UNPROTECTED COMPOSITE FRAMES WITH CHS COLUMNS
Performance under Fire Loadings

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INTRODUCTION

In last years, composite steel concrete solutions for building structures showed their high potentials for what concerns structural behaviour in fire. The full-scale fire tests undertaken at Cardington [1,2], revealed the possibility of adopting unprotected steel beams and the importance of column elements for the stability of a whole building in fire. At the same time, research studies conducted at the NRCC [3] reported on the inherent fire resistance of concrete filled columns.
Besides, methods for determining the fire resistance of steel and composite structures considerably developed and European codes are now presenting design tools consistent with both test data and material laws at elevated temperatures. However, because of simplifying assumptions often adopted in representing the behaviour in fire, such methodologies may result in over-conservative design.
In the work reported here, the structural fire performance of a composite moment frame with circular steel-concrete filled columns is investigated. The reference case study was defined within the framework of a European research project [4] where it was chosen among different structural solutions [5,6,7] in order to comply with both seismic and fire loadings. In this paper, attention is focused on structural fire performance under assigned thermal boundary conditions, identifying the collapse mechanism leading to failure. Moreover, the influence of restraining conditions on kind of structural failure was assessed by comparing the behaviour of four isolated beams under ideal support conditions. Finally, a numerical procedure [5] for the determination of interaction curves at elevated temperatures was applied for the verification of composite cross-sections in fire.

1 DESCRIPTION OF THE STUDY CASE

The reference study case was defined in the framework of a European Research Project [4]; in the choice of the structural scheme particular attention was given to the definition of the geometric layout to obtain an optimized solution balancing structural performance and cost effectiveness. As a result, the main frame was made of two bays spanning 7,5 and 10 m, respectively; the adoption of quite long spans for the main beams imposed the realization of secondary beams, in order to avoid the presence of propping systems for the composite deck.

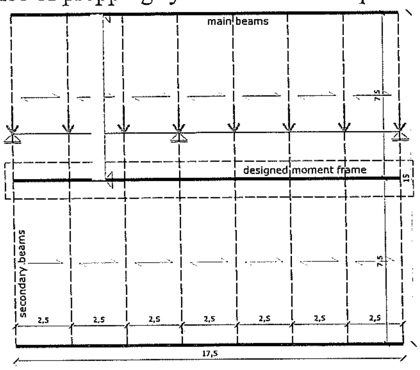


Fig. 1. Typical floor layout

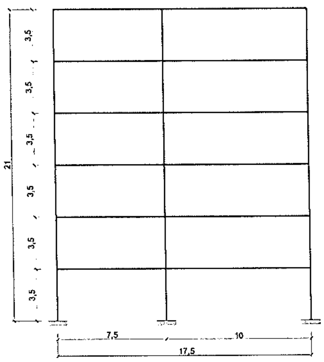


Fig. 2. Main moment frame - elevation

The secondary beams, spanning 7,5 m each, were placed at a distance of 2,5 m and were designed according to a simply supported scheme; the frame was braced in transversal direction. For what concerns the elevation, a medium-rise solution was adopted including five floors with 3,5 m inter-storey height; the schematic of the adopted layout is shown in Figs. 1 and 2. With reference to the defined geometry, several solutions including different kinds of structural elements were analyzed and compared [5, 6] in order to choose the most suitable one with respect to both seismic and fire loadings.
A "full" composite solution was chosen, including steel beams connected to a composite deck and composite circular steel-concrete filled columns.
The design of the frame [4] was made considering both static and seismic combination of actions; in particular, seismic design was made with reference to a peak ground acceleration PGA=0,35g following the ductile design approach of Eurocode 8 [10]. A high ductility class was chosen (q=6) and the dissipative structure was designed referring to the so-called "strong column-weak beam" concept. Finally, main beams were realized with IPE400 steel profiles in composite action with a 150 mm height composite deck realized with a 95 mm solid concrete slab casted on a 55 mm corrugated steel sheeting; composite columns had 457 mm external diameter with 12 mm thick steel tube. Full-strength connections were designed [4] and tested at the Testing facility of the University of Pisa.
Some structural details of the full-scale sub-assemblies are shown in Figs. 3, 4 and 5.

