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Post-Fire Axial Load Behaviour of Double Skin Composite Walls Incorporating Ultra-High Performance Concrete

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Abstract: Double skin composite walls (DSCWs) consist of two skins of profiled steel sheeting and in-fill of concrete. DSCWs are proposed to act as axial and lateral load resisting elements in buildings. This research concentrates on the structural performance of DSCWs (incorporating ultra-high performance concrete 'UHPC' of compressive strength greater than 120 MPa) subjected to elevated temperatures of up to 800°C maintained at a steady state duration of two hours. Performance of DSCWs is evaluated in terms of physical changes, residual axial load capacity/stiffness, axial load-deformation response, ductility, strain characteristics, and overall failure modes. The recommendations of this research will be useful for the development of guidelines for the post-fire axial strength of DSCWs and will aid in the development of fire protection measures.

1. Introduction

Profiled steel sheets are widely used in composite construction and the use of composite slabs (Wright et al. 1987) known as "fastrack construction" has grown rapidly since early 1980's, replacing traditional reinforced concrete flooring system. More recently, a novel form of double skin composite walling (DSCW) system comprising vertically aligned profiled steel sheeting and an infill of concrete (as shown in Fig. 1) was proposed (Wright et al. 1994; Wright and Gallocher 1995; Hossain and Wright 1995). Such composite walling has many advantages when used in conjunction with composite flooring and is applicable as vertical and lateral load resisting elements in buildings. In addition, such walls have also potential to be used in basements and blast resisting structures. The advantages of this system arise from the type of construction where profiled steel sheeting acts as a formwork for in-fill concrete in the construction stage and as reinforcement in the service stage (Hossain and Wright 1995; Wright and Gallocher 1995).

In such construction, the interaction between sheeting and concrete plays an important role in the composite action of the system. The interface shear bond failure may be a limiting criterion for the design of such system (Hossain and Wright 2004a-b). The bond between steel and concrete can be improved by embossments installed in the commercial profiled sheets available in North America or by installing other forms of shear connectors. The mechanical interlock at the sheet-concrete interface may govern the ductile and brittle failure of composite walls.

The behaviour of the composite walls under axial and shear loading was associated with the difficulty in the transfer of load between the steel skins and the concrete core and the buckling of the steel sheeting (Bradford et al. 1998; Wright 1998; Hossain 2000; Hossain and Wright 2004c). The problem of load transfer between steel and concrete can be overcome by providing additional shear connection devices at the head and foot of the wall in case of wall under axial load or by providing adequate connections between sheeting and concrete at the boundaries (Hossain and Wright 2004c). The sheet-concrete interface behaviour in a composite wall is complex as profiled ribs play an important role in providing mechanical bond when steel tends to slide over the concrete after the failure of chemical bond. In this case, the transverse shear bond perpendicular to the profiles (derived mainly from friction) plays an important role and mobilization of such bond will provide high shear resistance of composite walls (Hossain and Wright 1998).

