Stud Shear Connection Design for Composite Concrete Slab and Wood Beams

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Abstract: The stiffening and strengthening of wood floors with a thin collaborating concrete slab is a recent technique which appears particularly suitable for restoration work on ancient buildings. This research deals with the theoretical evaluation of the stiffness and strength of the connection between the wooden beam and the concrete slab. Toward this end, both the stiffness and the strength of the connection between the wooden beam and the concrete slab are theoretically assessed. The aim of the present research work is to define a simplified approach which allows the connection design to be based on deformation control. The stud connection is studied in the general case of wooden planks separating the concrete slab and the wooden beam. The initial stiffness of the connection is evaluated on the basis of the classical approach of the beam on elastic foundation, whereas the ultimate strength is based on the collapse mechanism with two plastic hinges in the stud shank. The failure mechanism leads to the definition of the minimum stud length. The results of the theoretical formulation are in good agreement with experimental results.

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Introduction

The growing interest in the rehabilitation of ancient buildings, even those of minor importance, has set the problem of the stiffening and strengthening of wooden floors. The major troubles and deficiencies of ancient wooden floors are excessive deformation under service loads and poor sound insulation. The employment of a thin collaborating concrete slab is appropriate for stiffening the wooden floor and reducing noise. This technology is simple and inexpensive, thus particularly suitable even for minor buildings. The concrete slab also allows one to both obtain a fire barrier and level the floor when the wooden beams have a large inelastic deflection. Furthermore, when properly connected to the walls, it provides the structure with a resisting floor diaphragm which improves the building performance under seismic actions. The stiffening of wooden floors depends entirely on the efficiency of the collaboration between concrete slab and wooden beam obtained by special connectors. In recent years, several types of connectors have been proposed and studied, including: stud connectors fixed into the wood with epoxy resin (Piazza and Turrini 1983), gang nails (Ronca et al. 1991; Giuriani and Frangipane 1993), common nails, screws, concrete connectors, tubular pins lodged in milled holes (Gelfi and Ronca 1993), and high strength nails (Ahmadi and Saka 1993).

The connection technique presented here employs dowels which are manufactured from ordinary smooth steel bars and are forced into calibrated holes drilled in the wood beam. This technique proved to be particularly suitable for rehabilitation works because of its easy installation process and high reliability. A particularly remarkable advantage of this technology is the possibility of inserting the dowels through the existing wooden planks, thus avoiding their removal. This technique has been extensively studied both experimentally and theoretically by Gelfi and Giuriani (1999a,b).

The stiffness improvement of the floor beam might be compromised by an excessive deformation of the connection. Accordingly, the limitation of the connection deformability should be the target of the connection design. The load-bearing capacity of the concrete–wooden beam depends on the connection strength too, but a design based on the deformability control usually provides sufficient bearing capacity.

In the present work, an analytical formulation of the stud connection stiffness is presented, so that the connection design can be based on the accepted beam deflection. A simplified solution is also proposed which is suitable for practice. This approach stems from the theoretical and experimental study on stud connectors for concrete–steel composite beams (Gelfi and Giuriani 1987).

Theoretical Evaluation of the Connection Stiffness

The connection stiffness evaluation is important because the composite concrete–wood beam deflection is strongly influenced by the slip between the beam and the slab. In the initial elastic range the stud behavior (Fig. 1) ideally resembles that of a traditional beam on elastic foundation (Patton-Mallory et al. 1997), both in the length embedded in concrete and in the length embedded in wood. The portion of the stud between the beam and slab is

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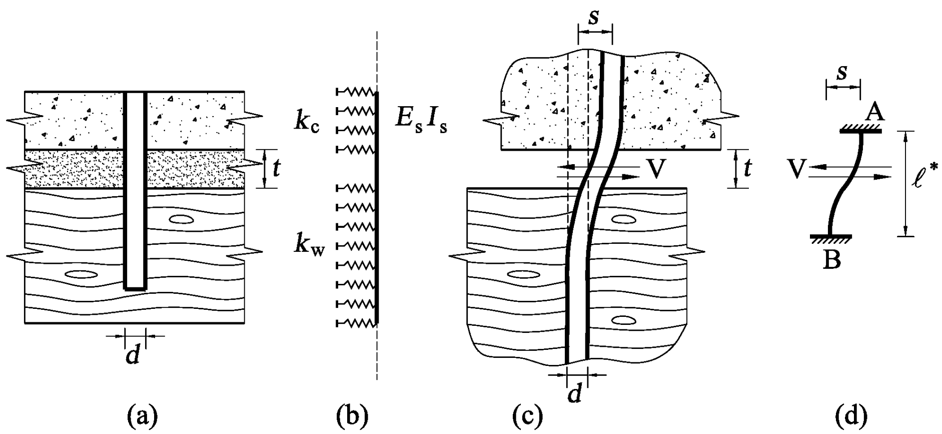

Fig. 1. Connection stiffness modelization

assumed to be free, because the interposed plank has a negligible stiffness being the load perpendicular to the grain. No clearance between the stud and wood is considered, being the studs forced into calibrated holes.

