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Fatigue behaviour of composite timber-concrete beams

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Abstract

Refurbishment of existing buildings often claims for strengthening and stiffening of timber floors. To avoid too heavy interventions, this need is particularly relevant in seismic zones and/or for historical buildings, not only to preserve historical value, but also to contain the masses. A solution commonly adopted is to substitute the screed with a thin reinforced concrete or lightweight reinforced concrete slab duly connected to the timber beams, in such a way that a composite timber-concrete floor is obtained, granting also a sufficient rigidity in the horizontal plane. Moreover, this solution has also the advantage to improve the acoustic performance.
Of course, the behavior of the composite structure depends on the rigidity of the shear connections. Since several type of shear connectors are available, the experimental assessment of its static and fatigue behavior is a prerequisite for a suitable design of the intervention. Aiming to compare their performances, an ad hoc experimental study has been carried out on three different types of shear connectors.
The fatigue tests have been performed on a composite wood-concrete beam. During each test, 15000 loading-unloading cycles have been applied, recording the deformations and the relative slip. After completion of the load cycles, static load has been applied till to collapse.
In the paper, the experimental tests and results are widely discussed, also in comparison with commonly used theoretical models, and relevant conclusions are drawn

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Keywords: Seismic vulnerability; Masonry buildings; Seismic resistance; Seismic risk index; Pushover methods.

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1. Introduction

In strengthening or repair of historical building the problem of preserving the architectural heritage is increasingly significant. This need, avoiding too heavy interventions, is particularly relevant in seismic zones, contributing to contain the masses. When the floors sustain painted or decorated ceilings, refurbishment of historical buildings, imposes strengthening and stiffening of existing timber floors to avoid their replacement; a typical solution consists in strengthening the deck with a reinforced concrete slab, so guaranteeing better distribution of vertical loads, but also increasing its stiffness and its acoustic performance. This solution, simple and suitable for small buildings too, allows also for the levelling of the deck, for the creation of a barrier between the floors in case of a fire, and, last but not least, the creation of horizontal diaphragms that once properly connected to the perimeter walls, give the building better seismic performance, improving the so-called ‘box behavior’. The increase in strength and flexural stiffness of the wooden flooring obtained by using the proposed solution depends on the effectiveness of the connection between the concrete slabs and the wooden beams, which in turn depend on the mechanical behavior of the connectors between the two elements. There are many types of connectors, but those mostly used are: screws, pins, plugs or connector, screwed or embedded in timber. These connectors should be designed to withstand shear stresses minimizing the timber-concrete slip at the interface. Many state-of-the-art studies are available concerning the static behavior of composite timber-concrete flooring: in addition to the theoretical studies, tests have been carried out to evaluate both the mechanical characteristics of different types of connectors and the mechanical characteristics of the reinforced flooring. On the other hand, no significant work has been found concerning studies on the fatigue behavior of these “composite” floorings. Concerns have risen regarding the behavior of the connectors if the flooring is subjected to repeated cycles of load, when the connectors tend to split the wood, thus increasing the relative slip and reducing the effectiveness of the link. These considerations gave rise to this study, which began with the planning of the flooring reinforcement of a typical Tuscan building, by following the highest standards and criteria. An ad hoc experimental test campaign, consisting in static and fatigue tests on real scale specimens, aiming to compare the actual fatigue behavior of three different types of wood-concrete connections is discussed in the present paper, allowing to draw some relevant conclusions.

2. Theoretical models for composite timber-concrete beam behaviour

Since the behavior of timber-concrete composite structures depends on the effective rigidity of the shear connectors, its evaluation is thus a key aspect in the development of theoretical models. Different models can be found in the literature to estimate the stiffness and the resistance of the connection. In Gelfi at al. (2002) a theoretical evaluation of the connection stiffness is reported idealising the stud behaviour as a beam on elastic soil (Patton-Mallory et al. 1997) together with an approximate formulation for the stud stiffness, suitable for design, modelling the connector as an embedded beam of ideal length, leading to a maximum error around 15% in the intervals of practical interest. In Eurocode 5, a linear model based on the work by Newmark et al. (1951) and Möhler (1956) and considering the wood-concrete slip, is recommended for design of mechanically jointed timber beams. This method, presented in the informative Annex B and known as Gamma method, is based on the theory of linear elasticity and provides an approximate solution of the differential equation of the problem. Considering a cosine, or sine, load function instead of a uniformly distributed load, a connection efficiency factor γ (Ceccotti, 2002) is defined, ranging from 0, for no connection, to 1 for fully composite action and rigid connection. The Gamma method offers an elegant solution, suitable for simply supported beams with smeared connections, uniform cross section and uniformly distributed load (Fernandez-Cabo et al. 2013). In the model,

