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Mathematical modelling of reinforced timber-gypsum fibreboard panel shear walls

Miroslav Premrov and **Peter Dobrila**
University of Maribor, Faculty of Civil Engineering, Smetanova 17, SI-2000 Maribor, SLOVENIA
e-mail: miroslav.premrov@uni-mb.si; peter.dobrila@uni-mb.si

SUMMARY

The paper presents mathematical models for solving reinforced panel shear walls, which are used as load-carrying capacity walls in the construction of prefabricated timber structures. The panels are composed of timber frame and fibre-gypsum boards which are reinforced with steel diagonals. Analytical solutions obtained by mathematical modelling with the fictive thickness and height of fibre-gypsum boards are proposed. The obtained computational results are compared with deflections and cracks measured on test samples. They are also compared with those obtained on the panels without reinforcement.

Key words: *timber, panel shear walls, fibre-gypsum boards, mathematical models, steel diagonals.*

1. INTRODUCTION

The presented panel shear walls are usually used as load-carrying capacity walls in the construction of prefabricated timber structures. Shear walls are regarded for design purposes as vertical cantilever beams. They consist of a timber frame and fibre-gypsum boards, which are fixed by mechanical fasteners to one or both sides of a timber frame (Figure 1). In such systems a greater part of the vertical load is usually borne by the timber frame. In engineering design a contribution of fibreboards is usually not considered to a total horizontal stiffness of the shear wall. This does not coincide with the real state. A horizontal load namely shifts a part of the force over the mechanical fasteners to the fibreboards. The boards thus also contribute to the shear (horizontal) stiffness of the walls.

Problems with cracks, which appear in fibre-gypsum boards usually appear especially in multi-storey buildings located in the seismic or wind areas (v_{ref}>*40 m/s*). The critical part of these panels under a horizontal load are fibre-gypsum boards and the stresses in the timber frame are usually not so high. To avoid cracks in the fibre-gypsum boards the producers usually use two boards on one side of the timber frame, so i.e. a total of four boards. Some solutions are presented in Refs. [1-4]. However, we tried to find another solution by reinforcing panels with diagonal steel elements. In this way a part of the force is shifted from the fibre-gypsum boards to the steel diagonals. The aim of our research was to determine computationally and experimentally the difference in the resistance and stiffness between the panel shear walls reinforced with steel diagonals and the unreinforced panels. Some test results obtained by using carbon fibres in laminated beams are presented in Ref. [5]. Investigation results of fiber reinforced hollow wood beams are presented in Ref. [6]. They show that fiber reinforcement increased the average strength and stiffness of the beams, compared to the unreinforced control samples, by *22%* and *5%*, respectively.

The solution of these problems with a finite element method can be very complex. It is especially difficult to consider a mechanical deformability of the fasteners.

In this way it is necessary to develop some simple mathematical models.

2. MATHEMATICAL MODELLING

According to the classical mechanical theory we developed new mathematical models for solving reinforced panel shear walls. Since it is necessary to have a simple mathematical model to solve such problems special attention is dedicated to considering a contribution of stiffness of steel diagonals which are built in the panels.

The derivation of the mathematical model is based on a continuity of horizontal displacements between reinforced and fictive normal panels, see Figure 1.

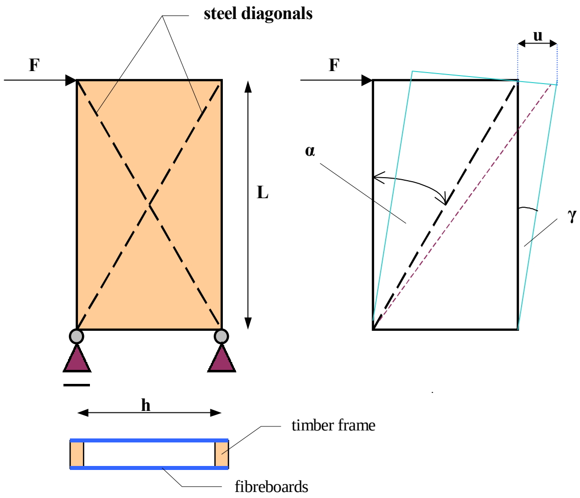

Fig. 1 Computational scheme of the model

Shear deformation in *one fibreboard* is:

