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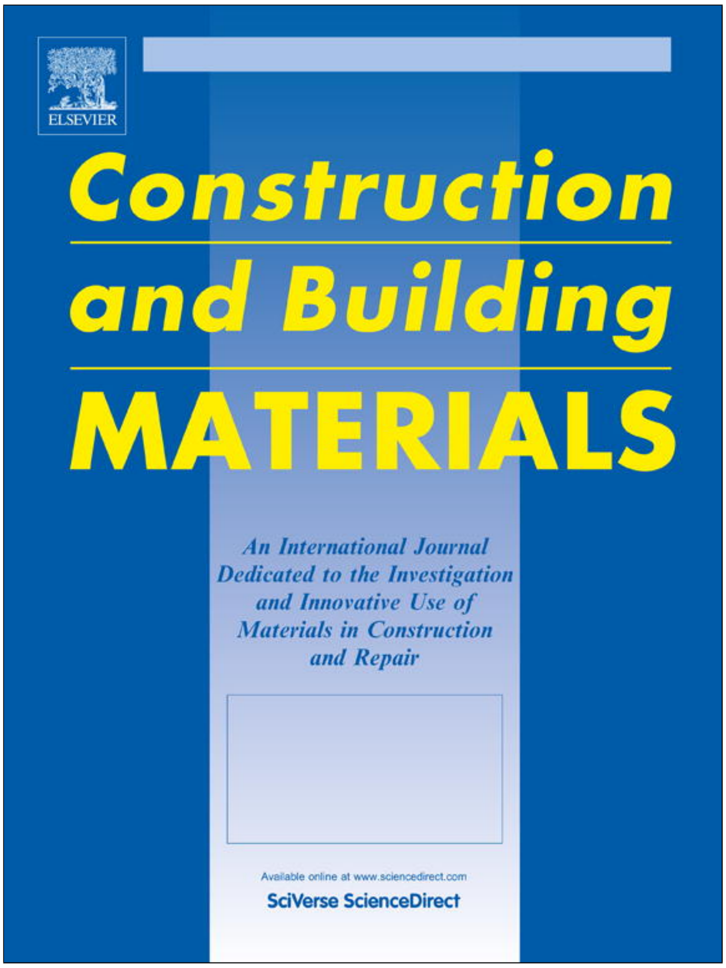

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Experimental analysis of timber–concrete composite beam strengthened with carbon fibres ☆

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HIGHLIGHTS

- ▶ We experimentally tested elements composed of a concrete plate connected to a timber beam.
- ▶ The beam is strengthened at the bottom of the tensile side with a CFRP strip.
- ▶ The measured results are compared with numerical results.
- ▶ Findings are important for the renovation principles of the old residential houses with timber floors.

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ABSTRACT

Reconstruction of old residential buildings is most often accompanied with a tendency to preserve the original features of the building, which leads to subsequent preservation of the floor elements. The ageing of the material and the consequent lower stiffness and load bearing capacity of the floor element along with the occasional change in the purpose of the facility and the consequent increase in the load require structural reinforcement of the floor elements. A possible solution lies in replacing the classic timber planks with a concrete slab and in the corresponding strengthening of the timber beam in the tensile zone. Therefore, the paper presents an experimental study performed on composite beam test samples composed of a concrete plate connected to a timber beam with dowel-type fasteners. Since the tension failure of the timber beam is the decisive resisting criterion, the beam is strengthened at the bottom of the tensile side with a carbon fibre-reinforced polymer strip. The measured results are compared with numerical results obtained by an already developed analytical model [1] based on Mohler's simplified formulation and on European standards for timber, concrete and steel structures. Since the experimental results demonstrate a relatively low deviation and additionally a good agreement with the numerical ones, the usage of the presented analytical model can be recommended to predict proper stiffness and load bearing capacity for these types of composite floor structures.

1. Introduction

The present times, characterised by specific circumstances in the sphere of climate change, witness an intensive focus of the sciences of civil engineering and architecture on searching for ecological solutions and construction methods that would allow for higher energy efficiency and, consequently, for reduced environmental burdening. Due to the fact that buildings represent one of the largest energy consumers and greenhouse gas emitters, energy saving strategies related to buildings, such as the use of eco-friendly building materials, reduction of energy demand for heating, cooling and lighting, reduction of green house gas (GHG) emissions, are strongly recommended.

Being a natural raw material timber helps the environment by absorbing and storing CO_2 while it grows and therefore plays an important role in reduction of the CO_2 emissions [2]. On the other hand, brick and concrete industries are responsible for about 10% of the global CO_2 emissions. Moreover, in spite of the smaller wall thickness timber buildings have better thermal properties than those built by using conventional brick or concrete construction methods. In comparison with other types of buildings, the energy-efficient properties of timber-frame buildings are excellent not only because well insulated buildings use less energy for heating, which is environmentally friendly, but also due to a comfortable indoor climate of timber-frame houses [3]. Considering the growing importance of energy-efficient building methods, timber construction will play an increasingly important role in the future.

☆ In memory of Mr. Matjaž Tajnik, victim of a tragic death on 17 May 2011, who carried out most of the presented experimental work.

