1. Introduction

The new targets on CO₂ emissions urged engineers for the development of lightweight solutions for the building industry. Sandwich panels, as very lightweight structures, became popular for their good properties on thermal, acoustic and fire but also on their mechanical properties with high load-bearing capacity at low weight [1]. The correct manipulation of the core thickness and strength may allow for a straightaway high payload [2]. However, this easiness in the sandwich panels manipulation contrasts with the complexity of the design. Depending on the geometry, loading and boundary conditions, different failure modes can occur, namely: face yielding, core shear, wrinkling, flexural buckling, shear crimping, face dimpling and local indentation [3,4]. Thus, although sandwich panels have been used since the sixties from previous century, there are no consistent standard rules covering all its design.

Major efforts have been done towards a unified design code. For example, the work on progress by CEN/TC 250/SC 3/WG 21 for the pr EN 1993-7 – Design of Sandwich Panels. The CEN Committee TCI28/SCI1 responsible for preparing a European product standard for sandwich panels, and the work of the Joint Committee ECCS TWG 7.9, are some examples.

1.1. Applications

Sandwich structures were primarily developed in the field of aeronautical and rocketry, making the shell of the spacecrafts able to resist the stresses of acceleration and landing [5,6]. Also, on other different industries such as automotive and marine industry [2]. Sandwich panels have a wide range of applications such as: roofing systems, facades, cladding walls [7,8], bridge decks [9] and prefabricated emergency housing systems [10,11]. Moreover, regarding to its structural properties, they are utilised for the restraint of columns [12], load bearing structures [13], impact loading [14] and for diaphragm action on roofs [15].

Typically, sandwich panels are used as covering elements of buildings and designed only to bear transverse loads – such as snow and wind – transferring them to the subtract by bending. Nowadays, an especial interest has arisen on the use of sandwich panels on small buildings – for instance, cooling and climatic chambers and clean rooms. Consequently, to the stress resultants arising from transverse loads, the normal forces must also be taken by the wall panels [16]. This design strategy has been transposed to other fields of engineering as in industrial construction.

Industrialised construction can be considered as a business strategy that transforms the traditional construction process into a manufacturing and assembly process [17]. Alongside with Modular Construction, normally creating cellular-type buildings of similar room sized units suitable for transportation [18]. Can include Volumetric Units, as for instances, modular rooms; Planar Construction of two-dimensional elements, such as walls and cladding panels for facades. As two-dimensional elements, sandwich panels can possess much of the properties requested for walls and their easiness of manipulation and production makes them very appealing for Modular Construction.

Regarding to the structural nature of volumetric units – also known as modules – can be found four-sided modules, where vertical loads are transmitted through the walls and sometimes also by additional steel angles and corner-supported modules where with vertical loads...
1.2. Sandwich components

The face sheets of the sandwich panels carry the tensile and compressive stresses in the sandwich. Conventional materials such as steel, stainless steel and aluminium are often used however other materials can be found. From the mechanical point of view of the steel, the yield stress is generally of reduced interest, as the load-bearing capacity is normally determined by wrinkling of face in compression rather than yielding of the face material.

When it comes to the core, it covers different purposes in sandwich panels. It must be stiff enough to keep the distance between face sheets and rigid (in shear) enough to minimise the relative slip, so face sheets cooperate with each other, maintaining the stiffness. In addition, there are other demands, such as: buckling, insulation, absorption of moisture, ageing, resistance, acoustics, and fire. A typical core is made of polyurethane foam, polystyrene, or mineral wool.

The core made of polyurethane foam is a continuous structure, while the core made of polystyrene or mineral wool is discontinuous. Mineral wool is cut from mineral wool slabs in elongated blocks called lamellae. The lamellae are rotated 90 degrees in the sandwich structure, which provides higher stiffness to the core. The contact surfaces between lamellae are not glued. Therefore, the core can be assumed as non-continuous with longitudinal and transversal discontinuities. However, in most cases assumed as continues for simplification. Although mineral wool has a lower insulation capacity in comparison to PIR and PUR, it exhibits a much better behaviour in fire which is highly convenient to what walls concerns, especially in the context of housing.

1.3. Background information on the proposed solution

Taking advantage of the benefits of sandwich panels in construction, in conjunction with industrialised construction, it is assessed the detail of the corners in sandwich walls. In this evaluation, it is privileged the quick assembly and other important factors such as thermal, acoustic, airtightness, material savings and fire properties of the walls, prior to load bearing capacity. Two solutions for corners in walls are under evaluation: first one based on the (a) composition of two typical flat panels and a second one done by means of an (b) angle panel in a monolithic element (see Fig. 1). First solution relies on the common practices used in corners, by prolonging one of the panels over the other one’s thickness. This solution requires that panels are restrained through angle profiles fastened against both panels face sheets. Whereas, in solution (b), the use of an angle panel, comprising of a monolithic panel with a corner shape, is considered, and so, no further work is required on site. In solution (a), the angle profile is fastened by means of screws and therefore connecting forces are discontinuous along the joint. On the other hand, in solution (b), the monolithic panel constituted of bended face sheets constitutes by itself the walls corner. Both solutions have been used for many years, however, the crescent interest in industrialised construction, and fast construction, has raised attention in these types of solutions, as solution (b).

