Numerical analysis of partially fire protected composite slabs

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Abstract: The paper presents a numerical investigation, done with the computer program SAFIR, in order to obtain simpler finite element models for representing the behaviour of the partially protected composite steel concrete slabs in fire situations, considering the membrane action. Appropriate understanding and modelling of the particular behaviour of composite slabs allows a safe approach, but also substantial savings on the thermal insulation that has to be applied on the underlying steel structure. The influence of some critical parameters on the behaviour and fire resistance of composite slabs such as the amount of reinforcing steel, the thickness of the slab and the edge conditions is also highlighted. The results of the numerical analyses are compared with the results of three full scale fire tests on composite slabs that have been performed in recent years.

Keywords: composite slabs, partially protected, fire design, membrane action

1. Introduction

Observations of actual building fires and large-scale fire tests conducted in a number of countries have shown 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. The paper focuses on concrete slabs connected to steel beams by means of headed studs, used in all the considered tests.

It is clear that there are large reserves of fire resistance in modern steel-framed buildings and that standard fire resistance tests on single unrestrained members do not provide a satisfactory indicator of the real performance of such structures.

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Results of fire tests made during 1995-1996 on full-scale steel concrete composite building constructed at the Building Research Establishment laboratory Cardington, U.K., indicated that the stability of composite steel-concrete framed buildings, where some of the steel beams are unprotected, can be maintained even when the temperature of the unprotected beams exceeds 1000ºC (STC, 1999), (Bailey et al., 1999). Wang (1996) used tensile membrane action which develops in the composite steel-concrete slabs to explain the excellent fire behaviour of the composite building in Cardington full-scale tests.

Another full-scale test on a fire compartment of the same composite building was performed in Cardington in 2003, within a research project aimed to study the tensile membrane action and the robustness of structural steel joints under natural fire. An assessment of the temperature development within the fire compartment and structural elements, and of the behaviour of the structure exposed to fire is presented by Wald et al. (2005, 2006) and Simões da Silva et al. (2005). The predicted local collapse of the structure was not reached during the test, in which the columns, the external joints and the edge beam of the composite slab were fire protected.

Recent full-scale fire tests on composite steel-concrete slabs confirmed a good fire performance when exposed to a long ISO fire (Zhao et al., 2008; COSSFIRE, 2006) even if some of the interior supporting beams of the slab panel are not protected.

The load transfer mode in the slab relies essentially on bending on short spans, with small deflections, at normal temperature. In fire situations, due to the large deflections, the load transfer changes to membrane behaviour along a larger slab panel. Considering this, a design method for composite steel concrete floor slabs in fire was developed by Bailey (2004), allowing designers to specify fire protection to only a portion of the steel beams within a given floor plate. The approach represents a further development of a previous simplified design method (Bailey and Moore, 2000), (Bailey, 2001), based on the experimental work at Cardington. In Bailey design method, the final displacement of the slab is calculated considering the yield-line model. An alternative method to calculate the load capacity of simply supported composite slabs considering the membrane action, by dividing the slab into one center-elliptic part and four rigid parts around, at the limit stage of load capacity, was presented by Li et al. (2007).

Numerical modelling of six full-scale fire tests carried out in 1995-1996 on the composite frame constructed at BRE laboratory at Cardington was performed by (Huang et al, 2002), using the computer program VULCAN, developed at the University of Sheffield. The study highlighted that for low temperatures of the steel unprotected beams, the influence of the concrete slabs on the structural behaviour of the building is small, but when the steel beam temperatures are higher than 500^{o}C, the slab became increasingly influential as part of the load-carrying mechanism.

Another finite element modeling (Moss and Clifton, 2004) of the Cardington test of 2003 as an assemblage of linked composite members, showed the three-dimensional nature of the floor system response, involving two-way action of the floor system, consistent with Bailey model.

A comparison of the Bailey design method with non-linear finite element modelling was performed by (Huang et al, 2004). One conclusion was that the simple design method may predict a greater enhancement of fire resistance due to tensile membrane than is apparent from finite element analyses, in particular for the case of the highly reinforced square slabs, for which the simple method predicts very large enhancement. Cases with reinforcement quantities which are typical for anti-crack meshes, as well as the more rectangular slabs, show less enhancement, and the disparity is less apparent.

In order to evaluate the fire resistance of a composite slab considering the membrane effect, a finite element analysis may nevertheless be required in particular situations. Yet, a complete and detailed numerical modelling of the membrane effect is quite complex and CPU time consuming, due to the simultaneous presence of beams and of orthotropic shells. If such a numerical simulation can be done in research centres and universities, it is not practically applicable for real projects that have to be analyzed in shorter time.

The FE numerical analyses presented in this paper simulate three full scale tests that have been performed in recent years: two have been performed by CTICM in France, FRACOF (Zhao et al., 2008) and COSSFIRE (2006), and one by the Czech Technical University of Prague, in Mokrsko, the Czech Republic (Chlouba & Wald, 2009), (Wald et al, 2010). The numerical analyses have been performed with the advanced calculation model SAFIR, developed at the University of Liege (Franssen, 2005).

The first objective of the paper is to derive the simplest possible models for representing the partially protected composite floors in fire situations that, based on simplifications and approximations, would nevertheless yield a sufficiently close to reality representation of the structural behaviour and a safe estimate of the load bearing capacity. This is why, the simple beam connections were modelled as hinges and the nominal values for material properties were used, as would be the case for a real design situation. This “a priori” predictive simulation approach (Beard, 2000) was considered, in the same manner, to determine the natural fire temperatures of Mokrsko test, by means of the Ozone program (Cadorin and Franssen, 2003, Cadorin et al. 2003), rather than to consider the measured temperatures. The second objective is to highlight the influence of some critical parameters on the behaviour and fire resistance of composite slabs.

2. Main features of the numerical simulations

The transient temperature distribution across the section of the beam and on the thickness of the shell finite elements is determined by means of linear triangular or quadrangular conductive elements. Thermal properties are temperature dependent and evaporation of moisture is considered by means of modified specific heat. Convection and radiation heat transfer are considered at the boundaries.

To describe the geometry of the cross sections in the beam elements, the fibre model is used. The cross section of the beam is subdivided into small fibres (triangles, quadrilaterals or both). The discretisation that is used across the section of the beams to calculate the temperature development is also used for integrating stresses and stiffness across the section in the subsequent structural analysis. Different boundary conditions can be applied on the surface of the element or on the surface of the insulation if relevant and, by varying these conditions from beam to beam, it is straightforward to consider some beams heated on two sides and others on 3 sides.

