Behaviour of stud connectors in wood-concrete composite beams

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Abstract

The stiffening and strengthening of wooden floors with a thin collaborating concrete slab is a recent technique, which appears particularly suitable for restoration of ancient buildings.

In this experimental research work a simple kind of connection is studied, suitable for practice, using materials that are normally available in the ordinary restoration yard. Tests are carried out both for studying the global behaviour of the composite beams and the local behaviour of the connections. They are conducted on two different composite beams, one with the concrete slab in direct contact with the wood beam, the other with interposed planks. Stud connectors are used which are obtained by ordinary smooth steel bars simply forced into calibrated holes drilled in the wooden beam. For the evaluation of the stiffness and resistance of the connectors pure shear tests were carried out. Experimental results concern both the local shear behaviour of the single connector and the global flexural behaviour of the composite beams.

The effectiveness of the proposed technique on the flexural stiffness and strength of the composite beams is shown. Ultimate load values are in a good agreement with those calculated by classical approaches.

1 Introduction

The wooden floors in old buildings often manifest structural and functional weaknesses so that strengthening interventions are required. These floors are generally designed for modest live loads that do not meet the requirements of the
actual codes. In addition, as a consequence of long-term loads, wooden floors often have excessive deformations that compromise their functionality. In any case the stiffening and strengthening of wooden floors is required to avoid damage to partition walls, to limit vibrations and to avoid noise due to walking.

In many rehabilitation interventions the substitution of the wooden floors with new structures, such as reinforced concrete slabs or steel floors, is proposed. This approach is not accepted for historical buildings which need the conservation of the existing structures. To recover and strengthen the wooden floors specific interventions are required.

Strengthening techniques, based on the collaboration of thin reinforced concrete slabs, have been proposed. The collaboration between the reinforced concrete slab and the wooden beams is achieved by connections whose efficiency influences the global behaviour of the floor. In recent years, several types of connectors have been proposed and studied by some authors. These provide adequate mechanical characteristics but often need techniques that can be difficult to propose for the ordinary restoration yard. The connections can be obtained by studs fixed with resin (Turrini & Piazza¹), gang-nails (Giuriani & Frangipane² and Ronca, Gelfi & Giuriani³), tubular pins (Gelfi & Ronca⁴).

In the present experimental study, a strengthening technique that is simple and suitable for ordinary constructions is proposed. A significant improvement in the flexural stiffness of the floor as well as an increase in its bearing capacity is achieved. The results concern the global behaviour of composite beams. Moreover particular attention is paid to the local behaviour of the connection to
provide an experimental support for further theoretical and numerical investigations. For this purpose, the shear-slip law of the single connector, necessary for any theoretical model, is also provided.

2 Technological aspects

The studs are obtained from common smooth steel bars which are simply forced into calibrated holes drilled in the wooden beams. The technique becomes particularly effective when the studs are embedded without removal of the existent planks, preserving in this way the original wooden floor ceiling. This is of great importance when the intervention concerns floors that are of particular historical-architectural interest.

Two types of connections are studied. In the first type, the collaborating concrete slab is in direct contact with the extrados of the wooden beam. Only a small part of the plank has to be removed (see figure 1a). The stud connectors work prevalently under shear stress and an adequate stiffness is obtained with modest diameters (φ 12 mm). In the following, this typology will be referred to as direct contact connection.

