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Analysis of timber-framed walls coated with CFRP strips strengthened fibre-plaster boards

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Abstract

This paper provides an experimental analysis of timber-framed walls, coated with carbon fibre-reinforced polymers (CFRP) strengthened fibre-plaster boards, usually used as main bearing capacity elements in the construction of prefabricated timber structures. The tensile strength of the fibre-plaster boards is lower than the strength of timber frame, therefore it is convenient to strengthen boards with high-strength materials in order to gain a higher capacity. It has been shown that the inclusion of CFRP diagonal strip reinforcement on the load-carrying capacity can be quite high and that it is maximized when the carbon strips are connected to the timber frame. On the other hand, the ductility itself was not significantly improved. The test samples proved an important distinction in behaviour in timber frame-fibreboard connecting area, dependant on the boundary conditions between inserted CFRP strips and timber frame. It has been shown that proposed simplified Eurocode 5 methods, applicable for wood-based sheathing boards, could be unsuitable for the problems presented.

Keywords: Timber structures; Walls; Fibre-plaster boards; Carbon fibres; Experiments

1. Introduction

Timber is commonly associated with lightweight construction although it is ubiquitous as a building material. Timber construction is an important part of the infrastructure in a number of areas around the world. Well-built timber structures usually maintain good performance under the influence of wind and especially earthquake forces. Wood itself is a very resilient material which has no high ductility in all directions. Although the only property with high deformation capacity is compression perpendicular to fibres, flexibility of mechanical fasteners usually provides high damping capacity between connected elements.

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In addition to the important applications of timber in bridges, railroad infrastructure, and many other applications, there is an increasing tendency worldwide toward building multi-level prefabricated timber structures with timber-framed walls as the main bearing capacity elements. Their load-carrying capacity becomes critical, especially when taller structures are subjected to heavy horizontal forces, particularly with structures located in seismic and windy areas. In this case it is sometimes necessary to reinforce the walls.

As the tensile strength of timber is usually not much lower than the compressive strength, the applications of fibre-reinforced polymers (FRP) or carbon fibre-reinforced polymers (CFRP) in timber have not been frequent as in masonry or especially in concrete structures. The potential of FRP in combination with steel and timber structures has only been explored recently. The main advantages of using FRP in particular compared to other materials (for example steel plates) are their corrosion resistance, light weight and flexibility, which allow convenient and easy transport to the place of errection.

The availability of advanced composite materials has stimulated much interest in reinforcement of timber elements, especially on *glued laminated beams*. Timber is an uncommon material for critical highway bridge structures, though several applications of strengthening using FRP and CFRP to gain higher ductility and bending resistance can be found in this field. Dagher and Breton (1998) reinforced laminated timber beams in the tensile area using FRP lamellas. The test results showed an essential increase in bending resistance. Stevens and Criner (2000) conducted an economic analysis of 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 presented FRP solution. The test results using carbon fibres in laminated beams are presented in Bergmeister and Luggin (2001).

Composite reinforcement on *sawn timber* elements is less common in literature although many applications exist, especially for retrofitted reinforcement. Timber beams reinforced with a layer of high-modulus composite material may be analysed using a transformed section of an equivalent wood (Johns and Lacroix, 2000), but the influence of composite reinforcement on the bending resistance of the timber elements is usually not particularly high. The reason is that, unlike concrete or masonry, the contribution of the tension zones to the bending resistance continues to be very high. Johns and Racin (2001) demonstrated their experimental studies using glass fibres to reinforce sawn timber sections. The test results presented here show that strength increases are far greater than those predicted by simple engineering bending calculations. They confirm that the composite material adjacent to the sawn wood, even wood of low quality, has an essential effect on the wood elements. Studies of the wood members reinforced with FRP materials in the form of sheets (Triantafillou, 1997) also show that the effectiveness of FRP reinforcement can be quite high, and that this is maximized when the fibres are placed in the longitudinal direction of elements.

Literature provides few investigations on *wood-based panels* strengthened with high-strength fibres (HSF). The use of HSF sheathing material does not increase the bearing capacity much if mechanical fasteners are applied to connect the *wood-based sheets* to the timber frame. Kent and Tingley (2001) presented experimental results for high-strength synthetic fibre reinforced panels bonded to hollow beams. They showed that a glass-aramid reinforced plastic (GARP) placed on the narrow dimension in the extreme tension zone increased an average strength and stiffness of elements (by 22% and 5% compared to the unreinforced test samples, respectively). Test experiments performed in EMPA on wood-based panels reinforced with Sika CarboDur strips demonstrated an essential increase in bending resistance by 43% (Zagar, 1999).

Use of HSF and CFRP for the repair and strengthening of timber elements opens new perspectives for timber structures design. Continuously decreasing prices of these materials make the new technology more economical and interesting. On the other hand, applying composite fibres to timber structures requires experience and higher quality of workmanship than traditional reinforcements.