Uniaxial mechanical properties of steel used in the beam elements and for the re-bars that are present in the shell elements follow the proposal of EN 1993-1-2 (2005); the stress-mechanical strain relationship is non-linear until 2%, with a horizontal plateau from 2 to 15% and a descending branch thereafter; unloading is plastic.

The plane stress constitutive model used for the concrete of the shell elements is an associate plasticity model with isotropic hardening. Tension is considered by a tension Rankine surface and is implemented in a smeared cracked model. Transient creep is implemented in an implicit manner.

Strength and stiffness of steel recover during cooling whereas this is not the case for strength in concrete. The fact that thermal expansion also recovers during cooling, although partially in concrete, is another factor that explains why the deflection reduces after an hour in the Mokrsko test, see section 5.

Connection between the concrete slab and the steel beams is modelled as perfect, having both types of elements sharing the same nodes. Connections between the unprotected steel beams and the protected edge beams are modelled as hinges. The choice of simple connections being used is motivated by the objective of obtaining a simple model that could be used in everyday practice as opposed to more complex models based on springs implemented in the component based method.

The transient geometrically non-linear behaviour of the structure is calculated in a dynamic analysis; acceleration forces are considered explicitly while numerical damping is used. The convergence criteria used in the iterations at each time step is based on the relative norm of the energy produced by out of balance forces multiplied by iterative displacements.

There is no criterion that would lead to “failure” of the simulations. The simulations keep on running until convergence is not possible, which is an indication of a possible run away failure with displacements increasing at a very fast rate, and it is up to the user to decide whereas the displacements reached at the last converged time step are acceptable or whereas a displacement criteria must be applied. Run away failure, with a vertical asymptote in the time-displacement curve, was observed in most simulations presented in this paper.

3. FRACOF test and numerical simulation

3.1 Test description

A typical composite steel-concrete slab, shown in Fig. 1 (Zhao et al., 2008), was adopted for this test. The slab of the designed test specimen covered an area of 7.35 m by 9.53 m, layed on 6.66 m by 8.735 m steel structure. The slab comprised four secondary beams, two primary beams, four short columns and a 155 mm thick floor slab realised with trapezoidal steel sheet of 0.75 mm thickness (height of the ribs of 58 mm). Normal weight concrete C30/37 was adopted in the design. The reinforcing steel mesh of 7 mm used in the slab was realised with S500 steel grade and had a grid size of 150x150 mm. The axis distance of the steel reinforcement from top of the slab was 50 mm. S235 steel grade was used for secondary beams and S355 for main beams. All steel beams were linked to the concrete slab with the help of headed studs, and to the columns with two common types of steel joints (flexible end plate and double angle web cleats).

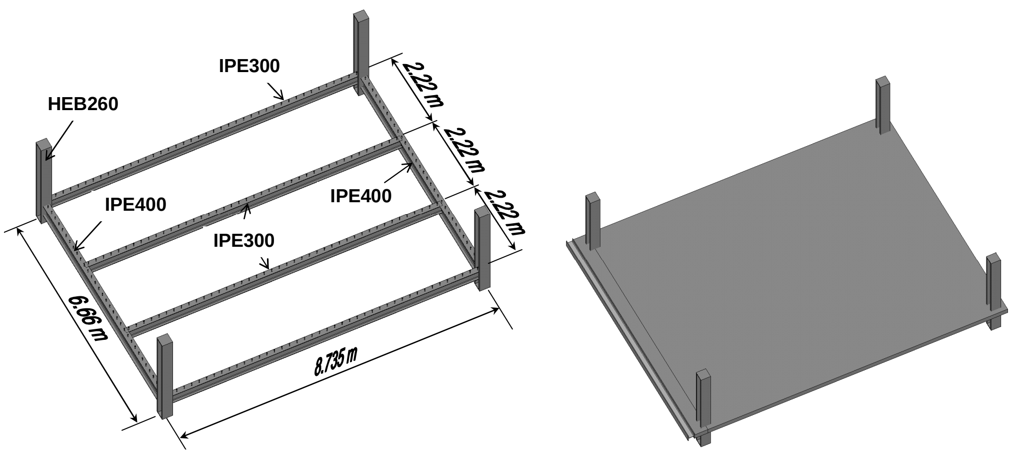


Fig. 1. Tested structure (Zhao et al., 2008)

During the fire test, the mechanical loading of the floor was applied using fifteen sand bags distributed over the floor leading to an equivalent uniform load of 3.87 kN/m². The two secondary beams and the composite floor were unprotected, while all the edge beams of the floor, namely all beams in direct connection with columns, were fire protected with fiber-based insulation to ensure a global structural stability under fire situations. The ISO fire exposure lasted up to 123 minutes.

3.2 Numerical analysis

Because the edge beams were placed on the boundary of the slab, the fire exposure was just on two sides. The properties of the insulation material that have been used in the simulation were the nominal ones (those given by the producer). The fiber-based insulation applied on the edge beams was characterised by: 128 kg/m^3 specific mass, 0 kg/m^3 water content and a temperature dependent thermal conductivity from 0.04 W/mK at 20°C, to 0.48 W/mK at 1200°C. The yield strength for secondary beams was 235 N/mm^2, while for the main beams was 355 N/mm^2. The yield strength of the rebars was 500 N/mm^2. The variation of the thermal conductivity, thermal elongation and specific heat of steel function of temperature was considered as given in EN 1993-1-2 (2005). Other parameters considered for steel are: 7850 kg/m^3 specific mass and 0.7 surface emissivity. A siliceous type of concrete was considered, with mechanical properties characterised by 30 N/mm^2 compressive strength and 2 N/mm^2 tension strength. The upper limit of the thermal conductivity, according to EN 1992-1-2 (2005) was considered for concrete. Other parameters considered for the concrete within the composite slab are: 2400 kg/m^3 specific mass, 46 kg/m^3 water content and 0.7 surface emissivity. Due to the ISO fire exposure, for all materials, the convection coefficient on heated surfaces was considered 25 W/m^{2}K, while the convection coefficient on unheated surfaces was 9 W/m^{2}K.

For the unprotected beams, the fire exposure was considered on three sides (without the top flange). Fig. 2 shows the comparison between the calculated temperatures and the measured ones by means of thermocouples, at mid-height of the web of the secondary unprotected beams.

