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Out-of-Plane Strengthening of Existing Timber Floors with Cross Laminated Timber Panels Made of Short Supply Chain Beech

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Abstract: Establishing short supply chains for timber has become important especially in Italy, which is an historically wood-importer country. Timber is an environmentally friendly construction material and a potential mean to reduce carbon footprint produced every year by the building sector. In addition to its sustainability benefits, reversible strengthening interventions can be attained for existing structures. As such, timber can be efficiently used to preserve and protect historical buildings which are, due to architectural and aesthetic values, fundamental components of the Italian cultural heritage. In this study, the use and potential of novel cross-laminated (X-Lam or CLT) timber panels made of Italian hardwood (i.e., beech) for strengthening of existing timber floors is investigated. A quantitative comparison between the mechanical performances of the proposed wood-based product and common retrofitting techniques, such as double-crossed timber planks and reinforced concrete slabs, is carried out in terms of bending stiffness (which is evaluated according to Eurocode 5), influence of weight and reversibility of intervention. It is shown that CLT panels represent a good compromise/alternative for the realisation of reversible and sustainable reinforcing interventions, with rather well promising performances.

Keywords: wooden floors; composite sections; strengthening interventions; out-of-plane stiffness; Cross Laminated Timber (CLT); beech (*Fagus sylvatica* L.)

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1. Introduction

In the Mediterranean countries, a considerable part of the ancient building stock is composed of masonry buildings with timber floor and roof systems. Being the floors susceptible both to in-plane and out-of-plane loads, it is crucial to define their behaviour under the combination of these actions as they play a key role in the definition of the building vulnerability and seismic response.

Floors in-plane stiffness both with the effectiveness of the connections with the shear-walls, ensure a lateral loads redistribution and an efficient holding of the load bearing walls, improving the seismic performance of the whole building (box-behaviour) [1]. Whereas, a high out-of-plane stiffness assures the compliance of deflections limits under the action of vertical loads, which mostly burden the structure for its entire life. Unfortunately, most of the traditional timber floors have low out-of-plane and/or in-plane stiffness; this condition both with the lack of connections to the main masonry walls which led to the vulnerability to seismic action of the ancient masonry buildings and represent a significant limitation [2]. In some cases, the existing floors could require additional strengthening and stiffening in the out-of-plane direction as they were designed to bear moderate loads and, in most cases, they suffer from excessive deflections with respect to current requirements. Permanent deflection due to creep can also reach critical values [3].

To face this latter condition several strengthening techniques have been developed to increase both the bending stiffness and load-bearing capacity of existing timber floors. One of the most widespread approaches consists in increasing the inertia of the floor joists by adding a reinforcement on its upper side; the results is a T-composite beam section having as web the original timber and as flange the new element, connected to the existing beam by the mechanical connections [4]. The aforementioned fasteners ensure the composite behaviour; while, their type and features are the main factors that govern the efficiency of the intervention. The solution of adding a top reinforced concrete slab of 40–50 mm height is one of the most widespread used for new floors and refurbishing and enhancing the performance of existing timber floors since it was developed in the 70 s.

Different mechanical connection systems have been applied in practice [5], such as for example dowel type fasteners, notches [6], studs [7], glued in rods [8], plates [9] or even the combination of various fastening systems [10]. Despite the observed advantages of hybrid timber-to-concrete (TTC) systems, like for example their increased load-bearing capacity, but also airborne sound transmission, structural fire rating, seismic performance and thermal mass, TCC solutions have also some significant drawbacks. Among others, there are differential deformations (namely those owing to shrinkage of concrete and moisture variation on timber [11], the lack of clear guidelines for their design (long term behaviour still remains a complex issue at the moment) and especially the significant increase in permanent loads, invasiveness, and low reversibility of intervention, which makes them unsuitable for use on valuable historic buildings [4].

To overcome all these disadvantages, the use of timber elements, instead of concrete slabs, has been proposed as an alternative, mechanical efficient option to strengthen existing timber floors [12]. Timber-to-timber (TTT) composite sections have undeniable advantages, compared to the aforementioned TTC sections. More precisely, these can be listed as the use of traditional materials and the 'dry' assembly methods, which promote the compatibility between materials as well the intervention reversibility and/or recoverability. Another relevant benefit concerns the characteristics of timber in terms of natural carbon sink and renewable material [13], which make this type of reinforcement technique rather efficient from an energetic/environmental viewpoint.

One of most awkward and discussed issues concerns the historical value preservation of building heritage, which has been often affected by invasive retrofit interventions aimed at reducing the carbon dioxide emissions or the seismic risk [14–16]. In particular, in the last decades, the indiscriminate demolition and replacement of timber floors and roofs with reinforced concrete (RC) slabs and steel beams appeared as really common strategy. It is thus clear that this practice has relevant negative consequences in terms of (i) increased seismic forces, given their inertial characteristics, (ii) low deformations and, consequently, low energy dissipation of RC member compared to timber elements [13], and (iii) additional weight on the existing wall/foundation system, which could not be able to provide sufficient low-bearing capacities [17]. The most widespread solutions adopted to realize TTT sections involve the use of different types of connectors [3,18–20] to join new timber planks or CLT panels to the existing floors. Both the TTC and TTT strengthening techniques are realized by minimizing the thickness of the reinforced floors, in order to preserve the existing floor level and internal room height. Nevertheless, despite the numerous advantages, the elastic moduli of timber products are relatively low compared to concrete and/or steel, hence TTT floors are characterized by very low bending stiffness, which makes them prone to excessive short- and long-term deflections, as well as sensitivity to human-induced vibrations [21,22]. Increasing the thickness of flat timber slabs is the easiest way to satisfy the deflection and vibration control requirements for long-span floors, but the application of excessively thick solid timber floors is not economically justifiable and minimally structurally efficient [22].

In this framework, the importance of CLT panels has become so relevant that several studies [23–27], have concerned panels made with different wood species compared to fir and spruce, which are currently the most used species for CLT industrial productions.

An interesting research case was carried out in Italy, where Sardinian maritime pine was employed to produce CLT panels. Preliminary tests carried out to evaluate the physical and mechanical properties, and thus to demonstrate the suitability of this locally grown species to produce CLT panels, can be found in [28]. To note that CLT panels made of some native hardwood species, such as beech, yellow-poplar and tropical hardwood (i.e., rubberwood, batai, etc.), were found as very performing. These literature evidences are particularly relevant, considering the impact that hardwood and new bonding techniques [29] could have on the production of CLT panels. In particular, it was highlighted in [26] the excellent out-of-plane and shear mechanical performance of the examined homogeneous beech or hybrid Corsican pine-beech CLT specimens. By comparing (via numerical simulations) the mechanical response of hardwood panels with those made of traditional C24 spruce, they proved to experience lower deflections and to be less deformable in both bending and shear loading conditions.

