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CONCEPTION OF SANDWICH STRUCTURAL PANELS COMPRISING THIN-WALLED STEEL FIBRE REINFORCED SELF-COMPACTING CONCRETE (SFRSCC) AND FIBRE REINFORCED POLYMER (FRP) CONNECTORS

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Summary: *In this paper, an innovative thermally efficient sandwich structural panel is proposed for the structural walls of a pre-fabricated modular housing system. Traditionally, sandwich concrete panels consist of conventional reinforced concrete wythes as external layers, polystyrene foam as core material and steel connectors. However, steel connectors are known to cause thermal bridges on the building envelope and possibly condensation and mould problems. Furthermore, the possibilities for thickness reduction/optimization of conventionally reinforced concrete layers are frequently limited by minimum cover requirements. To overcome these issues, the proposed sandwich panel comprises Fibre Reinforced Polymer (FRP) connectors and two thin layers of Steel Fibre Reinforced Self-Compacting Concrete (SFRSCC). This paper presents the basic conception of the proposed building system together with preliminary parametric numerical analyses to define the arrangement and geometry of the elements that constitute the sandwich panels. Finally, the feasibility of using the proposed connector and SFRSCC on the external wythes is experimentally investigated through a series of pull-out tests where failure modes and load capacity of the connections are analysed.*

1 INTRODUCTION

The main interest on using sandwich panels in building construction is related to the structural and thermal efficiency that can be achieved with this technology. By putting together thin, stiff and ductile concrete wythes, with a thermally-efficient core material, it is possible to obtain lightweight panels that are energy efficient and that can be easily handled and erected. This kind of structural solution has been extensively applied in precast structural panels, where the external layer also provides protection against mechanical damage, weather sheltering and vapour barrier, while the core provides thermal insulation [1].

Traditionally, these sandwich panels adopt conventionally reinforced concrete wythes and steel connectors to establish the connection between them. Commonly used shear transfer devices include wire truss connectors, bent wire connectors, and solid zones of concrete penetrating the foam [2-4].

Due to their high thermal conductivity, the steel connectors cause thermal bridging effects on the building envelope that result in increased heat flow and reduced internal thermal comfort, while creating conditions for the possibility of condensation and mould problems [5]. The existence of these thermal bridges increases the building energy demand for heating and cooling, and thus their avoidance is an essential challenge to achieve more thermally/energetically efficient and sustainable

buildings. Furthermore, with the steadily increasing cost of energy, the interest in having energetically efficient buildings is no longer limited to sustainability aspects, broadening its justification to actual economic reasons.

The use of glass fibre-reinforced polymer (GFRP) bars formed in a truss orientation instead of metal wire trusses was introduced by Salmon et al. [4]. Test results showed that the use of GFRP achieved a high level of composite action and provided thermal benefits similar to noncomposite insulated sandwich wall panels. Following the same concept, carbon fibre-reinforced polymer (CFRP) shear connection grid is introduced in the construction of sandwich wall panels in 2003 [6]. Nevertheless, both solutions have a high cost when compared with the traditional ones. In fact, the pultrusion manufacture process GFRP bars requires high cost specialized machinery, whereas the carbon fibre is still expensive for the proposed application.

This paper proposes an innovative sandwich panel with suitable structural and thermal performance, aimed for integration in an industrialized building system that also comprises precast sandwich elements for structural slabs. This system is intended to be cost competitive with focus on reduction of construction time, material optimization, while being environmentally friendly. The proposed sandwich panel for exterior bearing walls comprises FRP connectors of controlled cost, and two thin outer wythes of Steel Fibre Reinforced Self-Compacting Concrete (SFRSCC).

The role of SFRSCC is related to the inherent benefits of using this material instead of concrete with conventional reinforcement. Firstly, it allows reducing the volume of concrete necessary to produce the external layers, since the requirement of minimum cover for the reinforcements in conventional reinforced concrete structures is not applied to SFRSCC elements. So, it is possible to obtain more structurally efficient and lightweight elements. Another advantage is related to the possibility of reducing non-value adding activities on the production line and the related labour costs. Specifically, SFRSCC technology eliminates the tasks of placing the reinforcement (mesh or bars), and compacting/levelling concrete, thus allowing easier standardization of the production tasks. Furthermore, Fibre Reinforced Concretes (FRC) have several properties that make them attractive for this application: they generally present increased ductility, improved impact resistance and better water tightness due to the relatively high content of fine constituents and their improved crack-width control capacity.

This paper begins with a brief description of the proposed building system in which the devised sandwich panels are included. Then, through parametric analyses, efforts are made for assessing the best solutions for the geometry of the panel and arrangement/distribution of GFRP connectors. The parametric studies include both linear and non-linear analyses of the panel subject to axial loadings and wind pressure. This research work also comprises experimental parametric studies (pull-out tests) for the assessment of the relative effectiveness of the suggested connections between GFRP and SFRSCC.

2 PROPOSED BUILDING SYSTEM AND INNOVATIVE SANDWICH PANELS

The building system introduced in this paper is developed for single-storey residential, commercial or industrial buildings. It is composed of pre-fabricated sandwich structural panels for both façade walls and slabs, which are *in situ* connected to each other, so that a global structural behaviour of the building can be accomplished. The idea is to develop a structural exterior wall system that incorporates all the installations (water, electrical, network and telephone connections), thermal insulation and finishes. The entire system is prefabricated at a plant and transported to the construction site, where the remaining tasks to be performed are placement of the panel and connections to foundations and adjacent elements. By incorporating systems, the added value of panel is increased, the number of stages in the construction is diminished, the work efficiency is enhanced and, consequently, the overall cost of building is reduced.