The panel in solution (b) is manufactured in a specific apparatus where lamellae that constitute the core are placed so that the pre bended face sheets can be glued to mineral wool, and like ordinary panels, mineral wool lamellae are not glued together.

The benefit of the angle panel, in opposition to corners made by composition of panels, lies, most of all, in the fact that no steel is crossing through the wall thickness. This is the main factor that ensures the continuous and constant performance of the wall throughout the whole length, to what thermal, acoustic, airtightness and fire resistance concerns.

Moreover, for complementing previous solution, the angle panel connection to adjacent panels it is performed by means of a c-joint rather than the typical splice connection with steelprofiled sheets fastened to both panels. The c-joint requires no special tool for installation and has a long history of use in sandwich panels, see Fig. 2.

Unlike solution (a), in Fig. 1, where angle is fastened against wall panels, by means of fasteners discontinuously installed, the use of a c-joint in conjunction with angle panel ensures the quick mounting and the continuous restraint of adjacent panels throughout the whole length. This connection has been used for about 50 years in the industry, marine and offshore structures, and, among different advantages, it can also ensure a high level of airtightness, given its strong fitting to the slits, of the curled sheets, of the panels joints.

The angle panel in question is made of two steel sheets, which resemble to the thin-walled angular sections, providing a good insight to what local instabilities may concern [20,21].

Thus, given the importance of the elements in the corner of the modules, a sandwich angle is investigated, and its potential in load bearing capacity assessed, where for simplification the angle panel is analysed alone and without c-joints.

2. Theory

Sandwich panels have flexible cores, and their behaviour is therefore more complex than that of the plain plates. Therefore, it is important to understand the numerous failure modes of sandwich panels so that appropriate design criteria can be developed. They mainly carry applied loads as tensile and compressive stresses in the two face-sheets, whereas applied transverse forces predominantly are carried by the core material as shear stresses [16].

To assess angle panel full potential, this paper tackles the linear elastic buckling response of axially loaded angle sandwich panels, as the upper bound for load bearing capacity. The failures modes are obtained and framed for concentrically loaded angle panels with fixed

<table>
<thead>
<tr>
<th>Nomenclature</th>
</tr>
</thead>
<tbody>
<tr>
<td>List of symbols</td>
</tr>
<tr>
<td>$A_{F1}/A_{F2}$ Cross section area of face sheets</td>
</tr>
<tr>
<td>$A_r$ Area of the core</td>
</tr>
<tr>
<td>$B_s$ Bending rigidity of the panel</td>
</tr>
<tr>
<td>$D_f$ Bending stiffness of the steel faces</td>
</tr>
<tr>
<td>$E_{c_x}, E_{c_y} E_{c_z}$ Modulus of elasticity of the steel</td>
</tr>
<tr>
<td>$e_i$ Initial deflection, global imperfection</td>
</tr>
<tr>
<td>$f_y$ Nominal yield stress</td>
</tr>
<tr>
<td>$G_{c_x}, G_{c_y} G_{c_z}$ Shear modulus of the core</td>
</tr>
<tr>
<td>$G_A$ Shear rigidity of the core</td>
</tr>
<tr>
<td>$L$ Length of the angle panel</td>
</tr>
<tr>
<td>$N_{ki}$ Axial force of the panel</td>
</tr>
<tr>
<td>$N_w$ Axial wrinkling force</td>
</tr>
<tr>
<td>$t$ Thickness of the steel sheets</td>
</tr>
<tr>
<td>$U_x, U_y, U_z$ Displacements</td>
</tr>
<tr>
<td>$\lambda$ Slenderness</td>
</tr>
<tr>
<td>$\lambda_{ki}$ Bending slenderness</td>
</tr>
<tr>
<td>$\lambda_{GA}$ Shear slenderness</td>
</tr>
<tr>
<td>$\sigma_w$ Wrinkling stress</td>
</tr>
</tbody>
</table>
and pin-ended supports. The following chapters tackle global and local instabilities/buckling.

Elastic critical buckling load does not consider geometrical and material imperfections, therefore, it does not constitute the resistance of the panel, instead, it represents a theoretical value that sets the upper limit of the angle panel in terms of axial load, i.e., it provides the full potential of the panel as load bearing structure. Nonetheless, the requirements for the calculation of resistance of the panel, in resemble to EN 1993−1−1 [22] with steel profiles, are presented in the following chapter to foreseen future requirements as part of the assessment of angle panels.

