Experimental testing and analytical modelling of Skylark walls: external faces subjected to wind pressure

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Abstract

In the first part of this paper, the main results from a series of experiments focused on Skylark walls are presented. Three specimens were tested to evaluate the capacity of the external faces under uniform pressure. Specimens showed two main failure modes: 1) local bending, which is due to experimental constraints, and 2) failure of the tabs connecting the external face to the lateral panels. In the second part, an analytical formula is developed to calculate the maximum allowable pressure based on the observed failure modes.

1 Experimental testing

1.1 Experimental setup

Three column specimens were tested (Figure 1) to investigate their capacity to sustain wind withdrawing forces. Specimens comprise 3 mock columns made of plywood (Metsa plywood), assembled by using the following panels: 1) Front panel, i.e. the panel which is subjected to withdrawing wind in a real scenario 2) Lateral panels, which for testing purposes are fixed to the supporting steel plates 3) Stabilizing panels, which are introduced to replicate the stiffening effect of the second flange of the column (in this case removed to apply the load with the actuator).

The main panels are 1200 mm long, and 600 mm wide. They are connected to the side panels by 4 tabs (2 on each side), which are spaced 600 mm from each other. The load exerted by the hydraulic actuator was applied to the front panel by using a 800 x 500 x 10 mm steel plate (Figure 2).

The following instrumentation was installed for data acquisition:

- Four linear transducers (potentiometers) were used to track the panel vertical displacement. They measured the displacement between the front panel and the side panels.
- A 250 kN capacity load cell was used to measure the compression force. The load cell was mounted between the actuator and the steel plate.

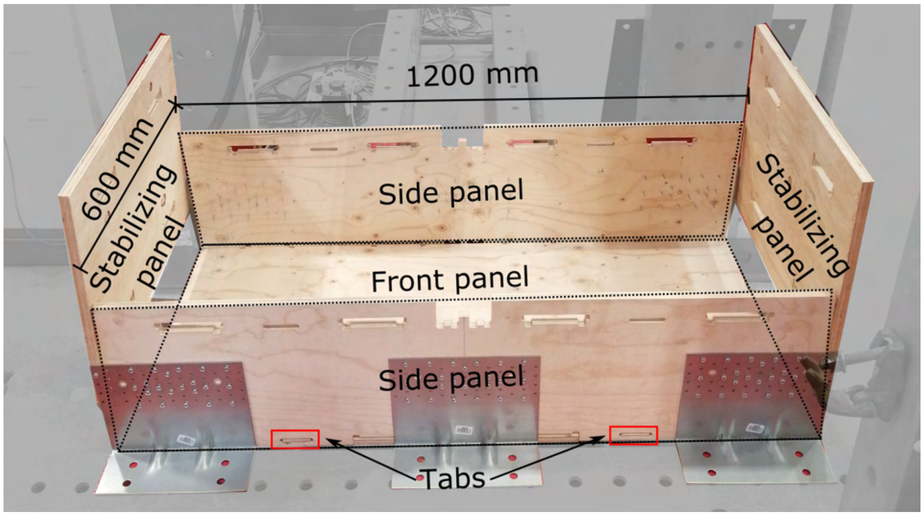

Figure 1: Geometry of the specimens.

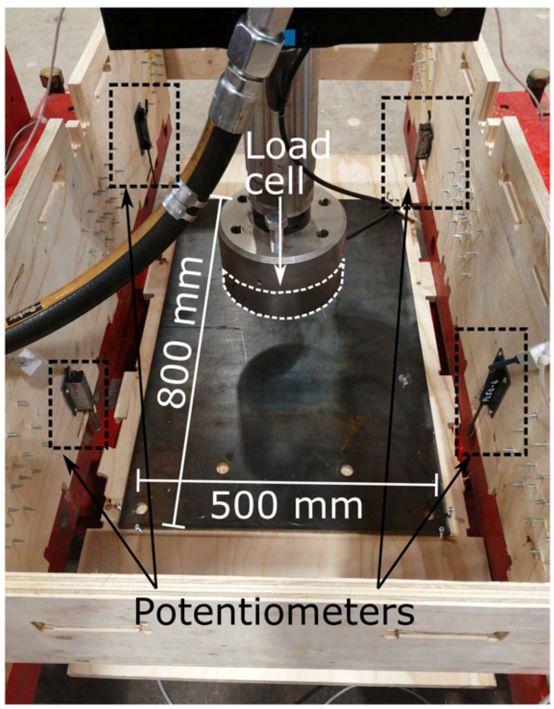

Figure 2: Experimental setup.

The loading protocol consisted of a monotonic displacement increasing until failure.

The loading rate was equivalent to 1 mm per minute.