Therefore, in the context of retrofitting interventions of existing timber floors, the application of novel homogeneous Italian beech CLT panels, which were experimentally investigated by Sciomenta et al. [26], as reinforcing element, is further considered and addressed in present study. For quantitative comparative purposes, moreover, the predicted mechanical performances are assessed towards some common strengthening techniques of typical use for wooden floors.

2. Reinforcing Interventions

Since wood is light and workable, through centuries it has been largely used for the realization of floors for ordinary buildings, and even in those with monumental importance. In the framework of typical timber floors, two main parts (frame and slab) can be variably designed, as a function and class of use of the primary structure.

In Italy, the frame is generally made of one or two orders of beams. In the second case, the primary order is composed of beams, while the secondary order consists of joists. The function of the frame is to bear structural permanent, non-structural permanent, and accidental loads, acting at the floor intrados and extrados. The slab usually presents a different structure which locally varies with the Italian region: in some cases, it is constituted by one or more layers of timber planks, while in other cases is consists of tiles. The primary roles of slab are to distribute the different design loads among the frame components and to increase the lateral stiffness of the frame, thus contributing to the distribution of the horizontal actions towards the main structural vertical elements. In this paper, a one-way system for timber floors (made of sawn wood joists and planks) is taken into account.

Three different strengthening techniques are considered to improve the bending stiffness for a case-study, existing timber floor consisting of sawn timber joists (with cross section of 160 mm by 200 mm, and spacing of 350 mm), and an upper layer of boards (30 mm in thickness).

All the interventions include the use of a reinforcement above the existing floor, see Figure 1, which is connected by means of screws.

The difference lies in the nature of strengthening element, which is respectively:

- a double-crossed timber planking, made of hardwood, with a mean modulus of elasticity parallel to the grain $E_{mean,II}$ = 14,237 MPa and a mean density ρ_{mean} = 756 kg/m^3;
- a CLT panel made of hardwood, with $E_{mean,II}$ = 14,237 MPa and ρ_{mean} = 756 kg/m^3 [26];
- a RC slab with secant modulus of elasticity E_c = 31,447 MPa and ρ_{mean} = 2500 kg/m^3.

To note that the first technique is fully reversible and commonly used in all practical situations, where a minimally invasive intervention—but mechanical efficiency—is required.

In contrast, the third system is clearly not reversible and can be used, where high floor stiffening is necessary, only in those buildings which are not protected by the Fine Art Monument. The second solution is halfway between (i) and (iii), as it constitutes a reversible intervention and anyway allows a quite significant floor strengthening.

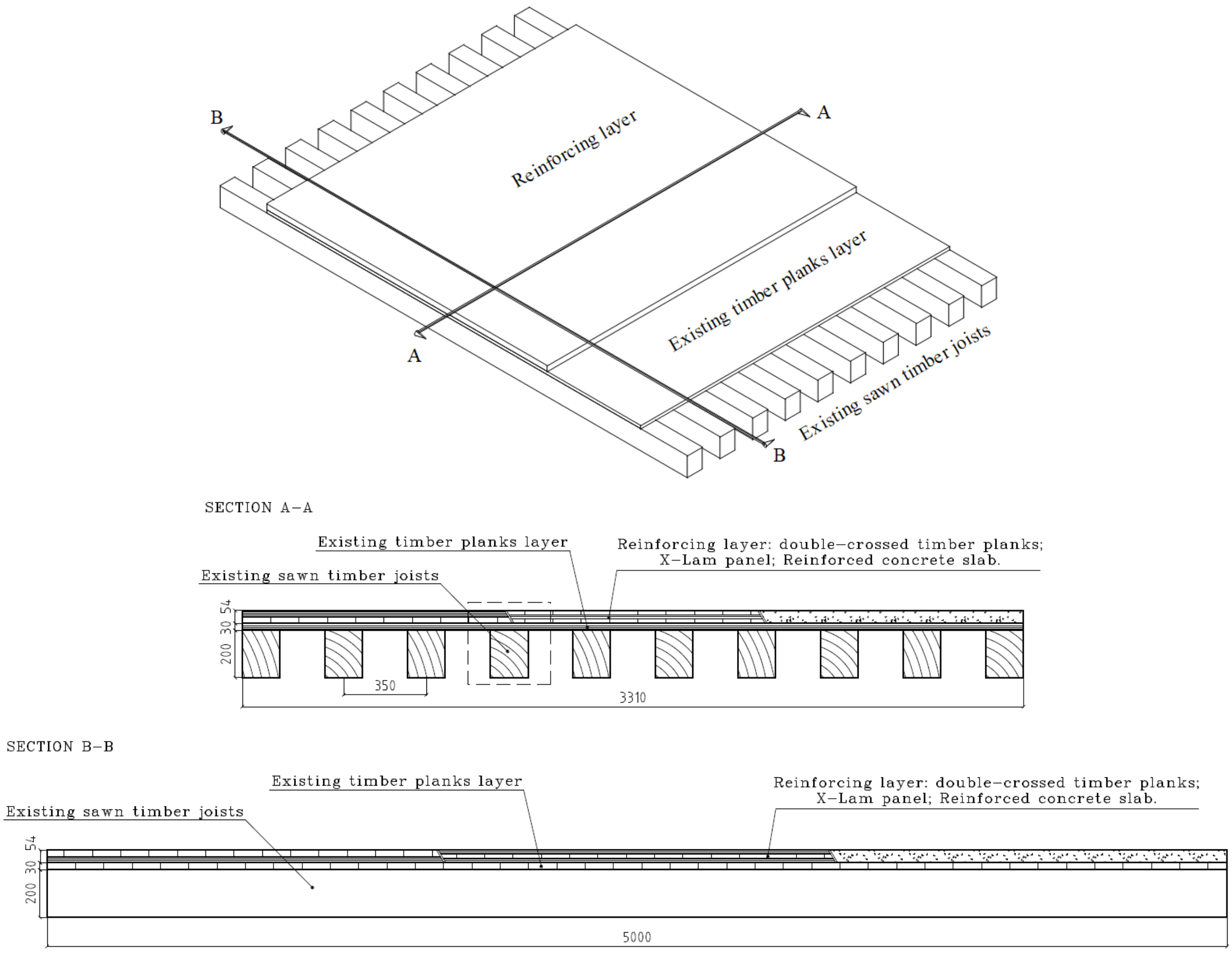


Figure 1. Three-dimensional view and cross-section details of the case-study floor object of intervention (dimensions in mm).

