Finite Element Analysis of Thin Precast Concrete Sandwich Panels

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ABSTRACT: The purpose of this study is to numerically investigate the performance of a thin Precast Concrete Sandwich Panel (PCSP) proposed for building retrofit. Standard precast concrete sandwich panels, constructed of steel reinforced concrete, are physically heavy and have significant thicknesses. A thin precast concrete over-cladding sandwich panel is presented in this paper which combines the state-of-the-art in ultra-high-performance concrete, carbon fibre shear reinforcement and vacuum insulation to allow for a slimmer design while abiding by thermal and structural constraints. Another precast concrete re-cladding sandwich panel is also referred to in this paper which uses phase change materials (PCM) in a thicker inner wythe to enhance the thermal storage properties of the concrete. The panels are modelled, and their structural integrity is investigated, using finite element techniques. The aim of the analysis is to provide an insight into the limiting parameters of these thin precast concrete cladding elements. The analysis has highlighted the concrete wythe thickness and the insulation stiffness as two important performance parameters.

KEY WORDS: Precast concrete; Sandwich cladding; Finite element.

1 INTRODUCTION

Concrete offers a number of advantages over other commonly used cladding materials, including its superior thermal resistance, fire resistance, durability and structural efficiency [1]. Standard precast concrete sandwich panels, constructed of steel reinforced concrete, are often designed with thicknesses that exceed 300mm, resulting in physically heavy cladding elements with significant embodied energy. The key components of a PCSP are presented in Figure 1.

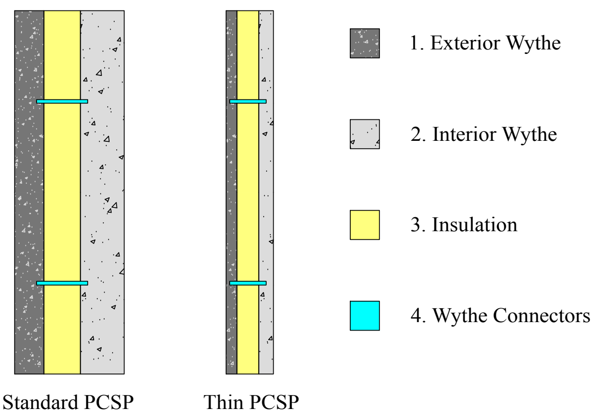

Figure 1. Components of a precast concrete sandwich panel

In standard PCSPs the exterior and interior concrete wythes (Components 1 and 2 in Figure 1) are reinforced with steel which require concrete cover to prevent corrosion. Adopting standard precast construction techniques means that wythe thicknesses of less than 70-80 mm are not feasible if reinforced with steel. Other research on thin-PCSPs have considered the use of Glass-fibre Reinforced Concrete (GRC) combined with textile reinforcement [2], [3] as well as geopolymer concrete [4], [5]. This research project proposes the use of a fibre reinforced Ultra High Performance Concrete (UHPC) in order to reduce wythe thickness.

The insulation (Component 3 in Figure 1) forms the filling between the two concrete wythes. This thin-PCSP uses vacuum insulation which has a far lower thermal conductivity (Kingspan's *Optim-R*^{TM} = 0.007 W/mK [6]) than the Extruded Polystyrene (XPS) insulation which is typically used in PCSPs (*Styrozone*^{TM} = 0.035 W/mK [6]).

The wythe connectors (Component 4 in Figure 1) are used to hold all the layers of the PCSP together. Depending on the quantity of connectors and structural properties of the connector's material, various degrees of composite action can be achieved [7]. The background and current innovations for wythe connectors are elaborated on in Section 2.

This study aims to provide insight into the structural behaviour of thin precast concrete sandwich panels using Finite Element (FE) methods. COMSOL Multiphysics® is the software used to carry out the FE modelling. This research forms part of a larger H2020 project (Project IMPRESS) which is focused on investigating a range of prefabricated innovative panels for buildings. Another of these PCSPs (Reinforced PCM concrete inner wythe – UHPC outer wythe) is presented in Figure 2. This panel has been tested under flexural loading conditions and its performance is compared in this paper with the thin-PCSP.

