Sandwich panels with GFRP faces – Study of mechanical behaviour and of their applicability in buildings

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Abstract: The growing need for building constructions for dwelling, in developing countries and due to some particular contingencies of such locations, has led to the necessity to find alternative constructive solutions to traditional methods of building. The advantageous features of the sandwich panels, such as the high stiffness/weight and strength/weight ratios, make them easy to use on the building site and can perform the functions of floor, walls and roof. The suitability of these sandwich panels in a prefabricated construction system was studied.

Three types of panels were studied, with epoxy resin reinforced with glass-fibre (GFRP) as facing materials and the following core materials: expanded polystyrene (EPS), agglomerated of expanded cork (ACE) and polypropylene honeycomb (FMPP). For this purpose, a study of the constituent materials and of the mechanical behaviour of sandwich panels was performed. Additionally, an experimental campaign was executed with the aim of doing a mechanical characterization of these panels. Next, based on the tests performed, the strength of the panels was calculated according to the methodology proposed in Annex D – "Design assisted by testing" of the Eurocode 0. Finally, based on a prototype building where the same type of panels had been applied, the safety of these panels was verified.

Based on the experimental tests and design resistant values calculated, it was found that none of the solutions related to sandwich panels applied in the studied building met all the required security checks. Thus, corrective measures were suggested.

This work was conducted in the research project “BMM – Bloco Multi-Modular”.

Keywords: Sandwich panels, GFRP, mechanical behaviour, experimental campaign, design assisted by testing.

1. Introduction

The main goal of this work is to evaluate the mechanical behaviour of composite sandwich panels with GFRP (Glass Fibre Reinforced Polymer) faces and the following core materials: expanded polystyrene (EPS), agglomerated of expanded cork (ACE) and polypropylene honeycomb (FMPP) in order to a possible use as floor, walls and covering of a pre-fabricated building. A study of the mechanical behaviour is presented, showing some expressions that can predict this behaviour at service and at collapse. Next, an experimental campaign with the aim of doing the mechanical characterization of these panels is performed. Additionally, with the results obtained in the experimental campaign, the design resistance values are calculated according to a methodology proposed by the annex D - "Design assisted by testing" of the Eurocode 0. Ultimately, a case study is shown, a building localized in Benavente, Portugal, that includes solutions utilizing the sandwich panels studied, verifying subsequently the safety of pavement, wall and covering.

2. Study of sandwich panels to apply in building prefabrication

2.1. Generalities

Due to their own features, sandwich panels can be very important elements in prefabricated buildings. They combine the advantageous characteristics of the faces and core, providing elevated ratios strength/weight and stiffness/weight, thermal insulation, long life at low maintenance costs and easiness to put in place. Nevertheless, there are some disadvantageous features such as behaviour in case of fire and creep [1]. A sandwich panel consists of two faces of reduced thickness, a thick layer named as core and adhesive situated in the interface between core and face. Taking into account the scope of this work, a brief description of will be given of GFRP as facing material and EPS, ACE and FMPP as cores materials.

GFRP is composed by glass fibres and a polymeric matrix. Generally it has the following features: an elevated ratio between strength and weight, high chemical and corrosion resistance, durability with low maintenance costs, low ductility and important creep behaviour [2].

EPS and ACE are formed by an interconnected network of solid struts, and, therefore, are called cellular solid materials. They have a similar mechanical behaviour, characterized by three phases at compression and two phases at tension. The compression stages comprise of two first phases approximately linear, with the first one significantly more stiff than the second one and a third non-linear phase where the stiffness increases progressively. The tension phases are approximately linear and relatively similar. Both materials can provide good thermal and sound insulation. The density of ACE is substantially bigger than EPS and regarding to the fire performance, ACE has a better behaviour [3, 4, 5].

FMPP is a two dimensional cellular solid called honeycomb and is made of polypropylene. It has some debilities regarding the fire behaviour and thermal insulation. Unlike the other two core materials, the mechanical behaviour is non isotropic. Thus, the performance depends on the force direction, where the stiffer direction is perpendicular to the plane of the FMPP [3].

2.2. Mechanical characterization

To predict the mechanical behaviour of a sandwich panel, the following assumptions were assumed: all the materials are firmly bonded together, the facing material is much stiffer and thinner than the core material, all materials are isotropic, the cross-sections remain plan and perpendicular to the longitudinal axes and the sandwich panel works only in one direction (beam behaviour). Note that some of these approximations can induce to significant errors. A sandwich panel beam is shown in the Figure 1 [6].

