study concluded that the fire resistance of frames constructed with Slim Floor slabs is at least as good as that of frames with conventional slab construction.

The composite action between the casted concrete and the steel beams is usually neglected in the design of Slim Floor beams. The beams may be calculated as pure steel elements. However, due to the presence of the concrete, the temperatures in the steel beams are not uniform, and a proper temperature distribution must be considered when calculating the fire resistance. The temperature distribution may be determined by a numerical analysis, using an appropriate program. Of course, this must be done for each particular situation, considering the dimensions of the steel beam inside the concrete and of the bottom flange exposed to fire on three sides. Anyway, for composite elements it is often necessary to use advanced calculation models in order to determine, at least, the temperature distribution inside the cross-section.

Newman (1995) used the 2D finite difference heat transfer programme TFIRE to perform the thermal analysis, for six unprotected isolated Slim Floor beams with precast concrete floor tested in fire at the

Steel Construction Institute (SCI). Ma and Makelainen (2006) used ABAQUS to model the fire behaviour of Slim Floor frames, considering a combination of shell, beam and rebar elements. The numerical study highlighted that the rotational and the axial restraints on the heated beam, in the plane of the frame, have significant influence on the entire frame behaviour in fire. Ellobody (2011) developed a complex 3D finite element model in ABAQUS, in order to investigate the behaviour of unprotected composite Slim Floor slabs exposed to different fires, considering also the interface between the steel beam and the composite concrete floor. The study showed that EN 1994-1-2 (2005) predictions for composite beams at elevated temperatures may be used, being generally conservative. The fire resistance of Korean asymmetric Slim Floor slabs depending on the load ratio was numerically investigated using ANSYS (Park *et al.* 2011), which showed that fire resistances of 60 minutes may be achieved with the considered systems for load ratios under 0.47 without additional fire protection. The authors also set a limit temperature on the bottom flange directly exposed to fire, according to the load ratio, that indicates the fire resistance.

The temperature distribution within the cross-section of a particular Slim Floor slab, using a hat-shaped steel profile, was investigated numerically using ABAQUS software (Schaumann and Kirsch 2011). This hat-shaped steel profile includes a cavity which must be properly considered in the numerical analysis, because neglecting the effect of the cavity radiation may lead to unsafe results. Using the results of the numerical analysis, the authors developed simple equations for the temperatures of the webs and flanges of the steel hat-shaped profile, which may be used to determine the bending capacity of the beam in fire conditions, for different fire resistance demands.

Such a simplified approach, given in the "Model Code for Fire Engineering" (Schleich *et al.* 2001), is available also for asymmetric Slim Floor beams of SFB type, in order to determine the temperature distribution within the steel profile. The method presented in this document gives the temperatures of the bottom wider plate, of the lower flange and on the height of the web of the steel profile for fire resistance demands of 60 and 90 minutes. No indication is given for the temperature in the concrete, in the area of possible supplementary reinforcement in the lower flange region.

In order to offer to the designer a tool to evaluate the temperatures in a Slim Floor system of IFB type (in which a wider lower flange or a narrow upper flange is welded onto a half hot-rolled profile), exposed to ISO fire, without the need of a complex numerical simulation, a parametric study was done by the authors, based on numerical simulations using SAFIR program (Franssen 2005), developed at the University of Liege for the analysis of structures under ambient and elevated temperature conditions. The aim was to propose simple equations for the calculation of temperature in various points in the cross-section, similar as those given in the "Model Code for Fire Engineering" (Schleich *et al.* 2001) for SFB Slim Floor beams, or for hat-shaped Slim Floor beams, determined by Schaumann and Kirsch (2011). The formulas available for SFB beams cannot be used for IFB beams, due to the different massivity of the components of the steel beam, especially due to the lower flange, which is exposed to fire on three sides in case of IFB beams. For SFB beams, the lower flange of the asymmetric profile has a part which is exposed on fire on three sides and a narrow part included in concrete. The study presented in the present paper also extends the availability of such analytical method for 120 minutes of fire resistance demand and offers formulas to calculate the temperature in the area of possible supplementary reinforcement, above the lower flange of the steel profile.

For the parametric study, the steel profiles of ArcelorMittal Company for IFB Slim Floor systems (ArcelorMittal 2008) were considered, for bottom plate thicknesses of 12-35 mm. The temperature on each cross-section was determined with SAFIR and some formulas have been developed, function of different parameters. Using these formulas to calculate the temperature in the bottom plate, top plate, web of the profile and inside the concrete (when supplementary reinforcement is considered) the bending capacity of the cross sections was calculated analytically, and validated through a mechanical analysis under elevated temperatures.

2. Verification of the numerical model

A fire test made at ETH Zurich in 1994, on a Slim Floor slab using a steel beam similar to the system analysed in the parametric study (IFB type A using prefabricated concrete elements), as shown in Fig. 2 (ProfilArbed 1995), was considered to verify the numerical model. The floor in the test comprises 3 steel beams of 2.71 m length, with a distance between beams of 2.4 m. The asymmetric beam was realized by an half of IPE400 and a welded plate 400 × 12, as a wider bottom flange, supporting precast concrete units. Concrete was poured in site, up to 4 cm above the top flange of profile. An upper reinforcement mesh was considered and two rebars were placed above the bottom flange of the steel beam. The floor was subjected to 120 minutes of standard ISO fire.