2 LITERATURE REVIEW

The first use of PCSPs is unknown but they have been used in construction for more than 60 years [8]. According to Gleich [9] the first type of PCSP was introduced in the 1960s as a fully composite panel that consisted of two wythes

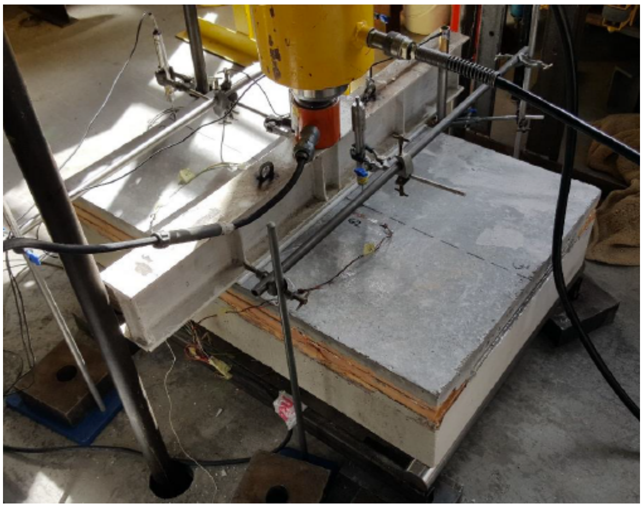

Figure 2. Precast concrete sandwich panel in a flexural test set-up

structurally connected with concrete ribs, which were later replaced by solid concrete zones to reduce thermal bridging while maintaining structural efficiency.

Steel trusses were later used to reduce the thermal bridging further compared to the concrete zones, but these steel trusses also present significant thermal bridging. As a result of the thermal inefficiencies found in the composite wall panels, the non-composite PCSPs which require minimal shear connectors became popular during the 1980s [9].

The majority of PCSPs currently are designed assuming non-composite behaviour with the inner leaf being the only structural wythe and the outer wythe acting as a rain-screen. The concrete wythes of non-composite panels were connected using smaller metallic connectors but, although smaller than their metallic truss predecessor, they also resulted in thermal bridging which has more recently been shown to be significant [10]. Fibre-reinforced polymer (FRP) connectors were introduced by *Thermomass* to replace the metallic connectors and reduce thermal bridging. Keenehan et al. [11] showed that these FRP ties used in the non-composite PCSPs almost eliminate the effects of thermal bridging. Salmon et al. [12] used FRP in a truss orientated shear connector to achieve a good degree of composite action and similar thermal performances to non-composite panels with FRP. An epoxy-coated composite grid made with cross-laid and superimposed carbon fibre called C-Grid® was developed by Chomarat™ and introduced into the construction market by AtlusGroup® in its composite CarbonCast© wall systems [13]. These shear connectors were tested as part of a pre-stressed wall system [14] and displayed close to 100 % composite action, provided an appropriate quantity and configuration of the shear grids are used.

A number of studies have used FE modelling to simulate the behavior of standard precast concrete sandwich panels under flexural [15] and thermal [16] loading conditions. Other studies have also modeled the behavior of PCSPs that incorporate novel materials such as using textile reinforced concrete [17], prestressed concrete [18] and using aerated concrete as the insulation [17].

3 METHODOLOGY

This paper presents the first stage in an FE analysis of the thin-PCSP. A number of simplifying modelling assumptions were made, as follows:

- Panel modelled in 2D
- Concrete and insulation modelled as linear elastic materials.
- Wythe connectors geometries are neglected.
- When maximum bending stresses exceed flexural strength of concrete, failure is assumed.
- Insulation and concrete layers are perfectly bonded

The panels are tested under flexural loading conditions numerically. A 3-point bending test in a load-controlled condition is first simulated to mimic laboratory testing. Additionally, a real wind loading condition is assumed and is explained in the following section.

Before applying FE modelling techniques, the PCSP is assessed for fully composite and non-composite behaviour. The composite behaviour of a PCSP is described in Figure 3.