The stiffness of the equivalent elastic soil can be obtained by means of tests which are similar to those recommended by STD-American Society for Testing and Materials (ASTM) 5764-970 (ASTM 1997). Fig. 2 shows the experimental curves corresponding to Alps spruce wood for different stud diameters ($d = 12/48$ mm). Force per unit thickness of wood is plotted as a function of the relative displacement. The average initial slope of the curves, represented by a dashed line in Fig. 2, corresponds to the stiffness k_w of the Winkler foundation. The stiffness k_w is approximately equal to 1,300 N/mm^2 and it does not significantly depend on the stud diameter. A similar result is presented in Gattesco (1998) (1,200 N/mm^2 for 16 mm stud diameter).

As for concrete stiffness k_c, few experimental data are available (Biolzi and Giuriani 1990). Gelfi and Giuriani (1987) proposed the relationship $k_c = E_c/\beta$, where E_c is the Young modulus of the concrete and $\beta = 2.5/3.3$ is a function of the ratio between the stud diameter and the stud spacing. As already mentioned, in the present study, the stud is assumed to behave as a Winkler beam of unlimited length, both in the concrete and in the wood. This simplification can be accepted because the diffusion zones, where stud deformations are significant both in concrete and in wood, are small and never larger than the stud length usually adopted.

The stud stiffness K_S (defined as $K_S = V/s$, where V = stud shear force and s = wood–concrete slip, Fig. 1) can be easily determined by imposing the continuity of the flexural deformations of the stud. By adopting the flexibility method approach, the compatibility equations in section H [Fig. 3(a)] are the following:

$$\eta_{11}V_1 + \eta_{12}M_2 + \eta_{10} = 0$$
$$\varphi_{21}V_1 + \varphi_{22}M_2 + \varphi_{20} = 0 \quad (1)$$

where the flexibility coefficients are

$$\eta_{11} = \frac{2\alpha_c}{K_c} + \frac{2\alpha_w}{K_w} + \frac{4\alpha_w^2}{K_w}t + \frac{4\alpha_w^3}{K_w}t^2 + \frac{t^3}{3E_sI_S} \quad \eta_{12} = \varphi_{21}$$