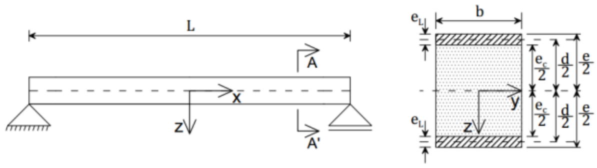


Figure 1 – Dimensions of sandwich beam

The bending stiffness (D) and the shear stiffness (U) are given, approximately, by the equations (1) and (2), respectively [6].

$$D = E_L \cdot \frac{b \cdot e_L \cdot d^2}{2} \tag{1}$$

$$U = b \cdot \frac{d^2}{e_c} \cdot G_c \tag{2}$$

Where E_L is the elasticity modulus of the faces and G_c is the core shear modulus.

Due to the fact that the facing material is much stiffer and thinner than the core material, the axial stress distribution over the height of a sandwich panel is given by (3) and (4) and the shear stress distribution is given by (5) and (6) [1], [6].

$$\sigma(z) = \frac{M}{d \cdot b \cdot e_L} \qquad \text{for} \qquad \frac{e_c}{2} \leq |z| \leq \frac{e}{2} \tag{3}$$

$$\sigma(z) = 0 \qquad \text{for} \qquad -\frac{e_c}{2} \leq z \leq \frac{e_c}{2} \tag{4}$$

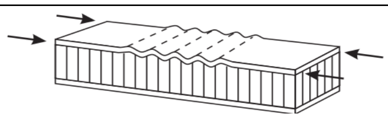
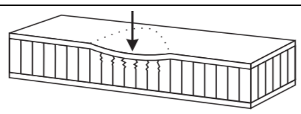
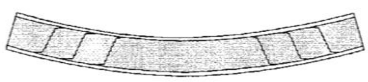
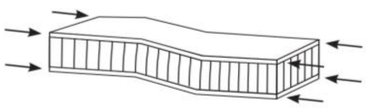
$$\tau(z) = \frac{V}{b \cdot d \cdot e_L}\left(\frac{e}{2} - |z|\right) \qquad \text{for} \qquad \frac{e_c}{2} \leq |z| \leq \frac{e}{2} \tag{5}$$

$$\tau(z) = \frac{V}{b \cdot d} \qquad \text{for} \qquad -\frac{e_c}{2} \leq z \leq \frac{e_c}{2} \tag{6}$$

Where $\sigma(z)$ is the axial stress, M is the bending moment, z is the vertical distance to the gravity centre of the panel and $\tau(z)$ is the shear stress.

In a sandwich panel, failure can occur in different ways. In the Table 1 some of the most common failure modes are shown.

Table 1 – Failure modes

Tensile failure of face [1]	
Wrinkling failure [7]	
Intra-cell wrinkling failure [8]	
Crushing failure [8]	
Core shear failure [1]	
Adhesive Failure [1]	
Buckling failure [8]	
Shear crimping [8]	

The bending moment resistance by tensile failure of face (M_R) is given by equation (7) [1]. The wrinkling stress ($\sigma_{cr,w}$) and intra-cell wrinkling stress ($\sigma_{cr,w}$) are presented in equations (8) and (9), respectively [1, 9]. The shear resistance (V_R) is given by equation (10) [1]. In the equations (11) and (12), the critical buckling load (P_b) and critical shear crimping ($P_{b,c}$) load are presented, respectively [8].

$$M_R = \sigma_{LtR} \cdot b \cdot e_L \cdot d \tag{7}$$

$$\sigma_{cr,w} = \frac{3}{2} \cdot \sqrt[3]{\frac{2 \cdot (1-\nu_c)^2 \cdot {E_{c,z}}^2 \cdot E_L}{3 \cdot (1+\nu_c)^2 \cdot (3 - 4 \cdot \nu_c)^2 \cdot (1 - {\nu_L}^2)}} \tag{8}$$

$$\sigma_{cr,c} = \frac{2 \cdot E_L}{(1 - {\nu_L}^2)} \cdot \left(\frac{e_L}{s}\right)^2 \tag{9}$$

$$V_R = b \cdot d \cdot \tau_{c,u} \tag{10}$$

$$P_b = \frac{\pi^2 \cdot D}{{L_e}^2 + \frac{\pi^2 \cdot D}{U}} \tag{11}$$

$$P_{b,c} = U \tag{12}$$

Where σ_{LtR} is the ultimate tensile stress of faces, ν_c and ν_L are the Poisson rations of core and faces, respectively, s is the cell size and $\tau_{c,u}$ is the ultimate shear stress of the core.