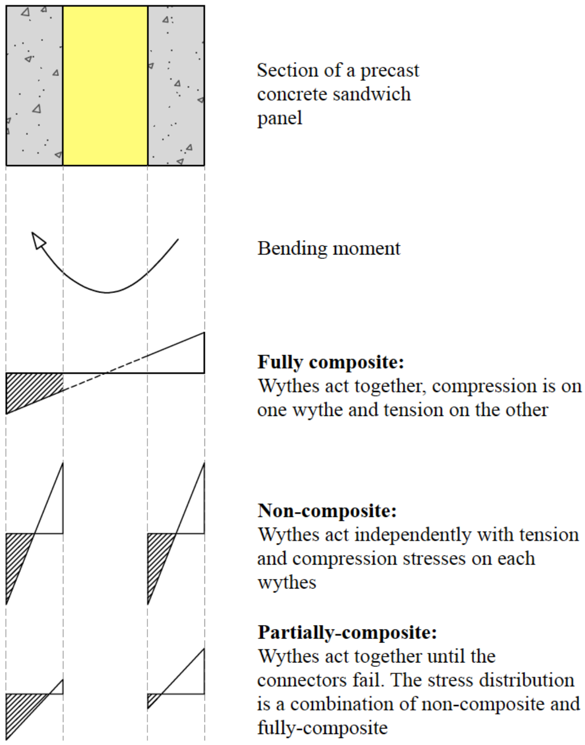

Figure 3. Approximate strain profiles for a section of a PCSP

3.1 Wind loading

The walls are simulated for various wind speeds (maximum gust wind speed during the recent Storm Ophelia was 43.3 m/s [19]) and are assumed to fail when the maximum principal stresses exceed the maximum tensile strength of the concrete. Equation (1) converts wind velocity, v (m/s) to wind pressure, P (N/m^2).

$$P = \frac{1}{2}\rho v^2 \tag{1}$$

where, ρ, is the density of air. The total force acting on the wall from the wind, F (N) is calculated by Equation (2):

$$F = PAc_d \tag{2}$$

where A is the area of the wall ($A = W_p \times H_p$) and c_d is the coefficient of drag (=1.4 for a short building). The total force is assumed to be distributed evenly between the two piers (as shown by the highlighted regions in Figure 4).

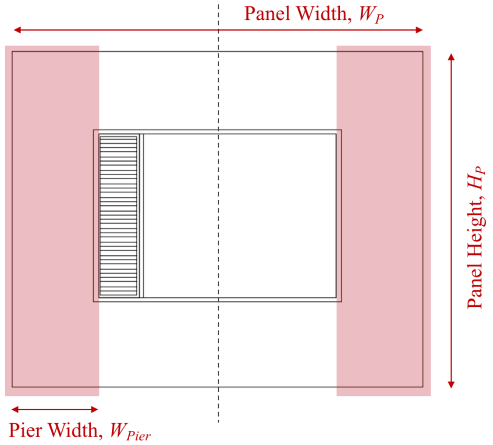

Figure 4. Elevation drawing of proposed precast concrete sandwich panel with a window highlighting the wind load bearing areas

The force from the wind on each individual pier is therefore taken as F/2 and the line load, w (N/m), is given by Equation (3).

$$w = \frac{F/2}{H_p} \tag{3}$$

For a single skin concrete panel, the elastic bending moment, M, may be calculated by Equation (4), and the bending stress, σ, may be calculated according to Equation (5).

$$M = \frac{wH_P^2}{8} \tag{4}$$

$$\sigma = \frac{My}{I} \tag{5}$$

where y is the distance from the stress point of concern to the neutral axis and I is the second moment of area (Equation (6)).

$$I = \frac{W_{pier}d^3}{12} \tag{6}$$

where d is the depth of the single concrete layer and the maximum bending stresses will occur at the outer most fibres when $y = d/2$. The analysis of a single skin concrete wall is used to estimate the maximum bending stress, σ_{max}, for both a fully composite (100%) and a fully non-composite (0%) PCSP. The maximum bending stresses for the composite and non-composite panel are calculated following the guidelines of the FIB report on precast concrete sandwich panels [1].

3.2 Finite Element modelling

In reality the behaviour of the PCSP will be somewhere in between 100% and 0% composite action of the two concrete wythes. To assess this behaviour FE techniques are used. The PCSPs are modelled in this paper using a linear finite element analysis and phenomenological failure criteria. Linear elastic behaviour is assumed in this model as a first step in assessing the behaviour of the composite walls. The material properties, required for modelling purposes, are presented in Table 1 and have been measured experimentally [20].

