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IN-PLANE STIFFNESS OF LARGE-AREA FRAMED FLOOR DIAPHRAGMS CONSTRUCTED WITH ORIENTED STRAND BOARD PANEL SHEATHING USING A SWISS CASE STUDY

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ABSTRACT: In a case study of a Swiss building, wood structural panels made from oriented strand board (OSB) are stapled continuously on the top side of a hollow-core timber slab to create one large-area framed floor diaphragm measuring 41.1 x 50 m^2. The diaphragm acts as horizontal bracing structure for six 41.1 m long steel trusses. The in-plane stiffness is investigated due to the internal stability load of the trusses in the ultimate limit state. A finite element model is developed using linear elastic material behaviour. The numerical total in-plane displacements are compared to the analytical results of the design method based on the *shear field beam theory*. The stiffness of the sheathing-to-framing connection and the size of the OSB panels are decisive factors for limiting the total in-plane displacement, which easily exceeds the limit value.

KEYWORDS: framed floor diaphragm, sheathing, OSB panel, in-plane displacement, finite element model, case study

1 – INTRODUCTION

In Switzerland, an increased number of public buildings such as school buildings and sports centres, characterised by a large floor area, are being constructed as hybrid timber constructions. Large-area framed floor diaphragms constructed with wood structural panels as part of a sheathing are used to transfer lateral forces to the vertical resisting elements and are integral in maintaining the structural integrity of buildings subjected to wind, seismic activity, and other horizontal loads. OSB panels are typically used as wood structural panels and offer a blend of strength, flexibility, and ease of installation. The paper deals with the total in-plane displacement of a large-area framed floor diaphragm. The applicability in modern construction is highlighted using the example of a case study of a Swiss building. Here, the simply supported framed floor diaphragm acts as bracing structure of six steel trusses, which must resist the uniformly distributed in-plane design load from the lateral buckling of the steel trusses. The in-plane stiffness is investigated.

2 – CASE STUDY

The case study relates to the new building of the Hagenmatt Root school complex in Switzerland. Six steel trusses each 41.1 m long (see Fig. 2a) support the ceiling of the sports hall as well as the timber construction of the upper floor. Prestressed hollow-core concrete slabs are supported at the bottom chords. The bracing is provided by a layer of in-situ concrete, to resist the horizontal earth pressure exerted on the underground sports hall. At the top chords, hollow-core timber slabs (see Fig. 1) span between the steel trusses (see Fig. 2c). Here, OSB panels are stapled continuously on top of the timber slabs and over the entire floor area (41.1 x 50 m^2). Along the four sides of the floor area, the timber slabs are framed by glulam beams. The framed floor diaphragm acts as a horizontal bracing structure and transfers the lateral forces to vertical steel cross-bracing elements in the four corners of the building. Three staircases without stabilization function are arranged at the centre of the floor area.

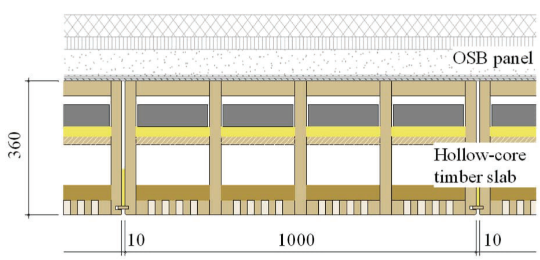

Figure 1. Cross-section of the hollow core timber slab (LFE360-REI60-AT3.1-silence12 according to [1]) sheathed with OSB panels.

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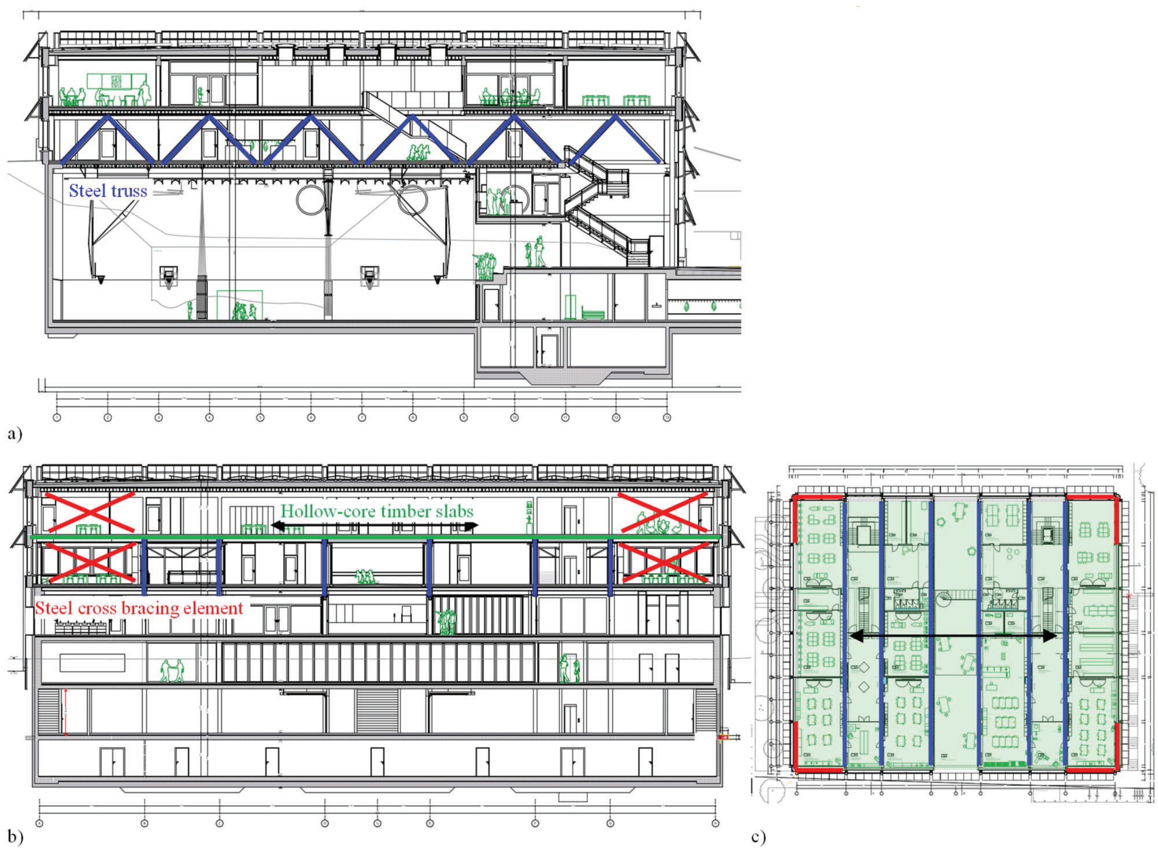