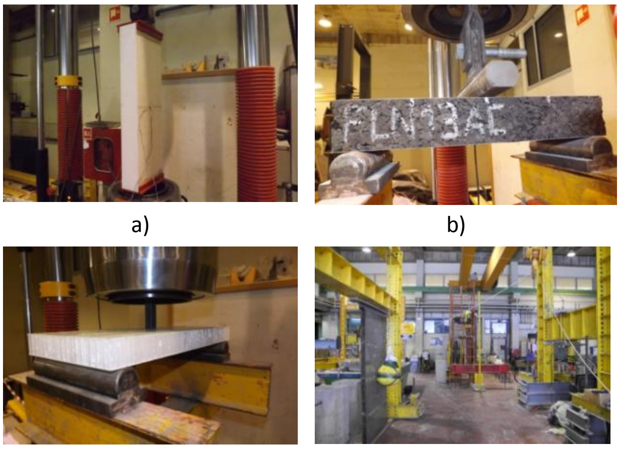
3. Experimental campaign

3.1. Introduction

The goal of this chapter is to characterize the mechanical behaviour of sandwich panels through experimental tests, in order to apply them as floor or wall. The experimental campaign can be divided into three parts. The main goal of the first part is to evaluate the mechanical properties of the constituent materials of sandwich panels studied, analysing also, some of the possible failure modes of these panels. It comprises tensile tests of GFRP faces, flatwise compressive tests, edgewise compressive tests and flexural tests with core shear failure. The second and third parts have the aim of doing an experimental study, taking into account a future utilization as floor and wall panels, respectively. The second part consists of concentrated load tests, flexural creep tests and flexural tests of floor solutions composed by sandwich panels. The last part comprises impact tests.

All sandwich panels were formed with the same type of faces, which had an expected thickness of 1 mm and were composed of a polymer matrix of epoxy resin and a glass fibre textile with an area density of 744 g/m^2 wherein 99 % of fibres were disposed at +45 ˚ and -45 ˚[10]. Regarding the core materials, EPS and ACE had an expected thickness of 80 mm and densities of 15 kg/m^3 to EPS and between 105 and 125 kg/m^3 to ACE [11, 12]. FMPP had an expected thickness of 40 mm and a density of 75 kg/m^3 [13].

In Figure 2 some of test configurations are presented.



a) b) c) d)

Figure 2 – Test configurations: a) flatwise compressive tests (core: EPS), b) flexural tests with core shear failure (core: ACE), c) concentrated load tests (core: FMPP) and d) impact tests (core: ACE)

3.2. Tensile tests of GFRP faces

The tests were conducted according with EN ISO 527-1 [14] and EN ISO 527-4 [15]. A total of 27 GFRP specimens were tested with the following nominal dimensions: height of 250 mm, 50 mm of width and a distance between grips of 150 mm. The samples showed a linear relation between stress and strain in a first stage and, in the last stage, also a linear relation but with a slope close to zero. The majority of failure modes identified were characterized by a sliding between fibres and the polymeric matrix and by the fact that fibres did not fail. The average, standard deviation and the coefficient of variation of maximum tensile stress ($\sigma_{t,max}$), strain for the maximum tensile stress ($\varepsilon_{\sigma,t,max}$) and the elasticity modulus (E) are presented in the Table 2.

Table 2 – Results of tensile tests

Statistical parameter	$\sigma_{t,max}$ [MPa]	$\varepsilon_{\sigma,t,max}$ [-]	E [Gpa]
Average	69.52	0.05888	6.97
Standard deviation	17.49	0.02438	0.99
Coefficient of variation	25%	41%	14%

3.3. Flatwise compressive tests

The tests were performed according to ASTM C365 [16]. Six samples with EPS and FMPP cores and five samples of ACE core were tested. All sandwich panels tested had a square section with a side equal to 100 mm. EPS and ACE cores showed a similar mechanical behaviour, containing three phases. In the first two phases, the relation between stress and strain is approximately linear with the slope of the first one significantly larger than the second stage. In the third stage, called densification, there was an increase of slope between stress and strain as the strain raised. All failure modes identified were by core crushing and, due to the material properties, there was not an instant where tension reduces when strain increases. Thus, according to the ASTM C365 the maximum stress is given by a strain equal to 0.02. FMPP exhibited, approximately, a linear relation between strain and stress until the maximum stress, where the cells wall buckled. The maximum stress (σ_{cmax}) and the elasticity modulus at compression are presented (E_c) in Table 3.