$$J_{eff} = J_0 + \gamma(J_\infty - J_0) \qquad \text{with } J_0 = J_w + \frac{E_c}{E_w} J_c \tag{1}$$

where J_{eff} is the effective moment of inertia of the composite beam, J_0 and J_∞ are the moments of inertia of the section without connection and with rigid connection, respectively, E_w and E_c the elastic moduli of wood and concrete and J_c and J_w the moments of wood and concrete section, respectively and γ the connection efficiency

$$\gamma = \frac{1}{1+\pi^2 \dfrac{E_w (J_\infty - J_0) s}{d_G^2 K_P L^2}} . \tag{2}$$

being L the beam span, d_G the distance between the concrete slab and the wood beam centroids, s the fasteners spacing and K_P the connector stiffness. According to EN1995-1-1, it can be assumed

$$K_P = K_{ser} = 2\rho_w^{1.5} \frac{d}{23} \text{ for SLS} \quad \text{and} \quad K_P = K_u = \frac{2}{3} K_{ser} \text{ for ULS} \tag{3}$$

where ρ_w is the mean density of wood and d the connector diameter. The normal stresses in the composite section subjected to a bending moment M can be then calculated according expressions

$$\sigma_c = -\frac{M}{d_G} \frac{\gamma(J_\infty - J_0)}{J_{eff} A_C} \pm \frac{E_c}{E_w} \frac{J_c}{J_{eff}} \frac{M}{W_C} ; \quad \sigma_w = \frac{M}{d_G} \frac{\gamma(J_\infty - J_0)}{J_{eff} A_w} \pm \frac{J_w}{J_{eff}} \frac{M}{W_w} \tag{4}$$

As suggested by Gelfi and Giurani (2003), starting from the deflection increment Δv at mid span, the maximum wood-concrete slip δ and the connector shear resistance V_P can be calculated as

$$\delta = \frac{\alpha \Delta v J_\infty}{L S_c} \quad ; \quad V_P = K_P \delta = K_P \frac{\alpha \Delta v J_\infty}{L S_c} , \tag{5}$$

while, for common values of the ratio between span and beam depth ($L / h \cong 20$), it can be assumed

$$\delta = 10 \Delta v . \tag{6}$$

3. Experimental test campaign

To evaluate the fatigue behavior of composite timber-concrete floor with different types of connectors, an experimental campaign has been carried out on real scale specimens. The experimental campaign aimed, inter alia, to evaluate the loss of stiffness and resistance of the connection system determined by the increasing of number of cycles. The experimental tests have been carried out on three real scale beams at the Laboratory of the Department of Civil and Industrial Engineering of the University of Pisa. The wooden beams, recovered from the dismantling of an old historical building in Lucca, were consolidated by using different types of connections based on:

- through-threaded bar connectors (Beam A);
- screw connectors (Beam B);
- a patented system, called CTL Maxi connectors (Beam C).

The samples of beams A, B and C, illustrated in Fig. 1, were designed according to geometry of typical flooring in Tuscany, characterized by main beams spacing 1,95 m, secondary framework of rafters spacing 0,29 m, covered with thin handmade bricks, and pavement (in most cases also constituted by terracotta bricks) on screed. The floor reinforcement consisted in a 0,90 m wide and 60 mm thick r.c. slab (concrete class C25/30), reinforced with a welded steel mesh Φ6, 150x150 mm, and 12 mm diameter connectors spacing 150 mm. The threaded rod connectors (Beam A), inserted into 14 mm diameter holes filled with epoxy resin, were embedded in the concrete for 70 mm and tightened with a tightening nut on each side. The connector head inside the concrete slab was obtained by a further nut, placed on the head of the threaded bar. The screw connectors (Beam B) consisted of a threaded part, about 80 mm long, screwed in the timber beam, and a headed non-threaded part, 70 mm long, embedded in the slab. The CTL Maxi connectors (Beam C) consisted in a 70 mm long steel stud, welded on a 50x50x4 mm square steel plate, with corners folded down to reduce the slip, connected to the beam with two 10x110 mm screws.

