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Structural Behavior of Hybrid Timber-Composite Beams

by

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Abstract

The development of timber-concrete composite structures has shown that composite systems are a very efficient solution to increase the load-carrying capacity and overall performance of timber structures. The weakness of these systems is clearly marked by the stiffness of the bond line, the natural limitations of the timber and defects in wood.

An innovative solution of these problems has been found in a new-type composite beam for structural rehabilitation and upgrading, combining polymer concrete in the compression zone, fiber-reinforced plastics in the tension zone and timber in between. All composite partners have been revised according the material formulation, structural performance of the composites, fracture and delamination behavior. Appropriate mechanical models for the numerical simulation based on a finite element approach have been developed. The results of the experiments have highlighted the limitations of the composite structure as well as the advantages of the various combinations. The test results show an increase of the load-carrying capacity up to 185% for short-term loading,
depending of the ratio between polymer concrete and timber. For long-term loading, the creep influence has been calculated with a rheological model to 1.8-2.5-times of the deflections compared to the unreinforced beams, depending also on the configuration.

Regarding economical and practical aspects, the presented composite system describe a good alternative to conventional solutions for structural upgrading in reconstruction.

Introduction

Economic design of large span and / or heavy loaded structures demands the use of high-performance materials, which means not only high absolute strength and stiffness but also a high ratio of strength and stiffness to self-weight. Particular advantages associated with fiber-reinforced plastics (FRP) over traditional reinforcing materials such as steel include, besides the low weight, the ease of handling and the corrosion resistance. This enhancement is very noticeable in flexural bending behavior, with FRP working in tension and timber in compression. This type of association brings good advantages when used in rehabilitation works and reinforcement of solid timber floors for new construction. It is a solution that does not overload the existing structure with additional weight, while significantly increasing its strength and stiffness and enhancing the serviceability by controlling the natural vibration mode, which is important in wooden structures.

It is a well-known feature of timber beams that they usually fail suddenly due to the breaking of fibers on the tensile face. This behavior may be changed by an adequate strengthening of the tensile face, which may lead to a more ductile failure induced by the gentle local buckling of the fibers in the compressive face. Such strengthening may be achieved by bonding a passive (slack) or active (pre-stressed) reinforcement of FRP with adhesives to the structural members. The development of a high quality in-service bond between the wood and the FRP laminate is essential to the composite interaction between the two materials. Adhesive bonding is a possible joining technology and it has the advantage of avoiding the stress concentrations induced by mechanical fasteners. Ideally, a rigid connection will lead to the most effective composite behavior, with the stress and strain distribution governed by the classic theory of strength of materials. This type of behavior needs a full connection that does not allow any slippage of the interface and will be obtained by a thin film of resin.

Apart from this, each piece of wood differs in the amount of stiffness-reducing defects such as knots, splits, and checks and therefore, it is hard to say at what stress level the reinforced beams would have failed if they had not been reinforced. The presence of FRP reinforcement arrest the crack opening, confines local rupture and bridges local defects in the timber. Therefore, the structure can support higher loads before failure. Moreover, as can be observed from experimental studies done, there is an increase of the load-carrying capacity because of the quoted crack opening arrest, but the limits are distinguishable in timber crushing under bending loads and the quality of the bond line itself.

A high strength material can be the optimal add-on to restore the load-bearing capacity in the compression zone. This add-on was found in an epoxy resin-bounded polymer concrete (PC). The idea of this application was focused on combining the favorable characteristics of the polymer concrete layer, the existing timber beam and FRP reinforcement in a composite structure. The disadvantages of the single components are compensated and the tension-stressed timber and FRP and the compression-stressed PC layer offering a much better load-bearing behavior.

A study of these two reinforcement techniques of timber floors under bending loads has been carried out at the Bauhaus-University of Weimar. The experimental and numerical study consists of two parts: First, the use of epoxy resin-based polymer concrete as strengthening material, whereby the removal of the suspended ceiling on timber floors is not necessary, and second, the use of structural adhesives on the building site, whereby the removal of the overhanging part of the structure as well as the inserted ceiling is not necessary. This reinforcing technique can be used for structural upgrading and repair of existing structures or for new constructions where the height of the beams is limited.

