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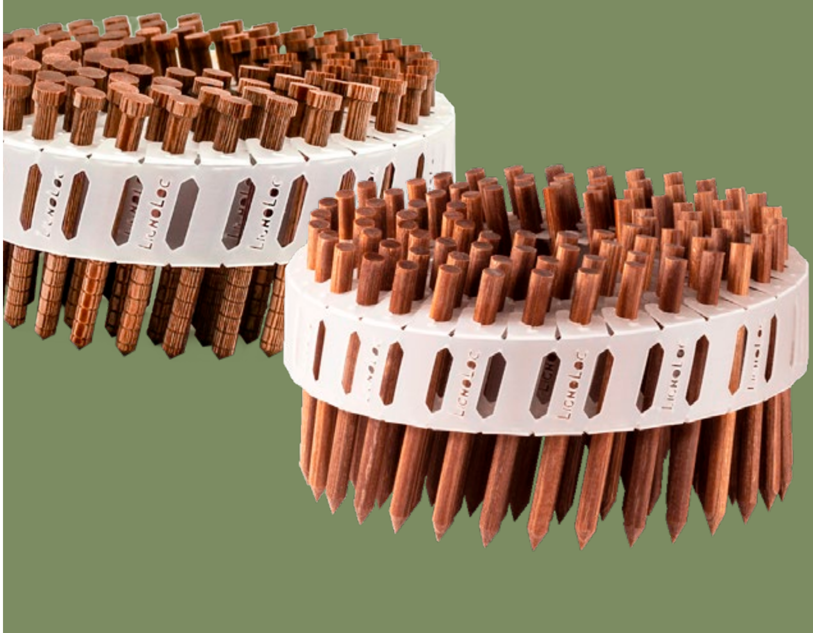
# Structural analysis of a shear wall in wood panel construction

According to DIN EN 1995-1-1:2010-12 with LIGNOLOC® wooden nails



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# 01 // System

A single-family home constructed using timber panel construction is considered. The exterior wall element presented below was identified as the most critical and will be used for the subsequent structural design.

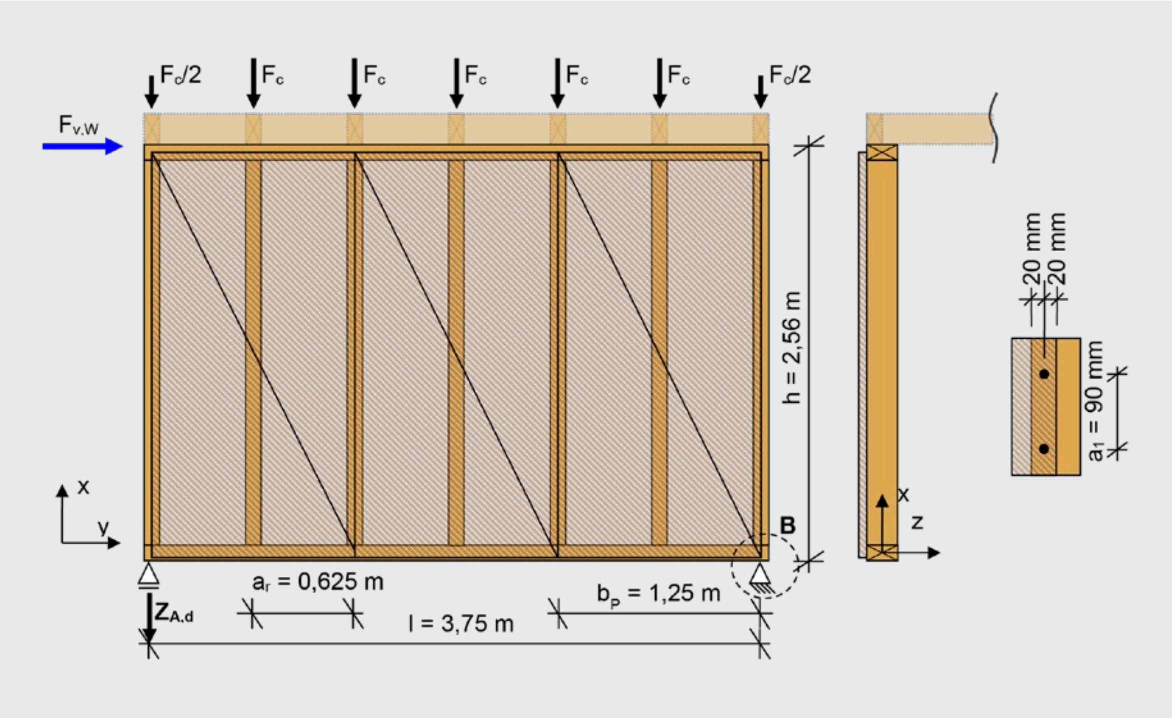
**Edge studs:** Softwood C24, $b_{Ri}$ x $h_{Ri}$ = 80 x 120 mm², NKL 1