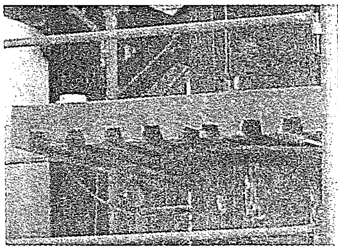
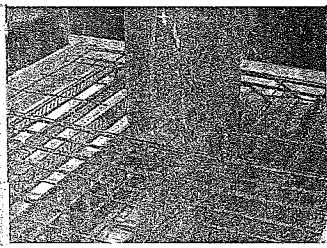
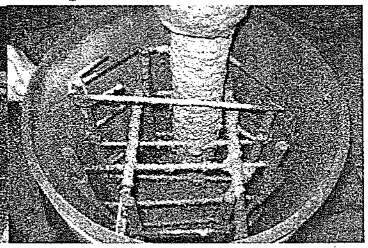




Fig. 3. Composite deck *Fig. 4.* Composite beam *Fig. 5.* Composite column casting

After assessing the seismic performance [5,7], structural fire behaviour of the frame was evaluated as described in the following sections of this work.

2 STRUCTURAL FIRE PERFORMANCE OF THE COMPOSITE FRAME

The structural fire behaviour of the frame was evaluated by making use of a two step analysis. The first step involved the prediction of temperature distribution inside the structural members (thermal analysis), while the second step had the main purpose of determining the response of the structure under both gravity loads and thermal actions (structural analysis). In both phases the special purpose programme SAFIR [8] was adopted.
The variation of thermal and mechanical material properties at elevated temperatures was taken into account according to the constitutive laws given in Part 1-2 of Eurocode 2 and Eurocode 3.
In thermo-mechanical analyses static loads were applied in order to comply with dispositions given in Eurocode1 Part1-2 assuming fire limit state design; the adopted combination coefficients were related to category B, corresponding to office buildings.

2.1 Thermal analysis of composite cross-sections

When performing thermal analyses, the ISO 834 Standard fire curve was adopted, being an internationally recognized standard fire, even if it is known [9] that its use may present many shortcomings; besides, no cooling effect was considered. For beam elements, a three sides exposure was adopted while column elements were considered uniformly heated.
The distribution of temperature at the midline centre of each cross-section is shown in Figs.6 and 7.

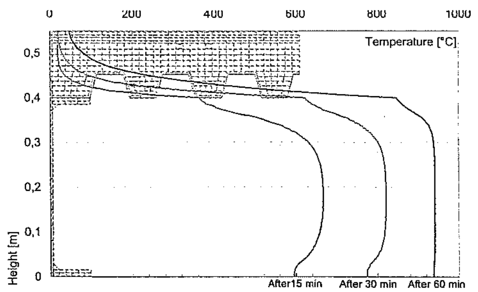


Fig. 6. Thermal analysis - composite beam

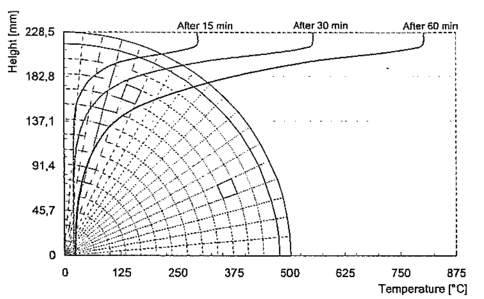


Fig. 7. Thermal analysis - composite column

The composite floor undergoes high thermal gradients due to the relative thermal properties of steel and concrete; anyway, while in the early stages temperature in the concrete slab is a lot lower than in steel and almost uniform over the thickness, in the later stages increasing thermal gradients develop in the slab and temperature in steel reaches a uniform distribution, approaching gas temperature.

In the composite column, temperature of the steel tube is uniform over the thickness reaching high values since the early stages of fire; conversely, the inner concrete core maintains values of temperature under 250° through all the duration of the fire.

Thermal analysis was used to define the position of steel re-bars both in the slab and in the column by making sure that they didn't reach temperatures higher than 400°C at least for 30 minutes.