2.1. Global buckling

Considering a sandwich panel that is uniaxial loaded and assuming no imperfections present the panel will buckle according to Euler theory. Euler load represents the smallest value for an axial load \( N_{cr} \) at which the element will not return to straight condition after being displaced in lateral direction. This sets the limit on the pure elastic response. Two modes of elastic buckling are possible: a Euler buckling mode involving bending of the sandwich column and a core shear mode [23].

The elastic buckling load due to bending rigidity is expressed as:

\[
N_{ki} = \frac{\pi^2 \cdot B_s}{L^2}
\]  

(1)

where \( B_s \) is the bending stiffness of a panel and later detailed concerning the angle panel. Regarding to the elastic buckling load due to shear rigidity can be estimated as follows:

\[
GA = G_c \cdot A_c
\]  

(2)

Thus, the elastic buckling load \( N_{cr} \) for a compression member combining both is given by:

\[
N_{cr} = \frac{N_{ki}}{1 + \frac{N_{wi}}{G_c \cdot A_c}}
\]  

(3)

The elastic buckling load is associated mainly to the length of the panel whereas shear rate \( GA \) is independent of the length.

If imperfections and 2nd order effects are neglected the axial load will culminate into the wrinkling of the panel in both face sheets. The “axial wrinkling force” \( N_{wi} \) can be calculated by the following equation,

\[
N_{wi} = \sigma_w \cdot (A_{F1} + A_{F2})
\]  

(4)

The axial force leading to wrinkling of the panel resembles the plastic normal force of a steel profile. Thus, the slenderness of the sandwich angle panel can be calculated as follows,

\[
\lambda = \sqrt{\frac{N_{wi}}{N_{cr}}} = \sqrt{\frac{\lambda_{ki}^2 + \lambda_{GA}^2}{\lambda_{cr}^2}}
\]  

(5)
The wrinkling wave is a periodic wave with the (half) wavelength \( \lambda \) investigated here. Wrinkling can occur simultaneously all over the surface which leads to:

\[
\sigma_w = k \sqrt{\frac{E_f E_c G_c}{500}}
\]

With bending slenderness:

\[
\lambda_b = \sqrt{\frac{N_w}{N_{Ei}}} \quad (6)
\]

and shear slenderness:

\[
\lambda_{GA} = \sqrt{\frac{N_w}{G_A}} \quad (7)
\]

At the same time shear rate imposes an upper limit of the elastic buckling load, the shear slenderness \( \lambda_{GA} \) represents a lower limit of the sandwich panel slenderness.

Global buckling constitutes the first buckling mode of a sandwich panel loaded by a centric axial force for a slenderness of the panel \( \lambda > 1 \). Whereas for \( \lambda < 1 \), the first buckling mode corresponds to wrinkling of the face sheets. Thus, for global buckling to occur, the panels must be thin, and the face sheets must be relatively stiff or thick. The global buckling load, however, is only reached for panels with rather unusual slenderness regarding building practice. To what length concerns, global buckling takes place on rather long panels although for most common dimensions, failure of panels occurs by wrinkling or shear failure [24].

The calculation of resistance of the panel based on the ultimate compression stress considers a global initial deflection, \( \varepsilon_0 \). According to EN 14509 [25] the tolerances are of 2,0 mm for each metre length, which leads to:

\[
\varepsilon_0 = \frac{1}{500} \cdot L \quad (8)
\]

Calculation of the resistance due to global buckling instability can be performed where amplification of deflections through 2nd order effects must be considered.

2.2. Local buckling

Local buckling can take the form of localised buckling, dimpling and wrinkling. To what local instabilities due to loading is concerned, wrinkling is perhaps the most dominant local instability and therefore investigated here. Wrinkling can occur simultaneously all over the surface of the face-sheets [26]. One way to calculate the critical wrinkling load is to consider the panel as a beam on an elastic foundation [23]. The wrinkling wave is a periodic wave with the (half) wavelength \( \lambda \) as shown in Fig. 3 (see Fig. 4).

Which follows the differential equation Eq. (9):

\[
P_k = D_f \frac{d^4 w}{d x^4} + P \frac{d^2 w}{d x^2} - \sigma_z = 0 \quad (9)
\]

where \( D_f \) is the bending stiffness of the steel facing alone, \( P \) is the compression load acting on the steel facing and \( \sigma_z \) the support pressure from the core, the elastic foundation.

Thus, wrinkling on sandwich panels are dependent on the material properties and geometry of the core and face-sheets and can manifests itself in different ways, as represented in Fig. 5 [23].