The above described prefabricated structural sandwich panel comprising thin-walled SFRSCC to be used as structural façade is shown schematically in Figure 1a), where the various components involved are identified, and a possible arrangement of connectors is shown. This wall system acts as

the primary load carrying system of the structure transferring the loads to the foundation of the structure. The single storey wall panels span vertically between foundations and floor/roof panels without the need for additional intermediate supports. Horizontal floor and roof panels' span shall behave as one-way slabs, continuously supported by the inner concrete layers of wall panels as shown in Figure 1b). This eliminates the need for beams and columns along exterior walls. The horizontal panels shall also act as diaphragms that transfer the lateral loads to the walls.

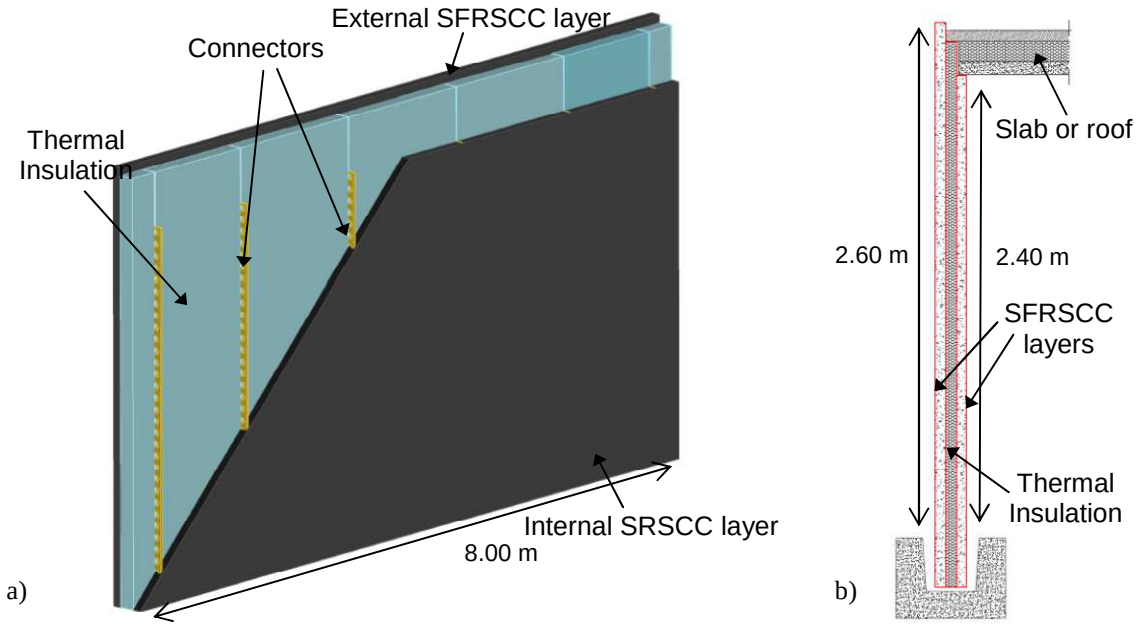


Figure 1: Proposed building system: a) components of the devised load-bearing sandwich wall panel; b) cross-section

3 PRELIMINARY PARAMETRIC STUDIES

To better understand the structural behaviour of the panel, and the potential for joint behaviour of the wythes, a set of systematic parametric studies are carried out. At an initial stage, focus is given to the optimum arrangements and properties of the parts of the sandwich panels. The FEM-based software FEMIX [7] is used for these analyses.

3.1 Common features: geometry, loads, supports and mechanical properties

In this work all the analyses are limited to a reference sandwich panel with 8.00 m length and 2.60 m height (the external and the internal SFRSCC layers with 2.60 m and 2.40 m height, respectively). In order to simplify the study, the thickness of the insulating material is kept equal to 100 mm. The thickness of the SFRSCC layers is one of the variables studied.

For a better explanation of load and support conditions, the description follows by providing a general explanation regarding the finite element model. The FEM mesh of the sandwich panels consists of Reissner-Mindlin flat shell elements to represent the internal and external SFRSCC wythes and the FRP connectors by their middle plan. The Gauss-Legendre integration scheme with 2×2 integration points is used in all elements. The initial models assume that bond between materials (SFRSCC and FRP) is perfect. The contribution of the insulating material is disregarded.

The panels are subjected to both horizontal (x direction) and vertical loads (z direction). It is considered that the roof/floors transfer the vertical forces to the load-bearing walls and into the

foundation. The lateral loads (i.e.: wind) are withstood by the sandwich wall panels that span between the floor/roof and the foundation. Floor/roof systems are considered to act as horizontal diaphragms, in the sense that horizontal loads are carried through these diaphragms into the shear walls (the sandwich wall panels in orthogonal direction). These supporting conditions are shown in Figure 2.

The vertical loads correspond to the gravity load of the wall panel and the load (R_{SL}) transferred by the slab (10.0 m of span), which is simulated by a centred line force applied to the upper edge of the internal SFRSCC layer (see schematic representation in Figure 2).

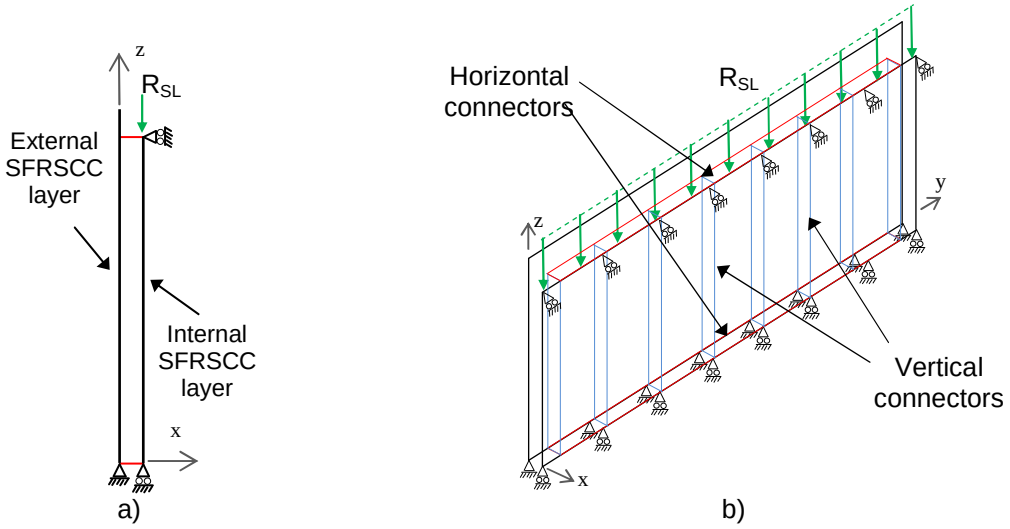


Figure 2: Schematic representation of support conditions and reaction of slab: a) cross-section; b) overall view

