Calibrating structural modelling simulation parameters of a lightweight temporary shelter using a lateral load test *in situ*

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Abstract

The importance of temporary housing facilities has been recently highlighted due to the occurrence of migrant flows, agricultural workers, and, more recently, the need for 'social distancing' strategies has become crucial to limiting the spread of the coronavirus 2019 (COVID-19) disease. They are built with different shapes, technology, structural and material systems. The structural system is often very simple because the module must be constructed in a short time by a few people. They have guaranteed the safety and well-being of the occupants and have to be designed in accordance with the rules and approved building codes. For these reasons, it is very important to design and verify the structural system with a high level of accuracy using a model and reliable structural analysis methods. Furthermore, it is essential to test the actual behaviour of the structure in use to validate the structural model simulated with the behaviour in situ. In this paper, we have illustrated a simple original test in situ to analyse the behaviour and survey the displacements of the shear wall prototype of a temporary home module in cork and timber loaded with a horizontal force. The comparison between the measured and the calculated displacements by means of finite element model software led to the evaluation of the accuracy of the structural model and the more realistic value of the connection's metal stiffness. A specific numerical function was obtained using a rational regression interpolation that relates the connections' stiffness value to the horizontal force. Knowing the actual value of the connection stiffness leads to a more reliable and safe design.

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Introduction

In recent years demand for lightweight temporary shelters has grown. They have been widely used in post-disaster recovery, earthquakes and hurricanes, as emergency housing for industrial and agricultural workers (Mills-Tettey, 1989; Moran *et al*., 2021), and low-income groups but also for ecotourism (Radogna, 2018). The importance of emergency housing facilities in Europe has been recently highlighted due to the occurrence of migrant flows, and more recently, the need for 'social distancing' strategies has become crucial to limiting the spread of the coronavirus 2019 (COVID-19) disease. In addition to quarantine and isolation procedures for those exposed to or infected with COVID-19, social distancing has been enforced amongst the general population to reduce the transmission of COVID-19 (Klochko, 2022).

In this context, providing new temporary accommodation to limit social contact or protect vulnerable people, such as the homeless or migrants, has become a critical issue in managing the pandemic emergency phases. These modules are lightweight structures that can be used for several purposes; they are designed and planned so that they can be erected, dismantled, upgraded, reused, relocated, and recycled in different configurations for various functions. After being used, they can be re-used for the same or new function (Arslan, 2007) or stored for future use. They are built with different shapes, technology, structural and material systems (Dev and Das, 2021). The structural system is often very simple because the module must be constructed in a short time by a few people. They are often built with low-cost materials to control expense. They guarantee the safety and well-being of the occupants (Barreca and Tirella, 2017) and have to be designed in accordance with the rules and approved building codes. For these reasons, it is imperative to design and verify the structural system with a high level of accuracy using a model and reliable structural analysis methods. The structural analysis software based on the finite element model (FEM) method is very reliable and can simulate the behaviour of more complicated and varied structures stressed by the load with a high degree of accuracy (Coemert *et al*., 2021). The precision does not depend only on the structural geometric model but also on the knowledge of the mechanical properties of utilised materials (Molari *et al.*, 2020). It is important to test the real behaviour of the structure in use in order to validate the structural model simulated with the behaviour in situ (Bian *et al.*, 2022). Using this comparison, it is possible to obtain important information necessary to improve the structural analysis model. In this paper, we have illustrated a simple original test *in situ* to analyse the behaviour of the shear wall prototype of a temporary home module in cork and timber loaded with a horizontal force. The value of geometric deformation caused by different horizontal force values was surveyed by a laser scanner. This allowed the analysis and evaluation of some mechanical properties of the shear wall prototype to improve the structural model and the behaviour simulation of the whole temporary building module, which was stressed by live and environmental loads using structural analysis software models.

The structural test facilities are equipped with specific testing machines to evaluate the deformation of timber structures, with multi-axial loads applied with hydraulic actuators and high-speed displacement and strain data acquisition systems (Granello *et al.,* 2022).