With the aim to mechanically compare the effect of three different reinforcing systems with the same dimensions, the thickness of herein proposed novel CLT panels, equal to 54 mm, is taken into account for the above options (i), (ii) and (iii). The connection between the existing timber floor and the reinforcement is assumed flexible, as it is realized by means of screws with variable diameter and spacing as described in the following paragraphs.

3. Comparison of Selected Intervention Techniques

3.1. Calculation Approach

The selected intervention techniques were applied to the case-study floor system schematized in Figure 1. Considering a single joist for the 5 m span floor, see Figure 2, the bending stiffness was first evaluated according to the mechanically jointed beams theory, as specified by Annex B of Eurocode 5 [30].

Such a linear elastic theory assumes (i) a simply supported boundary condition for the beam, (ii) a connection by mechanical fasteners with slip modulus K, between the individual parts of the composite section, (iii) a constant or uniformly varying spacing of fasteners, and (iv) a sinusoidal or parabolic distribution of bending moment.

In addition, the percentage weight increment was also calculated to underline the influence of reinforcing system on the permanent load acting on the floor, and thus to use it as a further influencing parameter for mechanical performance assessment.

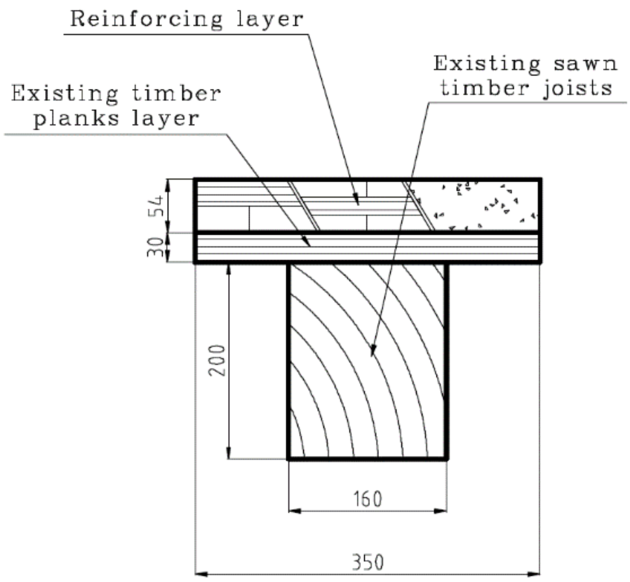


Figure 2. Cross-section of the examined composite system (dimensions in mm).

3.2. Slip Moduli and Performance

Considering the flexible connection between the existing timber floor and the reinforcing system, the slip modulus (both for ULS and SLS) increases progressively, going from the double-crossed planking to the RC slab.

For timber-to-timber connections, in particular, the mean density (ρ_{meam}) is calculated considering the mean densities of the two jointed timber members ($\rho_{m,1}$ and $\rho_{m,2}$), as in Equation (1):

$$\rho_{mean} = \sqrt{\rho_{m,1} \cdot \rho_{m,2}} \tag{1}$$

For concrete-to-timber connections, the mean density is based on the mean value of the timber element, and K_{ser} should be multiplied by 2. Assuming a mean density of the existing timber elements of 500 kg/m^3, the resulting mean densities are 615 kg/m^3 in the cases of D40 strength class timber planks and novel beech CLT panel, and 1000 kg/m^3 in the case of the RC slab.

For screw diameters (d) equal to 8, 10, and 12 mm, the corresponding slip moduli K_{ser} (for SLS) and K_u (for ULS) were thus evaluated according to Eurocode 5 [11], see Equations (2) and (3):

$$K_{ser} = \rho_{mean}^{1.5} \frac{d}{23} \tag{2}$$

$$K_u = \frac{2}{3} K_{ser} \tag{3}$$

In doing to, a reduction coefficient for K_{ser} (equal to 0.7) was also taken into account, in order to include a stiffness reduction due to the presence of existing planking [31–33]. As shown in Table 1, the so calculated slip moduli change depending both on the density of reinforcement and on the screw diameter.

Table 1. Calculated slip moduli for different reinforcements.

Reinforcing System	d = 8 mm		d = 10 mm		d = 12 mm	
	K_{ser} [N/mm]	K_u [N/mm]	K_{ser} [N/mm]	K_u [N/mm]	K_{ser} [N/mm]	K_u [N/mm]
(i) Timber/(ii) CLT	3712	2475	4640	3093	5568	3712
(iii) RC	5444	3630	6805	4537	8167	5444

Assuming the RC slab as reinforcing system, it can be seen that the K_{ser} slip modulus, and consequently the K_u slip modulus, increases of about 47%, compared to timber planks or CLT panels. A further comparison between the aforementioned formulation and the new one which has been recently developed in the draft of Eurocode 5, Annex K [34] was thus carried out. In particular, Equation (4) was used to evaluate the mean slip modulus per fastener in case of timber-to-timber connections:

$$K_{SLS} = 0.12\left(\rho_{mean,1} t_{h,1}^{0,1} + \rho_{mean,i}^{1,1} + \rho_{mean,2} t_{h,2}^{0,1}\right) d^{1,1} \quad (4)$$

where:

$\rho_{mean,1}$ and $\rho_{mean,2}$ is the mean density of member 1 or 2;
$\rho_{mean,i}$ is mean density of the interlayer;
$t_{h,1}$ and $t_{h,2}$ is the thickness of member 1 or 2;
d is the fastener diameter.

The so calculated values were greater than those in Table 1 of about 10–12%, affecting the bending stiffness of the composite section with a minimal overestimation of about 1–2%. For this reason, the slip moduli in Table 1 were adopted in the following, as they are plausible by comparison with the values determined experimentally in the literature, as for example in the case of hardwood-softwood connection with interlayer [19].

3.3. Bending Stiffness

For the composite section in Figure 2, composed of a single joist with 160 mm by 200 mm cross-section, an upper layer of timber planks, and a strengthening system, comparative bending calculations were carried out in accordance with Equations (5) and (6):

$$(EI)_{eff} = \sum_{i=1}^{2}\left(E_i I_i + \gamma_i E_i A_i a_i^2\right) \quad (5)$$

$$\gamma_i = \left[1 + \frac{\pi^2 E_i A_i s_i}{(K_i l^2)}\right]^{-1} \quad (6)$$

with:

(EI)$_{eff}$ = effective bending stiffness of the composite section;
E_i = modulus of elasticity of the *i*-th element;
I_i = second moment of area of the *i*-th element;
γ_i = gamma coefficient of the *i*-th element;
A_i = cross-sectional area of the *i*-th element;
a_i = distance between the geometric centre of the *i*-th element and the centre of the composite section;
s_i = spacing of fasteners;
K_i = K_{ser} for the serviceability limit state and K_u for the ultimate limit state calculations;
l = span of joist.