Strengthening with polymer concrete

Polymer concrete is a composite material formed by combining mineral aggregates such as sand or gravel with a monomer resin. Rapid-setting organic polymers are used in PC as binders. Studies on epoxy polymers have shown that curing method, temperature and strain rate influences the strength and stress-strain relationships. PC is increasingly used as an alternative to cement concrete in many applications. Today, PC is used for finishing work in cast-in-place applications, precast products, highway pavements, bridge decks and waste water pipes. PC exhibits a brittle failure. Improving its post-peak stress-strain behavior is important. The Development of better PC systems and characterizing the compressive strength in terms of constituents are essential for the efficient utilization of PC. However, the data on epoxy PC are rather limited, and there is an increasing interest in the deformation characteristics under working conditions in combination with other materials such as wood for composite structures.

The used polymer concrete for this application consists of the epoxy resin, an accelerator and special gravel with a grading of 0-4 mm (0.16”). The components were mixed together in a ratio of 14:2.67:83.33 by weight. Comparing the used PC with high-strength concrete C100/115 after curing for seven days on a temperature of 20°C, the mean compressive strength is about the same, whereas epoxy PC has a triple value of the bending strength of C100/115. A comparison with the usually on building site used concrete C25/30 is shown in Table 1.

The mechanical properties of the PC have been investigated for dependence on the ambient air temperature when cast on site and the curing time [1]. In the laboratory tests, only a small increase of the compressive strength with cumulative curing time and storage temperature could be observed. Low temperatures result in an interruption of the chemical reaction in cement bounded concrete. In epoxy resin bounded PC, the chemical reaction continues when the ambient air temperature arises. Higher temperatures reflect a higher early compressive strength sooner than by standard climate conditions.

For characterization of the structural performance of the composite structure, several lab tests have been carried out for different specimen configuration [2, 3]. Since these types of building systems have been realized as timber-concrete composite beams mainly used for bridges or revitalization of timber floors, steel dowel type connectors were normally used to transfer the shear forces. These steel connectors realize the transmission of shear stress in the contact area of timber and surface layer in timber-concrete composite structures. In the building system described here, only the natural adhesive bond between timber and epoxy PC should be sufficient to transfer the shear- and tension forces in the compound joint. The main practical advantages for the chosen system are:

- The section design can be done easily by a timber formwork on the level of the necessary construction height.
- All work can be done from top side; the suspended ceiling will remain unaffected.
- The floor below the reconstruction work can be used without any restrictions. The full load-carrying capacity is achieved after one day.

As result of the axial bonding and shear tests, the structural behavior of the compound between timber and epoxy PC can be assumed as rigid. This assumption could be confirmed by bending tests where failure occurred in the timber traction zone without complete plasticization of the compression region in all specimen and test series. The results of the bending test show an increase of the load carrying capacity of 150-200% for a layer thickness of 2.5 and 3.5 cm (1 and 1.4”).

Strengthening with fiber reinforced plastics

Over the past years, different investigations on FRP reinforcement, mostly carbon fiber FRP’s (CFRP), have been done to study the structural improvement when using this technique for timber structures [4, 5]. The solid timber beams were reinforced with a continuous carbon fiber lamella with intermediate modulus fibers within the clear span, otherwise over the full length embedded in the specimen at different section locations.

The CFRP layer was glued / embedded by means of a commercially available epoxy resin, consisting of Bisphenol-A-Epichlorhyd, Bisphenol F and Epichlorhydrin. The mean mechanical properties are shown in Table 2. The wood beams reinforced with CFRP strips revealed a behavior that is more ductile compared to unreinforced beams. The strength increase of the reinforced specimen was defined as the bending stress at the deflection in linear range before failure, divided by the bending stress of the un-reinforced specimen at the same deflection value. It has been calculated for all test series between 6% and 12%. The presence of CFRP reinforcement arrested crack opening, confined local rupture and bridged local defects in the timber, therefore this technique is very promising for structural enhancement in tension stressed areas.

Numerical simulation of the bond line behavior

The bond line behavior between timber and PC has been confirmed as rigid, so no further action in the numerical model is necessary. Between timber and FRP, debonding and delamination effects have been observed and should be included in an appropriate model. Decohesion along interfaces plays an important role in a wide variety of failure processes in structures when using chemical bonding as the optimal form of combining two surfaces with each other. The various theories of bonding developed over the past years can only limited explain the observed effects from laboratory tests and real structure monitoring. The development of finite element techniques, focusing on crack propagation and interlaminar damage, provides new tools to predict the structural performance enhancement and the serviceability of fiber-reinforced structures.