The test in situ is very important because it allows the behaviour of the structure in real-use conditions to be known. Although the procedure and the method are not simple and easy to apply and, to the authors' knowledge, no specific standard covers this testing type. Nevertheless, in situ, it is challenging to simulate the environmental loads on the structure and to monitor and record the deformations. For this reason, a good compromise could be to test a significant component of the structure and extend the results obtained to the whole. In this study, the main mechanical characteristics of the timber module structure were determined by simulating the wind action on a prototype of the shear wall of the module. The real deformation of the prototype, surveyed by a terrestrial laser scanner (TLS), was compared with the surface created by the structural model (Maiellaro *et al.,* 2015). The comparison of the points cloud of the deformation surveyed by TLS with the calculated deformation surface was made using specific software, allowing the structural model to be calibrated.

Materials and methods

The structure of the lightweight temporary shelter was designed as a succession of timber portal frames (Figure 1A) composed of spruce boards hinged together. They are made of 3 cm-thick and 16-cm-wide spruce boards with columns (uprights) (Figure 1B) of the same size as the horizontal beams in the roofing and the floor. This system allows for the complete interchangeability of the elements and flexibility and modularity of the structure whilst containing the production costs. The wooden structure has a 3-cm×16-cm T-shaped cross-section. The shear walls of the module are made of an OSB panel 1.5 cm thick. The panels are fixed to the columns by metal connections on the corners, designed to allow the fast and easy mounting of the wall (Figure 1C). Each OSB panel has a dimension of about 90×90 cm and four horizontal slat of wood 3×3 cm to stiffen the plane, two cork insulation and waterproofing layer (Barreca *et al.*, 2021) of 6 cm without structural function are applied on both sides (Barreca *et al.*, 2018).

The modular shelter has approximate dimensions of 5×3 m and comprises 6 timber portal frames distanced about 90 cm. The moment-resisting knee portal joint is designed using 4 bolts (grade 8.8) of a diameter of 8 mm. An innovative connection system (has been designed in order to join the panels to the columns in a fast and safe way (Figure 1D). The connector is a metallic plate 2 mm

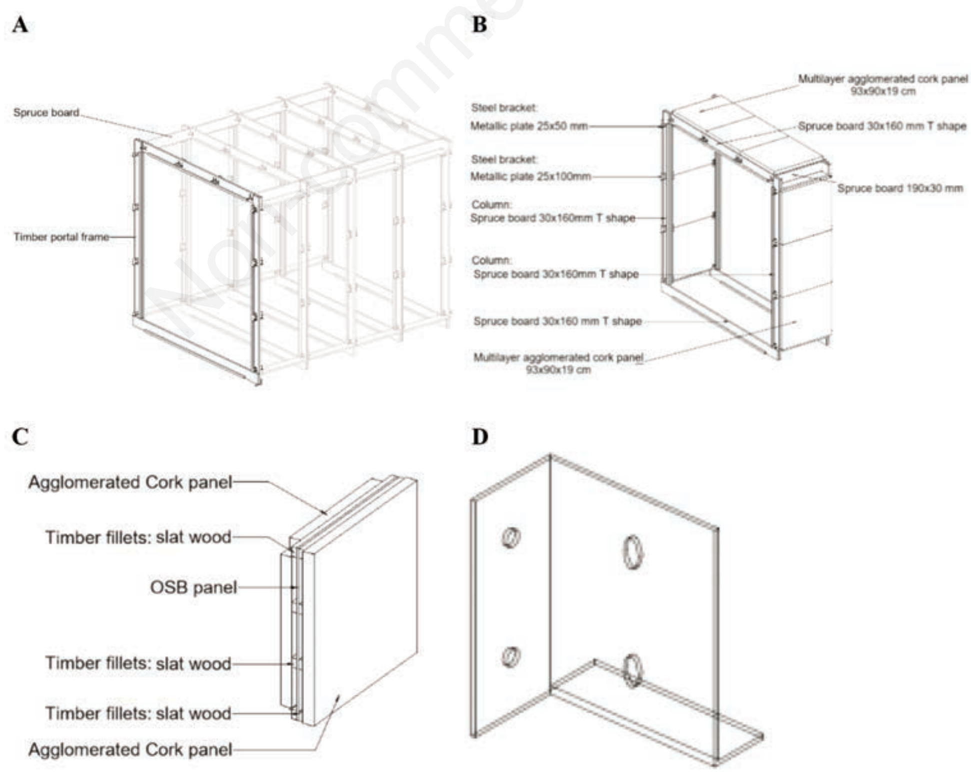

Figure 1. A) Structure of the lightweight temporary shelter; B) timber portal frames with sandwich panels in cork and timber; C) multilayer agglomerated Cork panel; D) metallic plate (steel bracket) to fix the panels to the timber column frame.

