# Slip Modeling in Timber-Framed Walls with Wood-Based or Fibre-Plaster Sheathing Boards

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*Abstract:* - The paper provides mathematical modelling for prefabricated timber-framed walls composed of a timber frame and two different types of sheathing boards. Since by wood-based boards (WBB) the tensile strength is similar to the compressive one, there are practically no cracks appearing in the boards. On the other hand, in case of fibre-plaster sheathing boards (FPB) the tensile strength is approximately 10-times lower than the compressive one and therefore cracks in the tensile diagonal board's direction usually appear. Based on analysis of experimental research results [1] special approximate mathematical models have been developed. The models enable simultaneously to consider the flexibility of mechanical fasteners in the connecting areas, as well as possible cracks appearing in the tensile area of the sheathing boards.

*Key-Words:* - Timber structures, Walls, Slip, CFRP, Modelling

## 1 Introduction

Nowadays, there are the strongest arguments for building timber frame buildings. Brand new and improved features, being introduced in the early 80s in the last century, brought about the expansion of timber frame buildings all over the world. The most important are the next introduced changes:

- transition from on-site construction to prefabrication in factory,
- transition from elementary measures to modular building,
- bigger input of glued-laminated wood in construction,
- development from micro-panel wall system to macro-panel wall prefabricated panel system.

Another reason for this kind of building is speed of build because of the high degree of prefabrication of elements in a factory (Fig. 1); the buildings are built in essential shorter period of time. This means that in the building stage just minimum of time is spent in inconvenient weather conditions. And also, the probability of later claims is lowered. Another argument for building is that at the same outer dimensions we get up to 10% bigger residential area. As mentioned, at lower maintenance costs, high thermal efficiency and lower probability of constructional failure, is the decision of investor pro or contra much easier.

Fig. 1. Factory preparing of prefabricated walls.

There are two mostly used timber-frame systems around the world:

- post and beam system,
- timber-frame panel system.

We will focus our presentation to the second one - the timber-frame panel system. The basic vertical load bearing elements are panel walls that consist of load bearing timber frame and sheating boards, while horizontal floor load bearing elements are slabs constructed of roof beams and load bearing wood-based sheating boards on the upper side of the roof beams (Fig. 2). Because all elemets are prefabricated, the ercetion is very fast so the slogan „two men building one house“ would be suitable. Consequently, the system is very useful for multi-storey building; therefore the interest in the world is getting bigger.

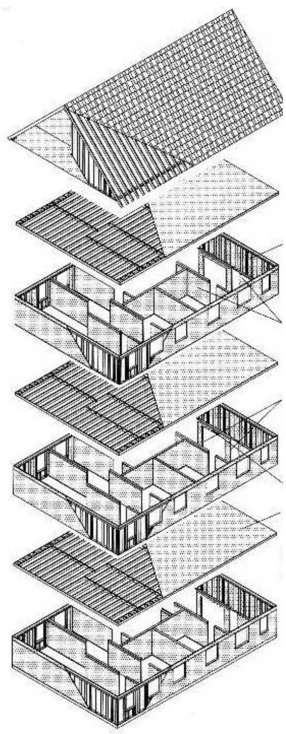


Fig. 2. Shematic presentation of the timber – frame panel construction.

As the influence of the horizontal load increases with the height of the structure, the wall's load-carrying capacity becomes critical when taller structures are subjected to heavy horizontal forces, particularly by structures located in seismic and windy areas.

In the presented research the treated walls consist of solid timber frame coated by sheets of board-material fixed by mechanical fasteners to one or both sides of the timber frame (Fig. 3). There are many types of panel sheet products available which may have some structural capacity such as wood-based materials (plywood, OSB, hardboard, particleboard, etc.) or plaster and fibre-plaster boards (FPB), made from gypsum, 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 namely their relatively good fire protection. For example, single gypsum sheathed board of 15mm thickness assures 45 minutes of fire protection. Additionally, gypsum is a healthy natural material and is consequently particularly desired for residential buildings. On the other hand, from a structural point of view the tensile strength of FPB is very low, approximately 10-times lower than the compressive one, and can not be compared with the overall strength of the timber frame at all.

In this research we will limit our attention to modelling of the walls with wood-based or FPB sheathing boards. A special attention will be focused to the comparision of the numerical results, including forming of cracks, destruction force and slip in the timber frame-board connecting area.