This paper presents results of experiments performed on sawn timber-framed walls coated with boards made from fibre-plaster material, recently the most frequently used in Central Europe. One of the most important reasons for an increased application of these types of gypsum products is their relatively good fire protection. For example, a single gypsum sheathed board of thickness 15 mm assures 45 min of fire protection (according to Knauf, 2002). Additionally, gypsum is a healthy natural material and is consequently particularly desired for residential buildings.

It may be useful to underline that the precise type of fibrous panel product used in the walls tested in this research is not common in North America, though presented results may be of interest to engineers attempting to develop techniques for the reinforcement of wood-framed walls sheathed with essentially brittle panel products.

Section 2 describes the problem of employing treated elements in heavy seismic or windy areas. Tensile strength of fibre-plaster boards (FPB) is very low, therefore boards are reinforced with CFRP strips, inserted in a tensile diagonal direction. Later, in Section 3, test configurations using different strips and boundary conditions are described and test results are analysed and discussed in Section 4.

2. Problem description

The treated wall is a composite element consisting of framed panels made from sheets of board-material fixed by mechanical fasteners to one or both sides of the timber frame (Fig. 1). There are many types of panel products available which may have some structural capacity such as wood-based materials (plywood, oriented strand board, hardboard, particleboard, etc.) or plaster boards and, more recently fibre-plaster boards. In the following analysis we limited our attention to the fibre-plaster boards (FPB).

In structural analysis panel walls for design purposes can be regarded separately as vertical cantilever beams with the horizontal force ($F_{\mathrm{H}} = F_{\mathrm{H,tot}}/n$) acting at the top (Fig. 1). Considered supports approximate an influence of neighbouring panel walls and assure an elastic-clamped boundary condition for the treated wall, as can be found for example in Faherty and Williamson (1989), Hoyle and Woeste (1989), Schulze (1996) and Eurocode 5 (EC5).

2.1. Experimental studies and design methods

Research activities regarding wood-based walls date back to the beginning of the last century. There are large volumes of experimental and numerical results on timber diaphragms and shear walls, but it is not

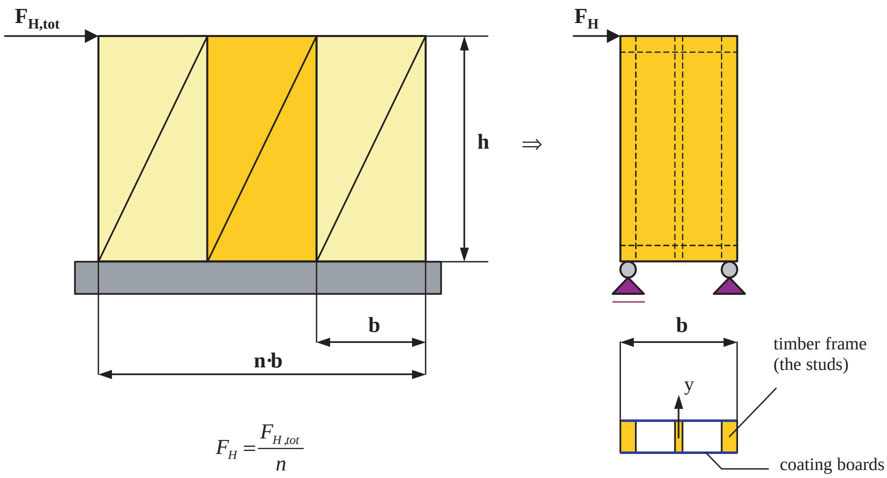

Fig. 1. Static design and cross-section of the treated panel wall.

possible here to present an extensive review of all available information. Thus, a brief overview of some references from previous decades is presented.

Experimental studies were conducted on the structural behaviour of wood-based diaphragms, on system components such as connections (Chou and Polensek, 1987; Polensek and Bastendorf, 1987) and on the spacing of fasteners (Van Wyk, 1986). The EC5 experimental method determines the load-carrying capacity of wall elements by testing the protoype structures in accordance with EN 594.

Many *design models* have been proposed in order to analyse and predict the behaviour of *wood-based* shear walls and diaphragms subjected to lateral loads. Källsner (1984) and Äkerlund (1984) proposed an agreeable approach to determine the load-carrying capacity of the wall unit, based on the following key assumptions:

- behaviour of the joints between the sheet and the frame members is assumed to be linear-elastic until failure,
- the frame members and the sheets are assumed to be *rigid* and *hinged to each other*.

The influence of shear deformations in the fibreboard can be additionally estimated by introducing the shear angle. Additionally, two models are presented based on the assumption that the load–displacement relation of fasteners is completely plastic. Källsner and Lam (1995) presented the walls load-carrying capacity as a function of fasteners spacing along the upper horizontal timber member assuming constant fastener spacing along all timber members.