The wind loading is calculated following the simplified procedure described in the Eurocode 1 – Part 1-4 [8]. The values that lead to the most unfavourable conditions considered in the Portuguese National Annex are chosen. The described load cases are included in three different relevant combinations (Ultimate Limit States – ULS): LC1 – external SFRSCC layer under pressure and internal SFRSCC layer under suction; LC2 – external SFRSCC layer under suction and internal SFRSCC layer under pressure; and LC3 – both layers under suction (see Figure 3 and

Table 1).

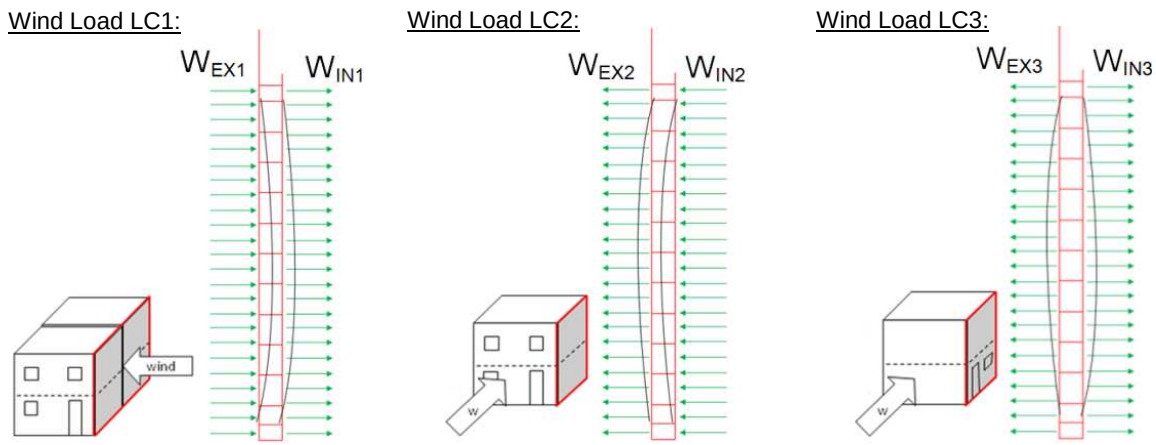


Figure 3: Wind load cases

Table 1: Adopted values for the load cases and load combinations

Load cases		
Gravity Load	G	-
Slab reaction (design value)	R_{SL}	-48.75 N/mm
LC1 – External wind force	W_{EX1}	+1.021e-03 N/mm^2
LC1 – Internal wind force	W_{IN1}	+6.381e-04 N/mm^2
LC2 – External wind force	W_{EX2}	-1.531e-03 N/mm^2
LC2 – Internal wind force	W_{IN2}	-4.466e-04 N/mm^2
LC3 – External wind force	W_{EX3}	-1.531e-03 N/mm^2
LC3 – Internal wind force	W_{IN3}	+6.381e-04 N/mm^2

Load combinations (ULS)	
LC1	$1.35 \cdot G + 1.0 \cdot R_{SL} + 1.5 \cdot (W_{EX1} + W_{IN1})$
LC2	$1.35 \cdot G + 1.0 \cdot R_{SL} + 1.5 \cdot (W_{EX2} + W_{IN2})$
LC3	$1.35 \cdot G + 1.0 \cdot R_{SL} + 1.5 \cdot (W_{EX3} + W_{IN3})$

For these preliminary studies it was assumed that 60 kg/m^3 of hooked ends steel fibres of 35 mm length, 0.55 mm diameter and 1100 MPa tensile strength were used for the production of SFRSCC. The quasi-isotropic GFRP laminate used for the connectors has a total thickness of 5.0 mm and the following distribution of internal fibres: 25% in the longitudinal direction (0º), 25% in the transverse direction (90º) and 50% in the directions ±45º. In these parametric studies, all the materials are assumed as having a linear-elastic isotropic behaviour. The material properties used in the numerical analysis are shown in Table 2. The indicated values are characteristic values determined from experimental tests with SFRSCC cylinders and with GFRP laminates.