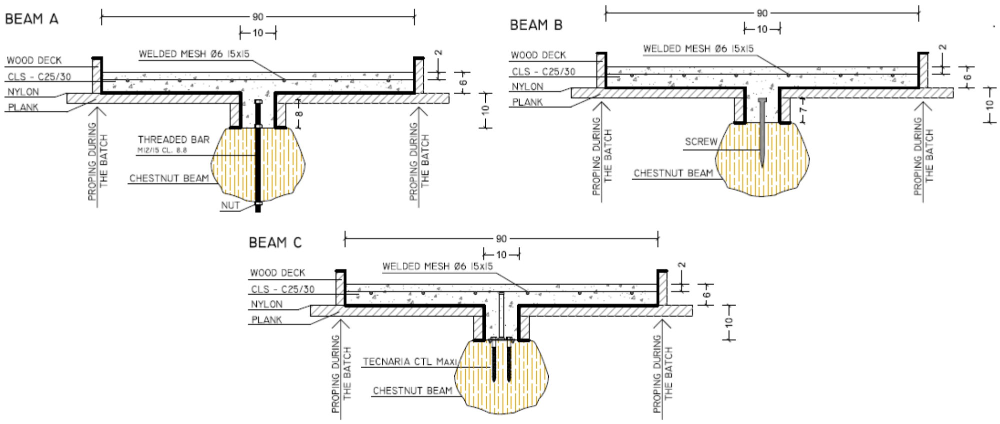

Fig. 1. Composite timber-concrete beam A, B and C.

Before packing the samples, to assess their mechanical characteristics as well as the deterioration degree, the wooden beams were classified according to the relevant Italian standard (UNI, 2004) (see Table 1), so deriving a characteristic bending strength $f_{m,k}$ =27 MPa and a characteristic shear strength $f_{v,k}$ =4 MPa.

Table 1. Beams' properties.

Element	Timber type	Category	Length [mm]	$\Phi_{eq,med}$ [mm]
Beam A	Chestnut	II	4020	235
Beam B	Chestnut	II	4000	220
Beam C	Chestnut	II	4100	230

3.1. Equipment, instrumentation and testing

The test rig consisted in a steel frame (Fig. 2a), where the simply supported beam samples were placed. The timber beam ends, which were supported on cylindrical steel hinges, were inserted into special metal housings, in order to prevent rotation around the longitudinal axis. The load was applied by means of a 450 kN hydraulic jack remotely controlled via a 500kN load cell and transferred to the slab of the sample by means of a load transfer dispositive, consisting of a metal structure having four hinged imprints with dimensions 444x138 mm (Fig. 2a). The test instrumentation equipment and its layout is illustrated in Fig. 2b. Displacements were measured by means of Linear Variable Displacement Transducers (LVDTs):4 LVDTs ($i_1 \div i_4$ in Fig. 2b) allowed to measure the displacement of the beam and of the slab; 5 LVDTs ($i_5 \div i_9$ in Fig. 2b) the vertical displacements; 8 LVDTs ($i_{10} \div i_{17}$ in Fig. 2b) the relative beam-concrete slip and LVDT i_{18} the longitudinal deformation of the intrados of the beam at the mid-span. For the measurement of slab's deformations, a resistive strain gauge (r) was used instead.

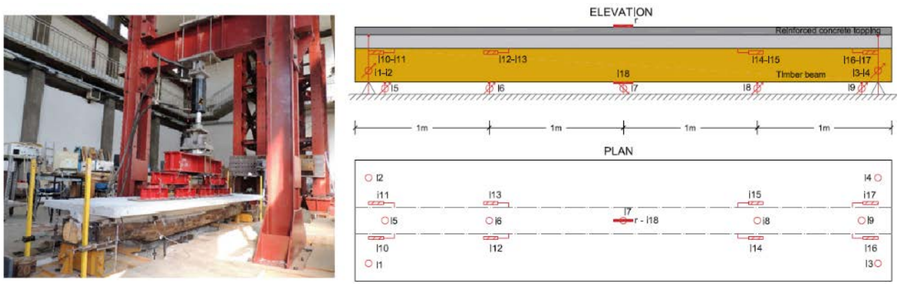

Fig. 2. (a) Lab test; (b) Layout of test equipment.