The exhibit of the different wrinkling cases is dependent upon the geometry, mainly thicknesses, and material properties. In Case I, the "Rigid base", only one face wrinkles. This phenomenon occurs for instance in bending, when only one face is in compression. The second mode (Case II) is the anti-symmetrical and the two face sheets wrinkle in phase. This shape is presented in sandwich configurations with a thin core (as capture in angle panel and shown in later Fig. 13). In Case III, known as symmetrical wrinkling, out-of-phase or "hourglass" wrinkling is a special case of rigid base wrinkling with a smaller distance to the rigid plane. This creates a firmer support of the face sheet, producing shorter wavelengths and making critical load higher [2].

Based on the previous wrinkling instabilities, wrinkling stress can then be derived in different ways, however, a common structure can be identified among different authors, as show below:

\[
\sigma_w = k \sqrt{\frac{E_f E_c G_c}{500}} \quad (10)
\]

Examples of that are introduced, for instance, in Plantema’s approach with \( k \) equal to 0.825. On the other hand, Hoff and Maartment [27] suggest a \( k \) of 0.51. In line with previous formulations of wrinkling, in EN 14509 [25], it is suggested similar approach but where \( k \), in the absence of large-scale tests, may conservatively be considered as 0.5. These are formulations used for regular sandwich panels for the calculations of resistance.

3. Parametric study

A parametric study/analysis, also known as a design sensitivity analysis (DSA), is the study of the influence of different geometric or physical parameters or both on the solution of the problem. A parametric study can be formed if the mapping between CAD and simulation models, where in this case by means of finite element models, are consistent. The parametric study is possible for concept design given the low number of variables to perturb [28].

A parametric study on the angle panel is carried out by means of a series of finite element models comprising different geometries to assess its different load responses. The study involved the development of a script in Python to operate in Abaqus for modelling and post processing of results, aided by a cluster for running calculations. This process has taken "only" about 10 days in total to perform thanks to supercomputer Kebnekaise at HPC2N and to the hours and processing nodes allocated for this end.

3.1. Geometry

Equal and non-equal-leg angles are considered with thin steel face sheets and a mineral wool core. The angle steel facings are made of similar and dissimilar thicknesses. The orthotropic core is composed of mineral wool where lamellae are perpendicularly oriented towards the angle faces.

The angle panel or corner panel, constitutes the connection of two concentric walls made of sandwich panels. The short segments of wall
of the angle panel are hereby considered as the legs of the corner angle. Thus, can be distinguished: a (1) left-leg length, (2) right-leg length, (3) left-leg thickness, (4) right-leg thickness and, to what steel facings concerns, (5) inner and (6) outer face-sheet total thicknesses and finally (7) angle length, as shown in Fig. 6.

In Fig. 7, the angle corner is composed of a bended (1) outer face-sheet and a (2) inner face-sheet made of steel and a soft core made of mineral wool. Mineral wool is considered as continuous along the full thickness of the wall so not to compromise the thermal and fire aspects of the walls. The mineral wool lamellae, arranged longitudinally along the wall, required to be prolonged in one of the walls creating naturally a longer lamella, Lamella 1; whereas, the other segment of wall, in the angle corner, has a shorter lamella on the right-leg, Lamella 2. Lamellae are oriented with its highest stiffness perpendicular to the face-sheets for stabilisation of both face-sheets. This direction - x-direction - is kept on both legs, as shown in Fig. 7, but given the nature of the corner, exposes inevitably its lower component of stiffness in the longer lamellae, at the outer face-sheet, in the right-leg, as highlighted further below in Fig. 18. It is worth mentioning that this discontinuity in stiffness triggers buckling, leading to the loss of load-bearing capacity of the angle sandwich panel, in the form of local buckling, which is further discussed in Section 6.1.

The sandwich corner angle entails two thin faces, each of which may have dissimilar thicknesses \( t_{\text{left,leg}} \) and \( t_{\text{right,leg}} \), separated by a thick core of low-density material (mineral wool) of which may have different thicknesses for left and right-leg \( c_{\text{left,leg}} \) and \( c_{\text{right,leg}} \), respectively) and Elastic Modulus of \( E_{x,x} \), \( E_{y,y} \) and \( E_{z,z} \), oriented as in Fig. 7, where \( E_{x,x} \) possesses the highest stiffness. The corner’s total length \( L \) and left-leg length \( l_{\text{left,leg}} \) and right-leg length \( l_{\text{right,leg}} \) may vary as well (see Fig. 6).

The parametric test includes four different leg lengths for the left- and right-leg and three different thicknesses for each leg, as per Fig. 6, and stated in Table 1; thirty lengths and two thicknesses of the steel sheets. Results lead to a total of 8640 finite element models which are generated for analysis, as in Table 1.

The chosen thicknesses for the parametric study are in line with common thicknesses used in sandwich panels. Whereas, for the leg lengths both lower and higher dimension are coincident with the thresholds for production of the angle panel. Regarding the lengths of panels, they are distributed so to capture both local and global buckling phenomena. A 500 mm step between consecutive lengths is used with the intention of aiding on framing the establishing of the boundaries for local and global buckling. These models are automatically generated within the software Abaqus by means of a script in Python developed for this end where each one of the previous parameters are defined in combination.

