DESIGN OF WALL DIAPHRAGMS ACCORDING TO EC 5

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Summary: *This paper presents basic concept for analyzing shear wall diaphragms in timber frame structures – platform frame construction. It also provides important instructions and notes on the factors that may have influence on the racking resistance of vertical panels. Providing lateral and torsional stability of high – storey buildings made of timber, in general, is achieved by bracings and / or formation of the roofs, floors and wall diaphragms. The basic requirements that must be met according to ultimate limit state are: static equilibrium and racking resistance of wall panels. For the racking design of timber diaphragms Eurocode 5 provides two simplified methods (Method A and Method B). The goal of the analysis is to show different aspects of modern calculation methods and their effectiveness compared to a traditional approach to the problem.*

Keywords: *Shear wall, Eurocode 5, racking resistance.*

1. INTRODUCTION

Vertical and horizontal loads acting on the timber structure cause certain effects that must be adequately analyzed. In light-frame buldings, such as platform type of structure, ensuring the stability of all timber members and elements represents very important task for structural engineers. Ensuring the stability of frame system of timber buildings can be achieved in several different ways. As one of the generally accepted ways is desinging of wall, floors and roof panels (diaphragms). Each of this structural elements needs to fulfil demands ultimate limit state (ULS) and serviceability limit state (SLS) and all other requirements which EC5 [1] requires. Depending on the stiffness of floor diaphragms and walls panels there are several different distribution of shear forces and

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failure mechanisms. Horizontal loads, such as wind and earthquakes, can cause shear, overturning (tilting) and sliding of the vertical panels (Fig. 1). These are the basic forms of loss of local stability of timber shear walls. The combination of these mechanisms of fracture and instability are also possible. Shear walls usually consist of lumber framing, connections, and panel sheathing attached with nails or screws (or any other dowel-type of fasteners). The panels must be made from sheets of wood based panel products compliant with EN 13986 [3] and, in the case of LVL panels, with EN 14279 [4]. Thickness of sheathing directy depends of the spacing betwen vertical studs and the desired racking resistance of wall.

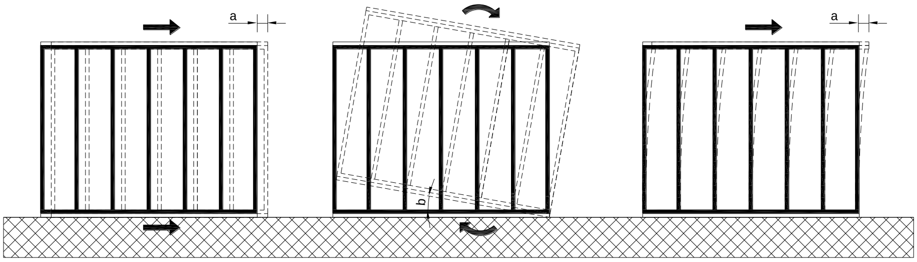

Figure 1. Sliding, overturning (tilting), and racking of timber shear-walls

According to EC5[1] the racking resistance of wall shall be determinated either by test (EN594) or by calculations, employing appropriate analytical methods or design models. European regulations [1] prescribes two alternative simplified methods of calculation and design of wall diaphragms.

2. SIMPLIFIED ANALYSIS OF WALL DIAPHRAGMS – METHOD A

These method should only be applied to wall panels with a tie-down on their end, that is the vertical member at the end directly connected to the construction below. The design racking load-carrying capasity of timber wall made up of several wall panels should be calculated according to following equations:

$$F_{v,Rd} = \sum F_{i,v,Rd} \tag{1}$$

$$F_{i,v,Rd} = \frac{F_{f,Rd} \cdot b_i \cdot c_i}{s} \tag{2}$$

where, $F_{i,v,Rd}$ the design racking load-carrying capacity of each panel against a horizontal force $F_{i,v,Ed}$ according to fig.2.

