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Strengthening and Stiffening Ancient Wooden Floors with Flat Steel Profiles

Natalino Gattesco and Lorenzo Macorini
University of Trieste, Department of Architectural Design and Urban Planning, Trieste, Italy

ABSTRACT: A strengthening technique for wooden floors of historical buildings aimed to increase both the flexural stiffness and the load capacity is herein proposed. The technique consists in placing steel plates above the existing boarding and connected to the wooden beams through steel dowels. Such a technique is high reversible and low invasive. In the study experimental investigations on the connection and on two floor types were carried out. The former allowed to determine the load-slip relationship of the connector, whereas the loading test on the two floors, carried out before and after being strengthened, permitted to quantify the stiffness increase. It was also made a numerical model which was calibrated on the basis of the experimental results on the floors and using the experimental load-slip relationship for the connection. The numerical simulation of the behavior of the two floors strengthened with the proposed technique was carried out up to the timber collapse.

1 INTRODUCTION

Most wooden floors of ancient buildings are excessively deformable under service loads and are not adequately stiff in their plane to prevent dangerous out of plane forces in the masonries when subjected to earthquakes, so that it is needed to increase their stiffness, both in plane and out of plane, as well as their bearing capacity. The stiffening and strengthening is often achieved by using a reinforced concrete slab over the timber decking (Piazza and Turrini 1983, Ronca et al. 1991, Gattesco 2001, Giuriani 2006). The aim of such a slab, adequately connected to the timber joists, is twofold: to form a wood-concrete composite structure to resist both bending moment and obtain the stiff diaphragm behavior. But this technique has some shortcomings concerning mainly the increase of dead load, which raises the seismic action, and the need for an additional structural depth over the existing decking, that is sometimes incompatible with the level of the floor. Moreover, this solution is frequently refused by the Cultural and Environmental Assets Service because it is considered with low “reversibility” and high invasiveness.

In the last years it is raised the interest for “dry” strengthening techniques, which employs planks, timber panels, steel sheets with thickness compatible with that of the floor (Giuriani 2004, Modena et al. 2005). These solutions belong to the so-called “reversible” techniques.

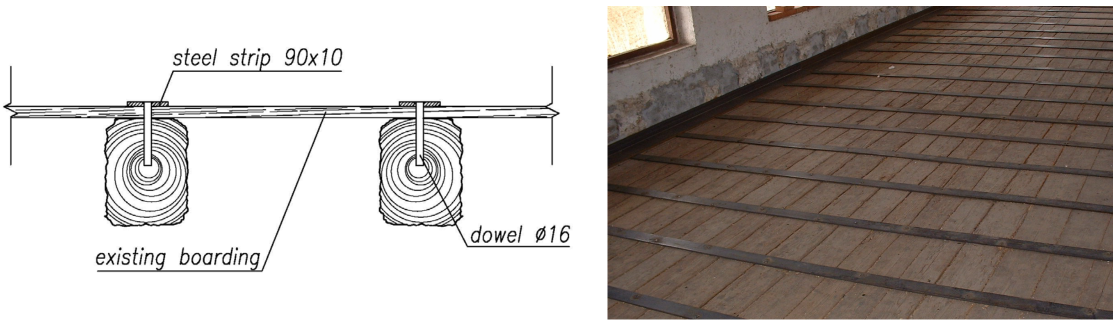
The aim of research work is to study the behavior of a particular technique obtained using flat steel profiles connected to wood joists with stud connectors. The studs are forced through few hammer blows into bore holes, drilled in the timber member, and welded to the steel profile. Moreover the scope is to investigate the characteristics needed to obtain adequate efficiency in terms of strengthening and stiffening of wooden floors. The strengthening system is applied over the existing boarding of the floor so to significantly simplify the execution of the intervention.

A numerical study is carried out in order to preliminarily compare different solutions in terms of both mechanical effectiveness and practical feasibility. The experimental investigation is carried out both on samples of single beams and on existing wooden floors with the purpose to quantify the increase in stiffness under the application of out of plane loads. The experimental results allow calibrating refined numerical models able to correctly describe the actual behavior of the floors and to extend the experimental results through broad numerical simulations.