$$tg\,\gamma \approx \gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} = \frac{\tau}{G_b} = \frac{F}{2\cdot(dA_{1b})\cdot G_b} = \frac{F}{2\cdot\frac{9}{10}\cdot(dA_{1b})\cdot G_b} \qquad (1)$$

where G_b represents the shear modulus of a fibreboard and dA_{1b} is a fictively enlarged cross-sectional area of one board as the influence of inserted diagonals. The value *9/10* is considered as a shear cross-section coefficient as a proportion between a shear and actual cross-section area. From Figure 1 it is evident that the horizontal displacement of the fibreboard (u_b) is:

$$tg\,\gamma = \frac{u_b}{L} \quad \Rightarrow \quad u_b = \frac{F\cdot L}{2\cdot\frac{9}{10}\cdot(dA_{1b})\cdot G} \qquad (2)$$

The axial force in the tensile steel diagonal is according to Figure 1:

$$S = \frac{F}{2\cdot \sin\alpha} \qquad (3)$$

The horizontal displacement of the *tensile steel diagonal* (u_s) is thus:

$$u_s = \int_l \frac{S\cdot\bar{S}}{E_s\cdot A_s}dx = \frac{F\cdot L}{2\cdot\sin^2\alpha\cdot\cos\alpha\cdot E_s\cdot A_{1s}} \qquad (4)$$

For the steel diagonal it is better to consider the net area ($A_{1s}{}^{0}$). If we consider a continuity of horizontal displacements of the fibreboard and the steel diagonal from Eq. (2) and Eq. (4), we get for the effective *cross-section of one reinforced fibreboard*:

$$dA_{1b} = \frac{10}{9}\cdot\frac{E_s}{G_b}\cdot\left(\cos\alpha - \cos^3\alpha\right)\cdot A_{1s}^{0} \qquad (5)$$

This is the part of one fibreboard, which is effective to the axial stiffness of one steel diagonal in the board. *The total fictive cross section of one reinforced fibreboard is thus*:

$$A_{1b}^{*} = A_{1b} + dA_{1b} = t\cdot h + \frac{10}{9}\cdot\frac{E_s}{G_b}\cdot\left(\cos\alpha - \cos^3\alpha\right)\cdot A_{1s}^{0} \qquad (6)$$

It is evident that the fictive cross section of a reinforced panel ($A_{1b}{}^{*}$) is bigger than a normal one (A_{1b}). The mathematical modelling of reinforced panels yields two possibilities:

a) to use the *fictive "height"* of fibreboards:

$$h^{*} = \frac{A_{1b}^{*}}{t} = h + \frac{10}{9}\cdot\frac{E_s}{G_b}\cdot\left(\cos\alpha - \cos^3\alpha\right)\cdot A_{1s}^{0}\cdot\frac{1}{t} \qquad (7)$$

b) to use the *fictive thickness* of fibreboards:

$$t^{*} = \frac{A_{1b}^{*}}{h} = t + \frac{10}{9}\cdot\frac{E_s}{G_b}\cdot\left(\cos\alpha - \cos^3\alpha\right)\cdot A_{1s}^{0}\cdot\frac{1}{h} \qquad (8)$$

According to the first possibility the height of a developed fictive fibreboard is of course bigger than of a normal one (see Figure 2b). The thickness is not changed. The same holds true if we consider the fictive thickness of the fibreboard. In this case the height of the fibreboard is not changed (see Figure 2c). Hovever, it is very important that for both cases the dimensions of a timber frame are not changed.