Since the joist as in Figure 1 are considered simply supported, the connection system is made of 8, 10, or 12 mm diameter screws with variable spacing, which is set equal to a minimum value (s_{min}) near the end-restraints and to a maximum value (s_{max}) at mid-span. The limit cases of absence of connection (K = 0) or fully rigid connection (K = ∞) are also presented in Table 2, in terms of limit bending stiffness capacities for the examined composite section.

Three cases of flexible connections (K_1, K_2, and K_3) with different values of s_{max} and s_{min} are analysed. K_1, K_2, and K_3 present a maximum spacing of 200 mm, 150 mm, and 100 mm, and a minimum spacing of 150 mm, 100 mm, and 50 mm, respectively.

As a major result, the calculated slip moduli affect the corresponding γ_1 coefficients according to Equation (6). In particular, the coefficient decreases with the increase of the slip modulus, as shown in Table 3 for the intermediate flexible case (K_2). The γ_1 values in the case of RC slab are indeed similar to those determined in earlier studies [11].

Table 2. Limit cases of calculated bending stiffness: $(EI)_0$ for $K = 0$ and $(EI)_\infty$ for $K = \infty$.

Reinforcing System	$(EI)_0$ [Nmm^2]	$(EI)_\infty$ [Nmm^2]
None	8.54×10^{11}	8.95×10^{11}
(i) Timber plank	8.62×10^{11}	2.60×10^{12}
(ii) CLT panel	9.19×10^{11}	3.03×10^{12}
(iii) RC slab	9.98×10^{11}	3.88×10^{12}

Table 3. Coefficients for different reinforcements (K_2).

Reinforcing System	*d* = 8 mm		*d* = 10 mm		*d* = 12 mm	
	γ_1 (SLS) [-]	γ_1 (ULS) [-]	γ_1 (SLS) [-]	γ_1 (ULS) [-]	γ_1 (SLS) [-]	γ_1 (ULS) [-]
(i) Timber	0.38	0.29	0.44	0.34	0.48	0.38
(ii) CLT	0.24	0.17	0.28	0.21	0.32	0.24
(iii) RC	0.17	0.12	0.20	0.15	0.24	0.17

Therefore, K_3 is the stiffer option, and all the three flexible cases are included between the limit cases, leading to bending stiffnesses greater than the case without connection and lower than the case of rigid connection. Both the ultimate limit state (ULS) and the serviceability limit state (SLS) are considered, see Tables 4 and 5, using the slip moduli K_u ad K_{ser}, respectively, in the evaluation of the coefficients γ_i.

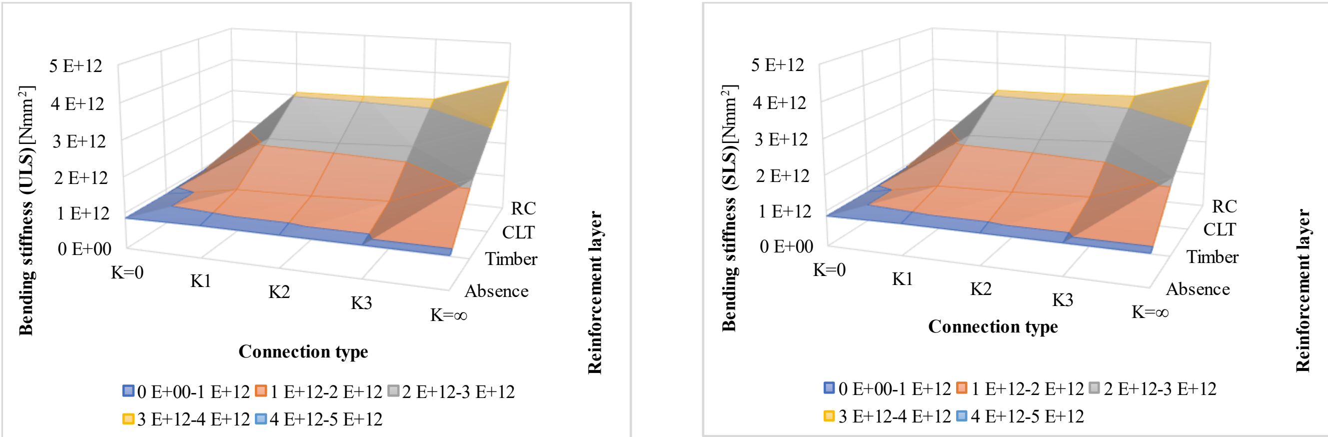
Table 4. Flexible connections (ULS): calculated $(EI)_{eff}$ for K_1, K_2, and K_3.

Reinforcing System	$(EI)_{eff}$ [Nmm^2]								
	K_1			K_2			K_3		
	d = 8 mm	*d* = 10 mm	*d* = 12 mm	*d* = 8 mm	*d* = 10 mm	*d* = 12 mm	*d* = 8 mm	*d* = 10 mm	*d* = 12 mm
None		8.93×10^{11}			8.93×10^{11}			8.94×10^{11}	
(i) Timber	1.42×10^{12}	1.45×10^{12}	1.48×10^{12}	1.47×10^{12}	1.51×10^{12}	1.54×10^{12}	1.57×10^{12}	1.61×10^{12}	1.65×10^{12}
(ii) CLT	2.13×10^{12}	2.16×10^{12}	2.19×10^{12}	2.18×10^{12}	2.22×10^{12}	2.25×10^{12}	2.28×10^{12}	2.33×10^{12}	2.37×10^{12}
(iii) RC	3.09×10^{12}	3.11×10^{12}	3.12×10^{12}	3.12×10^{12}	3.14×10^{12}	3.16×10^{12}	3.19×10^{12}	3.22×10^{12}	3.25×10^{12}

Table 5. Flexible connections (SLS): calculated $(EI)_{eff}$ for K_1, K_2, and K_3.

Reinforcing System	$(EI)_{eff}$ [Nmm^2]								
	K_1			K_2			K_3		
	d = 8 mm	*d* = 10 mm	*d* = 12 mm	*d* = 8 mm	*d* = 10 mm	*d* = 12 mm	*d* = 8 mm	*d* = 10 mm	*d* = 12 mm
None		8.93×10^{11}			8.93×10^{11}			8.94×10^{11}	
(i) Timber	1.48×10^{12}	1.51×10^{12}	1.54×10^{12}	1.54×10^{12}	1.58×10^{12}	1.61×10^{12}	1.65×10^{12}	1.69×10^{12}	1.72×10^{12}
(ii) CLT	2.19×10^{12}	2.22×10^{12}	2.25×10^{12}	2.25×10^{12}	2.29×10^{12}	2.33×10^{12}	2.37×10^{12}	2.43×10^{12}	2.47×10^{12}
(iii) RC	3.12×10^{12}	3.15×10^{12}	3.17×10^{12}	3.16×10^{12}	3.19×10^{12}	3.22×10^{12}	3.25×10^{12}	3.29×10^{12}	3.33×10^{12}

As shown in Table 4 and Figure 3a, the ULS bending stiffness of the composite section with 8 mm screws increases from 59% (K_1) to 76% (K_3) with the double-crossed timber planks system, from 139% (K_1) to 155% (K_3) with the homogeneous beech CLT panel, and from 246% (K_1) to 257% (K_3) with the RC slab.