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The use of timber in construction is gaining ever more support, especially in regions with vast forest resources since it reduces the energy demand for transport if the building material is available from the local area, [2,4]. Respecting all given facts, timber as a material for load bearing construction represents a future challenge for residential and public buildings.

Energy-efficient construction has become an unavoidable fact. Unfortunately, we frequently tend to forget that new residential buildings, which we mostly lay focus on, represent only a minor part of the existing housing stock. More attention should be therefore drawn to the renovation of older buildings – the biggest air pollutants, whose energy related properties no longer satisfy the requirements of the existing legislation. It is thus most vital to consider not only energy-efficient new buildings but to attend to energy-related refurbishment of older residential buildings and houses as well. Such undertaking should imply the renovation of the building's envelope, which could be realized by adding thermal-insulation materials and replacing the doors and windows [5]. On the other hand, we ought to strive for the maximum preservation of the original features of the building and for the subsequent preservation of the floor elements.

Since the floor elements in old residential buildings usually consist of timber beams with planks simply connected to the beams, the renovation or reconstruction process usually brings about certain construction-related problems, such as a change in the purpose of the facility and the consequent increase in the load on the floor elements, a decrease in the strength due to the ageing of the material and the subsequent lower load bearing capacity of the floor element. The floor elements in such cases therefore have to be adequately reconstructed and strengthened, which is more specifically presented in two possible steps described in Section 2 of this paper including a summary of the similar previous studies. The most important facts about the structural behaviour of such reconstructed and strengthened composite floor elements are given in Section 3. Experimental analysis performed on test samples with different types of CFRP strengthening is presented in Section 4. The measured results are additionally supported and compared with the already presented numerical results in [1].

2. Reconstruction of old timber floors

The floors in old residential buildings usually consist of timber beams placed in the tensile zone and of timber planks simply connected to the timber beams. In order to increase the bending and shear resistance of the floor elements, two main successive steps of reconstruction are to be taken, as is schematically presented in Fig. 1:

- the timber planks are removed and replaced with the concrete slab,
- the timber beams are reinforced with carbon fibre-reinforced polymer (CFRP) strips glued to the bottom of the beam.

The first step of merely replacing the timber plank with the concrete slab (Fig. 1a) involves three possible failure mechanisms of such a composite floor element under the bending load: compressive failure of the concrete slab, shear failure of the fasteners in the concrete–timber connecting area and bending failure in the tensile area of the timber beam. In case the last failure mechanism has a decisive role, the floor element needs further reinforcement comprising only that of the timber beam, which is performed by gluing CFRP strips to the bottom-tensile side of the beam, as shown schematically in Fig. 1b. It is important to stress that the second step of strengthening is significant only in case when the tensile strength criterion in the timber beam is decisive for the failure and when there is no risk of other two possible types of failure.

We know that the chosen materials of reconstruction (concrete, CFRP strips and the adhesive) are less eco-friendly than timber. Especially, the exposure of the CFRP and the dust due to the ageing in the closed space may be seriously harmful to user of the building. Especially, the exposure of the CFRP and the dust due to the ageing in the closed space may be seriously harmful to user of the building. Of course, according to the given facts, the strengthened floor elements are less eco-friendly than the original elements with the timber planks, but with this subsequent preservation of the timber beams we still can keep the original features of the building. Additionally, the costs of employing CFRP are in comparison with other strengthening materials at the moment rather high. However, the main advantages of using CFRP strips in particular compared to other optional materials of strengthening in this case (for example steel plates) are their corrosion resistance, light weight and flexibility, which allow convenient and easy transport to the place of erection. Continuously decreasing prices of these materials make the new technology more economical and interesting. Therefore the use of CFRP for the repair and strengthening of timber elements opens new perspectives for timber structures design.

2.1. Reconstruction with a concrete slab

Using the existing timber floor we can develop an efficient composite system made of timber members in the tensile zone, a concrete layer in the compression zone and a timber–concrete connection between them. The benefits of such reconstruction-strengthening procedures, sometimes applicable even to newly constructed buildings, are increased stiffness and load bearing resistance, better sound insulation and fire resistance as well as cost and environment related advantages gained when the existing supporting timber structure is used as framework.

The behaviour of the described floor construction depends on the connection between the timber beam and the concrete slab, i.e. on the fasteners used. Owing to the negative impact humidity has on the strength of timber, it is of outmost importance to install hydro-isolation in the flat area between the concrete slab and the timber beam in order to prevent humidity invasion from the first to the latter. Afterwards, the timber beam equipped with hydro-isolation undergoes the process of installation of the steel dowels which connect the concrete slab to the timber beam (Fig. 2a). Next, a minimum reinforced concrete slab is concreted onto the timber beams with the installed steel dowels in the following way: the timber beam with steel dowels is simply laid onto the concrete plate (Fig. 2b).

The experiments showed that the use of steel fibre reinforced concrete – SFRC displays better characteristics than the use of the classic concrete, regardless of the type of fasteners connecting the concrete slab and the timber beam. The experimental study involving the use of steel fibre reinforced concrete by Holschemacher et al. [6] demonstrated a 27% higher final load bearing capacity of the fasteners and a few times higher ductility, in comparison with the classically reinforced concrete. Practical examples of this kind of reconstruction procedure performed on the existing timber floor can be found in many European cities (e.g. Leipzig, Tübingen, etc.), as presented by Holschemacher et al. [6], Kenel [7] and Schanzlin [8].