Table 2: Assumed values for the material properties in the parametric studies

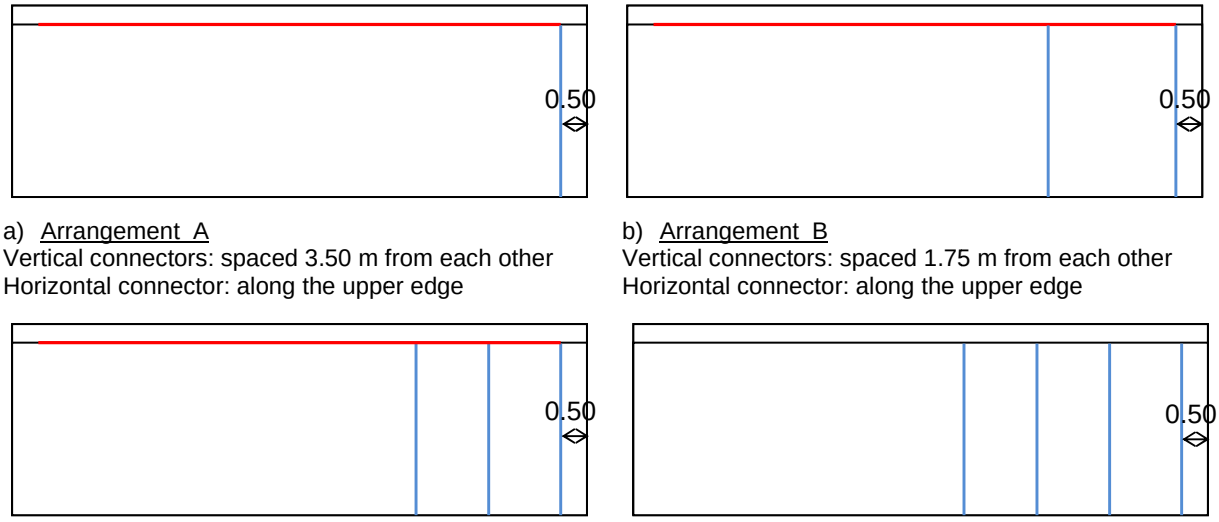
SFRSCC	
Young´s Modulus (E_{cm})	31000 MPa
Poisson ratio (ν_c)	0.15
Tensile strength ($f_{ct,k}$)	2.9 MPa
Compressive strength ($f_{c,k}$)	52.0 MPa
GFRP laminate	
Young´s Modulus (E_{pm})	13950 MPa
Poisson ratio (ν_p)	0.50
Tensile strength ($f_{pt,k}$)	330.7 MPa

3.2 Effect of the arrangement of the connectors and thickness of SFRSCC layers on the maximum stresses

In this section, the effect of the arrangement adopted for the connectors of the sandwich panels on the maximum tensile principal stresses in the panel is studied. Both vertical and horizontal continuous connectors are considered. The arrangements studied are described in Figure 4. The thickness of the GFRP flat shell connectors are held constant at 5 mm, whereas SFRSCC flat layers are 75 mm thickness, which is a typical thickness for concrete layers of sandwich panels [1].

Figure 5 shows the evolution of the maximum principal tensile stress (σ_1) in both SFRSCC layers and also in the GFRP connectors for the four types of panel studied (positive values correspond to tensile stresses). From these results, it is possible to infer that as the spacing between the vertical connectors is reduced the stress level in both SFRSCC layers and vertical connectors are diminished. It may be further noted that the omission of horizontal connectors (arrangement D) does not imply relevant changes in terms of principal stresses in SFRSCC and GFRP. It can be also observed that, independently of the arrangement, the stress level in the GFRP connectors is always much lower than the strength of the material (Table 2). For SFRSCC layers of 75 mm, the maximum principal tensile

stress does not reach the tensile strength of SFRSCC in any of the studied arrangements. For arrangements B, C and D the stress level is similar in both layers, what is desirable once, for simplicity of process, the material adopted for both layers is the same. Thus, from these obtained results it can be concluded that an arrangement as simple as the arrangement D, with only vertical connectors, can be adopted for the panels keeping the stress level in the SFRSCC layers low enough to avoid cracking. For this reason, the arrangement D is chosen for the following studies.



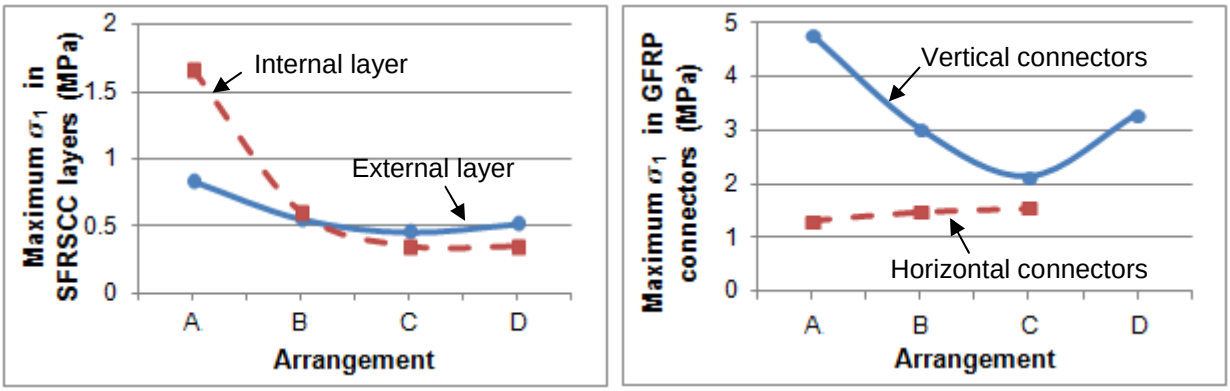
a) Arrangement A
Vertical connectors: spaced 3.50 m from each other
Horizontal connector: along the upper edge

b) Arrangement B
Vertical connectors: spaced 1.75 m from each other
Horizontal connector: along the upper edge

c) Arrangement C
Vertical connectors: spaced 1.00 m from each other
Horizontal connector: along the upper edge

d) Arrangement D
Vertical connectors: spaced 1.00 m from each other
Horizontal connector: none

Figure 4: Arrangements of connectors studied: a) A; b) B; c) C and d) D (units in metres)



a) b)

Figure 5: Influence of connector arrangement on the maximum principal tensile stresses (σ_1) of: a) SFRSCC layers; b) GFRP connectors

Figure 6 presents the effect of the thickness of SFRSCC layers on the maximum value of the principal tensile stresses in these layers. The maximum stresses obtained for a panel with the arrangement D and thickness of concrete layers of 35, 45, 55, 65 and 75 mm are presented. From the

results, it may be observed that even for sandwich comprising layers with 35 mm thickness, the maximum principal tensile stresses are lower than the half of the tensile strength considered for the adopted SFRSCC. It is also observed that the reduction of the thickness of SFRSCC layers from 75 to 35 mm is accompanied by a slightly increase of the maximum tensile stresses in the connectors from 3.3 to 5.0 MPa, what is not significant if compared with the tensile strength of GFRP laminate.