Because the prefabricated concrete elements do not participate to the flexural capacity of the cross-section, they were considered with the corresponding material properties only in the thermal analysis, to determine the temperature distribution. Fig. 3 shows the mesh of half of the symmetric cross-section.

As shown in Fig. 4, the temperatures calculated in the lower rebars are slightly higher than the corresponding temperatures recorded during the test. After 120 minutes of ISO fire, at which the test was stopped, the temperature in the rebars does not exceed 400°C, which, according to EN 1993-1-2 (2005), is the temperature from which it is considered that the yield strength of steel decreases. Therefore, the rebars maintain the full yield strength for the duration of the test.

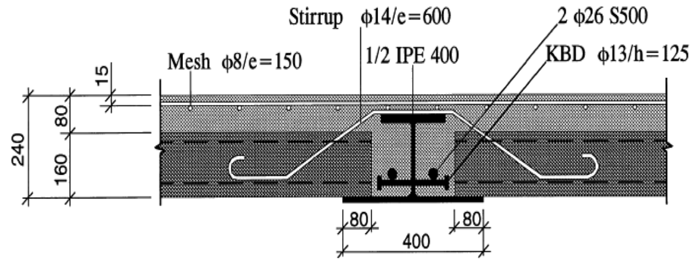


Fig. 2 Cross-section of the tested floor (ProfilArbed, 1995)

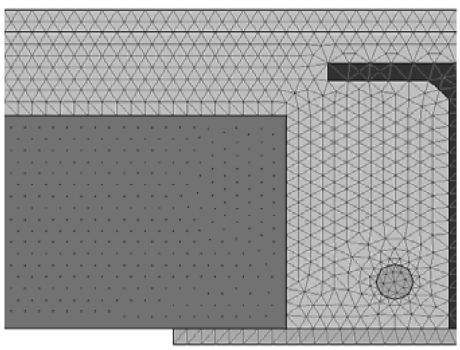


Fig. 3 Numerical model

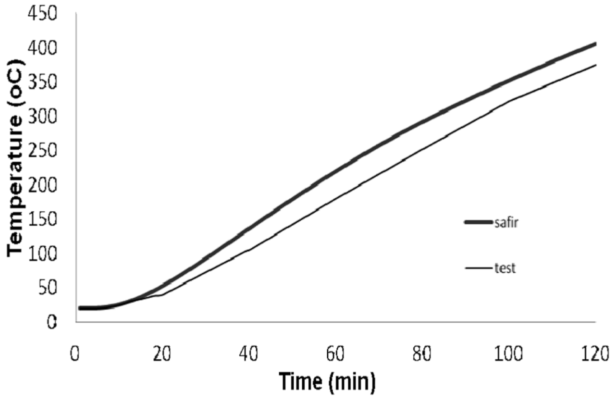


Fig. 4 Temperature in rebars

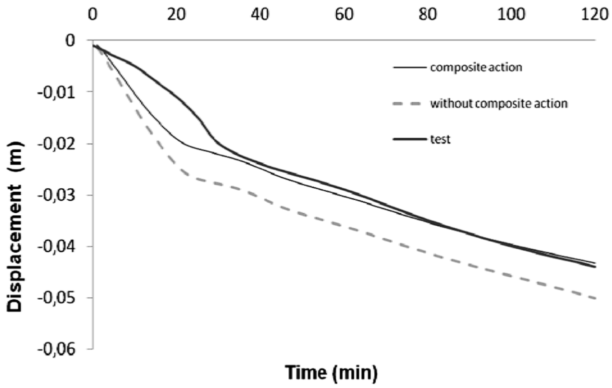


Fig. 5 Displacement evolution at mid-span

In the mechanical analysis under elevated temperatures, two situations were analysed. First, all the elements of the cross-section participate to the bending capacity (excepting for the prefabricated concrete elements), considering an effective width of 25% of the span. Second, no composite action is considered and only the steel beam with the extended bottom flange, together with the lower reinforcement, participates to the bending capacity of the slab. The results are presented in Fig. 5.

If the composite action is considered, Fig. 5 demonstrates a good agreement between the time-displacement curves resulted from the numerical simulation and from the test. Neglecting the composite action, the numerical model follows a similar path, but with higher displacements.

In order to propose an analytical model to calculate the bending capacity of IFB Slim Floor beams with precast concrete units, no composite action was considered by the authors. Therefore, for the distribution of the temperature on the cross-section, simple equations were determined only for the temperatures in the asymmetric steel beam and in the lower reinforcement.