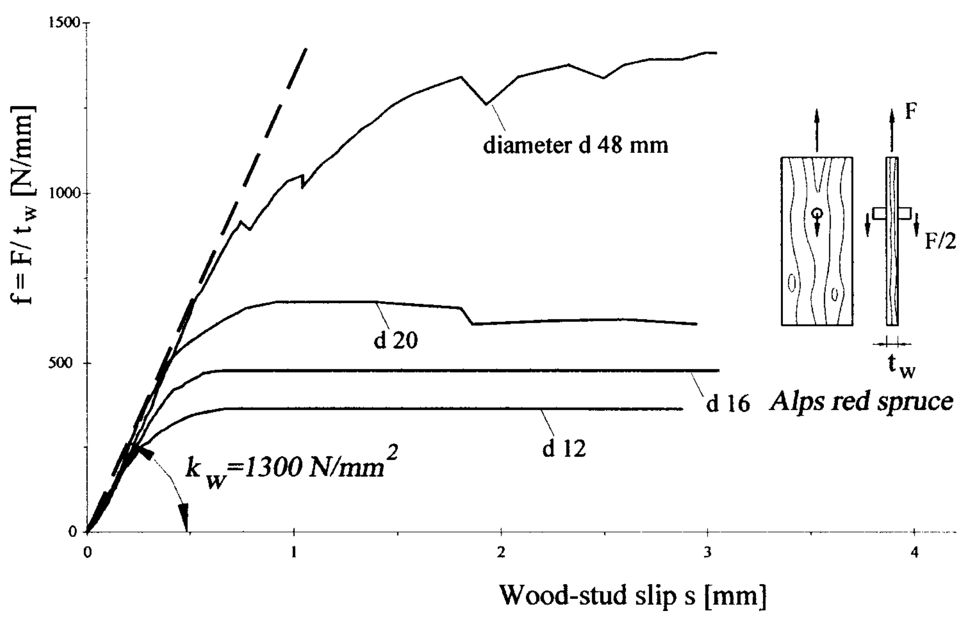

Fig. 2. Experimental curves for the evaluation of wood stiffness k_w

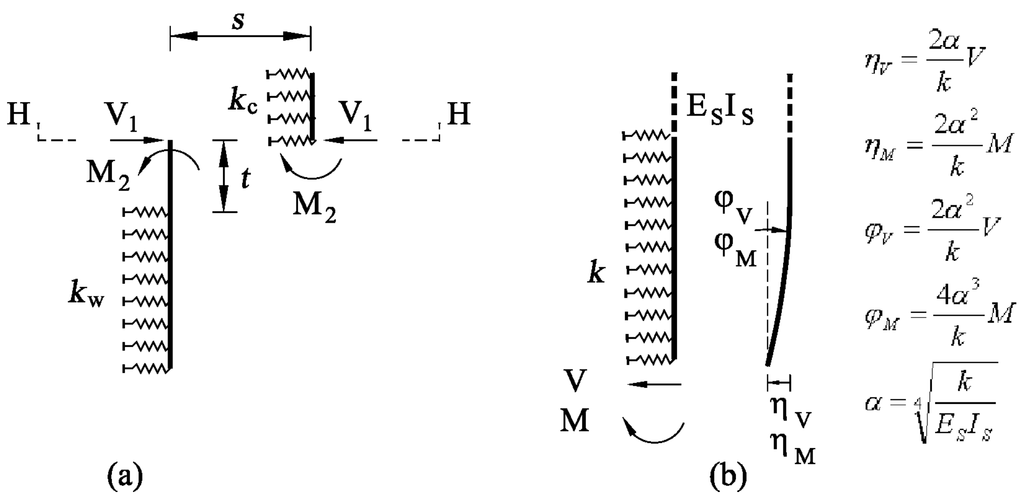

Fig. 3. Scheme for stud stiffness calculus

$$\varphi_{21}=\frac{2\alpha_c^2}{K_c}-\frac{2\alpha_w^2}{K_w}-\frac{4\alpha_w^3}{K_w}t-\frac{t^2}{2E_sI_S}\quad \varphi_{22}=\frac{4\alpha_c^3}{K_c}+\frac{4\alpha_w^3}{K_w}+\frac{t}{E_sI_S}$$

$$\eta_{10}=-s\quad \varphi_{20}=0$$

with

$$\alpha_c=\sqrt[4]{\frac{k_c}{4E_SI_S}};\quad \alpha_w=\sqrt[4]{\frac{k_w}{4E_SI_S}}$$

The flexibility coefficients stem from the solution of the semi-infinite beam on Winkler foundation, loaded by a shear force V_1 and a bending moment M_2 at the beam free end (Fig. 3). Fig. 3(b) shows displacements η_V, η_M, and rotations φ_V, and φ_M, induced at the beam end by the imposed loads V and M, respectively. The flexibility coefficients are given by the addition of the displacements and rotations of the beam embedded in concrete (index c), of the beam embedded in wood (index w) and of the unconstrained beam segment of length t between the concrete and wood.

By eliminating M_2 in Eq. (1), the shear force V_1 can be obtained as function of the slip s. The connection stiffness is therefore

$$K_S=\frac{V_1}{S}=\frac{12(\alpha_c\alpha_w)^3E_SI_S}{Z} \tag{2}$$

where $Z=3(\alpha_c^2+\alpha_w^2)(\alpha_c+\alpha_w)+3t\alpha_c\alpha_w(\alpha_c+\alpha_w)^2+3(t\alpha_c\alpha_w)^2(\alpha_c+\alpha_w)+(t\alpha_c\alpha_w)^3$. This solution is only formally exact, because it is based on the simplified assumption of the beam of unlimited length on a perfectly elastic foundation. A reduction of the connection stiffness due to the finite length of the stud might be expected although it is negligible when the stud length typically used in the construction practice is adopted. The problem of the stud length will be further discussed in the following paragraph. A further approximation for the expression of K_S, which is more suitable for design purposes, is here proposed.