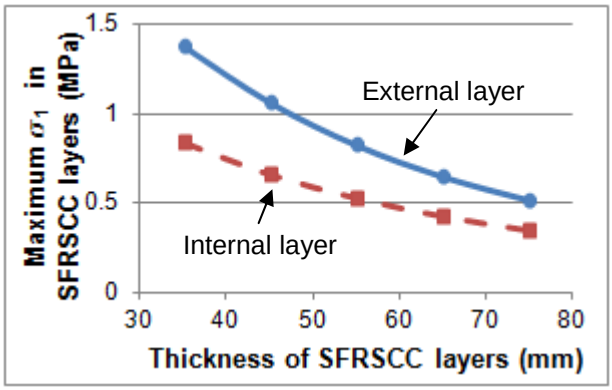
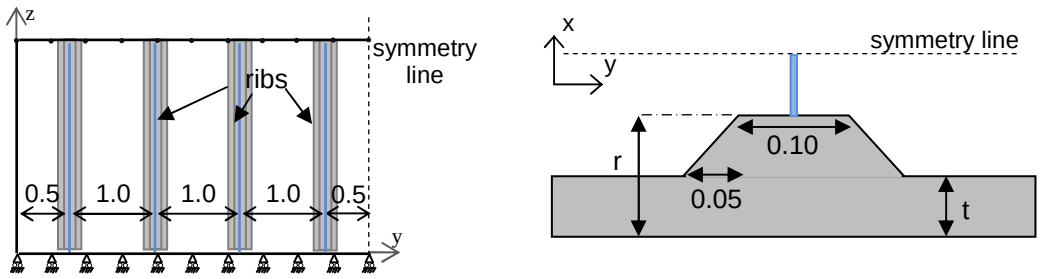


Figure 6: Influence of thickness of the SFRSCC layers on the maximum principal tensile stress

3.3 Ribbed SFRSCC layers: influence of thickness on the maximum stresses

The results shown in the sections 3.2 permit to conclude that it is possible to build the house system using a sandwich panel with the arrangement D, and comprising SFRSCC layers as thin as 40 mm, keeping its parts working in a low stress level. However, the use of such thin SFRSCC layers leads to potentially problematic stress concentrations in the vicinity of the connections and may also represent a difficulty to assure proper bond conditions with the GFRP connectors. In order to bypass these issues, the use of ribbed SFRSCC layers is proposed and its viability is evaluated. As shown in Figure 7, each SFRSCC panel comprises 8 ribs (spaced of 1.0 m from each other) conveniently arranged in the vicinity of the connectors. The current sections of the SFRSCC layers are t = 40 mm thick. The results are computed for panels with two rib thicknesses (r = 60 and 80 mm) and results are compared with the results obtained to a sandwich panel comprising flat concrete layers (r = 40 mm). All the other conditions and properties are kept the same presented in section 3.1.



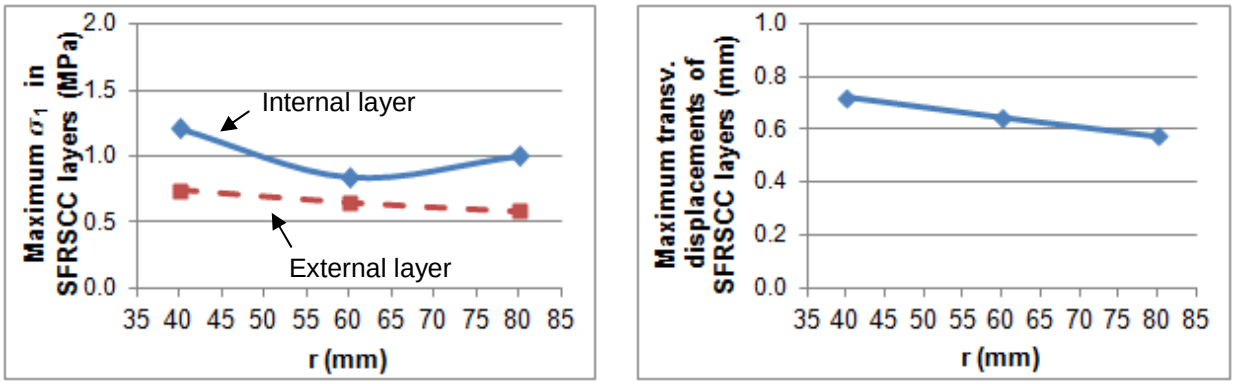
a) b)

Figure 7: Sandwich panels comprising ribbed SFRSCC layers a) arrangement of ribs and connectors SFRSCC layers; b) detail of ribbed section (units in metres)

The maximum principal stresses in both SFRSCC layers and maximum transverse displacements (*x* direction) are computed and the results are shown in Figure 8. Results shown in Figure 8 permit to conclude that the adoption of ribbed SFRSCC layers slightly reduces both the maximum tensile stress and maximum transverse displacement in the panel.

4 NONLINEAR ANALYSIS

The mechanical behaviour of the sandwich panel with consideration of the post-cracking behaviour (nonlinear analysis) is assessed by using the FEMIX program. The geometry, mesh and support conditions are those of the parametric study described in section 3. Only the arrangement D of connectors is studied and the SFRSCC layers are ribbed as shown in Figure 7 (t = 40 mm and r = 80 mm).



a) b)

Figure 8: Influence of thickness of SFRSCC ribbed section on: a) maximum principal tensile stresses (σ_1) in both SFRSCC layers; b) maximum transverse displacement (*x* direction)

The SFRSCC fracture properties adopted in the numerical analysis are shown in Figure 9. The indicated values are determined experimentally and the tensile strain-softening diagram is obtained from inverse analysis [9]. The trilinear diagram is used to model the softening behaviour of SFRSCC, assuming a crack bandwidth (l_b) equal to the square root of the area associated with the corresponding integration point. The linear analyses showed that the GRFP connectors always remains at stress levels well below their strength limits, therefore they are here simulated assuming linear elastic behaviour (properties presented in Table 2).

