DEVELOPMENT AND TESTING OF A SANDWICH PANEL WITH UHPC AND PCM CONCRETE LAYERS

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ABSTRACT. Precast concrete sandwich panels provide a thermally efficient alternative to conventional brick and mortar construction and improve the energy efficiency of existing buildings. Thisproject comprised the design and testing of a sample re-cladding panel composed of a phase change material (PCM) in the concrete inner wythe (for thermal efficiency) and a thin ultra-high performance concrete (UHPC) outer wythe, joined compositely using a C-grid shear connector. Six different concrete mixes were prepared and structurally tested in compression and flexure. A concrete sandwich panel wascast using two of the best performing mixes and subsequently tested in three-point bending to investigate its flexural performance. The strongest PCM and UHPC concretes had average compressive and flexural strengths of 25MPa and 5.1MPa, and 121MPa and 9.2MPa respectively. The 900mm span panel tested in flexure reached its serviceability limit at 10kN, with ultimate peak load occurring at 97kN. Post-peak behaviour illustrated the role of the shear connector in allowing composite action to occur.

Keywords: PCM, Sandwich panels, Thermal efficiency, UHPC.

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INTRODUCTION

A European research project, IMPRESS, has an ultimate goal of improving the energy efficiency of mid-20th century buildingsthrough developing a high performance thin concrete sandwich panel which can be used as are-cladding panel to renovate these buildings [1]. The thermal and hygrothermal performance is well understood [2], but how these panels would perform structurally has not yet been investigated. This paper examines the concrete mix design and structural testing of prototype panels. The key objectives were to design Phase Change Material (PCM) and Ultra-high performance concrete (UHPC) for the inner and outer wythes respectively and to evaluate their composite action in a scaled panel flexural test.

METHOD

After several trial mixes, the mixes decided on for the main PCM and UHPC pours were as shown in Table 1.

Table 1. Mix constituents (in kg/m^3)

Constituent	**PCM**	**UHPC**
Rapid Hardening cement	220	254
GGBS	220	254
Micro silica	0	120
10mm Limestone aggregate	958	760
Medium sand	700	715
Betacarb filler	0	400
Water	225	200
Superplasticiser	4.6	18
35mm hooked steel fibres	(40)*	(40)*
Micro encapsulated PCM	120	0

* = for fibre mixes only

Coupon and Cube Specimens

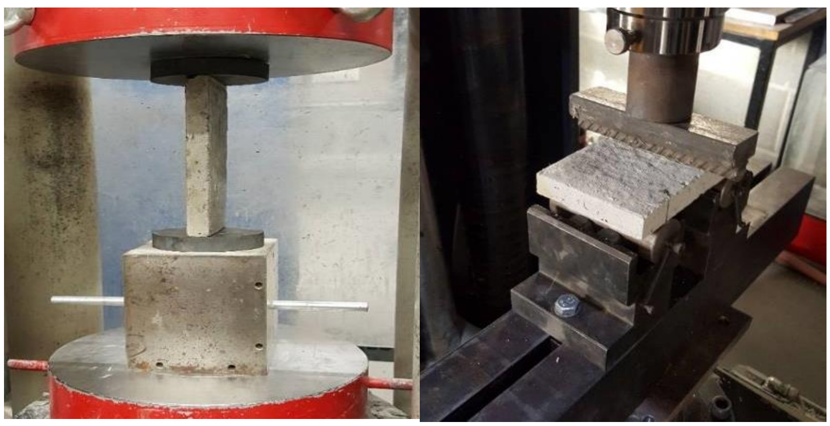
Normally, IS-EN standards [3] specify the dimensions for compression and flexural testing of cubes/prisms as 100mm and 160*40*40mm respectively. However, as the minimum wythe thickness was 20mm, it was decided to create 160*40*20mm novel concrete coupons in place of the standard prisms. Thus, they would have a significantly larger length to depth ratio of 8/1 compared to the standard prism’s 4/1. These could also be used for compression tests post flexure, but standard 100mm cubes would also be cast to ensure reliable and comparable concrete strength test results at 28 days.

Six coupons were poured for each mix, three each for compression and flexural testing, which were subsequently averaged. After establishing the 28 day cube strength in the normal way, a more difficult compression test was undertaken which involved loading along the longitudinal axis, as shown in Figure 1. Neoprene pads had tobe used to support/load the samples to distribute the load better and reduce end-friction and the effect of eccentricities, though the specimen's slenderness did make lateral instability more likely.