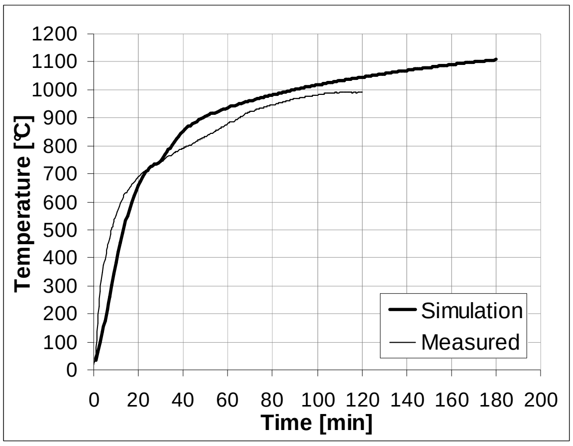


Fig. 2. Temperature in the secondary unprotected beams (mid-height of the web)

In the structural analyses, only the concrete located above the trapezoidal sheets has load bearing capabilities. However, the presence of the ribs is important for the temperature distribution in the concrete and in the rebars. For the thermal distribution, in order to obtain a simple numerical model, the cross section of the slab containing ribs has been replaced by a section with an equivalent thickness calculated according to EN1994-1-2 Annex D (EN1994-1-2, 2005). Fig. 3 shows a good agreement between the evolution of the measured and simulated temperature in the slab at the rebar level, considering this simplification.

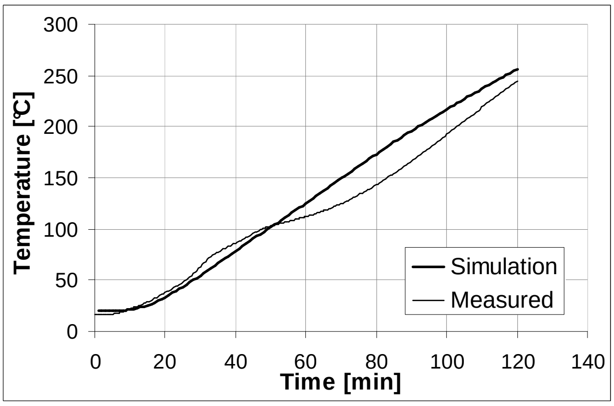


Fig. 3. Temperature evolution in the slab at rebar level

The primary and secondary beams have been idealised using beam elements, and the slab using shell elements. For the structural analysis, only the concrete that is present above the corrugated steel profiles was considered, while the concrete underneath only forms a thermal protection equivalent to the protective effect of the ribs. According to the joints details from the test, the beam-to-column and beam-to-beam connections were modelled as pinned. The rebars have been modelled as an equivalent steel layer on the thickness of the shell element, in amount of 256 mm²/m. Even though at the test the load was "concentrated" by using sand bags, in the simulation the load was considered uniformly distributed. For the material properties, the nominal values have been used, not the measured ones.

In Fig. 4, the calculated deformed shape and the membrane stresses of the slab are shown, at 165 minutes. At this moment, in the simulation, the structure failed due to large deflections of the secondary edge beams. The membrane action, characterised by the equilibrium between the compression of the concrete on the edges of the slab and the tension in the rebars from the middle of the slab, was overreached, and the slab could not uphold the load any longer. The chart shows the comparison between the measured and the calculated deflection at the centre of the slab.

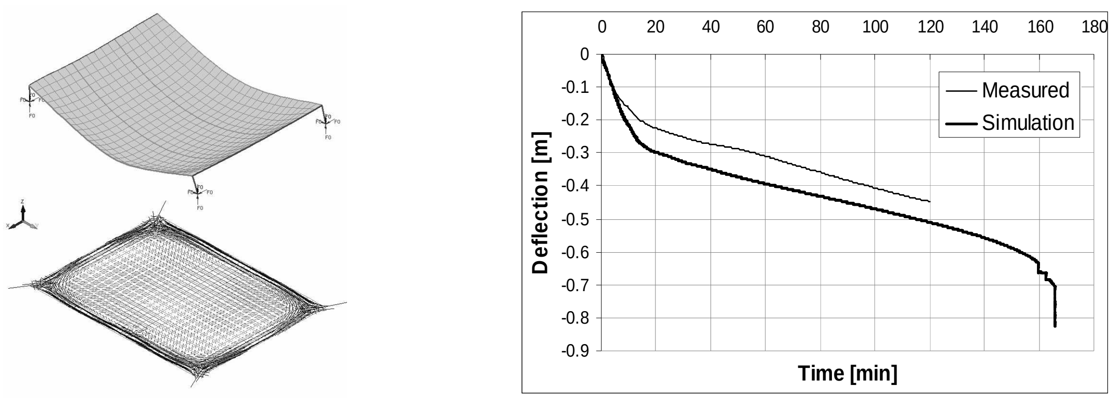


Fig. 4 Deformed shape and membrane forces – Deflection in the middle of the slab

4. COSSFIRE test and numerical simulation

4.1 Test description

The concrete slab covered an area of 6.66m by 8.5m, as shown in Fig. 5 (COSSFIRE, 2006). The composite steel and concrete floor was made of five secondary beams, four primary beams, six short columns and a 135 mm thick slab realised with trapezoidal steel sheet of 0.75 mm thickness (height of the ribs of 58 mm). Normal weight concrete C30/37 was adopted in the design. The reinforcing steel mesh with 8 mm diameter used in the slab was realised of S500 steel grade and had a grid size of 200x200 mm. The axis distance of the steel reinforcement from the top of the slab was 35 mm. As in case of the FRACOF test, the steel beams were linked to the concrete slab with help of headed studs, and to the columns with flexible end plate and double angle web cleats.

During the fire test, the mechanical loading of the floor was applied using sand bags, distributed over the floor, leading to an equivalent uniform load of 3.75 kN/m². The interior secondary beams and the composite slab were unprotected. All the boundary beams of the floor were fire protected by fiber-based insulation, in order to ensure a global structural stability under fire situations. As for the FRACOF test, the ISO fire exposure was stopped after 123 minutes.