Figure 2. Hagenmatt Root school complex in Switzerland (modified drawings from Bürgi Burkhard von Euw GmbH) – a) Transversal and b) longitudinal section; c) Floor plan of the 2nd floor.

3 – BACKGROUND

3.1 FRAMED FLOOR DIAPHRAGM

The framed floor diaphragm consists of three components, which contribute to the load-bearing capacity and the stiffness of the system: 1) the framing members acting as compression and tension chords (i.e. glulam beams), 2) the sheathing (i.e. OSB panels), and 3) the fasteners connecting the sheathing to the framing members as well as to the timber slabs (i.e. staples). The verification of a tension diagonal in former design standards was mechanically incorrect because a tension diagonal requires the buckling of the sheathing, which should be avoided. The approach has therefore been replaced by the so-called *shear field theory* [2]. Here, the framing members are joined by hinges, the in-plane forces are continuously transferred into the sheathings, and the edges of the wood structural panels of the sheathings are continuously connected with mechanical fasteners to the supporting structure of the floor system. This approach results into a continuous shear flow along the edges of the wood structural panels. The load-bearing capacity of the connections is decisive for the design, i.e. ductile failure of the staples is preferred over brittle failure modes, such as buckling or shear failure of the sheathing.

3.2 BRACING STRUCTURE OF TRUSSES

The simply supported, large-area framed floor diaphragm acts as bracing structure for the six steel trusses (see Fig. 2c). Based on its in-plane flexibility, the requirement for a rigid support condition can no longer be met (see Fig. 3). The framed floor diaphragm must be dimensioned while considering its flexibility. The design action, that is decisive for the ultimate limit state design, is caused by the internal instability of the steel trusses, parallel to the timber slabs.

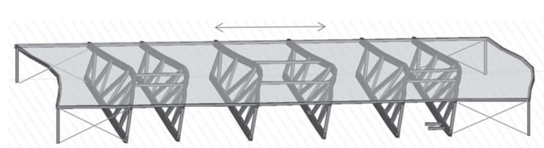

Figure 3. Framed floor diaphragm as bracing structure of the steel trusses.

The horizontal design action depends on the sum of the mean design compressive forces $N_{i,Ed,mean}$ in the upper

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chords of the six steel trusses and is evenly distributed in the plane (see Fig. 4). The internal stability load $q_{\mathrm{Ed,ULS}}$ has been determined by (1) according to [3, 4]:

$$q_{\mathrm{Ed,ULS}} = \frac{\sum N_{i,\mathrm{Ed,mean}}}{30 \cdot l} = \frac{6 \cdot (1-k_{\mathrm{crit}}) \cdot M_{\mathrm{Ed,max}}/h_{\mathrm{truss}}}{30 \cdot l} \quad (1)$$

where k_{crit} is a factor which considers the reduced bending strength due to lateral buckling, and $M_{\mathrm{Ed,max}}$ is the maximum bending moment acting on the steel trusses with the height h_{truss}. The displacement of the bracing structure due to the internal stability load $q_{\mathrm{Ed,ULS}}$ should not exceed a limit value of $l/500$; i.e. 41100/500 = 82.2 mm.

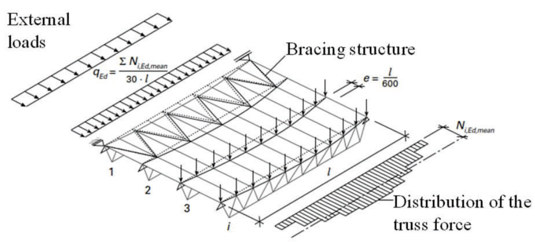

Figure 4. Trusses stabilized laterally by a bracing structure (modified from [4]).

3.3 STIFFNESS PROPERTIES

3.3.1 SERVICEABILITY LIMIT STATE

The mean values of the stiffness properties are used for the serviceability limit state (SLS), as listed in Table 1. The mean modulus of elasticity E_{mean} and the mean shear modulus G_{mean} are taken for the wooden framing members and the OSB panel sheathing according to the Swiss design standard for timber structures [4, 5]. The mean slip moduli of the connection parallel to the grain $K_{\mathrm{mean,0}}$ and perpendicular to the grain $K_{\mathrm{mean,90}}$ are determined by (2)-(3), in kN/m:

$$K_{\mathrm{mean,0}} = 2 \cdot 60 \cdot d^{1.7} \quad (2)$$

$$K_{\mathrm{mean,90}} = \frac{1}{2} K_{\mathrm{SLS,0}} = 60 \cdot d^{1.7} \quad (3)$$

based on the diameter d of the staples (with a minimum angle to the grain of at least 30°).