3. Parametric study-Temperature distribution

For the thermal analysis, the emissivity used was 0.7 for steel and 0.8 for concrete, on heated surfaces

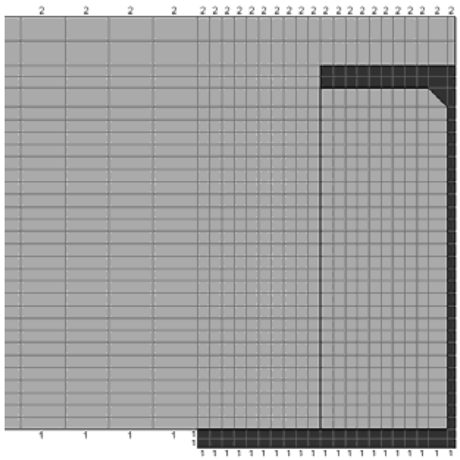


Fig. 6 Numerical model of the cross-section

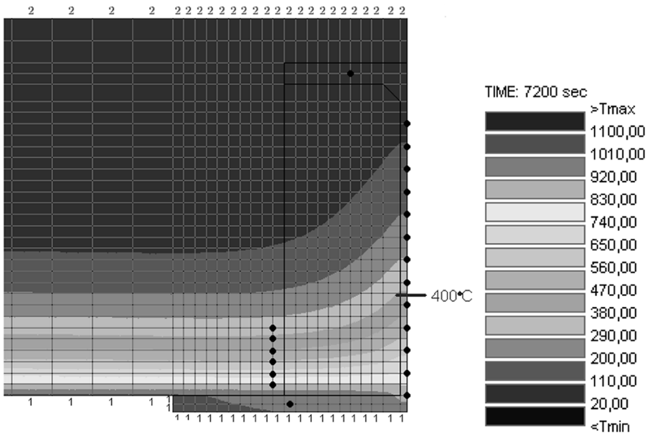


Fig. 7 Points monitored on the cross-section

as well as on unheated surfaces, whereas the coefficient of convection was 25 W/m^{2}K on heated surfaces and 4 W/m^{2}K on unheated surfaces for both materials. The upper limit of the thermal conductivity was considered for concrete.

The numerical model is shown in Fig. 6. The cross-section of the beam is exposed to ISO fire only from bellow, the temperature in the air on the top of the floor being considered 20^{o}C.

The thermal analysis was done for 30, 60, 90 and 120 minutes of fire exposure, for each cross-section considered. Temperatures from relevant points of the cross-section were extracted from the numerical analysis, and the distance from the top of the bottom flange from which the temperatures are below 400^{o}C was also monitored. For all cases, the 400^{o}C isotherm was found to be on the height of the web, even after 120 minutes of fire exposure. Therefore, the top plate retains its full strength, and the parametric study further concentrated on the temperature distribution in the bottom flange, web and concrete, in the area of possible supplementary lower reinforcement (in the hypothesis that the temperature in the reinforcement is equal to the temperature in the concrete, in the same location). Fig. 7 shows the points in which the temperature was monitored and the temperature distribution on the cross-section for a given case, after two hours of standard ISO fire exposure, by highlighting the 400ºC limit in the web of the asymmetric beam.

3.1 Temperature in the bottom flange

In a first approach, the temperature in the bottom plate was calculated using the simple method presented in EN1993-1-2, table 4.2 (EN 1993-1-2, 2005), considering the section factor $A_m/V = (b + 2t)/(bt)$, for the flange exposed on three sides. As it will be shown in section 4, this leads to very conservative values of the bending capacity of the floor calculated analytically, when compared to the results of numerical simulations. For Slim Floor slabs, the temperature in the lower flange, which is directly exposed to fire, has the strongest influence on the fire resistance. Therefore, another method for the calculation of the temperature in the bottom flange was considered.

The temperature was recorded after 30, 60, 90 and 120 minutes of ISO fire, in the point shown in Fig. 7, situated at a quarter of the width of the bottom flange. It was found that this temperature depends essentially on the thickness of the bottom plate. Therefore, in order to derive simple formulas for the temperature evolution, the temperatures were represented as shown in Fig. 8, function of the thickness of the plate, for the different fire resistance demands. First and second order functions were found to fit better with the scatter, as Fig. 8 shows, and were used to represent the temperature in the bottom flange.

The following equation is proposed to determine the temperature in the bottom flange, in which T_i is in ºC and the plate thickness t_{pl} is in mm

$$T_i = A_i t_{pl}^2 + B_i t_{pl} + C_i \tag{1}$$

in which the coefficients A_i, B_i and C_i are given in Table 1.

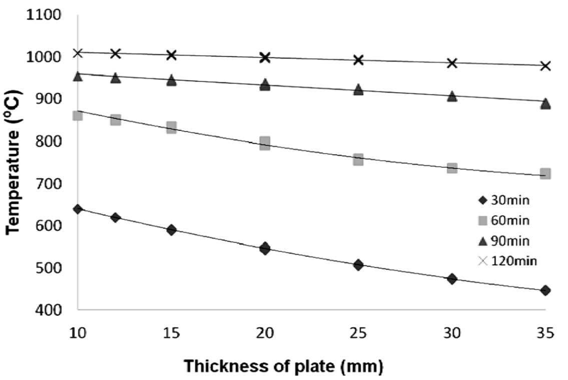


Fig. 8 Temperature in bottom flange function of plate thickness

Table 1. Coefficients for temperature calculation in the bottom flange

Time (min)	A_i	B_i	C_i
30	0.113	-12.80	760
60	0.130	-11.80	980
90	-	-2.60	990
120	-	-1.25	1025

3.2 Temperature in the web of the steel profile

The temperature on the height of the web is hardly influenced by its thickness, but is strongly influenced by the distance from the bottom flange and, in a smaller amount, by the thickness of the bottom flange t_{pl}.