thick with two wings (steel bracket). One wing is fixed to the vertical column by two bolts, whereas the other one is a useful support and track for the panel while being fixed to the plate. These connectors created a restraint for the panels, with a grade between the pinned and the fixed support. To stiffen the OSB panel, 4 timber fillets were applied to the sides and 2 to the border. Each frame is fixed to the ground by a couple of screw foundations (Güralp, 2000), which creates a very practical and quick system to establish a solid base to support the timber structure and the loads on it. This foundation solution is easy to install, remove, and re-use and represents a valid alternative to fixed foundations, such as concrete ones. This foundation does not require the ground to be levelled. It does, however, raise the floor above the ground, and in this way, the rainwater flows below the structure.

Finite element model analysis of the structure

The 3D analysis of a complete structure can be highly complex (Harvey and Hubert, 2022). These difficulties are not linked to the geometric dimension, but they are caused by many elements, including individual fasteners, and by the behaviour simulation of the elements' junctions and the supports. In this study, the structural analysis of the lightweight temporary shelter was carried out using SISMICAD, which is an industrial structural simulation software based on the finite element model (Croce *et al*., 2022) (Figure 2). The lightweight temporary shelter has a length of 5 m, a width of 3 m and a height of 3 m. In order to move closer to the theoretical design values of Eurocode 5, the portal frame structure has been developed, considering timber to be an elastic orthotropic material. However, structuring the wall panel with the precise formulation of connections between the timber frame and the OSB panels of sheathing was particularly difficult, introducing a large number of unknowns into the numerical analysis. Therefore, the process of modelling became complicated. The joints of the OSB panels are in the corners, and the border timber fillets are attached to the timber portal steel brackets through a bolt (grade 8.8) of 8 mm diameter. The stiffness of the connection was simulated by a semi-rigid reaction of the moment-resisting connection employing a spring which replicated the elasticity in terms of force per elongation. The structural analysis process was carried out by simulating the pressures that could act on its during its lifetime.

In particular, the snow load on the roof (60 daN·m^2) in accordance with Italian Standard (NTC 2018) EN 1991-1-3 (Pietro Croce *et al*., 2018), the live load on the floor (200 daN·m^2) under EN 1991-1-1, and the wind horizontal load on the walls following EN 1991-1-4 (Ministero delle Infrastrutture e dei Trasporti 2018) were considered. A wind velocity of 51 m·s^{-1} corresponding to the F2 of the enhanced Fujita Scale (EF Scale) of tornado damage intensity (Doswell, Brooks, and Dotzek, 2009) was used to simulate emergency conditions. The F2 represents the damage caused by a strong tornado, such as roofs torn from frame houses; mobile homes demolished; boxcars pushed over; large trees snapped or uprooted. The wind pressure on the walls and the stress on the module were simulated from two directions, X and Y. The seismic forces in directions X and Y were calculated with reference to a horizontal peak ground acceleration equal to a_g=3.065 m·sec^{-2}, with a probability of exceedance in 50 years, equal to 5% (Table 1). The wind forces on the structure are higher than the seismic forces because the module has a low weight (Table 1).

For this reason, the wind pressure was the only horizontal load considered in the structural analysis. The investigations of the structural load path and system behaviour of light-frame (LF) wooden structural systems highlighted the poor performance of these buildings when subject to extreme weather events (such as hurricanes and tornadoes), whereas, due to their light weight, these buildings are resistant to earthquakes (Malone *et al.,* 2014). The shear walls are the timber structural components, lateral to the portal frames, which are more stressed by horizontal forces, and the horizontal load resistance of the building depends on them.