Two simplified computational methods are given in the final draft of EC5 (2002) in order to determine the load-carrying capacity of the wall diaphragm. The first simplified analysis—*Method A*, is identical to the "Lower bound plastic method", presented by Källsner and Lam (1995). This method defines the wall's shear resistance ($F_{v,d}$) as a sum of all the fasteners' shear resistances along the loaded edges in the form of:

$$F_{v,d} = \sum F_{f,Rd} \frac{b_i}{s} \cdot c_i \tag{1}$$

$F_{f,Rd}$ is the lateral design capacity per fastener; b_i is the wall panel width; s is the fastener spacing.

$$c_i = \begin{cases} 1 & \text{for } b_i \geqslant b_0 \\ \dfrac{b_i}{b_0} & \text{for } b_i \leqslant b_0 \end{cases} \quad \text{where } b_0 = h/2 \tag{2}$$

This is only an approximated and simplified definition, which can be applicable for wood-based panels where the strength is relatively high and the elements tend to fail because of fastener yielding.

The second simplified analysis—*Method B* is applicable to walls made from sheets of wood-based panel products only, fastened to a timber frame. The fastening of the sheets to the timber frame should either be by nails or screws, and the fasteners should be equally spaced around the perimeter of the sheet. According to Method A the sheathing material factor (k_n), the fastener spacing factor (k_s), the vertical load factor ($k_{i,q}$) and the dimension factors for the panel (k_d) are included in the design procedure in the form of:

$$F_{v,d} = \sum F_{f,Rd} \frac{b_i}{s_0} \cdot c_i \cdot k_d \cdot k_{i,q} \cdot k_s \cdot k_n \tag{3}$$

where

$$s_0 = \frac{9700 \cdot d}{\rho_k} \tag{4}$$

d is the fastener diameter; ρ_k is the characteristic density of the timber frame.

Presentation of both EC5 (2002) design methods and the test method (EN 594, 2003) is almost a compromise and not obvious in all details, though there is a fundamental difference in the methods, namely in the way of vertical anchoring of the stud to the tension side of the wall unit.

Analytical models were also developed to predict the dynamic response of the timber shear walls, Stewart (1987), Dolan and Foschi (1991). Finally, Kasal et al. (1994) developed a three-dimensional finite element model to investigate the responses of complete light-frame wood structures.

2.2. Influence of fibre-plaster coating boards

In the above mentioned methods boards made of *wood-based materials* are not mathematically considered as a composite part of the wall unit. There is a fundamental assumption that the horizontal force is transformed over the mechanical fasteners in the connection area to the effective tensile diagonal of the board and from there to the support (Fig. 2). The board's thickness is thus defined according to the tensile diagonal force (T), the corresponding effective width (b_{eff}) and to the tensile strength of the sheathing material. Simplified forms for practical use can be found for example in Schulze (1996) or Brüninghoff (1988).

The influence of boards on the total design racking strength of the wall can also be found in a very simple form in *Simplified Method B* in EC5 (2002), which is practically modified formula of *Method A*.

A simplified formula for horizontal deflection at the top of a wall considering cantilever-bending deflection (w_{t}), shear deflection of the wood-based sheathing boards (w_{b}), flexibility of timber-sheathing connections (w_{c}) and deflection due to anchorage details (w_{a}) can be found in Faherty and Williamson (1989) and Hoyle and Woeste (1989):

$$w = w_{\mathrm{t}} + w_{\mathrm{b}} + w_{\mathrm{c}} + w_{\mathrm{a}} = \frac{8 \cdot F_{\mathrm{H}} \cdot h^3}{E_{\mathrm{t}} \cdot A_{\mathrm{t}} \cdot \mathrm{b}} + \frac{F_{\mathrm{H}} \cdot h}{G_{\mathrm{b}} \cdot t} + 0.376 \cdot h \cdot e_{\mathrm{n}} + d_{\mathrm{a}} \tag{5}$$

E_{t} is the elastic modulus of timber elements; A_{t} is the area of boundary vertical timber element cross-section; G_{b} is the modulus of rigidity of coating boards; t is the effective thickness of coating boards; e_{n} is the nail deformation.

All the above mentioned methods are usually unsuitable for treated walls sheathed with fibre-plaster boards (FPB). The main assumptions do not exactly coincide with the real state of FPB, in which the tensile strength is evidently lower than the compressive strength. Consequently, cracks in a tensile zone usually appear under heavy horizontal loads before stresses on the fasteners reach their yielding point, and the

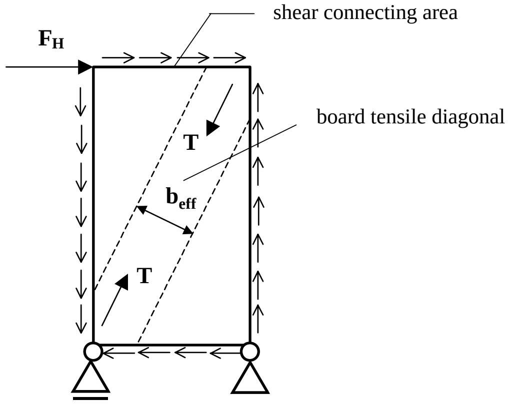

Fig. 2. Considered force distribution.