**Inner studs:** Softwood C24, $b_{Ri}$ x $h_{Ri}$ = 80 x 120 mm², NKL 1, $a_r$ = 0,625 m (stud spacing)

**Bottom/Top plates:** Softwood C24, $b_S$ x $h_S$ = 60 x 120 mm², NKL 1

**Sheathing:** OSB/4, $t_P$ = 18 mm, applied on one side according to DIN EN 12369-1:2001

**Fasteners:** LIGNOLOC® wooden nails, d = 3.7 mm, l = 50 mm, $a_1$ = 90 mm




| LOAD TYPE | CHARACTERISTIC VALUE | LOAD DURATION CLASS | $\psi_0$ |
|---|---|---|---|
| **Permanent load** | $F_{c,G,k}$ = 2.0 kN | Permanent | - |
| **Imposed load** | $F_{c,Q,k}$ = 5.0 kN | Medium-term | 0.7 |
| **Snow load < 1000 m** | $F_{c,S,k}$ = 3.0 kN | Short-term | 0.5 |
| **Wind load (in plane)** | $F_{v,W,k}$ = 5.0 kN | Short/very short-term | 0.6 |
| **Wind load (out-of-plane)** | $w_k$ = 0.4 kN/m² | Short/very short-term | 0.6 |

# 02 // Application requirement

### For the simplified design of shear walls according to method A of DIN EN 1995-1-1:2010-12

## Anchorage

Is end anchorage provided? ✓ 9.2.4.2 (1)

## Sheathing

Is the width of each panel at least h/4? 9.2.4.2 (2)

$$b_p = 1{,}25\,m \geq \frac{h}{4} = \frac{2{,}56}{4} = 0{,}64\,m\,\checkmark$$

Is there a maximum of one horizontal panel joint? ✓ 9.2.4.2 (NA.20)

Are the panel edges rigidly connected in shear? ✓

## Fasteners as per ETA-23/0041, analogous to unpre-drilled nails per EN 1995-1-1:2010-12

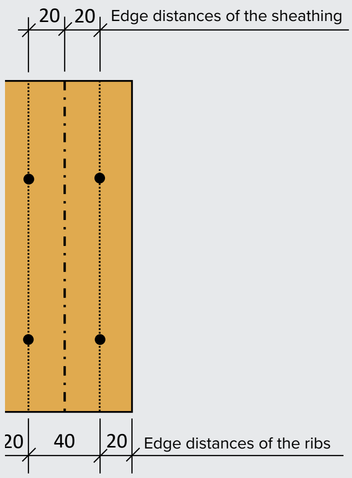
Is a consistent fastener spacing provided along all edges? 9.2.4.2 (2)

$a_1$ = 90 mm ✓

Fastener Spacing Verification

$a_1$ = 90 mm ≤ 150 mm (Nails) ✓ 10.8.2 (1)

$a_1$ = 90 mm ≤ 80 · d = 80 · 3,7 = 296 mm ✓ 8.3.1.3 (NA.12)