The stud stiffness K_S is imposed equal to that of the double embedded beam of Fig. 1(d) of ideal length ℓ^*:

$$K_S=\frac{12E_SI_S}{\ell^{*3}} \tag{3}$$

which becomes equal to Eq. (2) when

$$\ell^*=\frac{\sqrt[3]{Z}}{\alpha_c\alpha_w}=f(k_c,k_w,t,d)$$

Function $f(k_c,k_w,t,d)$ can be expanded around the reference values (k_{co},k_{wo},t_o,d_o) of its four independent variables:

$$\ell^*=f(k_{co},k_{wo},t_o,d_o)+\left(\frac{\partial f}{\partial k_c}\right)_o(k_c-k_{co})+\left(\frac{\partial f}{\partial k_w}\right)_o(k_w-k_{wo})+\left(\frac{\partial f}{\partial t}\right)_o(t-t_o)+\left(\frac{\partial f}{\partial d}\right)_o(d-d_o)+\cdots$$

It is possible to demonstrate that all terms of an order higher than one can be neglected. Consequently, the series is truncated after the first-order terms. Adopting as a reference the most recurring values for the concrete and the wood stiffness, for the interposed plank thickness and for the stud diameter ($k_{co}=10{,}000$ N/mm^2, $k_{wo}=1{,}300$ N/mm^2, $t_o=25$ mm, and $d_o=16$ mm), it follows that

$$\ell^*\cong 17.3-0.000572k_c-0.00894k_w+0.880t+4.34d \tag{4}$$

with k_c and k_w expressed in Newtons per square millimeter, and t and d in millimeters.

Note that K_S expressed as in Eq. (2) was not expanded with Taylor series because, unlike the ideal length ℓ^*, it does not rapidly converge.

If the parameters vary within intervals of practical interest

$$7{,}000<k_c<14{,}000 \text{ N/mm}^2;$$

$$1{,}000<k_w<1{,}400 \text{ N/mm}^2;$$

$$12<d<20 \text{ mm};$$

$$0<t<50 \text{ mm}$$

the differences between the "exact" solution of Eq. (2) and the approximate solution given by Eqs. (3) and (4) are negligible, being the maximum difference at most equal to 15%.

In Fig. 4, K_S is plotted as a function of plank thickness t, for different values of stud diameter d and for significant values of k_w and k_c. Curves given by Eqs. (3) and (4) (dashed lines) are very close to those given by the exact solution of Eq. (2) (continuous lines).

In Fig. 5, the theoretical results given by Eqs. (3) and (4) are compared with the experimental data by Gelfi and Giuriani (1999b). The theoretical straight line (dashed) is in good agreement with the initial stiffness of the experimental results.

A more expressive formulation can be obtained for the most commonly used materials (Alps red spruce wood and ordinary

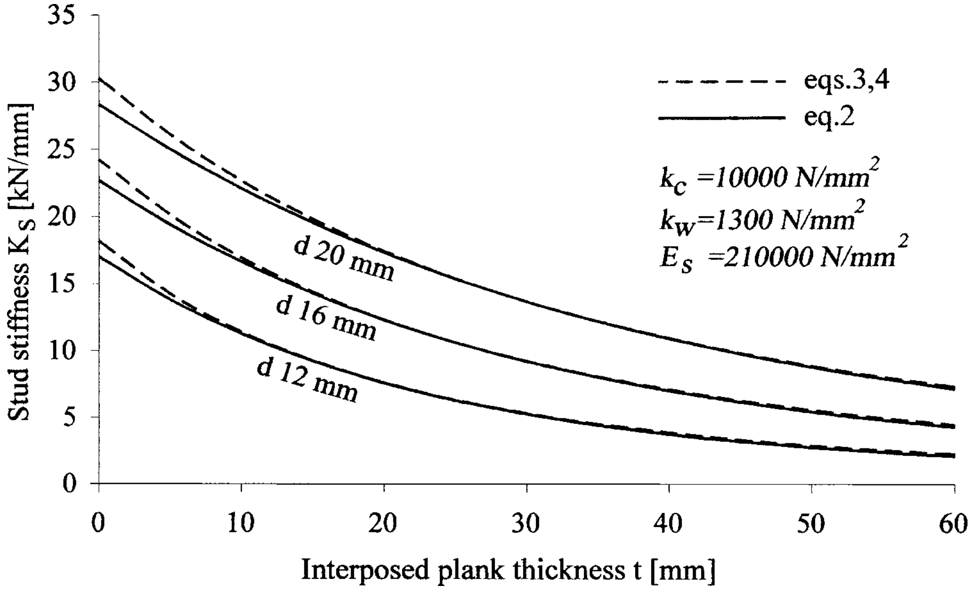

Fig. 4. Stud stiffness K_S as function of plank thickness t and diameter d

concrete). Assuming $k_w = 1{,}300\ \text{N/mm}^2$ and $k_{co} = 10{,}000\ \text{N/mm}^2$, ℓ^* becomes

$$\ell^* \cong 0.042 + 0.88t + 4.34d \cong t + 4.34d \qquad (4')$$

Considering the relationship $I_S = \pi d^4/64$, Eq. (3) can be rewritten as follows:

$$K_S = 124{,}000 \frac{d}{(4.34 + t/d)^3} \qquad (3')$$

This simplified expression well emphasizes the dependence of the connection stiffness on the stud diameter and has an approximation similar to that of Eq. (4).

