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Some Remarks on the Design of Timber Composite Structures Exposed to Fire

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Summary

When a timber-based composite structure is exposed to fire, it is of primary importance to know the changes of the stiffness that the single component elements are subject to, especially due to the decrease in resistant sections. On the basis of some physical tests and numerical simulations carried out, it is shown that the knowledge of the static performance of a timber-based composite structure during fire needs a sufficiently accurate determination of the bending stiffness of beams and slabs, therefore of the thermal decay of timber elements; it is shown that some standards' decay models are unable to accurately predict the behaviour of those composite structures.

1. Foreword

Composite timber structures have been proposed and successfully experimented for a long time, also for strengthening existing timber elements, frequently suffering for various kinds of static deficiencies (material decay, inadequate resistance and stiffness). Among these, the wood-concrete mixed structures are worthy of note for newly built-houses too, since some quality criteria (fire resistance, sound proofing, vibration control) are easily fulfilled by the wood-concrete systems used as floor structures [4, 5]. It is widely accepted that the distribution of stresses between the component elements is strongly influenced by the stiffness (flexural and axial) of the component elements themselves and of the connection systems [6]. When the structure is exposed to fire, it is of primary importance to know the changes of the stiffness that the single component elements are subject to, due to a decrease in resistant sections (timber) or to variations in the mechanical parameter values which characterise them (timber, concrete, connector). This information is required in order to be able to set prediction mathematical models for the behaviour of the composite timber structures exposed to fire and particularly to determine their resistance to fire, relative to the load bearing capacity.

Significant results, coming from some tests in furnaces on full-size models of timber composite floors, are discussed and some proposals are given regarding both the simplified numerical models used to provide an adequate simulation of the real behaviour of those structures exposed to fire and the design criteria to improve their resistance under fire. To this end, *insulation* and *integrity* requirements, which have to be guaranteed by the structure in accordance with the current standards, should not be forgotten. Also on the basis of those physical tests and the numerical simulations carried out, it is clearly shown that the knowledge of the composite structure static performance during a fire needs a sufficiently accurate determination of the flexural behaviour of beams and slabs and, therefore, of the thermal decay of timber elements. It is also evident that some decay models proposed by standards are unable to accurately predict the behaviour of such timber-based composite structures.

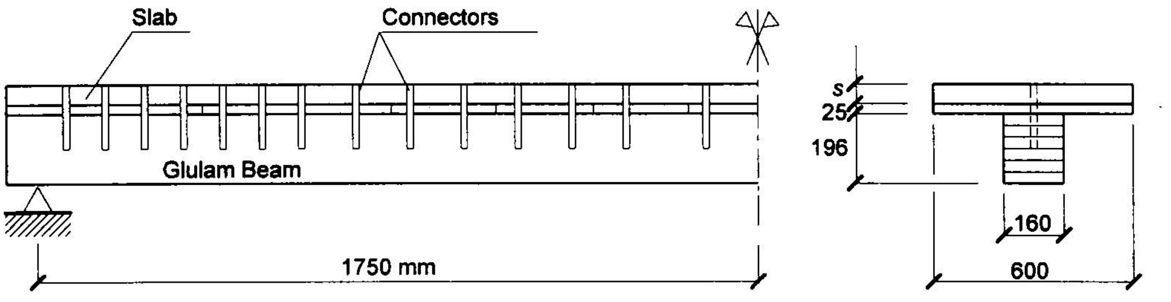

Fig. 1 - Full size samples of the tested timber-based composite structure.

2. Experimental Part

2.1 Test Samples

Tests were carried out on four timber-based composite structures, made with glulam timber beams of spruce (Picea abies Karst.), from the same production lot, with a cross-section of 160 x 196 mm . Such a choice was compulsory in order to achieve consistent data from the point of view of thermal decay. Elastic modulus characterization tests were, of course, carried out on each single beam as well as on the assembled composite structures, prior to the physical tests inside the furnace. These tests made also it possible to accurately check at room temperature the numerical model used for the structure's static analysis. The length of each structure is equal to ≈3.50 m, the centre distance between timber beams of the floor (slab width of the specimen) is equal to 0.60 m (Fig. 1). All samples were assembled by means of glued steel connectors [7], except the S3 specimen for which screws* (inserted without pre-drilling) were used. Some single timber beams(1) were also tested for comparison purpose (Table I).

Model Id.	Slab		Connectors		Test Load P_{sl}	
	Type	Thickness s (mm)	Ø (mm)	Pitch (mm)	P_{sl} (kN)	q_{sl} (kN·m^{-2})
S1	Concrete	50	14	140÷220	13.66	13.01
S2	Laminated Wood Panel	48	14	100÷200	12.53	11.93
S3	Laminated Wood Panel	48*	4*	30÷90*	9.15	8.71
S4	Plywood Board	80	14	100÷200	12.55	11.96
Beam(1)	——	—	—	—	4.20	3.96

Table I - Full size models of different timber-based composite structures.