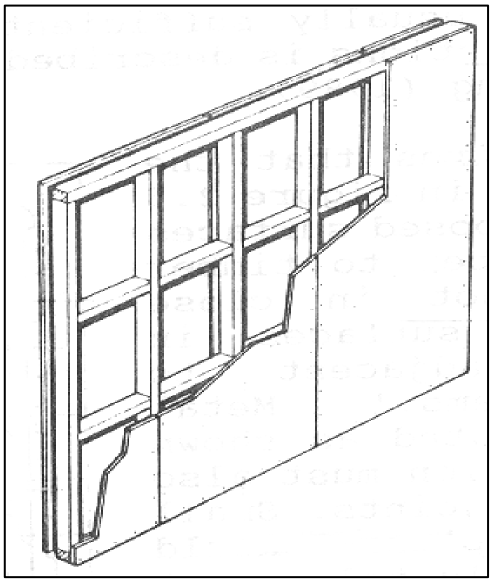

Fig. 3. Composition of the wall.

In structural analysis panel walls for design purposes can be regarded separately as vertical cantilever beams with the horizontal force ($F_H$=$F_{H,tot}$ /$n$) acting at the top (Fig.x4). 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 [2] – [4].

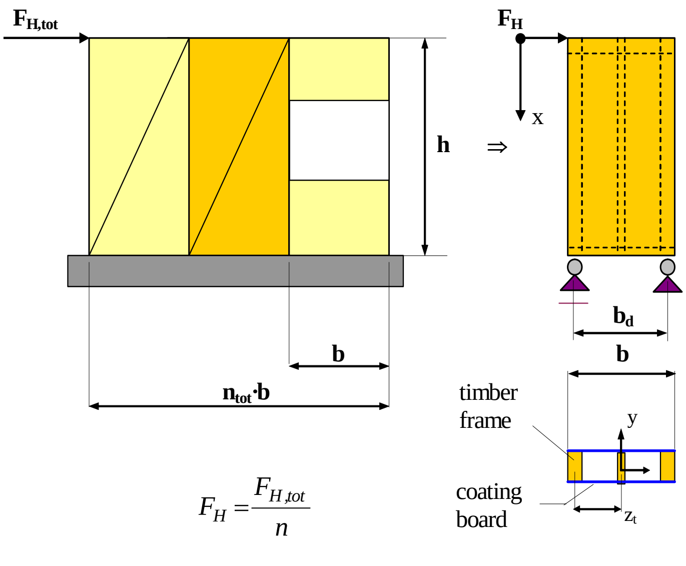


Fig. 4. Static design for the wall system in one level.

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 [5] and Äkerlund [6] 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 [7] 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.

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 [2] and [4]:

$$w = w_t + w_b + w_c + w_a = \frac{8 \cdot \nu \cdot h^3}{E_t \cdot A_t \cdot b} + \frac{\nu \cdot h}{G_b \cdot t} + 0.376 \cdot h \cdot e_n + d_a \quad (1)$$

$\upsilon$ ....... maximum shear due to design loads at the top of the wall,
$E_t$ ….. elastic modulus of timber elements,
$A_t$ ….. area of boundary vertical timber element cross section,
$G_b$ …. modulus of rigidity of coating boards,
$t$ …… effective thickness of coating boards,
$e_n$ ….. nail deformation,
$d_a$ ...... deflection due to anchorage details.

Analytical models were also developed to predict the dynamic response of the timber shear walls, [8] – [9]. Finally, Kasal et al. [10] developed a three-dimensional finite element model to investigate the responses of complete light-frame wood structures.

# 2 Modelling of walls with FPB

We will limit our analysis to the horizontal force ($F_H$) influence acting at the top of the wall (Fig. 4). By employing FPB as a coating material a horizontal load shifts a part of the force over the mechanical fasteners to the FPB and the wall acts like a deep composite beam with a semi rigid connecting area between the timber frame and FPB [2].

For design purposes a simplified design method for mechanically jointed beams according to Annex B of Eurocode 5 [3] is widely used. Expression of the so called »γ-method« is based on the differential equation for the partial composite action with the following fundamental assumptions [11, 12]:
a) Bernoulli’s hypothesis is valid for each sub-component,
b) slip stiffness is constant along the element,
c) material behaviour of all sub-components is linear elastic.

The effective bending stiffness $(EI_y)_{eff}$ of mechanically jointed beams which empirically considers the flexibility of fasteners via coefficient $\gamma_y$, taken from Eurocode 5 [3], can be written in the form of:

$$(EI_y)_{eff} = \sum_{i=1}^{n} E_i \cdot \left(I_{yi} + \gamma_{yi} \cdot A_i \cdot a_i^2\right) = \sum_{i=1}^{n_{timber.}} \left(E_i \cdot I_{yi} + E_i \cdot \gamma_{yi} \cdot A_i \cdot a_i^2\right)_{timb.} + \sum_{j=1}^{n_{board}} \left(E_i \cdot I_{yi}\right)_{FPB} \quad (2)$$

where $n$ is the total number of elements in the considered cross-section and $a_i$ is a distance between global y-axis of the whole cross-section and local $y_i$-axis of the i-th element with a cross-section $A_i$.

