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FIRE RESISTANCE OF COMPOSITE FLOORS: NUMERICAL EVALUATION OF DESIGN PARAMETERS

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Abstract. *Designing a composite floor at ambient temperature is a relatively simple task. Considering fire conditions, the floor designed at ambient temperature has to be verified to accomplish requirements of fire resistance. Concerning to stability in fire during the required time of fire resistance — RTFR passive fire protection may be needed. The passive protection of columns and of main border beams avoiding protection of secondary beams is accepted as the more economic fire protection strategy. This work numerically evaluates the influence of the total area of reinforcement bars and of the characteristic resistance of concrete in looking for a stable panel configuration in fire. Conclusions indicate that the greater the area of reinforcing bars the smaller the central node displacement as expected. But it is observed that the increasing concrete characteristic resistance does not implies smaller central node displacements in face of the intense cracking of concrete at the center of the panel.*

Keywords: *Composite floors; design parameters; passive fire protection; secondary beams.*

INTRODUCTION

Composite floors are frequently used in steel structured buildings for commercial, office and residential occupations. A typical modulus is made of four columns, four main border beams, a number of secondary beams and a concrete slab supported by a steel deck. The number of columns may vary, but practical observations show that typical moduli vary from 5 to 10 m both width and length. As a consequence, in such buildings a compartmented fire is the most probable fire scenario.

The design of composite floors at ambient temperature is a very simple task. Technical standards like the ABNT NBR 6118:2014, ACI 318-11 and the ANSI/SDI C-2011 are applicable. Considering fire conditions, the floor and its supporting beams and columns once designed at ambient temperature are verified to accomplish requirements of fire resistance. Concerning to stability in fire during the required time of fire resistance — RTFR passive fire protection may be needed.

The passive protection of columns and of main border beams avoiding protection of secondary beams is accepted as the more economic fire protection strategy based on the experience of countries where steel structures are intensively used like United Kingdom. Nevertheless, to avoid fire protection of secondary beams it is needed to show that the floor and its supporting structure remains stable in fire at least during the RTFR.

It is noticeable that the considered structural modulus is not supposed to be redesigned in fire conditions: it has to be only verified in fire conditions against acceptance criterion. As columns and border beams are fire protected, when using FEM analyses, limiting criterion of maximum calculated panel deflection in fire is imposed. By panel it is here understood the set of concrete slab supported by steel deck and secondary beams. Panel deformation is measured through the displacements of some control nodes adequately chosen by the engineer responsible for fire behavior structural analysis.

When the limiting criterion is over passed at any control node, it is said that the entire substructure is unstable in fire for the RTFR. In this case, the secondary beams should be protected against fire. It may happen that the cost of passive fire protection of secondary beams may compete with the cost of redesign of the floor for example increasing one or more of the following parameters: slab height, sections of secondary beams, area of reinforcement bars and characteristic resistance of concrete. Moreover, in the general case, fire analysis is made when the steel structure is under manufacture. This fact limits the options of redesign the structure looking for a better strategy of fire protection to a few slab design parameters.

This work numerically evaluates the influence of the total area of reinforcement bars and of the characteristic resistance of concrete in looking for a stable panel configuration in fire. It is supposed that changing these parameters it may be possible to pass from an unstable to a stable condition in fire even more when the control node displacement is slightly greater than the limiting criterion.

1 METHODOLOGY

An analysis of a substructure extracted of a real building was chosen in this research. All dimensions are the ones determined by the design method at ambient temperature. A Finite

Element Method program specially suited for fire behavior analyses of composite floors called VULCAN is used. VULCAN is a product of Vulcan Solutions Ltd. of Sheffield, UK and was tested against experimental results from Cardington Fire Tests (Huang; Burgess; Plank, 1999). Nowadays, VULCAN analytical results are accepted in several countries as reliable data of the fire behavior of a structure.

A VULCAN substructure model is prepared considering all geometrical details and physical properties. Fire is identified with the standard-test of ABNT NBR 5628:2001. Columns and main border beams are protected so that their heating regime is linear from ambient temperature to 550 °C at the end of the RTFR. The secondary beams and the slabs are supposed not protected and its heating regime follows the standard-test curve.