2 STRENGTHENING TECHNIQUE

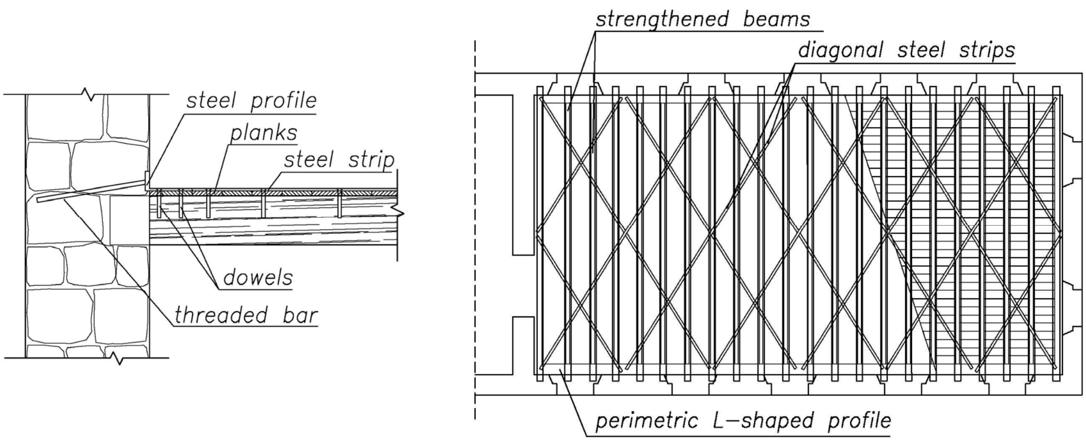
The strengthening technique consists in placing, above the existing boarding in correspondence of every wooden joist, a thin steel plate fixed to the beams through steel dowels (Fig. 1). The connectors are forced through few hammer blows into bore holes, drilled in the timber member, and welded to the steel profile. The dowels were driven in the timber member for 120 mm. This technique may be considered “dry” because does not uses any gluing material to bond the dowels to wood or any other material to be cast in place.

The composite system obtained concerns the wooden joist connected to a flat steel profile, 90 mm wide and 10 mm thick, through steel dowel connectors. The two members are separated by the existing boarding, having a thickness ranging between 20 mm and 30 mm, which are normally not considered as effective part of the composite system. Actually the existing boarding partly contrast the steel plate to buckle. As shown in some studies available in the literature on the wood-concrete composite floors (Gelfi and Giuriani 1999, Gattesco 2001), when the boarding is not removed above the timber joists a larger diameter for the dowels is needed in order to obtain the same connection stiffness (e.g. 12 mm without boarding, 16 mm with boarding interposed).



(a) (b)

Figure 1 : Strengthening technique: (a) cross section, (b) stiffened floor example.



(a) (b)

Figure 2 : Details of the complete intervention: (a) floor-wall connection, (b) lattice truss system.

A horizontal lattice truss needs to be realized to give to the floor good in plane stiffness so as to be able to transfer horizontal seismic action to the shear walls and to hinder out-of-plane displacements of the walls perpendicular to the seismic direction. Such a structure may be obtained placing a perimetric L-shaped steel profile anchored both to the floor joists, through dowels and by welds to reinforcing steel plates, and to the masonry walls, through driven dowels injected with cement grout (Fig. 2a). A system of diagonal steel strips was welded to the perimetric steel profile so to complete the lattice truss (Fig. 2b). The steel strengthening system is normally covered by an upper wooden flooring laid on listels.

3 EXPERIMENTAL INVESTIGATION

Several experimental tests on specimens that simulate the behavior of the connection in the composite floor system were firstly carried out so to determine the relationship between the shear force and the slip. Then two *in situ* tests on different existing floors of an ancient masonry building (late 19th century), used as water mill, were performed.

3.1 Experimental tests on the connection

A special experimental test was designed so to correctly simulate the actual behavior of the connectors in the floor joists. The specimens concern two flat steel profiles (Fe 430), with a cross section 90x10 mm, connected through couples of steel dowels (Fe 510), 16 mm diameter, at two opposite sides of a timber element of Northern Italy spruce, having a cross section of 170x300 mm (Fig. 3). Between the steel plates and the timber element some pieces of plank, 23 mm thick, are interposed so to simulate the floor boarding. The connectors are driven inside calibrated holes, made in the timber element, through several hammer blows. Appropriate steel devices are set at both ends of the specimen to allow grasping it to the loading machine (Fig. 3).