The second moment of area for timber about the local $y_i$-axis $(E_i I_{yi})_{timber}$ is in comparison with other values very small and may be neglected. In this case the above equation results in an approximation:

$$(EI_y)_{eff} \approx \sum_{i=1}^{2} \left(E_i \cdot \gamma_{yi,\mathrm{mod}} \cdot A_i \cdot a_i^2\right)_{timb} + \sum_{j=1}^{2} \left(E_i \cdot I_{yi}\right)_{FPB} \approx \gamma_{yi,\mathrm{mod}} \cdot (EI_y)_{timber} + (EI_y)_{FPB} \quad (3)$$

## 2.1 Modelling of fasteners flexibility

We can mention from Eqs. (2) and (3) that the bending stiffness strongly depends on the stiffness coefficient of the fasteners ($\gamma_{yi}$). It can be defined via the fastener spacing ($s$) and the slip modulus per shear plane per fastener ($K$) using Eurocode 5 [3] in the form of:

$$\gamma_y = \frac{1}{1 + k_y}; \qquad k_y = \frac{\pi^2 \cdot A_{t1} \cdot E_t \cdot s}{L_{eff}^{\,2} \cdot K} \quad (4)$$

In the proposed mathematical model the value of the modulus ($K$) varies according to the lateral force ($F_1$) acting on one fastener, which can be determinated according to the shear force ($V_z$), the spacing between fasteners ($s$), the effective shear stiffness $(ES_y)_{ef}$ in the connecting plane and on the effective bending stiffness $(EI_y)_{eff}$ of the cross-section in the following form:

$$F_1 = \frac{(ES_y)_{eff}}{(EI_y)_{eff}} \cdot \frac{s}{2} \cdot V_z \qquad (5)$$

The value of $K$ directly depends on the slip ($\Delta$) in the timber frame - FPB connecting area. As long as behaviour of fasteners is almost elastic (Fig. 5a) the value of $K$ is maximal ($K=K_{ser}$) and it is constant (Fig. 5b). The value of $K_{ser}$ depends on national codes. With an increasing part of plasticity the value of $K$ decreases. We propose the *three-linear interpolation diagram* to simulate the behaviour of fasteners depending on the value of $F_1$ (Fig. 5b). It is important to determine three fundamental diagram points:

$$F_1 \le N_{al} \quad \Rightarrow \quad K = K_{ser} \qquad (6a)$$

$$F_1 = F_{f,Rd} \quad \Rightarrow \quad K = K_u = \frac{2}{3} \cdot K_{ser} \qquad (6b)$$

$$F_1 = F_{f,Rk} \quad \Rightarrow \quad K = 0 \qquad (6c)$$

$N_{al}$ …. allowable lateral load-bearing capacity per shear plane per fastener,
$F_{f,Rd}$ … design lateral load-bearing capacity per shear plane per fastener,
$F_{f,Rk}$ … characteristic lateral load-bearing capacity per shear plane per fastener.

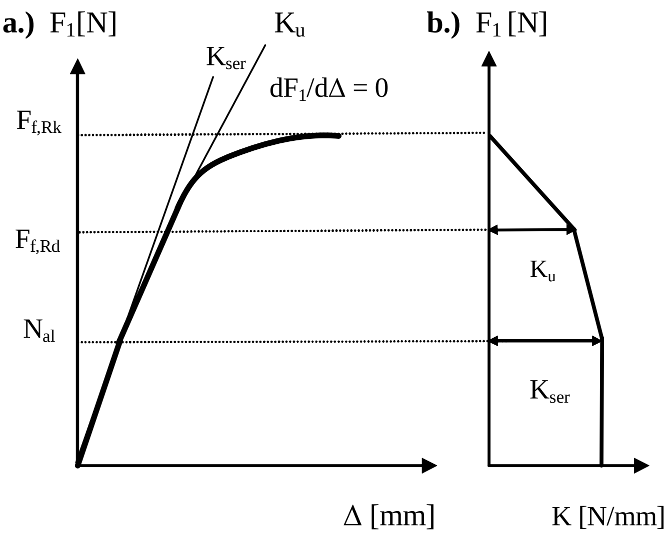


Fig. 5. a.) $F_1$–$\Delta$ diagram, b.) Three–linear diagram for K.