3.2. Analysis method

For the analysis of the angle panel, a series of numerical and analytical analysis are conducted.

In what concerns numerical analysis, a pure elastic buckling is conducted and so no imperfections are added. Eigenvalue buckling analysis is used to estimate the critical loads. An incremental load pattern was defined in “Buckle step” [29] in opposition to unit load to improve speed in calculation. A general eigenvalue buckling analysis can provide useful estimates of collapse mode shapes. Only first buckling mode is computed as it provides sufficient information on the critical loads. Moreover, software allows for the captured of local buckling in the form of overall wrinkling of the facings sheets and overall bowing of angle panel as per global buckling typical shape. In order that global buckling occurs, the panels must be slender, and steel facings must be relatively thick [16]. The finite element models do not take in consideration imperfections, so the elastic load increases up to the elastic buckling load.

Regarding analytical calculations, different authors are analysed, and same methods used in regular sandwich panels are applied to angle panels.

4. Numerical calculations

Numerical calculations are carried out by means of finite element modelling of a sandwich angle panel using Abaqus software.

4.1. Finite element analysis

Numerous failure mechanisms are investigated and described in various research papers, for instance, it can be mentioned: local instability (wrinkling of facing) [30,31], shear of the core [32,33], global buckling [34], debonding, based on concentrated dynamic loads [35] and indentation caused by the crushing of the soft core under the acting
force [36]. Naturally, the analysis of these phenomena requires the use of appropriate models [37].

Regarding the core, besides the discontinuous nature of the mineral wool, most studies on core assume an equivalent homogeneous material and the perfect bonding is assumed between the facesheets and the core. Regarding the core, few papers concern the discontinuous core of mineral wool. There are considerable difficulties in finding complete information about the properties of mineral wool used in sandwich panels. One of the few papers devoted to the structural behaviour of sandwich panels with mineral wool concentrates on wrinkling phenomenon is [38]. In the paper [39], the results of laboratory tests and numerical simulations of panels with a hybrid core were presented. The influence of different proportions of mineral wool and polyurethane foam on the results of the four-point bending tests were investigated. It should be clearly noted that several research projects dedicated to panels with mineral wool core dealt with issues of fire resistance [40]. Structures with a discontinuous core are very interesting because of their complex structural behaviour, impossible for providing an accurate analytical description.

The linear elastic buckling response was investigated using Abaqus simulation software on different angle sandwich panels’ geometries. Three-dimensional finite element models of the angle panels were built. For sake of simplification and comprehension of the angle behaviour, it is disregarded the longitudinal joint used in coupling of adjacent panels, c-joint in Fig. 2, as it would stiffening the panels edges and so overestimating the effect of the angle panel.

The angle sandwich panels are modelled with two thin-walled angular sections – that constitutes the steel facings – perfectly bonded to an equivalent homogeneous and orthotropic core of mineral wool.

The failures modes are obtained and framed for concentrically loaded angle panels with fixed and pin-ended supports. Equal and non-equal-leg angles are considered with thin steel face sheets and a mineral wool core. The angle steel facings are made of similar and dissimilar thicknesses.

### 4.1.1. Modelling

Steel facings are modelled with shell elements where fineness of the mesh is adjusted to capture wrinkling. A 10 mm, equally spaced mesh – longitudinally and transversely – is considered to accommodate different wave lengths correspondent to the expected eigenmode of the wrinkling at the steel facing [10,41].

Core of the panel is also modelled with equal mesh size of 10 mm with solid elements – bricks C3D8R – so to match with the mesh of the steel facings, for a good agreement between both.

The interface steel facings and mineral wool core is modelled as tied constraint to the centre of geometry of the angle, therefore, no eccentricities are introduced (see Fig. 8).

Material properties of steel facings data are based on nominal values (see Table 2).

### 4.1.2. Boundary conditions

The ends of the panel are tie constraint to the centre of geometry (CG). The angles sandwich panels are then concentrically loaded at the centre of geometry of the angle, therefore, no eccentricities are introduced (see Fig. 8).

Some applies for supports which are located at the centre of geometry. The present study tackles the angle sandwich panels with fixed and pin-ended supports (see Fig. 9).