Have the minimum fastener spacing requirements in the studs been met? Table 8.2

$$a_{1,min,VH} = (5 + 5/\cos\alpha)\cdot d = (5 + 5(\cos 0^\circ))\cdot 3{,}7 = 37\,mm \leq 90\,mm\,\checkmark$$

$$a_{2,min,VH} = 5\cdot d = 5\cdot 3{,}7 = 18{,}5\,mm \leq 40\,mm\,\checkmark$$

$$a_{4,c,min,VH} = 5\cdot d = 5\cdot 3{,}7 = 18{,}5\,mm \leq 20\,mm\,\checkmark$$

Have the minimum fastener spacings in the OSB panel been met?

$$a_{4,c,min} = 3\cdot d = 3\cdot 3{,}7 = 11{,}1\,mm \leq 20\,mm\,\checkmark$$ 8.3.1.3 (NA.13

# 03 // Load on the edge rib

## Characteristic axial forces in the edge stud

From permanent load: $F_{Ri,G,k} = 0{,}5 \cdot F_{c,G,k} = 0{,}5 \cdot 2{,}0\,kN = 1{,}0\,kN$

From imposed load: $F_{Ri,Q,k} = 0{,}5 \cdot F_{c,Q,k} = 0{,}5 \cdot 5{,}0\,kN = 2{,}5\,kN$

From snow load: $F_{Ri,S,k} = 0{,}5 \cdot F_{c,S,k} = 0{,}5 \cdot 3{,}0\,kN = 1{,}5\,kN$

From wind load: $F_{Ri,W,k} = F_{v,W,i,k} \cdot h/l = 5{,}0\,kN \cdot 2{,}56\,m/3{,}75\,m = 3{,}41\,kN$

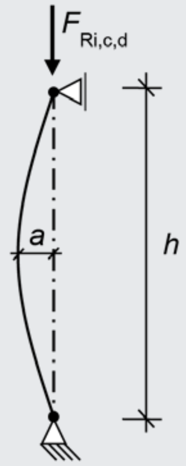
## Design verification based on the following load case (wind as leading action)

$$E_d = E\left\{\sum_{j\geq 1}\gamma_{G,j} \cdot G_{k,j} \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i>1}\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right\}$$

$$F_{Ri,c,d} = 1{,}35 \cdot F_{Ri,G,k} + 1{,}5 \cdot F_{Ri,W,k} + 1{,}5 \cdot 0{,}7 \cdot F_{Ri,Q,k} + 1{,}5 \cdot 0{,}5 \cdot F_{Ri,S,k}$$

$$F_{Ri,c,d} = 1{,}35 \cdot 1{,}0\,kN + 1{,}5 \cdot 3{,}41\,kN + 1{,}5 \cdot (0{,}7 \cdot 2{,}5\,kN + 0{,}5 \cdot 1{,}5\,kN) = 10{,}21\,kN$$

**NOTE:** According to DIN EN 1990/NA:2010-12, all possible combinations of actions must be considered. In the design of timber structures, different kmod values must be applied depending on the load duration class. To maintain clarity in this example, only a single load combination is considered. However, for practical structural verification, all relevant load cases must be analysed in full.




## Design value of the bending moment due to imperfection

10.2 (1)

$$M_d = F_{Ri,c,d} \cdot a = F_{Ri,c,d} \cdot \tfrac{h}{300} = 10{,}21\,kN \cdot \tfrac{2{,}56}{300} = 0{,}09\,kNm$$

## Design value of the bending moment due to wind

$$M_{d,Wind} = \left(1{,}5 \cdot 0{,}4\,kN/m^2 \cdot 0{,}625\,m/2\right) \cdot 2{,}56^2 m^2/8 = 0{,}154\,kNm$$

$$M_{d,ges} = M_d + M_{d,Wind} = 0{,}09\,kNm + 0{,}154\,kNm = 0{,}24\,kNm$$

# 04 // Shear flow in the composite and sheathing layers

## Design value of horizontal action

$$F_{v,d} = \gamma_Q \cdot F_{v,W,k} = 1{,}5 \cdot 5{,}0\,kN = 7{,}5\,kN$$

## Design value of shear

$$s_{v,0,d} = \frac{F_{v,d}}{l} = \frac{7{,}5\,kN}{3{,}75\,m} = 2{,}0\,\frac{kN}{m}$$