The temperature was recorded after 30, 60, 90 and 120 minutes of ISO fire, in the points from the web shown in Fig. 7. The temperatures were represented as shown in Figs. 9-14, function of the distance from the top of the bottom plate, for the different fire resistance demands and for a given thickness of the bottom plate. Exponential functions were found to fit better with the scatter, as Figs. 9-14 show, and

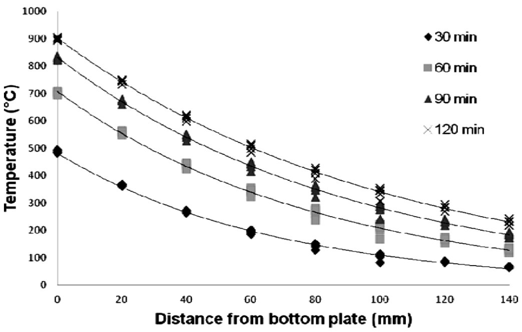


Fig. 9 Web temperature for 12 mm bottom plate

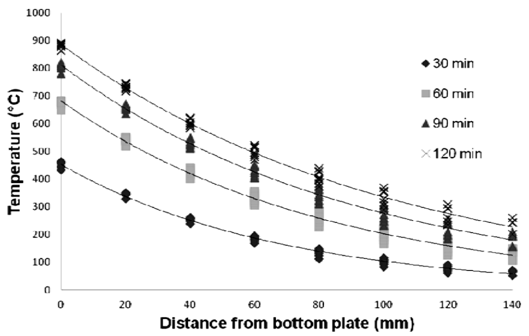


Fig. 10 Web temperature for 15 mm bottom plate

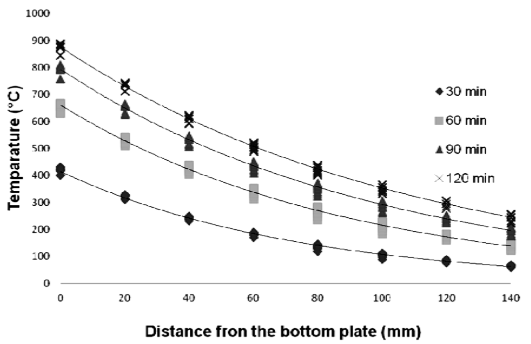


Fig. 11 Web temperature for 20 mm bottom plate

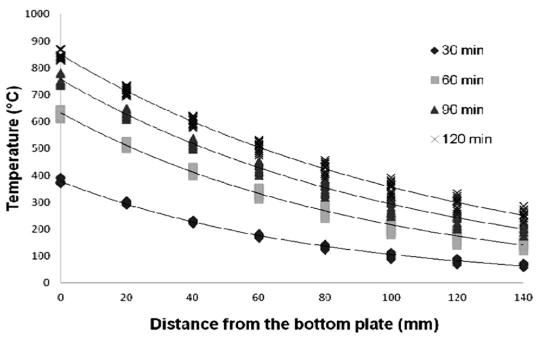


Fig. 12 Web temperature for 25 mm bottom plate

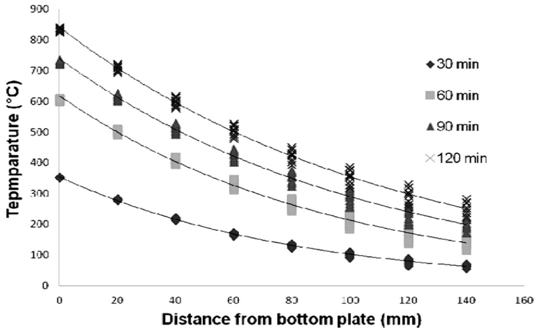


Fig. 13 Web temperature for 30 mm bottom plate

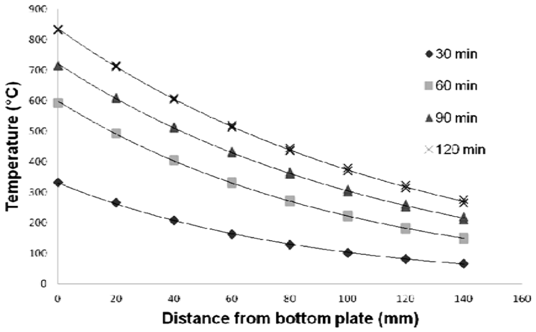


Fig. 14 Web temperature for 35 mm bottom plate

were used to represent the temperatures in the web.

The following equation is proposed to determine the temperature in the web, in which T_w is in °C, while the distance z along the height of the web measured from the top of the bottom flange and the plate thickness t_{pl} are in mm

$$T_w = k_1 e^{k_2 z} \tag{2}$$

with

$k_1 = A_w \ln(t_{pl}) + B_w$

$k_2 = C_w \ln(t_{pl}) + D_w$

in which the coefficients A_w, B_w, C_w and D_w are given in Table 2.

3.3 Temperature in the rebars above the bottom flange

The temperature in the rebars above the bottom flange was considered equal to the temperature of the concrete at the same location. As in case of the web temperature, the temperature on the height of the concrete is strongly influenced by the distance from the bottom flange and, in a lesser extent, by the thickness of the bottom flange.