3.3.2 ULTIMATE LIMIT STATE

According to the Swiss standard [4], reduced stiffnesses should be used for the ultimate limit state (ULS). The design values of the mean modulus of elasticity E_{d}, the mean shear modulus G_{d}, and the mean slip modulus of the connection $K_{\mathrm{d},i}$ are taken by (4)-(6) as follows:

$$E_{\mathrm{d}} = {\eta_{\mathrm{mod}}}/{\gamma_{\mathrm{M}}}\, E_{\mathrm{mean}} \quad (4)$$

$$G_{\mathrm{d}} = {\eta_{\mathrm{mod}}}/{\gamma_{M}}\, G_{\mathrm{mean}} \quad (5)$$

$$K_{\mathrm{d},i} = {\eta_{\mathrm{mod}}}/{\gamma_{\mathrm{M}}}\, K_{\mathrm{u},i} \quad (6)$$

with $K_{\mathrm{u},i} = \frac{2}{3} K_{\mathrm{mean},i}$ (7)

where ${\eta_{\mathrm{mod}}}/{\eta_{M}}$ is the factor accounting for the effect of the duration of load divided by the partial factor for the stiffness property, E_{mean} and G_{mean} are the mean modulus of elasticity and the mean shear modulus, and $K_{\mathrm{u},i}$ is the design value of the mean slip modulus of the connection. The latter $K_{\mathrm{u},i}$ depends on the mean slip modulus of the connection $K_{\mathrm{mean},i}$ and the direction of the grain, i.e. parallel (i=0°) or perpendicular (i=90°) (see (2) and (3)).

For long-term analyses, the design values of the final mean modulus of elasticity $E_{\mathrm{d,fin}}$, and the final mean shear modulus $G_{\mathrm{d,fin}}$ are taken according to EN 1995-1-1 [3] by (8)-(9) as follows:

$$E_{\mathrm{d,fin}} = \frac{E_{\mathrm{d}}}{(1+k_{\mathrm{def}})} \quad (8)$$

$$G_{\mathrm{d,fin}} = \frac{G_{\mathrm{d}}}{(1+k_{\mathrm{def}})} \quad (9)$$

where k_{def} is the factor for the evaluation of creep deformation accounting for the relevant service class, i.e. service class 1.

The connection is constituted of two wood-based elements having different time-dependent behaviour. The design value of the final mean slip modulus of the connection $K_{\mathrm{d,fin},i}$ depends again on the direction of the grain, i.e. parallel (i=0°) or perpendicular (i=90°), and is taken by (10) as follows:

Table 1. Stiffness properties of the framing members (C24, 160 x 360 mm²), of the sheathing (OSB/3, d = 25 mm), and of the connection (staples d = 1.53 mm, n_v = 2, a_v = 35 mm) according to [4, 5], depending on the limit state design.

Limit state		SLS			ULS							
		E_{mean}	G_{mean}	K_{mean}	$\eta_{\mathrm{mod}}/\gamma_{\mathrm{M}}$	E_{d}	G_{d}	K_{d}	$1+k_{\mathrm{def}}$	$E_{\mathrm{d,fin}}$	$G_{\mathrm{d,fin}}$	$K_{\mathrm{d,fin}}$
		[kN/mm²]	[kN/mm²]	[kN/m]	[-]	[kN/mm²]	[kN/mm²]	[kN/m]	[-]	[kN/mm²]	[kN/mm²]	[kN/m]
C24		11.00	0.69	/	0.59	6.47	0.41	/	1.60	4.04	0.25	/
OSB/3		3.80	1.08	/	0.58	2.22	0.63	/	2.50	0.89	0.25	/
Staples	0°	/	/	247	0.58	/	/	96	1.90	/	/	51
	90°	/	/	124	0.58	/	/	48	1.90	/	/	25

$$K_{\mathrm{d,fin},i} = \frac{K_{\mathrm{d},i}}{(1+k_{\mathrm{ef}})} = \frac{K_{\mathrm{d},i}}{(1+2*\sqrt{k_{\mathrm{def,1}} \cdot k_{\mathrm{def2}}})} \tag{10}$$

where $k_{\mathrm{def,1}}$ and $k_{\mathrm{def,2}}$ are the deformation factors for the two connected timber elements.

3.4 DETERMINATION OF THE TOTAL IN-PLANE DISPLACEMENT

The total in-plane displacement of the framed floor diaphragm is determined using an analytical design method documented in [6] by Lignum, the umbrella organization of the Swiss forestry and timber industry. The design method is based on the extended shear field theory, specifically the so-called *shear field beam theory* by Kessel et al. [7, 8] and is included in the current version of EN 1995-1-1:2004, i.e. prEN 1995-1-1:2024 [9]. Depending on the limit state design, the stiffness properties of Table 1 are taken as mean values, design values of the mean values (see (4) - (6)) or design values of the final mean values (see (8) - (10)).

Equation (11) determines the total in-plane displacement $u_{\mathrm{tot,an}}$ for a floor area of the length l and the height h and sheathed by n sheathings:

$$u_{\mathrm{tot,an}} = u_{\mathrm{N}} + \frac{1}{\sum \frac{1}{\sum_{i=1}^{n} \frac{1}{u_{\mathrm{V}}}}} + \frac{1}{\sum \frac{1}{\sum_{i=1}^{n} \frac{1}{u_{\mathrm{K}}}}} \tag{11}$$

In the present case study, the framed floor diaphragm is sheathed by one OSB panel sheathing. For n=1 sheathing, Equation (11) can be simplified by (12) as follows:

$$u_{\mathrm{tot,an}} = u_{\mathrm{N}} + u_{\mathrm{V}} + u_{\mathrm{K}} \tag{12}$$

The in-plane displacement from the axial deformation of the framing members $u_{\mathrm{N,an}}$ is determined by (13):

$$u_{\mathrm{N,an}} = \frac{5 \cdot q_{\mathrm{Ed}} \cdot l^4}{384 \cdot (EI)_{\mathrm{ef}}} \tag{13}$$

with $$(EI)_{\mathrm{ef}} = \sum\left(E_{\mathrm{ch}} A_{\mathrm{d,ch}} z_{\mathrm{ch}}{}^2\right) \tag{14}$$

$$= 2 \cdot E_{\mathrm{ch}} \cdot A_{\mathrm{ch}} \cdot \left(\frac{h}{2}\right)^2$$

where q_{Ed} is the uniformly distributed in-plane design load applied to the floor diaphragm, and $(EI)_{\mathrm{ef}}$ is the effective bending stiffness, which depends on the framing members' modulus of elasticity E_{ch}, cross-section area A_{ch} and distance to the centroid of the system z_{ch}, as well as on the height h of the framed floor diaphragm. The design method disregards the bending stiffness of the OSB panel sheathing and the influence of the connection between the sheathing and the framing members on the effective bending stiffness.