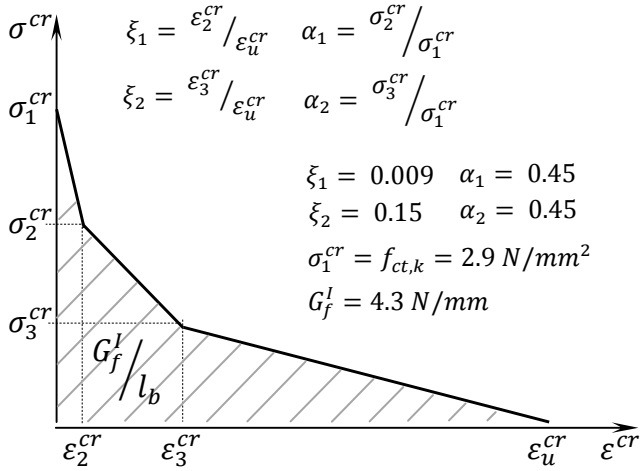


Figure 9: Trilinear tension softening diagram for the SFRSCC

For this study only the slab reaction and the wind load corresponding to the load combination LC1 is considered (pressure in the external layer and suction in the internal layer). Firstly, the slab reaction

is applied and then, the wind loads are gradually applied, multiplying an increasing wind load factor *k* to the characteristic value of the wind loadings. The evolution of the maximum crack opening in both SFRSCC layers as function of *k* is computed and depicted in Figure 10.

Although the first crack opening in the SFRSCC layers appears for a wind load factor of 8, a maximum crack opening of 0.3 mm is only attained for a wind load factor equal to 36 (equivalent to 24 times the wind load factor for the ULS). For this wind load factor, the correspondent transverse displacement of a middle point of internal and external concrete layers are, respectively, 8.9 mm (L/270) and 9.2 mm (L/261). For this wind load factor the maximum tensile and compressive stresses in the GFRP connectors are, respectively 105 and 100 MPa, what means that, even for transversal loads as high as 24 times the corresponding to ULS, the rupture of the GFRP connectors is not expected.

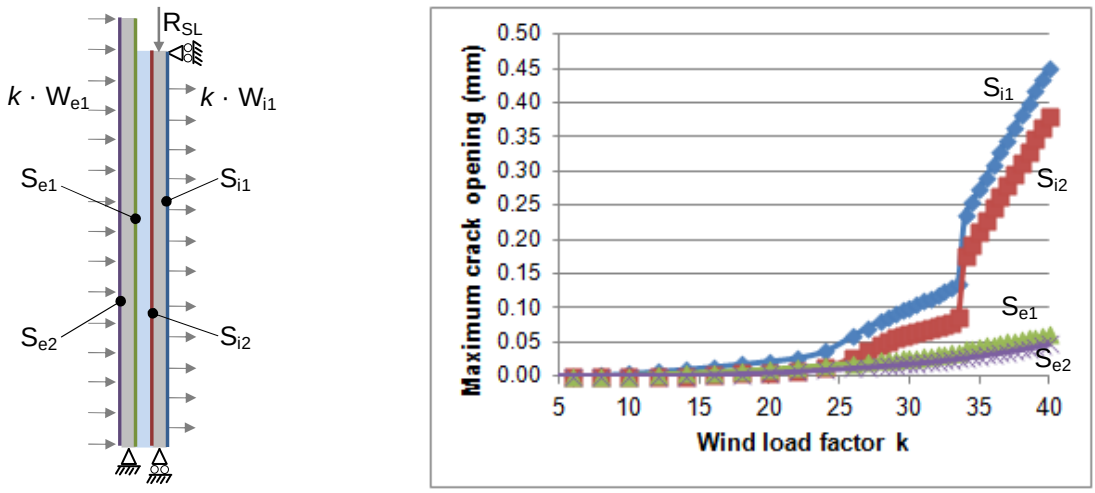


Figure 10: Evolution of maximum crack opening in the SFRSCC layers as function of *k* wind load factor

5 EXPERIMENTAL EVALUATION OF EFFECTIVENESS OF GFRP-SFRSCC CONNECTIONS: PULL-OUT TESTS

The GFRP connectors used in sandwich panels are subjected to tensile loads during the stripping and handling phases. Thus, these connections may be designed to withstand the self-weight of the bottom SFRSCC layer and the suction forces generated between the concrete and the form. To investigate this, a series of pull-out tests is performed. Four types of connectors are investigated: one adhesively bonded (C) and 3 types of embedded connectors (L, T and Y) (see Figure 11). The adhesively bonded type consists of an inverted T GFRP profile bonded to the hardened concrete surface through an epoxy layer. The embedded type connectors are designed to benefit from mechanical interlock with SFRSCC layers. They are pre-positioned before pouring the first concrete layer. Among the embedded connectors the following alternatives, depicted in Figure 11, are tested: simply perforated plate (L), T-shaped profile (T) and Y-shaped profile (Y).

The specimens consist of a GFRP connector and a 400 mm x 200 mm SFRSCC block (Figure 11) with 40mm thickness and ribbed at the intersection with the GFRP. The connectors are manufactured by Vacuum Assisted Resin Transfer Molding (VARTM) process [10] using a glass fibre quasi-isotropic mat as reinforcement and polyester resin matrix. The properties of the produced laminates are those indicated in section 3. The SFRSCC composition is: cement CEM I 42.5R, limestone filler, coarse river sand, crushed granite 5-12 mm, hooked end steel fibres (length of 35 mm, diameter of 0.55 mm and a yield stress of 1100 MPa), water and a superplasticizer based on polycarboxilates. The characteristic

values for the compressive strength and the modulus of elasticity of SFRSCC obtained from tests with cylinders are shown in Table 2.