Analysis of the shear wall deformations

The mechanical behaviour of a timber shear wall, strained by loads, can be simulated through complex FEM with a fine analytical resolution (Figure 3B). However, a complete model is characterised by a large number of degrees of freedom, in addition to the mechanical and geometric structural characteristics, the reliability of the results depends on the right restraints and connections simulated. The stiffness of the connections of the OSB panels with the lateral portal columns was simulated in the FEM structural model using spring elements in the corners of the panel. The value of the rigidity of the springs [K_s] for the setup of FEM software to simulate in the structural model the joint slip of the metal bracket between the wood columns and the OSB panels, following section 7 of the Eurocode 5 (Fonseca *et al*., 2022). The Ks value is known for the common metal connections but has to be evaluated when a new connection is used. In this study, a specific experimental test was conducted to evaluate the stiffness of the designed steel bracket connections for the shear walls of the module.

Set up of the *in situ* experiment

A scale 1:1 specimen was installed on the soil ground by means of fixed supports to evaluate the deformation of the shear wall in use. The specimen was loaded with a horizontal force applied on the top utilising a steel cable (⌀2 cm) linked to a winch

Table 1. Horizontal load on the structure.

Load [daN]	X	Y
Wind [daN]	5069.352	7529.533
Seismic [daN]	2498.506	3012.055

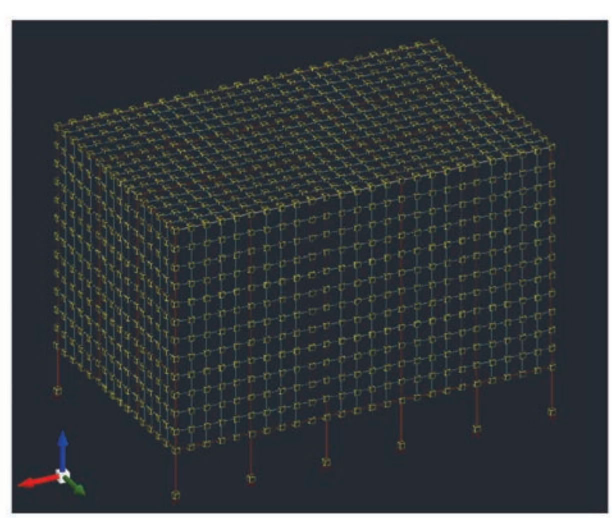

Figure 2. Finite element model of the timber module.

with a digital load cell. Vertical loads were not applied to generate exclusively shear stress. The load was divided between the two timber columns in an equal measure using a rigid bracket. A terrestrial laser scanner (TLS), Leica C10, was positioned a 2.00 m distance in front of the shear wall (Figure 4). The steel brackets for the panels were coloured in blue to eliminate the laser reflection error. Furthermore, for each bracket, a special target (8-inch-high reflectivity target) was applied to improve the surveyed measure precision. The steel cable was positioned horizontally, and a load with a step of 1 kN was applied by a manual winch (Figure 5A) fixed to an unmovable point. The digital load cell mounted on the cable measured the force value applied in real-time. For each load step, the wall deformations were surveyed using the laser scanner (Figure 6A). The deformations were measured until they reached the load value of 8 kN; after this value, the OSB panels showed clear signs of failure. The deformation survey carried out by the TLS had a standard deviation of about 1.9 mm with a level of confidence of 68%, but, above all, it was possible to define the points cloud of the shape deformation of the wall using the Cyclone 9.2 software (Barreca *et al*., 2017). The target points were surveyed.

With a resulting standard deviation of 0.1 mm, and their displacements measure were used to compare the shear wall deformation with the model's results (Janßen *et al*. 2019). The max value force of 8 kN was applied in eight steps; for each step, an increment of 1 kN was applied; subsequently, the wall deformation and the target point (node) displacements components v_x, v_y, v_z were surveyed using the TLS (Figure 6B). The time of load test was approximately four hours because the load was applied very slowly to allow for the settlement of the wall components. The residual wall displacement at the end of the test load was surveyed when the specimen was unloaded.

A displacement analysis of the shear wall specimen was carried out using the point cloud of every step using the Cloud-to-Cloud tool of processing CloudCompare software (Figure 6) (Rajendra *et al*., 2014).