Minimum Embedding Stud Length

The connection stiffness and bearing capacity significantly depend on the values of the embedded lengths both in the concrete and in the wood. Stud lengths greater than those stemming from the collapse approach, which will be discussed later, neither affect the bearing capacity nor significantly increase the connection stiffness, which is practically equal to that of the ideal unlimited embedded length. Therefore, the minimum stud length given by the collapse approach can be used for practical application.

The stud connector maximum bearing capacity corresponds to the collapse mechanism with two plastic hinges in the stud shank. The connection resistance of a stud embedded in two different

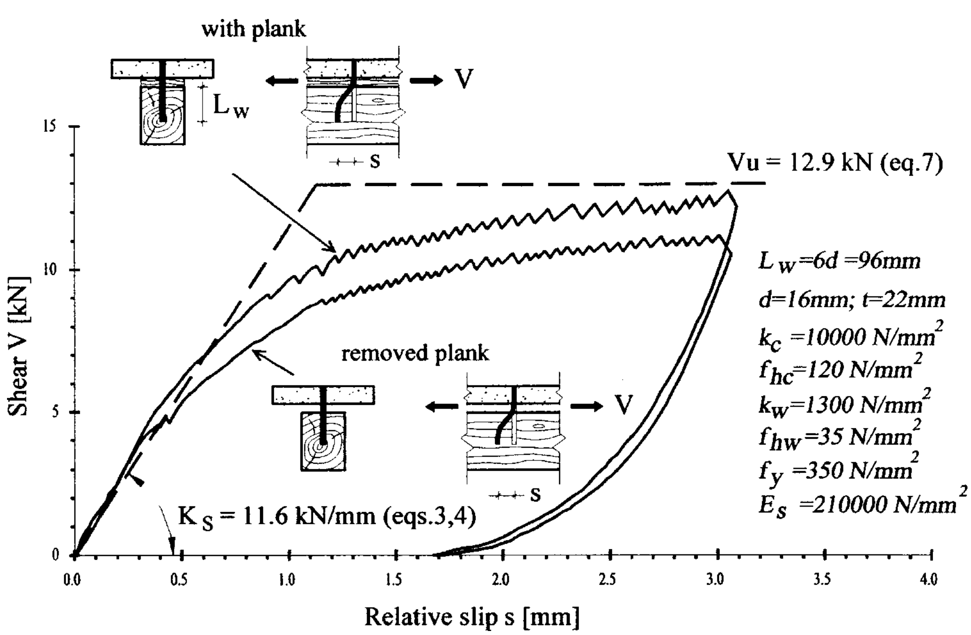

Fig. 5. Experimental (Gelfi and Giuriani 1999b) and theoretical results comparison

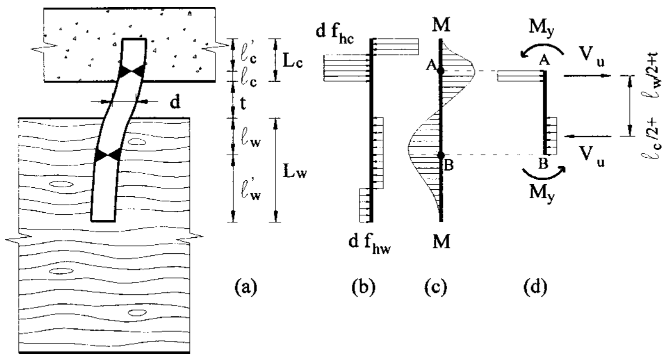

Fig. 6. Collapse mechanism and model for stud resistance calculation

materials with interposed gap t between the concrete and wood can be determined by adopting an approach which is similar to that of the "European yield model" (Johansen 1949; European 1993; Patton-Mallory et al. 1997).

The concept of effective length presented in Gelfi and Giuriani (1987) is discussed here. The stud load-bearing capacity is equal to the resultant of the wood bearing stress f_{hw} acting on the effective length ℓ_w or to the resultant of the concrete bearing stress f_{hc} acting on the effective length ℓ_c (Fig. 6). For the collapse mechanism with two plastic hinges to develop [Fig. 6(a)], minimal additional embedded lengths in wood and concrete [ℓ_c' and ℓ_w', respectively, in Fig. 6(a)] must be added to the effective length to balance the bending moments that develop in the plastic hinges A and B.