Cross sections of I steel bars are divided in 12 elements and slab cross sections are divided in a number of layers. The temperatures of these transversal sectional elements and layers may vary describing a temperature profile. The temperature profile on beams sections varies linearly from 0.8 on the top flange, 1.0 on the mid height of the web and 1.0 on the bottom flange. Temperature profile on slabs is supposed varying linearly from 1.0 at its bottom surface and 0.2 at its upper surface.

Slabs supported by steel deck are modeled using effective thickness in each direction. Connectors are modeled and partial interaction is admitted. The loading cases of ABNT NBR 14323:2013 are not considered, but more conservative ones are taken as the total fixed load plus 20%, 40% and 60% of accidental loads in each case.

A composite floor is analyzed herein noting that the expression "composite floor" means a substructure modulus as described before. To identify each one of the 45 different cases analyzed three symbols are used: CXX stands for concrete of f_{ck} = XX MPa; QYY stands for CA-60 steel rebar mesh of YY mm^2/m in each direction; LR0N stands for the loading combination equal "fixed loading + 0.N x accidental loading".

2 PANEL'S STABILITY CRITERION

The concept of structural collapse in fire is not directly stated in norms like IT-08/11 or in NBR 14432:2001 nor in NBR 14323:2013. However, from these standards it is concluded that fire collapse arises from the loss of stability or from the loss of the resistive capacity of the structural elements under the action of the structural stresses. Although not explicit, these are localized phenomena because in these normative texts the fire is supposed to be compartmented.

The aim of the structural stability analysis in fire is to avoid the collapse of the structure which has to be understood as to avoid the occurrence of excessive deformations. The BS 476 Part 20 standard uses the following criteria for excessive deformation control in furnace beam and column tests:

(a) Beams are considered in structural collapse when the vertical displacement of the control node is equal to or greater than the twentieth of its span;

(b) Columns are considered in structural collapse, when, after thermal expansion, they return to their initial length.

These are merely conventional criteria and are aimed at avoiding possible damage to measuring equipments in the standard furnace test. In fire structural stability analyzes these criteria can be applied to beams, slabs and columns as it was suggested by the observations of Cardington Fire Tests. In the specific case of slabs, herein it is proposed to attenuate the displacement limit of the control node from L/20 to L/15, where L is the smallest span of the panel, except in situations where large deformations of floors may interfere adversely on escape routes.

3 RESULTS

Structural steel profiles used are shown in Figure 1. Steel has a yielding stress f_{yk} = 345 MPa. Slab height is 140 mm with continuous depth of 66 mm. ASTM A 653 G40 steel deck is used with 8 mm thickness. Parallel slab stiffness is 0.69 and perpendicular stiffness is 0.19. Fixed loading is 2.9 kN/m^2 and accidental loading is 6.25 kN/m^2. The RTFR is 90 minutes. The control node is the very central node of the panel. Table 1 summarizes the control node displacements.