The temperature was again recorded after 30, 60, 90 and 120 minutes of ISO fire, in the points within the concrete above the bottom flange shown in Fig. 7, which are located in the zone of the possible positions of the rebars. The temperatures were represented in a similar manner as for the web temperature distribution, function of the distance from the top of the bottom plate, for the different fire resistance demands, for a given thickness of the bottom plate. Exponential functions similar to the ones for web temperature distribution were found, as Figs. 15-20 show.

Table 2. Coefficients for temperature calculation in the web

Time [min]	A_w	B_w	C_w	D_w
30	-140.70	832.42	0.00317	-0.0230
60	-103.80	968.60	0.00232	-0.0182
90	-108.60	1146.70	0.00198	-0.0154
120	-70.44	1124.40	0.00158	-0.0134

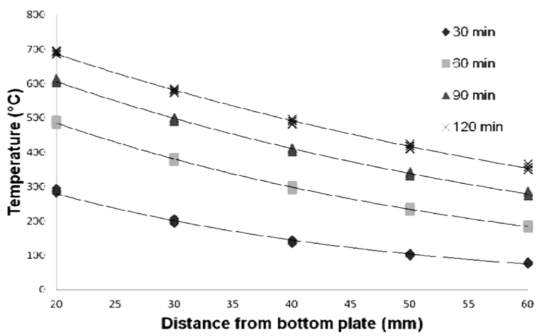


Fig. 15 Concrete temperature for 12 mm bottom plate

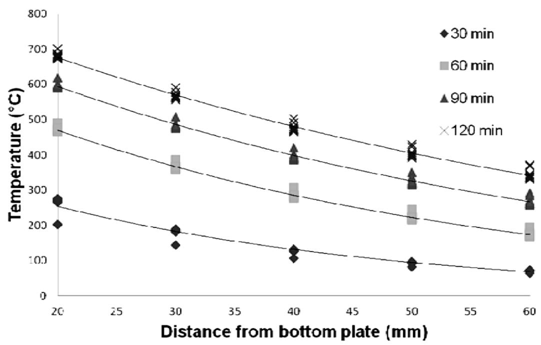


Fig. 16 Concrete temperature for 15 mm bottom plate

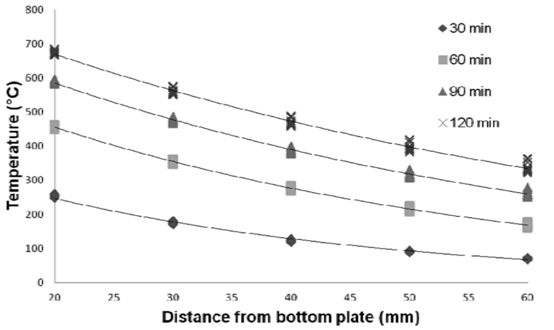


Fig. 17 Concrete temperature for 20 mm bottom plate

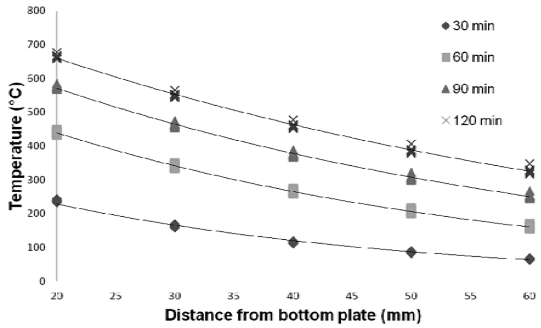


Fig. 18 Concrete temperature for 25 mm bottom plate

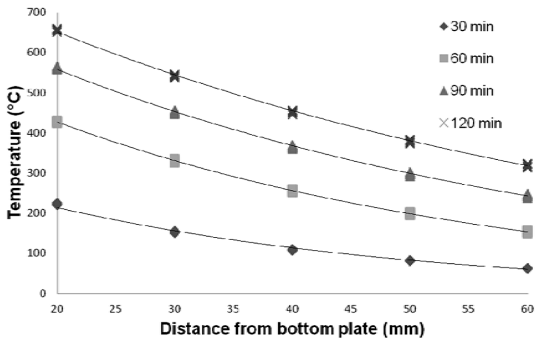


Fig. 19 Concrete temperature for 30 mm bottom plate

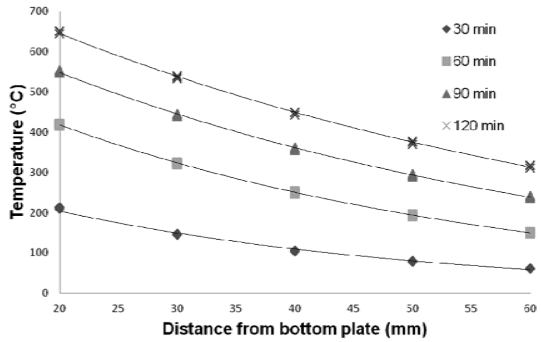


Fig. 20 Concrete temperature for 35 mm bottom plate

The following equation is proposed to determine the temperature in the concrete (rebars), in which T_c is in °C, while the distance z measured from the top of the bottom flange and the plate thickness t_{pl} are in mm

$$T_c = k_3 e^{k_4 z} \tag{3}$$

with

$$k_3 = A_c \ln(t_{pl}) + B_c$$

$$k_4 = C_c \ln(t_{pl}) + D_c$$

in which the coefficients A_c, B_c, C_c and D_c are given in Table 3.