For intermediate values of $F_1$ linear interpolation is used according to Fig. 5b. The slip ($\Delta$) in the timber frame - FPB connecting area under the force $F_H$ is calculated in the form of:

$$\Delta = \frac{F_1}{K} \qquad (7)$$

## 2.2 Modelling of cracks in FPB

The horizontal force forming the first tensile crack ($F_{H,cr}$) in FPB is defined according to the normal stress criteria and the tensile strength of FPB ($f_{bt}$):

$$\sigma^{`}_{b,max} = \frac{M_y \cdot E_b}{\left(EI_y\right)_{eff}} \cdot \frac{b}{2} \;\Rightarrow\; M_{y,cr} = \frac{2 \cdot f_{bt} \cdot (EI_y)_{eff}}{E_b \cdot b} \qquad (8a)$$

$$F_{H,cr} = \frac{M_{y,cr}}{h_d} = \frac{2 \cdot f_{bt} \cdot (EI_y)_{eff}}{E_b \cdot b \cdot h_d} \qquad (8b)$$

The tensile strength ($f_{bt}$) of fibre-plaster sheeting material is very low. Consequently, cracks in tensile area of FPB usually appear.

Four major assumptions are considered in the presented modelling of the cracked cross-section [12]:

a.) The tensile area of the fibreboards is neglected after the first crack formation.

b.) The stiffness coefficient of the fasteners in the tensile connecting area ($\gamma_{yt}$) is assumed to be constant and equal to the value by appearing the first crack.

c.) The stiffness coefficient of the fasteners in the compressed connecting area ($\gamma_{yc}$) is not constant and depends on the lateral force acting on one fastener, as it is declared in Eqs. (6).

d.) The normal stress distribution is assumed to be linear. This simplification can be used only by assumption that behaviour of timber frame in tension is almost elastic until failure and that the compressive normal stress in timber and in FPB is under the belonging yield point.

Position ($x_{II}$) of a new neutral axis ($y_{II}$) is computed according to the presented computational scheme (Fig. 6) by respecting the equilibrium criteria in x-direction:

$$F_{cb} + F_{ct} - F_{tt} = 0 \qquad (9)$$

$$\begin{aligned} F_{tt} &= \sigma^c_{tt} \cdot A_{t1} \\ F_{ct} &= \sigma^c_{ct} \cdot A_{t1} \\ F_{cb} &= \frac{\sigma^c_{cb} \cdot x_{II} \cdot 2t}{2} = \sigma^c_{cb} \cdot x_{II} \cdot t \end{aligned} \qquad (10)$$

$$x_{II} = \frac{-C_2 + \sqrt{C_2^2 - 4C_1 \cdot C_3}}{2C_1};$$

$$C_1 = t,$$

$$C_2 = A_{t1} \cdot n \cdot (\gamma_{yc} + \gamma_{yt}), \qquad n = \frac{E_t}{E_b} \tag{11}$$

$$C_3 = -n \cdot A_{t1} \cdot \left[\gamma_{yt} \cdot \left(b - \frac{a}{2}\right) + \gamma_{yc} \cdot \frac{a}{2}\right]$$

Because of crack appearing the effective bending stiffness $(EI_y)_{eff}$ is now decreased according to Eq. (2):

$$(EI_y)^{II}{}_{eff} = E_b\, I_b + E_t\, I_t =$$

$$E_b \cdot 2t \cdot \frac{x_{II}{}^3}{3} +$$

$$+ E_t \cdot \left\{ \frac{2a^3 \cdot c}{12} + \frac{d^3 \cdot c}{12} + A_{t1} \cdot \left(\gamma_{yc} \cdot z_{cII}{}^2 + \gamma_{yt} \cdot z_{tII}{}^2\right) \right\}$$

$$z_{cII} = x_{II} - \frac{a}{2}; \quad z_{tII} = b - \frac{a}{2} - x_{II} \tag{12}$$