In the second type, the slab is casted over the plank (figure 1b). The connectors crossing the plank are subjected also to flexural stress. Therefore, their diameter must be greater (φ 16 mm) to obtain both adequate stiffness and

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<td><strong>Wood (red spruce)</strong></td>
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<tr>
<td>Average relative humidity R.H. (^1)</td>
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<tr>
<td>Elasticity modulus (E_w) (^2) (∥ to grain)</td>
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<td>Average flexural strength (f_{wm}) (^3) (∥ to grain)</td>
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<td>Average compression strength (f_{c_o}) (^4) (∥ to grain)</td>
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<td>Bearing strength beneath the 12 studs (f_h) (^5) (∥ to grain)</td>
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<td><strong>Concrete</strong></td>
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<td>Average cylindrical strength (f_{cm}) (^6)</td>
</tr>
<tr>
<td>Secant elasticity modulus (E_c) (^7)</td>
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<tr>
<td><strong>Steel</strong></td>
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<td>Yielding stress (f_y)</td>
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<td>Tensile strength (f_u)</td>
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\(^1\) Measured with electronic moistmeter.  
\(^2\) Average value measured on 9 prismatic samples 20x20 mm according to UNI ISO 3349.  
\(^3\) Average value measured on 9 prismatic samples 20x20 mm according to UNI ISO 3133.  
\(^4\) Parallel to grain, mean of 10 prismatic samples 20x20 mm according to UNI ISO 3787.  
\(^5\) Average value of two tests on a 20 mm thick plank.  
\(^6\) Average value of 9 cylindrical samples.  
\(^7\) According to UNI 6556, mean of tree samples.
strength. In the following, this typology will be referred to as interposed plank connection. Table 1 lists material mechanical characteristics.

The stud is obtained from a smooth bar, which is rounded off at its end to facilitate the insertion in the wood. The shearing machine should be avoided because it provokes the ovalization of the connector terminal section with consequent slacks due to the widening of the hole during the stud insertion.

The holes are made by ordinary drills for steel with a diameter slightly smaller than that of the stud (respectively $\phi$ 11.75 mm and $\phi$ 15.75 mm) and the stud is forced in the hole with a hammer. The concrete slab is reinforced with welded mesh.

3 Shear tests of the connectors

The geometry of the samples (see figure 2) was studied in a way to simulate the behaviour of a single connector inside the composite beam. Every sample represents the ideal length of beam including one connector. The area of contact between the reinforced concrete slab and the wooden beam is equal to the
spacing of the connectors. In this way the concrete-wood friction is comparable to that in the beam.

Six samples of the first type, with φ 12 studs and concrete in direct contact with the wooden beam and six others with φ 16 studs and interposed plank, were investigated. Two samples for each type had different stud insertion depths equal to 3, 4, 6 times the diameter respectively.

The test bench (see figure 3), already used to investigate steel to concrete connectors (Gattesco & Giuriani\textsuperscript{5}), is designed to impose increasing slip between the wood and the concrete. The concrete slab is embedded in the frame (a) (see figure 4) while the flange (b) is glued to the wooden beam with epoxy resin. Force F is applied by means of a threaded bar which is instrumented with strain gauges. The force acts in the plane of the concrete-wood interface for the first type connection and in the mid-plane of the plank for the second type. The shear load does not coincide exactly with the position of the inflexion point of the connector because of the different stiffness of the connector embedded in the concrete and in the wood. Therefore, a secondary moment, even if modest, occurs in the connector shank. It provokes relative rotation between the beam and the slab so that the connecting rods (a) indicated in figure 2 were inserted. At both the extremities of these connecting rods, a sort of hinge was obtained by reducing the section. They were inserted in holes having a greater diameter to allow the relative displacement between the wood and the concrete. The axial force in these rods was measured by means of strain gauges and therefore the
secondary moment and the position of the inflexion point were evaluated. The
connecting rods do not appreciably reduce the shear load applied to the
connector, since they oppose a negligible shear force of only 30 N for a slip of 1
mm.

The relative displacement between the concrete slab and the wooden beam
is measured by two micrometric mechanical gauges (see figure 3).

![Figure 4: Shear tests loading scheme.](image)

For each sample, three loading-unloading cycles were carried out, with a
shear load ranging between zero and 3 kN. This load corresponds approximately
to the shear force in the connectors of a composite beam under service load. The
loading was applied by imposing increasing displacement corresponding to load
increments of 0.1 kN at each step. The measure of the shear force was made at
the end of every load cycle after a settlement time of 5 minutes. The fourth load
cycle was carried out up to a nominal rupture limit corresponding to a 3 mm slip.