Table 3 – Results of flatwise compressive tests

Statistical parameter	EPS		ACE		FMPP	
	σ_{cmax} [Mpa]	E_c [Mpa]	σ_{cmax} [Mpa]	E_c [Mpa]	σ_{cmax} [Mpa]	E_c [Mpa]
Average	0.047	2.60	0.062	2.98	1.520	62.00
Standard deviation	0.003	0.20	0.005	0.22	0.296	10.13
Coefficient of variation	5%	8%	7%	8%	19%	16%

3.4. Edgewise compressive tests

Edgewise compressive tests were conducted in agreement with ASTM C364 [17]. Four samples of each kind of panel were tested. The EPS and ACE specimens had the following nominal dimensions: height equal to 600 mm and a width equal to 200 mm, while FMPP specimens had the following dimensions: height of 300 mm and a width of 100 mm. All specimens showed an approximately linear relation between force and displacement until failure. Regarding failure modes, in the specimens with EPS and ACE cores the most common failure modes were face wrinkling with core crushing and face wrinkling with adhesive bond failure, respectively. In the samples with FMPP core the most common failure was by shear crimping. The maximum compressive axial stress at faces (σ_{clmax}) on each type of sandwich panel are given in Table 4.

Table 4 – Results of edgewise compressive tests

Statistical parameter	EPS	ACE	FMPP
	σ_{clmax} [MPa]	σ_{clmax} [MPa]	σ_{clmax} [MPa]
Average	16.69	22.37	29.69
Standard deviation	6.95	18.90	38.34
Coefficient of variation	42%	28%	25%

3.5. Flexural tests with core shear failure

The tests were performed according to ASTM C393 [18] and ASTM D7250 [19]. A span of 435 mm and a three point loading configuration were adopted. Two panels of EPS core, five panels of ACE core and three panels of FMPP core were tested. The panels with EPS cores had the particularity that the faces were not the faces studied in 3.2. These faces had a thickness between 2.5 and 3.0 mm and it was assumed that the elasticity modulus was equal to the average value presented in 3.2. All specimens had a nominal length of 500 mm and a nominal width of 250 mm for EPS and FMPP panels and 200 mm for ACE panels. All types of panels showed a linear relation between force and displacement in a first stage where the shear deformation had an important impact. For sandwich panels with FMPP cores, this was the only phase verified, while in the other type of panels, two subsequent stages were verified. The second stage is characterized by a non linear relation between forces and displacements, where the stiffness decreased progressively. In the last stage, there was a negligible range of force as the displacement increased. The failure modes identified were core shear failure in ACE samples, core shear failure and wrinkling with adhesive bond failure in FMPP samples and facing failure by bending and axial force of the upper face. This last mode occurred due to the significant core crushing in the midspan loading zone. The maximum axial stress at faces, the maximum shear stress in the core, the shear stiffness and core shear modulus are given by Table 5.

Table 5 – Results for flexural shear tests with core shear failure

EPS				
Statistical parameter	σ_{lmax} [MPa]	τ_{cmax} [MPa]	U [kN]	G_c [MPa]
Average	4.94	0.06	47.17	2.24
Standard deviation	0.07	0.01	7.47	0.34
Coefficient of variation	2%	11%	16%	15%
ACE				
Statistical parameter	σ_{lmax} [MPa]	τ_{cmax} [MPa]	U [kN]	G_c [MPa]
Average	12.45	0.05	24.62	1.49
Standard deviation	1.38	0.01	3.88	0.24
Coefficient of variation	11%	13%	16%	16%
FMPP				
Statistical parameter	σ_{lmax} [MPa]	τ_{cmax} [MPa]	U [kN]	G_c [MPa]
Average	48.07	0.24	97.98	9.40
Standard deviation	8.97	0.03	7.53	0.74
Coefficient of variation	19%	12%	8%	8%

3.6. Concentrated load tests

These kinds of tests were conducted without following any standard. Three specimens of sandwich panels with FMPP cores were tested with nominal dimensions equals to those presented in 3.5 .The configuration of tests was equal to the explained in 3.5, except for the fact that the midspan loading was applied by a steel bar with a square cross section aside of 20 mm. The relation between force and displacement exhibited two linear stages with stiffness in the first stage equal to about the double of the last stage. The failure modes presented were all preceded by core crushing and occurred by face wrinkling with adhesive bond failure and failure of the upper face along the perimeter of the steel bar. The maximum force (F_{max}), the displacement to the maximum force (δ_{Fmax}), the vertical stress in the area underlying the steel bar (σ_{max}), the stiffness in the first stage (K_1) and the stiffness in the second stage (K_2) are presented in Table 6.