Since the time-dependent effects have a big influence on rearrangement of stresses in a section composed of materials with different creep behaviour, as it is in our case, a special attention should also be dedicated to this problem. The problem of time-dependent effects presented in the connection between the concrete plate and the timber beam are numerically discussed in the study Tajnik et al. [1], where it is approximated with the effective slip modulus $K_{ser,fin}$ to take into account a timber final creep

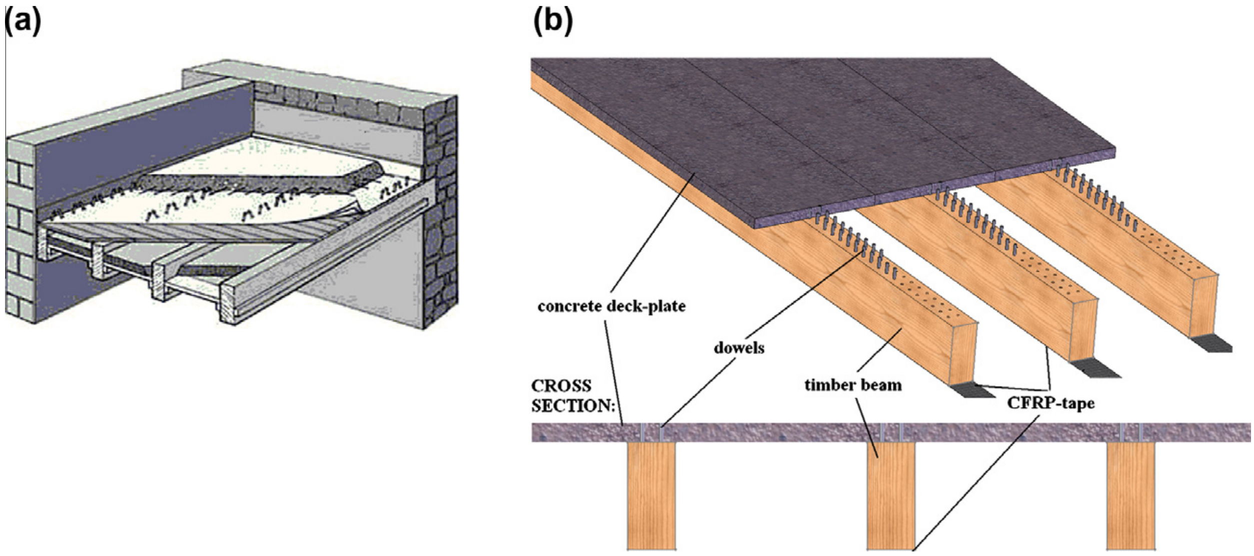

Fig. 1. Types of reconstruction of old timber floors with a concrete slab (a) and additionally with CFRP strips (b).

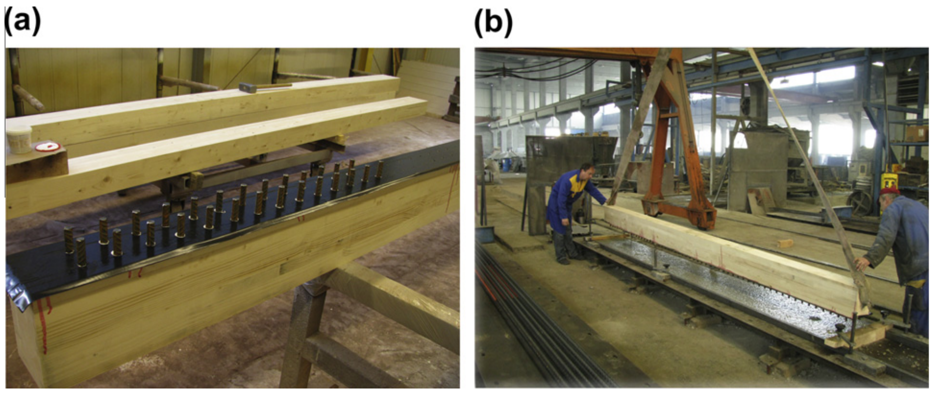

Fig. 2. (a) Installation of the steel dowels onto the timber beam. (b) Laying of the timber beam onto the concrete slab.

coefficient, a factor for a quasi permanent combination and a concrete creep coefficient. It is important that even in ultimate limit state calculations creep should not be neglected as confirmed by the research of Kavaliauskas et al. [9], Takač et al. [10] and Bob and Bob [11]. Ceccotti et al. [12] also recommends push-out tests to obtain realistic values of the connection stiffness and emphasises the significance of time dependent effects in an analysis of such structures.

2.2. Additional strengthening with CFRP strips

In many cases the above described strengthening concept is not sufficient. It is therefore recommendable to use additional carbon fibre-reinforced polymer (CFRP) strip at the bottom-tension side of the timber beam to gain higher bending resistance and stiffness of the floor in buildings or decks on bridges, as shown schematically in Figs. 1b and 3. Additional strengthening of the tensile zone with CFRP strips can increase the bearing load capacity in bending of the beam by approximately 15% [13], or even more – depending on the type of CFRP strip, which consequently leads to an even better use of each component of the composite section.