# 05 // Calculation of the governing anchorage forces

**Note:** The maximum uplift force can be determined for the load cases “permanent load” and “wind”. Since the effect of “permanent load” is stabilising, it must be multiplied by the partial safety factor γG,stb = 0.9 in accordance with DIN EN 1990/NA:2010-12, Table NA.A.1.2(A).

## Force equilibrium about point B

$$Z_{A,d} = \frac{1}{l} \cdot \left[\gamma_Q \cdot F_{v,W} \cdot h - \gamma_{G,stb} \cdot F_{c,G,k} \cdot \left(a_r + 2 \cdot a_r + 3 \cdot a_r + 4 \cdot a_r + 5 \cdot a_r + 6 \cdot a_r \cdot 1/2\right)\right]$$

$$Z_{A,d} = \frac{1}{3{,}75} \cdot \left[1{,}5 \cdot 5{,}0\,kN \cdot 2{,}56 - 0{,}9 \cdot 2{,}0\,kN \cdot \left(0{,}625 + 2 \cdot 0{,}625 + 3 \cdot 0{,}625 + 4 \cdot 0{,}625 + 5 \cdot 0{,}625 + 6/2 \cdot 0{,}6252\right)\right]$$

$$Z_{A,d} = -\,0{,}28\,kN$$

Due to the acting overpressure, no mechanical fasteners are needed to resist uplift in this case.

# 06 // Verification of the edge stud

## In-plane buckling

6.3.1 (NA.5)

$$\left.\begin{array}{l} a_r = 62{,}5\,cm \leq 50 \cdot t_P = 50 \cdot 1{,}8\,cm = 90\,cm \\ \\ h_{Ri}/b_{Ri} = 120/80 = 1{,}5 \leq 4 \end{array}\right\}$$

No Buckling

## Out-of-plane buckling

6.3.2

$$\sigma_{c,0,d} = \frac{F_{Ri,c,d}}{A} = \frac{10{,}21\,kN \cdot 10^{-3}}{0{,}12 \cdot 0{,}08\,m^2} = 1{,}06\,MN/m^2$$

$$\sigma_{m,d} = \frac{M_{d,ges}}{W} = \frac{0{,}24\,kNm \cdot 10^{-3} \cdot 6}{0{,}12^2 \cdot 0{,}08\,m^3} = 1{,}25\,MN/m^2$$

$$f_{c,o,d} = k_{mod} \cdot \frac{f_{c,0,k}}{\gamma_M} = \frac{1{,}0 \cdot 21\,MN/m^2}{1{,}3} = 16{,}15\,MN/m^2$$

$$f_{m,d} = k_{mod} \cdot \frac{f_{m,k}}{\gamma_M} = \frac{1{,}0 \cdot 24\,MN/m^2}{1{,}3} = 18{,}46\,MN/m^2$$

## Buckling coefficient

according to Schneider (24th ed.), table 9.29a or equation (6.25)

$$\lambda_y = \frac{l_{ef}}{i_y} = \frac{2{,}56}{0{,}289 \cdot 0{,}12} = 73{,}8$$

Interpolation of table values: $k_{c,y} = 0{,}51$

## Lateral-torsional buckling coefficient

according to Schneider (24th ed.), table 9.32 or equation (6.34)

$$\frac{l_{ef} \cdot h}{b^2} = \frac{2{,}56 \cdot 0{,}12}{0{,}08^2} = 48 \leq 135$$

$$k_{crit} = 1{,}0$$

## Verification

equation (NA.60)

$$\eta = \frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,o,d}} + \frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} = \frac{1{,}06 MN/m^2}{0{,}51 \cdot 16{,}15} + \frac{1{,}25 MN/m^2}{1{,}0 \cdot 18{,}46} = 0{,}20 \leq 1{,}0 \checkmark$$