<table>
<thead>
<tr>
<th>Left-leg thickness</th>
<th>Right-leg thickness</th>
<th>Left-leg lengths</th>
<th>Right-leg lengths</th>
<th>Thickness of the outer face-sheet</th>
<th>Thickness of the inner face-sheet</th>
<th>Angle length</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>50</td>
<td>125</td>
<td>125</td>
<td>0.7</td>
<td>0.7</td>
<td>500</td>
</tr>
<tr>
<td>80</td>
<td>80</td>
<td>150</td>
<td>150</td>
<td>1.0</td>
<td>1.0</td>
<td>1000</td>
</tr>
<tr>
<td>100</td>
<td>100</td>
<td>175</td>
<td>175</td>
<td>1.0</td>
<td>1.0</td>
<td>14500</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15000</td>
</tr>
</tbody>
</table>

Table 2

<table>
<thead>
<tr>
<th>Density, t</th>
<th>Thickness, t</th>
<th>Elastic Modulus, (E_t)</th>
<th>Poisson ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg m(^{-3})</td>
<td>mm</td>
<td>N mm(^{-2})</td>
<td></td>
</tr>
<tr>
<td>7850</td>
<td>0.7 and 1</td>
<td>210 × 10(^{3})</td>
<td>280</td>
</tr>
</tbody>
</table>

Table 3

<table>
<thead>
<tr>
<th>Density, (t)</th>
<th>Elastic Modulus, (E_t)</th>
<th>Shear modulus, (G_t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg m(^{-3})</td>
<td>N mm(^{-2})</td>
<td>N mm(^{-3})</td>
</tr>
<tr>
<td>180</td>
<td>4.75</td>
<td>0.74</td>
</tr>
<tr>
<td>1.23</td>
<td>3.02</td>
<td>0.52</td>
</tr>
<tr>
<td>3.14</td>
<td></td>
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</tr>
</tbody>
</table>

Fig. 8. Ends tied constraint to the centre of geometry (CG).
Since thin faces are used the assumption taken for fixed and pin-ended supports can be used [43].

4.1.3. Model validation

For validation of the finite element model properties, three angle sandwich panels are tested (see Fig. 10).

As previously mentioned, the model considers a full bond between mineral wool and face sheets. However, the debonding seen in the figure occurs after the first drop in load capacity and continued thereafter without further increase of load.

The tests are conducted with the fix-ended angle panels due to easiness in performing the connection and testing, leading to local instability. The numerical model, namely the mineral wool mechanical properties of the angle panel, are fine-tuned (calibrated) against testing results as shown in Fig. 11. Panels are manufactured with high quality control to reduce deviations from nominal values.

A difference in numerical and testing results is observed. Results in Fig. 11 show that for Test 1 the analysis under predicts the failure load. While for Test 2 the analysis over predicts and for Test 3 fits to the numerical results. This is because model calibration was conducted to best fit the three tests which would naturally lead to scatter in results. However, some aspects that may have originated the deviation from the numerical results may be related with the scattering nature of the mineral wool and its manipulation. For example, mineral wool suppliers provide information in the product declaration that density, which is very much related to mechanical properties of mineral wool, may vary of about 10% from nominal values. For instances, in a typical mineral slab, can often be observed small but distinct agglomeration of small rocks in the wool, constituting a large discontinuity in material properties. In opposition to mineral wool, steel, and the adhesive, are more stable materials but still prone to geometrical imperfection which may arise from manufacturing.

In sum, modelling of the angle panels considers testing results in calibration. The only factor considered in calibration is the mineral wool strength, in all directions, but with especial weight on the direction parallel to angle thickness, where buckling is triggered. Other factors, such as, geometrical imperfections, are disregarded in the model due to the limitations intrinsic to elastic buckling analysis. For this reason, geometrical imperfections are minimised in manufacturing of angle panels by means of employment of a rigorous quality control in production to minimise deviations from nominal dimensions, such as: length, thicknesses, straightness, angle and bowing of the steel sheets.

Thus, the parametric study on the angle panel is carried comprising a series of finite element models with different geometries based on elastic buckling analysis by means of eigenvalue analysis used in the software. The script developed in Python operates in Abaqus for modelling and post processing of results, aided by a cluster for processing calculations.

4.2. Mesh sensitivity

To achieve reliable results, a mesh sensitivity study is conducted based on the elastic buckling response of the angles. As mentioned, the finite element model implied the choice of shell elements for the steel facings while, in the solid core, brick elements are considered.

The choice on the shortest angle for this study with 200 mm equal-leg length and 500 mm total length lies on the fact that it exhibits the higher critical load response and more pronounced wrinkling (see Fig. 13). Local instability – in the form of wrinkling – is to be expected for shorter angle panels. The fineness of the discretisation that is required is related to the expected wrinkling wavelength. Furthermore, a good agreement between the mesh size of the shell and the brick elements shall, as much as possible, be coincident, for a more accurate and easiness in convergence of the models. These two factors are judged upon the elastic buckling response mainly in the form of wrinkling due to its sensitivity to the mesh density. The study implies the variation in size for each one of the brick elements – that constitute the core – and shell elements in the facing of the angle. Ranging from 2 to 16, on intervals of 2 mm sized elements (see Fig. 12).