It is rather important, from the purely technological point of view, to glue the CFRP strips onto a clean, previously prepared surface at the bottom of the timber beams and thus ensure a completely stiff connection (Fig. 4).

Since the tensile strength of timber is similar to its compressive strength, strengthening applications of fibre reinforced polymers (FRPs) in timber structures have not been so frequently used as in concrete or masonry structures. The main advantages of FRP in comparison to other materials, for example steel plates, are their corrosion resistance, light weight and flexibility allowing convenient and easy transport to the place of erection. The use of (HSF) high strength fibre and CFRP for the reconstruction and strengthening of timber elements opens new perspectives for a timber structure design, particularly as it does not affect the appearance of the timber. Dagher and Breton [14] reinforced laminated timber beams in the tensile area using FRP lamellas. The test results showed an essential increase in bending resistance. The test results using carbon fibres in laminated beams are presented also by Bergmeister and Luggin [15]. Stevens and Criner [16] conducted an economic analysis of fibre-reinforced polymer (FRP) glulam beams. The results showed practical applicability of FRP reinforced elements, especially for bridges of greater spans, where beam dimensions can be substantially reduced using the FRP solution presented.

Composite reinforcement on sawn timber elements is less common in literature although many applications exist, especially for bending reinforcement. Timber beams reinforced with a layer of high-modulus composite material may be analysed using a transformed section of equivalent wood ([17]). Johns and Racin [18]

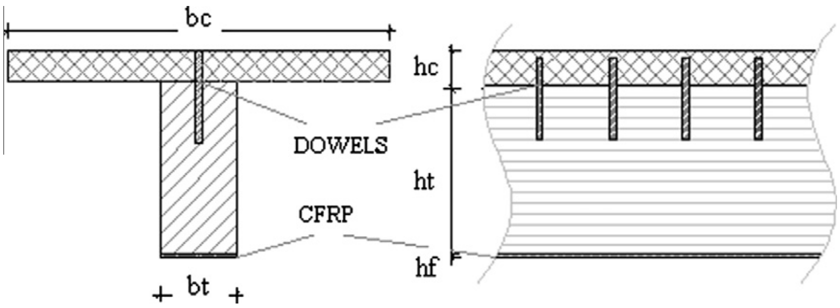

Fig. 3. Reinforcement by gluing CFRP strips on the tensile edge of the beam.

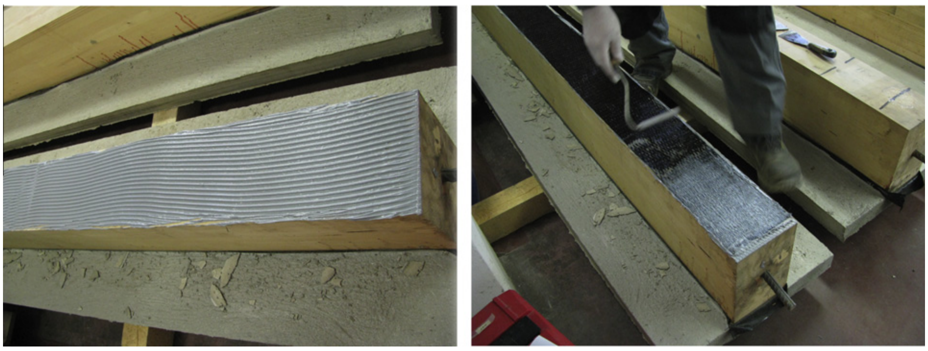

Fig. 4. Gluing of CFRP strips to the timber frame.

demonstrated their experimental studies using glass fibres to reinforce sawn timber sections. In the study of Dourado et al. [19] the influence of patch thickness and adhesive filleting was analysed experimentally and numerically. The main conclusion was that repairing leads to a remarkable gain in the bearing capacity of the damaged beam – a simple bonding repair with a patch of 2.0 mm thickness revealed a considerable increase of 62% in the bearing load. However, adhesive filleting did not induce a clear increase in the load bearing capacity. Investigation results for fibre reinforced hollow wood beams are presented by Kent and Tingley [20]. They show that a glass-aramid reinforced plastic (GARP) reinforcement increased the average strength and stiffness of the beams, compared to the non-reinforced control samples, by 22% and 5%, respectively.

3. Basic design model

Structural behaviour of timber–concrete composite members, governed by the shear connection between timber and concrete, can be predicted by the elasto-plastic model presented by Frangi and Fontana [21] or as a simplified elastic model appropriate for everyday engineering practice. For design and parametric study purposes, a simplified design method for mechanically jointed elements (Moehler's formulation) according to Annex B of EN 1995-1-1 [22] has been implemented. The expression of the so-called "γ-method" is used in equations with the following fundamental assumptions:

- bernoulli's hypothesis is valid for each sub-component (Fig. 5),
- material behaviour of all sub-components is linear elastic (Fig. 5),
- the distances between the dowels are constant along the beam (Fig. 3).