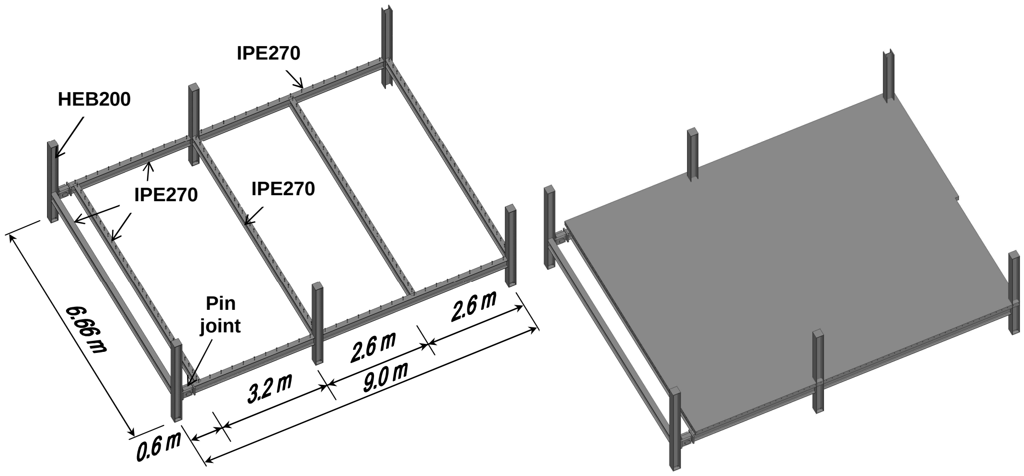


Fig. 5 Tested structure (COSSFIRE, 2006)

4.2 Numerical analysis

The protected beams were placed on the edges, leading to a fire exposure on two sides. For the unprotected secondary beams, the fire exposure was considered on three sides, as in case of FRACOF model. The same type of insulation as for FRACOF model was considered for the edge beams, characterised by: 128 kg/m^3 specific mass, 0 kg/m^3 water content and a temperature dependent thermal conductivity from 0.04 W/mK at 20°C, to 0.48 W/mK at 1200°C. The yield strength for both secondary and main beams was 235 N/mm^2. The variation of the thermal conductivity, thermal elongation and specific heat of steel function of temperature was considered as given in EN 1993-1-2 (2005). Other parameters considered for steel are: 7850 kg/m^3 specific mass and 0.7 surface emissivity. A siliceous type of concrete was considered, with mechanical properties characterised by 30 N/mm^2 compressive strength and 2 N/mm^2 tension strength. The upper limit of the thermal conductivity, according to EN 1992-1-2 (2005) was considered for concrete. Other parameters considered for the concrete within the composite slab are: 2400 kg/m^3 specific mass, 46 kg/m^3 water content and 0.7 surface emissivity. Due to the ISO fire exposure, for all materials, the convection coefficient on heated surfaces was considered 25 W/m^{2}K, while the convection coefficient on unheated surfaces was 9 W/m^{2}K.

Fig. 6 shows the very good agreement between the calculated temperatures and the measured ones by means of thermocouples, at mid-height of the web of the secondary unprotected beams. The FE simulation was carried out for 180 min, even if the experimental program lasted for 123 min, in order to define the failure of the composite assembly.

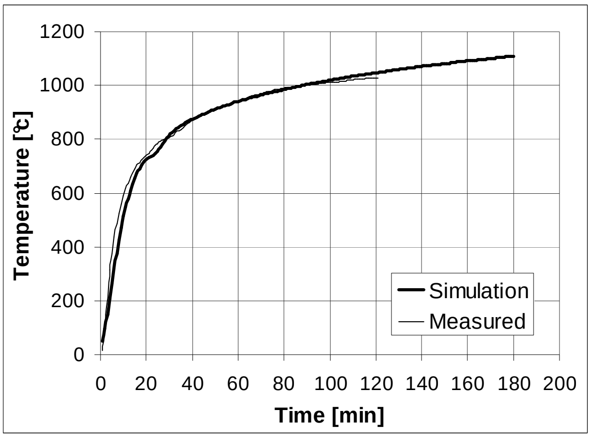


Fig. 6 Temperature in the secondary unprotected beams (mid-height of the web)

For the thermal distribution, as in case of the FRACOF numerical model, the cross section of the slab containing ribs has been replaced by a section with an equivalent thickness calculated according to EN1994-1-2 Annex D (EN1994-1-2, 2005). Fig. 7 shows the variation of the measured and the calculated temperature in the slab at the rebar level.

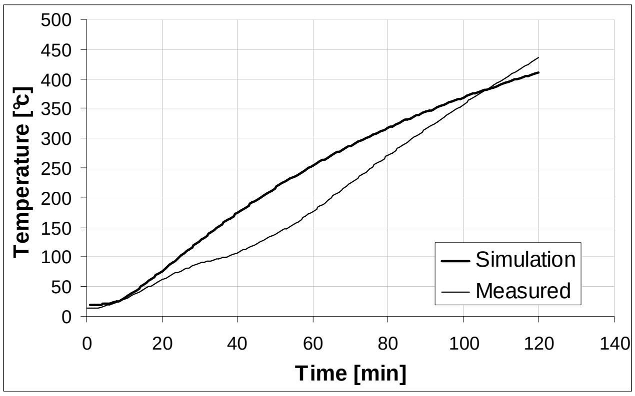


Fig. 7. Temperature variation in the slab at rebar level

The primary and secondary beams have been idealised with beam elements and the slab with shell elements of uniform thickness. The beam-to-column and beam-to-beam connections have been modelled as pinned. The rebars have been idealised as a steel layer in amount of 251 mm^2/m. In the simulation the load was considered uniformly distributed. For the material properties, the nominal values were considered.

In Fig. 8, the calculated deformed shape and the membrane forces of the slab after 149 minutes are shown. At this moment the composite slab failed, in the same manner as for the model of FRACOF structure, due to the large deflections of the secondary edge beam. In the chart, a comparison between the measured and the calculated deflection in the centre of the slab is shown.

After about 60 minutes a difference can be observed between the measured and the calculated deflection curves. In the test, for one of the secondary edge beams, damage of the insulation was observed, which was confirmed by an increase in temperature near the upper flange. For the mentioned edge beam, the measurements also show an increase of deflection at the middle of the span (see Fig. 9), affecting in this way the deflection in the middle of the slab. This effect, which could not be predicted before the test, has not been incorporated in the simulation.