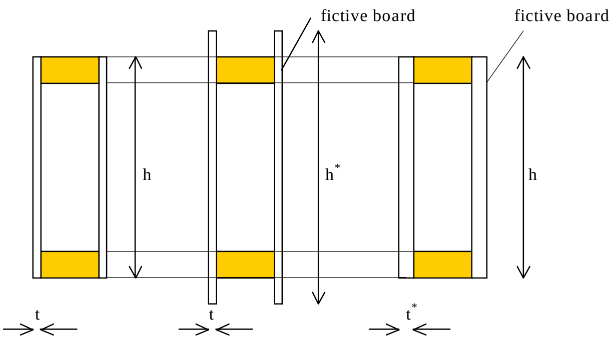

Fig. 2 Mathematical models: a) Normal panel (without reinforcement); b) Panel with a fictive height; c) Panel with a fictive thickness

In the proposed mathematical models it is not difficult to consider a mechanical flexibility of fasteners between fibreboards and a timber frame. By using Eurocode 5 [7] it can be easily considered with the slip modulus (K_{ser}) from Table 4.2 and the coefficient γ according to the equations *(B2a)* and *(B2e)*.

3. TEST SAMPLES

The experiments were performed on six test samples. Three of them were panel shear walls without steel diagonals (test samples T_1, T_2 and T_3) and the three were reinforced with steel diagonals $2\times(2\times60)$ *mm* of BMF-Holzverbinder type [8] (test samples T_4, T_5 and T_6).

3.1 Dimensions of the test samples

All test samples were *255 cm* long and *125 cm* high. They were, according to Eurocode 5 [7], Section 5.4.3, rigidly clamped into a support by bolts and INP steel profiles. The static equilibrium of the shear wall shown in Figure 3 requires the wall to have a tension anchorage at the uplifting end. In practice, such an anchorage will be needed at each end of the wall since the lateral load can be imposed in either direction along the wall. In our experiments the test samples were loaded at the free edge with a vertical force F_v, which symbolically represents the lateral force in real design (rotation for *90°*).

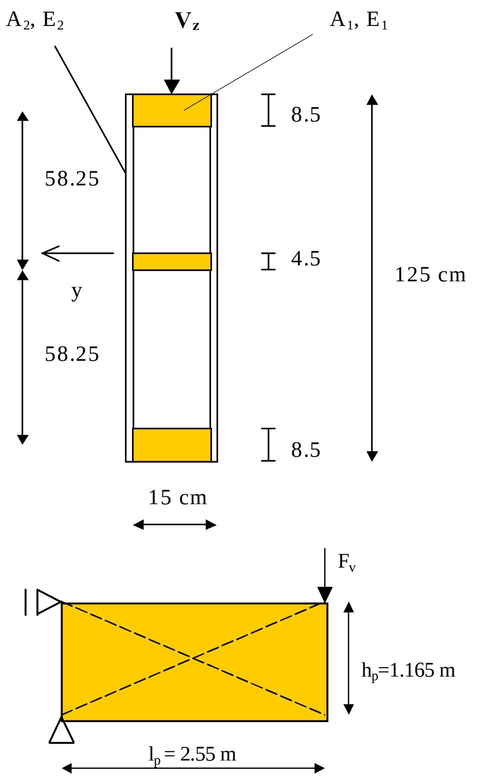

Fig. 3 Scheme of the static system and of the composed cross section of the test samples

The cross-section of all six test samples is composed of (Figure 3):

- a timber frame made of: timber columns *(2×8.5×12 +1×4.5×12) cm*, and timber beams *(2×8.5×12) cm*,
- Fermacell fibre-gypsum boards [9] with the thickness of *1.5 cm*. They are fixed to the timber frame by steel staples ϕ *1.53 mm* at the constant distance $s=9.1$ *cm*.

3.2 Material properties

The considered properties of the timber frame were of the class C22 according to Eurocode 5 classification [7]. The value of the modification factor (k_{mod}) was assumed to be *0.9* (for a short-term load). The relative humidity of timber was less than *20%*. The fibre-gypsum boards were of the Fermacell type. The material properties of the boards were taken from Ref. [9]. Table 1 presents all material properties of the test samples.