Additionally, two important further assumptions are considered in the proposed model:

- contribution of a potential tension area in the concrete slab is neglected in the load bearing capacity of the beam cross-section,
- slip modulus is taken in the elastic region for the serviceability limit state (K_{ser}) and in the plastic region for the ultimate limit state (K_u). The secant slip modulus can be determined in practice from push out tests according to Werner [23] and EN 26891 [24] which determine K_{ser} at $0.4F_{ultimate}$ and K_u at $0.6F_{ultimate}$. Ceccotti et al. [12] additionally recommend the use of K_{colaps} at $0.8F_{ultimate}$ for experimental collapse loads.

The effective bending stiffness $(EI_y)_{eff}$ in accordance with EN 1995-1-1 [22] can be written in the form:

$$(EI_y)_{eff} = E_c \cdot \left(\frac{h_c^3 \cdot b_c}{12} + \gamma_{ct} \cdot A_c \cdot z_c^2\right) + E_t \cdot \left(\frac{h_t^3 \cdot b_t}{12} + A_t \cdot z_t^2\right) + E_f \cdot (A_f \cdot z_f^2) \tag{1}$$

where E_c is the mean modulus of elasticity of concrete, E_t the mean modulus of elasticity of timber and E_f is the mean modulus of elasticity of the carbon strip. As shown in Fig. 5 z_i is the distance from overall neutral axis to the centre of gravity of each sub-component, A_c, A_f, A_t are cross section areas of each sub-component and h_c, h_f, h_t are thicknesses of each sub-component. The stiffness coefficient (γ_{ct}) in the connecting area between concrete and timber is calculated according to [22] in the form of:

$$\gamma_{ct} = \frac{1}{1 + \frac{\pi^2 \cdot E_c \cdot A_c \cdot S_i}{K \cdot l_{eff}^2}} \tag{2}$$

where l_{eff} is the effective length of the composite beam and s_i is the effective spacing between the fasteners. The stiffness coefficient in the plane between the carbon strip and the timber web, which are glued together, is considered as fully connected, therefore $\gamma_f = 1.0$. A normal stress in the composite section for each sub-component can be obtained according to the given assumptions in the form of:

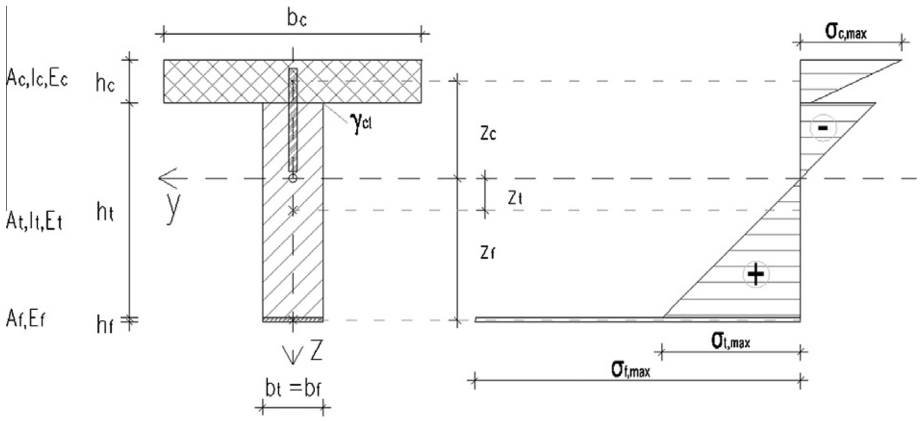

Fig. 5. Composite cross-section with a diagram of normal stress distribution.

$$\sigma_d = \frac{M_y \cdot E_i}{(EI_y)_{eff}} \cdot (\gamma_i \cdot z_i \pm \Delta z) \tag{3}$$

where M_y is the bending moment acting on the composite section, Δz the distance from the centre of gravity of each sub-component to the edge (or any fibre) of each sub-component and E_i is the modulus of elasticity of each sub-component. The considered linear stress distribution is schematically presented in Fig. 5.

4. Experimental analysis

A detailed numerical analysis based on the mathematical model in Section 3 is presented in Tajnik et al. [1]. A selected set of results will serve the purpose of comparison with the measured values obtained through the experimental analysis, which will be the only relevant factor indicating the suitability of choice of the presented mathematical model. The decision to seek experimental confirmation of the model additionally rests on the fact that the sphere of science sees the appearance of numerous articles denying the suitability of using the "γ-method" which is based on Eurocode 5 [22]. Moreover, studies like de Goes and Junior [25] tend to appear and confirm this method based on experimental measurements and finite element models (FEMs) calculations on road bridges.

4.1. Test configuration

The longitudinal section and the cross section of the analysed four test specimens P1, P2, P3 and P4 are presented in Fig. 6. The dimensions were determined according to the simplification for production, suitability for transportation and practical importance of information which the measured values can provide. The test specimen dimensions and the fasteners arrangement density underwent additional careful selection based on the numerical results of the previously described mathematical model. The aim was to ensure previously mentioned failure of the timber beam and after that the failure of the CFRP strips, which was the only sensible step to take in order to test the strengthened CFRP specimen and especially the influence of the CFRP reinforcement in order to recognize practical advantages of using such kind of reinforcing.