Table 6 – Results for concentrated load tests

FMPP					
Statistical parameter	**F_{max} [kN]**	**δ_{Fmax} [mm]**	**σ_{max} [MPa]**	**K_1 [kN/mm]**	**K_2 [kN/mm]**
Average	3.65	17.05	9.12	0.39	0.21
Standard deviation	0.25	2.41	0.63	0.01	0.01
Coefficient of variation	7%	14%	7%	4%	3%

3.7. Flexural creep tests

The tests were performed in agreement with ASTM C393 [18] and ASTM C480 [20]. The set-up configuration and the nominal dimensions were the same as explained in 3.5. The tests were realized in uncontrolled environment and the midspan load applied was 1 kN. A total of two specimens of sandwich panels with FMPP cores were analysed with test duration of 1032 h. Creep coefficients for 30 and 50 years were estimated through a potential regression curve based on the two samples tested. The displacements recorded after 1032 h were, approximately, twice the initial displacements. The equation (13) gives the displacement (δ_t in mm) in function of time (t in years). In Table 7, coefficient of determination of regression, the displacements and creep coefficients (φ_t) calculated through the regression are given.

$$\delta_t = 2.138 + 3.255 \cdot t^{0.258} \tag{13}$$

Table 7 – Results for the potential regression in flexural creep tests

FMPP				
Coefficient of determination	**$\delta_{t=30\ years}$ [mm]**	**$\varphi_{t=30\ years}$ [-]**	**$\delta_{t=50\ years}$ [mm]**	**$\varphi_{t=50\ years}$ [-]**
0.973	9.97	3.67	11.08	4.18

3.8. Flexural tests of floor solutions composed by sandwich panels

The tests were performed according to ASTM E529 [21]. Three types of floor solutions were analysed and one specimen of each type was tested. A simple supported beam with a span of 4 m and a four point configuration loading with a distance between midspan loads of 1.2 m. The solutions type A (Figure 3) and B (Figure 4) were only constituted by sandwich panels with the difference that the solution type B had an extra GFRP face on the bottom of sandwich panels with EPS core. The solution type C (Figure 5) comprised a sandwich panel with FMPP core and a Z section made of cold formed steel.

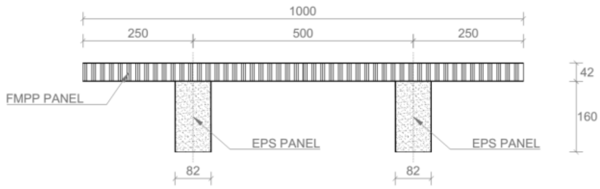


Figure 3 – Solution type A

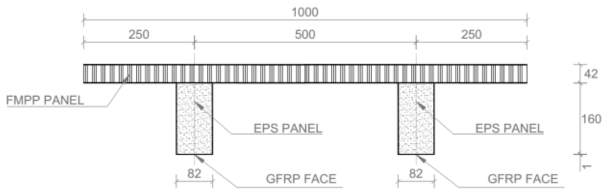


Figure 4 – Solution type B

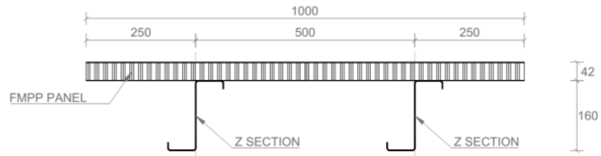


Figure 5 – Solution type C

Load levels were established and in each one of them a total unloading was procedure followed by loading until the next level. In terms of strength and stiffness, solution type C showed the best results and solution A showed the worst results. The failure modes identified were: wrinkling with adhesive bond failure in the upper face of the sandwich panel with FMPP core in the specimen type A, tension failure of the faces of sandwich panel with EPS core in specimen type B and by lateral torsional buckling in the specimen type C. Note that, in the specimen type C, it was demonstrated that the glued connection between the cold formed section and the sandwich panel was significantly favorable. The displacement values for each load level (δ) and the total load applied (F) are presented in Table 8.