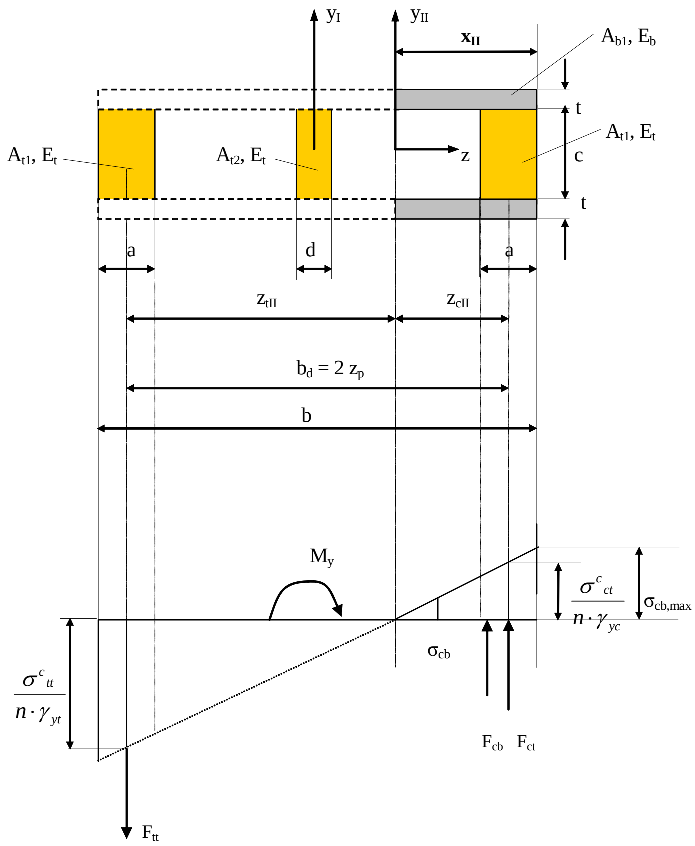


Fig. 6: Mathematical model for the cross-section with a tensile crack in FPB.

If we declare as a characteristic destruction condition the case when the tensile normal stress in timber ($\sigma_{tt,max}$) achieves the characteristic tensile timber strength ($f_{t,0,k}$), the characteristic horizontal destruction force ($F_{H,k}$) is computed in the following form:

$$F_{H,k} = \frac{f_{t,0,k} \cdot (EI_y)^{II}{}_{eff}}{E_t \cdot \left(\gamma_{yt} \cdot z_{tII} + \frac{a}{2}\right) \cdot h} \tag{13}$$

In engineering design it is important to reinforce the FPB if there is any possibility of cracks appearing. Experimental study of CFRP strip reinforcing can be found in [13]. Semi-analytical mathematical models with the fictive enlarged thickness of the boards are proposed in [12] or [14] and will not be presented or discussed in this paper.

## 3 Modelling of walls with wood-based sheathing boards

Of course we can model the walls with wood-based sheathing boards with the described procedure of the composite model described in Section 2, respecting an important fact that the tensile strength of the wood-based boards is similar to the compressive one. Consequently, it can be aspect that there is practically no crack appearing in the tensile area of the boards. Therefore, it can be predicted that the stresses in the fasteners reach their yielding point before any cracks in the boards are formed. Another important restriction is material behaviour of the wood-based sheathing boards. Assumptions, given in Section 2 for FPB, that material behaviour of all sub-components is linear elastic and that the tensile area of the sheathing board is neglected after the first crack formation, for wood-based sheathing boards it doesn't state. After the first crack formation the stress-strain situation is quite different (especially around the crack) and this change the tensile area and couldn't be neglected. Consequently, two simplified computational methods, based on general assumption that fasteners yielding point is reached before any crack appearing in the sheathing boards, are given in Eurocode 5 [3] 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 [7]. This method defines the wall's characteristic shear resistance ($F_{v,k}$) as a sum of the fasteners' lateral capacity ($F_{f,Rk}$) in the shear loaded edges in the form of:

$$F_{v,k} = \sum F_{f,Rk} \cdot \frac{b_i}{s} \cdot c_i \tag{14}$$

where $b_i$ is the wall panel width and $s$ is a fastener spacing. The parameter $c_i$ is empirical described in the form of:

$$c_i = \begin{cases} 1 & for\ b_i \geq b_0 \\ \dfrac{b_i}{b_0} & for\ b_i \leq b_0 \end{cases} \quad where\ b_0 = h/2 \tag{15}$$

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:

$$F_{v,k} = \sum F_{f,Rk} \cdot \frac{b_i}{s_0} \cdot c_i \cdot k_d \cdot k_{i,q} \cdot k_s \cdot k_n \tag{16}$$

where coefficient $s_0$ depends on the fastener diameter ($d$) and wood characteristic density ($\rho_k$) in the form of:

$$s_0 = \frac{9700 \cdot d}{\rho_k} \tag{17}$$

$$k_d = \begin{cases} \dfrac{b_i}{h} & for\ \dfrac{b_i}{h} \leq 1.0 \\ \left(\dfrac{b_i}{h}\right)^{0.4} & for\ \dfrac{b_i}{h} > 1.0\ and\ b_i \leq 4.8\,m \\ \left(\dfrac{4.8}{h}\right)^{0.4} & for\ \dfrac{b_i}{h} > 1.0\ and\ b_i > 4.8\,m \end{cases} \tag{18}$$

$$k_{i,q} = 1 + \left(0.083 - 0.0008 \cdot q_i^2\right) \cdot \left(\frac{2.4}{b_i}\right)^{0.4} \tag{19}$$

where $q_i$ is the equivalent uniformly distributed vertical load acting on the wall ($kN/m$) and should be determined using only permanent actions and any net effects of wind together with the equivalent actions arising from concentrated forces, including anchorage forces, acting on the panel. For the purposes of calculating concentrated vertical forces should be converted into an equivalent uniformly distributed load on the assumption that the wall is a rigid body e.g. for the load $F_{i,vertEd}$ acting on the wall as shown in Fig.7.