The aim was to ensure previously mentioned failure of the timber beam and the CFRP strips, which was the only sensible step to take in order to test the strengthened CFRP specimen.

The production of the tested beams was performed in three successive steps: (a) production of the glulam beam of the quality GL34h with the constant height of h_t = 250 mm and width of b_t = 160 mm, drilling of holes for the dowels, gluing of the hydroinsulation and insertion of the steel dowels of diameter d = 16 mm into the timber beam (Fig. 2a). The dowels were placed in two parallel rows at the distance of 50 mm between the rows (Fig. 6); (b) on the timber beam with already inserted dowels the concrete slab of a constant thickness of h_c = 60 mm and compressive strength of 30 MPa with a reinforcement S500 placed in the centre of the slab and with a minimum cross-section of As = 1.39 cm^2/m (Fig. 6) was produced. The timber beam with steel dowels is simply laid onto the concrete slab (Fig. 2b); (c) finally, the CFRP strips of the thickness of 1.2 mm and with the width of 150 mm which was constant in the whole length of the tested beam were glued to the bottom side of the timber beam (Fig. 4).

A risk of shear failure in the selected specimen does not exist in practice. Prediction resting on numerical results gained through the analytical calculation model estimates that shear failure occurs only at 220% overload above bending failure in the tensile area of the timber beam. To suit the failure a minimum possible

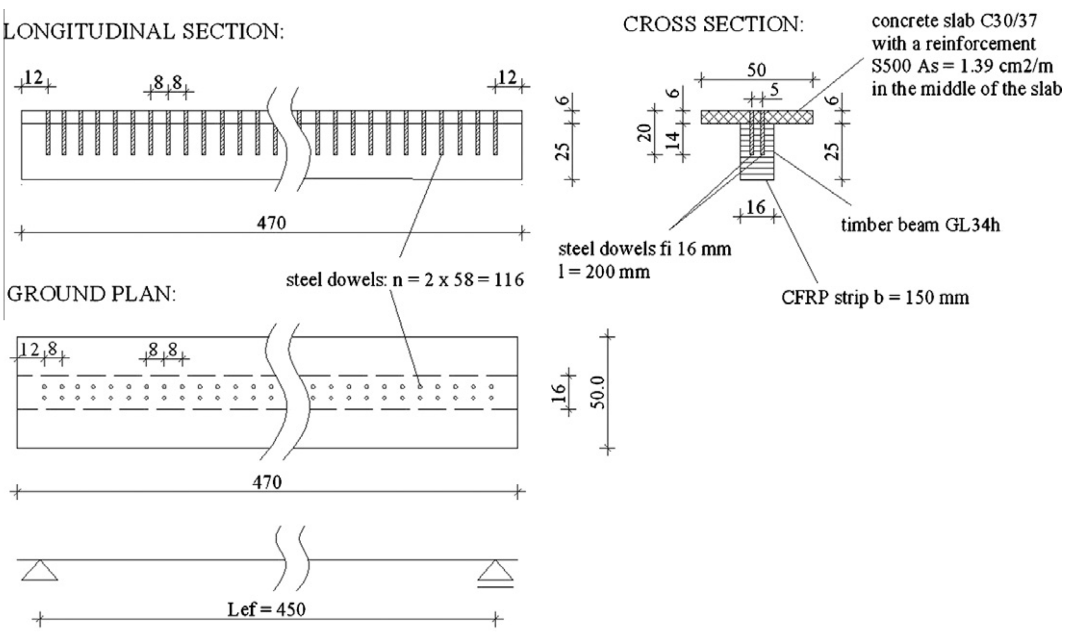

Fig. 6. Geometry of the test specimens.

Table 1
Properties of the materials used.

	Concrete C30/37	Timber GL34h	SikaCarboDur-H514
E_m (MPa)	31,939.00	11,600.00	300,000.00
$f_{m,k}$ (MPa)	–	34.00	–
$f_{t,0,k}$ (MPa)	–	23.50	1300.00
$f_{c,0,k}$ (MPa)	30.00	24.00	–
ρ_k (kg/m^3)	–	430.00	–
ρ_m (kg/m^3)	2400.00	516.00	1600.00

E_m mean value of modulus of elasticity.
$f_{m,k}$ characteristic bending strength.
$f_{t,0,k}$ characteristic tensile strength (for timber parallel to grain).
$f_{c,0,k}$ characteristic compressive strength (for timber parallel to grain).
ρ_k characteristic density.
ρ_m mean density.

regulations-abiding installation of the fasteners in the longitudinal range of 80 mm was selected which subsequently ensures high stiffness and load bearing capacity of the connection.

For the purpose of the testing we deliberately decided for a slightly atypical three-point bending test (although it is common practice to conduct a four-point bending test), with the load being concentrated in the middle of the 4.5 m span. Such load distribution does in fact bring a risk of partial bending-shear failure but it simultaneously offers far more reliable results about the impact the stiffness of the fasteners has due to constant distribution of the shear flux across the entire shear plane between the concrete slab and the timber beam. For the same reason we opted for constant distribution of the dowels across the entire shear plane along the timber beam. An additional reason was to avoid potentially unreal results in case of using the equation for effective distance between the fasteners (s_{ef}) following Eurocode 5 [22]. We tested four identical specimens.