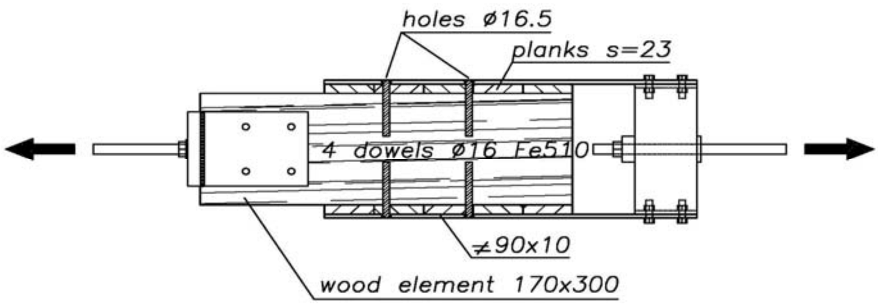


Figure 3 : Specimen for tests on the connection.

Three equal specimens were subjected to test using a hydraulic testing machine available in the Laboratory for Testing Materials and Structures of the University of Trieste. During the tests the load and the slip between steel elements and timber member were registered. The slip was measured through four linear variable displacement transducers (LVDT) with maximum elongation of 25 mm. The load-slip curves of the three experimental tests are illustrated in Fig. 4. The curves show a linear branch up to roughly a load of 10 kN, followed by a curvilinear path with a slightly inclined asymptote. The maximum load F_{max} and the stiffness K_s of the first branch are summarized in Table 1. Tests were stopped when the slip reached a value close to 25 mm, because it would have no interest to proceed. The average value of the stiffness resulted equal to 9.3 kN/mm and the average load capacity was 23.2 kN.

Table 1 : Results of tests on shear connectors.

Specimen	F_{max} (kN)	K_s (kN/mm)
S1	24.14	9.37
S2	22.10	9.20
S3	23.37	9.31

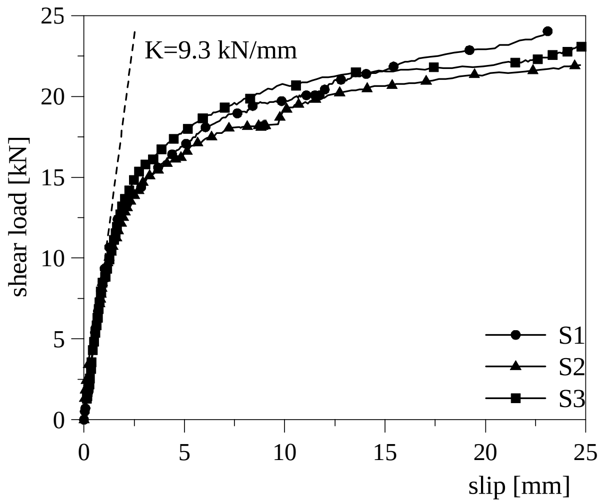


Figure 4 : Results of tests on the connection: load-slip relationship of the single dowel.

3.2 Experimental tests on floors in situ

The effectiveness of the proposed strengthening technique for ancient wooden floors was checked carrying out *in situ* loading tests on two types of floors of a 19th century building, identified as floor 1 and floor 2. For this purpose tests were performed before and after the strengthening intervention so to evidence the stiffness increase. Tests were carried out using a flexible water tank increasing the load up to the maximum estimated service load. The vertical displacements in different points of the floor (Fig. 5) were surveyed using linear variable displacement transducers (LVDT).

The considered floors consist in wooden beams (spruce), with cross section 170x200 mm, net length 6900 mm set at 550 mm on center. A boarding with average thickness of 23 mm is nailed to the beams extrados. The floor 1 includes, in addition, a transversal wooden beam, with same cross section as the longitudinal joists, supported at one end by a wall and at the other end by a I-shaped steel profile (HEA 200) parallel to wooden joists (Fig. 5a). So that this floor presents a significant bi-dimensional behavior, whereas the behavior of the floor 2 is more closely similar to that of simple beams (Fig. 5b).