4. Calculation of the bending capacity

For the mechanical analysis under elevated temperatures, steel was considered with the following properties: $E = 2.1\text{E}^{11}$ N/m^2, yield strength for the profile $f_y = 355$ MPa, yield strength for reinforcement $f_y = 500$ MPa. As stated above, the mechanical properties of the concrete were ignored (no composite action).

The bending capacity under elevated temperatures $M_{pl,Rd,\theta}$ was determined taking into account that a full plastic moment is developed when the section is fully yielded in bending. The following

Table 3. Coefficients for temperature calculation in concrete (rebars)

Time [min]	A_c	B_c	C_c	D_c
30	-6.90	612.67	0.00009	-0.0342
60	-4.06	834.64	-0.00005	-0.0240
90	-2.71	970.63	-0.00005	-0.0181
120	-1.37	1043.80	-0.00005	-0.0150

hypotheses were considered: the material behaviour is ideal rigid-plastic, the yield strength in tension is equal to the yield strength in compression, all the material of the cross-section is yielded and the cross-section carries no axial force. The plastic design bending moment $M_{pl,Rd,\theta}$ was calculated by a classical integration on the depth of the cross-section, considering strips of 1 mm thickness. For each strip the stress resultant $F_i = (A_i * f_{y,\theta,i})/\gamma_{a,fi}$, is calculated, where A_i is area of the 'i' strip, $f_{y,\theta,i}$ is the yield strength of 'i' strip for temperature and $\gamma_{a,fi}$ is a partial safety factor, considered with the unit value, as recommended in the fire parts of the Eurocodes.

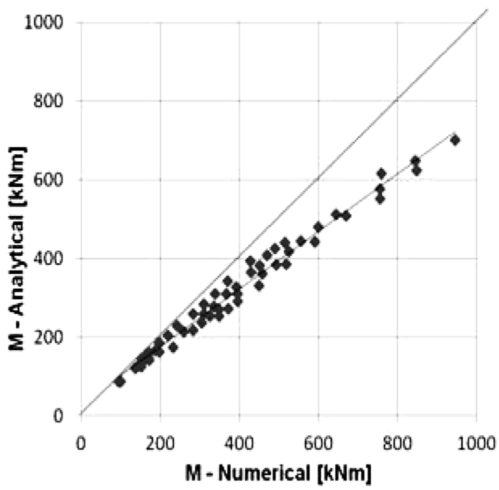
To verify the calculated values of the plastic bending moment using this analytical approach, numerical simulations using SAFIR were performed, by considering for each section a simply supported Slim Floor beam, loaded with uniform bending moment. The value of the applied bending moment was determined in order to obtain the requested fire resistance demand of 30, 60, 90 or 120 minutes. The bending capacity under elevated temperatures calculated by SAFIR for a given case, is then equal to the value of the bending moment imposed to the simply supported beam in the numerical analysis, in order to obtain a given fire resistance demand.

In a first step, the simplified method provided in EN1993-1-2 (2005), using the section factor for the bottom flange was considered to evaluate the temperature in this element, together with Eq. (2) for the temperature distribution in the web of the steel profile. For all cases, the fire resistance time in SAFIR exceeded, with an important amount, the fire resistance time predicted by the analytical method. As mentioned in section 3, this is due to the fact that the evaluation of the temperature in the bottom flange using the EN1993-1-2 method yields to values that are too conservative, compared with the temperatures determined by the parametric numerical study. For example, Fig. 21 shows that all the values of the plastic bending moment $M_{pl,Rd,\theta}$ calculated with SAFIR are considerably higher than the corresponding values calculated using the analytical method, for the fire resistance demand of 60 minutes.

In order to increase the load bearing capacity of a given cross-section under ISO fire, in the absence of fire protection, additional reinforcement is needed. Two bars ø28 were considered as supplementary lower reinforcement. The bending capacity was again determined analytically for the fire resistance demands of 60, 90 and 120 minutes, considering for the temperature in the rebars Eq. (3), for the web Eq. (2) and for the bottom plate the method based on the section factor.

Fig. 22 shows, for example for the fire resistance demand of 60 minutes, that all the values of the plastic bending moments $M_{pl,Rd,\theta}$ determined with SAFIR are considerably higher than the corresponding values calculated using the analytical method, when supplementary reinforcement is considered. Using two rebars ø28 as lower reinforcement, the bending capacity in fire increased with 25-55% in comparison with the capacity of the same sections without rebars.