$F_{f,Rd}$ - lateral design capacity of an individual fastener; b_i - wall panel width.

s - fastener spacing ; $c_i = 1$ for $b_i \geq b_0$ and $c_i = \frac{b_i}{b_0}$ for $b_i < b_0$; $b_0 = \frac{h}{2}$ (is half the height of the timber wall panels)

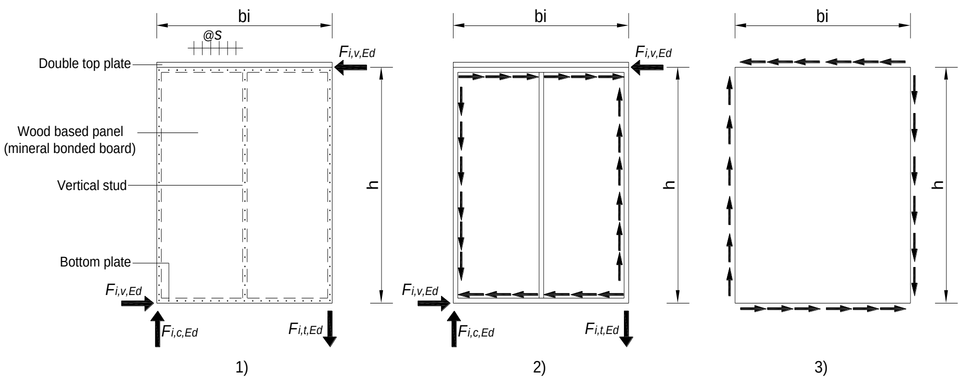

Figure 2. Loads and forces acting on wall panel (1), framing (2), and sheet (3) [1]

$$F_{i,c,Ed} = \frac{F_{i,v,Ed} \cdot h}{b_i} \quad ; \quad F_{i,t,Ed} = \frac{F_{i,v,Ed} \cdot h}{b_i} \tag{3}$$

The external forces (reactions) are determinated based on Figure 2. In case that wall diaphragms consist a panels which contain a door or window opening, these panels should not be considered to contribute to the racking load-carrying capacity. Special attention should be paid to the shear buckling of the sheet (OSB, Plywood…), but if the following condition are met, shear buckling of the sheet may be disregarded:

$$\frac{b_{net}}{t} \leq 100 \tag{4}$$

where: b_{net} - clear distance between studs; t - thickness of the sheet.

Also, buckling and capacity of vertical wall studs should be checked as columns subjected either to compression or combined compression and bending. In places where vertical studs are joined with bottom and top rail (horizontal members), the compression perpendicular to the grain stresses should be verify. The procedure, which EC5 recommended is method A, but final choice may be given in the National annex.

3. SIMPLIFIED ANALYSIS OF WALL DIAPHRAGMS – METHOD B

The concept and formulas given in method B are based on a semiempirical theory assuming that the wall is a rigid body, which is not completely correct [2]. This simplified procedure define a wall assembly which is comprised of one or more walls with each wall formed from one or more panels. Basic form of external wall assembly is presented in Figure 3 with marked elements. The design procedure are similar to previous method with some diferences.

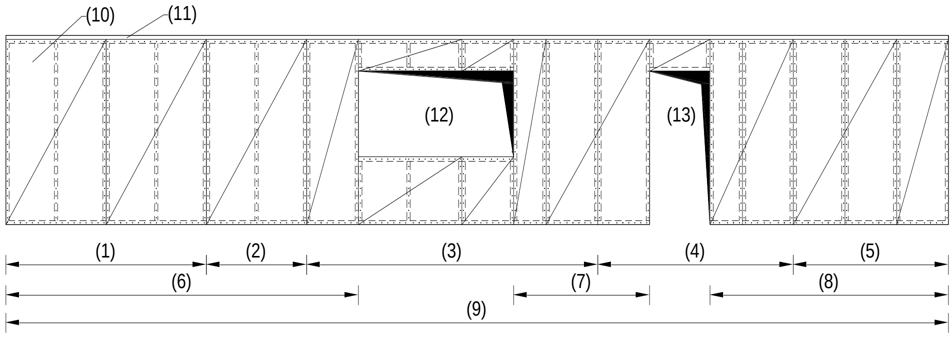

Figure 3. Timber wall assembly [1]
Key: 1÷5 –Wall panel; 6÷8 –Wall; 9 –Wall assembly;
10– Sheet; 11–Head binder; 12–Window; 13–Door