Material properties for the timber of quality GL24h are taken from EN1194 [26] and for the concrete from Eurocode 2 [27]. The values for the CFRP strips of SikaCarboDur-H514 are taken from Sika [28]. All material properties are listed in Table 1.

Fig. 7. Installation of the measuring instruments.

4.2. Test results

The ultimate failure force (F_u), the maximal cantilever bending deflection (w) under the acting force (F) and the slip (Δ) between the concrete slab and the timber beam were all measured (Fig. 7). For the point of view of statistics, the following finding is of significant importance: similar load bearing and stiffness characteristics were demonstrated by all specimens with the exception of one which showed a certain deviation in the load bearing capacity due to the expected higher deviation of the average timber strength. The specimen in question remained part of the analysis since it displayed a suitable stiffness level.

Fig. 8 shows steps of one of the specimen's failure where the first evident failure was that of the timber beam (Fig. 8a), followed by the failure of the CFRP reinforcement (Fig. 8b) and finally by the total failure of the specimen – that of the concrete

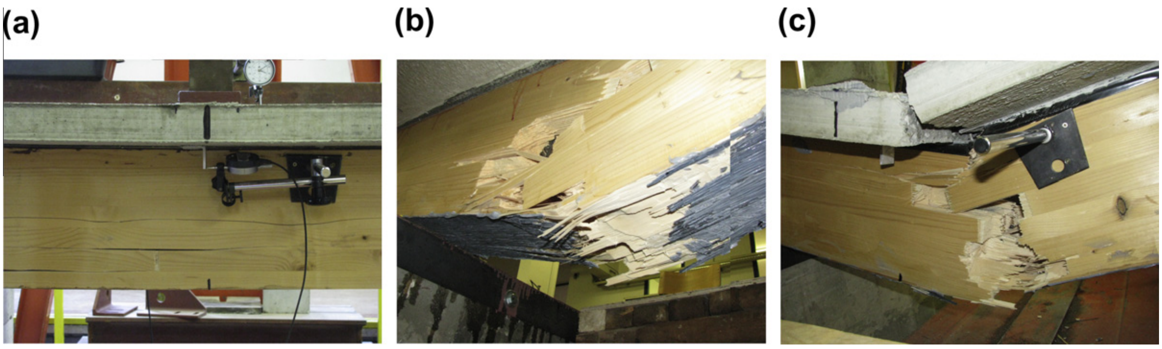

Fig. 8. Tensile failure of the timber beam (a), failure of the CFRP reinforcement (b) and total failure of the specimen (c).

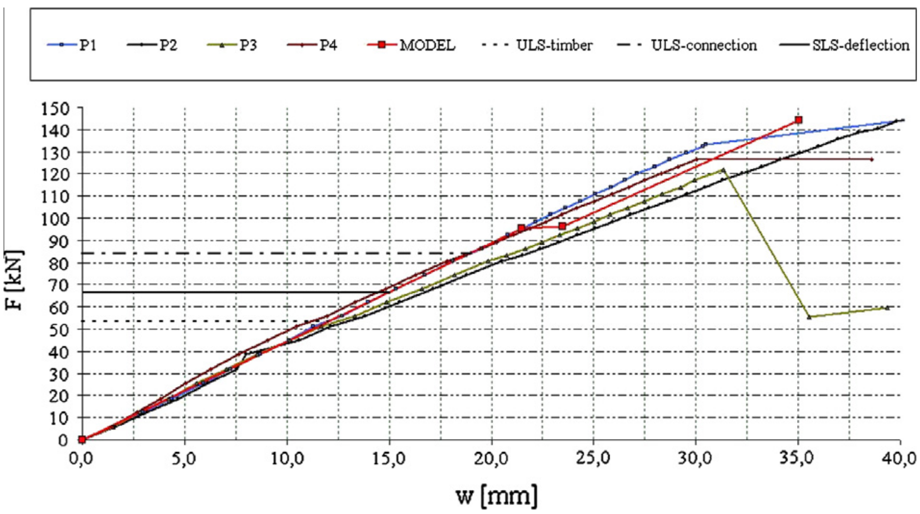

Fig. 9. *F*–*w* diagram of the test samples and the calculated numerical results.

Table 2
Measured test results and the calculated numerical results.

Test specimen	Failure mode: timber in tension $F_{u,timber}$ (kN)	Deflection by force $F_{u,timber}w$ (mm)	Failure mode: concrete in compression $F_{u,concrete}$ (kN)	Force by $w = 15$ mm (kN)
P1	133.10	30.47	144.11	66.79
P2	140.48	38.90	152.70	60.16
P3	122.01	31.34	143.54	62.47
P4	126.32	30.05	148.23	68.97
Numerical model [1]	125.20	30.79	147.67	67.20

slab, which classifies the failure as brittle, i.e. non-ductile failure (Fig. 8c). The fasteners, i.e. the connection between the concrete slab and the timber beam, were not at risk, according to expectations.