$$q_i = \frac{2a \cdot F_{i,vert,Ed}}{b_i^2} \tag{20}$$

where $a$ is the horizontal distance from the force $F_H$ to the leeward corner of the wall and $b_i$ is the length of the wall (see Fig. 7).

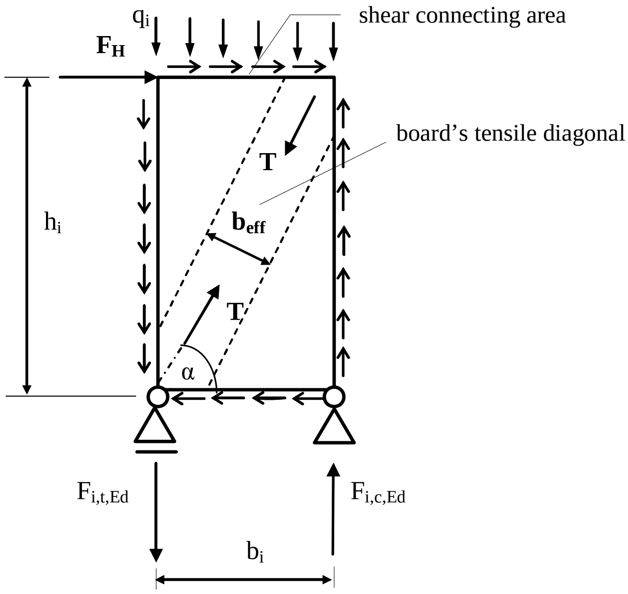


Fig. 7: Considered force distribution.

The fastener spacing factor ($k_s$) is computed in a form of:

$$k_s = \frac{1}{0.86 \cdot \dfrac{s}{s_0} + 0.57} \tag{21}$$

where $s$ is the spacing of the fasteners around the perimeter of the sheets. The sheathing material factor ($k_n$) is in a form of:

$$k_n = \begin{cases} 1.0 & for\ sheath.\ on\ one\ side \\ \dfrac{F_{i,v,Rd,max} + 0.5F_{i,v,Rd,min}}{F_{i,v,Rd,max}} & for\ sheath.\ on\ both\ sides \end{cases} \tag{22}$$

where $F_{i,v,Rd,max}$ is the design racking strength of the stronger sheathing and $F_{i,v,Rd,min}$ is the design racking strength of the weaker sheathing.

It should be underlined that the both simplified mehods can be applicable only for wood-based panels where the tensile strength is relatively high and the elements tend to fail because of fastener yielding and not because of crack appearing in the sheathing boards.

# 4 Numerical example

## 4.1 Geometrical and material properties

Numerical analysis is performed for the panel wall of actual dimensions *h=263.5 cm* and *b=125 cm*, composed of timber studs (*2x9x9cm* and *1x4.4x9cm*) and timber girders (*2x8x9cm*). The boards of the thickness *t=15 mm* are fixed to the timber frame using staples of *Φ1.53 mm* and length *l = 35 mm* at an average spacing of *s = 75 mm* (Fig. 8). Examples with two different material types (FPB and plywood) of the sheathing boards will be separately analysed and compared.

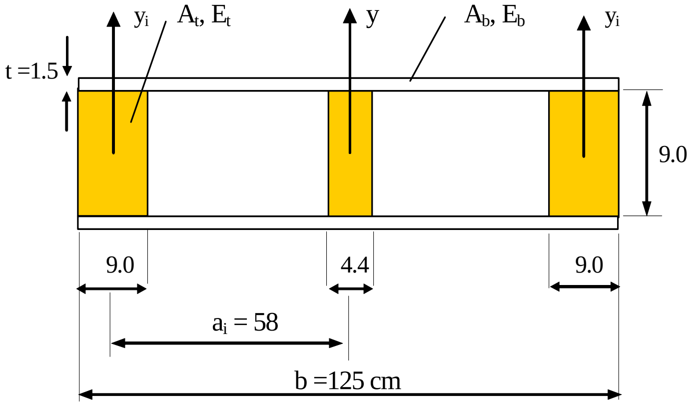


Fig. 8. Cross-section of the test sample.