For wall assembly the design racking strength should be calculated as a sum of racking strength of all components (panels) (equation 1.) The racking strength of a wall *"i"* is defined by folowing equation:

$$F_{i,v,Rd} = \frac{F_{f,Rd} \cdot b_i}{s_0} k_d \cdot k_{i,q} \cdot k_s \cdot k_n \tag{5}$$

where: b_i is the wall panel length (in m'); $s_0 = \frac{9,7d}{\rho_k}$ is the basic fastener spacing (in m'); d is the fastener diameter (in mm); ρ_k is the characteristic density of the timber frame (in kg/m³); k_d is the dimension factor for the wall defined by:

$$k_d = \begin{cases} \dfrac{b_i}{h} & \text{for } \dfrac{b_i}{h} \le 1,0 \\ \left(\dfrac{b_i}{h}\right)^{0,4} & \text{for } \dfrac{b_i}{h} > 1,0 \quad \text{and } b_i \le 4,8m \\ \left(\dfrac{4,8}{h}\right)^{0,4} & \text{for } \dfrac{b_i}{h} > 1,0 \quad \text{and } b_i > 4,8m \end{cases}$$

$k_{i,q} = 1 + \left(0,083q_i - 0,0008q_i^2\right) \cdot \left(\dfrac{2,4}{b_i}\right)^{0,4}$ is the uniformly distributed load factor for wall *"i"* and q_i is the equivalent uniformly distributed vertical load acting on the wall (in kN/m), with $q_i \geq 0$.

$$q_i = \frac{2 \cdot a \cdot F_{i,vert.Ed}}{b_i^2} \quad \text{According to Fig.4}$$

$$k_s = \frac{1}{0{,}86 \dfrac{s}{s_0} + 0{,}57} \quad \text{is the fastener spacing factor}$$

$$k_n = \begin{cases} 1{,}0 & \text{for sheating on one side} \\ \dfrac{F_{i,v,Rd,\max} + 0{,}5 F_{i,v,Rd,\min}}{F_{i,v,Rd,\max}} & \text{for sheating on both sides} \end{cases}$$

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

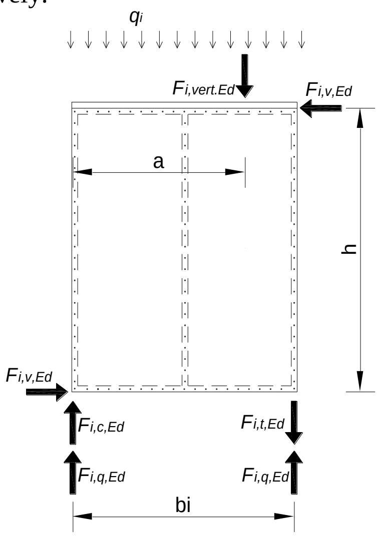

Figure 4. Determination of equivalent vertical acrtion qi and reaction forces from vertical and horizontal actions [1]

The external forces (reactions), as in the previous procedure (method A) should be determinated based on Figure 4 and equation (3). When tensile forces are transmited to the construction below, the panel should be anchored with stiff fasteners. Of course this method, like the previous one, prescribes controlling the buckling and capacity of vertical wall studs and stresses at local areas, especially the compression perpendicular to the grain. If the condition (4) is met shear buckling of the sheet may be disregarded according to EC5.

4. DESIGN OF VERTICAL STUD WALLS

In general case stud walls may be subjected either to axial compression or combined compression and bending which are typically causes the wind loading. The strength of the wall is primarily derived from the studs and all concentrated loading should be located directly over the studs and not in the span area of the header plate. In cases where the sheathing material is able to provide adequate lateral restraint, the risk of buckling of the studs about the *z–z* axis can be ignored [5].

I. DESIGN OF STUD WALLS SUBJECTED TO AXIAL COMPRESSION

a) Relative slenderness ratio $\lambda_{rel,y} > 0,3$ and $\lambda_{rel,z} > 0,3$

$$\frac{\sigma_{c,0,d}}{k_{c,\mathrm{y}} \cdot f_{c,0,d}} \leq 1,0 \quad \text{and} \quad \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} \leq 1,0 \tag{6}$$

i.e. $\sigma_{c,0,d} \leq k_{c,y} \cdot f_{c,0,d}$ and $\sigma_{c,0,d} \leq k_{c,z} \cdot f_{c,0,d}$

where $\sigma_{c,0,d} = \dfrac{N_d}{A}$ is the design compressive stress parallel to the grain; N_d is the design axial load on the stud; A is the cross-sectional area;

$f_{c,0,d} = \dfrac{k_{\mathrm{mod}} \cdot k_{sys} \cdot f_{c,0,k}}{\gamma_m}$ is the design compressive strength parallel to the grain.