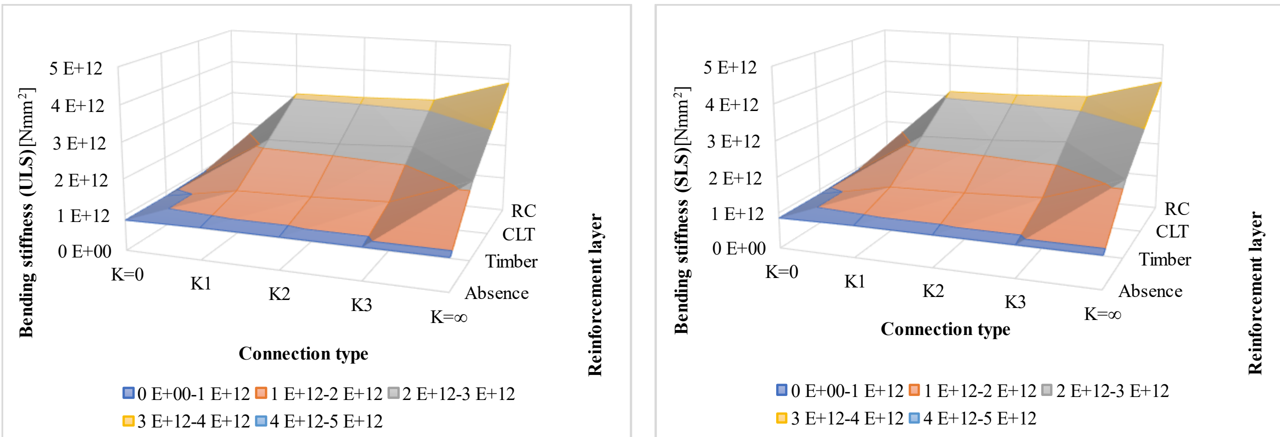


(**a**) (**b**)

Figure 3. Example of bending stiffness variation (8 mm diameter screws): ULS (**a**); SLS (**b**).

The SLS bending stiffness, see Table 5 and Figure 3b, shows a greater increment, with a range from 66% (K_1) to 85% (K_3) in the case of crossed timber planking, from 145% (K_1) to 165% (K_3) in the case of X-Lam panel, and from 250% (K_1) to 264% (K_3) in case of RC slab.

The ULS bending stiffness for the composite section with 10 mm diameter screws, see Figure 4a, increases from 63% (K_1) to 81% (K_3) with the double-crossed timber planks system, from 142% (K_1) to 161% (K_3) with the homogeneous beech CLT panel, and from 248% (K_1) to 260% (K_3) with the RC slab.



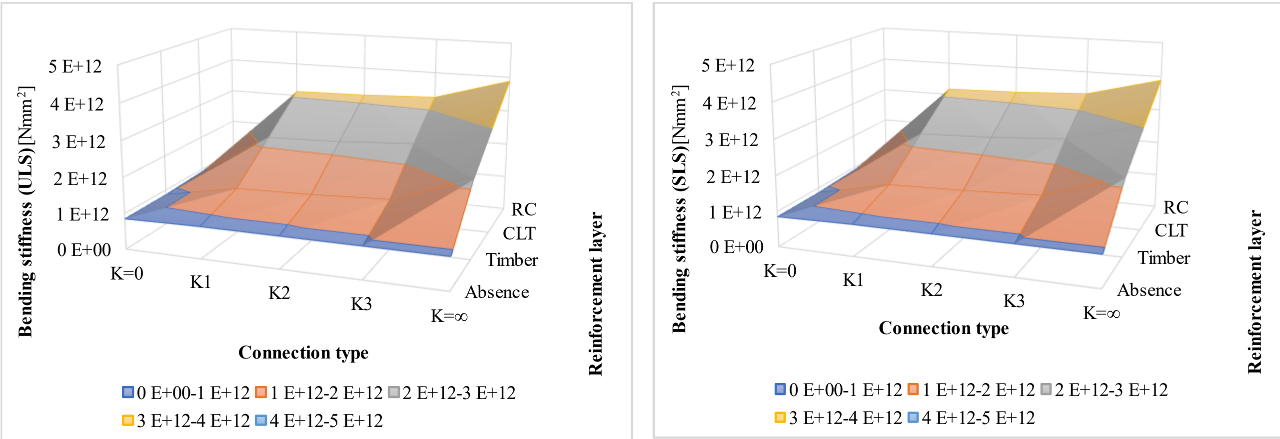
(**a**) (**b**)

Figure 4. D graphs of bending stiffness variation (10 mm diameter screws): ULS (**a**); SLS (**b**).

The SLS bending stiffness in Figure 4b shows a greater increment, with a range from 69% (K_1) to 90% (K_3) in case of crossed timber planking, from 149% (K_1) to 172% (K_3) in case of CLT panel, and from 252% (K_1) to 268% (K_3) for of RC slab.

The ULS bending stiffness in Figure 5a for the composite section with 12 mm diameter screws increases from 66% (K_1) to 85% (K_3) with the double-crossed timber planks system, from 145% (K_1) to 165% (K_3) with the homogeneous beech CLT panel, and from 250% (K_1) to 264% (K_3) with the RC slab.

The SLS bending stiffness in Figure 5b shows a greater increment, with a range from 73% (K_1) to 92% (K_3) in the case of crossed timber planking, from 153% (K_1) to 177% (K_3) in case of CLT panel, and from 255% (K_1) to 272% (K_3) for the RC slab.



(**a**) (**b**)

Figure 5. D graphs of bending stiffness variation (12 mm diameter screws): ULS (**a**); SLS (**b**).

With respect to the limit cases with $K = 0$ and $K = \infty$, it is also worth to note that the flexible cases K_1, K_2 and K_3 present bending stiffness values which are necessarily comprised between the above limits, but are rather similar to each other for the same reinforcing system. The typical bending stiffness variation and sensitivity is shown in Figure 6.