Because the shear stress in the plastic hinges A and B, where the stud bending moment is maximum [Fig. 6(c)], is zero, the equation expressing equilibrium of the stud segment AB [Fig. 6(d)] can be written as

$$V_u(\ell_c/2+\ell_w/2+t)-2M_y=0 \tag{5}$$

Since the following equations relate V_u and ℓ_c to ℓ_w:

$$V_u=f_{hw}\cdot d\cdot \ell_w; \quad \ell_c=\ell_w f_{hw}/f_{hc}=\ell_w/\beta; \quad \beta=f_{hc}/f_{hw}$$

the shank effective length in the wood can be obtained

$$\ell_w=\sqrt{\frac{2\beta}{1+\beta}}\sqrt{\frac{2M_y}{f_{hw}d}+\frac{\beta}{1+\beta}\frac{t^2}{2}}-\frac{\beta}{1+\beta}t \tag{6}$$

Considering that the stud plastic moment is given by

$$M_y=f_y d^3/6$$

where f_y = stud yield stress, Eq. (6) can be rewritten as follows:

$$\ell_w=\frac{d}{1+f_{hw}/f_{hc}}\left(\sqrt{\frac{2}{3}\frac{f_y}{f_{hw}}\left(1+\frac{f_{hw}}{f_{hc}}\right)+\left(\frac{t}{d}\right)^2}-\frac{t}{d}\right) \tag{6'}$$

The stud bearing capacity is then:

$$V_u=f_{hw}\ell_w d \tag{7}$$

In Fig. 5, the theoretical value V_u obtained from Eq. (7) is compared with experimental data by Gelfi and Giuriani (1999b).

The additional length embedded in wood ℓ_w' can be derived from the equilibrium equation:

$$f_{hw}\cdot d\cdot \ell_w'^2/4-M_y=0$$

from which

$$\ell_w'\geqslant\sqrt{\frac{4M_y}{f_{hw}d}}=d\sqrt{\frac{2}{3}\cdot\frac{f_y}{f_{hw}}} \tag{8}$$

Consequently, the whole length embedded in wood has to be larger than

$$L_w\geqslant \ell_w+\ell_w' \tag{9}$$

In the same way, the whole length embedded in concrete can be calculated. In particular, the effective length ℓ_c and the additional length ℓ_c' can be derived from Eqs. (6), (6′), and (8) exchanging index c with index w, or in a simpler way from the following relations:

$$\ell_c=\ell_w/\beta \quad \ell_c'=\ell_w'/\sqrt{\beta}$$

from which

$$L_c\geqslant \ell_c+\ell_c' \tag{9'}$$

For practical applications it is important to point out that lengths larger than those given by (9) and (9′) do not imply any increase of the shear strength. Nevertheless, if these minimum values are used, the resulting stiffness can be 30% smaller than the stiffness derived from Eqs. (1) and (2), where infinite lengths were assumed. However, a slight increase in minimum lengths allows one to establish the theoretical maximum stiffness. By adding one diameter both in wood and in concrete, 90% of the maximum stiffness can be obtained. Therefore, the following stud lengths are proposed for design:

$$L_{w,\mathrm{tot}}\geqslant L_w+d \quad L_{c,\mathrm{tot}}\geqslant L_c+d \tag{10}$$

For common material mechanical characteristics, the wood embedded length $L_{w,\mathrm{tot}}$ is about five times the stud diameter d, whereas the concrete embedded length $L_{c,\mathrm{tot}}$ is never larger than three diameters. It is worth noting that these stud lengths are enough to induce the collapse mechanism with two plastic hinges. The minimum stud total length is therefore

$$L_{\mathrm{tot}}=L_{w,\mathrm{tot}}+t+L_{c,\mathrm{tot}} \tag{10a}$$

Finally, if no planks separate the concrete slab and the wooden beam ($t=0$), Eq. (7) equals Eq. 6.2.1.f in Eurocode 5 (European 1993), which is proposed for the evaluation of the shear resistance of connection of very thick wooden elements

$$V_u=\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_y f_{hw}d} \tag{7'}$$

Connection Design Based on Deformation Control

The connection design of wood–concrete composite beams concerns mainly the evaluation of the stud diameter, length, and spacing. The usually adopted stud diameter varies in the range of 12 to 20 mm. In the case of 20–30 mm thick interposed planks, a 16 mm stud diameter appears to be a good compromise solution to assure performance while limiting the number of connectors.