The analysis of delamination is commonly divided into the study of crack initiation and crack propagation. Delamination initiation analysis is usually based on stresses and interaction criteria of the interlaminar stresses in conjunction with a characteristic distance as function of geometry and material properties. Crack propagation is usually predicted using the Fracture Mechanics (FM) approach. This approach avoids the difficulties associated with a stress singularity at the crack front but requires the presence of a pre-existing delamination. When used in isolation, neither the strength-based approach nor the FM approach is adequate for a progressive delamination failure analysis. Both approaches have to be considered together using special interfacial decohesion elements, placed between composites material layers or in the structure where cracking can occur (Figure 1). These elements are surface-like and are compatible with general bulk finite element discretizations of the solid model. Cohesive elements, or decohesion elements, bridge nascent surfaces and govern their separation in accordance with the cohesive material law.

Different types of interface damage laws have been developed so far for different structural and thermal problem formulations using linear, bilinear, polynomial or exponential functions. For the combination of the anisotropic properties of wood with FRP reinforcement in an interface damage law, the interaction of shear and normal stresses have to be considered. This problem can be solved by using a delamination analysis and an exponential interface damage law. The proposed exponential interface law consists of a continuously differentiable stress-crack opening-behavior, robustness regarding numerical problems and takes both, shear and tension stiffness, into account [6].

The cohesive zone model consists of a constitutive relation between the traction T acting on the interface and the corresponding interfacial separation δ (displacement jump across the interface) developed from the simple and convenient universal binding law furnished by SMITH and FERRANTE, where e = exp(1).

$$T = e\,\sigma_c\left(-\delta/\bar{\delta}\right)\exp\left(-\delta/\bar{\delta}\right) \qquad (1)$$

The definition of traction and separation depend on the finite element and the applied material model. It is based on the local corotational element coordinate system. Decohesion response was specified in terms of a surface potential $\Phi(\delta)$ relating the interface tractions and the relative tangential and normal displacements δ_n and δ_t across the interface [6], [7]. The resulting work of normal separation and tangential separation can be related to the critical values of the energy release rates. The surface potential is defined to

$$\begin{aligned}\Phi(\delta) &= e\,\sigma_c\,\bar{\delta}_n\left(1+\exp\left(-\Delta_n\right)\cdot\zeta\right)\\ \text{with}\quad \zeta &= \left(1-r+\Delta_n\right)\frac{1-q}{r-1}-\\ &\quad -\left(q+\Delta_n\frac{r-q}{r-1}\right)\exp\left(-\Delta_t^2\right)\end{aligned} \qquad (2)$$

σ_c maximum normal traction at the interface

$\bar{\delta}_n$ normal separation where the maximum normal traction is attained with $\delta_t = 0$

$\bar{\delta}_t$ shear separation where the maximum shear traction is attained at $\Delta_t = 1/2\sqrt{2}$

q ratio between normal separation work and shear separation work

δ_n^* value of δ_n after complete shear separation with $T_n = 0$

$\Delta_n = \delta_n/\bar{\delta}_n,\ \Delta_t = \delta_t/\bar{\delta}_t,\ q = \Phi_t/\Phi_n,\ r = \delta_n^*/\bar{\delta}_n$.

In most of the computations all cohesive surfaces are taken to have identical cohesive properties which simplifies the potential from eq. (2) with $q = 1$ and $r = 0$. The potential lead by derivation on the displacements to the stresses if $\delta < \delta_{max}$ where the traction components T are coupled to both normal and tangential crack opening displacements and leads to separation at $\Delta_n = 1$, so that $T_n \equiv 0$ for $\Delta_n \geq 1$. The traction components in normal and tangential direction are given in eq. 3 and 4.

$$T_n = e\,\sigma_c\,\Delta_n\,\exp\left(-\Delta_n - \Delta_t^2\right) \quad (3)$$

$$T_t = 2\,e\,\sigma_c\left(\bar{\delta}_n / \bar{\delta}_t\right)\Delta_t\left(1+\Delta_n\right)\exp\left(-\Delta_n - \Delta_t^2\right) \quad (4)$$

The main advantage of the cohesive zone modeling is that, when it is known where fracture may occur a priori, a cohesive zone may be placed anywhere along element interfaces in that areas, to take these effects into account. Furthermore, using decohesion elements, both onset and propagation of delamination can be simulated without previous knowledge of crack location and propagation direction and therefore suitable for structural design and evaluation of timber composite beams.