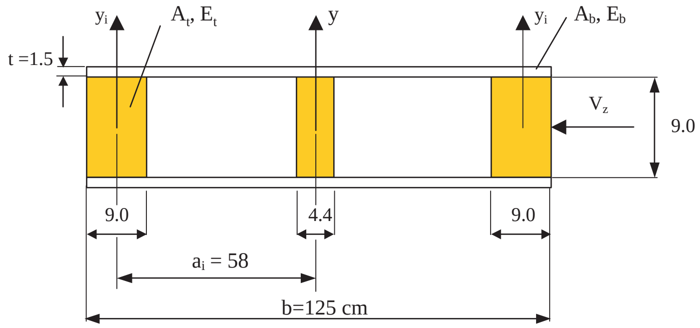

Fig. 3. Cross-section of test samples.

fibreboards do not behave usually as rigid elements (Dobrila and Premrov, 2003). However, by employing FPB as a coating material, a horizontal load shifts a part of the force over the mechanical fasteners to the fibreboard and the wall acts like a deep beam. Distribution of the horizontal force by composite treatment of the element depends on the proportion of stiffness. The effective bending stiffness $(EI_y)_{\text{eff}}$ of mechanically jointed beams which empirically considers the flexibility of fasteners via coefficient γ_y, taken from EC5 (2002), can be written in the form of:

$$(EI_y)_{\text{eff}} = \sum_{i=1}^{n} E_i \cdot (I_{yi} + \gamma_i \cdot A_i \cdot a_i^2) = \sum_{i=1}^{n_{\text{timber}}} (E_i \cdot I_{yi} + E_i \cdot \gamma_i \cdot A_i \cdot a_i^2)_{\text{timber}} + \sum_{j=1}^{n_{\text{board}}} (E_i \cdot I_{yi})_{\text{board}} \tag{6}$$

where n is the total number of elements in the considered cross-section and a_i is the distance between global y-axis of the whole cross-section and local y_i-axis of the ith element with a cross-section A_i (see Fig. 3). The second moment of area for timber about the local y_i-axis $(E_i.I_{yi})_{\text{timber}}$ is in comparison with other values very small and may be neglected. In this case the above equation results in an approximation:

$$(EI_y)_{\text{eff}} \approx \sum_{i=1}^{2} (E_i \cdot \gamma_i \cdot A_i \cdot a_i^2)_{\text{timber}} + \sum_{j=1}^{2} (E_i \cdot I_{yi})_{\text{board}} \approx \gamma_y \cdot (EI_y)_{\text{timber}} + (EI_y)_{\text{board}} \tag{7}$$

It is evident that the force distribution in this case strongly depends on the stiffness coefficient of the connecting area (γ_y), which mostly depends on the fasteners slip modulus (K_{ser}) and fasteners disposition, as well as on the type of the connection. An experimental analysis on the influence of fasteners spacing on behaviour of the treated walls can be found in Dobrila and Premrov (2003).

2.3. Strengthening of fibre-plaster boards

As described, the FPB are usually a weaker part of the presented composite system, because their tensile strength is evidently smaller than the wood strength of all members in the timber frame. Thus, especially in multi-level buildings located in seismic or windy areas, cracks in FPB usually appear. In these cases the FPB lose their stiffness and therefore their resistance should not be considered at all. Stresses in the timber frame under a horizontal loads are usually not critical.

There are several possibilities to reinforce panel walls in order to avoid cracks in FPB:

- by using additional boards. The boards are usually doubled:
  - symmetrically (on both sides of a timber frame),
  - non-symmetrically (on one side of a timber frame),

- by reinforcing boards with steel diagonals,
- by reinforcing boards with carbon or high-strength synthetic fibres.

In Dobrila and Premrov (2003) we presented the first possibility experimentally using *additional FPB*, which gave higher elasticity of elements, whilst bearing capacity and especially ductility were not improved in the desired range. Wolf (2001) presented theoretical parameter-study of the influence on racking resistance by inserting an additional *interior sheet* (web-sheet) using an elastic model. In the so-called I-framed wall the additional interior sheet is glued into routs of the upper side of the sole plate and to the bottom side of the top plate, respectively.

With the intention to improve the resistance and especially the ductility of the walls it is more convenient to insert *diagonal steel strips*, which have to be fixed to the timber frame. In this case only a part of the horizontal force is shifted from boards over the tensile steel diagonal to the timber frame after the appearance of the first crack in the tensile zone of FPB (Dobrila and Premrov, 2003). An enlarged effective cross-section of FPB (A^*_{1b}) can approximately be computed considering the compatibility conditions between the actual reinforced and fictitious unreinforced element. The computational procedure is described in details in Premrov and Dobrila (2002) and will not be presented here. With regard to the fictive enlarged cross-section of FPB we proposed two approximate analytical models using either fictitious thickness (t^*) or fictitious width (b^*) of fibreboards:

$$t^* = \frac{A^*_{1b}}{b} = t + \frac{1}{\chi} \cdot \frac{E_s}{G_\mathrm{b}} \cdot \sin^2\alpha \cdot \cos\alpha \cdot A_{1s,0} \cdot \frac{1}{b} \tag{8}$$

$$b^* = \frac{A^*_{1b}}{t} = b + \frac{1}{\chi} \cdot \frac{E_s}{G_\mathrm{b}} \cdot \sin^2\alpha \cdot \cos\alpha \cdot A_{1s,0} \cdot \frac{1}{t} \tag{9}$$

In the above equations α represents the angle of inserted steel diagonals with the net area ($A_{1s,0}$). A non-dimensional coefficient χ is shear cross-section coefficient defined as a proportion between the shear and actual cross-sectional area of the FPB with the shear modulus (G_b).