# 07 // Verification of bottom plate compression

## Overhangs

$$ü_{li} = Min\begin{Bmatrix} 30\,mm \\ a \\ l \\ l_1/2 \end{Bmatrix} = Min\begin{Bmatrix} 30\,mm \\ - \\ 80\,mm \\ (625-80)/2 = 272{,}5\,mm \end{Bmatrix} = 30\,mm$$

$$\sigma_{c,90,d} = \frac{F_{Ri,c,d}}{A_{ef}} = \frac{10{,}21 \cdot 10^{-3}}{0{,}12 \cdot (0{,}08+0{,}03)} = 0{,}77\,MN/m^2$$

## Perpendicular-to-grain bearing coefficient

Continuous support, sole plate made of softwood (VH)

$$l_1 = (0{,}625 - 0{,}08) = 0{,}545\,m \geq 2 \cdot h_{Ri} = 2 \cdot 0{,}08\,m = 0{,}16\,m$$

$$\Rightarrow k_{c,90} = 1{,}25$$

*6.1.5 (3)*

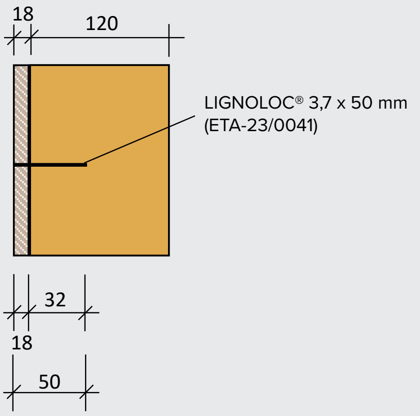
$$f_{c,90,d} = k_{mod} \cdot \frac{f_{c,90,k}}{\gamma_M} = 1{,}0 \cdot \frac{1{,}2 \cdot 2{,}5\,MN/m^2}{1{,}3} = 2{,}31\,\frac{MN}{m^2}$$

*9.2.4.2 (NA.21)*
*(20% increase in load-bearing capacity)*

## Verification

$$\eta = \frac{\sigma_{c,90,d}}{k_{c,90} \cdot f_{c,90,d}} = \frac{0{,}77\,MN/m2}{1{,}25 \cdot 2{,}31\,MN/m^2} = 0{,}27 \leq 1{,}0\,\checkmark$$

# 08 // Verification of the LIGNOLOC® fasteners




## Embedment strength of the OSB board

Gl. (8.22)

$$f_{h,1,k} = 65 \cdot d^{-0,7} \cdot t_P^{0,1} = 65 \cdot 3{,}7^{-0,7} \cdot 18^{0,1} = 34{,}73\, N/mm^2$$

$$f_{h,1,d} = f_{h,1,k} \cdot \frac{k_{mod,1}}{\gamma_M} = 34{,}73\, N/mm^2 \cdot \frac{1{,}0}{1{,}3} = 26{,}7\, N/mm^2$$

## Embedment strength of solid wood (α=0°)

according to the ETA

$$f_{h,2,k} = 0{,}082 \cdot \rho_k \cdot d^{-0,3} = 0{,}082 \cdot 350\, kg/m^3 \cdot 3{,}7^{-0,3} = 19{,}38\, N/mm^2$$

$$f_{h,2,d} = f_{h,2,k} \cdot \frac{k_{mod,1}}{\gamma_M} = 19{,}38\, N/mm^2 \cdot \frac{1{,}0}{1{,}3} = 14{,}91\, N/mm^2$$

$$\beta = \frac{f_{h,2,d}}{f_{h,1,d}} = \frac{14{,}91\, N/mm^2}{26{,}7\, N/mm^2} = 0{,}56 \leq 1{,}0$$

## Design value of the yield moment

ETA table B.1

$$M_{u,k} = 1200\, Nmm$$

$$M_{u,d} = M_{u,k} \cdot \frac{k_{mod,M}}{\gamma_M} = 1200\, Nmm \cdot \frac{0{,}9}{1{,}3} = 830{,}77\, Nmm$$