Strain gauges were used in testing of the coupons, placed on the tensile side of the coupons for the flexural tests, and on both flat faces for the longitudinal compression tests.

Test Sandwich Panel Design and Manufacture

Atest panel, designed to be similar to the recladding panels for the IMPRESS project, was 900mm long, 600mm wide and 200mm deepwith six main components (see Figure 2). The inner wythe was a 120mm thick PCM concrete containing two layers of 16mm diameter rebar (with 20mm cover and 60mm between the layers, at 175mm centres). The next layer was a highly efficient



(a) (b)

Figure 1. Sample test equipment for (a) longitudinal compression and (b) three-point flexure

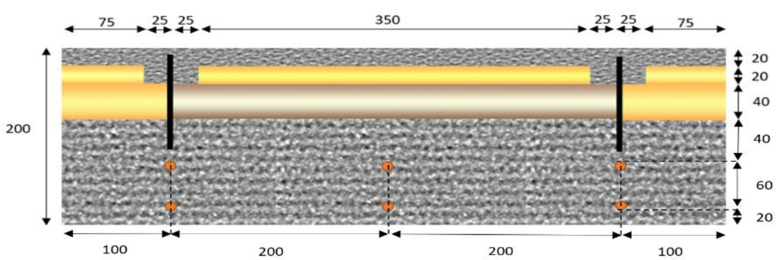


Figure 2. Sandwich panel cross-section

Image	Description
	UHPC
	PIR Insulation
	Vacuum Sealed Insulation
	Shear Connectors
	Steel Rebar
	PCM Concrete

vacuum sealed insulation which made up most of the insulating core of the panel but supplemented between the shear connector downstands by a layer of Polyisocyanurate (PIR) insulation which had varying depths depending on its location in the panel cross-section. The final part of the panel was the 20mm UHPC outer wythe with steel fibres, connected to the inner wythe by two longitudinal strips of C-grid polymer shear connectors which were themselves anchored in 20mm UHPC downstands in the outer wythe.

Sandwich Panel Testing

A three-point bending test was performed on the panel which was placed on two roller supports and had a line load applied at its centre. Five linear variable differential transformers (LVDTs) and four strain gauges were placed (Figure 3) to determine the displacement of and strains in the panel during testing. The loading was applied using a hydraulic jack with an initial loading rate ofabout 25N/s. Once the UHPC layer failed, as it did first, the loading rate was increased to up to 90N/s till failure.

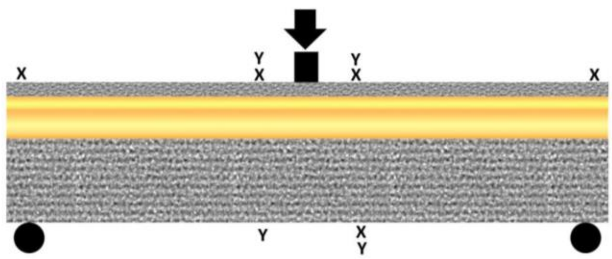


Figure 3. X = Transducer, Y = strain gauge

RESULTS AND ANALYSIS

A. Material behaviour

The non-standard compression test on the coupons gave rise to lower compression strength (averages 13.6 MPa vs. 22.4 MPa for cubes - see Table 3), as expected due to the aspect ratio, but without a neoprene bearing would have had larger variability due to problems of lateral instability, local crushing and end shearing. The effect of fibres was to increase the strength marginally in each case, especially for the coupon tests where the fibres bonded the concrete together preventing crack propagation.By using strain gauges, the Young's modulus of the fibre mixes was found to be approximately 23 and 42.5GPafor the PCM and UHPC mixes respectively, which wereused in the finite element analysis, discussed in the last section of the paper.

B. Panel Flexural Test

The panel test setup can be seen in Figure 4. The UHPC layer failed with a single central flexural crackupon a loading of 10.1kN to the panel. For practical purposes, this can be considered the serviceability limit failure point of the panel. However, full ultimate failure of the panel did not occur until 97kN, so the panel had very significant post-cracking load carrying capacity. The composite action between concrete layers is highly important in sandwich panels [5]. So, improved shear connection between the concrete layers to improve composite action may improve the serviceability limit, but not necessarily the ultimate limit. A thicker UHPC concrete layer would also have a similar effect.