The in-plane displacement from the shear deformation of the sheathing $u_{\mathrm{V,an}}$ is determined by (15):

$$u_{\mathrm{V,an}} = \frac{q_{\mathrm{Ed}} \cdot l^2}{8 \cdot (GA)_{\mathrm{p,ef}}} = \frac{q_{\mathrm{Ed}} \cdot l^2}{8 \cdot \kappa \cdot G_{\mathrm{p}} \cdot A_{\mathrm{p}}} = \frac{q_{\mathrm{Ed}} \cdot l^2}{8 \cdot \kappa \cdot G_{\mathrm{p}} \cdot h \cdot t_{\mathrm{p}}} \tag{15}$$

where G_{p} is the OSB panel sheathing's shear modulus, A_{p} is its cross-section area, and κ is the shear correction factor, i.e. for rectangular cross-sections κ =5/6.

The in-plane displacement from the deformation of the sheathing-to-framing connection $u_{\mathrm{K,an}}$ is determined by (16):

$$u_{\mathrm{K,an}} = \left(\frac{l}{l_{\mathrm{p}}}\right) \cdot \frac{q_{\mathrm{Ed}} \cdot l \cdot a_{\mathrm{v}}}{4 \cdot K_0 \cdot n_{\mathrm{v}} \cdot h} + \left(\frac{l}{h_{\mathrm{p}}}\right) \cdot \frac{q_{\mathrm{Ed}} \cdot l \cdot a_{\mathrm{v}}}{4 \cdot K_{90} \cdot n_{\mathrm{v}} \cdot h} \tag{16}$$

where h_{p} and l_{p} are the height and the length of the single OSB panel ($l_{\mathrm{p}} > h_{\mathrm{p}}$), K_0 and K_{90} are the slip moduli of the connection parallel and perpendicular to the grain, in kN/m, a_{v} and n_{v} are the spacing and the number of rows of the staples. For the slip moduli K_0 and K_{90} between the OSB panels, a distinction is made as to whether the joint is parallel (0°) or perpendicular (90°) to the grain of the timber slab (see (2) and (3)).

4 – NUMERICAL INVESTIGATIONS

4.1 FINITE ELEMENT MODEL

A finite element (FE) model is generated using AXISVM X6 to analyse the shear flexibility of the large-area framed floor diaphragm of the case study (see section 2). In accordance with section 3.1, the framed floor diaphragm of length l of 41.1 m and height h of 50 m consists of three components: 1) the pinned framing members (GL24h, 160 x 360 mm^2), 2) one sheathing (OSB/3, t= 25 mm), and 3) the joints modelling the continuous connection of the sheathing on the framing members and the timber slab without taking contact into account (see Fig. 5a und b).

The edge joint models the stapled connection of the sheathing on the framing members and is defined by the shear stiffness of the edge joint K_{e}, in kN/m/m:

$$K_e = \begin{pmatrix} K_{\mathrm{e,x}} \\ K_{\mathrm{e,y}} \\ K_{\mathrm{e,z}} \end{pmatrix} = \begin{pmatrix} K_{\mathrm{e,x}} \\ K_{\mathrm{e,y}} \\ K_{\mathrm{e,z}} \gg K_{\mathrm{e,x}} \end{pmatrix} \tag{17}$$

with $$K_{\mathrm{e,x}} = \frac{K_0 \cdot n_{\mathrm{v}}}{a_{\mathrm{v}}} \tag{18}$$

$$K_{\mathrm{e,y}} = \frac{K_{90} \cdot n_{\mathrm{v}}}{a_{\mathrm{v}}} = \frac{1}{2} K_{\mathrm{e,x}} \tag{19}$$

where $K_{\mathrm{e,x}}$ and $K_{\mathrm{e,y}}$ are the in-plane properties of the edge joint (the x-axis is parallel to the framing members), and $K_{\mathrm{e,z}}$ is the out-of-plane property of the edge joint (the z-axis is perpendicular to the plane of the sheathing). The torsional stiffness of the joint is disregarded [10]. The joint properties are defined by K_0 and K_{90}, the slip moduli

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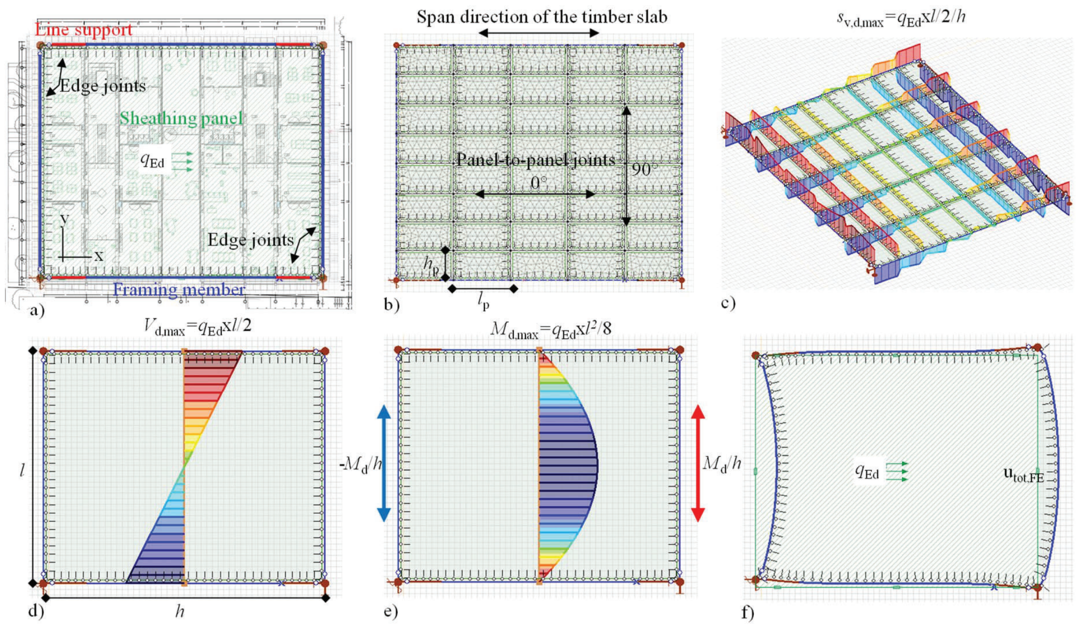

Figure 5. Finite element model of the framed floor diaphragm – a) Elements of the framed floor diaphragm; b) Panel-to-panel joints; c) Shear flow in the edge joints and panel-to-panel joints; d) und e) Internal forces; f) Numerical total in-plane displacement $u_{tot,FE}$.