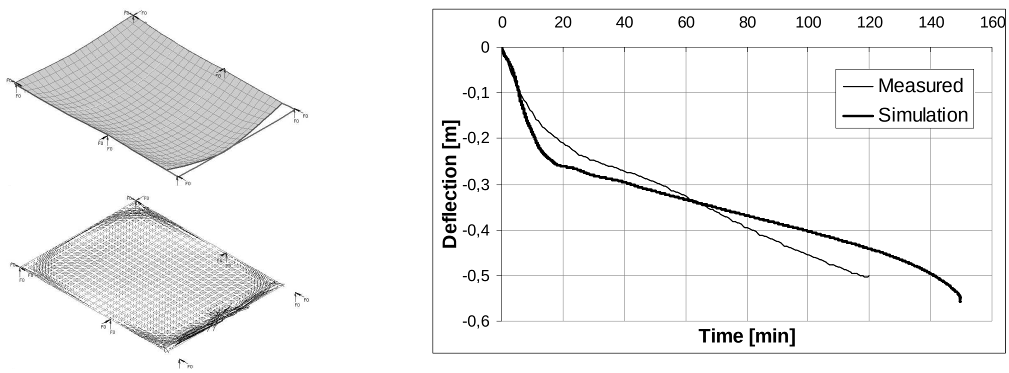


Fig. 8 Deformed shape and membrane forces – Deflection in the middle of the slab

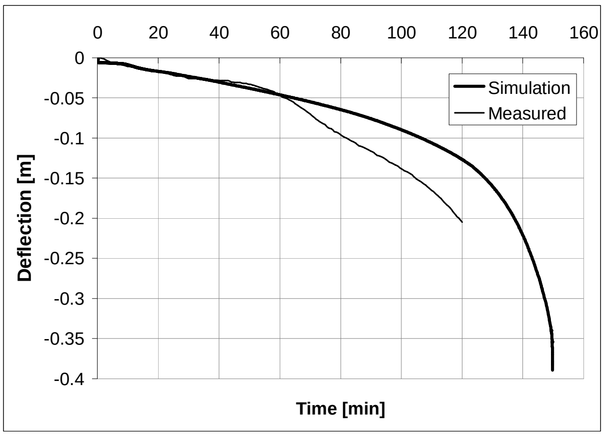
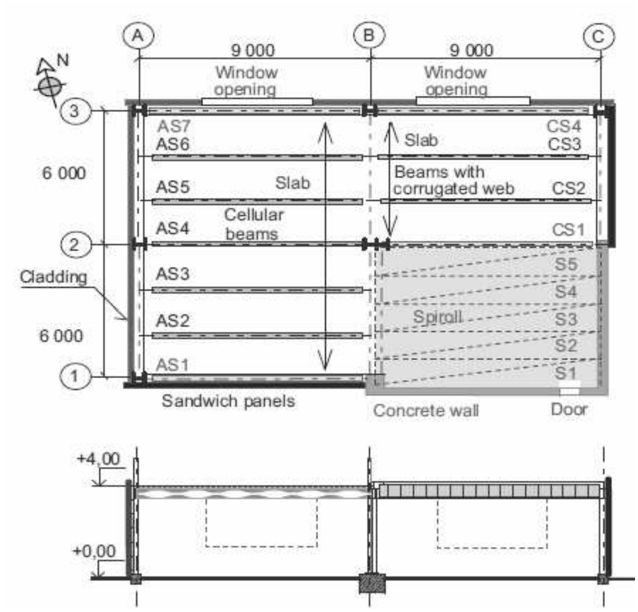


Fig. 9. Deflection of the secondary edge beam

5. MOKRSKO test and numerical simulation

5.1 Test description

The experimental structure, shown in Fig. 10a (Chlouba & Wald, 2009, Wald et al, 2010), represents one floor of an administrative building of 18 x 12 m. The composite slab on the castellated beams was designed with a 9 to 12 m span and on beams with corrugated webs with a 9 to 6 m span. The composite floor has 120 mm thickness with trapezoidal steel sheet of 0.75 mm thickness (height of the ribs of 60 mm). The concrete used for the composite slab was classified as C30/37, and was reinforced by a smooth mesh of ø5 mm 100/100 mm with 500 N/mm² strength. The axis distance of the steel reinforcement from the top of the slab was 40 mm. The height of the castellated Angelina beams with the sinusoidal openings, design by Arcelor Mittal (ASI-AS7), made of IPE270 section from S235 steel grade, was 395 mm. As shown in Fig. 10b (Chlouba & Wald, 2009; Wald et al, 2010), the particular cross-section of Angelina beams is obtained from cutting and re-welding of a common hot-rolled profile. The geometrical parameters which define uniquely the shape of the opening, are governed by functional requirements. The edge beams were from IPE400 sections and S235 steel grade. The beam to beam and beam to column connections were designed as header plate connections. The fire protection of the columns, primary and edge beams was designed for 60 minutes fire resistance, by board protection.



a)

b)

Fig. 10 Tested structure (Chlouba & Wald, 2009; Wald et al, 2010)

The dead load of the tested structure reached 2,6 kN/m². The variable load of 3.0 kN/m² was simulated by sand bags. The fire load consisted of wooden piles of 35.5 kg/m² simulating a fire load density of 620 MJ/m². The openings of 2.54 m height and a total length of 8.00 m with 1.0 m parapet ventilated the compartment. To allow a smooth development of fire, no glazing was installed.

Under the composite floor with castellated beams, a temperature of 935°C, was measured after 60 minutes. The slab failed after 62 minutes, at a time which was presumably the beginning of the cooling phase of the fire, with the measured temperature of the lower flange of the beam at the mid span of 895 °C.

5.2 Numerical analysis

The numerical simulation was performed for the 9x6m^2 zone, where the slab is supported by Angelina beams (between axes A-B and 1-3, see Fig. 10). The columns and the cross braces were not modelled. Therefore, the thermal analysis was realised only for the Angelina beams, the composite slab, and two types of protected edge beams, with fire exposure on three sides and respectively on two sides.

For the numerical model, the gypsum board fire protection considered for the edge beams was characterised by: 648 kg/m^3 specific mass, a temperature dependent thermal conductivity ranging from 0.12 to 0.27 W/mK in the 20 to 800°C temperature range and the effect of moisture evaporation and different endothermic chemical modifications being introduced in the enthalpy formulation. The yield strength for all the steel beams was 235 N/mm^2. The variation of the thermal conductivity, thermal elongation and specific heat of steel function of temperature was considered as given in EN 1993-1-2 (2005). Other parameters considered for steel are: 7850 kg/m^3 specific mass and 0.7 surface emissivity. A siliceous type of concrete was considered, with mechanical properties characterised by 30 N/mm^2 compressive strength and 2 N/mm^2 tension strength. The upper limit of the thermal conductivity, according to EN 1992-1-2 (2005) was considered for concrete. Other parameters considered for the concrete within the composite slab are: 2400 kg/m^3 specific mass, 46 kg/m^3 water content and 0.7 surface emissivity. Due to the natural fire exposure, for all materials, the convection coefficient on heated surfaces was considered 35 W/m^{2}K, while the convection coefficient on unheated surfaces was 9 W/m^{2}K.