Material properties for the timber of quality C22 are taken from EN 338 [15], for the FPB Knauff plasterboards from [16] and the plywood boards from [17]. All material properties are listed in Table 1.

Table 1. Properties of used materials.

| | Timber C22 | FPB Knauf | Swedian (S) Plywood* |
|---|---|---|---|
| $E_{0,m}$ [N/mm$^2$] | 10000 | 3000 | 9200 |
| $G_m$ [N/mm$^2$] | 630 | 1200 | / |
| $f_{m,k}$ [N/mm$^2$] | 22.0 | 4.0 | 23.0 |
| $f_{t,0,k}$ [N/mm$^2$] | 13.0 | 2.5 | 15.0 |
| $f_{c,0,k}$ [N/mm$^2$] | 20.0 | 20.0 | 15.0 |
| $\rho_k$ [kg/m$^3$] | 340 | 1050 | 410 |
| $\rho_m$ [kg/m$^3$] | 410 | 1050 | 410 |

* *The values are given for 12 mm typical thickness of the board.*

## 4.2 Calculation and numerical results

a) Since the computational model according to Fig. 3 is considered, $h_d = 254.5\ cm$ and $b_d = 116\ cm$.

b) Lateral load-bearing capacity of the staples:
According to Eurocode 5 [3] using Johansen expressions we obtain for the characteristic lateral load-carrying capacity for the staples ($F_{f,Rk}$) and for the belonging designed value ($F_{f,Rd}$) respecting $k_{mod} = 0.9$:

FPB:
$F_{f,Rk} = 659.69\ N$
$F_{f,Rd} = 456.71\ N$

WBB (plywood):
$F_{f,Rk} = 516.74\ N$
$F_{f,Rd} = 357.74\ N$

Allowable lateral load-bearing capacity per shear plane per fastener ($N_{al}$) is not declared in Eurocode 5 [3] thus it can be obtained for the both types of the boards using Brüninghoff [18]: $N_{al} = 203.03\ N$.

c) Slip modulus ($K_{ser}$) for the staples is computed using Eurocode 5 [3]:

FPB:

$$\rho_m = \sqrt{\rho_b \cdot \rho_t} = \sqrt{1050 \cdot 410} = 656.12\ kg/m^3$$

$$K_{ser}^{(FPB)} = \frac{\rho_m^{1.5} \cdot d^{0.8}}{80} = \frac{656.12^{1.5} \cdot 1.53^{0.8}}{80} = 295.215\ N/mm \qquad (23)$$

WBB:

$$\rho_m = \sqrt{\rho_b \cdot \rho_t} = \sqrt{410 \cdot 410} = 410\ kg/m^3$$

$$K_{ser}^{(WBB)} = \frac{\rho_m^{1.5} \cdot d^{0.8}}{80} = \frac{410^{1.5} \cdot 1.53^{0.8}}{80} = 145.827\ N/mm \qquad (24)$$

d.) The stiffness coefficient $\gamma_{yi}$ before any cracks appearing in the boards is obtained using Eq. (4), the effective bending stiffness $(EI_y)_{eff}$ of the un-cracked cross-section is calculated using Eq.(2), the horizontal force forming the first tensile crack in board ($F_{H,cr}$) is calculated using Eq.(8) and the characteristic horizontal load-carrying capacity ($F_{H,k}$) for FPB is calculated using Eq.(13), $x^{(FPB)}_{II} = 42.636\ cm$. The calculated results for FPB and WBB are listed in Table 2. Since the proposed procedure is not recommended for WBB after the first crack formation (see Section 3), the result of $F_{H,k}$ for WBB is not presented.

e.) The wall's characteristic shear resistance ($F_{v,k}$) as a sum of the fasteners' lateral capacity ($F_{f,Rk}$) in the shear loaded edges using Eurocode 5 Method A is calculated using Eqs. (14) and (15). The results for Method B are calculated using Eqs. (16) - (22). All calculated values are presented in Table 2. Since Eurocode 5 [3] does not recommend Method B for FPB the results are not presented.