The test setup adopted in the tests is shown in Figure 12. The setup includes the hydraulic machine, the 50 kN load cell, the clamp, the supports, the transducers and the electronic data acquisition unit. The vertical load is applied at top extremity of the FRP through the clamp. The lower supports consist on a pair of steel bars with 40 mm width and 500 mm length placed over the SFRSCC block, as shown in Figure 12, and anchored to a bottom steel plate. Slip between the GFRP and concrete is measured in the front and rear sides of the specimen by applying one transducer in the FRP surface nearby the concrete top surface. A metallic bar is adopted as reference point to measure the slip relatively to a point on the concrete surface aligned with the supports. Another transducer measures the midspan deflection of the concrete block and this value is subtracted from the measures of the transducer fixed to the FRP to obtain the relative slip between the FRP and the concrete block. All the specimens are tested under monotonic loading at a constant displacement rate of 0.03 mm/s. Two test specimens per each connector type are carried out.

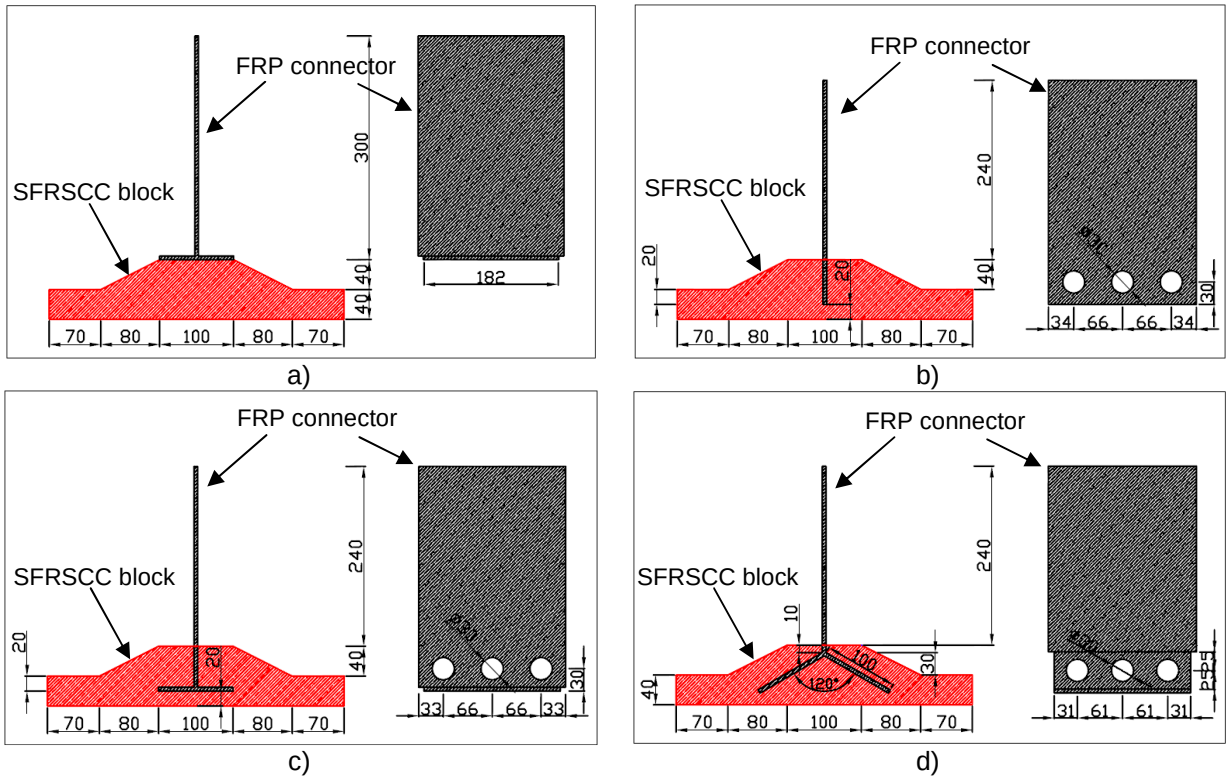


Figure 11: Details of pull-out specimens: a) adhesively bonded – type C, b) embedded – type L, c) embedded – type T, d) embedded – type Y. (dimensions in mm)

None of the specimens failed by rupture of the FRP connectors, which remained nearly intact at the end of testing, as it can be seen in the failure mechanisms shown in Figure 13a. Representative mean force *vs.* average slip relationships are depicted in Figure 13b. The mean failure loads per unit length for the specimens of C, L, T and Y type are, respectively, 47.85 kN/m, 48.65 kN/m, 63.45 kN/m and 95.65 kN/m. Figure 13b shows that the embedded connectors present a significant ductility after peak load, as the values obtained for ultimate slip are always higher than 7 mm. Despite the load capacity of adhesively bonded connector to be similar to the obtained for the L connector, the failure modes of these types of connectors are very different. While the adhesively bonded connectors fail in a brittle way, the L connectors show a ductile behaviour, presenting an ultimate slip higher than 20 mm.

Considering the arrangement of panel shown in Figure 7 and an adhesion to mould of 2.0 kN/m^2, the tensile load on connectors during stripping is estimated on 3.5 kN per metre of connector for a panel stripped flat. Therefore, the obtained results permit to conclude that all the studied connectors have a high safety factor during stripping. A mean safety factor of 13.6, 13.9, 18.1 and 27.2 is obtained, respectively for the C, L, T and Y connectors.