Alongside the steel diagonals' influence these models enable simultaneous consideration of the fasteners' flexibility between the board and the timber frame and any appearing cracks in the tensile area of the FPB. Unreinforced panels (without steel diagonals) can be computed using actual dimensions of the fibreboards. Numerical results presented in Premrov and Dobrila (2002) on diagonally steel reinforced elements show good agreement with measurements performed on the test samples.

As the tensile strength of FPB is obviously lower than the compressive strength and corresponding capacity of timber frame, the treated elements tend to fail because the cracks are forming in the tensile area of the FPB, therefore this tensile area could be reinforced with *high-strength materials*. This strengthening concept is such that the composites would contribute to tensile capacity when the tensile strength of FPB is exceeded. No FRP applications on the treated fibre-plaster boards were found in the literature.

3. Test configuration

Three sample groups from total of nine test samples were tested in order to carry out appropriate experimental research on the influence of CFRP strengthened walls. All test groups consisted of three panel walls of actual dimensions $h = 263.5$ cm and $b = 125$ cm. The cross-section presented in Fig. 3 was composed of timber studs ($2\times9\times9$ and $1\times4.4\times9$ cm), timber girders ($2\times8\times9$ cm) and Knauf fibre-plaster boards (Knauf, 2002) of thickness $t = 15$ mm. They were fixed to the timber frame using staples of $\varnothing$ 1.53 mm at an average spacing of $s = 75$ mm.

The static model according to Fig. 1 was used for all groups of test samples. The samples were actually rotated by 90° according to Fig. 1 and they were therefore subjected to vertical force acting at the end of the elements (Fig. 4a). The tensile support was simulated with three M16 bolts and with two thin steel plates ($5\times120\times600$ mm. These steel plates were anchored to the rigid steel frame over 2 [NP10 using two tensile M16 bolts of length $l_1 = 210$ mm.

The FPB were reinforced in the *tensile diagonal area* using SikaWrap-230C strips (Sika, 2003) made from carbon high-strength fibre reinforced polymers of thickness 1.2 mm. Strips with different widths (300 or 600 mm) and of different boundary conditions were glued to the FPB.

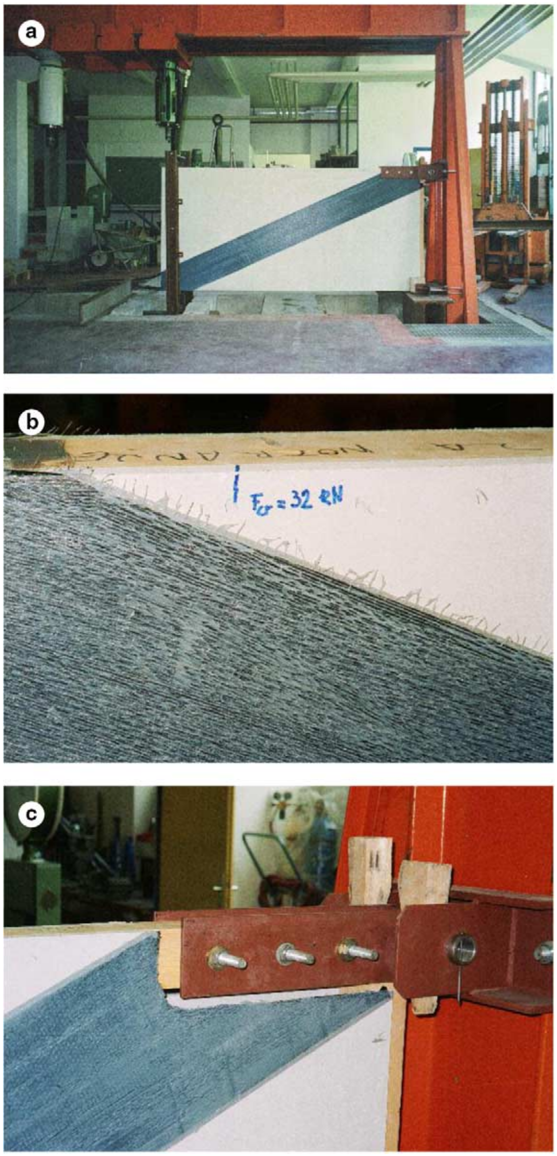

Fig. 4. (a) G1: the static system, (b) G2: the CFRP strip is glued on the FPB and additionally to the timber frame, (c) G3: the CFRP strip is not glued to the timber frame.