In a second step, in the evaluation of the bending capacity of the Slim Floor beams using the analytical method, the simplified method provided in EN1993-1-2, using the section factor for the bottom flange was dropped and equation (1) was considered to evaluate the temperature in this element. For the temperature distribution in the web of the steel profile, and for the temperature in the rebars, the



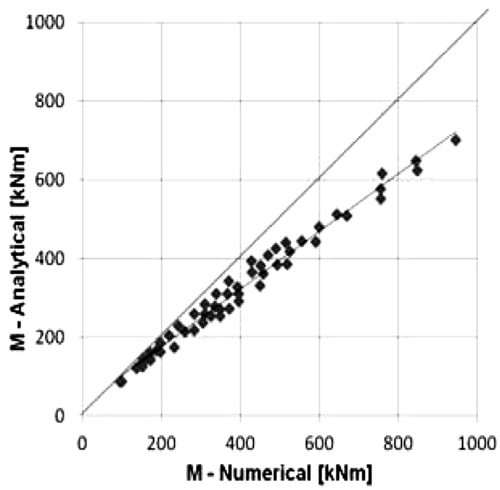


Fig. 21 $M_{pl,Rd,\theta}$ at t = 60 minutes, without reinforcement Fig. 22 $M_{pl,Rd,\theta}$ at t = 60 minutes, with reinforcement

same equations were considered, i.e. (2) and (3).

A better fit of the results given by the analytical method and by SAFIR were obtained, in absence of the rebars, as Figs. 23-26 show. The results obtained with SAFIR showed that the beams can retain without reinforcement, at least, the following load ratios (defined as the ratio between $MM_{pl,Rd,\theta}$ determined with SAFIR and the design resistance of the beam for time t = 0, $M_{fi,t,Rd}$, determined according to EN 1993-1-2): 0.5 at 30 minutes of ISO fire; 0.17 at 60 minutes of ISO fire; 0.12 at 90 minutes of ISO fire and 0.11 at 120 minutes of ISO fire. For 60, 90 and 120 minutes of fire resistance, the following maximum load ratios are possible, respectively: 0.99, 0.43, 0.29 and 0.26. These results are illustrated in Fig. 27.

A better fit of the results given by the analytical method and by SAFIR were also obtained for the sections with lower reinforcement considering Eq. (1), as Figs. 28-30 show. The results show that the beams can retain with this supplementary reinforcement, at least, the following load ratios: 0.34 at 60