Table 3. Concrete sample mean strengths

Mix	CUBE (MPA)	Coupon Compression (MPa)	Coupon Flexure (MPa)
PCM	22.4	13.6	5.0
PCM Fibre	25.1	16.0	5.1
UHPC	117.7	27.8	5.7
UHPC Fibre	121.3	35.7	7.5

There were five distinct phases during the testing of the panel as shown from A-F in Figure 5. By examining the data obtained from LVDTs and strain gauges, it was possible to deduce certain events, as follows:

i) Phase 1 (A-B)

In this phase the entire panel surface moved down almost uniformly (Figure 6), but by very little, as indicated by less than 0.5mm of displacement observed on all top surface LVDTs. The strain gauge behavior was elastic and was quite symmetrical with each at about 15 microstrain by theend of this phase. The negative values in the top surface indicated compressive strain, while thepositive values at the bottom indicated tensile strain indicating composite action, but whether the

Figure 4. Panel testing setup

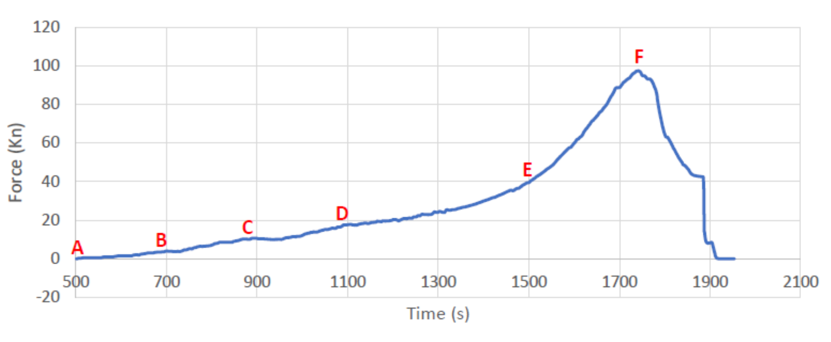


Figure 5. Force vs time for the panel test

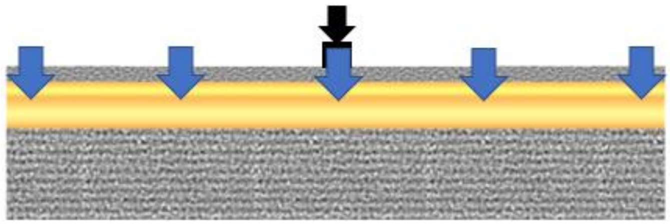


Figure 6. Phase 1 diagram

shear connector or insulation was responsible is not known. However, it is expected that the shear connectors and insulation restrained the UHPC layer and this resulted in a bending moment diagram like thatshown in Figure 7. It should be recognized that the shear connectors run along the length of the panel near the edges (see Figure 2) and thus there are no shear connectors running across the 600mm width at the support or mid span.

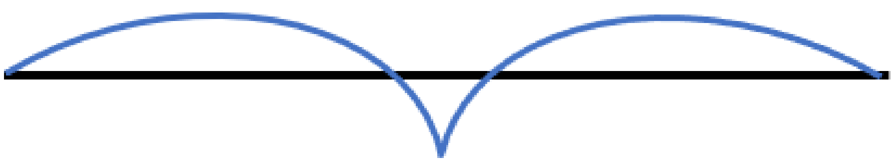


Figure 7. Phase 1 bending moment in the UHPC

ii) Phase 2 (B – C)

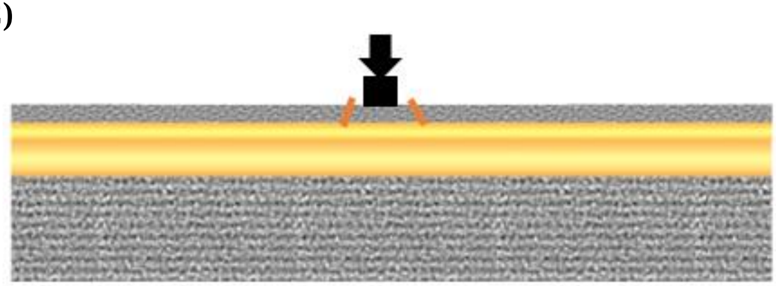


Figure 8. Phase 2 diagram

Cracks began to appear near the line load in the top wythe (Figure 8). This was observed where the top two strain gauges changed at a much quicker rate of increase for the strain. LVDTs also showed a slightly increased rate of displacement which provided further evidence for this. It is thought that these cracks were flexural in nature and propagated upwards from the bottom of the UHPC wythe. The bottom two strain gauges continued to increase at a relatively linear pace, which indicated that there was no cracking in the PCM layer at this point, but some load transfer.