![Table 2: Connector initial stiffness and ultimate resistance.](table)

Figures 5 and 6 show the shear force-slip curves of the samples with a
connector insertion length in the wood equal to four times the diameter. Table 2
shows the values of the initial secant stiffness, evaluated for a 0.05 mm slip, and
the values of the nominal failure resistance (3 mm slip). The initial stiffness of
the samples with concrete in direct contact has a very high value due to the initial
cement-wood bond. This stiffness in practice quickly drops due to shrinkage,
cyclic loads and thermal effects.
Figure 5: Shear force-slip curves for direct contact connection.

Figure 6: Shear force-slip curves for interposed plank connection.

Figure 7: First loading cycles and initial stiffness.
4 Experimental behaviour of composite beams

Two wood-concrete composite beams were tested to study the role of the local behaviour of the connection on the global flexural stiffness and strength.

The two beams (figure 8) differ in the types of connectors. In the first the concrete slab is in direct contact with the wooden beam and the connectors have a diameter $\phi = 12$ mm. In the second, the concrete slab is casted over the plank with $\phi 16$ connectors (interposed plank connection). In both of the beams, the connector insertion length is of 4 diameters. The characteristics of the materials and the positioning modality of the connectors are the same as the ones used in the shear tests (see § 2).

Figure 8 shows the geometrical and mechanical characteristics of the beams, in particular the mechanical characteristics of the transformed sections, where the concrete slab has been considered as wood equivalent. The transformed “all wood” area has been assumed as having a fictitious width equal to $b^* = b E_c/E_w$, being $E_c/E_w = 3.26$ the ratio between concrete and wood elastic moduli and $b$ the width of the slab. The width of the concrete slab has been adopted equal to the beam spacing of a common wooden floor (50 cm). The connectors are spaced 10 cm apart in the zones near the supports (up to 1/4 of the span) and 20 cm apart in the mid zone of the beam; they are joggled to avoid longitudinal splitting of the wood. The concrete slab is reinforced with a $\phi 6$ mm welded mesh 100x100 mm of FeB44k steel.

Maximum stresses of concrete $\sigma_c$ and wood $\sigma_w$ are indicated. They were calculated with the transformed section approach for a bending moment value $M = 6.56$ kNm corresponding to service conditions. The service load was assumed equal to 2.75 kN/m for simulating a floor subjected to a total (dead plus live) uniformly distributed load equal to 5.50 kN/m². Stress $\sigma_{ow}$, calculated in the assumption that only the wood has load bearing capacity, is also indicated; note the very high value $\sigma_{ow} = 17$ N/mm².

Since the clear span is 4.00 m, the maximum shear at service load is $V = 5.50$ kN. Shear force $F$ of the most loaded connectors near the supports was evaluated according to the theory of the transformed section (3.50 kN and 3.73 kN respectively for the connectors $\phi 12$ and $\phi 16$). It corresponds to about 1/3 of the value of the ultimate shear resistance of the samples (9.5 kN and 11.5 kN respectively for the connectors $\phi 12$ and $\phi 16$). As a matter of fact this theory overestimates the maximum value of the shear force, because the slip between the concrete slab and the wooden beam, which also occurs under service loads (Giurani², Ronca³), provokes a redistribution of shear force along the connection (Giuriani⁴), with consequent reduction of the maximum value.