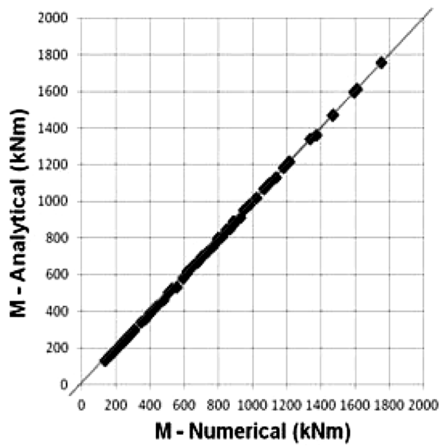


Fig. 23 $M_{pl,Rd,\theta}$ at t = 30 minutes, without reinforcement

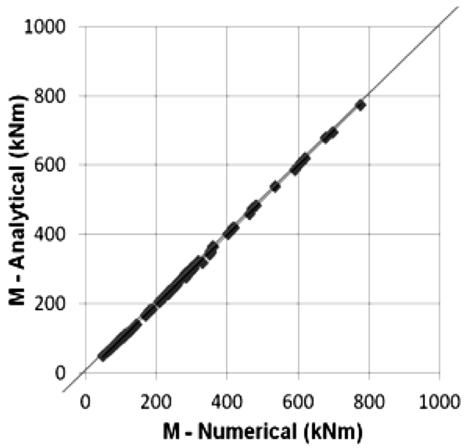


Fig. 24 $M_{pl,Rd,\theta}$ at t = 60 minutes, without reinforcement

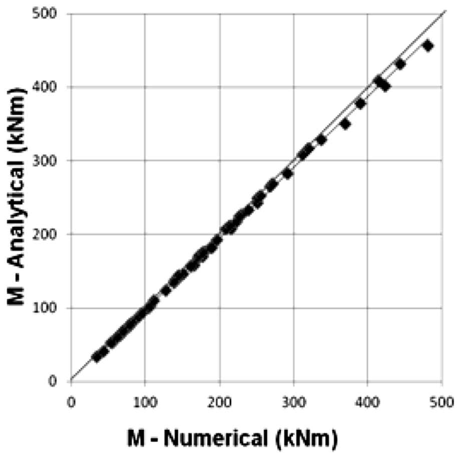


Fig. 25 $M_{pl,Rd,\theta}$ at t = 90 minutes, without reinforcement

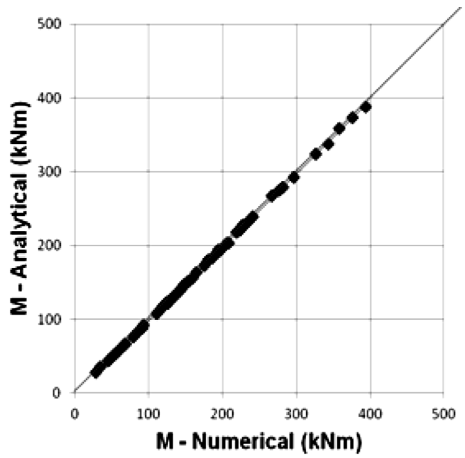


Fig. 26 $M_{pl,Rd,\theta}$ at t = 120 minutes, without reinforcement

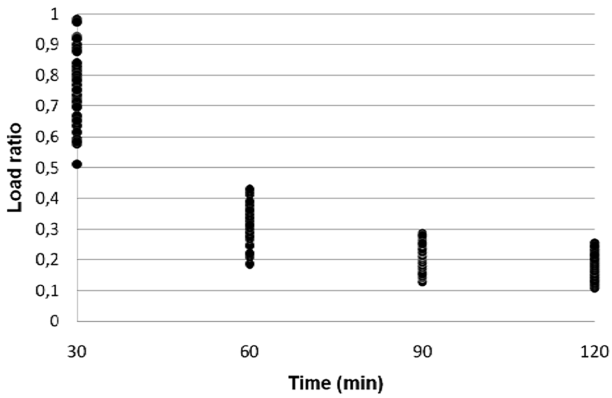


Fig. 27 Load ratios, without reinforcement

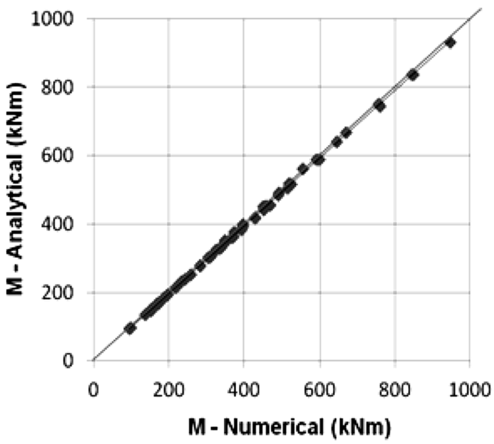


Fig. 28 $M_{pl,Rd,\theta}$ at t = 60 minutes, with reinforcement

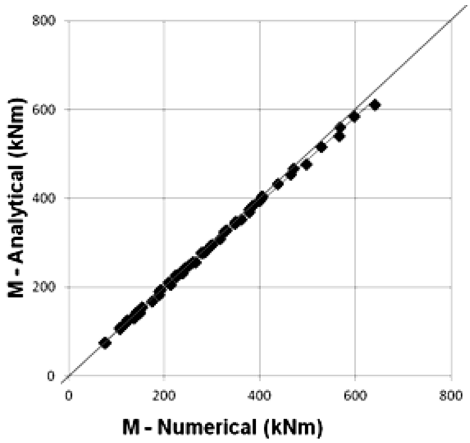


Fig. 29 $M_{pl,Rd,\theta}$ at t = 90 minutes, with reinforcement

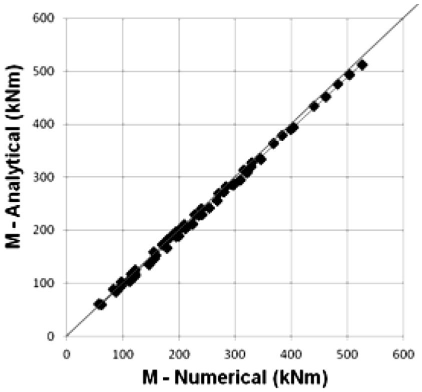


Fig. 30 $M_{pl,Rd,\theta}$ at t = 120 minutes, with reinforcement

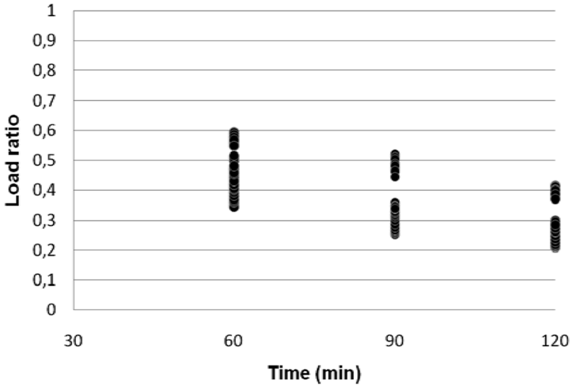


Fig. 31 Load ratios, with reinforcement

minutes under ISO fire; 0.25 at 90 minutes under ISO fire and 0.23 at 120 minutes under ISO fire. For 60, 90 and 120 minutes of fire resistance, the following maximum load ratios are possible, respectively: 0.60, 0.55 and 0.51. These results are illustrated in Fig. 31.

5. Conclusions

The bending capacity under ISO fire of the composite Slim Floor slabs, with asymmetric beams in which a wider lower flange or a narrow upper flange is welded onto a half hot-rolled profile, was investigated numerically, considering the conservative hypothesis that no composite action exists.

The investigated Slim Floor beams present 30 minutes of fire resistance with load ratios of 0.5 or more, without any additional reinforcement, while for a fire resistance demand of 60 minutes, a maximum load ratio of 0.6 is possible. If no fire protection is considered, higher bending capacities under ISO fire may be obtained by using additional reinforcement above the bottom flange.

In a parametric study, the temperatures on the cross-sections were determined numerically and simple equations have been developed for the temperature distribution. Using these equations, the bending capacity of the beams with or without reinforcement above the bottom flange may be calculated, by means of a classical plastic approach. The simplified analytical approach gives good results in comparison with numerical analyses at elevated temperatures.

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