Finite element deformation analysis

A FE simulation of the shear wall was carried out using SISMICAD software. The OSB panels of the wall were modelled using shells interconnected with springs to the column nodes. The brackets were modelled utilising the springs, and the constant K simulated the stiffness of the bracket; for the model, the value was considered variable from 200 to 1000 daN/m. The FE model allowed the software to simulate the behaviour of the wall loaded with horizontal forces applied to upper nodes (Figure 7A). The total horizontal force was applied to the model varying from 1 kN to 8 kN, with steps of 1 kN. For each incremental load step, the displacement components u_x, u_y, u_z of the FE model's nodes were calculated on varying values of spring constant K from 200 to 1000 daN/m with steps of 50 daN/m.

Spring stiffness evaluation

The comparison between the measured and the calculated displacements led to the evaluation of the accuracy of the FE model and the value K spring constant applied.

Each target point surveyed was calculated by (Eq. 1) the distance to the homologous point of the calculated model.

The distance error (Eq. 2) and the average distance error (Eq. 3) (Wargocki *et al*., 1999; Bo *et al*., 2012) were used to assess the fitting between the model deformation and surveyed wall deformation (Table 2).

$$d^i = \sqrt{(v_x^i - u_x^i)^2 + (v_y^i - u_y^i)^2 + (v_z^i - u_z^i)^2} \qquad (1)$$

$$E_T = \sum_i^m (d^i) \qquad (2)$$

$$E_a = \frac{1}{m}\sum_i^m (d^i) \qquad (3)$$

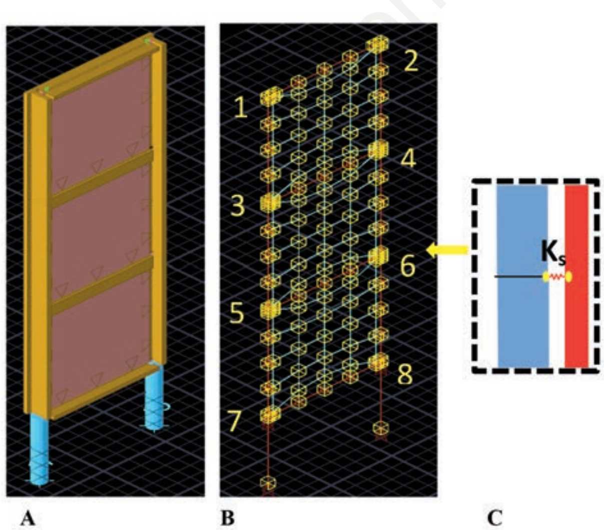

Figure 3. Finite element (FE) shear wall model: A) shear wall model; B) FE model; C) spring connection of K_s stiffness.

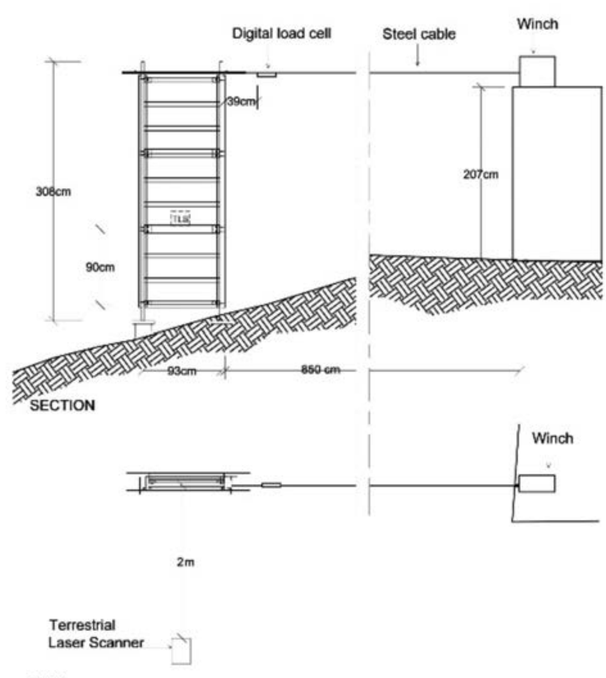

Figure 4. Scheme of the test apparatus.

where: $v^i_{x,y,z}$: are the measured displacements; $u^i_{x,y,z}$: are the calculated displacements.