2.2 Testing Equipment

The tests were performed inside a furnace normally used for fire-resistance tests on elements to be exposed vertically to fire, especially equipped for performing fire-resistance tests also on load-bearing elements normally installed horizontally. The furnace consists mainly of a combustion chamber 3.00 m wide by 3.00 m high by 1.00 m in depth; the entry opening is closed-off by means of a special wall, consisting of a strong outer frame filled in with heat resistant bricks, 150 mm thick, according to the elements to be assembled and tested. The wall is assembled on a trolley so the furnace can be opened quickly at the end of the test and the fire on the tested element can be extinguished and stopped with a jet of water or other means. Heating of the furnace, depending on the selected heating programme, is accomplished using 9 natural gas radiant burners, with flames no longer than 20 cm, arranged on the bottom wall, in front of the element to be tested. The instruments located outside the furnace include a temperature programmer, a pressure regulator that operates the flue gate, a temperature recorder with a 0÷1300 °C scale, a computer for the acquisition and visualisation of all temperature, pressure and displacement data.

2.3 Testing Procedures

Each element to be tested is assembled diagonally inside the furnace, in order to exploit the maximum possible length (l=3.50 m), with one side against the closing wall, so that only three sides are exposed to fire. The supporting system comprises two counter plates tightly fitted to both ends with a threaded bar, bolted to the external counter frame and adequately insulated. The element is loaded in the centre line by means of a special transmission pin that goes through the wall and connects to an external lever mechanism fixed to the counter frame.

2.3.1 Thermal action and duration of tests
The furnace was heated according to the heating programme of the international ISO 834 (1975) standard: $T\text{-}T_o= 345\cdot\log_{10}(8\cdot t+1)$, where T is the temperature and t the time (minutes). The fire tests were extended for about 60 minutes, therefore permitting investigations on the timber floor elements after fire (the so-called "residual" structures).

2.3.2 Load conditions
The proposed structures have been designed on the basis of some numerical analyses (i.e. by the F.E. model described in [5, 6, 3]); it was imposed, for all the timber beams under real test condition, a value equal to ≈5.0 MPa for the maximum stress at the beam section's lower fibre. This initial stress state could be assumed as probably being present in correspondence with a fire event. Table I gives data relative to the distributed loads q_{sl} equivalent (for the value of maximum bending moment) to the concentrated loads P_{sl} applied during the fire tests inside the furnace. It is worthy of note that all the timber-based composite structures with glued steel connectors exhibit load values ≈3 times the one shown by the single timber beam. According to current simplified calculation criteria [2] and with a charring rate β_0 =0.7 mm/min, the same "residual" beams, after 60 minutes exposure to fire, present maximum stress values that are still feasible (about 13 MPa). It is to be noted that, for the single timber beam, the stress value of 13 MPa (after exposure to fire) determined the load value constant during the fire test.

3. Experimental Results and Numerical Models

The physical data collected during each fire test included temperature data (inside the furnace, on the slab to quantify the degree of insulation provided by each floor, in the connectors to see that the temperature threshold, at which point the glue softens, was not exceeded and inside the timber beam at different depths) and floor displacements (see [3]): this was done not only for checking the mathematical models, but also for comparing similar models with those proposed by current standards, in particular ENV 1995 [1] and Italian U.N.I. 9504 [2]. Due to lack of space, only the diagrams of maximum temperatures read on the top surface of the slabs are reported in Fig. 2, as a function of time (in seconds). At a first glance, it is immediately evident how the floor comprising a concrete slab (50 mm thick) and wooden boards (25 mm) is also able to guarantee the insulation requirement with sufficient safety for 60 minutes exposure to fire (T_{max}=105 °C). The same result could be obtained by the concrete slab alone, but with a ≈100 mm thickness.

Numerical modelling of composite floors' behaviour to fire, like those referred to, must principally concerns two aspects: modelling the structure's static behaviour and modelling the section's thermal decay process. As far as the first aspect is concerned, direct reference is made to the flexible connection composite structures' analysis model, already presented on other occasions [6]: remember that the beam and the slab (and also, generally speaking, the connection itself) vary during exposure to fire (geometry or mechanical characteristics). As far as concerns thermal decay models of timber section during a fire, three models are here presented, of which two from standards and one proposed by the authors. The first model is the one proposed as the "effective cross-section method" by ENV 1995 [1], the other is the one proposed by current Italian standard [2] referring only to the charring process; for both these models β_0 =0.7 mm/min for softwood glulam. Research up till now has made it possible to highlight the decay phenomena of the wood's mechanical characteristics in a layer under the charred surface, due to the high temperatures present (>100 °C). Such phenomena, partly responsible for the decrease of stiffness beyond the theoretical value achieved on a standards basis, can still be found on residual specimens under post-fire conditions.