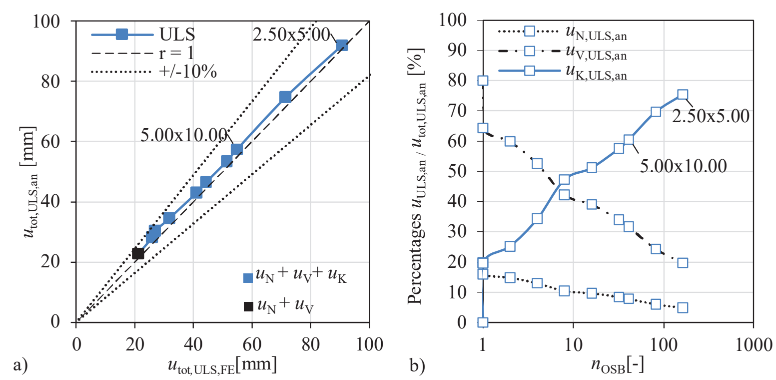

Figure 6. Analytical total in-plane displacement $u_{tot,ULS,an}$ according to (12) for the ultimate limit state design – a) Comparison to the numerical total in-plane displacement $u_{tot,ULS,FE}$; b) Percentages of the in-plane displacements from the axial deformation of the framing members $u_{N,ULS,an}$, from the shear deformation of the sheathing $u_{V,ULS,an}$, and from the deformation of the sheathing-to-framing connection $u_{K,ULS,an}$ depending on the division of the sheathing into individual OSB panels (number of OSB panels n_{OSB} with logarithmic scaling).

of the connection parallel and perpendicular to the grain of the framing members, in kN/m, the spacing of the staples a_v, and the number of rows of the staples n_v.

If the OSB panel sheathing is divided into individual OSB panels, the panel-to-panel joint models the stapled connections of two OSB panels on the timber slab (see Fig. 5b), i.e. the flexibility is doubled. The panel-to-panel joints are defined by half the shear stiffness of the edge joint, either parallel to the timber slab $K_{p,0}$ or perpendicular to the timber slab $K_{p,90}$, in kN/m/m:

$$K_{p,0} = \begin{pmatrix} K_{p,0,x} \\ K_{p,0,y} \\ K_{p,0,z} \end{pmatrix} = \begin{pmatrix} K_{e,x}/2 \\ K_{e,y}/2 \\ K_{e,z} \end{pmatrix} \tag{20}$$

$$K_{p,90} = \begin{pmatrix} K_{p,90,x} \\ K_{p,90,y} \\ K_{p,90,z} \end{pmatrix} = \begin{pmatrix} K_{e,y}/2 \\ K_{e,x}/2 \\ K_{e,z} \end{pmatrix} \tag{21}$$

where $K_{p,i,x}$ and $K_{p,i,y}$ are the in-plane properties of the panel-to-panel joint (the x-axis is parallel to the joint), and $K_{p,z}$ is the out-of-plane property of the panel-to-panel joint (the z-axis is perpendicular to the plane of the sheathing). $K_{e,x}$ and $K_{e,y}$ are taken according to (18) and (19).

The timber slab and the steel trusses are disregarded in the FE model. Line supports in x- and y-direction model the vertical steel cross-bracing elements in the four corners of the framed floor diaphragm. The material behaviour is defined as linear elastic using the stiffness properties of Table 1. The in-plane design load applied to the floor diaphragm q_{Ed} is uniformly distributed over the height h. The sheathing is discretised into a triangular mesh with an average element side length of 1.5 m (see Fig. 5b).

4.2 VERIFICATION

The numerical results of the FE model are shown in Fig. 5c-e. The in-plane design load q_{Ed} generates a shear force distribution V_d and a moment distribution M_d over the length l of the simply supported framed floor diaphragm. The axial force in the framing members results from the moment M_d divided by the height h, with a maximum value at midspan $M_{d,max}/h$. In the FE model, the OSB panel sheathing contributes to the bending stiffness of the framed floor diaphragm ($E_{mean} > 0$, see Table 1). The contribution is higher, when the sheathing is not divided into single OSB panels but defined as one single OSB panel. In that case, the maximum axial force in the framing members is slightly smaller. The shear force V_d is introduced from the sheathing via the connections into the framing members and the timber slab and generates the shear flow $s_{v,d}$. The maximum shear flow $s_{v,d,max}$ results at the supports. At midspan, the total in-plane displacement $u_{tot,FE}$ is generated (see Fig. 5f).

4.3 VALIDATION

For comparison with the analytical design method, the total in-plane displacement is investigated for the ULS design. The in-plane design load q_{Ed} is defined as the internal instability load $q_{Ed,ULS}$ (see section 3.2). It is determined with $q_{Ed,ULS} = 56.40$ kN/m based on the combination of actions for persistent and transient design situation with the leading variable action of 3.00 kN/m^2 (category C1) according to SIA 260 [11]. The internal stability load $q_{Ed,ULS}$ is evenly distributed over the height h ($q_{Ed,ULS} = 1.13$ kN/m^2). The design values of the stiffness properties are taken for the ULS from Table 1.