The average distance error between the measured displacements and the displacement of calculated nodes is variable both at the variance of the horizontal force and at the variance of the spring constant K (Figure 7C). A two-way ANOVA test (α=0.05) was carried out, and a value of 1.998 (F_{crit}=2.087) was obtained for the Horizontal Forces, and a value of 26.786 (F_{crit}=1.709) was obtained for the springs constant K, these values confirmed the statistic dependence between K and errors in displacement evaluation. The K equation was obtained using a regression rational interpolation (Eq. 4), which relates the K value to the horizontal force to obtain the minimum E_T. The function graph highlighted two different K behaviours; in the first part, the K value grows with the increase of horizontal force, and after peaking at a value of 300 daN, the K decreases slowly. This behaviour is probably due to the setting in the initial phase (backlash bolt, foundation screws, *etc.*) after this first phase, the K decreases linearly with the growth of the horizontal forces.

$$K = \frac{a+bF}{1+cF+dF^2} \quad (4)$$

where:
a=2.27·102; b=2.15; c= –3.79·10^{-3}; d=1.08·10^{-5}; correlation coefficient (r): 9.91·10^{-1}.

Figure 5. Test setup: A) the winch with the steel cable to apply the horizontal force; B) the shear wall specimen installed on the soil ground and the laser scanner positioned in front of it.

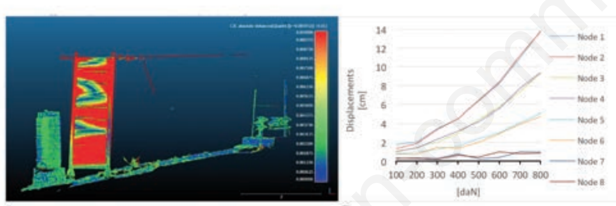

Figure 6. A) Comparison between the final test configuration displacements and the initial configuration displacements of the shear wall specimen through the CloudCompare software; B) node displacements for each step of the horizontal force.

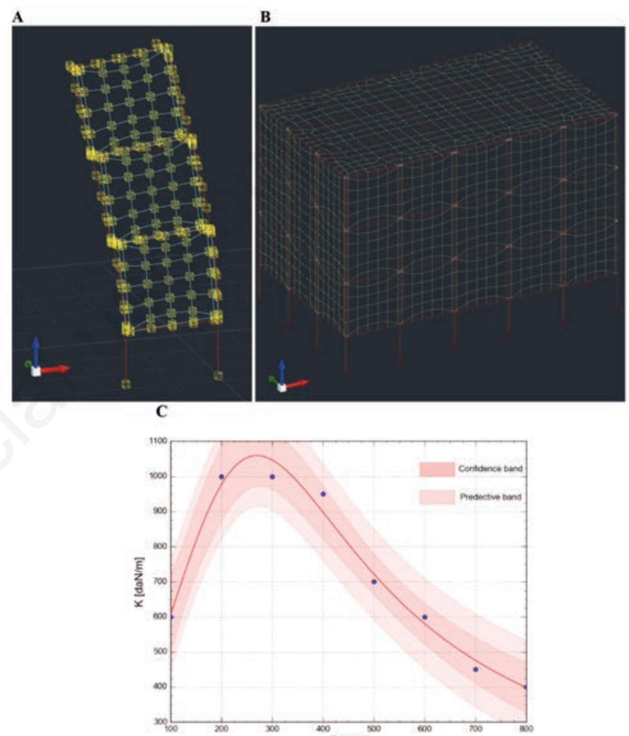

Figure 7. A) Finite element (FE) deformation model of the shear wall; B) FE model for non-linear analysis; C) regression graph: spring stiffness constant and horizontal force.

Table 2. Average distance error between measured displacements and displacement of calculated nodes.