## Required minimum embedment depth 1 (OSB/4)

according to the ETA

$$t_{1,req} = \sqrt{\frac{\beta}{1+\beta} + 1} \cdot \sqrt{\frac{4 \cdot M_{u,d}}{0{,}75 \cdot f_{h,1,d} \cdot d}} = \sqrt{\frac{0{,}56}{1+0{,}56} + 1} \cdot \sqrt{\frac{4 \cdot 830{,}77\, Nmm}{0{,}75 \cdot 26{,}7\, \frac{N}{mm^2} \cdot 3{,}7}} = 7{,}81\, mm$$

$$t_{1,req} = 7{,}81\, mm \leq t_{vorh} = 18{,}0\, mm \checkmark$$

## Required minimum embedment depth 2 (VH)

according to the ETA

$$t_{2,req} = \sqrt{\frac{1}{1+\beta}} + 1 \cdot \sqrt{\frac{4 \cdot M_{u,d}}{0{,}75 \cdot f_{h,2,d} \cdot d}} = \sqrt{\frac{1}{1+0{,}56}} + 1 \cdot \sqrt{\frac{4 \cdot 830{,}77\,Nmm}{0{,}75 \cdot 14{,}91\,\frac{N}{mm^2} \cdot 3{,}7}} = 11{,}48\,mm$$

$$t_{2,req} = 11{,}48\,mm \leq\ t_{vorh} = 32{,}0\,mm$$

## Design of the load-bearing capacity per nail

$$F_{v,Rd,Na} = \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{1{,}5 \cdot M_{u,d} \cdot f_{h,1,d} \cdot d}$$

$$F_{v,Rd,Na} = \sqrt{\frac{2 \cdot 0{,}56}{1+0{,}56}} \cdot \sqrt{1{,}5 \cdot 830{,}77\,Nmm \cdot 26{,}7\,N/mm^2 \cdot 3{,}7\,mm}$$

$$F_{v,Rd,Na} = 297{,}30\,N$$

## Shear resistance of the sheathing

$$S_{v,0,R,d} = \frac{F_{v,Rd}}{a_1} = \frac{297{,}30 \cdot 10^{-3}\,kN}{0{,}09\,m} = 3{,}30 kN/m$$

## Verification the shear action

$$\eta = \frac{S_{v,0,d}}{S_{v,0,R,d}} = \frac{2{,}0\,kN/m}{3{,}30\,kN/m} = 0{,}61 \leq 1{,}0\,\checkmark$$

## Verification of the sheathing

Load-bearing capacity of the sheathing considering the arrangement

9.2.4.2 (NA.16)

$$f_{v,0,d} = \frac{k_{mod} \cdot 0{,}33 \cdot f_{v,k,OSB}}{\gamma_M} = \frac{1{,}0 \cdot 0{,}33 \cdot 6{,}9}{1{,}3} = 1{,}75\,kN/m^2$$

## Buckling of the sheathing

Buckling of the sheathing

$$\frac{a_r}{35} = \frac{625}{35} = 17{,}86\,mm < \ t_P = 18\,mm$$

## Verification of the sheathing

$$\eta = \frac{F_{v,Rd,Na}/(t_P \cdot a_1)}{f_{v,d}} = \frac{297{,}30\,N/(18\,mm \cdot 90\,mm)}{1{,}75\,N/mm^2} = 0{,}10 \leq 1{,}0\,\checkmark$$

# 09 // Horizontal deformation

## Conditions

panel length $l = 3{,}75\,m \geq h/3 = 2{,}56/3 = 0{,}85\,m$

panel width $b_p = 1{,}25\,m \geq h/4 = 2{,}56/4 = 0{,}64\,m$

+ Panel supported by a rigid substructure
+ no increase in the fastener load-bearing capacity according to EC 9.2.4.2 (5) considered

⟶ no verification required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