Table 1. Material properties

Material	Stiffness (GPa)	Tensile Strength (MPa)
UHPC	50	12
PCM-Concrete	10	2
Insulation	2.5×10^{-3}	-

First the FE model is validated for a single skin concrete wall. The maximum bending stress calculated following Equations (3) to (6) for a 0.14m thick wall (σ_{max}= 1.8MPa) match with those results found with the FE analysis, as shown in Figure 5.

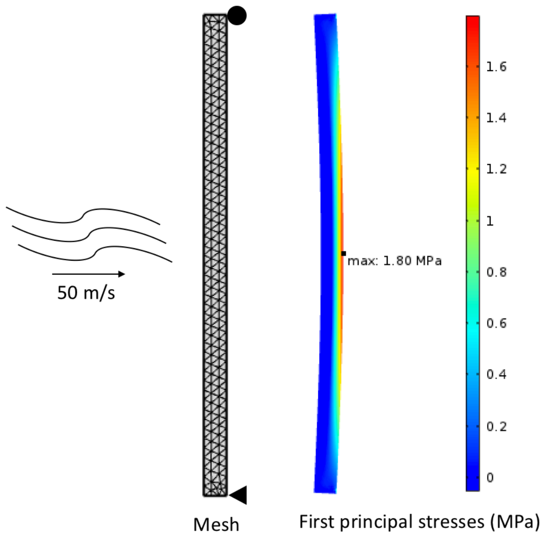

Figure 5. Finite Element model of plain concrete wall, displaying maximum bending stresses

Displacements in this, and all subsequent FE models in this paper, are scaled up by a factor of 20 to visualise the deflections better.

4 RESULTS AND DISCUSSION

The results of the FE study are presented in two sections; first the two panels developed for the project IMPRESS are compared on a smaller scale (Section 4.1); then the performance of the thin-PCSP is assessed with variable parameters (e.g. insulation thickness, wythe thickness and insulation stiffness) (Section 4.2).

All three layers of the PCSP are modelled in COMSOL Multiphysics but for visual clarity only the stress distribution of the two concrete wythes are presented and the insulation layer is hidden in the results.

4.1 "Recladding" vs "Overcladding" panel

Experimental loading conditions are applied in this section for modelling the two different panels. A point load is applied at 10 kN increments in the model. The two wythes of the "overcladding" panel have a similar stiffness while the two wythes of the "recladding panel" have different stiffnesses, therefore they respond differently when loaded. The stress distributions for both panels are presented in Figure 6.

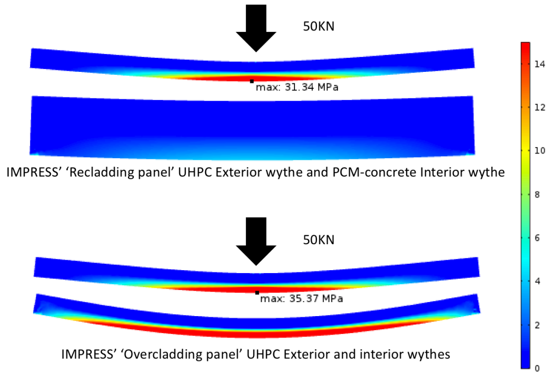

Figure 6. Behaviour of two different PCSPs under a 50kN point load

The behaviour found during testing of the PCSPs is similar to the behaviour found by the model in that the top wythe fails first. Failure of the PCSP would occur before these loads could be reached but this is a linear elastic analysis and cracks are not accounted for in the model. The linear elastic behaviour of the extreme bottom fibres of both wythes is presented in Figure 7.