1.2 Failure mechanism

In all 3 specimens the failure was initiated by a bending failure near the steel plate (Figure 3a,b and c). This is believed to be due to the particular experimental setup. Since the steel plate is significantly stiffer than the timber flange, the force could not be equally distributed on this flange. This means that the force distribution does not accurately represent what would occur in the case of wind pressure.

Specimen 1 (Figure 3d and e) failure of the tabs in shear and 2) failure of the region surrounding the tabs. These failure modes are believed to be possible under wind pressure loads.

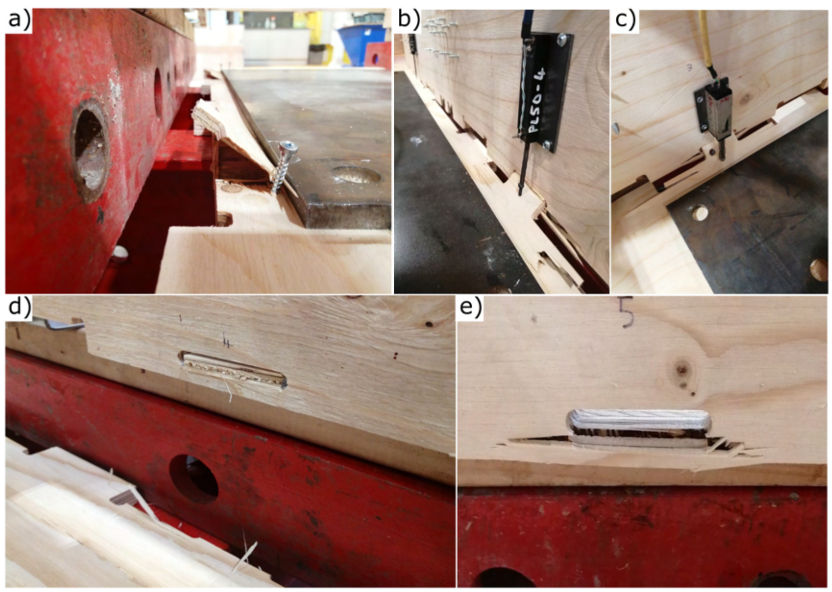

Figure 3: Experimental setup.

1.3 Failure load

The response of the specimens in terms of pressure (exerted by the actuator) vs vertical displacement is reported in Figure 4. Note that the vertical displacement was taken as the average between the measurements of the 4 linear transducers. The pressure on the specimen was taken as the force exerted by the actuator divided by the area of the front panel, i.e., 0.72 m ^{2}. Specimens failed in between 2.5 kPa and 3.2 kPa.

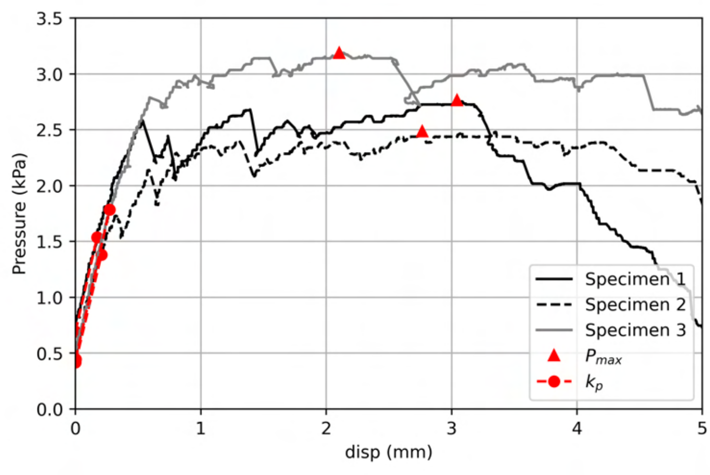

Figure 4: Pressure vs displacement response of the specimens.

Table 1: Experimental results of the tested specimens.

Specimen	P_{max} **(kPa)**	k_p **(kPa/mm)**
S1	2.76	4.88
S2	2.49	4.67
S3	3.19	4.93

2 Analytical model

A set of design equations is provided to calculate the observed failure modes occurring in the tabs (Figure 5).

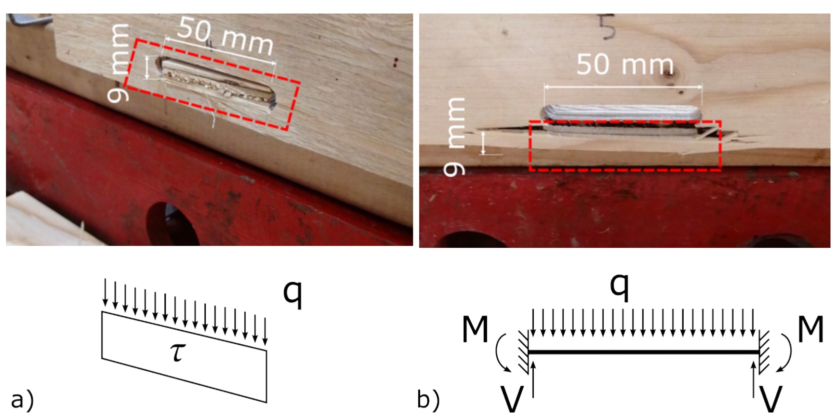

Figure 5: Failure modes at the base of the analytical model.