Test loads, arranged following the scheme in Fig. 3, were determined aiming to reproduce into the beam, at the wood-concrete interface, the shear stresses of the real floor slab, induced at SLS by the characteristic combination, considering imposed load for meeting room/conference (q_{imp}=4 kN/m^2). The equivalences between the two static schemes in terms of maximum shear provide a minimum test load $P_{\min,test} \simeq 20kN$, corresponding to the effect permanent loads, and a maximum test load $P_{\max,test} \simeq 50kN$. The number of loading/unloading cycles to be carried out, 15 000 cycles, was defined in a conventional way, considering a real situation and estimating the actual number of cycles to which the floor under examination would be subjected in a reference period of approximately 30-40 years. The cyclic part of the test was performed controlling the force control, while the cycle to failure (increasing monotonically) was carried out controlling the displacement, detect any softening phenomenon.

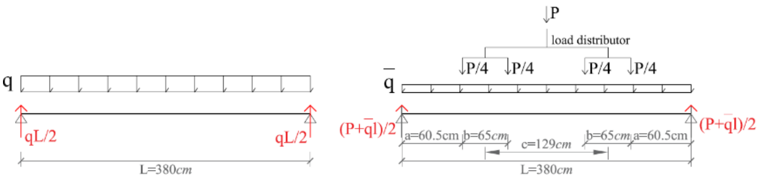

Fig. 3. Load scheme on the real floor and load scheme during the lab tests.

4. Experimental results

4.1. Fatigue tests

The three composite beams were tested in fatigue performing 15 000 cycles in the load range $P_{\min,test}$ - $P_{\max,test}$ previously defined. The test results are shown against the number of cycles in the diagrams of Fig. 4, in terms of:

- vertical displacement at the mid-span of the beam (Fig. 4a);
- slip between concrete slab and wooden beam, evaluated by means of the transducers i_{10}, i_{11}, i_{16} and i_{17} located at the end of the beams (Fig. 4b);
- deformation of concrete slabs at the extrados (Fig. 4c);

where curves in red refer to Beam A, curves in blue to Beam B and curves in black to Beam C.

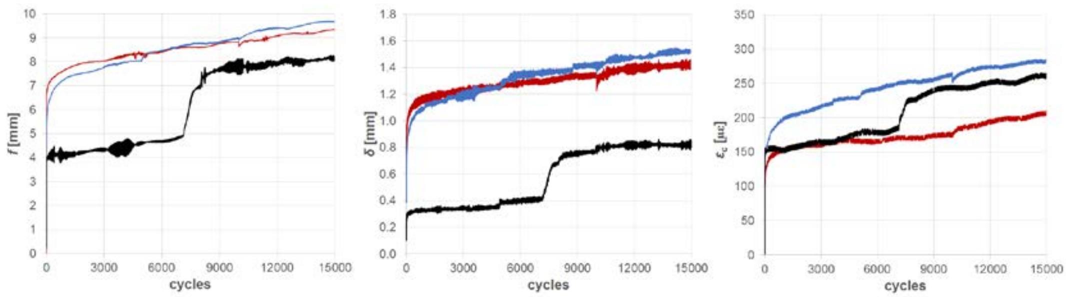

Fig. 4. (a) deflection-number of cycles diagram curves; (b) slip-number of cycles curves; (c) concrete strain- number of cycles curves.

As expected, fatigue influences the behavior of the composite beams leading to progressive degradation of the connection as shown by the increase of the deformation with the number of cycles. Indeed, it can be observed that:

- the vertical displacement significantly increases with the number of cycles: after 15 000 cycles there is an increase of 50-60% in beams A and B, and of about 100% in beam C, with reference to the initial values;
- the mean slip at the end of the beam considerably increases with the number of cycles, attaining for all the tested beams values bigger than 100% in comparison with the initial ones;
- also the mean deformation of the concrete slabs at the extrados significantly increases with the number of cycles, attaining increments of about 100% with reference to the initial values.

Moreover, it can be easily noticed from the diagram in Fig. 4 that beam A and B show similar fatigue behavior. In effect, the considered deformation parameters are similar and increase nearly constantly with time, while beam C exhibits lower deformation parameters, except for the concrete strain, that present an abrupt increase after around 7000-7500 cycles. The load-deflection and load-wood-concrete slip diagrams are reported in Figures 5a and 5b, respectively, for the three beams in virgin conditions before fatigue tests (dotted lines), and after 15000 cycles (solid lines).