3.3 Geometrical properties of the cross section

Table 2 presents all geometrical properties of the normal and reinforced test samples. Equations (7) and (8) are used for the proposed mathematical models according to Figure 2b and Figure 2c. The Table 2 presents the geometrical properties according to different values of the assumed slip modulus of the staples (K_{ser} and $K_u=2/3K_{ser}$), obtained by Eurocode 5, Table 4.2. The expected designed vertical force ($F_{v,d}$) is according to [7] Eq. (5.4.3a):

$$F_{v,d} = \sum_{i=1}^{m} R_d \cdot \left(\frac{b_i}{b_1}\right)^2 \cdot \frac{b_1}{s} = R_d \cdot n \qquad (9)$$

where n represents the whole number of the staples with spacing s on the bound $h_p^{(0)}=125$ *cm*, b_1 is the width of the widest sheet, b_i is the width of other sheets and m is the number of the sheets. In this case the resistance of the boards is not considered. The shear force is just the sum of the shear resistance of the staples (R_d). For our test samples we get $F_{al}=5.58$ *kN* ($K=K_{ser}$) and $Fv,d=8.74$ *kN* ($K_u=2/3K_{ser}$). The proposed values for the case $Ku^*=1/3K_{ser}$ are appended.

Table 1 Material properties of the timber and of the Fermacell fibre-gypsum boards

	$E_{0,mean}$ [MPa]	$E_{90,mean}$ [MPa]	G_{mean} [MPa]	$f_{m,k}$ [MPa]	$f_{t,0,k}$ [MPa]	$f_{c,0,k}$ [MPa]	$f_{v,k}$ [MPa]	ρ_{mean} [kg/m³]
timber	10000	330	630	22	13	20	2.4	410
	$E_{m,z}$ [MPa]	E_D [MPa]	G_0 [MPa]	G_{90} [MPa]	$zul\ \sigma_m$ [MPa]	$zul\ \sigma_{z,0}$ [MPa]	$zul\ \tau_0$ [MPa]	ρ [kg/m³]
fibreboards	3000	1900	1200	1200	1.1	0.5	0.3	1000

Table 2 Geometrical properties of the test samples

	N_{al} = 203 N F_{al} = 5.58 kN	R_d = 318 N $F_{v,d}$ = 8.74 kN	R_k = 459 N $F_{v,k}$ = 12.62 kN
K_j [kN/cm]	$K_{ser} = 3.37$	$K_u = 2/3\ K_{ser} = 2.25$	$K_u^* = 1/3\ K_{ser} = 1.22$
$K_{yj} = 17.52/K_j$	5.22	7.79	14.3
$\gamma_{yj} = (1 + K_{yj})^{-1}$	0.161	0.114	0.065
$(EI_y)_{ef}$ [kN/cm²] x 10^8			
normal panels	2.592	2.267	1.928
reinforced panels			
fictive height	3.336	3.010	2.671
fictive thickness	2.807	2.482	2.143
$(ES_y)_{ef}^*$ [kN/cm²] x 10^8			
normal panels	0.957	0.677	0.386
reinforced panels			
fictive height	0.957	0.677	0.386
fictive thickness	0.957	0.677	0.386
$(ES_y)_{ef}^{max}$ [kN/cm] x10^6			
normal panels	2.714	2.435	2.144
reinforced panels			
fictive height	3.268	2.988	2.697
fictive thickness	2.973	2.693	2.402
ratio reinf./norm. panel			
$(EI_y)_{ef}$			
fictive height	1.287	1.328	1.385
fictive thickness	1.083	1.095	1.112
$(ES_y)_{ef}^{max}$			
fictive height	1.204	1.227	1.258
fictive thickness	1.095	1.106	1.120