2.1 Shear failure of the tab

The shear capacity $F_{s,1}$ of the tab (Figure 5a) can be calculated as:

$$F_{s,1} = f_s \cdot A_{s,1} \tag{1}$$

where $A_{s,1} = 50 \cdot 9 = 450\ mm^2$ is the shear area of the tab, and f_s is the shear strength of the material.

By introducing a linear load q acting above the tab, and by substituting the geometrical properties, Equation 1 becomes:

$$q = F_{s,1}/l \tag{2}$$

where l is the length of the tab equal to 50 mm.

2.2 Failure of the strip below the tab

The strip of timber below the tab (Figure 5) can be thought as a fixed-fixed beam subjected to distributed load q. Supports are subjected to both shear V and moment M. Its shear capacity $F_{s,2}$ is equal to:

$$F_{s,2} = f_s \cdot A_{s,2} \tag{3}$$

where $A_{s,2} = 9 \cdot 18 = 162\ mm^2$ is the shear area of the strip.

The moment capacity M_2 is equal to:

$$M_2 = W \cdot f_{t,0} \tag{4}$$

where $W = 18 \cdot 9^2/6 = 243\ mm^3$ is the section modulus of the strip, and $f_{t,0}$ is the the tensile capacity of the material parallel to the grain.

Following the Eurocode 5 [1] recommendations, the combined shear-flexural failure occurs if:

$$\frac{V}{F_{s,2}} + \frac{M}{M_2} = 1 \tag{5}$$

By substituting the distributed load q into equation 5:

$$\frac{ql}{2A_{s,2}f_s} + \frac{ql^2}{12Wf_{t,0}} = 1 \Rightarrow q = \frac{1}{\frac{l}{2A_{s,2}f_s} + \frac{l^2}{12Wf_{t,0}}} \tag{6}$$

2.3 Tributary area and final formula

In the current geometry of walls (Figure 6), the tributary area A_t for each couple of tabs is equal to $A_t = 600 \cdot 564 = 338400\ mm^2$.

Considering a wind pressure ρ acting on such a tributary area, the distributed load q on a single tab is equal to:

$$q = A_t\rho/2l \tag{7}$$

which can be re-written as:

$$\rho = 2ql/A_t \tag{8}$$

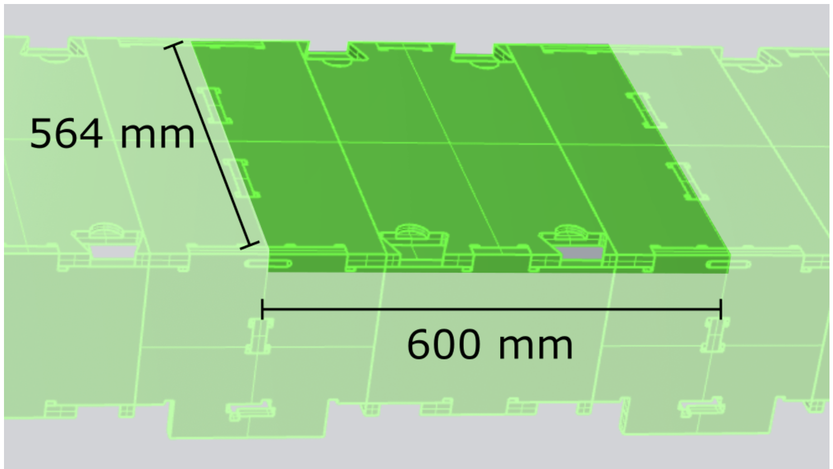

Figure 6: Tributary area for each couple of tab.

2.4 Pressure capacity

By considering both failure mechanisms possible, the pressure capacity ρ_{rd} of the wall cassettes is equal to the minimum failure load between them:

$$\rho_{rd} = \min \begin{cases} \frac{2A_{s,1}f_s}{A_t} \\ \frac{2}{\frac{A_t}{2A_{s,2}f_s} + \frac{l \cdot A_t}{12Wf_{t,0}}} \end{cases} \tag{9}$$

By substituting the actual geometrical properties into equation 9, it leads to:

$$\rho_{rd} = \min \begin{cases} \frac{f_s}{376} \\ \frac{1}{\frac{4700}{9f_s} + \frac{235000}{81f_{t,0}}} \end{cases} \tag{10}$$

Acknowledgements

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References

[1] Normalisation Comite Europeen de. *Eurocode 5 - Design of Timber Structures. Part1-1: General rules and rules for buildings*. Brussels, Belgium, 2004.