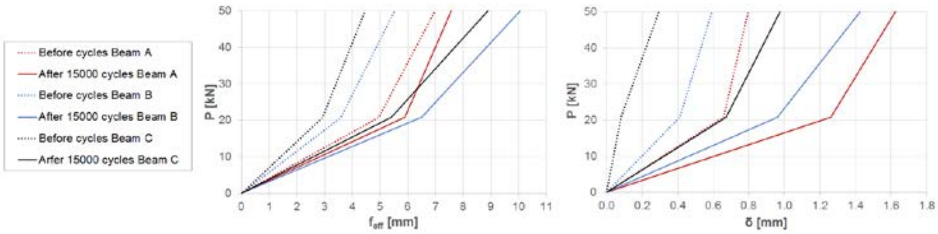

Fig. 5. (a) load-slip diagrams; (b) load-concrete strain diagrams in virgin conditions (dotted lines) and after fatigue tests (solid lines).

4.2. Monotonic test till collapse

After the 15 000 cycles fatigue test, a monotonic static test until collapse was performed on the three beams, controlling the displacement. Test results are summarized in Fig. 6 in terms of load-deflection (a), load-slip (b) and load-concrete deformation (c) curves (Beam A in red, Beam B in blue and Beam C in black).

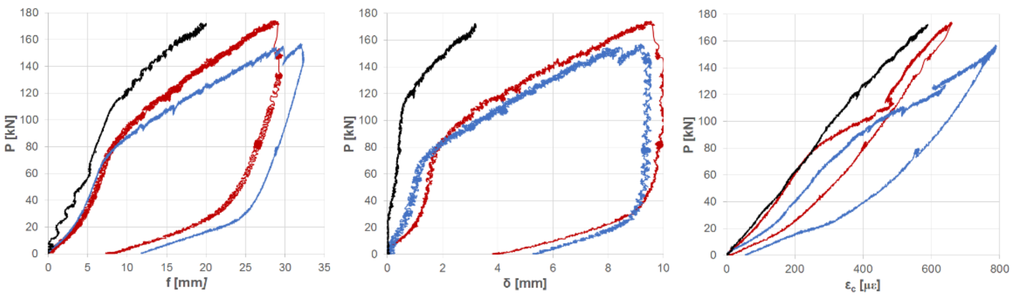

Fig. 6. (a) load-deflection diagram; (b) load-slip diagram; (c) load-concrete strain diagram.

From the analysis of the diagrams of Fig. 6, it can be observed that:

- until collapse, beams A and B show a similar behavior characterized by high deformations, while beam C, although presenting a higher initial stiffness, is characterized by a lower level of plastic deformations;
- ultimate load is around 160 kN-170 kN for all the three investigated beams;
- after an initial phase, for loads bigger of 25-30 kN, beams A and B exhibit increasing stiffness, that decreases sensibly when the load reaches 65-70 kN, to remain nearly constant until the collapse;
- in the load range 0-30 kN, beam C is characterized by very high stiffness, which slightly reduces in the range 30-120 kN; for loads bigger than 120 kN the stiffness sensibly decreases, remaining nearly constant until collapse;
- comparison in terms of maximum displacement and slip confirms that beam C is stiffer than beams A and B: being vertical deflection and slip of beam C around 2/3 and 1/3, respectively, of those measured on beams A e B.

Finally we can also notice the different collapse mechanism of the three connections systems (Fig. 7). In beams A and B, first the stresses transferred by the stems of the connectors lead to the upset of the holes, subsequently, there is plasticization of the connectors which bend, showing huge deformations. On the contrary, in beam C, due to the presence of the metal plate and “crampons” embedded in the timber beam, a satisfactory behavior of the connection system is guaranteed, characterized by higher stiffness and smaller slip between concrete slab and timber beam.

Fig. 7. Shear connectors after the test: (a) beam A, (b) beam B and (c) beam C.

5. Theoretical experimental comparison

The experimental values in terms of deflection, timber-concrete's slips and concrete strain, have been also compared with the theoretical ones, evaluated by means of Möhler theory presented in §2. To simplify the calculations, the beams have beam assumed having a circular cross section with a constant diameter of 235 mm, nearly corresponding to average area of the real beams, and the geometrical and static characteristics of timber-concrete section have been derived accordingly. Stresses in the composite beams were derived from eqn. (4) and maximum slips were assessed both using eqn. (5) and simplified formula (6). The stiffness of the connections was evaluated using eqn. (3) for through-threaded bar connectors (beam A) and screw connectors (beam B), while for the CTL Maxi connectors expressions given by the producer, based on laboratory tests, were used (Tecnaria, 2012).