Table 3 Shear forces on one staple varying slip modulus

N_{1i} / $V_{zi} = F_{vi}+G$	N_{1i} [N] $K = K_{ser}$	N_{1i} [N] $K = K_u = 2/3\ K_{ser}$	N_{1i} [N] $K = K'_u = 1/3\ K_{ser}$
F_{vi} [kN] V_{zi} [kN]			
Normal panels	$N_i = 0.01680\ V_{zi}$	$N_i = 0.01359\ V_{zi}$	$N_i = 0.00819\ V_{zi}$
F_{v1} = 5.58 V_{z1} = 6.870	$115 < N_{al}$		
F_{v2} = 10.0 V_{z2} = 11.288	$190 < N_{al}$		
F_{v3} = 14.5 V_{z3} = 15.788	$N_{al} < 265 < R_d$	$N_{al} < 215 < R_d$	
F_{v4} = 15.0 V_{z4} = 16.288		$N_{al} < 221 < R_d$	
F_{v5} = 20.0 V_{z5} = 21.288		$289 \approx R_d$	
Reinforced panels fictive height fictive thickness	$N_{1i} = 0.01305\ V_{zi}$ $N_{1i} = 0.01551\ V_{zi}$	$N_{1i} = 0.01023\ V_{zi}$ $N_{1i} = 0.01241\ V_{zi}$	$N_{1i} = 0.00658\ V_{zi}$ $N_{1i} = 0.00820\ V_{zi}$
F_{v1} = 5.58 V_{z1} = 6.870	$90 < N_{al}$ $107 < N_{al}$		
F_{v2} = 10.0 V_{z2} = 11.288	$147 < N_{al}$ $175 < N_{al}$		
F_{v3} = 15.0 V_{z3} = 16.288	$213 > N_{al}$ $253 > N_{al}$	$167 < N_{al}$ $202 = N_{al}$	
F_{v4} = 18.5 V_{z4} = 19.788	$N_{al} < 258 < R_d$ $N_{al} < 307 < R_d$	$202 = N_{al}$ $N_{al} < 246 < R_d$	
F_{v5} = 20.0 V_{z5} = 21.288		$N_{al} < 218 < R_d$ $N_{al} < 264 < R_d$	
F_{v6} = 25.0 V_{z6} = 26.288		$N_{al} < 269 < R_d$ $R_d < 326 < R_k$	$173 < N_{al}$ $N_{al} < 216 < R_d$
F_{v7} = 30.0 V_{z7} = 31.288		$R_d \tilde{}\ 320 < R_k$ $R_d < 388 < R_k$	$206 \tilde{}\ N_{al}$ $N_{al} < 257 < R_d$
F_{v8} = 35.0 V_{z8} = 36.288		$R_d < 371 < R_k$ $R_d < 450 < R_k$	$239 < R_d$ $N_{al} < 298 < R_d$

By comparing the results we can see that the bending $(EI_y)_{ef}$ and the shear stiffness $(ES_y)_{ef}$ are bigger for the model with the fictive height than for the fictive thickness of the fibreboards. The difference is smaller by the shear stiffness. The using of the slip modulus of the fasteners (K) depends on shear forces on the staples.

By considering the contribution of the boards to the whole resistance a shear force on one staple depends on the effective shear stiffness in the connecting area $(ES_y)_{ef}^*$ and on the effective total bending stiffness $(EI_y)_{ef}$ of the model:

$$N_1 = \frac{T_x}{2} \cdot s \cdot V_{zi} = \frac{(ES_y)_{ef}^*}{(EI_y)_{ef}} \cdot \frac{s}{2} \cdot V_{zi} \qquad (10)$$

The values for normal and reinforced panels are presented in Table 3. The dead weight of all panels is $G=1.288$ kN. The total shear force is thus $V_{zi}=F_{vi}+G$.

We can note that the shear forces on one staple, according to the Eq. (10), are smaller than those obtained by Eq. (9). This is clear, while in Eq. (9) the shear stiffness of the boards is not considered. It is also evident that the forces on the staples are smaller in the models with the fictive height of the boards. It is important that, as long as the force (N_{1i}) is smaller than N_{al}, we should use $K=K_{ser}$ for the slip modulus. In the case $N_{al}<N_{1i}<R_d$ the design modulus $K=2/3K_{ser}$ must be used. In other cases ($R_d<N_{1i}<R_k$) we recommend to take a very small slip modulus, smaller than $K=2/3K_{ser}$. It may also be more convenient and more accurate to take intermediate values of K ($0.9K_{ser}$, $0.8K_{ser}$, ..., $1/3K_{ser}$).

3.4 The loading procedure

All panels were first loaded by a vertical force $F=2.0$ kN. The experiments were continued at intervals of *2 kN / 5 min* up to force F_{cr}, when the first crack appeared. Then we continued with the same intervals up to force F_u, when the stresses on the manometer started to decline. This meant that the destruction of the panel was approaching.