At the end of this phase, the cracks near the line load had fully formed and the UHPC layer could be considered to have cracked right through at this point (at a 10.1kN load). This serviceability failure was indicated most clearly by the top two strain gauges reducing in compressive strain. The two center LVDTs also recorded slightly increased rates of displacement which further supported this evidence. The bending moment diagram in Figure 9 illustrates what had occurred in the UHPC by the end of this phase. It is important to note that in the next phase, when the edges of the top wythe wish to lift, they were restrained by the shear connectors running between support (left to right in Figure 9) and so two hogging moments becomes more evident in the top wythe.

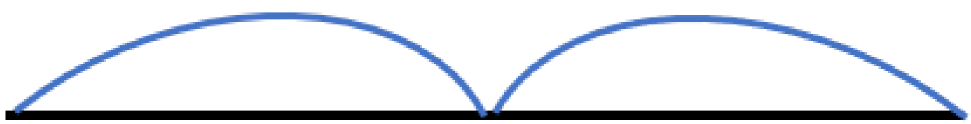


Figure 9. End of phase 2 UHPC bending moment

iii) Phase 3(C – D)

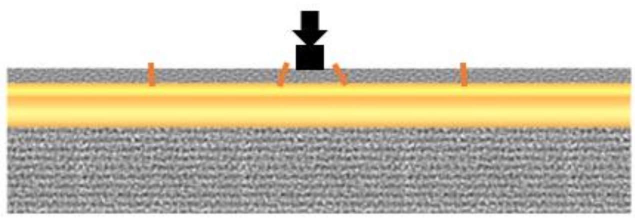


Figure 10. Phase 3 diagram

Cracks began to appear where the largest hogging moments were located (Figure 10 and 11) – the presence of the shear connector holding down the top wythe enhances this effect. These cracks had formed fully by the end of this phase which could be observed in the top two strain gauges by a significant drop in positive tensile strain and a highly visible crack.   The top surface LVDTs showed increased rates of displacement from this point onwards, which provided further support for this interpretation, Since there was no longer a hogging moment in the UHPC (Figure 12) , the downward displacement of the UHPC increased. The UHPC can be considered to have completely failed by this point and thus there were no longer any significant bending moments in this layer. Due to this, the PCM layer began to restrain most of the load and the bottom strain gauges showed this as they begin to experience larger rates of strain increase with increasing load.

Figure 11. Cracks appearing on top of top wythe due to hogging moment

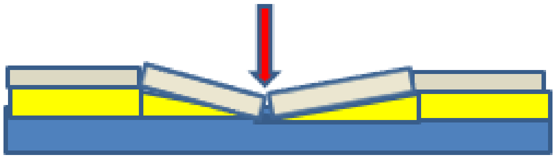


Figure 12. Failure mechanism in the top wythe

iv) Phase 4(D – E)

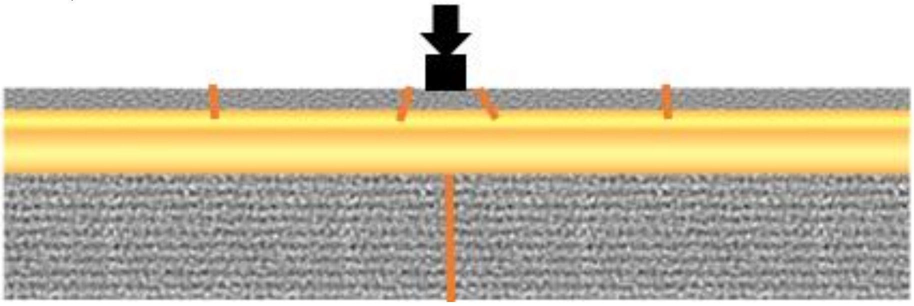


Figure 13. Phase 4 diagram

Since the top layer had failed, the readings from the gauges and LVDTs on the PCM layer were focused on instead. In this phase cracks began to form in the PCM layer (Figure 13), which were fully formed by the end of this phase (at a 40kN load). From observing the force against time graph (Figure 5), it was observed that the relationship between force and time changed at this point. This was due to the rebar taking on the loading, rather than the concrete. In the bottom two strain gauges it was observed that the increase in tensile strain dropped, further indicating cracking of the PCM concrete as this indicated that the local flexural cracks on the bottom face had relieved some of the tension in the concrete.