The test bench is shown in figure 9. Beam (a), turned upside down, is put on the two rocker arms (b) hinged at the supports (c). Loads are applied imposing vertical displacements of the extremities by means of screws acting on the steel heads (e). The applied forces are measured from the micrometric gauges of the dynamometric lozenge-rings (d).
Direct contact connection

Welded mesh ø6
100x100 mm

Connector ø12

Interposed plank connection

Plank

Connector ø16

Connector spacing 100 mm
Connector spacing 200 mm

Characteristics of the transformed "all wood" sections with $E_c/E_w = 3.26$

- $y_0 = 48 \text{ mm}$
- $J_{id} = 28000 \text{ cm}^4$
- $E_w = 9500 \text{ N/mm}^2$
- $J_w = 3714 \text{ cm}^4$
- $S = 1832 \text{ cm}^3$
- $J_{id}/J_w = 7.75$

Theoretical stresses:
- $\sigma_c = 3.60 \text{ N/mm}^2$
- $\sigma_n = 4.14 \text{ N/mm}^2$
- $\sigma_{cw} = 13.87 \text{ N/mm}^2$

NOTE: $\sigma_c, \sigma_n, \sigma_{cw}$ are the maximum stresses in the concrete and in the wood calculated with the transformed section approach, under a service load of 5.50 kN/m², that provokes at mid span a moment $M=6.56 \text{ kNm}$. $\sigma_{cw}$ is calculated in the hypothesis that only the wood has bearing capacity.

Figure 8: Geometrical and mechanical characteristics of the two composite beams
The load scheme in figure 9 corresponds to that of a floor beam resting on two walls with a 4 m clear span, equal to the length of the concrete slab, and a 4.37 m design span. The two steel heads (e), that extend the wooden beam, are not in contact with the slab in order not to disturb its slip with respect to the wood. The dead load of the experimental beam produces negligible internal actions because it is supported by the rocker arms (b). Gauges were positioned to measure both the deflection and the relative slip between the slab and the wooden beam.

![Test bench and loading scheme](image)

Figure 9: Test bench and loading scheme

On both beams four load cycles were carried out: two up to a total load \( Q = 6.0 \) kN, which is equivalent to about 60% of the total service load that was assumed for the design of the beam; the third up to a value \( Q = 22.9 \) kN, which is about twice the total service load, and the fourth up to rupture. The tests were carried out by controlling the deformations. The increase of deflection at each step was 0.3–0.4 mm in the first two cycles, 0.5–1 mm in the third cycle and 1–2 mm in the fourth cycle; the loading ratio corresponded to a load step every 5 minutes.

Figures 10 and 11 show the load-deflection and load-slip diagrams that demonstrate an almost linear elastic behaviour in both of the beams under service loads (\( Q \leq 9.48 \) kN). In particular, the beam with the direct contact connection (studs \( \phi 12 \)) has a linear behaviour that coincides with that of the transformed section theory. As a matter of fact up to a load value equal to about 7 kN, which represents 75% of the service load value, the beam deformation is characterised by the “bonding” between the wood and the concrete (see figure 10b).
Figure 10: Load-deflection diagrams of the two composite beams.
Figure 11: Wood-concrete relative slip diagrams of the two composite beams.

Note that load $Q = 9.48$ kN causes a bending moment $M = 6.56$ kNm at mid-span equal to that of a distributed load of 2.75 kN/m that is equivalent to a load of 5.50 kN/m$^2$ on the floor, as above mentioned. For this value, in table 3 the experimental values $f_e$ of the deflection are compared with the theoretical values calculated both for the composite beam with the transformed section theory ($f_t$) and for the wood beam without concrete slab ($f_{w0}$).

The table shows that the composite beams, with perfect connections, would have a theoretical stiffness, about 8 times superior to that of the isolated wood beam ($f_{w0}/f_t = 7.75 - 8.42$ respectively for the two beams). The deformability of the connections slightly reduces this stiffness and causes an increase in the deflection

<table>
<thead>
<tr>
<th>Connection type</th>
<th>experimental $f_e$ [mm]</th>
<th>theoretical $f_t$ [mm]</th>
<th>only wood $f_w$ [mm]</th>
<th>$f_w/f_t$</th>
<th>$f_w/f_{w0}$</th>
<th>$f_w/f_e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direct contact with plank</td>
<td>5.57</td>
<td>4.77</td>
<td>37.0</td>
<td>7.76</td>
<td>1.17</td>
<td>6.64</td>
</tr>
<tr>
<td></td>
<td>7.69</td>
<td>5.83</td>
<td>49.1</td>
<td>8.42</td>
<td>1.32</td>
<td>6.39</td>
</tr>
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</table>
of about 20-30% respectively ($f_c/f_t = 1.17\div1.32$).