Table 1
Properties of used materials

	$E_{0,m}$ [N/mm^2]	G_m [N/mm^2]	$f_{m,k}$ [N/mm^2]	$f_{t,0,k}$ [N/mm^2]	$f_{c,0,k}$ [N/mm^2]	$f_{v,k}$ [N/mm^2]	ρ_m [kg/m^3]
Timber C22	10,000	630	22	13	20	2.4	410
Fibre-plaster board	3000	1200	4.0	2.5	20	5.0	1050
SikaWrap-230C	231,000	–	–	4100	–	–	1920

$E_{0,m}$ is the average value of the modulus of elasticity (for timber parallel to the grain); G_m is the average shear modulus (for timber and FPB only); $f_{m,k}$ is the characteristic bending strength (for timber and FPB only); $f_{t,0,k}$ is the characteristic tensile strength (for timber parallel to the grain); $f_{c,0,k}$ is the characteristic compressive strength (for timber parallel to the grain); $f_{v,k}$ is the characteristic shear strength (for timber and FPB only); and ρ_m is the average density.

The first group (G1) of three test samples was additionally reinforced with two CFRP diagonal strips (one in each FPB) of width 300 mm which were glued on the FPB using Sikadur-330 LVP. The strips were additionally glued to the timber frame (Fig. 4a and b) to ensure the transmission of the force from FPB to the timber frame.

The second group (G2) of three test samples was additionally reinforced with two CFRP diagonal strips of width 600 mm. The strips were glued on FPB and to the timber frame as in G1 (Fig. 4b) to ensure the transmission of the force from FPB to the timber frame.

The third group (G3) of three test samples was additionally reinforced with two CFRP diagonal strips of width 300 mm as in G1 but they were *not glued to the timber frame* (Fig. 4c).

Material properties for the test samples for all groups were the same (Table 1). Values for timber of quality C22 are taken from EN 338 (2003), the characteristics of fibre-plaster boards from Knauf (2002) and for carbon strips Sika (2003) data were used.

Considering the presented dimensions and material properties the fasteners slip modulus (K_{ser}) and coefficient γ_y can be computed using EC5 (2002) standards:

$$\rho_{\mathrm{mean}} = \sqrt{\rho_{\mathrm{b}} \cdot \rho_{\mathrm{t}}} = \sqrt{1050 \times 410} = 656.12\ \mathrm{kg/m^3};\ K_{\mathrm{ser}} = \frac{\rho_{\mathrm{mean}}^{1.5} \cdot d^{0.8}}{120} = 196.81\ \mathrm{N/mm} \tag{10}$$

$$k_{yi} = \frac{\pi^2 \cdot A_{t1} \cdot E_{\mathrm{t}} \cdot s}{L_{\mathrm{eff}}^2 \cdot 2 \cdot K_i} = \frac{\pi^2 \cdot 9^2 \times 1000 \times 7.5}{(2 \times 263.5)^2 \cdot 2 \times 196.81} = 5.48;\ \gamma_{yi} = \frac{1}{1 + k_{yi}} = \frac{1}{1 + 5.48} = 0.154 \tag{11}$$

The effective bending stiffness $(EI_y)_{\mathrm{eff}}$ of the unstrengthened test samples can be obtained using Eq. (6):

$$\begin{aligned}(EI_y)_{\mathrm{eff}} &= 300\frac{2 \times 1.5 \times 125^3}{12} + 1000 \cdot \left(\frac{2 \times 9^4}{12} + \frac{4.4^3 \times 9}{12} + 2 \times 9 \times 9 \times 58^2 \times 0.154\right) \\ &= 2.3168 \times 10^8\ \mathrm{kN\,cm^2}\end{aligned} \tag{12}$$

4. Test results and analysis

The force forming the first crack (F_{cr}) in the FPB, the crushing force (F_u), the maximal cantilever bending deflection (w) under the acting force (F) and the slip (Δ) in the tensile area between the FPB and the timber frame were all measured. The measured values for the unstrengthened (UNS) test samples were taken from Dobrila and Premrov (2003) and included for information and comparison only.

Average force forming the first crack (F_{cr}):

G1: $F_{cr,1} = 24.28$ kN G3: $F_{cr,3} = 35.90$ kN
G2: $F_{cr,2} = 32.13$ kN UNS: $F_{cr,uns} = 17.67$ kN

Average crushing force (F_u):

G1: $F_{u,1} = 40.33$ kN G3: $F_{u,3} = 36.26$ kN
G2: $F_{u,2} = 46.27$ kN UNS: $F_{u,uns} = 26.02$ kN

It is evident that the elastic resistance (force forming the first crack) essentially increased for all kinds of CFRP strengthened test samples, but mostly for samples G3, where the CFRP strips were not fixed to the timber frame. The CFRP influence was not so obvious at samples G1, where carbon strips of the same dimensions were additionally glued to the timber frame. Of course, when comparing samples G1 and G2, the influence of strengthening depends on the width of the inserted diagonal strips. It is also interesting to mention that for all groups of test samples, the cracks dispersed before they reached the CFRP strips and did not extend to the strip at all (Fig. 5).