The floors were strengthened using the proposed technique: a steel strip (90 mm wide and 10 mm thick) was connected to the wooden beams by means of driven dowels. The connectors were distributed along the beam length as illustrated in Fig. 6.

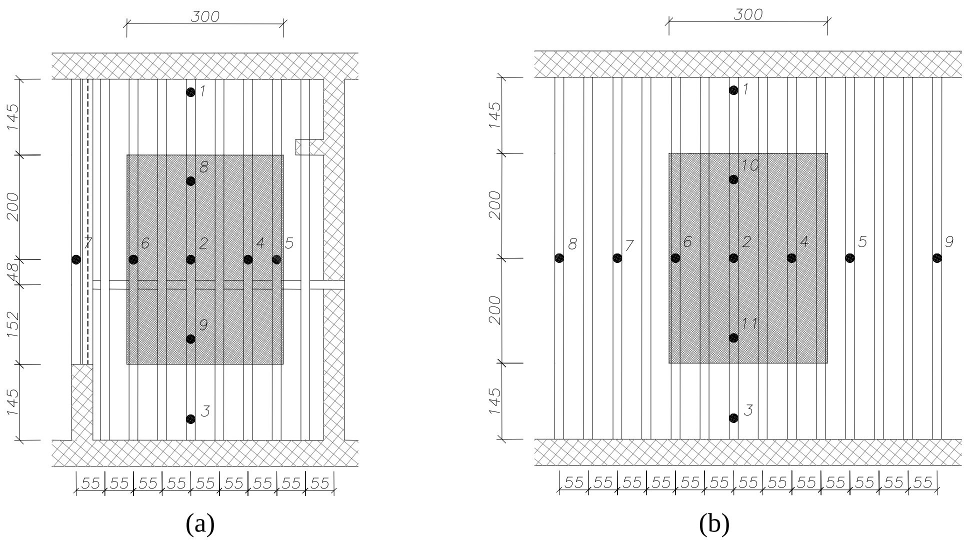


Figure 5 : Loaded portion of the floors and position of the transducers: (a) floor 1, (b) floor 2.

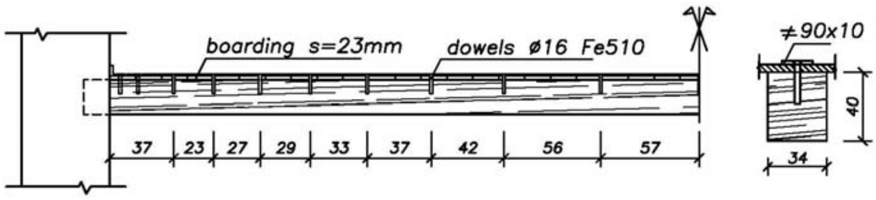


Figure 6 : Distribution of dowels along half a beam and cross section.

The maximum values of the vertical displacement (measure point 2 – Fig. 5) in correspondence of the service load, equal to 3.00 kN/m^2, are reported in Tab. 2 for both floors; the two columns evidence the values before and after the strengthening. The results show that a significant flexural stiffness increase in the strengthened floors is reached: 60% in floor 1 and 112% in floor 2. Only longitudinal beams were strengthened so that the lower stiffness increase in floor 1 is due to the fact that the transversal contribution becomes less effective after consolidation because of the unchanged stiffness of the transversal beam.

In Figs. 7-8 are plotted the vertical displacement profiles of the floor 2 before and after the strengthening, respectively. The two diagrams of each figure concern the displacement profiles along the axis of the beam in the center of the loaded zone (a) and the profiles along a transversal direction in the floor mid-span (b). The curves put clearly in evidence that the stiffness after the strengthening intervention is more than doubled, with respect to that of the floor before consolidation ($K_{s,after}$=2.1 $K_{s,before}$).