The total in-plane displacement $u_{tot,ULS,FE}$ was taken from the FE model. Fig. 6a shows the results of the comparison between the FE and the analytical model for the total in-plane displacement $u_{tot,ULS,an}$ according to (12). In the analytical and the FE model, the division of the OSB panel sheathing into individual OSB panels is extended step by step. The number of OSB panels n_{OSB} is therefore increased step by step. First, the sheathing is defined as one single OSB panel disregarding the shear flexibility of the edge joints but modelling them as rigid in shear. In the next step, the shear flexibility of the edge joints is considered based on the slip modulus of the connection parallel to the grain $K_{d,0}$, in kN/m, and the spacing and the number of rows of the staples a_v and n_v. Then, the number of the panel-to-panel joints is increased by halving stepwise the sheathing first in width and then in length (see Table 2). At the end, the sheathing is divided into OSB panels of the sizes of 5.00 x 10.00, 5.00 x 5.00, and 2.50 x 5.00 m^2 (h_p x l_p, defined as height and length of the OSB panels, with $h_p < l_p$).

Fig. 6a shows a perfect correlation with a coefficient value of 1.00 between the analytical and the FE models. The analytical design method provides results on the safe side and within an error corridor of 10%. The difference between the analytical and numerical total in-plane displacements is expressed by the root mean square error (RMSE) and results in 2.4 mm. This agrees with the mean absolute error (MAE) of 2.3 mm. The small difference between RMSE and MAE illustrates that all errors are of similar magnitude.

4.4 INFLUENCE OF OPENINGS

After validation of the FE model, the influence of the big openings of the staircases is investigated. The openings of 13.80 x 7.70 m^2 and 2 x 6.90 x 5.50 m^2 are included in the FE model (see Fig. 7). The openings are framed with pinned framing members (GL24h, 160 x 360 mm^2). Edge joints model the stapled connection of the sheathing on the framing members according to (17).

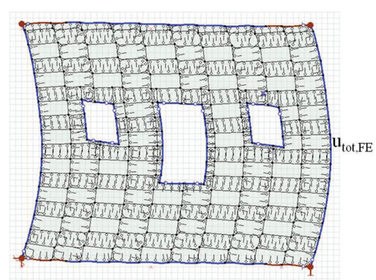

Figure 7. Deformed Finite element model of the framed floor diaphragm including the openings of the staircases.

Without openings, the numerical in-plane displacement reaches a value of 90.8 mm for OSB panels of the size of 2.50 x 5.00 m^2 (see Fig. 6a). With openings, this results in 93.3 mm. The influence on the total in-plane displacement is less than 5%, as the openings are located close to the shear zero point of the shear flow (see Fig. 5c). For other arrangements of openings closer to the shear flow maxima, the influence might be higher.

5 – PARAMETER STUDY

5.1 NUMBER OF JOINTS

Fig. 6b shows the percentages of the individual in-plane displacements in the analytical total in-plane displacement $u_{tot,ULS,an}$ with an increased number of OSB panels n_{OSB}, i.e. an increased number of panel-to-panel joints. The results are shown with logarithmic scaling.

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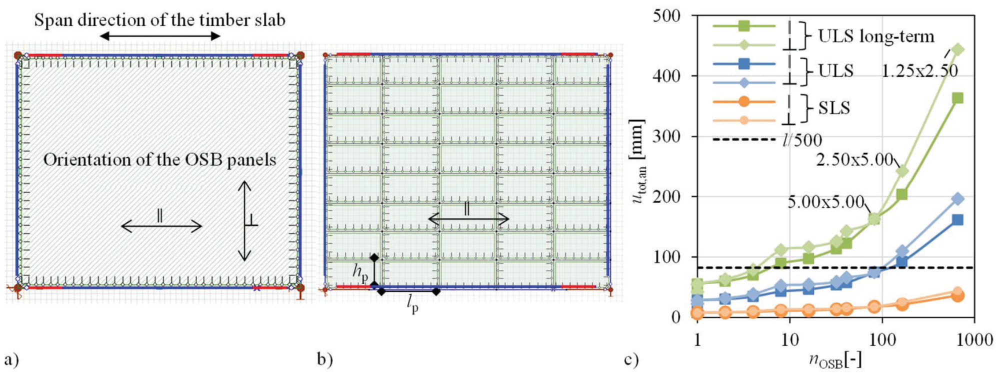

Figure 8. Analytical total in-plane displacement $u_{tot,ULS,an}$ according to (12) depending on the limit state design (SLS, ULS, ULS long-term) – a) Orientation of the OSB panels (parallel or perpendicular to the timber slab); b) Division of the sheathing into single OSB panels oriented parallel to the timber slab with the height h_p and the length l_p ($h_p < l_p$); c) Total in-plane displacements depending on the division of the sheathing into individual OSB panels (number of OSB panels n_{OSB} with logarithmic scaling).

Table 2. Analytical total in-plane displacements $u_{tot,SLS,an}$, $u_{tot,ULS,an}$ and $u_{tot,ULS,fin,an}$ according to (12) depending on the height and length of the single OSB panel (hp < lp), the number of OSB panels n_{OSB}, and the orientation of the OSB panels (parallel or perpendicular to the timber slab, see Fig. 6a). The results exceeding the limit value of l/500 are highlighted in bold.