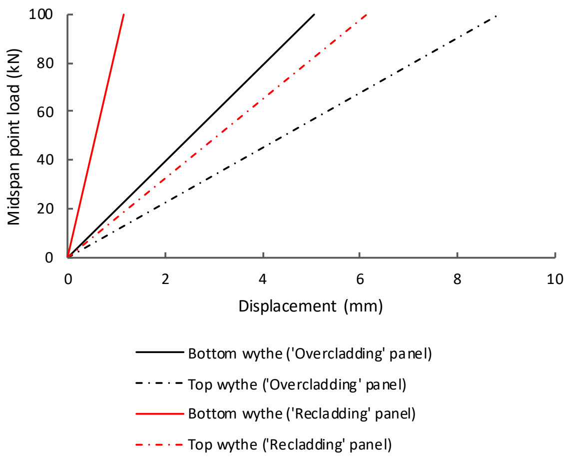

Figure 7. Load vs displacement of the two different PCSPs at the bottom fibres of each wythe

These linear curves show the change in stiffness of the different wythes in the different panels. It is evident from the model that the thicker ("recladding") PCSP has a greater differential stiffness between the two wythes when compared with the thinner ("overcladding") PCSP. More importantly, under a given elastic load, the outer wythe of the overcladding panel is more heavily stressed due to the partial composite action with the thin inner wythe. The effect of wythe thickness and stiffness are assessed in greater detail in the following section for different wind loading conditions.

4.2 Parametric analysis

In this section the wind loading conditions are applied as a Uniformly Distributed Load (UDL) following Equations (1) to (3). The behaviour of the sandwich panels are different as a result, as presented in Figure 8. It is evident from this figure that under the UDL the exterior wythe of the thin-PCSP does not experience the highest bending stress.

Applying the theory from Equations (3) to (6) for the thin-PCSP would give a maximum bending stress of 11 MPa for a fully non-composite panel and only 1.1 MPa for a fully composite panel. The FE model (which does not include any shear connecttions) shows a maximum bending stress of 8.79MPa on the back wythe. This means that, based on the assumptions made in this model and a UHPC tensile strength of 12MPa, the thin-PCSP could withstand wind speed higher than 50 m/s.

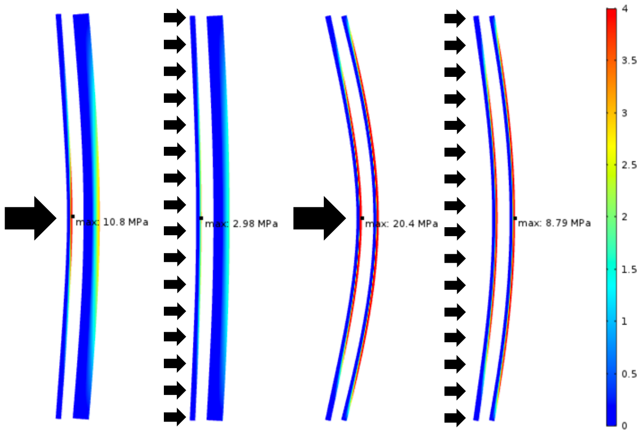

Figure 8. Behaviour of both cladding panels under point load and uniformly disturbed loading conditions

4.2.1 Concrete wythe thickness

The first parameter assessed is the wythe thickness. The FE figures for the parametric analysis present the direct bending stresses in place of the first principle stresses. It can be seen from Figure 9 that there is evidently an increase in deflection and maximum bending stress in the panel as the thickness of the wythes are decreased.

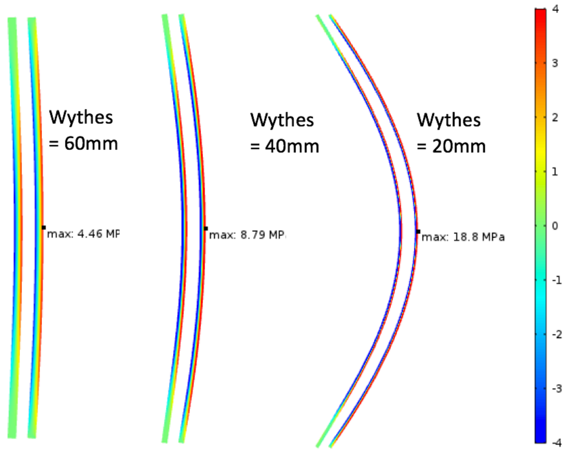

Figure 9. Finite element models of PCSPs with different wythe thickness displaying magnitude and location of maximum bending stress (MPa) for a wind speed of 50 m/s