2.2 Thermo-mechanical analysis of the composite frame

The composite cross-sections analysed in previous section were then used to make up the planar frame model adopted for mechanical analysis. The whole structure was modelled with 180 beam elements; three kinds of cross-sections were adopted for beams depending on the effective width of concrete slab as defined in Eurocode 4.1-1; one cross section's type was used for columns.

The schematic of the planar frame and applied boundary conditions are shown in Fig. 8.

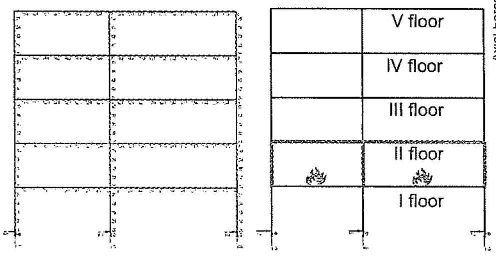


Fig. 8. Planar model and boundary conditions

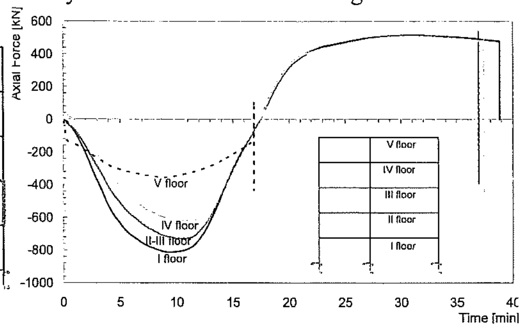


Fig. 9. Axial force at different floor levels

To choose the most severe fire scenario, preliminary analysis were developed applying the heating curve at each floor level, separately; results are shown in Fig.9 for axial force in longest span beam. Heating higher floor levels induced smaller values of the peak compressive force, except for the beams placed at 2nd and 3rd floor levels whose behaviour was characterized by the same curve. In each case, the zero value of axial force was attained for the same time of fire exposure and the same amount of tensile axial force was induced; moreover, changing the position of the heated beam element seemed not to have a significant influence on fire resistance rating. Beams placed at last floor level, made an exception to the observed behaviour since the amount of restraint offered by the column elements seemed inadequate to prevent the beams' failure due to run-away deflections.

Fire acting at the 2nd floor level was chosen as reference study case, and analysis results focused on the longest span beam, presenting the higher vertical displacements through the duration of the fire. Three stages were identified in its behaviour, namely: restrained thermal elongation characterized by increasing axial compressive forces, thermal bowing meaning thermal curvature inducing high displacements at mid-span and catenary action characterized by a cable-like behaviour in which loads are carried by tensile axial force. The observed behaviour is summarized in Figs. 10 and 11.

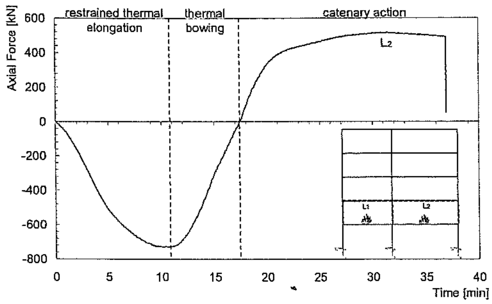


Fig. 10. Evolution of axial force

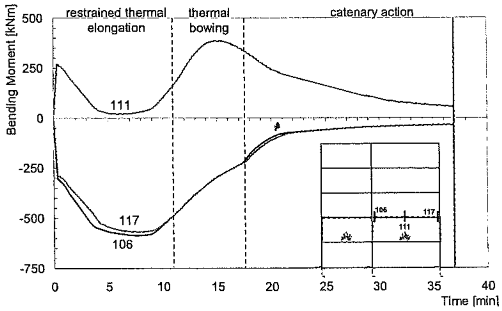


Fig. 11. Evolution of bending moment

Analysis results were evaluated with reference to a suitable criterion making sure that the end of simulations corresponded to some physical phenomena and not to numerical instabilities.
To this purpose, in order to identify the formation of elementary mechanism the state of stress at the three critical cross-sections was investigated; in particular, stress in the steel flanges was compared with both the thermally reduced Eurocode 3 (Part 1-2) proportional and yield limit stresses.
The evolution of stresses for bottom and top flange is shown in Figs. 12 and 13.