The most significant diagrams, shown in Fig. 8, allow to underline that:

- the theoretical load - deflection diagram (Fig. 8a) for beam C is substantially equivalent to that obtained during the laboratory tests, while experimental deflections of beams A and B are greater than predicted;
- the theoretical load - strain graphs (Fig. 8b) are roughly an interpolation of the three experimental tests carried out on the beams A, B and C, but in any case not significantly different than expected;
- the theoretical load - slip diagrams (Fig. 8c) are slightly different from those obtained experimentally on safe side in the case of the beam C and on unsafe side for beams A and B, for which the effective stiffness of the connections is lower than theoretically assessed.

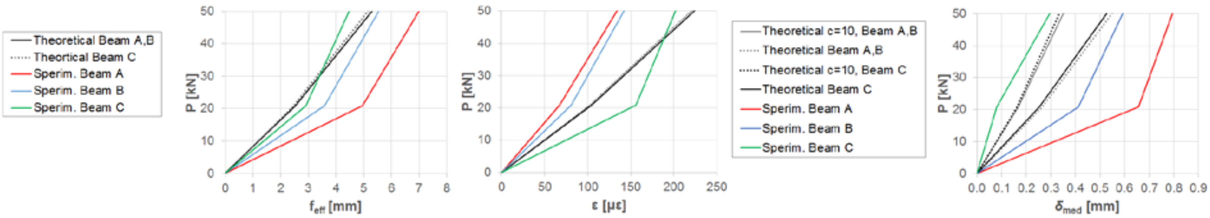

Fig. 8. (a) load-deflection diagram; (b) load-concrete strain diagram; (c) load-slip diagram.

It is also interesting to compare the bending moment in the wooden beam at the collapse, $M_{w,u,sp}$, with the bending strength of the wooden beam alone, $M_{w,u}$, based on the already mentioned characterization according to the UNI 11119 (UNI, 2004). In the calculations $M_{w,u,sp}$, has been derived under the simplifying assumption that the total bending moment is provided by the beam and the slab, according to their stiffness characteristics. A theoretical bending strength $M_{w,u}$=12.5 kNm is obtained while the experimental one is $M_{w,u,sp}$ =18.3 kNm. It appears clearly how the actual bending strength of the beam is higher than the estimated one. Finally, the experimental ultimate bending moment $M_{u,sp}$=97 kNm is compared with the theoretical bending strength determined according the Gamma method presented in §2, $M_{u,t}$=102 kNm. It is pointed out that the comparison shows that the experimental value differs from the theoretical by about 5%.

6. Conclusions

The main aim of the present work has been the assessment of the behavior of wooden beams reinforced with concrete slabs. The study of such kind of composite structure plays a central role in the recovery of existing wooden flooring, considering that this flooring is extremely widespread in ancient buildings and very often lacking in terms of strength or, even more frequently, in terms of stiffness. Types of connectors most commonly used have been examined and their fatigue behavior has been studied. In particular, three real scale beam samples reinforced with a composite slab were tested; the wooden beams, recovered from a historical building in Lucca, were reinforced with a solid concrete slab by means of three different types of connectors: threaded through rods, screws, and CTL MAXI connectors. Fatigue tests consisting of 15000 loading/unloading cycles were carried out on the samples, in order to simulate the behavior of the reinforced beams during repeated loading over a significant time period (30-40 years). At the end of the fatigue test, each sample was tested monotonically controlling the displacement till to collapse. The study confirmed that this kind of concrete-wood composite beams is sensitive to loading/unloading cycles which cause significant increases in the timber-concrete slips, and consequently in deflection and deformation of the two materials. This may be regarded as fatigue effect, which appears to be far from negligible (considering this type of structure), and was manifested through the progressive damage of the system as a whole.

With regard to fatigue degradation and the effects on the safety of the structure due to load cycles, it should be noted that range and number of cycles to which the samples were subjected were extremely severe. In fact, from one side it should be considered from one side that 15000 cycles correspond to a lifetime of about 50 years for a building, like for example a congress/meeting room, intensely used (about 300 times a year) and from the other side that the loading range of each cycle corresponds to the characteristic load combination for SLS. In such conditions, all the beams, although damaged (especially during the final stage), demonstrated ultimate loads certainly exceeding the demand.It can therefore be concluded that the reinforcement of wooden flooring with reinforced concrete, has a value that is anything but negligible, both from a structural and an economic point of view. In effect, reinforcing timber flooring with a reinforced concrete slab, not only allows to considerably increase the stiffness (2 to 5 times) but also contributes to build horizontal diaphragms that once properly connected to the perimeter walls, give the building a better anti-seismic performance, favoring what is usually called 'box behavior'.

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