The ultimate load (see figure 10) is practically the same for the two beams (respectively $Q = 66.2$ and $63.8$ kN, equal to 7 and 6.7 times the service load). The deflections are 146 and 106 mm and the maximum slips between the wood and the concrete are 12 and 7 mm, respectively for the beam with studs $\theta 12$ and $\theta 16$. In both cases, the collapse occurred due to the tensile failure of the wood. Cracks in the lower part of the concrete slab, as a consequence of the great curvature of the beam, also formed; so the neutral axis of the concrete slab was much closer to the compressed concrete fibre.

The experimental values of the ultimate load and of the correspondent bending moment are in good agreement with those calculated assuming: a) perfectly plasticized connection, b) wood with linear elastic behaviour up to rupture and c) stress block for compression stresses in the concrete as consequence of cracking. With reference to the following mechanical characteristics (see table 1 and 2):

- wood ultimate strength: $\sigma_{wu} = f_{wm} = 69$ N/mm$^2$
- concrete ultimate strength: $\sigma_{cu} = f_{cm} = 31$ N/mm$^2$
- $\theta 12$ connection ultimate shear strength: $F_u = 9.5$ kN
- $\theta 16$ connection ultimate shear strength: $F_u = 11.5$ kN

the total shear force of the half beam is obtained. This shear force, corresponding to 14 connectors, has the value $D=133$ kN for the beam with connectors $\theta 12$ and $D = 160$ kN for the beam with connectors $\theta 16$. The depth of the stress-block is equal to 8.6 mm (10.4 mm for $\theta 16$ connectors) and therefore the resultant of the compression stresses $N_c$ on the slab has eccentricity, with respect to the centroid of the wood beam, equal to $z = 147$ mm ($z = 139$ mm for $\theta 16$ connectors). For equilibrium, tensile axial force $N_w$ of the wooden beam equals compressive force $N_c$ of concrete. Being $N_c=N_w=D$, the value of the ultimate bending moment is:

$$M_u = Dz + (\sigma_{wu} - D/A_w)J_w/(h_w/2) = 48.5 \text{ kNm} \quad (M_u = 45.0 \text{ kNm})$$

where $A_w$ and $h_w$ are the wood area and section depth (see figure 8). These values are in a good agreement with the experimental ones equal to 45.8 and 44.2 kNm respectively.

5 Conclusions

In this paper experimental results are presented both concerning the local behaviour of one stud connector and the global behaviour of the composite beams. The following concluding remarks can be expressed:

- The proposed technique, based on stud connectors simply forced into calibrated holes, without resins, appears to be simple and suitable for the restoration interventions.
- The stud connection technique with interposed plank between the concrete slab and the wooden beam is particularly simple and effective. The $\theta 16$ stud connectors with interposed plank showed a behaviour which was very similar
to that with $\phi$ 12 studs and concrete in direct contact with the wood.

- Insertion lengths greater than four times the stud diameter do not considerably improve the connection stiffness and strength.

- The actual deflection of the composite beams under service loading resulted as not being very different from that given by the theory of the transformed section; a $20\%-30\%$ increase in deflection occurred due to the slip between concrete slab and wooden beam.

- The experimental behaviour of the composite beam up to failure is in good agreement with the theoretical one assuming the complete plasticization of the connection and the elastic behaviour of the wood beam up to the failure. The differences between the experimental and theoretical values at rupture are less than $6\%$.

References