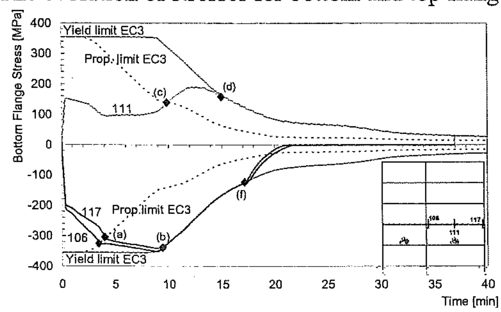


Fig. 12. Stress in the bottom flange

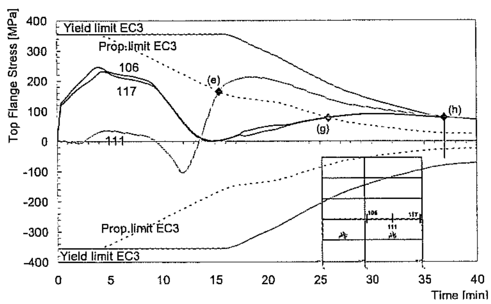


Fig. 13. Stress in the top flange

The bottom flange stress at supports quickly increases in compression, while the top flange starts out in tension from cold meaning that the beam at supports is initially subjected to large bending moments resulting both from restrained rotation and thermally induced P-δ effects. The tensile stress in the bottom flange decreases at mid-span while the compressive stress at support reaches the proportional limit (a) and when the bottom flange yields in compression (b), the proportional limit in tension at mid-span is achieved (c). Afterwards, the bottom flange at supports follows the yielding branch in compression (from (b) to (f)) while at mid-span it yields in tension (d), this causing the top flange reaching the tensile proportional limit almost at the same time (e). When the compressed bottom flange at supports leaves the yielding branch (f) to unload, the third load-bearing mechanism (catenary action) begins and from now on the steel beam at the three critical cross-sections is characterized by tensile stresses. Later, top flange at supports reaches the proportional limit in tension (g) and finally the numerical simulation stops when the top flange reaches the same value of the thermally reduced tensile yield limit at the three critical cross-sections simultaneously (h).

COMPARISON WITH ISOLATED BEAMS UNDER IDEAL SUPPORT CONDITIONS

n order to assess the influence of restraining conditions on the behaviour of heated elements, four solated beams under ideal support conditions were analyzed including the following cases: pin-oller (P-R), pin-pin (P-P), fixed-slide (F-S) and fixed-fixed (F-F). Results were compared with the ame curves obtained in previous section for F-B case, as shown in Figs. 14 and 15.