Using the details from the test related to the fire load, the openings and the boundaries of the compartment, a fire curve has been obtained with Ozone program (Cadorin and Franssen, 2003, Cadorin et al. 2003) and used further in the thermal analysis. This curve is compared in Fig. 11 with the measured gas temperatures in the test.

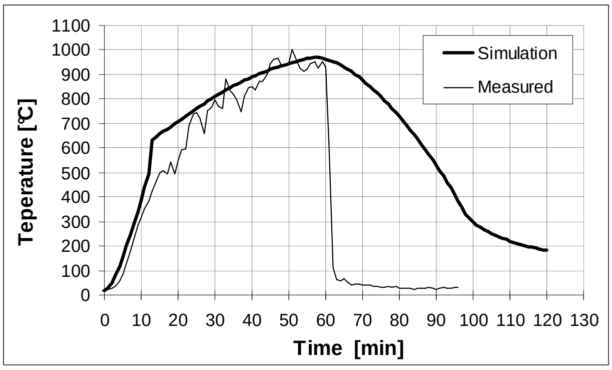


Fig. 11. Comparison between the Ozone fire curve and the gas measurements

The influence of web openings on the behaviour of steel beams at elevated temperatures was studied numerically by Yin and Wang (2006). It was emphasised that the presence of the openings have substantial influence on the critical temperatures of axially unrestrained beams, while for restrained beams the effect of web openings on the beam‘s large deflection behaviour is smaller. In order to simplify the numerical model, the castellated Angelina beams of the Mokrsko test were modelled using beam elements considering a minimum cross-section, for the entire length of the beam. The minimum cross-section consists of an upper and a lower T, representing the flanges, together with the parts of the web above and bellow the opening. Fig. 12 shows the comparison for the temperature in the lower flange of the Angelina beams. For the thermal distribution, the section of the slab containing ribs has also been replaced by a section with equivalent thickness, in a similar way as for the FRACOF and COSSFIRE numerical models.

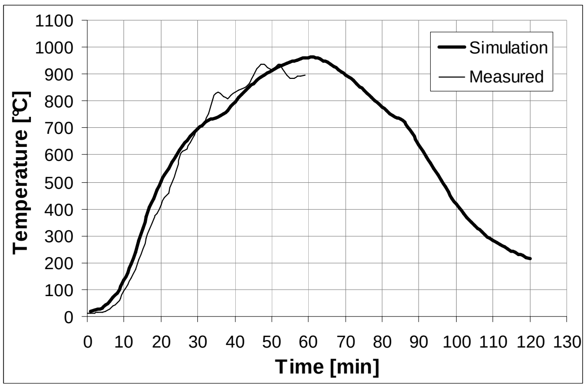


Fig. 12. Temperature in lower flange of the Angelina beams

In the numerical model, the edge beams and the unprotected Angelina beams were idealised using beam elements, and the slab using shell elements. Vertical supports have been used for representing the columns, and horizontal restrains for the cross braces and for the continuity of the slab.

Fig. 13 shows the calculated deformed shape and the membrane forces of the slab at failure, namely a concrete failure in the corner of the slab. Failure of the numerical model appeareed after 56 minutes, due to lack of convergence in the concrete constitutive model, appearing for large strains and softening behaviour that developed at that stage. The chart in Fig. 13 shows the deflection curve from the simulation compared to the measured deflection from the test for the middle area of the slab.

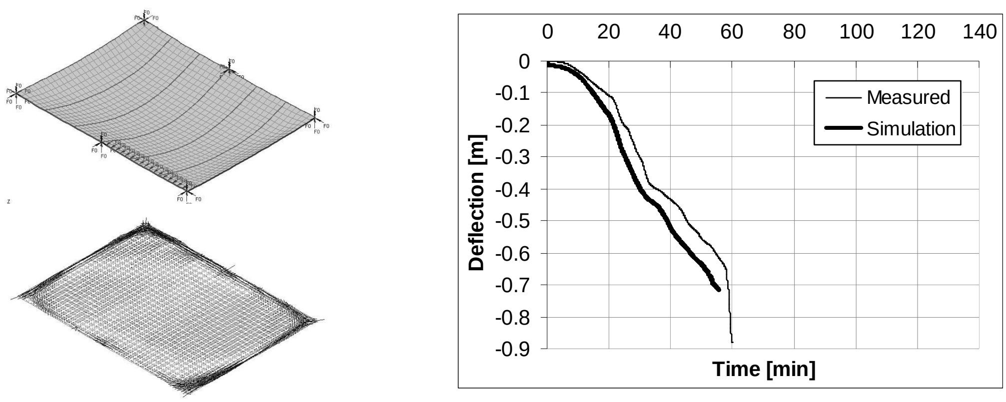


Fig. 13. Deformed shape and membrane forces – Deflection in the middle of the slab

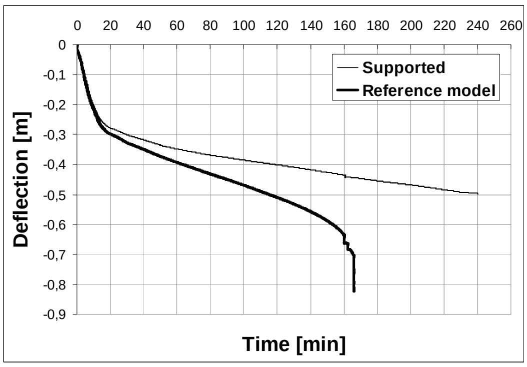
6. Sensitivity study

For the three tests, a numerical sensitivity study has been performed in order to see the influence of a number of parameters on the mechanical response of a composite slab. For each parameter, supplementary simulations have been done and then compared with the reference numerical models presented above. The influence of the vertical supports on the edges, of the presence of the unprotected secondary beams, of the thickness of the slab and of the amount of reinforcement was investigated.

6.1 Influence of the vertical supports on the edges

For the three tests, a model was built in which all the edges of the composite slab were fully restrained vertically. The aim was to investigate whether this simplified model may lead to a safe estimate of the fire resistance time, thus avoiding considering the fire protection on the edge beams in the finite element model. The influence of various degrees of edge-beam protection in the development of the membrane mechanism in composite slabs was studied by Abu et al. (2008). The numerical analyses under elevated temperatures have shown that the tensile membrane action is lost when the temperatures in the protected beams exceed about 600°C. At this stage, plastic hinges form in the edge beams.