Combined FRP and PC reinforcement

With the obtained knowledge from the experimental tests and numerical investigations, it is now possible to describe and design any possible combination of the three materials as a composite structure. For the first estimation of the structural behavior of the composite structure, the tested specimen with PC reinforcement with a section of 12/14 cm (4.7/5.5”) and a span of 2.20 m (7.2 ft) have been evaluated. The thickness of the PC layer has been chosen to 2.5 and 3.5 cm (1 and 1.4”) placed on top with an additional CFRP reinforcement on bottom, cross section 50 x 1.4 mm (2 x 0.06”), material properties according to Table 2. The compound between the laminate and timber has been determined to be free from defects and delamination. Figure 2 shows the results of the structural analysis as strength increase by comparison of the bearable bending moments. The most economical configuration here is type (e) with only a thin PC layer on top. The increase of the ultimate bending moment m_u by 200% evidences the double load-carrying capacity of this composite system and optimal performance on the building site [8].

This approach has been repeated for a solid timber floor, cross section 22/26 cm (8.7/10”), span 7 m (23 ft) with PC reinforcement of 3 cm (1.2”) and high modulus CFRP reinforcement 100 x 1.4 mm (4 x 0.05”). The bond line has been described with the cohesive zone model and anisotropic parameters for the timber section. The results of the numerical simulation for a maximum allowable deflection of 3.5 cm (1.4”) were similar to the results shown in Figure 2. Again, the advantage of the additional reinforcement is significant regarding the structural performance and of the composite beam. Due to the easy to realize construction, this type of composite is very attractive when additional stiffness or less construction height is needed. Overall, this building system is time and labor-cost saving and suitable for difficult floor and workspace situations.

Example – Reconstruction of Mansfield Castle

Accompanying to the theoretical investigations, both systems were tested for new constructions and upgrading of existing structures independently and together under practice-related conditions on site. First practical insights have shown primarily doubts regarding cleanliness and feasibility could not confirmed and the work progress was done accurate and efficient after short briefing of the construction worker.

For the new occupancy and floor design of the Mansfield Castle, an increase of the existing dead loads and the life loads for the structural design of the waffle slab above the „Blue Hall” (Figure 3, 4) was required. The main girders got a deflection of 9 cm (3.5“) over the past years. After removing of the ceiling cover, large longitudinal and inclined cracks in the girders were visible. Due to high loading from the secondary ceiling girders and the specific construction of the waffle slab, the combination of described reinforcement systems were chosen for the main girder – upgrading of the tension zone with carbon fiber strips (Figure 5) and an additional PC layer on top (Figure 6).

The structural design was done using the described finite-element model with cohesive law for the bond line. The material properties for the numerical model are shown in Table 3. When comparing the stress distribution over the section height (Figure 7) small tension stresses in the polymer concrete layer are present in areas where the main girder section is cut off for the secondary girder joints. The magnitude of these stress peaks in the bond surface was calculated to 40% of the design value $f_{cm,d}$ and covered by additional four solid glass fiber rebar’s with a diameter of 15 mm (0.6”) in the PC layer.

Conclusions

A new-type composite beam for structural rehabilitation and upgrading, combining polymer concrete in the compression zone, fiber reinforced plastics in the tension zone and timber in the center has been investigated numerically and tested in practice. The used mechanical model show good agreement with recent test results and addresses structural nonlinearities and FRP debonding. With the proposed composite beam, the load-carrying capacity can be nearly doubled compared to solid timber structures, where the construction work remains significant low. Regarding economical and practical aspects, the presented composite system describe a good alternative to conventional solutions for structural upgrading in reconstruction. It is suitable for difficult floor and workspace situations.

Authors

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Figure 1: FRP debonding between FRP reinforcement strip and timber at the support