The theoretical stud length stems from Eq. (10). For the sake of simplicity, for ordinary wood, concrete, and steel, embedding lengths of five and three times the diameter can be adopted in wood and in concrete, respectively. The stud spacing is usually determined for the most loaded connectors, and it can be increased where the connection shear flow decreases. For simply supported floors subjected to uniform loads, a minimum constant spacing is typically used near the supports up to a quarter of the span, and is doubled in the midzone. The minimum spacing should be evaluated according to the design target for wood–concrete composite floors which is based, as already mentioned, on deflection control under service load.

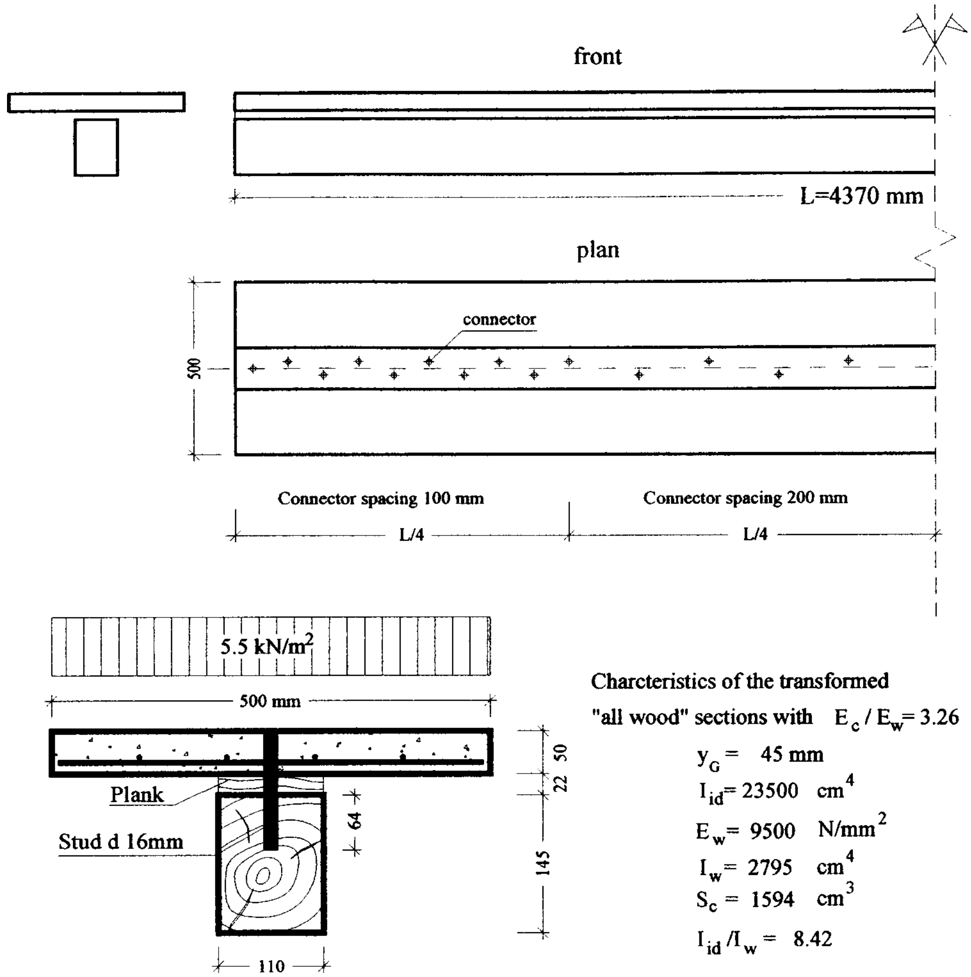

Fig. 7. Design example

The beam deflection increase produced by the slip of the connections in simply supported beams under uniform load is about eight-to-ten times the maximum slip (Ronca et al. 1991; Gubana 1995). Accordingly the minimum spacing can be derived by limiting the maximum slip. A small slip design value (for example $s_d = 0.2/0.3$ mm) induces a limited increment of the deflection which is acceptable and not greater than about 10 s_d (i.e., 2/3 mm).

Given the connector stiffness K_s [Eqs. (3) and (3′)], the design shear force V_{sd} for the most loaded connector can be obtained imposing the design slip s_d

$$V_{Sd} = K_S s_d \tag{11}$$

The design shear force V_{Sd} has to be equal to the shear flow q acting along the beam axis multiplied by the stud spacing a, that can be therefore expressed as

$$a = V_{Sd}/q = K_S s_d/q \tag{12}$$

The shear flow should be determined by means of a nonlinear approach, but, for the sake of simplicity, the classical no-slip theory can be adopted, being the shear flow redistribution negligible under service load. The maximum shear flow is therefore $q = V_b S_c / I_{id}$, where V_b = maximum shear force of the beam and, referring to the transformed "all wood" section; I_{id} = second area moment; and S_c = first area moment of the slab. Substituting this expression in Eq. (12), the minimum stud spacing becomes