Table 8 – Results for flexural tests of floor solutions composed by sandwich panels

Load level	Type A		Type B		Type C	
	F [kN]	δ [mm]	F [kN]	δ [mm]	F [kN]	δ [mm]
1	1.00	17.57	1.00	8.09	1.00	1.21
2	2.00	41.33	2.00	17.34	2.00	2.28
3	3.00	73.91	3.00	27.81	3.00	3.38
4	5.00	197.27	5.00	50.83	5.00	5.37
5	-	-	7.00	82.25	7.00	7.45
6	-	-	-	-	12.00	12.28
7	-	-	-	-	20.00	20.60
Maximum	5.13	260.17	9.89	215.90	30.64	35.58

3.9. Impact tests

The tests were performed in agreement with ISO 7892 [22]and EOTA TR 001 [23], taking into account a future utilization as internal or external walls. In Table 9, the criteria's adopted are presented as well as remaining information required. The impact bodies with 0.5 and 1.0 kg are steel spheres, with a diameter of 50 mm and 62.5 mm, respectively and correspond to a hard body impact. The 50 kg impact body is a spheroconical bag containing glass spheres with a diameter of 3 mm and correspond to a soft body impact. Two specimens of sandwich panels with EPS cores and one with ACE core were tested. The specimens had the following nominal dimensions: 2250 mm of height and 1170 mm of width. All sandwich panels fulfilled the criteria shown in Table 9.

Table 9 – Criteria's and energy impacts for impact tests

Limit state	Impact body mass [kg]	Number of impacts	Impact energies [N.mm]	Criteria
Serviceability limit state	0.5	3	1, 3 e 6	No penetration , no degradation and no collapse
	50.0	3	100, 200 and 300	
Ultimate limit state	1.0	1	10	
	50.0	1	700	

4. Security check of sandwich panels applied in a study case

4.1. Introduction

For the security check of sandwich panels applied in the study case, first the design resistance values were calculated according to the methodology proposed in Annex D – "Design assisted by testing" of the Eurocode 0 [24] in the section: "D7 Statistical determination of a single property". The assessment adopted was "direct assessment of the design value for ULS verifications". Nevertheless, the characteristic resistance values are presented in order to give an idea about the material partial factor. The log-normal distribution, a coefficient of variation unknown and a design conversion factor of 1.0 were considered in all cases. In order to include geometrical deviations in the design resistance values and because the goal of the manufacture is to produce panels with the same height dimensions, the design resistance values were calculated in function of the maximum forces.

4.2. Bending moment resistance

As shown in equation (3), it is reasonable to assume that bending moment resistance depends only on the faces stresses. Thus, according with the experimental campaign, the bending moment resistance is conditioned by the face wrinkling failure. Therefore, for the panels with EPS and ACE cores, the edgewise compressive tests were used and for FMPP core panels the flexural tests with core shear failure were used. Despite the fact that four specimens of each panel were tested in edgewise compressive tests and three specimens of FMPP core panel were tested in the flexural tests with core shear failure, it was considered for calculations, that eight specimens of faces were tested for EPS and ACE core panels and four for FMPP core panels. This is justified, because, in fact, eight faces were tested (two of each panel) and the moment resistance depends on only one face and because four is the minimum number of specimens that the Eurocode 0 allows in these conditions. The values of characteristic M_{Rk} and design M_{Rd} bending moment resistances are presented in Table 10.

Table 10 – Characteristic and design bending moment resistances

	EPS	ACE	FMPP
M_{Rk} [kN.m/m]	0.71	1.24	1.52
M_{Rd} [kN.m/m]	0.30	0.53	0.61

4.3. Axial force resistance

The axial compression force resistance calculated does not take into account the buckling failure, since this failure mode depends on support conditions and panel length. The edgewise compressive testes were the only tests used to calculate the resistances. Thus, the number of specimens considered was four, because in this case failure depends on two faces. In relation to the axial tension force resistance, it depends only on faces and, therefore, the values are equal in the different type of panels studied and the values calculated were based on tensile tests of GFRP faces. The values of characteristic ($N_{C,Rk}$) and design ($N_{C,Rd}$) axial compression force resistances, as well as the values of characteristic ($N_{T,Rk}$) and design ($N_{T,Rd}$) axial tension force resistances are given in Table 11

Table 11 – Characteristic and design axial compression force resistances

	EPS	ACE	FMPP
$N_{C,Rk}$ [kN/m]	13.87	24.25	49.78
$N_{C,Rd}$ [kN/m]	0.97	1.78	6.64
$N_{T,Rk}$ [kN/m]		59.82	
$N_{T,Rd}$ [kN/m]		38.89	

4.4. Bending moment – axial force interaction

The bending moment – axial force design interaction curves are showed in Figure 6. Note that the horizontal straight lines in EPS and ACE panels arise from the different number of specimens considered in the calculation of M_{Rd} and $N_{C,Rd}$ was different.