Orientation of the OSB panels				Parallel (‖)			Perpendicular (⊥)		
	h_p [m]	l_p [m]	n_{OSB} [-]	$u_{tot,SLS,an}$ [mm]	$u_{tot,ULS,an}$ [mm]	$u_{tot,ULS,fin,an}$ [mm]	$u_{tot,SLS,an}$ [mm]	$u_{tot,ULS,an}$ [mm]	$u_{tot,ULS,fin,an}$ [mm]
Rigid joints	41.10	50.00	1	7.2	22.7	42.1	7.2	22.7	42.1
Shear-flexible joints based on the slip modulus of Table 1	41.10	50.00	1	8.5	28.3	55.0	8.5	28.3	55.0
	20.60	50.00	2	9.0	30.4	59.9	9.4	31.7	63.0
	10.30	50.00	4	10.0	34.6	69.6	11.0	38.7	79.1
	5.10	50.00	8	12.0	43.0	**89.2**	14.3	52.5	**111.2**
	5.10	25.00	16	12.9	46.5	**97.2**	14.8	54.6	**116.1**
	5.10	12.50	32	14.5	53.4	**113.3**	15.8	58.9	**125.9**
	5.00	10.00	41	15.4	57.4	**122.4**	17.5	66.0	**142.5**
	5.00	5.00	82	19.5	74.7	**162.6**	19.5	74.7	**162.6**
	2.50	5.00	164	23.6	**92.0**	**202.8**	27.7	**109.4**	**243.0**
	1.25	2.50	658	40.0	**161.4**	**363.6**	48.2	**196.1**	**443.9**

The percentage of the in-plane displacement from the axial deformation of the framing members $u_{N,ULS,an}$ decreases to 5% and from the shear deformation of the sheathing $u_{V,ULS,an}$ to 20%, while the percentage of the in-plane displacement from the deformation of the sheathing-to-framing connection $u_{K,ULS,an}$ increases up to 75% by reducing the dimensions of the OSB panels.

The analytical design method neglects the contribution of the sheathing to the effective bending stiffness. However, the influence of the sheathing on the effective bending stiffness decreases with an increased division of the sheathing into single OSB panels, i.e. with an increased flexibility of the sheathing. In practical examples for OSB panels of the size of 2.50 x 5.00 m² or smaller, the contribution of the sheathing to the bending stiffness is negligible for this size of floor plan. The decisive in-plane displacement results from the deformation of the sheathing-to-framing connection $u_{K,ULS}$.

5.2 THE LIMIT STATE DESIGN

For the ULS design, the total in-plane displacement reaches values of more than 50 mm and exceed with OSB panels of the size of 2.50 x 5.00 m² the limit value of 82.2 mm (see section 3.2). Fig. 8c shows the analytical total in-plane displacement $u_{tot,an}$ as a function of the number of OSB panels n_{OSB} for the three limit state designs (SLS, ULS, ULS long-term). The results are shown with logarithmic scaling. Furthermore, the limit value $l/500$ for the in-plane displacement of 82.2 mm is included. The sheathing is divided into OSB panels of 1.25 x 2.50 m². The stiffness properties for the different limit states are taken from Table 1. The internal instability load for SLS $q_{Ed,SLS}$ is defined by dividing the internal instability load for ULS $q_{Ed,ULS}$ by the factor of 1.8. The internal instability load for ULS with long-term behaviour $q_{Ed,ULS,fin}$ is defined as the quasi-permanent part, i.e. 80% of $q_{Ed,ULS}$. Table 2 lists the analytical results.

The SLS results remain below the limit value of 82.2 mm. Due to the 1.8 times higher in-plane design load and the reduction of the stiffness values according to (4)-(7) (see Table 1), the ULS results increase by a factor of 4. The use of commonly used OSB sizes such as 2.50 x 5.00 or 1.25 x 2.50 m^2 exceeds the limit value. Taking long-term behaviour into account, the stiffness properties of the ULS design are decreased by a factor of (1+k_{def}) according to (8)-(10) (see Table 1). The ULS results for long-term behavior increase further by a factor of 2. Consequently, the results already exceed the limit value when defining the first perpendicular joint (n_{OSB} = 8, Table 2).

5.3 ORIENTATION OF THE OSB PANELS

In the case study, the orientation of the OSB panels is defined as parallel to the span direction of the timber slab (‖) (see Fig. 8b). The influence is investigated when the OSB panels are oriented either parallel or perpendicular to the span direction of the timber slab (‖ or ┴, see Fig. 8a). When oriented perpendicular to the timber slab (┴), the number of panel-to-panel joints perpendicular to the timber slab (90°, see Fig. 5b) increases. Since the decisive in-plane displacement results from the deformation of the sheathing-to-framing connection $u_{K,an}$, the total in-plane displacement $u_{tot,an}$ increases, as well (see Fig. 8c). For all limit state designs, the highest increase of 20% results when starting to divide the sheathing into perpendicular joints (n_{OSB} = 8, Table 2). When dividing the sheathing into theoretically squared OSB panels of the size of 5.00 x 5.00 m^2, the results are identical between parallel (‖) and perpendicular (┴).

5.4 NUMBER OF SHEATHINGS

A measure to increase the in-plane stiffness of the framed floor diaphragm is to sheath the timber slabs with two layers of OSB panels. By placing the second sheathing at the bottom of the hollow-core timber slab, the two-sided sheathing would result to a doubled in-plane stiffness. Since the bottom side of the timber slab is intended to have a high-quality wood appearance, it could only be reinforced on the top side.

The FE model is extended by adding a second layer of sheathing on top of the first sheathing with t_2 = 25 mm for the ULS design. The idea is to shift the panel-to-panel joints of the second sheathing by 0.5 x l_p and 0.5 x h_p, which leads to an offset between the panel-to-panel joints of the first and the second sheathings (see Fig. 9a und b). The sheathings are divided into thirds in length and halved in height. The in-plane design load applied to the floor diaphragm $q_{Ed,ULS}$ is uniformly distributed over the two sheathings. The second sheathing is framed with line-to-line connecting elements modelling the stapled connection of the second sheathing on the framing members and is defined by the shear stiffness of the edge joint according to (17). Besides these connecting elements, the two sheathings are not in contact. The panel-to-panel joint of the second sheathing models the stapled connection of two OSB panels on the first OSB panel sheathing and is defined by the shear stiffness $K_{p,2}$, in kN/m/m:

$$K_{p,2} = \begin{pmatrix} K_{p,2,x} \\ K_{p,2,y} \\ K_{p,2,z} \end{pmatrix} = \begin{pmatrix} K_{e,x}/2 \\ K_{e,x}/2 \\ K_{e,z} \gg K_{e,x} \end{pmatrix} \quad (22)$$

where $K_{p,2,x}$ and $K_{p,2,y}$ are the in-plane properties of the panel-to-panel joint (the x-axis is parallel to the joint), and $K_{p,z}$ is the out-of-plane property of the panel-to-panel joint (the z-axis is perpendicular to the plane of the sheathing). $K_{e,x}$ is taken according to (18). The design values of the stiffness properties are taken for the ULS from Table 1.