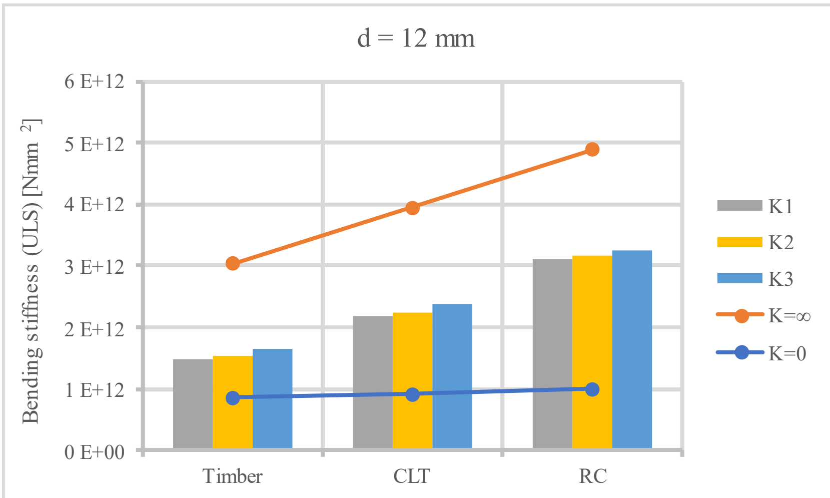


Figure 6. Comparison between limit cases and flexible cases, in terms of ULS bending stiffness for 12 mm diameter screws.

Analysing the results in terms of slip moduli, the values assessed according to EC5 underestimates the experimental values expected from tests in similar timber-to-timber joints [19]. Consequently, even the bending stiffness is underestimated, due to the approximations of the method by [30] and on the presence of the interlayer [31,33], in agreement with other studies [35]. The three flexible cases K_1, K_2, K_3, which represent three possible and realistic choices of screws spacing, were considered to evaluate their influence on the effective bending stiffness. The estimated values in the case of CLT panel as reinforcement, in particular, are greater than those in the case of double-crossed timber planking, although the thickness and density of the material are the same, because of the presence of two layers with the grain direction parallel to the span direction. Considering the RC slab, the Eurocode 5 provides approximated formulas to determine the mean density to be used to evaluate the slip moduli and the consequent bending stiffness. This kind of retrofit technique, although has better mechanical performances, is however not always applicable in historic buildings, where indeed reversible interventions are preferable [3,4,12,13].

A parameter regarding the performance of the flexible connections, indicated as η, can be further analysed, with respect to the two limit cases K= 0 and K= ∞, by calculating it according to Equation (7):

$$\eta = \frac{(EI)_{eff} - (EI)_0}{(EI)_\infty - (EI)_0} \quad (7)$$

The so obtained η values, calculated for the K_2 case, are presented in Table 6 and Figure 7.

Table 6. Values for different reinforcement (K_2).

Reinforcing System	d = 8 mm		d = 10 mm		d = 12 mm	
	η (SLS) [-]	η (ULS) [-]	η (SLS) [-]	η (ULS) [-]	η (SLS) [-]	η (ULS) [-]
(i) Timber	0.39	0.35	0.41	0.37	0.43	0.39
(ii) CLT	0.63	0.60	0.65	0.61	0.67	0.63
(iii) RC	0.75	0.74	0.76	0.74	0.77	0.75

Figure 7. Typical trend of η parameter for 12 mm diameter screws K_2 case.

The η parameter, which usually does not exceed the unit, increases with increasing density of the reinforcing system and with decreasing screws spacing and, consequently, with increasing of the slip modulus.

Therefore, assuming constant effective screws spacing (K_2 case), it increases both going from the timber planks reinforcement to the RC slab and going from ULS to SLS.

3.4. Weight

The considered reinforcing material densities were set in 756 kg/m^3 for double-crossed timber planking and novel beech CLT panel and 2500 kg/m^3 for RC slab.

Regarding the composite system, the additional reinforcing element constitutes about the 40% of the structural permanent loads in the two timber-based solutions, and about the 69% in the case of the RC slab, see Figure 8.

Considering the flexible cases with 12 mm diameter screws, both the weight and bending stiffness increasements are noteworthy to be highlighted, so as to identify a suitable compromise between the most efficient mechanical strengthening technique for the existing floor and the corresponding increase of permanent loads.

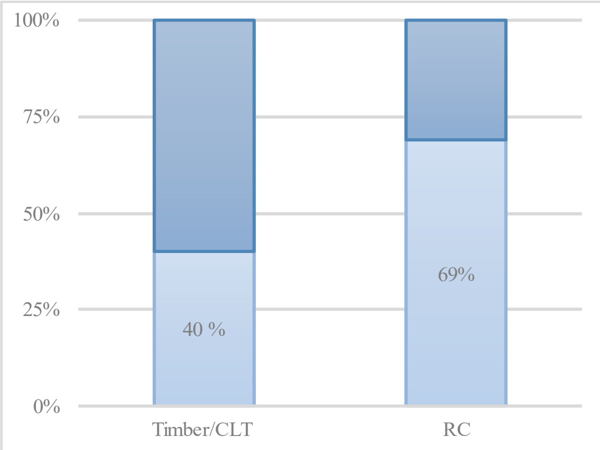


Figure 8. Percentage weight variation of reinforcement, compared to the total permanent weight of timber floor with reinforcing intervention.

In the K_1 case, the ULS bending stiffness increases of 66%, 145% and 250% for double-crossed timber planks, novel CLT panel, and RC slab, respectively. In the K_2 case, the ULS bending stiffness increases of 73%, 152% and 254% for double-crossed timber planks, novel CLT panel, and RC slab, respectively. In the K_3 case, the ULS bending stiffness increases of 85%, 165% and 264% for double-crossed timber planks, novel CLT panel, and RC slab, respectively.

To sum up, see Figure 9, the use of timber planks as reinforcing element determines a ULS bending stiffness equal to $(EI)_{eff}$ = 1.65 × 10^{12} Nmm^2 in the stiffer case (K_3), which is about 33% lower than the CLT panel with K_1 value (($(EI)_{eff}$ = 2.19 × 10^{12} Nmm^2).