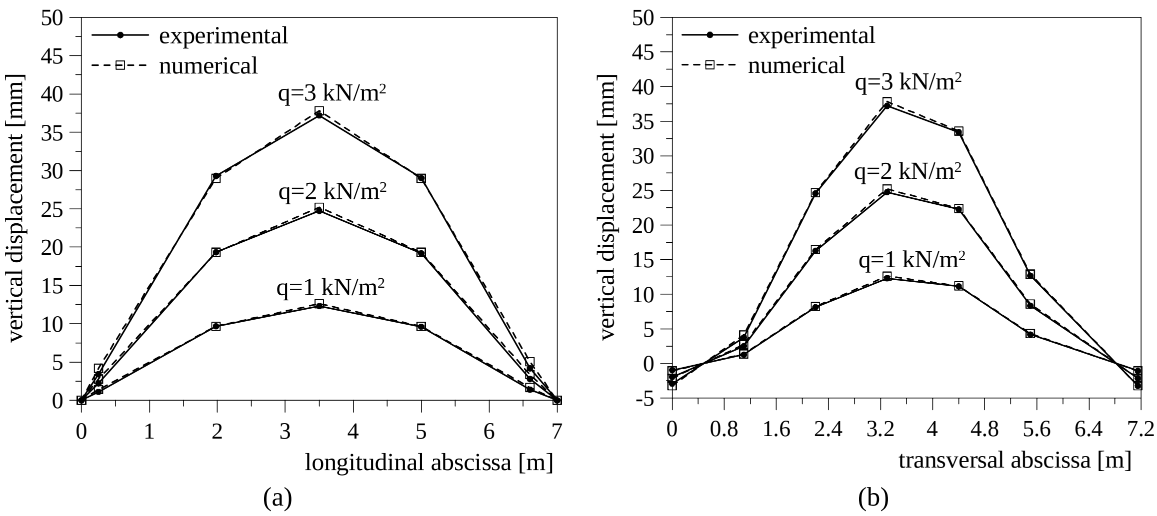


Figure 7 : Vertical displacements of the plain floor 2: (a) longitudinal direction, (b) transversal direction.

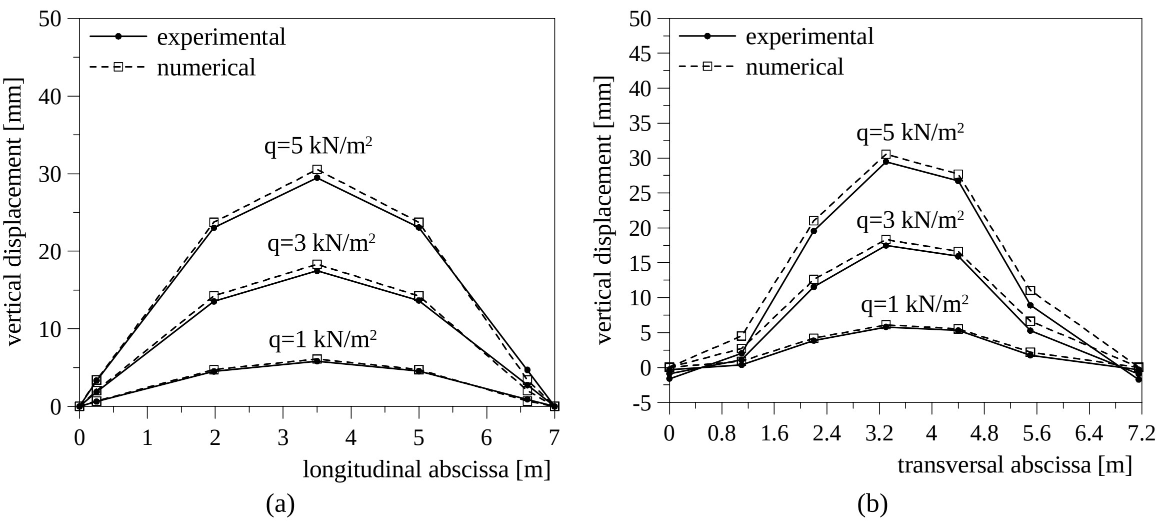


Figure 8 : Displacements of the strengthened floor 2: (a) longitudinal direction, (b) transversal direction.

Table 2 : Maximum deflection measured during the tests in correspondence of the load 3.0 kN/m^2.