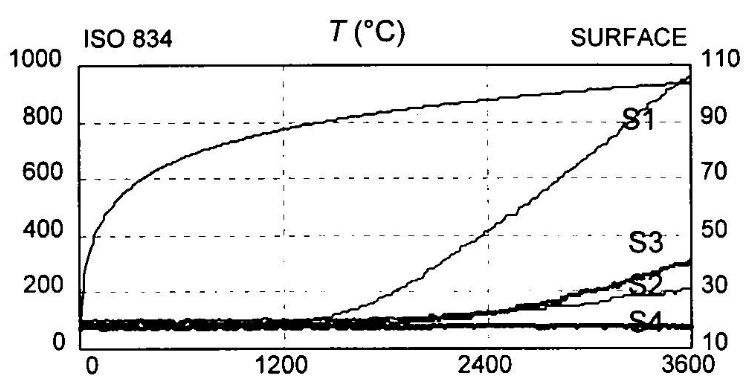

Fig. 2 - Maximum temperatures at the unexposed floor surfaces.

In fact, the behaviour of a "residual" timber structure (once the charred layer has been removed completely down to the bare wood) still does not correspond to the assumed initial mechanical parameters for the whole residual section. On the other hand, the displacement values present at the end of a furnace test, differ from those found for the residual structure, but tested at room temperature. As regards the so-called irreversible decay phenomenon, it should be investigated further still also, and above all, at a microstructure level. However, from an engineering point of view and assuming that the charring rate values proposed by various standards are valid, it can be assumed to be limited to a layer immediately beneath the charred one. The results of characterisation mechanical tests run up to now on residual elements (both single and composite), appear to confirm the values ranging from 2 to 5 mm in thickness of highly decayed material, under the charred layer.

The simplified cross-section of timber beam for the "engineering" type decay model, used to simulate the physical tests that have been carried out up till now, could possibly be that proposed in Fig. 3. The following variables are given in the same figure:

- $d_{ef} = d_{char} + d_{add}$, where $d_{add} = d_r$ (during fire) or $d_{add} = d_i$ (after fire, at room temperature);
- d_i = thickness of the layer subject only to irreversible decay phenomena;
- d_r = thickness of the layer subject to irreversible and reversible decay phenomena.

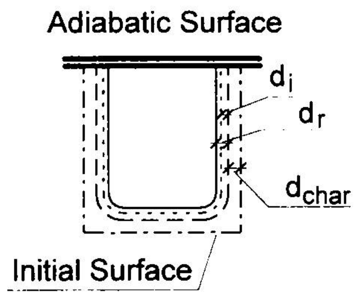

Fig. 3 - The "effective cross-section" method [1], applied also for the residual timber section.

The actual values assumed by the quantities given, depend on the several variables involved and on the dimensions of the timber beam itself (or on the *S/V* ratio between fire-exposed surface and volume of heated material). This highly simplified model could be of immediate use, having to remove from the standard's charred layer another layer, which has a constant value at least from a certain time of exposure to fire (≈ 20÷30 minutes). The bending stiffness parameter ($E{\cdot}J$) is, therefore, found starting from a fictitious notional section but the modulus of elasticity E is constant (equal to the initial value at room temperature). In this investigation, such a model has been set on the basis of some fire tests on single timber beams; it was later verified also on different types of composite structures that were being studied. The excellent behaviour of the simple decay model described above, can be seen clearly in Fig. 4, where curves ***a*** (according to [2]), curves ***b*** (according to the "effective cross section" method of ref. [1]) and, lastly, curves ***c*** (according to the decay model of Fig. 3) are compared with experimental curves ***Exp***. As it can be seen, the decay models currently proposed by standards and based only upon the assumption of a charring rate [2] are unable to predict the behaviour of a composite structure during a fire.