On the other hand, when comparing the measured results of the crushing force, a greater improvement can be noticed in the groups where the CFRP diagonals were glued to the timber frame. Compared to the unstrengthened test sample, the crushing force in samples G2 was increased by 78%. In samples G3 the crushing force practically coincided with a force forming the first crack, so cracks hardly appeared at all, which is not a good solution to ensure better ductility, necessary for seismic design.

Further information on the behaviour of tested elements can be obtained by calculation of the "safety" (c_i) and "ductility coefficients of FPB" (d_i) in the following forms:

$$c_1 = \frac{F_{u,1}}{F_{cr,1}} = 1.66,\ c_2 = \frac{F_{u,2}}{F_{cr,2}} = 1.44,\ c_3 = \frac{F_{u,3}}{F_{cr,3}} = 1.01;\ c_{uns} = 1.47 \tag{13}$$

$$d_1 = \frac{u_{(F_{u,2})}}{u_{(F_{cr,2})}} = \frac{55.06}{19.67} = 2.80,\ d_2 = \frac{u_{(F_{u,2})}}{u_{(F_{cr,2})}} \frac{63.15}{23.71} = 2.66,\ d_3 = \frac{u_{(F_{u,3})}}{u_{(F_{cr,3})}} \cong 1.0;\ d_{uns} = 2.71 \tag{14}$$

It can be seen from the calculated non-dimensional coefficients that the CFRP strips should be fixed (glued) to the timber frame in a certain way to assure satisfactory safety and the ductility of the coating fibreboards. In this case only, a part of the horizontal force is shifted from the FPB over the tensile strip to the timber

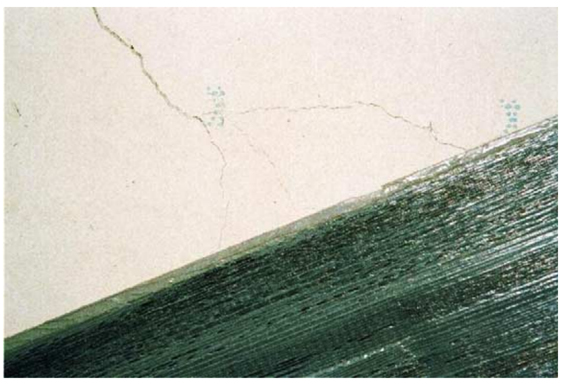

Fig. 5. Dispersion of cracks near the CFRP strip.

frame after the appearance of the first crack in the fibreboard. This principle is practically the same as if reinforced with BMF steel diagonals, where slightly higher safety ($c = 1.93$) and "ductility" ($d = 2.97$) were achieved, Dobrila and Premrov (2003).

The coating boards of the test samples in which the CFRP strips were not fixed to the timber frame (samples G3), demonstrate typical non-ductile behaviour ($d_3 \approx 1.0$). It is probably the consequence of the fact, that basically CFRP behave as linear-elastic up to the failure.

For further analysis it is important to present measured cantilever deflections (w) under the acting force (F, Fig. 6) and slips (Δ) in the connecting area (Fig. 7).

First of all, it is convenient to compare the predicted bending stiffness, calculated with Eq. (12), with results measured. The deflection measured at force $F = 4$ kN was $w_{\text{meas}} = 1.212$ mm and if shear deformations are neglected we arrive at:

$$(EI_y)_{\text{eff,exp}} = \frac{F \cdot h^3}{3 \cdot w_{\text{meas}}} = \frac{4 \times 263.5^3}{3 \times 0.1212} = 2.0127 \times 10^8 \text{ kN cm}^2 \tag{15}$$

There is quite a difference between experimental stiffness $(EI_y)_{\text{eff,exp}}$. and the computed values of bending stiffness $(EI_y)_{\text{eff}}$ from (Eq. (12)), the reason lies in flexibility of bolts at the tensile support and in shear deformations neglection.

It is evident from Fig. 6 that, similarly to the classical reinforcement with BMF steel diagonals presented in Dobrila and Premrov (2003), there is practically no influence on stiffness of any reinforcement before appearance of cracks in the unstrengthened FPB. This is logical because in this case the reinforcement is practically not activated at all and its stiffness in comparison to the stiffness of uncracked FPB is small. After appearance of the first crack in the unstrengthened test samples ($F_{\text{cr,uns}} = 17.67$ kN) the influence of the CFRP strips is obvious and it depends on the strip's dimensions as well as on the boundary conditions between the strips and the timber frame.

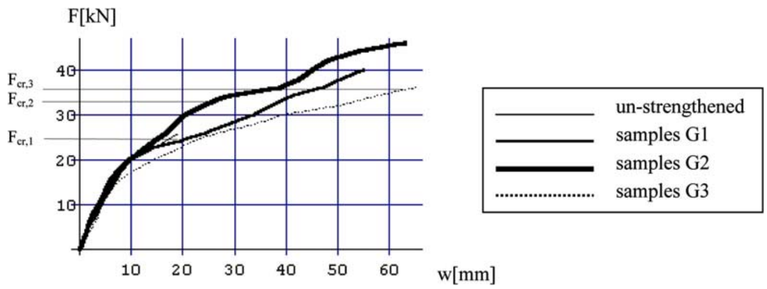

Fig. 6. Measured average bending deflections (w) under the force F.