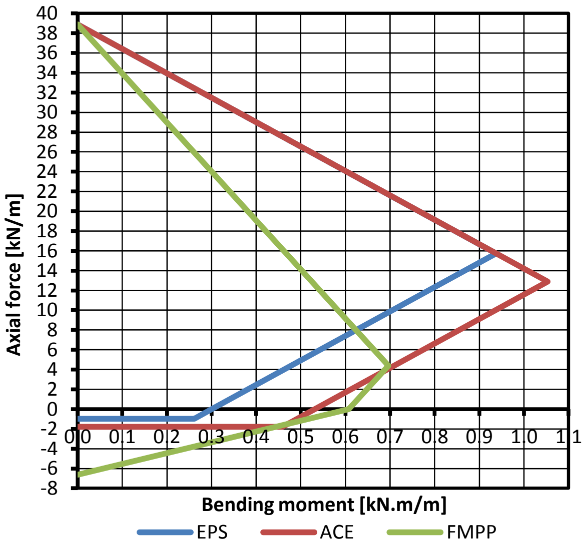


Figure 6 – Bending moment – axial force interaction

4.5. Shear resistance

The shear resistance was calculated through the flexural tests with core shear failure. Since the Eurocode 0 imposes a minimum of three and four specimens for the calculation of characteristic and design values, respectively, these values were adopted in cases where the number of samples was inferior. The values of characteristic (V_{Rk}) and design (V_{Rd}) shear resistances are presented in Table 12.

Table 12 – Characteristic and design shear resistances

	EPS	ACE	FMPP
V_{Rk} [kN/m]	3.74	3.15	7.01
V_{Rd} [kN/m]	1.87	1.48	2.79

4.6. Considerations regarding the design resistances

The design resistance values were calculated only through the experimental campaign and, therefore, there are some aspects that should be noted. The considerable statistical uncertainty due to the reduced number of specimens tested, the option of taking the design conversion factor of 1.0, the adoption of unknown coefficient of variation and a lognormal distribution are some factors that could change significantly the design values. An analysis in order to evaluate the impact of a smaller statistical uncertainty was performed, assuming that the number of specimens tested was 30 in all test types and the statistical parameters remain equal. This analysis was not done for tensile tests, since the number of specimens is almost 30. The results of this analysis are shown in Table 13.

Table 13 – Design values assuming that 30 samples were tested and statistical parameters remain equal

	EPS	ACE	FMPP
M_{Rk} [kN.m/m]	0.77	1.33	1.84
M_{Rd} [kN.m/m]	0.47	0.83	1.51
$N_{C,Rk}$ [kN/m]	18.23	31.71	61.21
$N_{C,Rd}$ [kN/m]	10.85	19.05	41.32
V_{Rk} [kN/m]	4.30	3.42	8.46
V_{Rd} [kN/m]	3.71	2.71	6.95

4.7. Case study

The case study is called *Casa Moçambique*, is localized in Benavente, Portugal and is intended for single-family housing. It is a two floor building with an implantation area of 112 m^2 and a covered area of 128 m^2. The wall and roof solutions are constituted by sandwich panels with EPS cores and the elevated floor solution is equal to the solution type C, presented in 3.8. The elevated floor plant, the roof plant and two cutaway views are shown in Figure 8. In this figure, T represents tie (steel bar with 10 mm of diameter), VS secondary beam (equal to the steel section presented in 3.8) and VP principal beam (similar to beam presented in 3.8 but with a height of 260 mm). Images of the case study in a state close to finished are presented in Figure 7.

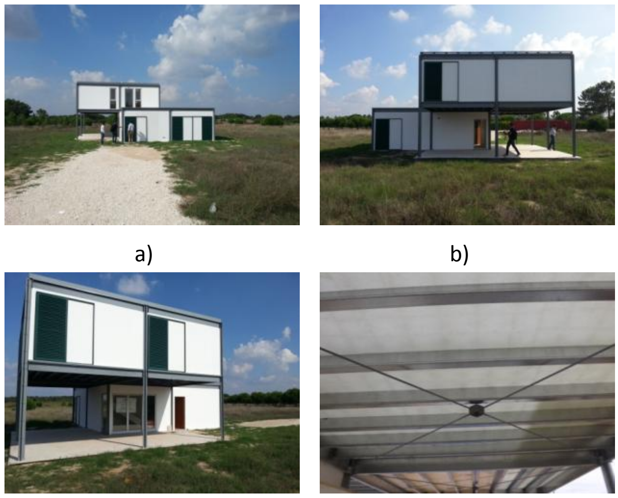


Figure 7 – *Casa Moçambique*: a), b) and c) side views and c) view of the floor solution