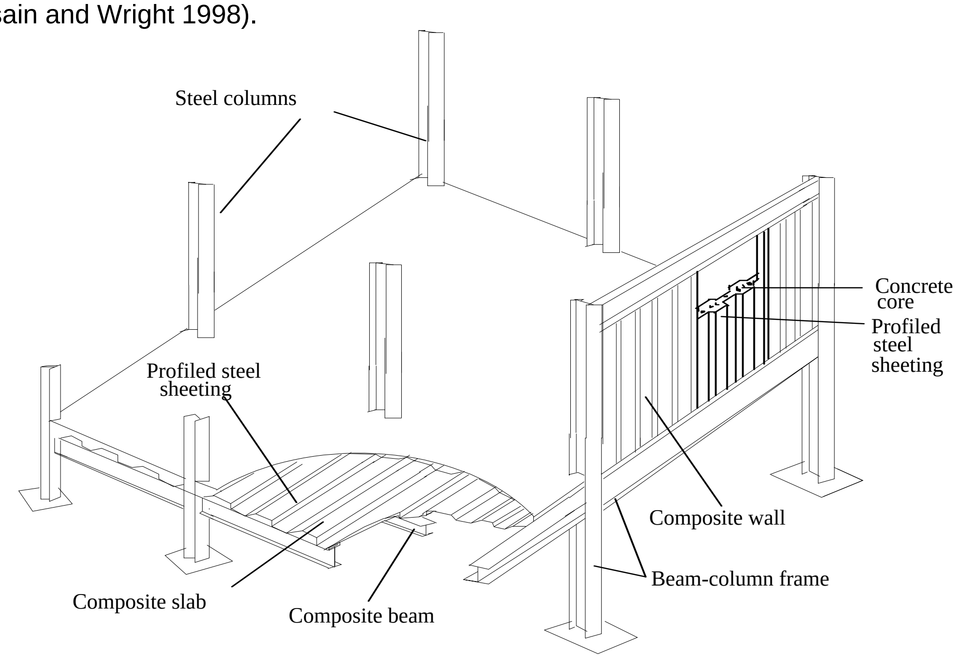

Figure 1: Schematic of composite wall in a building

The knowledge of the behavior of such walling systems being exposed to high temperatures is an important element in the design of this system in a building. Moreover, the use of ultra high performance concrete (UHPC) can increase the strength, ductility and energy absorbing capacity of such composite walls. UHPC is a high-strength, ductile material formulated by combining powders (Portland cement, silica fume, quartz flour and fine silica sand), high-range water reducer, water, and steel or organic fibers (Collepardi et al. 1997; Richard and Cheyrezy 1995, Bonneau et al. 1997, Reda et al. 1998, Tafraoui et al. 2009). Over the last years**, s**elf-consolidating UHPC has been developed with superior strength (>120 MPa), ductility and durability - which translates to speedy construction, reduced maintenance and a longer life span for the structure (Hossain et al. 2012). The steel fibers limit the crack propagation and delay crack formation by behaving as crack arresters or bridging mechanism in the UHPC. UHPC mixtures having compressive strength, flexural strength and modulus of elasticity of more than 100 MPa, 10 MPa and 60 GPa, respectively have been developed (Acker and Behloul 2004, Buitelaar 2004, Tafraoui et al. 2009).

UHPC offers advantages such as ability to be virtually self placing, speed of construction, and improved aesthetics as well as superior durability against corrosion and superior abrasion/ impact resistance - which translates to reduced maintenance and a longer life span for the structure (Bonneau et al. 1997, Reda et al. 1998). UHPC has the capacity to deform and support flexural and tensile loads, even after initial cracking. Better durability characteristics of UHPCs are the result of improved micro-structural properties of the mineral matrix (maximum compactness and a small disconnected pore structure due to combination of fine powders and their chemical reactivity) and the bond between the matrix and the fiber. UHPC has been successfully used in various types of construction including bridges, seismic-resistant and explosion-resistant structures (Hartmann and Graybeal 2001, Buitelaar 2004, Hajar et al. 2004, Bierwagen and Abu-Hawash 2005, Hossain et al. 2012).

Although the structural performance of DCSWs under axial and shear loading has been investigated, their fire resistance has not yet been studied especially with UHPC. This paper presents the results of an experimental investigation studying the behavior UHPC based DSCWs subjected to elevated temperatures ranging from 0 to 800^{o}C for a duration of two hours.

2. Experimental Investigation

An extensive experimental investigation on the fire resistance of composite walls with different types of high performance concrete (HPC) as concrete in-fill is now in progress. The test results of 10 composite walls with UHPC subjected to elevated temperatures are the subject matter of this paper.

2.1 Wall Details

The dimensions and connection details of the walls are presented in Figure 2. Composite wall specimens were constructed by attaching two profiled steel sheets of thickness 0.61 mm, by using steel bolts at each trough, in five rows at spacing of 110 mm at the top and bottom and 140 mm at the center. Shear interaction between the steel sheeting and UHPC was improved through the use of steel hooks used as shear connectors that were placed in the crests, at the top and bottom of the wall, which were attached to the steel sheeting by riveted connections. The overall dimension of 1/6th scale model walls was 540 x 320 mm.