Specimen	Max deflection (mm)	
	Before strenthening	After strengthening
Floor 1	19.9	12.4
Floor 2	37.2	17.5

4 NUMERICAL SIMULATIONS

A numerical model based on the finite element method has been built using ABAQUS code. The purpose was to simulate the whole behavior of the structure up to the collapse and to make possible extending the investigation to other types of floor. In the model the composite joist is schematized by means of two parallel beam elements, which represent the wooden member and the flat steel profile, respectively, connected together through non linear springs. The springs are settled horizontally on the middle plane of the boarding in correspondence of the position of each dowel. Rigid links connect the spring ends to the nodes of the main elements (Fig. 9).

In the analyses of the floors tested experimentally, the load distribution of boarding was taken into consideration including in the model adequate transversal beam elements. The main parameters of the mechanical model were calibrated on the basis of the experimental results on loading tests on the floors and the laboratory tests on the connection device.

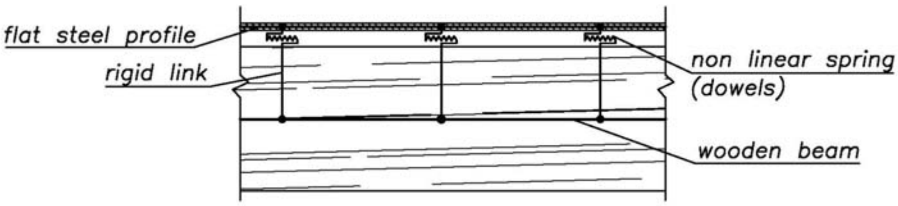


Figure 9 : Modeling of the composite system.

The first numerical analyses were carried out to simulate the two loading tests on the floors not yet strengthened, assuming a linear elastic behavior for timber, with the scope to derive the elastic modulus of the wood. In this way a modulus E_{0mean}=12000 MPa was achieved minimizing the standard deviation of the numerical values of the vertical displacements against the experimental values (Fig. 7). The obtained value is coherent with the expected one for a wooden member made of Northern Italy spruce.

Afterwards, the numerical simulations of the loading tests on the two strengthened floors were carried out. Timber material was assumed linear elastic up to failure, whereas an elasto-plastic relationship was assumed for the steel of the flat profiles. In particular for steel an elastic modulus Es=207000 MPa and a yielding stress f_{yk}=275 MPa (Fe430) were considered. For the connection, springs characterized by a piece wise linear relationship, describing properly the experimental results (Fig. 4), were assumed.

The results of the numerical analyses on the strengthened floor 2 are shown in Fig. 8. A good agreement between the numerical and the experimental curves can be observed, which confirms the accuracy of the model. Once completed the validation of the numerical model, the simulations of the behavior of the floors 1 and 2 were carried out up to the collapse. In such a way it was possible to quantify the bearing capacity of the floors, that is the capacity corresponding to the first reaching of the characteristic strength in one component material of the structure. Actually the analyses proceeded until the average experimental tensile strength of wood was reached ($f_{wt0,av}$=36 MPa). The load capacity q_f was then calculated in correspondence of the limit stress state in the timber member for combined bending and axial force, with reference to the characteristic strength values for wood ($f_{wt0,k}$=14 MPa and $f_{wm,k}$=23 MPa). Similarly the design load capacity q_d was calculated with reference to design strengths of wood, obtained with the relations

$$f_{wt0,d} = f_{wt0,k} \cdot \frac{k_{mod}}{\gamma_w}, \qquad f_{wm,d} = f_{wm,k} \cdot \frac{k_{mod}}{\gamma_w}, \tag{1}$$

where γ_w is the partial factor for wood and k_{mod} is the modification factor for duration of load and moisture content. Assuming, as indicated in ENV 1995-1-1 (2004), γ_w= 1.3 and k_{mod}=0.8 (medium duration variable loads) the strength values are $f_{wt0,d}$=8.6 MPa and $f_{wm,d}$=14.1 MPa. From this load, taking into account the partial factors for actions, the corresponding maximum service load q_s may be derived

$$q_s = \frac{1}{\gamma_G + n(\gamma_Q - \gamma_G)} \cdot q_d \tag{2}$$

where n is the ratio between variable and total load, γ_G and γ_Q are the partial factors for permanent and variable loads, respectively. For floor 2 the design load capacity is equal to 7.0 kN/m^2 and the corresponding maximum service load evaluated with Eq. (2), assuming n=0.9, γ_G= 1.4 and γ_Q= 1.5, is equal to 4.7 kN/m^2. The maximum deflection obtained with this service load is equal to 28.7 mm, that corresponds to the maximum allowable instantaneous deflection (l/250). In Fig. 10 the load-deflection curve of the floor 2 strengthened with the proposed technique is illustrated. On the diagram are evidenced with horizontal dashed lines, the bearing capacity q_f, the design load capacity q_d and the corresponding maximum service load q_s.