$$a = \frac{K_S J_{id}}{V_{Sd} S_c} s_d \tag{13}$$

A design example is proposed for a wooden floor subjected to a total service load of 5.5 kN/m^2, with the geometrical and mechanical characteristics shown in Fig. 7 which correspond to those used in the experimental tests by Gelfi and Giuriani (1999a). The beam span and the beam spacing are 4,370 and 500 mm, respectively. The stud diameter is $d = 16$ mm and the interposed plank thickness is $t = 22$ mm. The wood beam is of Alps red spruce ($k_w = 1{,}300$ N/mm^2) and the slab is of ordinary concrete ($k_c = 10{,}000$ N/mm^2); therefore the connector stiffness is $K_s = 10{,}600$ N/mm [Eq. (3′)].

The design shear force for the connector, corresponding to a design slip $s_d = 0.3$ mm, is,

$$V_{Sd} = K_S s_d = 3.45 \ \text{kN}$$

The maximum shear force of the beam is $V_b = 6.00$ kN, yielding to a shear flow of approximately $q = 41$ N/mm. By substituting V_b, I_{id}, and S_c (as indicated in Fig. 7) in Eq. (13), a connector spacing $a = 78$ mm is obtained. In the aforementioned tested beam, the spacing a was rounded up to the value $a = 100$ mm at the supports and $a = 200$ mm in the middle.

The deflection induced by the service load according to the classical no-slip theory is 5.8 mm, corresponding to 1/749 of the span. The deflection increase due to the connection deformability is approximately 2/3 mm, so that the total deflection is about 8/9 mm, which is less than 1/500 of the span. The tested beam gave a deflection of 7.7 mm. It is worth noting that the wooden beam alone, lacking the collaborating slab, would have a much larger deflection of 49 mm ($L/90$).

Concluding Remarks

The rehabilitation of wooden floors frequently requires stiffening works to enhance their structural performance under service loads. The widely adopted technique of a thin collaborating concrete slab connected to the wooden beams allows an increase in nominal stiffness of one order of magnitude, and can approximately double the bearing capacity of the original wooden floor. Stud connectors are suitable to provide the composite beam with sufficient stiffness and bearing capacity, even when a plank is interposed between the beam and the slab. As a weak connection can compromise the structural response of the composite beam, particular attention must be paid to the connection design concerning the stud spacing, length and diameter. Toward this end, the following remarks can be drawn:

1. The simplified theoretical formulation for the connection stiffness [Eq. (3′)] makes the connection design based on the deformation control possible and suitable for practical applications. The connector spacing can be determined as a function of an acceptable value of the beam deflection increment which is caused by the slip between concrete slab and wood beam. The stud spacing can be derived from Eq. (13), by fixing a design value for the maximum slip, which is about 1/10 of the deflection increment.
2. The stud length can be determined adopting the value provided by Eq. (10*a*), which stems from the stud bearing capacity approach.
3. A stud diameter equal to 16 mm, for wooden floors with interposed plank and ordinary wood, concrete and steel, guarantees both the required stiffness and strength, and limits the number of stud connectors thus reducing the damage to the wood beam.
4. The connection stiffness theoretical formulation stems from the classical theory of the beam of unlimited length on elastic foundation but it can be applied to the study of the stud behavior if the stud lengths are at least equal to the minimum values given by Eq. (10*a*). If this requirement is met, then the differences between the solutions, based on the hypothesis of limited and unlimited length, are negligible for practical applications, being at most equal to 15%.
5. Both the theoretical connection stiffness and bearing capacity are in good agreement with experimental data.

Notation

The following symbols are used in this paper:

a = stud spacing;
d = stud diameter;
E_c, E_w, E_s = concrete, wood, and steel Young's modulus;
f_{hc}, f_{hw} = concrete and wood bearing resistance;
f_y = steel yield stress;
I_S = second area moment of the stud section;
K_S = connection stiffness;
k_c, k_w = concrete and wood Winkler foundation stiffness;
ℓ^* = ideal length;
ℓ_c, ℓ_w = concrete and wood effective lengths;
L_c, L_w = minimum stud total lengths embedded in concrete and wood;
M_y = stud plastic moment;
q = shear flow;
s = slip between concrete slab and wood beam;
t = interposed plank thickness;
t_w = wood thickness in bearing test;
V = shear force transmitted to a connector;
V_u = ultimate strength of a single stud connection; and
β = concrete and wood bearing resistance ratio (f_{hc}/f_{hw}).

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