The measured results for the failure force in tensile area of the timber beam ($F_{u,timber}$), the force of the total failure of the specimen – that of the concrete slab ($F_{u,concrete}$) and the measured deflection by the force $F_{u,timber}$ are presented in Table 2. Additionally, the measured value of the force by a known deflection of $w = 15$ mm is presented to determine the bending stiffness of the test sample. The calculated results using the proposed design model from Section 3 are given to compare the measured and the numerical values.

For information only, the calculated load-bearing capacity of the un-strengthened element with the concrete slab can be calculated from [1] and it is $F_{u,timber,uns.} = 99.36$ kN. The load-bearing capacity of the original structure composed of the timber beam and timber planks placed perpendicular to the grains of the beam can be calculated considering only the contribution of the timber beam and it is $F_{u,timber,orig.} = 50.37$ kN. These results for the increasing in the bending resistance are close to the gains in a load-bearing capacity obtained by the experiments with similar strengthening concepts in Ajdukiewicz and Brol [29].

Fig. 9 shows F–w diagrams of all four specimens up to the point of failure. For the purpose of comparison the diagrams contain additionally drawn calculation stiffness which was calculated according to the mathematical model in Section 3. Further items added to the scheme of the diagrams predicted upon the model are (horizontal lines): the design value of failure in the tensile area of the timber beam (ULS-timber), the provisional design load bearing capacity of the connection, i.e. of the fasteners (ULS-connection) and the provisional load bearing capacity according to the limit value for instantaneous deflection $L/300 = 15$ mm (SLS-deflection). The interrupted red line represents a clearly visible leap appearing in the mathematical model, at the crossing from the fasteners shift module in the range of serviceability limit state – SLS (K_{ser}) into the fasteners shift module in the range of ultimate limit state – ULS (K_u) at the approximate force of 96 kN. It needs to be stressed that the fasteners' stiffness and that of the timber beam should be taken into separate consideration despite their connection through Eqs. (1) and (2), since the first necessary step is to define a borderline between serviceability limit state (SLS) and ultimate limit state (ULS) for the fasteners with respect to the failure load bearing capacity by Johansen's terms. It means defining the limit value up to which the connecting area's behaviour has linearly-elastic features, by the shift module for SLS (K_{ser}), and the limit value at which the connecting area's behaviour is non-linear, by the shift module for ULS (K_u). Attention should also be paid to the fact that the above limits do not necessarily coincide with those of the composite timber beam as whole, unless the decisive role in the timber beam failure is seen in the connection itself or in the fasteners between the slab and the timber beam. This finding is highly relevant to the designer's decision and his preference for non-ductile bending failure in the tensile area of the timber beam or the ductile – yielding failure of the fasteners. The failure of the timber beam in tension was practically decisive, there was (especially by test samples P3 and P4) practically no post-breaking stability of the element after that, as it can be seen from Fig. 9. The deflections rapidly increased and they were not under control anymore.

As seen in the diagrams of the test samples in Fig. 9, we decided for the non-ductile failure mechanism in the bottom tensile area of the timber beam. The presented measured results show a relatively small discrepancy in the four samples, which can justify a relatively small number of the chosen identical samples. With regard to defining the ultimate failure load bearing capacity, the diagram furthermore shows a good agreement between the results gained through the calculation analytical model [1] and the results of the experimental specimens P1–P4, with the maximum deviation between sample P1 and the model being 8.2%.

In addition, our findings reveal good agreement in predicting deformation or the stiffness of such constructions since the difference between the calculation-predicted value and the average measured value is as little as 30.36 mm/30.00 mm = 1.012% or 1.2%, where shifts located at areas of the predicted failure were observed. The serviceability limit state range ($L/300 = 15$ mm), where predicting deformation becomes even more important from the designer's perspective, also shows the average deviation of 15.75 mm/15.00 mm = 1.05% or 5.0%.

5. Conclusion

The use of high strength fibres (HSFs) and carbon-fibre reinforced polymers (CFRPs) for the reconstruction and strengthening of timber elements opens new perspectives for the timber structures design. Continuing reduction in the price of these materials make the new technology more economical and interesting. On the other hand, in comparison with traditional reinforcement the use of fibre composites in timber buildings calls for experience and higher quality of construction works. This leads to a conclusion that the presented composite floor system, consisting of the concrete slab and the timber beam reinforced with CFRP, means a highly interesting potential way of increasing the load bearing capacity of old timber floors in the future. At the same time this method preserves the original features of the existing buildings.

The presented experimental study certainly reached its purpose since it leads to a conclusion – based on the analysed results and their relatively low deviation – that it largely confirms, in spite of the limited number of samples, the adopted analytical calculation model. We can thus claim that the analytical calculation model, meticulously presented and analysed in [1], proves to be most suitable for the calculations of the load bearing capacity and the calculation or prediction of deformation of the floor constructions in question. Extensive numerical parametric analyses, carried out in [1] with variations including timber quality, the height of the timber beam and the installation of the fasteners in the connection showed that the contribution of the CFRP strips in the form of additional strengthening can improve the bending resistance up to 26% and in the bending stiffness up to 18%.

We would like to emphasise that the presented concept of reinforcing is recommended only for strengthening old timber floor structures to assure a higher load-bearing capacity. In new timber floor elements a higher load-bearing capacity can be easily achieved by using timber beams of higher dimensions or by using a timber of a higher quality.

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