The results in Figure 9 display the behaviour for a 50 m/s wind speed and show that the PCSP with the 20mm wythes would fail under this loading. The influence of wind speed on the maximum bending stress in the PCSPs is further analysed in Figure 10. These plots show that a PCSP with 20mm wythes would fail under a wind speed of about 40 m/s, the PCSP with 40mm wythes would fail under a wind speed of 60 m/s while the thicker PCSP with 60mm wythes would be able to withstand wind loads in excess of 80 m/s. Furthermore, these results show that the difference between the two wythes becomes more significant for thinner sections with the inner wythe of the 20mm wythes PCSP failing before the external wythe.

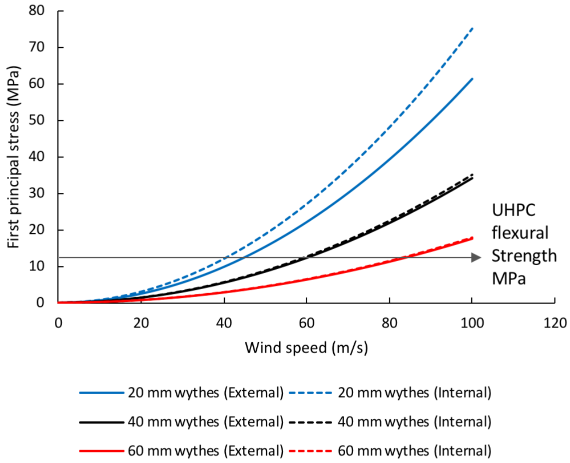

Figure 10. Maximum stress of both internal and external wythes vs wind speed of PCSPs with different wythe thickness

4.2.2 Insulation thickness

Decreasing the insulation thickness not only creates challenges for achieving thermal requirements of the PCSP but it also results in greater bending stresses in the PCSP (Figure 11); that is, based on the PCSPs achieving some degree of composite action. The results from Figure 11 also show the first wythe to fail is dependent on the thickness of the insulation as a result of the different load distributions between the two concrete wythes. All PCSPs with the insulation thicknesses presented in this figure withstand the 50 m/s wind load.

By assessing the impact of the wind speed on the maximum bending stresses further, as per Figure 12, it is evident that the influence of the insulation thickness is not as critical as the influence of the wythe thickness. It is also noted from this figure that differences in maximum bending stress between the two wythes is less for the PCSP with thicker insulation.

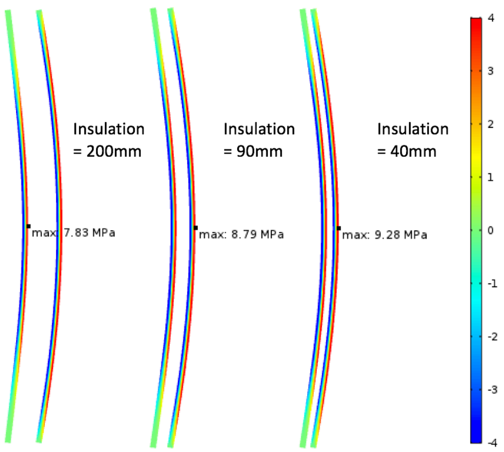

Figure 11. Finite element models of PCSPs with different insulation thickness displaying magnitude and location of maximum bending stress (MPa) for a wind speed of 50 m/s

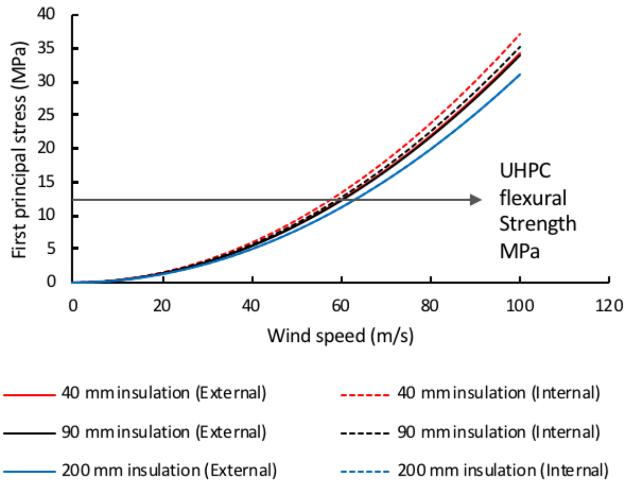

Figure 12. Maximum stress of both internal and external wythes vs wind speed of PCSPs with different insulation thickness