For the FRACOF and COSSFIRE tests, the slab with full vertical fixity on the edges resisted a longer time to the fire exposure than the slab with the real flexible supports, as shown in Fig. 14. The plastic hinge that otherwise formed in the secondary edge beams was avoided and the collapse of the slab was not reached after 4 and respective 3 hours of ISO fire exposure. For the Mokrsko test, the deflection of the edge beams calculated for the reference case is very small; the deflection at failure is of 25 mm, compared with the deflection of 650 mm in the centre of the slab. This seems to indicate that the flexibility of the edge beams did not play a role in the failure mode. This is confirmed by the numerical simulation, in which the collapse occurred at the same time as when vertical deflections are considered on the edges of the slab; the two curves can hardly be distinguished in Fig. 14c.



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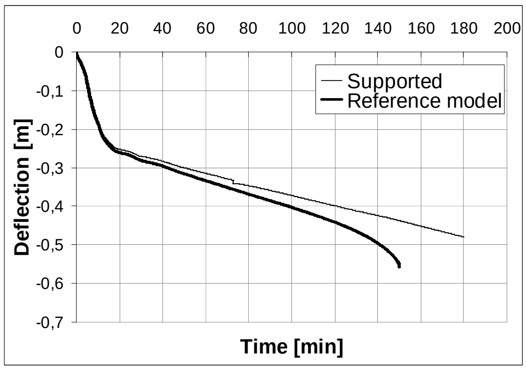
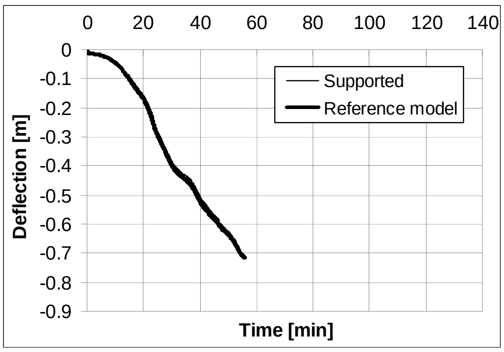


b)



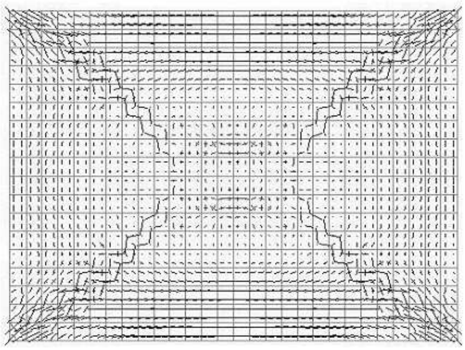
c)

Fig. 14. Influence of the vertical supports on the edges: a) FRACOF b) COSSFIRE c) Mokrsko

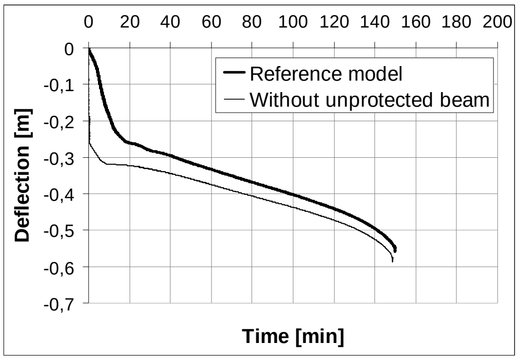
6.2 Influence of the presence of the unprotected secondary beams

For all tests, a model has been considered in which the unprotected secondary beams were neglected, or just a part of the section has been modelled. For the Mokrsko test, the secondary beams were castellated beams, and the question was how to model these, or whether it is really necessary to model these at all. Fig. 15 shows the deflection for the reference numerical models and for the models without the unprotected beams.

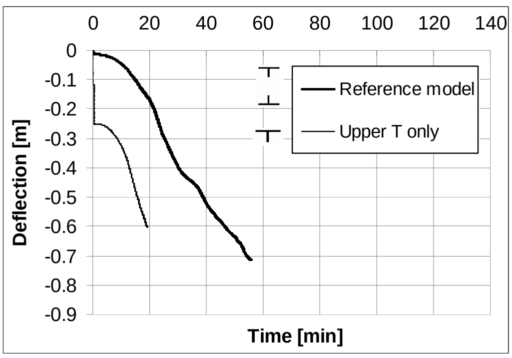
For the FRACOF specimen, when the unprotected beams are neglected, as shown in Fig. 15a, yield lines form in the slab during application of the load at room temperature, leading to the failure of the slab before the total load can be applied. For the COSSFIRE specimen, when the unprotected beams are not present, the load can be applied but the slab enters from the beginning into tensile membrane. It can be observed in Fig. 15b that the deflection under load at ambient temperature is very small for the reference model for which the unprotected beams are modelled, while it reaches 270 mm if the these beams are not included in the numerical model. As the fire develops, the deflection curve converges towards the same curve as the one obtained when the unprotected beams are present in the numerical model. For the Mokrsko specimen, using the minimum section for the Angelina beams (by means of the upper and lower T, thus considering the presence of the openings for the entire length of the beams), leads to a good correlation with the test. In the case where just the upper T is considered to represent the cross-section of the Angelina beams, an early failure of the slab occurs.



a)



b)



c)

Fig. 15 Presence of the unprotected beam: a) FRACOF b) COSSFIRE

c) Mokrsko

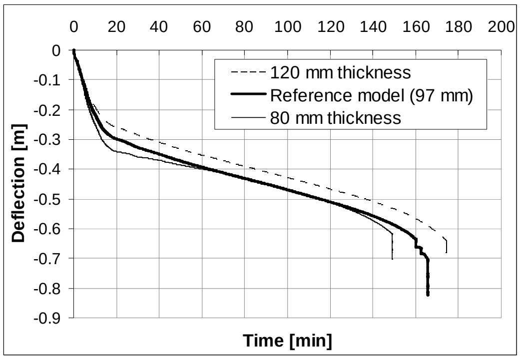
This seems to indicate that considering a simpler numerical model, in which the presence of the unprotected beams is ignored (or considered in a limited way), if this model leads to stability after a significant duration, the displacements calculated with this simple model are a good approximation, on the safe side, of the displacements that would be calculated by a more complete model. If, on the other hand, the simpler model without the unprotected secondary beams leads to failure, a complete model must be considered.