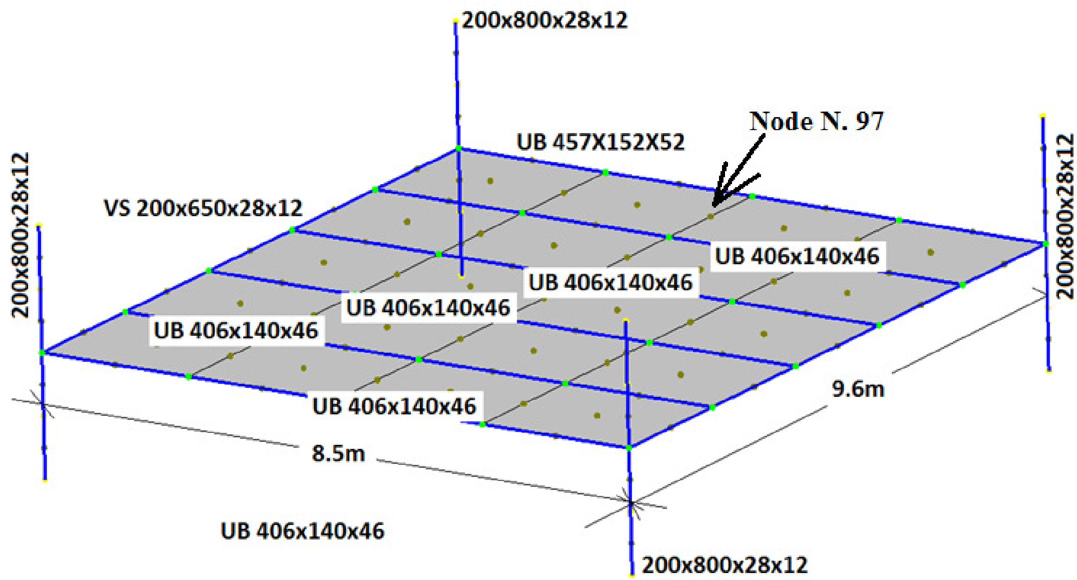

Figure 1. Structural geometric dimensions and steel sections.

It is observed in Table 1 that in all loading cases the substructure is stable during the RTFR = 90 minutes as the maximum calculated displacement is 369 mm < 425 mm which corresponds to 8500 mm/20. It is noticeable that the fire protection of columns and main border beams is designed to reach a steel temperature of 550 °C at the end of the RTFR.

Plotting the control node displacements as a function of the area of reinforcement bars, Figure 2, it is found very good linear relationships with an average correlation index R^2=0.995 for the three loading combinations. As it is expected, the greater the areas of rebar mesh the smaller the control node displacements. It is observed that the tendency lines are almost parallel with slopes of 0.47, 0.48 and 0.41 for LR06, LR04 and LR02, respectively.

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Table 1. Control node displacements (mm) at the end of RTFR.

L	C	Q92	Q159	Q196
0.2	20	190	158	151
	25	194	147	144
	30	172	151	137
	35	191	162	155
	40	190	173	137
0.4	20	283	251	234
	25	279	244	212
	30	265	248	224
	35	264	245	228
	40	284	247	224
0.6	20	369	324	293
	25	342	324	289
	30	353	311	295
	35	332	311	300
	40	325	305	297

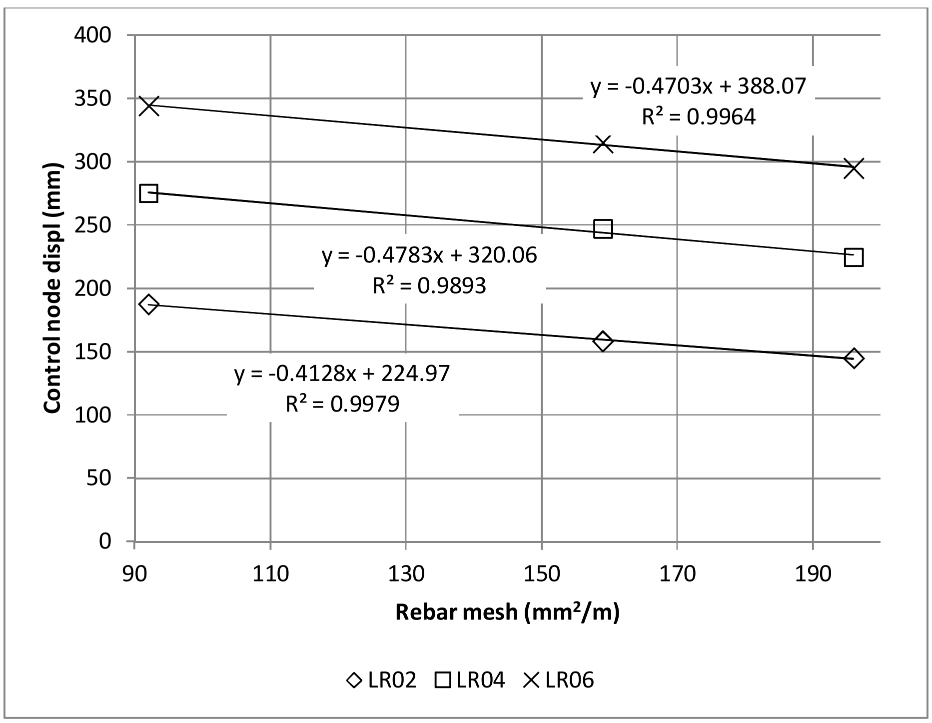

Figure 2. Node displacements as a function of total rebar mesh area for loading combinations.