Horizontal force [daN]	Spring constant K [daN/m]																
	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000
100	5.64	5.04	4.63	4.34	4.14	3.99	3.88	3.81	3.78	3.77	3.80	3.85	3.90	3.95	4.00	4.06	4.11
200	12.09	10.53	9.35	8.43	7.69	7.09	6.60	6.19	5.86	5.58	5.34	5.15	4.99	4.87	4.77	4.69	4.64
300	16.28	13.96	12.19	10.80	9.71	8.84	8.15	7.61	7.20	6.87	6.62	6.42	6.28	6.17	6.09	6.05	6.03
400	20.45	17.34	15.01	13.19	11.76	10.64	9.79	9.17	8.70	8.35	8.09	7.90	7.77	7.68	7.64	7.63	7.65
500	21.79	17.89	14.92	12.62	10.85	9.57	8.72	8.14	7.75	7.53	7.45	7.51	7.66	7.86	8.11	8.37	8.65
600	24.90	20.42	17.11	14.72	13.04	11.90	11.14	10.68	10.50	10.55	10.77	11.10	11.49	11.90	12.33	12.76	13.18
700	24.54	19.86	17.34	15.72	14.75	14.35	14.40	14.79	15.37	16.07	16.81	17.55	18.28	18.97	19.63	20.25	20.83
800	24.35	20.07	17.63	16.31	15.95	16.35	17.24	18.36	19.55	20.72	21.83	22.88	23.85	24.76	25.59	26.37	27.09

Structural analysis of the building module

A new structural analysis was carried out to evaluate the resistance of the module against external loads (Figure 7B).

The connections of the walls to the timber portals of the module were simulated through springs with stiffness which was evaluated with Eq. 4. The analysis was conducted using an approach involving a process of iterative refinement of the model. In fact, step by step, the spring constant K was automatically recalculated with reference to the horizontal force applied to the node and calculated in the previous step. The FEM was developed, in accordance with Eurocodes (EN 1990), using a non-linear analysis to consider the deformation of the springs, which led to a different model behaviour to the loads. The deformations of the model, calculated with the OSB panel connection simulated by a spring with variable stiffness, were bigger than the model without. In particular, the FE model highlighted a ductile behaviour with a higher incremental deformation at low loads, whereas it showed lower incremental deformation at high loads. This capacity allows the structure to withstand better cyclic forces, such as seismic or wind gusts, because part of the energy is dissipated in the deformation work (Bovo *et al*., 2020). The verification of the wood module's elements was conducted under Eurocode 5 and highlighted that the module structure could be resistant to a wind velocity bigger than 72 m/sec, which corresponds to the F3 of the enhanced Fujita Scale (EF Scale) of tornado damage intensity (Doswell, Brooks, and Dotzek, 2009). The F3 level represents the damage caused by a strong tornado, such as: *Roofs and numerous outside walls blown away from frame homes, all trees in its path uprooted and/or lofted, two-story homes having their second floor destroyed, high rises have many windows blown out, radio towers blown down, metal buildings (i.e., factories, power plants, and construction sites) are heavily damaged, sometimes completely destroyed. Large vehicles such as tractors, buses, and forklifts are blown from their original positions*, which has an only 4.9% mean probability of happening. The analysis of the structure for this load condition highlighted that the max horizontal force for the shear wall was 677.83 daN and the max displacement was 3.97 cm; this force is lower than the max force applied in the experimental test.

Conclusions

This paper has presented a method to calibrate a FE structural model of a timber building module stressed by external horizontal loads. The module's structure is based on the OSB panels joined by brackets to a wooden skeleton to resist in-plane lateral forces. A more realistic simulation of the structural behaviour and stress deformation of the timber module was carried out introducing variable stiffness connections of the brackets to the panels in the FE model. This modelling gave more reliability to the value of the results, which highlighted the greater resistance of the structure to the wind force than the simpler initial FE model. This more realistic deformation value of the structures allows the designer to verify the structure's compatibility with other components, such as the waterproof layer or service technology systems. To obtain this greater reliability, a more complex model is necessary and harder computation analysis which therefore takes longer computational time to resolve. In fact, to analyse the first structural model, it was necessary to solve a mathematical system with 432 linear equations. To analyse the more reliable model, it was necessary to solve a mathematical system with 25,326 non-linear equations because the introduction of the variable stiffness connections brought the non-linear behaviour of the force-displacement relationship into consideration. It would be possible to develop specific structural software to optimise the analysis procedure based on a more efficient algorithm to supply technicians with an easy tool to design and facilitate the diffusion and use of these building modules.

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