b) Relative slenderness ratio $\lambda_{rel,y} > 0,3$ and $\lambda_{rel,z} \leq 0,3$

$$\sigma_{c,0,d} \leq k_{c,y} \cdot f_{c,0,d} \tag{7}$$

c) Relative slenderness ratio $\lambda_{rel,y} \leq 0,3$ and $\lambda_{rel,z} \leq 0,3$

$$\frac{\sigma_{c,0,d}}{f_{c,0,d}} \leq 1,0 \tag{8}$$

II. DESIGN OF STUD WALLS SUBJECTED TO COMBINED BENDING (OUT OF PLANE) AND AXIAL COMPRESSION

a) Relative slenderness ratio $\lambda_{rel,y} > 0,3$ and $\lambda_{rel,z} > 0,3$

*If the lateral torsional buckling of the stud will not arise (relative slenderness ratio for bending of each stud is $\lambda_{rel,m} \leq 0,75$

$$\frac{\sigma_{c,0,d}}{k_{c,\mathrm{y}} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1,0 \quad \text{and} \quad \frac{\sigma_{c,0,d}}{k_{c,\mathrm{y}} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1,0 \tag{9}$$

$\sigma_{m,z,d} = 0$; $\sigma_{m,y,d} = \frac{M_{y,d}}{W_y} = \frac{6M_{y,d}}{b \cdot h^2}$ is the design bending stress about the major (y-y) axis of the stud;

$f_{m,y/z,d} = \frac{k_{\text{mod}} \cdot k_h \cdot k_{sys} \cdot f_{m,k}}{\gamma_m}$ is the design bending strength about major axis.

**If the lateral torsional buckling of the stud can appear (relative slenderness ratio for bending of each stud is $\lambda_{rel,m} > 0,75$

$$\left(\frac{\sigma_{m,y,d}}{k_{crit} \cdot f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} \leq 1,0 \tag{10}$$

b) Relative slenderness ratio $\lambda_{rel,y} > 0,3$ and $\lambda_{rel,z} \leq 0,3$

$$\frac{\sigma_{c,0,d}}{k_{c,\text{y}} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1,0 \tag{11}$$

c) Relative slenderness ratio $\lambda_{rel,y} \leq 0,3$ and $\lambda_{rel,z} \leq 0,3$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1,0 \tag{11}$$

5. CONCLUSION

In the modern design of timber structures it's necessary to fulfill all the requirements that are EC5 prescribes. Regardless of the racking resistance of the timber shear walls it's always recommended to design bracings and other elements that could provide lateral stability of structure. The basic requirements that must be met according to ULS are: static equilibrium and racking resistance of wall assembly wich is shown above. Also, simplified methods "Method A" and "Method B" are presented with all relevant factors and parameters. In both cases the main concept is to determine the racking strength of single shear wall and based on this, get the total racking strength of wall assembly as a sum of all. The difference between Method A and B depends of the boundary condition being assumed. These methods, probably will be replaced in future by a unique method because in certain cases can give inaccurate results [5]. The aim of this paper is to provide important instructions and notes for analyzing shear wall diaphragms in timber frame structures (platform frame construction) and this problem bring closer to the wider professional community.

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МЕТОДЕ ПРОРАЧУНА ДРВЕНИХ ЗИДНИХ ПАНЕЛА ПРЕМА ЕВРОКОДУ 5

Резиме: *Рад презентује концепт прорачуна зидних дијафрагми скелтних дрвених конструкција према Еврокоду 5. У раду се анализирају фактори који утичу на отпорност на смицање вертикалних панела. Просторна стабилности дрвених конструкција зграда, у општем случају, постиже се конструисањем спрегова и/или формирањем кровних, међуспратних и зидних дијафрагми. Основни захтеви који морају бити испуњени према граничном стању носивости су: статичка равнотежа панела и отпорност на смицање. За анализу носивости зидног панела (дијафрагме), с обзиром на положај оптерећења, Еврокод 5 прописује две упрошћене методе (Метод А и Метод Б). Циљ анализе је да покаже различите аспекте прорачуна савремених метода и њихову ефикасност у односу на традиционалне концепте али и да стручној јавности приближи специфичну методологију коју треба применити приликом пројектовања дрвених конструкција приземних и вишеспратних објеката.*

Кључне речи: *Зидни панел, еврокод 5, смицање, стабилност.*