It is noted in Fig. 12 that the shell elements are more sensitive to shell variations then brick elements. This can be noticed by the fact that for bricks sizes, only two intervals of buckling responses are detected, while in shell elements, a total of seven intervals of buckling responses possibilities are observed. A mesh of 10 mm, for shell and brick elements is chosen given that it is one of which gives the highest elastic buckling response to the least shell size and closest to Test 1 load response.

4.3. Analysis

As mentioned before, to analyse the local buckling phenomenon, the shortest angle with 200 mm equal leg length, 500 mm total length and 50 mm thicknesses is chosen, as it exhibits the higher elastic buckling response. The wrinkling shapes from this short panel are shown in the following Fig. 13.

The buckling shape of the angles in previous figure exhibit an anti-symmetrical shape, as the two face sheets wrinkle in-phase, which is typical from sandwich configurations with a thin core, as mentioned and shown in Fig. 5. Whereas thicker angle panels, as 100 mm, exhibit a symmetrical wrinkling with out-of-phase wrinkling (see Fig. 14).

However, when it comes to the buckling shapes having the two legs in comparison, it is noticed an anti-symmetrical shape in the first two buckling models, whereas, in the last two buckling modes shown, both legs buckle symmetrically. Finally, it is also worth noticing that elastic buckling responses for different buckling modes are relatively close from each other which reinforces the unstable nature of sandwich panels.

Regarding flexural buckling, numerical results shown a clear bow in the panels in longer angle panels. However, results from parametric
study show that, in a narrow interval, angle panels exhibit a couple buckling, where wrinkling and bending of the panel, typically from global buckling, are identified in combination. Since a 500 mm step is used in the parametric study, the phenomenon of couple buckling is only possible to be identified in few panels, showing how narrow the interval can be. The following plot (Fig. 15) gives a perspective of the local and global intervals for same panel mentioned in Fig. 13 but covering all its lengths.

Considering other angle panels, with equal leg lengths, it can be identified that the threshold for local global buckling in pin-ended and fix-ended panels is linear, as shown in Fig. 16. Therefore, it is important to identify the boundaries of these two main instability phenomena.

It is noticed that, similarly to the theoretical length factors for Euler’s critical load in columns, also the fixed and pin-ended angle panels thresholds, represented in Fig. 16, take approximately 0.5 and 1, respectively. This is evident in the angle 200 mm leg lengths angles, where the threshold for pin-ended panels is half of the fix-ended threshold. However, as the lengths decrease, difference between thresholds tend to increase, diverging from 0.5.

5. Analytical calculations

Analytical calculations are undertaken based on literature and norms available so to substantiate the numerical calculations and setting the basis for new formulations.

5.1. Global buckling

As described in Eq. (1), global buckling can be approximated by Euler’s buckling formulation. For this end, bending rigidity requires the calculation of the second moment of inertia where steel and mineral wool shall be calculated independently.

5.2. Calculation of the bending rigidity of the angle panel

Components as steel sheets and core are assumed to be fully bonded together, where face material is much stiffer than the core. The bending rigidity, \( EI \), used in typical panels, where modulus of elasticity \( E \) and the second moments of area \( I \) should then be decomposed in a sum of the flexural rigidities for each component. The flexural rigidity shall be measured about the centroid of the entire section.

Thus, for the calculation of the critical load, bending rigidity, \( EI \), often denoted as \( B \), in sandwich panels, must be calculated first. The angle is breakdown in three basic parts (as in Fig. 17): the left leg, right leg and the short middle segment where both coincide. The reason lies
on the fact that these components resemble an ordinary panel so then
the moments of inertia are simpler to be calculated.

Unlike the ordinary sandwich panels, in the calculations of the
principal moments of inertia for the angle panels, it is required a
consistent manipulation of axis, through translation and rotation of
each one of the axes of each component relatively to the centre of
gometry of the angle coincident with the principal axis.

Then, linear elastic critical load can be calculate based on the
calulation of $B_s$, by including the geometrical and material properties
of the steel sheets and core material. The orientation of the axis in the
right-leg length should be consistent with the orientation of the mineral
wool lamella, as show in Fig. 7.

6. Discussion and conclusions

6.1. Local buckling

Local buckling in the form of wrinkling can occur simultaneously all
over the surface of the face-sheets [2] and is dependent on the material
properties of the core and face-sheets and is identified in numerical
calculations. A way to calculate critical wrinkling load is to consider the
panel as a beam on an elastic foundation [43]. As described in chapter
2.2 several authors suggest the use of a similar equation where only a
$k$ value varies. This formulation was developed for the calculations of
resistance of regular panels, however with the intention to understand
wrinkling in angle panels, the employment of same formulation is
investigated, based on the numerical results.