For validation, an equivalent one sheathed system with t_{eq} = 50 mm is modelled, in parallel (see Fig. 9c). The edge joints of the equivalent system are defined by the shear stiffness of the edge joint $K_{e,eq}$, in kN/m/m:

$$K_{e,eq} = \begin{pmatrix} K_{e,eq,x} \\ K_{e,eq,y} \\ K_{e,eq,z} \end{pmatrix} = \begin{pmatrix} 2 \cdot K_{e,x} \\ 2 \cdot K_{e,y} \\ K_{e,eq,z} = K_{e,z} \end{pmatrix} \quad (23)$$

where $K_{e,x}$, $K_{e,y}$ and $K_{e,z}$ are taken according to (17). The total in-plane displacements of both system, $u_{tot,ULS,FE}$ and $u_{tot,ULS,FE,eq}$, are compared with each other.

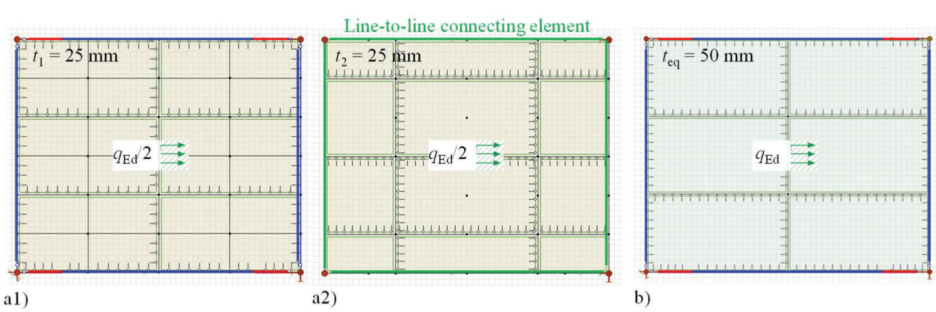

Figure 9. Finite element model of the framed floor diaphragm – With a 2-layer sheathing a1) First sheathing t_1 = 25 mm and a2) Second sheathing t_2 = 25 mm; b) Equivalent system with a 1-layer sheathing t = 50 mm.

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The shear stiffness of the panel-to-panel joints of the equivalent system are calibrated to comply with the in-plane stiffness of the two sheathed system. The shear stiffness of the equivalent panel-to-panel joints parallel to the timber slab $K_{p,0,eq}$ or perpendicular to the timber slab $K_{p,90,eq}$, in kN/m/m, are calibrated to:

$$K_{p,0,\mathrm{eq}} = K_{p,0} + K_{p,2} = \begin{pmatrix} \mathbf{2} \cdot K_{\mathrm{e,x}}/2 \\ \mathbf{1.5} \cdot K_{\mathrm{e,y}}/2 \\ K_{\mathrm{e,z}} \end{pmatrix} \quad (24)$$

$$K_{p,90,\mathrm{eq}} = K_{p,90} + K_{p,2} = \begin{pmatrix} \mathbf{1.5} \cdot K_{\mathrm{e,y}}/2 \\ \mathbf{2} \cdot K_{\mathrm{e,x}}/2 \\ K_{\mathrm{e,z}} \end{pmatrix} \quad (25)$$

where $K_{e,x}$, $K_{e,y}$ and $K_{e,z}$ are taken according to (17).

The two sheathed framed floor diaphragm results in a mean value of the total in-plane displacements of the two sheathings of 20 mm while the equivalent system results in 19.6 mm. The difference is less than 2%. With increase of 1.7 times, a one sheathed system with $t_{eq} = 25$ mm would result in 33.2 mm.

6 – CONCLUSIONS

Based on the *shear field beam theory*, the analytical results are compared with the results of the FE model of a simply supported structure. Equation (12) provides very good approximations of the total in-plane displacement of the case study's large-area framed floor diaphragm of over 2000 m^2 with vertical steel cross-bracing elements in the four corners. For OSB panels of the size of 2.50 x 5.00 or 1.25 x 2.50 m^2, the total in-place displacements exceed the limit value of l/500 in the ULS design. The in-plane flexibility of the framed floor diaphragm is too high. The internal stability load $q_{Ed,ULS}$ would results in smaller values when determined according to the Swiss design standard for steel structures [12]. However, even with 2-layer OSB panel sheathing, the criteria could not be met. Therefore, the authors decided to use a horizontal steel truss system, integrated in the hollow core timber slab, to brace the steel trusses. This decision also simplifies the cooperation between the steel and timber construction companies.

The following conclusion can be drawn from the parameter study for future projects:

- The analytical design method can be used to fulfill wind requirements in the SLS. Depending on the building class, e.g. of hospitals with a high infrastructure function, the criteria in the SLS need to be fulfilled for earthquake loads.
- The total in-plane displacement increases significantly in the ULS and even more when including long-term behavior.
- The decisive in-plane displacement results from the deformation of the sheathing-to-framing connection. The total in-plane displacement is increased by each joint. Therefore, the size of the OSB panels should be chosen as big as possible.
- The joints perpendicular to the grain of the timber slab have a higher influence on the in-plane stiffness than joints parallel to the grain. The OSB panels should be oriented parallel to the span direction of the timber slab.
- Depending on their size, openings located at the shear zero point might be negligible. For large openings, further investigations are required.
- A 2-layer OSB panel sheathing on top of the framed floor diaphragm increases the in-plane stiffness by over 1.5 times.

Further measures can be implemented to increase the in-plane stiffness. One example could be modeling a staggered OSB panel arrangement, considering the contact surfaces between the OSB panels.

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