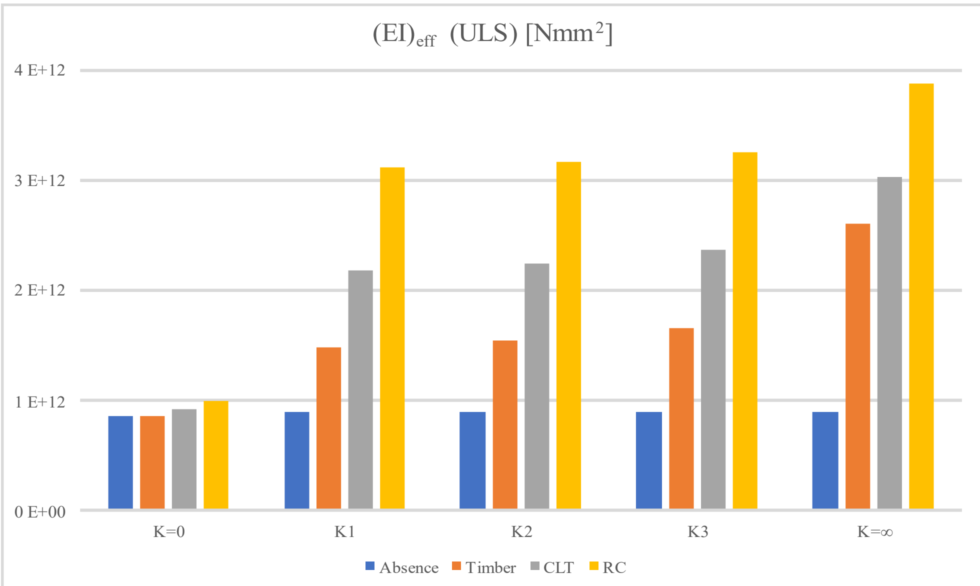


Figure 9. Quantitative comparison of selected reinforcement techniques in terms of $(EI)_{eff}$ at ULS.

On the other hand, the CLT panel in stiffer case (K_3) leads to a ULS value of $(EI)_{eff}$ = 2.37 × 10^{12} Nmm^2, which is lower down to about 32% than the RC slab in K_1 case, with $(EI)_{eff}$ = 3.12 × 10^{12} Nmm^2.

Similarly, the SLS bending stiffness of the CLT panel in K_1 case is larger of about 31% than timber planks with K_3 value, and the stiffer CLT panel case (K_3) value is lower of about 28% than the RC slab in K_1 case.

Considering the stiffer case K_3 with 12 mm diameter screws, the ULS bending stiffness is $(EI)_{eff} = 2.37 \times 10^{12}$ Nmm^2 for CLT panel as reinforcement, against $(EI)_{eff} = 1.65 \times 10^{12}$ Nmm^2 for timber planks and $(EI)_{eff} = 3.25 \times 10^{12}$ Nmm^2 for RC slab.

In this regard, from the comparison in terms of bending stiffness and weight percentage increase, shown in Figure 10, it is clear that the CLT panel allows to obtain a significant increase of ULS bending stiffness, and also efficiently limit the increase of structural permanent loads on single joists, at around 40%.

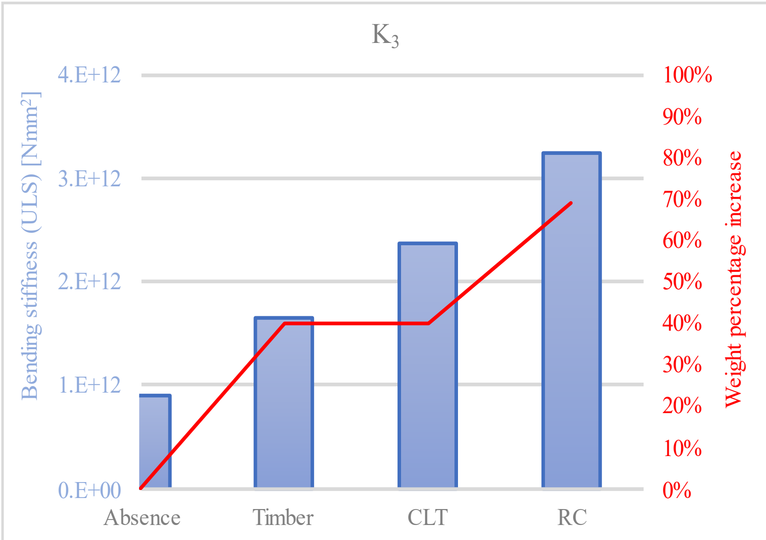


Figure 10. ULS bending stiffness-to-weight increase for K_3 case.

4. Bending Moment and Normal Stresses

Considering the K_2 flexible case with 12 mm diameter screws, it can be important to show how the chosen reinforcing system affects the moment distribution between the joist and the reinforcement itself along the span of the single joist. Alongside the structural permanent loads due to the joists, the existing floor and the reinforcing system, non-structural permanent loads and accidental loads are considered. The design distributed load per single joist considered is equal to 1.90 kN/m, and, for a span of 5 m, it produces a reaction at the supports equal to 4.75 kN for a simply supported beam. Analysing the bending moment distribution $M_n(x)$ along the 5 m span, it can be divided between the elements (reinforcement and joist) of the composite section. Using subscript n = 1 for the reinforcing element and subscript n = 2 for the joist, moments and axial forces can be determined according to the following Equations (8)–(11):

$$M_1(x) = \frac{E_1 I_1}{(EI)_{eff}} M(x) \tag{8}$$

$$M_2(x) = \frac{E_2 I_2}{(EI)_{eff}} M(x) \tag{9}$$

$$N_1(x) = \frac{\gamma_1 E_1 A_1 a_1}{(EI)_{eff}} M(x) \tag{10}$$

$$N_2(x) = \frac{\gamma_2 E_2 A_2 a_2}{(EI)_{eff}} M(x) \tag{11}$$

In order to respect the condition expressed by Equation (12):

$$M(x) = M_1(x) + M_2(x) + N_1(x) \cdot a_1 + N_2(x) \cdot a_2 \tag{12}$$

Considering the bending moment distributions (Figure 11 left graphs), M_1 for the $K = 0$, K_2, and $K = \infty$ cases increases when the reinforcement stiffness increases, and M_2 decreases progressively. Consequently, normal stresses can be evaluated according to Annex B of Eurocode 5 [11], Equations (13) and (14):