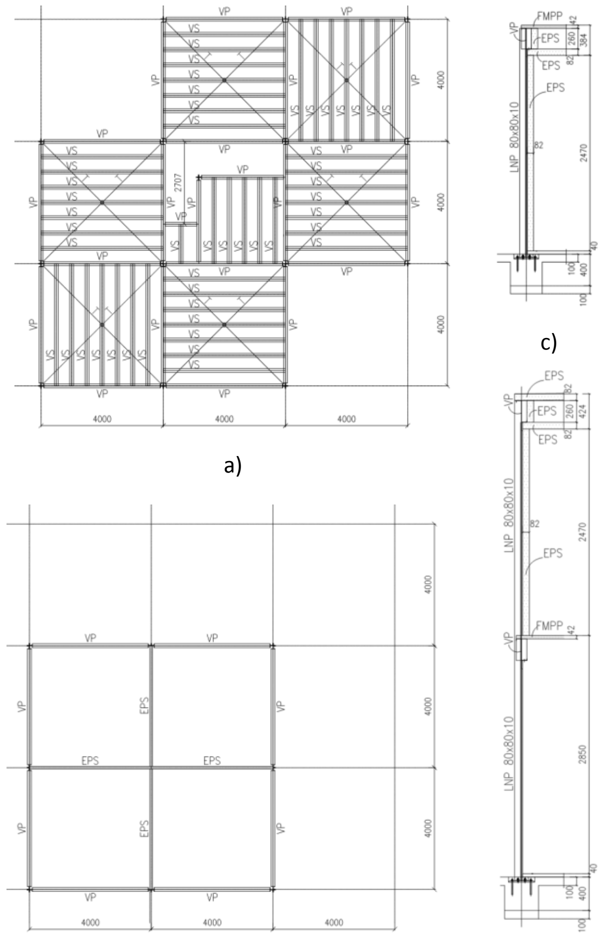


Figure 8 – *Casa Moçambique*: a) Plant of elevated floor, b) roof plant, c) and d) two cutaway views

4.8. Security checks of sandwich panels solutions used in the case study

The security checks were done in accordance with Eurocode 0 [24] and Eurocode 1 [25]. The verifications for ultimate limit state are presented in Table 14 and the verifications for serviceability limit state are presented in Table 15. The verifications for serviceability limit state comprises deflections (taking into account creep) and maximum axial stresses in the faces, in order to avoid creep failure. Furthermore, verifications for ultimate limit state were done assuming the scenario presented in Table 13. These verifications are shown in Table 16.

Table 14 – Summary of verifications for ultimate limit state

Function	Leading variable action		
	Distributed imposed load	**Horizontal load/ concentrated imposed load**	**Wind**
Floor	✓	X	N.A.
Roof	X	X	X
External wall	N.A.	X	X
Internal wall	X	N.A.	✓

✓ – Checks; X – Does not check; N.A. – Not applicable

Table 15 – Summary of verifications for serviceability limit state

Function	Leading variable action	
	Distributed imposed load	**Concentrated imposed load**
Floor	✓	X

✓ – Checks; X – Does not check; N.A. – Not applicable

Table 16 – Summary of verifications for ultimate state, assuming the scenario presented in Table 13

Function	Leading variable action		
	Distributed imposed load	**Horizontal load/ concentrated imposed load**	**Wind**
Floor	✓	✓	N.A.
Roof	X	X	X
External wall	N.A.	✓	X
Internal wall	✓	N.A.	✓

✓ – Checks; X – Does not check; N.A. – Not applicable

5. Conclusions

EPS and ACE cores showed mechanical characteristics very similar. In relation to FMPP core, unlike remaining types of cores where the behavior can be considered isotropic, this presents a different behavior in plane and in the vertical direction (stiffer direction). The sandwich panels with FMPP core demonstrated a significantly better behavior in terms of strength and stiffness.

The floor solutions composed only with sandwich panels presented a significantly worse mechanical behavior compared with a solution with cold steel formed sections of dimensions similar to the sandwich panels.

Regarding impact loads, sandwich panels with EPS and ACE cores fulfilled all requirements to apply as internal or external walls.

None of the panels applied on the case study checked all verifications. This can be justified by the fact that the amount of specimens to each type of tests performed was small and to the elevated coefficients of variation registered. Regarding the first reason appointed, through an analysis performed, it was concluded that an increase in the number of specimens is not enough to fulfill the verifications. Therefore, it is recommended that the producer improves the quality of sandwich panels fabricated, thus increasing the average values and reducing the coefficients of variation. In a second approach, it is suggested a change of the sandwich panel solutions, reducing the spans that panels have to support. In relation to the safety problem due to concentrated load, it is suggested to use a thick coating material in order to degrade the load along the thickness of this.

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