4.2.3 Insulation stiffness

The last parameter assessed in this paper is the insulation stiffness. Although the insulation stiffness is significantly lower than the UHPC (2.5 MPa << 50,000 MPa) the results in Figure 13 show that the insulation stiffness does have an impact on the overall behaviour of the PCSP.

Figure 14 shows that a PCSP with an insulation stiffness of 2.5 MPa (XPS insulation) would fail under a wind loading of approximately 60 m/s while a PCSP with an insulation stiffness of 10 MPa (Phenolic foam insulation) would withstand wind speeds above 75 m/s. It would also appear that there exists a balance between the insulation stiffness and the amount of load taken by each concrete wythe.

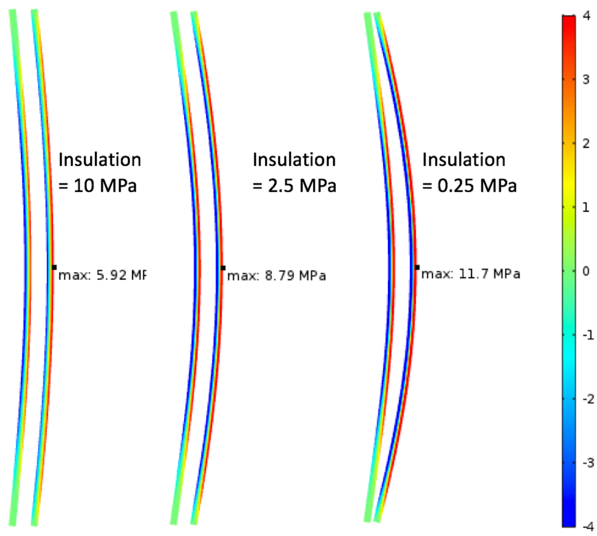

Figure 13. Finite element models of PCSPs with different insulation stiffness displaying magnitude and location of maximum bending stress (MPa) for a wind speed of 50 m/s

There are a number of assumptions and limitations when modelling this 3D geometry in 2D. The major assumption made is the neglecting of the shear ties which could have a significant influence on the degree of composite action between the two concrete wythes. The model will be upgraded to 3D in future work and will be validated against a series of experimental results that are proposed and scheduled for testing.

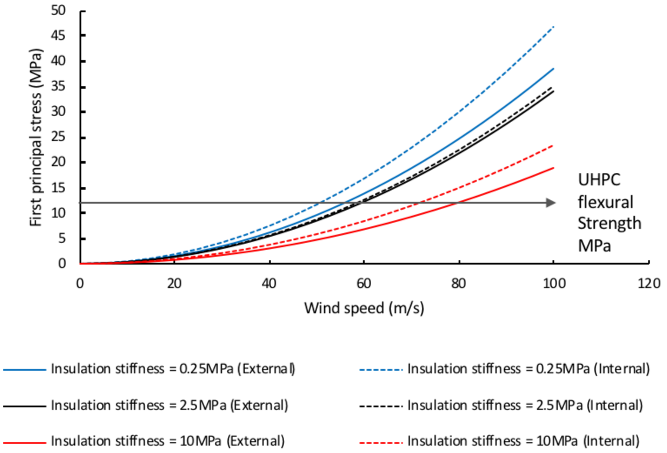

Figure 14. Maximum stress of both internal and external wythes vs wind speed of PCSPs with different insulation stiffness

5 CONCLUSION

This paper presents results that represent the behaviour of a thin-PCSP under flexural loading conditions. The results have shown that, provided some degree of composite action is achieved between the two wythes of concrete, the thin-PCSP can withstand significant wind loading conditions when formed of an ultra-high-performance concrete. The results showed that both concrete wythe thickness and insulation stiffness have a significant impact on the performance of the PCSP.

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