$$\sigma_i = \frac{\gamma_i E_i a_i M}{(EI)_{eff}} \tag{13}$$

$$\sigma_{m,i} = \frac{0.5\ E_i h_i M}{(EI)_{eff}} \tag{14}$$

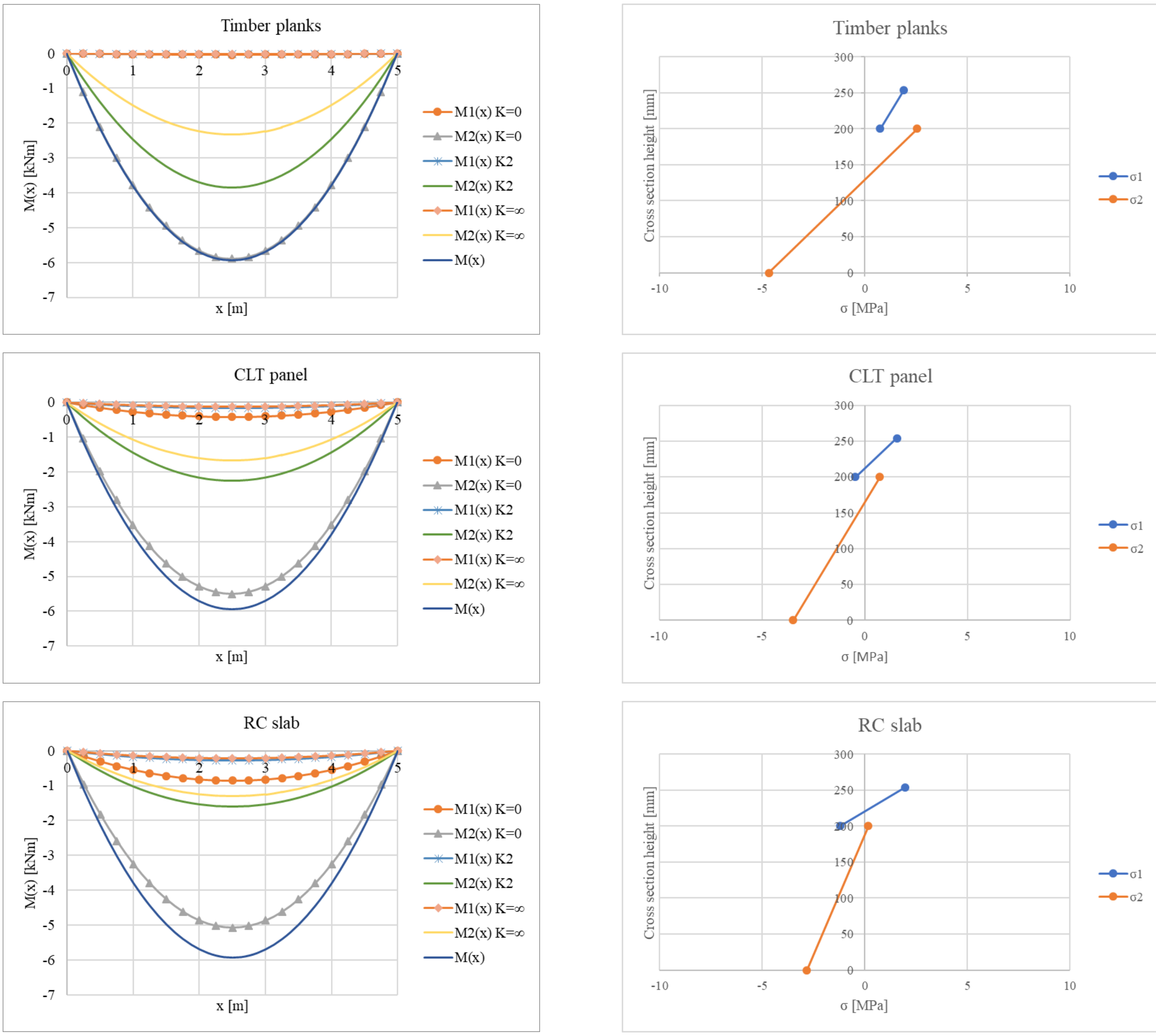


Figure 11. Bending moment distributions along the span for the $K = 0$, K_2, and $K = \infty$ cases, and maximum normal stress distributions along the cross-section height for each reinforcement.

Considering the most stressed cross section in terms of bending moment, i.e., the one in the middle of the span, the variation of the normal stresses can be analysed along the height of the cross section, Figure 11 right graphs. Comparing the different reinforcements, the normal stresses in the joist, σ_2, decreases as the stiffness of the reinforcing system increases, and even in this comparison CLT panel determines an intermediate behaviour between timber planks and RC slab.

5. Conclusions

The sustainable exploitation of wood resource for products other than firewood, where the latter currently represent its main use in Italy, can increase the interest in the management of rural areas, which are usually underdeveloped and abandoned. The use of short supply chain wood-based products for the restoration of historical and ancient buildings can be a significant practice and it could lead to several benefits over time such as the economic enhancement of local forests, the preservation of the architectural heritage with minimally invasive interventions, and sustainability advantages due to the use of wood in the construction sector.

To promote this kind of applications, comparative analysis regarding the out-of-plane strengthening of existing one-way floor were performed. Three different retrofit interventions were considered: double-crossed timber planks, CLT panels and reinforced concrete slab. The first and the third are two of the most commonly used techniques; the second one considers thin homogeneous beech cross-laminated timber panels recently mechanically characterized. In order to focus the comparison on the nature of the reinforcing elements, the thickness of the reinforcements was assumed equal to that of the CLT panels. The comparison in terms of out-of-plane bending stiffness was conducted by evaluating slip moduli and effective bending stiffness according to the mechanically jointed beam theory, Annex B of Eurocode 5, both at ULS and at SLS.

A total of three flexible cases, i.e., K_1, K_2 and K_3 slip moduli for screwed connections characterized by different spacing along the span of the member, and three screw diameters, were considered. In addition, the performance of the flexible connections was assessed with respect to the two limit cases corresponding to the absence of any connection ($K = 0$) and to the presence of a fully rigid connection ($K = \infty$). From the comparisons proposed in this work, it was shown that the choice of CLT panels leads to a range of SLS bending stiffness increase from 145% (K_1 and d = 8 mm) to 177% (K_3 and d = 12 mm), against the ranges from 66% (K_1 and d = 8 mm) to 92% (K_3 and d = 12 mm) for double-crossed timber planks and from 246% (K_1 and d = 8 mm) to 272% (K_3 and d = 12 mm) for the RC slab solution. Although based on the use of hardwood, this solution can also minimize the weight increase for the examined geometry, in about the 40% part of the structural permanent loads, against the 69% term determined for the RC slab, and it allows to realize a fully reversible intervention.

Even though the RC slab leads to the greatest bending stiffness values, this solution is in fact not always applicable to existing buildings, especially when subjected to superintendence of architectural heritage restrictions. In addition, concrete and steel are considered less environmentally friendly than wood, and for this reason are often less preferable in the construction sector, in order to pursue sustainability goals.

Due to the growing interest in retrofitting solutions based on engineered wood products, the application of novel Italian beech CLT panels proved to represent a significant alternative for the retrofitting of existing timber floors, especially when belonging to the built cultural heritage. In this way, even further innovative structural applications could be detected and optimised for local species, specifically to beech and in general to hardwood, which both represent a consistent percentage of the Italian wooden areas.

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