Table 2. Numerical results for $\gamma_{yi}$, $(EI_y)_{eff}$, $F_{H,cr}$, $F_{H,k}$ and $F_{v,k}$.

| | $\gamma_{yi}$ | $(EI_y)_{eff}$ [kNcm$^2$] $\cdot 10^8$ | $F_{H,cr}$ [kN] | $F_{H,k}$ [kN] | $F_{v,k}$ [kN] Met. A | $F_{v,k}$ [kN] Met. B |
|---|---|---|---|---|---|---|
| FPB | 0.203 | 2.584 | 13.53 | 39.58 | 21.99 | / |
| WBB | 0.112 | 5.114 | 52.42 | / | 17.22 | 16.07 |

It is evident that $F_{H,cr}^{(WBB)}$ is much higher than $F_{H,cr}^{(FPB)}$. Therefore, it can be concluded, that there is practically no possibility of any cracks forming by using plywood as a sheathing board. As described in Section 3, $F_{v,k}$ means the force by which a full yielding of fasteners would appear. Of course, in case of FPB, where the tensile strength is very low, $F_{v,k}$ is much more higher than $F_{H,cr}$.

f.) Calculated values for lateral force acting on one fastener ($F_1$) are obtained by Eq. (5) and presented in Table 3.

g.) The results for slip ($\Delta$) in the timber frame–sheathing board connecting area are calculated using Eq. (7) and presented in Table 3.

Table 3. Numerical results for lateral force acting on one fastener ($F_1$) and for slip ($\Delta$) in the connecting area.

| $F_H$ [kN] | $F_1^{(FPB)}$ [N] | $F_1^{(WBB)}$ [N] | $\Delta_{FPB}$ [mm] | $\Delta_{WBB}$ [mm] |
|---|---|---|---|---|
| 5.0 | 69.289 | 19.279 | 0.235 | 0.132 |
| 10.0 | 138.579 | 38.558 | 0.469 | 0.264 |
| 13.53 = $= F^{(FPB)}_{H,cr}$ | 187.497 $< N_{al}$ | 52.170 | 0.635 | 0.358 |
| 15.0 | 198.189 | 57.838 | 0.671 | 0.397 |
| 20.0 | 258.064 | 77.117 | 0.922 | 0.529 |
| 25.0 | 306.057 | 96.396 | 1.224 | 0.661 |
| 30.0 | 352.426 | 115.674 | 1.532 | 0.792 |
| 35.0 | 394.036 | 134.953 | 1.859 | 0.924 |
| 39.58 = $= F^{(FPB)}_{H,k}$ | 437.011 $< F_{f,Rd}$ | 152.613 | 2.138 | 1.045 |
| $F^{(WBB)}_{H,cr}$ | / | 202.12 $\approx N_{al}$ | / | 1.384 |

It is interesting to compare calculated results for maximal horizontal deflection ($w$) at the top of the wall using Eq. (1) with results obtained by the proposed semi-analytical model presented in Section 2. Using Eq. (1) we get for $F_H = 10\ kN$ considering $w_t$ and $w_b$ ($\nu = 0.04$) for FPB:

$$w = \frac{0.04 \cdot 263.5^3}{1000 \cdot 201.6 \cdot 125} + \frac{0.04 \cdot 263.5}{120 \cdot 1.5} = 2.91\,mm \tag{25}$$

Maximal cantilever horizontal deflection using the proposed semi-analytical model is calculated considering the bending and the shear part in the form of:

$$w = \int_{x=0}^{x=h_d} \frac{F_H \cdot x^2}{(EI_y)_{eff}} dx + \int_{x=0}^{x=h_d} \frac{F_H}{G_b \cdot A_{s,eff}} dx = 2.77\,mm \tag{26}$$

It is obvious that the agreement between the both methods is relative good. Of course, the calculated deflection assuming composite behaviour of the wall element is a little smaller as by using Eq. (1) where only a part of a composite bahaviour is considered.

## 4 Conclusion

It is obviously from the presented results that in case of FPB by first crack forming the force acting on one fastener ($F_1$) is strongly under its characteristic lateral load-carrying capacity ($F_{f,Rk}$). Therefore, our prediction that cracks in FPB appear before the stresses in the fasteners reach their yielding point, was completely correct. Force, forming the first tensile crack in the board, is namely strongly under the characteristic load-carrying capacity obtained with the shear model. Consequently, it is not recommended to use Eurocode 5 Methods A or B to define the lateral load-carrying capacity of the wall.

On the other hand, by using plywood as a sheathing material, force forming the first tensile crack in the board is evidently higher than by FPB. The reason is in higher tensile strength of the board, which is in range of the strength of the timber frame. Consequently, the stresses in the fasteners reach their yielding point before any cracks appearing in the sheathing boards and therefore, the «Lower bound plastic method« (Eq.14 or Eq.16) can be used to determine the wall's load carrying capacity.

Comparing the obtained results for WBB using Method A in Method B it is obvious that there is only a small difference in the calculated characteristic lateral load-carrying capacity ($F_{v,k}$) between the both methods.

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