In Fig. 11 the load-slip curve for the outer dowels of the joist in the center of the loaded zone is plotted. It can be noted that the connector behaves linearly up to the maximum service load; beyond this load it starts to plasticize. In Figs. 12-13 the values of the slip and the shear force on dowels along half span are illustrated, respectively, for different loading levels. A significant slip increase can be noted when the load exceeds the maximum service load q_s (Fig. 12). In fact almost all the connectors plasticize for a load greater then the load q_s, as clearly shown in Fig. 13.

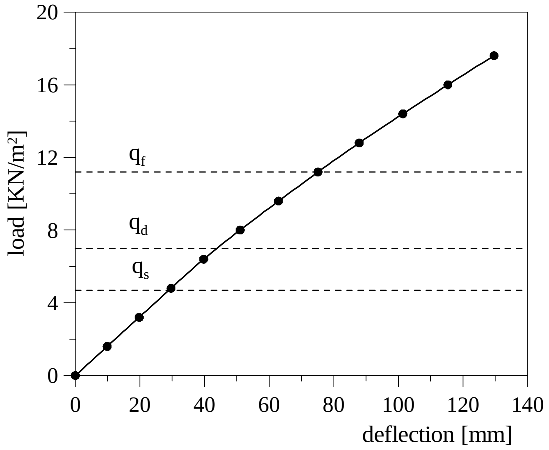


Figure 10 : Load-deflection for strengthened floor 2.

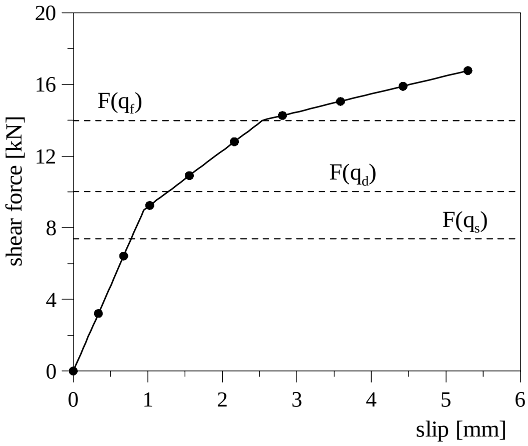


Figure 11 : Load-slip for outer dowels of floor 2.

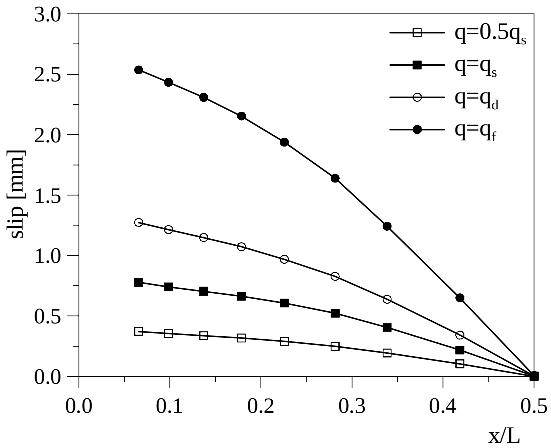


Figure 12 : Steel-wood slip along half span for floor 2.

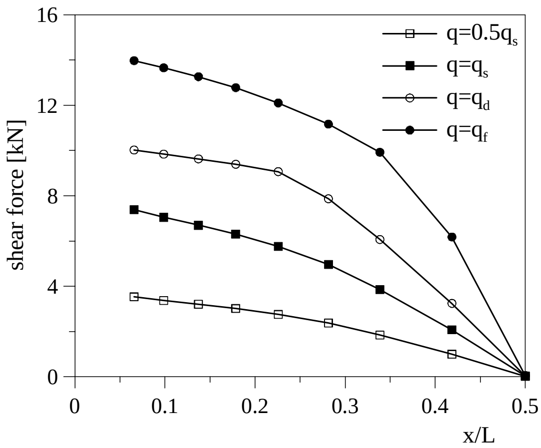


Figure 13 : Shear force on dowels along half span for floor 2.