It can be of a certain interest to analyse the static behaviour of the composite structure with reference to the values of a parameter γ suitable to synthetically describe the static efficiency of the connection [7]. The parameter γ_{sl}, to which it seems appropriate the name *efficiency* with regard to serviceability limit states, can be expressed by means of the following equation:

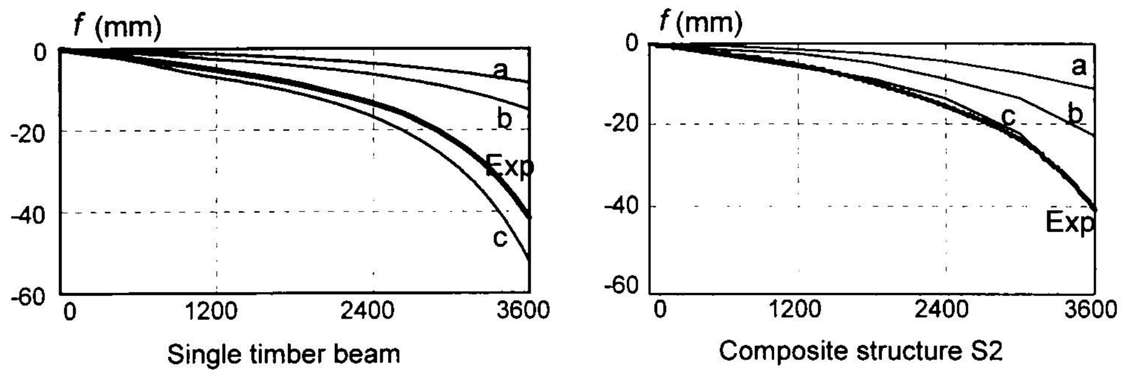

Fig. 4 - Diagram's deflection vs. time (seconds).

$$\gamma_{sl} = \frac{(E \cdot J)^* - (E \cdot J)_b}{(E \cdot J)_\infty - (E \cdot J)_b} \;. \tag{1}$$

The parameters ($E{\cdot}J$) in the preceding formula are the bending stiffness of the beams simply in parallel (suffix b), of the composite beams with rigid connection (suffix ∞) or with the actual connection (apex *) at the same service load. The limit values of γ are recognisable for the value γ=0 (connection with no stiffness) and for γ=1 (rigid connection). It is to be noted that the values of the parameter γ are to be referred to a particular loading arrangement and to a specific load value. If the real displacement values are analysed for the composite structures under investigation before and after the fire test, without variation of loads, the diagram in Fig. 5 can be obtained. The *efficiency* of the connection at service load γ_{sl} has increased for all structures and this is a direct consequence of the connection behaviour, nearly unchanged during the fire event as imposed at the design phase, and of the simultaneous remarkable decay of timber beams.

Fig. 5 - Values of the parameter γ_{sl} before and after the fire event.

As regards the behaviour of the so-called *residual* structure after fire, Table II shows the values of the equivalent distributed loads q_u at failure, determined by means of mechanical tests at room temperature under the same loading arrangement, compared with the values q_{sl} kept constant during fire test. The values of q_u demonstrate that the problem of evaluating the structure's residual resistance subsequent to a non-destructive fire event is of fundamental importance especially for timber-based composite structures.

With reference to the design phase and to the material partial safety factor imposed for the bending strength of timber element after fire, it can be noted that only the composite structures with high values of the initial static efficiency (S1, S2, S4) are able to exhibit values of safety factor for the whole structure greater than the initial one. This is a consequence of the material's anisotropy characteristics and the random defects inside the wooden mass: the decay due to the thermal decomposition process shows continuously new "defects" of the timber elements and consequently it is more than likely that the composite structure S3 (with lower value of the initial static efficiency) exhibits the smallest value of the ratio q_u/q_{sl}.

	Timber-based composite structures	q_{sl} ($kN \cdot m^{-2}$)	q_u ($kN \cdot m^{-2}$)	q_u/q_{sl}
S1	Concrete slab (glued steel connectors)	13.01	33.33	2.56
S2	Laminated wood panel slab (glued steel connectors)	11.93	27.66	2.32
S3	Laminated wood panel slab (screw connectors)	8.71	15.13	1.74
S4	Plywood boards slab (glued steel connectors)	11.96	26.54	2.22
Beam(1)	Single timber beam	3.96	10.50	2.65

Table II - The static performance of the residual timber composite structures after fire.

Worthy of another mention is the problem related to the determination of the ultimate limit state of glued connections due to heat, when the glue is sensitive to high temperatures. This problem is quite easily resolved at the design stage: all that is necessary is to guarantee an adequate protection for the glue beneath the charred layer. The thickness of wood required for the glue used here (with a "softening" temperature of ≈80 °C) has given a value ranging from 20 to 25 mm.

4. Concluding Remarks

Timber-based composite structures are generally very sensitive to the bending and axial stiffness of the component elements as well as to the stiffness (due to shear force) of the connection. Consequently, if the structure is exposed to fire, it is essential to have a sufficiently accurate knowledge of the stiffness variations that the single elements are subject to, in order to set prediction models for the composite structure's behaviour and, especially, to determine its resistance to fire. The physical tests performed up till now reveal that actual standard's decay models are not adequate in order to accurately define the bending stiffness of timber elements during fire and, therefore, the behaviour of the whole composite structure.

Acknowledgements

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