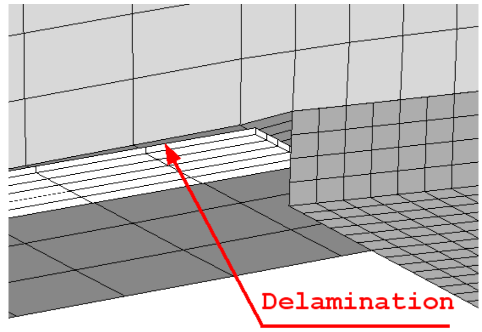

Figure 2: Load-bearing capacity increase compared to solid timber beams in %

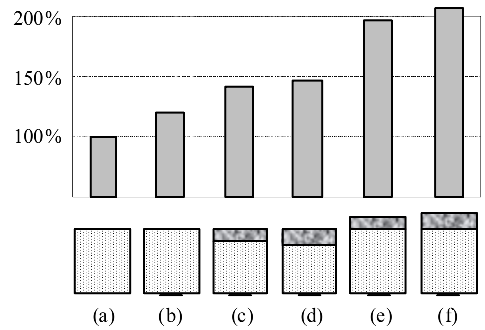

Figure 3: Waffle slab, bottom view

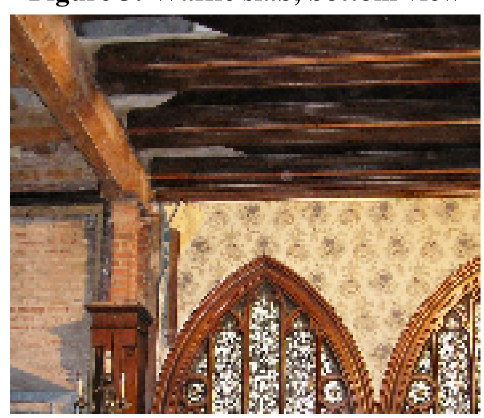

Figure 4: After reconstruction work

Figure 5: Gluing of the FRP strips

Figure 6: PC reinforcement from upper floor

Figure 7: Finite element mesh

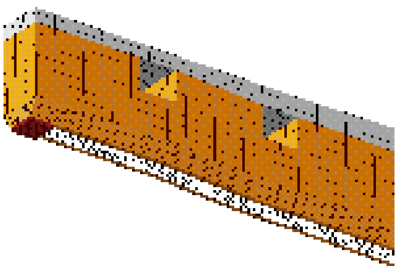

Figure 8: Longitudinal stresses

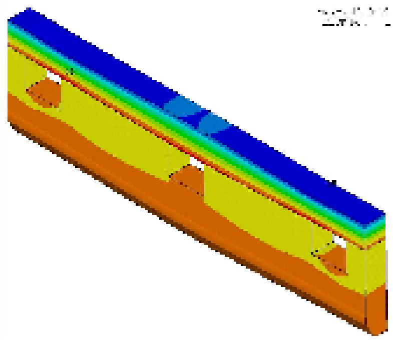

Figure 9: Shear stresses

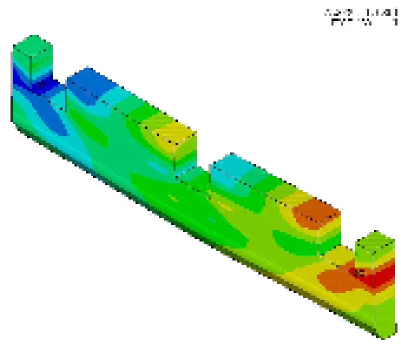

Table 1: Mean material properties of used polymer concrete

Material property	Epoxy PC				Concrete C25/30				Ratio
Density	2	g/cm³	125	lb/ft³	2.4	g/cm³	150	lb/ft³	0.83
Tensile MOE	19,600	MPa	2,843	ksi	30,000	MPa	4,351	ksi	0.64
Bending strength	30	MPa	4.35	ksi	5.5	MPa	0.8	ksi	5.45
Compressive strength	110	MPa	16	ksi	30	MPa	4.35	ksi	3.37

Table 2: Mean material properties of used FRP reinforcement

Material property	Epoxy matrix				CFRP			
Tensile MOE	2,800	MPa	406	ksi	164,000	MPa	23,786	ksi
Tensile strength	75	MPa	11	ksi	2,200	MPa	319	ksi

Table 3: Mean material properties for numerical simulation

Modulus	Timber				CFRP				GFRP			
E_{11}	9,000	MPa	1,305	ksi	210.000	MPa	30,458	ksi	38.000	MPa	5,511	ksi
E_{22}	315	MPa	46	ksi	14.000	MPa	2,031	ksi	11.500	MPa	1,668	ksi

E_{33}	616 MPa	89 ksi	14.000 MPa	2,031 ksi	11.500 MPa	1,668 ksi
G_{21}	740 MPa	107 ksi	7.368 MPa	1,069 ksi	4.000 MPa	580 ksi
G_{32}	42 MPa	6 ksi	491 MPa	71 ksi	4.000 MPa	580 ksi
G_{31}	620 MPa	90 ksi	491 MPa	71 ksi	4.000 MPa	580 ksi

Interface			
σ_c = 2.20 MPa (319 psi)	τ_c = 4.50 MPa (653 psi)	δ_n = 0.50 mm (0.020”)	δ_t = 0.57 mm (0.022”)