Figure 3 shows control node displacements as a function of concrete characteristic resistance for loading combinations. The force of the linear relationship is indicated by the correlation index R^2. It is seen that no linear relation exists in the cases of LR02 and LR04 and that a linear relation may be admitted for the loading combination LR06. The expected slab behavior in ambient temperature is smaller displacements for increasing concrete characteristic resistance.

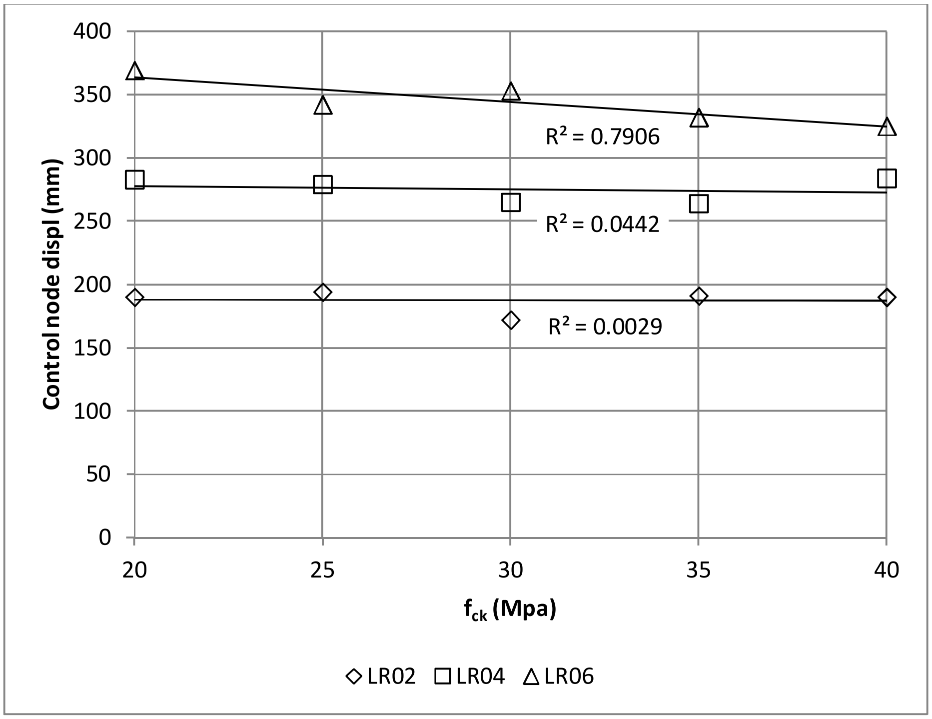

Figure 3. Node displacements as a function of concrete characteristic resistance.

The relationships of Figure 3 show themselves more complex than appears at first sight. Looking for an explanation, one may raise a hypothesis based on membrane action of slabs in fire. As Bailey (2002) states a slab in fire supports loading by tensile membrane action occurring in the center (Tension zone) and a compressive membrane action forming a supporting 'ring' around its perimeter (Compression zone), Figure 4. Thus, when the node displacement is related with concrete characteristic resistance, membrane action suggests that if the node is located at the tension zone where concrete is fractured, displacement is governed by the tensile capacity of the reinforcement and by the remaining compressive capacity of the concrete. Although the steel reinforcement mesh area is the same on the cases of Figure 3, the depth of cracks on the compression zone varies with the loading. Therefore, the relationship between the central node displacement and the characteristic resistance of concrete is not conclusive.