6.3 Influence of the thickness of the slab

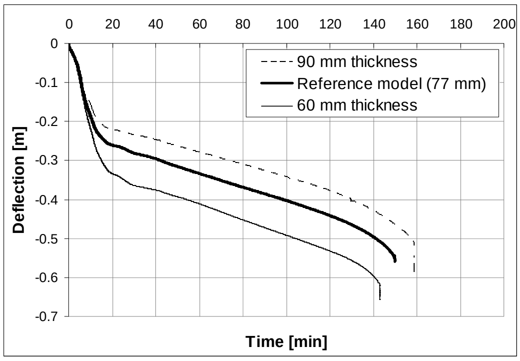
Numerical models with different thickness of concrete covering the steel trapezoidal sheets (effective slab thickness) were considered. Fig. 16 shows that a higher thickness generally leads to lower deflections although the trend is not continuous as can be seen on Fig. 16a and 16c. This may be due to the fact that the deflections result from different effects such as thermal and mechanical elongation of the steel bars in the slab that tend to increase when the thickness decreases, but also thermal gradient across the thickness of the slab that tends to decrease with decreasing thickness. For the FRACOF and COSSFIRE tests, fire resistances over 120 minutes are achieved even with values of the effective slab thickness lower than in the reference tests. For the Mokrsko test, the behaviour of the slab is modified completely, from failure to stability, by increasing the effective slab thickness from 60 mm for the reference model to 70 mm, while a value of 50 mm leads to fire resistance times bellow 40 minutes. For the 70 mm slab, the deflection recovers partly during the cooling phase that is considered in the natural fire.

6.4 Influence of the amount of reinforcement

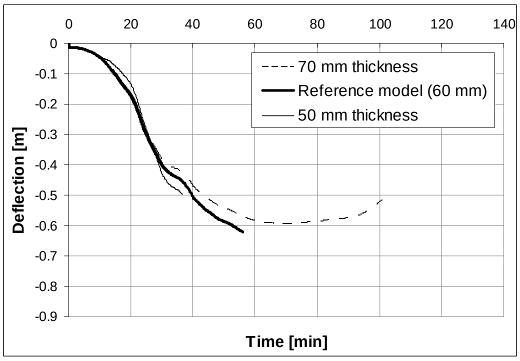
Numerical models with different quantities of reinforcements were considered with nearly unchanged results for the FRACOF and COSSFIRE tests, as shown in Fig. 17. A significant improvement is observed for the Mokrsko test, with a behaviour changed from failure to stability when the amount of steel is increased by 44%, see Fig. 17c.



a)

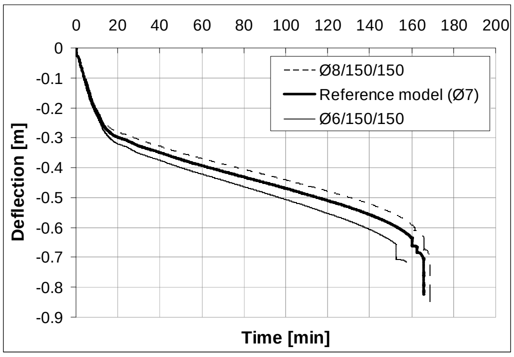


b)

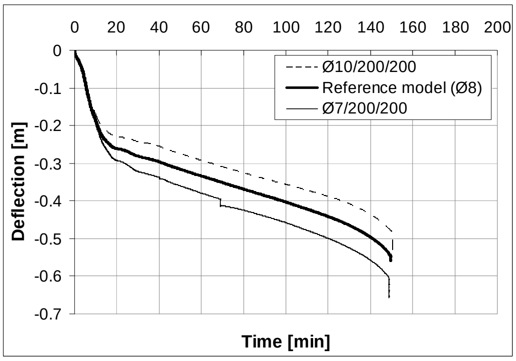


c)

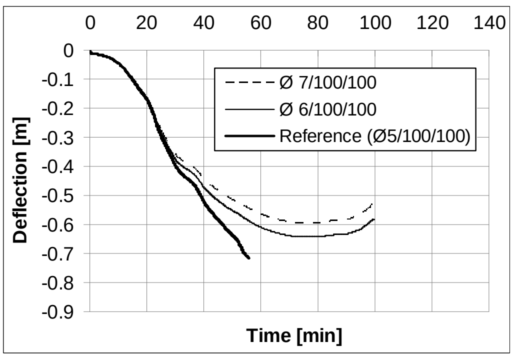
Fig. 16. Influence of the slab thickness: a) FRACOF b) COSSFIRE c) Mokrsko



a)



b)



c)

Fig. 17. Influence of the reinforcement: a) FRACOF b) COSSFIRE c) Mokrsko

7. Conclusions

Using the available information from three real scale fire tests, FRACOF, COSSFIRE and Mokrsko, numerical simulations have been done using the computer program SAFIR.

The primary and secondary beams have been idealised using beam elements, and the slab using shell elements. In order to obtain a simple numerical model, the cross section of the slab containing ribs has been replaced by a section with an equivalent thickness calculated according to EN1994-1-2 Annex D. Using this approximation, the values of the simulated temperatures in the rebars are in the safe side compared with the values of the temperatures obtained from tests. The comparison between the numerical results and the test results shows good correlation, indicating that the simplified numerical model may be used in design. Differences of time resistance for FRACOF and COSSFIRE tests could not be addressed because the fire exposure in the tests was stopped after 120 minutes. For the Mokrsko structure, the fire resistance time in the numerical simulation is almost the same as the failure time observed in the test, with a difference of 5 minutes, in the safe side for the numerical model.

An important option when the numerical model is built, is to consider or not the vertical restraints along the edge of the composite floor. For the FRACOF and COSSFIRE specimens, the failure highlighted by the numerical analysis was caused by plastic hinges forming in the secondary edge beams. When the edges of the slabs are completely restrained vertically, the plastic hinges forming in the secondary edge beams are avoided and the fire resistance times are significantly increased for these two slabs. For the Mokrsko test, the presence of the vertical restraints did not changed the failure time, which suggests that the flexibility of the edge beams did not play a role in the failure mode. Considering the results of the numerical simulations for the three cases, it is recommended to avoid the vertical restraints on the edge of the composite floor, even if this would simplify the numerical model.

The presence of the secondary unprotected beams is generally necessary in the numerical model. A simpler model, without these beams may be considered, however, if this model leads to stability after a significant duration.

There seems to be a critical thickness of the concrete slab, with lower values leading to premature failure and higher values decreasing somehow the vertical displacements but not increasing significantly the fire resistance time. In a similar manner, an increase of the amount of reinforcement does not seem to have significant effect, on the condition that the minimum amount that is required to avoid premature failure has been provided.

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