5 CONCLUSIONS

In this research work a technique to strengthen and stiffen the wooden floors of ancient buildings is presented. The proposed technique consists in connecting to each joist a flat steel profile placed above the boarding. The connection is made with steel dowels driven into calibrated holes through several hammer blows and welded to the steel element. This technique respects the conservation requirements and is highly reversible and very low invasive.

This strengthening method allows to considerably improve the flexural behaviour of the floors. It is possible to increase even the efficiency of the in-plane floor behaviour under seismic actions by adding both some perimetric L-shaped steel profiles, linked to the masonry walls, and some steel strips placed diagonally and welded to the perimetric elements, so to form a lattice truss. In this way the strengthened floor is able to transfer the seismic actions to the shear walls and to prevent the out of plane displacements of the masonry walls set perpendicularly to the seismic direction.

The flexural effectiveness of the proposed strengthening techniques was evaluated both through experimental tests on two different types of wooden floors and by numerical simulations. Moreover some experimental tests on the connection device allowed defining the dowel load-slip relationship used in the numerical model.

The experimental *in situ* tests have been carried out on two different types of floor of a 19th century building, which houses a water mill. The first floor is formed by parallel joists stiffened by a wooden cross beam; the second one is simply formed by parallel joists. A wooden boarding 23 mm thick, is settled over the joists.

The loading tests, performed both before and after the strengthening work, allowed to define the flexural stiffness increase. In the most common floor, with simply parallel joists, the flexural stiffness after the strengthening was more then doubled (2.1).

The numerical model includes the nonlinear behavior of the connection obtained from the experimental tests on the connectors and it was calibrated on the basis of the experimental results performed on the two floors. The numerical simulation of the experimental tests up to failure allowed determining the bearing capacity and the corresponding maximum service load. In conclusion both the bearing capacity and the stiffness in service resulted considerably incremented with the intervention proposed. Such a numerical tool allows to reliably study the behavior of this type of floors up to the ultimate limit state.

REFERENCES

ENV 1995-1-1, (2004) Eurocode 5. Design of Timber Structures. Part 1-1: General Rules and Rules for Buildings, *CEN European Committee for Standardization*, Bruxelles.

Gattesco, N. 2001. Experimental Study on Different Dowel Techniques for Shear Transfer in Wood-Concrete Composite Beams, *Creative Systems in Structural and Construction Engineering*, Shing (ed.), Balkema, Rotterdam, pp. 487-492.

Gelfi, P. and Giuriani, E. 1999. Stud shear connectors in wood-concrete composite beams, *Proc. 1st International RILEM Symposium on Timber Engineering*, Stockholm, Sweden, 13-15 Sept. 1999, RILEM Publication S.A.R.L., pp. 245-254.

Giuriani, E. 2004. L'organizzazione degli impalcati per gli edifici storici, *L'Edilizia, Speciale Legno Strutturale*, n. 134, pp. 30-43.

Giuriani, E. 2006. Rinforzo di solai in legno mediante soletta collaborante di calcestruzzo, in L. Dezi and N. Gattesco (eds) *Strutture Composte – Nuove costruzioni, recupero, ponti*, International Centre for Mechanical Sciences, CISM, Udine, Italy, ISBN 88-85137-20-2, 2006, pp. 311-322.

Modena, C., Valluzzi, M.R., Garbin, E., da Porto, F. 2004. A strengthening technique for timber floors using traditional materials, *Proceedings of the Fourth International Seminar on Structural Analysis of Historical Constructions*, Padova, Italy, 10-13 Nov. 2004, pp. 911-921.

Piazza, M. and Turrini, G. 1983. Una tecnica di recupero statico dei solai in legno, *Recuperare*, Vol. 5, 6, 7, Milano.

Ronca, P., Gelfi, P. and Giuriani, E. 1991. The Behavior of a Wood-Concrete Composite Beam under Cyclic and Long Term Loads, *Structural Repair and Maintenance of Historical Buildings, 2nd International Conference STREMAH 91*, Seville, Spain, Vol. 1, pp. 263-275.