v) Phase 5 (E – F)

In this phase the rebar was taking most of the stress in the panel in tension, with concrete at the top of the PCM layer taking the compressive stress (Figure 14). This continued until the end of this phase, where the steel failed, and the entire panel could then be considered to have reached ultimate failure (at 97 kN load). This failure could be seen in the force against time graph by a large drop off in force after this point. The huge swings in strain in the bottom gauges further indicated this. Finally, all the LVDTs showed reduced displacements at this point.

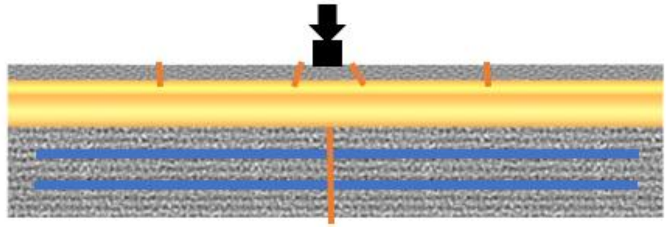


Figure 14. Phase 5

C. Finite Element Modelling

Finite element modelling was carried out using COMSOL Multiphysics® software to explore the structural behaviour of the UHPC and PCM-concrete sandwich panel. The panel was modelled in 2-D as a linear elastic model. In this initial model the shear connectors were omitted and it was assumed that there was a perfect bond between the concrete and insulation layers. Failure of each type of concrete was assumed when the bending stresses exceeded the flexural strength of the concrete. The stiffness of the insulation layer was assumed to be 10MPa to reflect the stiffness of the insulation layer of the laboratory test panel.

A three-point bending test was modelled to simulate the load test carried out in the laboratory. A point load was applied to the mid-span of the panel and was increased up to 50kN. Due to the difference in thickness and material stiffness of the sandwich panel layers, the layers have significantly responded differently to the applied load. The stress distribution for each of the concrete layers are displayed in Figure 15.

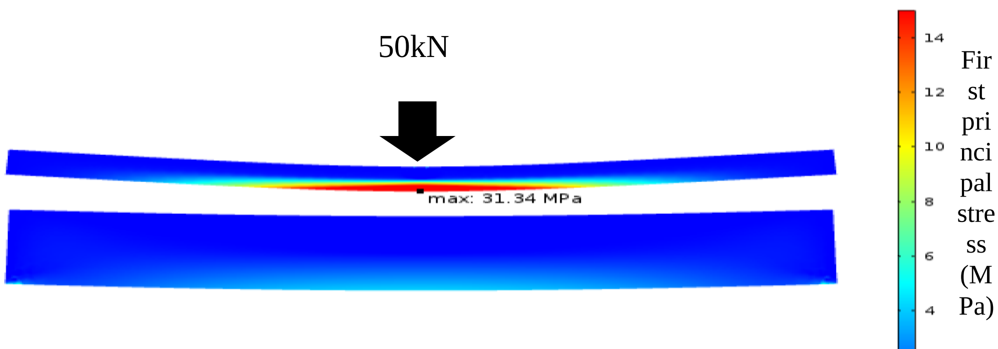


Figure 15. Behaviour of panel under point load

The results from the model reflected the structural behaviour of the panel during the laboratory load test by showing that there was very little load transfer to the lower PCM-concrete layer prior to the failure of the UHPC layer. This behavior was expected due to the relatively low stiffness of the insulation layer. Failure of the UHPC layer occurred at 10kN in the laboratory test however as the finite element analysis assumes linear elastic material behaviour cracks are not accounted for in the model. Further FEM modelling was carried out on this panel configuration and the results are discussed further in separate papers [9, 10].

CONCLUSIONS

This paper has illustrated the mechanism by which a composite sandwich façade panel would resist an external point load, whereby the thin UHPC outer wythe initially transfers the load to the PCM inner wythe, but in failing elastoplastically, sudden collapse does not occur and the ultimate load is some 9 times larger than the serviceability limit, suggesting a very practical and safe structural element in practice.

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