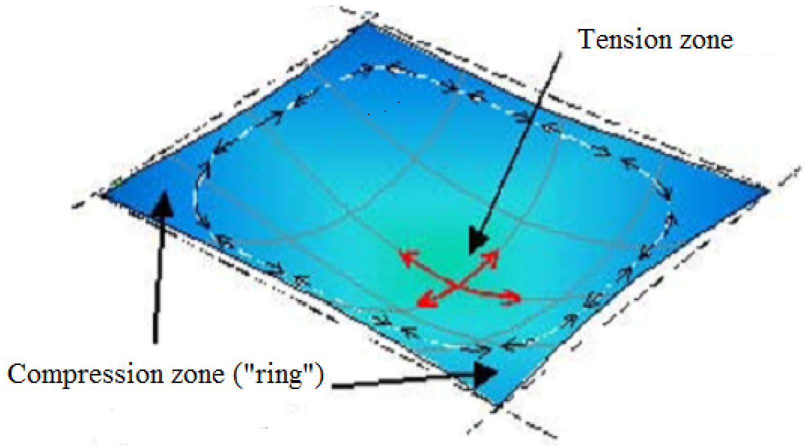

Figure 4. Compression and tension zones in membrane action

(Source: Bailey, 2002)

To test this hypothesis, Figure 4 shows the vertical displacements of the node 97 located in the compression zone (see Figure 1) as a function of the characteristic resistance of concrete. Loading combination is LR02 and total area of steel reinforcement bars is 92 mm^2/m. It is observed that a good linear relationship is found between displacements and f_{ck}.

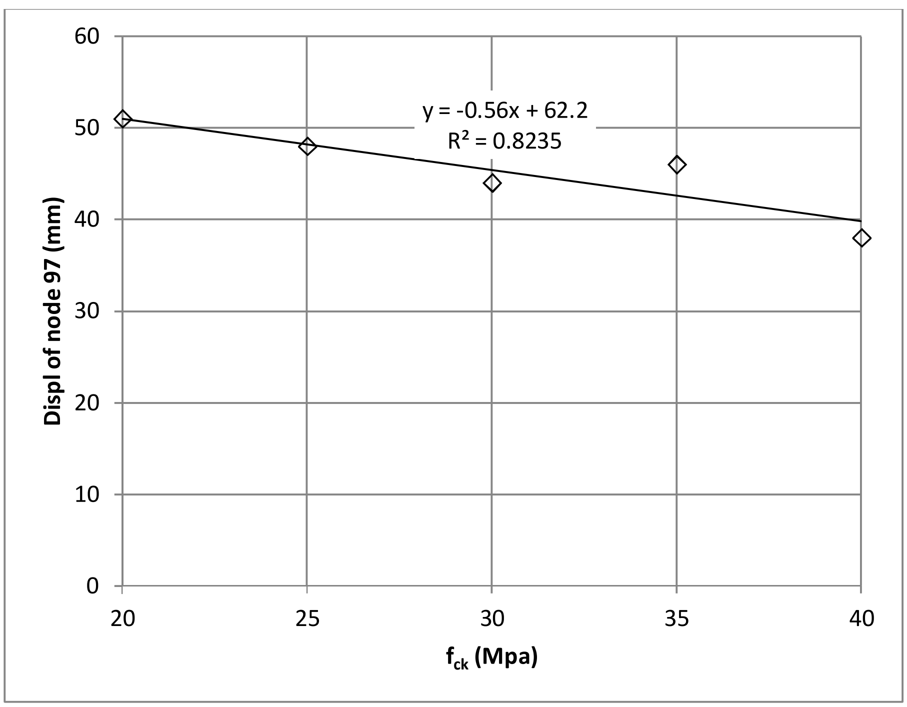

Figure 5. Node 97 displacements as a function of f_{ck}.

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4 CONCLUSIONS

The FEM analyses of a composite floor were used to investigate the influence of the characteristic resistance of concrete and of the total area of steel reinforcement on the slab's vertical displacements. Findings indicate that good linear relationships exist between vertical displacement of the slab's central node and the total area of steel reinforcement bars. But, the dependence of the slab's vertical displacements and the concrete characteristic resistance was found not conclusive. A future investigation of this dependence may consider the membrane behavior of slabs in fire. Prior investigation described indicates that a linear dependence exists between vertical displacements of nodes located in the slabs' compression 'ring' and the characteristic resistance of concrete. On the other hand, for nodes located in the tension zone the displacements seem to be dependent of the extension of the fractured zone and of the total steel reinforcement area.

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