4. TEST RESULTS

4.1 Cracks

The curve of the first crack in fibre-gypsum boards in all test samples propagated from the most tensioned fibre at the connection of the first bolt to the neutral axis of the composed cross section. At the same time we also noticed that the crack in the opposite diagonal corner was not formed at all, not even before destruction. This indicates that the panel shear walls at great loads behave like a thin-wall (*L/H>2*) and not like a truss.

Let us compare the measured cracks in the normal test samples (T_2 and T_3) with the reinforced test samples (T_4, T_5 and T_6). The test sample T_1 was eliminated because the rotation of the wall was too big (the slip modulus K_{ser} was too small).

4.1.1 The average force at the formation of the first crack (F_{cr})

Test samples without steel diagonals:

$$F_{cr} = \frac{14.35 + 14.83}{2} = 14.59 \text{ kN}$$

Test samples with steel diagonals:

$$F_{cr}^x = \frac{18.23 + 18.63 + 18.65}{3} = 18.50 \text{ kN}$$

$$\boxed{\frac{F_{cr}^x}{F_{cr}} = \frac{18.50}{14.59} = 1.27}$$

4.1.2 Comparison of the width of cracks at forces $F=F_{cr}$ and $F=20$ kN

Test samples without steel diagonals:

$$\delta_R = \frac{1.1 + 1.4}{2} = 1.25 \text{ mm}$$

$$\delta_{20} = \frac{8.6 + 9.0}{2} = 8.8 \text{ mm}$$

Test samples with steel diagonals:

$$\delta_R^x = \frac{0.3 + 0.3 + 0.1}{3} = 0.233 \text{ mm}$$

$$\delta_{20}^x = \frac{0.6 + 0.5 + 0.2}{3} = 0.430 \text{ mm}$$

4.2 Destruction force

The difference in the measured average destruction force (F_u) between the reinforced and the normal test samples was greater than the difference in cracks.

Test samples without steel diagonals:

$$F_u = \frac{21.02 + 19.34}{2} = 20.18 \text{ kN}$$

Test samples with steel diagonals:

$$F_u^x = \frac{34.40 + 36.60 + 36.20}{3} = 35.73 \text{ kN}$$

$$\boxed{\frac{F_u^x}{F_u} = \frac{35.73}{20.18} = 1.77}$$

This means that the resistance of the reinforced test samples was at the average 77% greater than the resistance of the normal panels.

4.3 Vertical displacements

Figure 4 and Table 4 present the measured vertical displacements on all test samples. A ratio between deflections of the normal and the reinforced panels is also added. It is evident that the stiffness of the reinforced test samples (T_4, T_5, T_6) is much greater than in those without steel diagonals (T_1, T_2, T_3). The difference is more evident at greater forces, especially after the first crack appears. More details about the measured results can be found in Ref. [10].

Table 4 Average values of the measured vertical displacements

F_v [kN]	v_{normal} [mm]	$v_{reinforced}$ [mm]	$v_{normal}/v_{reinforced}$
4	5.67	5.50	1.03
6	8.57	8.45	1.01
8	11.70	11.20	1.04
10	14.65	12.77	1.15
12	17.70	15.07	1.17
14	21.00	18.87	1.11
16	25.70 (c)	22.10	1.16
18	34.60	24.77	1.40
20	40.60	29.50 (c)	1.38
22		33.37	
24		37.27	
26		42.67	
28		48.37	
30		53.03	
32		61.50	
34		69.50	

It is evident from Table 4 that there is practically no difference in the measured deflection between normal and reinforced panels when the force is less than *8 kN*. After that a difference of vertical displacements constantly increases with the force. It is especially obvious after the formation of a first crack.

4.4 Comparison with the proposed mathematical models

From Table 2 we can find out that a ratio in a bending stiffness between reinforced and normal panels depends on the slip modulus and for $K=K_{ser}=3.37$ is:

- *1.287* by using the proposed model with a fictive height of a board,
- *1.083* by using the proposed model with a fictive thickness of a board.

But for $K=2/3K_{ser}$ a ratio is:

- *1.328* by using the proposed model with a fictive height of a board,
- *1.095* by using the proposed model with a fictive thickness of a board.

By comparing the measured results from Table 3 with the proposed mathematical models we recommend to use the mathematical model with the fictive thickness under a load before the first crack appears. After that it is recommended to use the model with the fictive height.