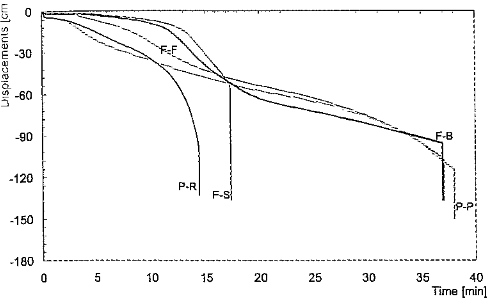


Fig. 14. Vertical displacement at mid-span

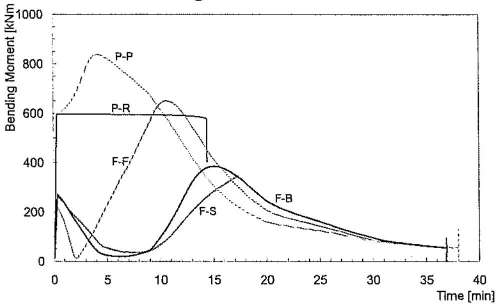


Fig. 15. Bending moment at mid-span

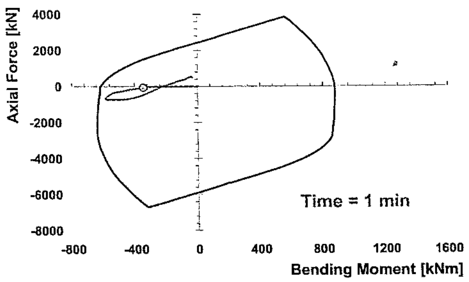
n absence of axial restraint, (P-R and F-S) the achieved fire resistance rating is much lower since he beam fails because of uncontrolled vertical displacements (run-away failure). Conversely, the resence of rotational restraint had much less influence on fire resistance rating, while reducing oth vertical displacements and bending moment at mid-span, at least in early stages (F-S and F-F). Bending moment was almost constant in the P-R case since no additional P-δ moment was induced s happened in the P-P case where axial compressive forces add to sagging bending; evolution of ending moment in F-S and F-F cases showed the influence of rotational restraint. In fact, after eaching the load imposed value, bending moment starts to decrease because of the higher amount f bending moment induced at supports until the formation of plastic hinges; afterwards, bending noment increases again at mid-span then two situations occur: run-away failure for F-S case and atenary action for F-F case.

The comparison with the framed beam (F-B) showed that its behaviour resulted half-way between F-S and F-F ideal cases; in fact, in the early stages it was similar to the F-S case but when this beam ails because of run-away deflections, catenary action in the framed beam begins and its behaviour pproaches the F-F case achieving the same fire resistance rating.

INTERACTION CURVES AT ELEVATED TEMPERATURES

Results from previous sections showed the importance of assessing realistic restraining conditions or heated elements in a structure, since this parameter can have a great influence on the observed ehaviour in fire. In particular, the importance of considering the presence of axial restraint was utlined since its presence improves fire resistance rating. Therefore, including axial force in fire lesign of composite beams could be a fundamental issue; unfortunately, simple calculation models rovided by Eurocode 4.1-2, Hence, to cope with the necessity of adopting easy to use design tools s a practical means of assessing fire resistance of structural elements in common design practice vithout accepting gross approximations and without being forced to adopt advanced calculation ools, some alternative design procedures are needed.

A firs step in this direction was made by developing interaction curves at elevated temperatures [5]; n doing so, account was taken of both the variation of mechanical material properties with emperature and the effective temperature distribution in cross-sections. By making use of such urves, the load bearing capacity in fire of critical cross-sections in the analyzed frame was valuated and compared with the effects of actions coming from global analysis. The obtained urves for section 106 together with the evolution of internal actions are shown in Figs. 16 and 17. Before the fire, the domain had its maximum dimensions; the point inside the frontier represents the ictual value of internal actions while the solid line represents all the possible M-N couples.



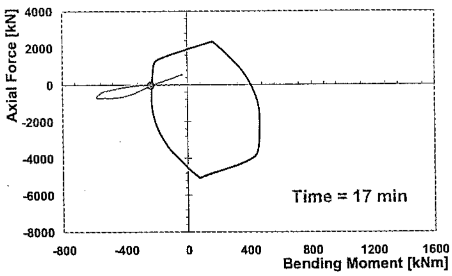


Fig. 16. Interaction curve at support after 1 min *Fig. 17.* Interaction curve at support after 17 min

With increasing time of fire exposure, the dimensions of domain reduce while grossly maintaining the original shape, even if the reduction of bending moment is higher than the reduction of axial force. Correspondingly, the point representing the actual M-N couple moves along the solid line; when the point belongs to the frontier, it means that the cross-section has reached its load-bearing capacity in fire (see Fig. 17). The present check represented an alternative way of assessing the capacity of a cross-section in fire accounting for the interaction of axial force and bending moment.

5 SUMMARY AND ACKNOWLEDGMENT

The designed frame showed a really satisfactory behaviour in fire, achieving a fire resistance rating of almost 40 minutes even if unprotected. For the time being, further research is going on aiming at the assessment of behaviour of composite connections in fire considering their importance especially when dealing with structures in seismic-prone areas.

Results presented in this work were obtained in the framework of the following European research project: RFCS Steel RTD Programme, Contract n. RFSR-CR-03034 [4]. Nevertheless the opinions expressed in this paper are those of the writers and do not necessarily reflect those of the sponsors.

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