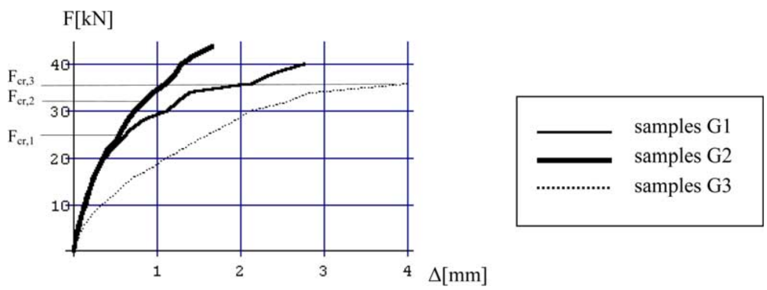

Fig. 7. Measured average slips (Δ) in the connecting area.

Closer look at the graph in Fig. 6 at $F > 17$ kN reveals an obvious difference in the behaviour of the test samples when the CFRP strips were glued to the timber frame (samples G1 and G2) or if they were not (samples G3). Beside the fact that samples G1 and especially G2 demonstrated higher load-carrying capacity than samples G3, it is also important to mention that samples G1 and G2 produced substantially smaller slip than samples G3, which never exceeded 1mm at the first crack forming (Fig. 7). Therefore it can be assumed that the yield point of the fasteners was not achieved before cracks appeared at all. Consequently, the walls tend to fail because of the crack forming in FPB. In this case of strengthening the ductility of the whole wall element (see Fig. 6 for samples G1 and G2) practically coincides with the "ductility" of FPB, as proposed with d_1 and d_2 coefficients.

In contrast, in *G3* model, where the CFRP strips were unconnected to the timber frame, the slip (Δ) between the FPB and the timber frame was evidently higher than in samples G1 and G2, and exceeded 3mm when the first crack in FPB appeared (Fig. 7). The load–displacement relation ($F - \Delta$) of the fasteners was in this case at the force which produced first cracks almost completely plastic. Since the tensile strength of FPB is essentially improved, the walls tend to fail because of fastener yielding and therefore, as it is described in Section 2.1, the "Lower bound plastic method" (EC5 Simplified Method A) can be used to determine the wall's load carrying capacity (Eq. (1)). Although the fibreboards in samples G3 demonstrated practically no deformation capacity ($d_3 \approx 1.0$, Eq. (14)) the ductility is formed ($F - w$ diagram, Fig. 6) over the fasteners yielding (Fig. 7).

The increasing slip in samples G3 directly effects decreasing bending stiffness, which is experimentally noticed when one compares samples G3 with G1 and G2 at higher forces, especially when $F > 25$ kN (Fig. 6). On the other side it can be calculated from Eqs. (10)–(12) over the decreasing values for the fastener slip modulus (K) and coefficient γ_y.

5. Conclusions

Since the tensile strength of the coating fibre-plaster boards (FPB) is obviously lower than the compressive one, the treated elements tend to fail because cracks are forming in the tensile area of the FPB. Therefore, in order to avoid cracks, strengthening with CFRP strips, placed in a tensile diagonal direction of FPB, as a high strength material can be recommended.

As shown, there is practically no influence on the element stiffness of any reinforcement before cracks appeared in the unstrengthened FPB. However, after the first cracks in unstrengthened FPB appeared, the test samples demonstrated an important difference in behaviour dependant on the boundary conditions between the inserted CFRP strips and the timber frame.

If strips are glued to the timber frame the fasteners produced substantially smaller slip, which never exceeded 1mm when the first cracks appeared (Fig. 7). Therefore it can be assumed that the yield point of the fasteners is not achieved before cracks appeared at all and the elements tend to fail because cracks appear in the tensile area of FPB. Therefore, it is not recommended to use EC5 simplified methods to predict the element resistance. Simple mathematical models with a fictive enlarged cross-section of FPB are proposed in Premrov and Dobrila (2002). In such cases of strengthening ductility of the whole element practically coincides with ductility of FPB. Ductility and load-carrying capacity additionally depend on the dimensions of the inserted CFRP strips.

In the case where the CFRP diagonals are unconnected to the timber frame, the slip between the FPB and the timber frame is evidently higher and the load–displacement relation of the fasteners is, after the cracks appeared, almost perfectly plastic. Since the tensile strength of FPB is with CFRP highly improved, the walls tend to fail because of fastener yielding, similar as at wood-based sheathing boards. Therefore the "Lower bound plastic method" (Eq. (1) or Eq. (3)) can be used to determine the wall's load carrying capacity. In this case of strengthening the ductility of the walls can be assured over the fastener yielding and does not depend on the deformation capacity of coating boards at all.

As we know the costs of employing CFRP are at the moment rather high. Experimental results presented here justified these high costs with much higher forces forming the first crack, load-carrying capacity and stiffness increase, especially at CFRP reinforced elements, with strips connected to the timber frame.

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