5. DISCUSSION AND CONCLUSIONS

It is evident from the measured and analytical results that the resistance of reinforced panel walls is greater than the resistance of panels without inserted steel diagonals. The measured results show a good coincidence with the analytical results obtained with the proposed mathematical models. The models with the fictive height or thickness of fibreboards are presented in the paper. We recommend a mathematical model with a fictive thickness under a load before the first crack appears. After that it is recommended to use the model with the fictive height.

From the relation between the forces forming the first crack it is evident that the inserted steel diagonals

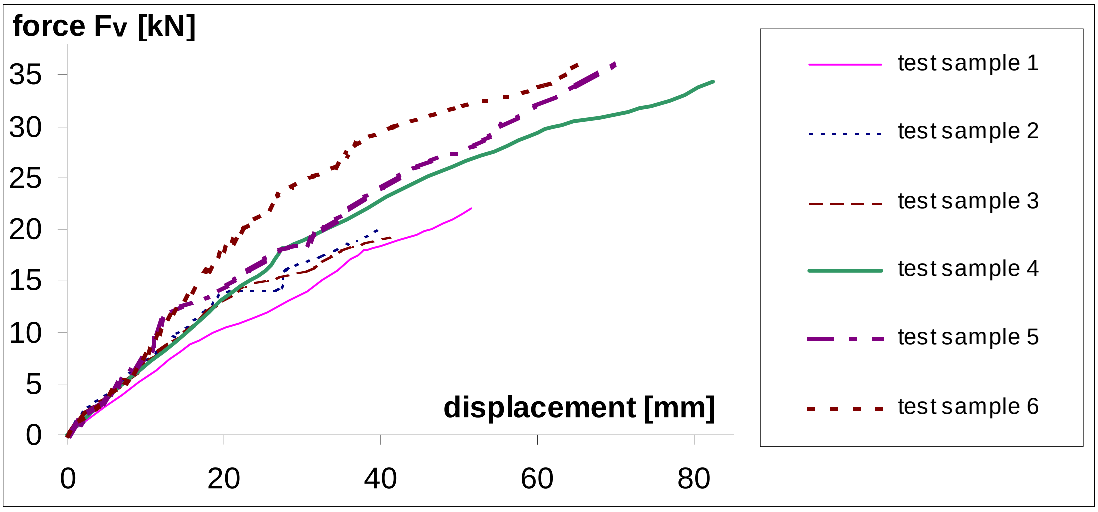

Fig. 4 Comparison of the measured vertical displacements

are not important. The maximum load can be only *27%* greater than in panels without inserted diagonals. The costs of panel reinforcing can be higher. But the proportion between destruction forces shows that the resistance of the reinforced panels is *77%* higher.

Consequently, we recommend the steel diagonals in the construction of multi-storey buildings located in the seismic or windy areas ($v_{ref} > 40\ m/s$). The cracks in the reinforced panels are scarcely perceivable and they also disappear after the action of the short-term load. In this case the use of steel diagonals is highly recommended.

In the described tests the stress calculation was performed for bending moments with a shear force. However, in reality the axial compression stresses also exist due to the dead load of the panels and vertical actions on the floors.

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MATEMATIČKO MODELIRANJE ARMIRANIH DRVENO-GIPSANIH PANELNIH POSMIČNIH ZIDOVA

SAŽETAK

U radu su predstavljeni matematički modeli za proračun armiranih panelnih posmičnih zidova, koji se upotrebljavaju kao nosive stijene kod montažnih drvenih konstrukcija. Paneli su sastavljeni od drvenog okvira i vlaknastih gips ploča, koje su ojačane čeličnim dijagonalama. U radu su predložena rješenja kod matematičkog modeliranja s fiktivnom širinom i visinom vlaknastih gips ploča. Dobiveni rezultati su uspoređeni s progibima i pukotinama, izmjerenima na testiranim uzorcima. Također su uspoređeni s rezultatima, izmjerenim na neojačanim uzorcima.

Ključne riječi: *drvo, panelni posmični zidovi, vlaknaste gips ploče, matematički modeli, čelične dijagonale.*