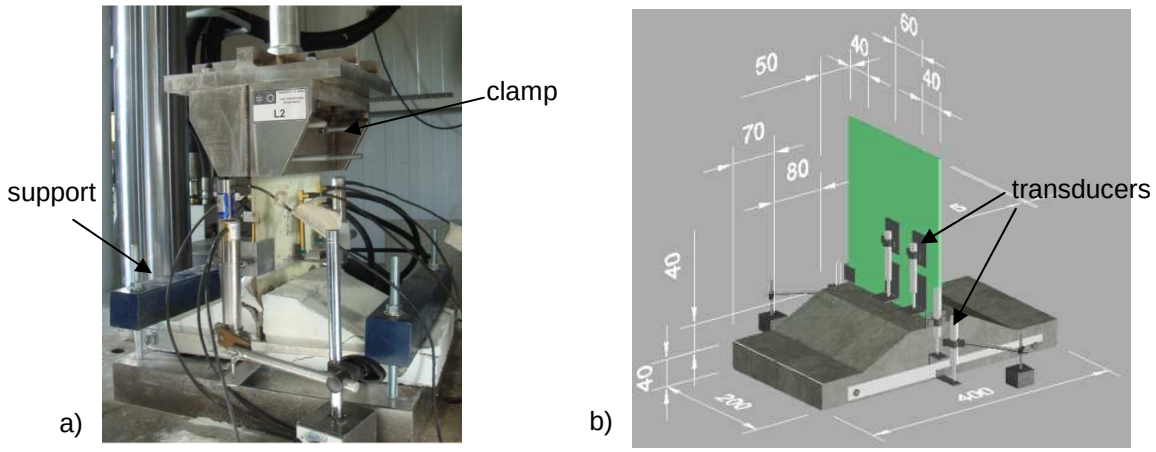


Figure 12: Pull-out test setup: a) overview, b) detail of pull-out specimens (dimensions in mm)

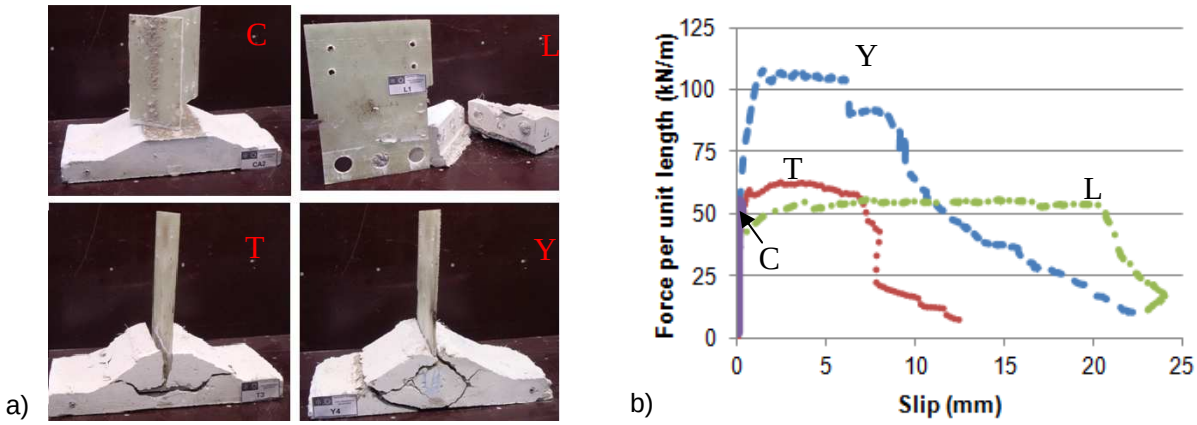


Figure 13: Pull-out test results: a) typical failure mechanisms, b) load-slip curves.

9 CONCLUSIONS

In this study an innovative sandwich panel comprising SFRSCC layers and GFRP laminate connectors is devised and optimized through a FE linear modelling. The structural behaviour of the sandwich panel by considering the post-cracking behaviour (nonlinear analysis) of the SFRSCC is also assessed. Pull-out tests were carried out using different types of GFRP connectors and the ultimate load and failure modes were analysed. Based on the obtained results, the following conclusions can be derived.

The maximum tensile stress in the SFRSCC layers is significantly affected by the arrangement of the GFRP connectors. However, for a panel configuration comprising vertical GFRP connectors spaced 1 m from each other, the omission of horizontal connectors does not imply relevant changes in

the principal stresses in SFRSCC and GFRP. Regardless the arrangement and thickness of SFRSCC layers, the stress level in the GFRP connectors is always much lower than the strength of the material, what permits to conclude that, disregarding the local effect in the connections, the weakest components in the panel are the SFRSCC layers. However, even for those panels comprised of SFRSCC layers as thin as 35 mm, the maximum principal tensile stresses are lower than the characteristic tensile strength of SFRSCC. Furthermore, the adoption of a ribbed configuration for the SFRSCC layers can be the solution to materialize the GFRP–SFRSCC embedded connections. This solution shows a slightly improvement of the overall behaviour of the sandwich panel due to the increase of stiffness, leading to a lower tensile stress level in the SFRSCC layers and to a reduction of the maximum deflection when the panel is submitted to a design governing load combination. The results obtained from the material nonlinear simulations of the sandwich panels showed that the proposed configuration for the panel presents a ductile behaviour, even for wind load factor 24 higher than the load factor corresponding to the ULS.

From the experimental pullout-tests, it can be concluded that the GFRP Y type connector provided the highest load carrying capacity. Despite the smaller load carrying capacity of the other tested embedded connectors, they still present similar load capacity that is at least 13 times the tensile loadings developed during the stripping of a panel flat. All the embedded solutions present a significant ductility after peak load. Among them, is important to emphasize the behaviour of the simply perforated connectors that, despite its simplicity, develop a very appreciable residual resistance to high values of slip.

ACKNOWLEDGEMENTS

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