As wrinkling shape in Fig. 18 reveals, a larger protuberance is
noticed at the area where Lamella I is in contact to the steel sheet on the
right leg length. This part of the mineral wool lamella is coinciding with
its direction of lower stiffness (direction y). This suggests that the lower
stiffness of mineral wool exposed to the steel sheet promotes an early
buckling at the short segment, indicated in Fig. 18; since the elastic
foundation of the steel facings have its lowest stiffness here.

Given that wrinkling formula is solely based on the stiffness of
the mineral wool, disregarding geometry and properties of steel, the
phenomena observed previously can then be expressed by breaking
down the wrinkling formula (Eq. (10)) in two components regarding
the segments of face sheets in interaction with mineral wool: (1) where face sheets meet mineral at its highest stiffness (Lamella 1 and 2, on x direction) and (2) where face sheets meet mineral at its lowest stiffness (Lamella 1 on direction y). Eq. (11) express this variation by means of the correct choice of the modulus elasticity and shear modulus, for the directions x and y, which represent the highest and lowest stiffnesses, respectively, which consequently will lead to the highest (σw,1) and lowest wrinkling stress (σw,2).

$$\sigma_{w,1} = k \sqrt{\frac{E_f E_{c,z}}{E_c G_{c,z}}}; \quad \sigma_{w,2} = k \sqrt{\frac{E_f E_{c,y}}{E_c G_{c,y}}}; \quad k = 0.7$$  \hspace{1cm} (11)

A value of 0.7 for k in the formula of the wrinkling is computed. This value arises from an iterative process of fitting to numerical results. The k-factor calculated is slightly above k-factors in literature where it may be due to effect of the stabilising corner of the angle panel.

Once the wrinkling stresses are calculated, the wrinkling force ($N_w$) can be estimated by multiplying with cross-section area of the steel faces. Where the area relates to regions described before, i.e., the cross-section area of the steel face segment supported by the highest stiffness of mineral wool ($A_{w,1}$) and cross section of the lowest stiffnesses of the mineral wool ($A_{w,2}$).

$$N_w = \sigma_{w,1} A_{w,1} + \sigma_{w,2} A_{w,2}$$ \hspace{1cm} (12)

Results show a deviation of 10% for all the different wrinkling responses of angle panels when it comes to compare numerical to analytical values.

6.2. Threshold for local and global bucklings

As referred before, a threshold for local and global buckling in pinned and fix-ended panels can be identified numerically, as it shows a clear path which follows the line represented in Fig. 19. Analytically, by equalising previous equations for global buckling (Euler buckling formulation, Eq. (1)) to local buckling (Eq. (11)), it is possible to extract the length of the angle panel that matches the threshold line represented in previous figure. The analytical results, match then the numerical results through Eq. (13).

$$L_{couple\ buckling} = \frac{\pi^2 \cdot B_i}{k \cdot \sqrt{\frac{E_f E_c G_{c,z}}{E_c G_{c,z}} \cdot (A_{inner} + A_{outer})}}$$ \hspace{1cm} (13)

where for angle panel lengths smaller than $L_{cb}$, calculated in Eq. (13), local buckling in the form of wrinkling is expected, whereas for higher $L_{cb}$ values, global buckling occurs. This equation may help engineers to predict the instability phenomena expected to occur and its thresholds being then able to take measures regarding to foreseen instabilities.
6.3. Final considerations

The parametric study on the angle panel is carried out by means of a series of finite element models comprising different geometries to assess its different load responses. Results are compared with analytical formulations based on the theory of sandwich panels and conclusions are traced.

The upper limit of the load bearing capacity of the angle panel is calculated by means of the linear elastic buckling response. Regarding to resistance, like EN 1993-1-1, same approach could be developed for angle panels, requiring for it a series of tests for the development of the buckling curves required in calculations.

Global and local buckling are identified and framed. They showed to be in good agreement with numerical and analytical results. The numerical calculations, through the parametric study, helped identifying couple buckling shapes, where local and global buckling are visible simultaneously. The knowledge on the thresholds for local and global instabilities are important for the prior choice of the formula used for calculation of the angle panel load capacity. For this end, a formula based on the length of the corner angle panel is adjusted. It is also adjusted a coefficient that sets the axial wrinkling load of the corner angle. Once more, given that numerical results are based on theorical values, without imperfections, the formula proposed does not comprises imperfections, but it should provide the designers with a good insight of possible load bearings capacities of angle panels and which formula to use whether local or global buckling are to be expected.

Finally, can be said that a corner angle can withstand a significantly high load, considering its short size. Regarding modular construction, this panel may set a step forward on the development of frameless structures which have very much to benefit from the quality on offsite construction for reducing imperfections.

CRediT authorship contribution statement

Pedro Andrade: Writing – review & editing, Writing – original draft. Ove Lagerqvist: Methodology, Conceptualization, Supervision. Rui Simões: Supervision, Formal analysis. Gabriel Sas: Supervision, Funding acquisition.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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