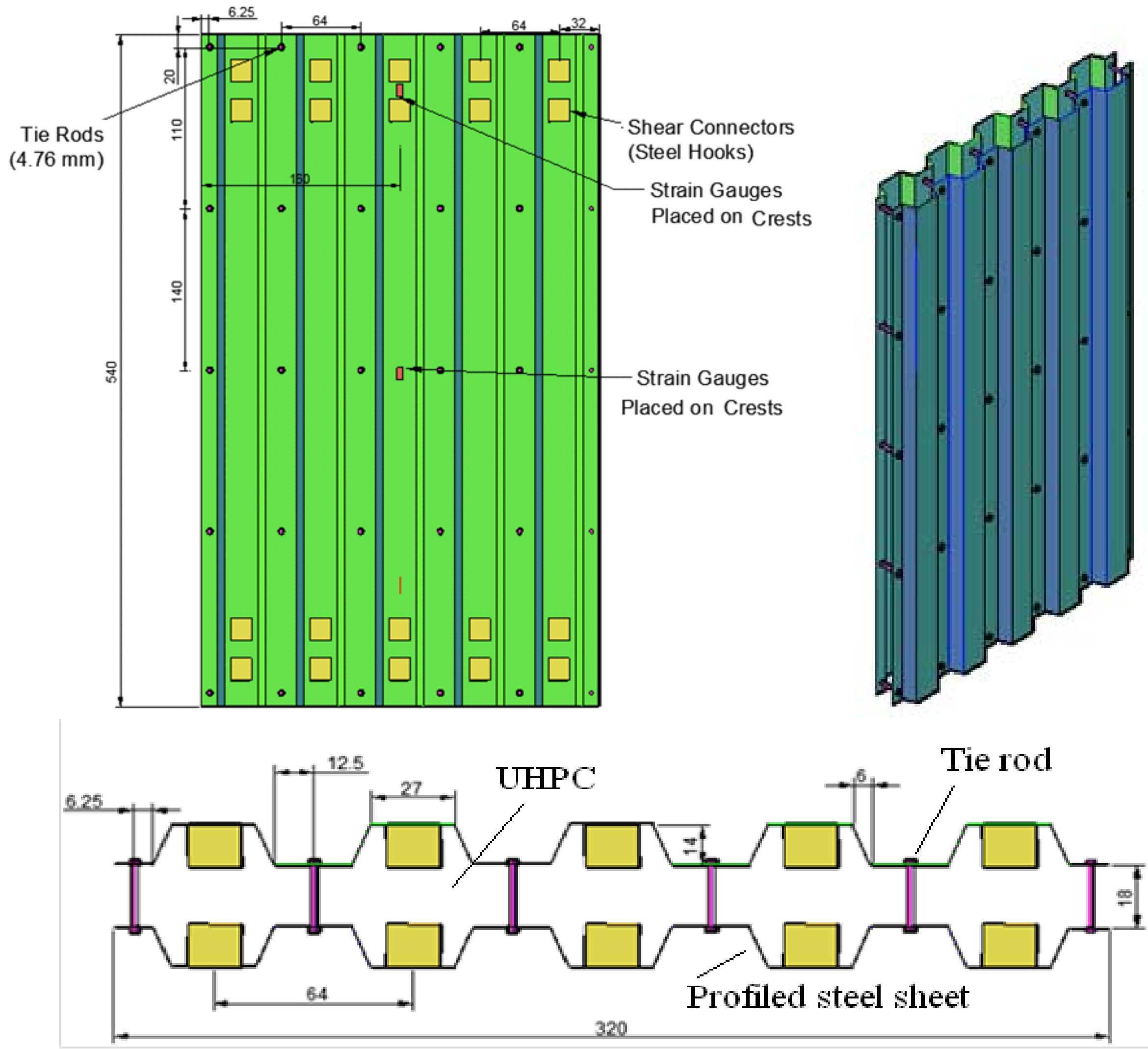

Figure 2: Geometric dimensions of composite walls (dimensions in mm)

2.2 Material Properties, Casting, Curing, Instrumentation and Testing of Composite Walls

After assembling the pair of sheeting with connection devices (Figure 2), they were installed in a casting assembly. Flowable UHPC was machine mixed and poured into the wall from the top without vibration or consolidation. The walls were cast vertically and parallel to the profile and were cured in this position for the first two days before stripping off from the casting assembly. They were then air cured at room temperature until testing. Control specimens in the form of cylinders and cubes were also cast to determine the strength of UHPC at the age of wall testing.

UHPC was made of Type 10 general purpose cement, silica fume, silica sand of 110 micrometer nominal size, steel fibers and superplasticizer. The steel fibers used were 0.2 mm in diameter and 13 mm in length with a tensile strength of 2160 MPa, modulus of elasticity of 210 GPa, and melting point higher than 800°C. The UHPC had water to cementitious material ratio of 0.22 and a steel fiber content of 9% by mass of dry material. The mean 28-day compressive strength of UHPC was 120 MPa. The profiled steel sheet had yield stress of 354 MPa, ultimate strength of 420 MPa and modulus of elasticity of 207 GPa as determined from coupon test according to ASTM A370.

Composite wall specimens were cured for 28 days and then subjected to various temperatures (0, 400°C, 600°C and 800°C) in a furnace for a steady state duration of 2 hours as per ASTM E119a before being cooled down to room temperature. The test specimens were removed from the furnace after the specified duration of heating, and then strain gauges were installed before being tested. Strain gauges were installed at key locations (at the top and centre) of the composite wall as shown in Figure 2. After instrumentation, the walls were transported to the compression testing machine. All the walls were subjected to axial loading until failure. Axial load, axial deformation and strains were recorded using a data acquisition system throughout the loading history. During loading, the overall behaviour of the walls including failure modes was observed.

3. Results and Discussions

3.1 Failure Mode of the Profiled Steel Skin Concrete Composite Wall

The failure diagrams in Fig. 3 show the region of failure with steel sheet buckling and concrete cracking/crushing. The failure of the control composite wall (20^{o}C) and those exposed to 400^{o}C was due to cracking/crushing of concrete at the top accompanied by buckling of the steel sheeting in the same region. The region of failure was confined to the portion of the wall confined by the first and second row of bolts (acting as sheet-concrete connectors) from the top. It was also observed that the buckling of steel sheeting occurred between the first and the second line of bolts from the top (Fig. 3).

Control 400°C 600°C 800°C

Figure 3: Failure modes of DSCWs (20^{o}C~800^{o}C)

The ultimate failure of walls subjected to elevated temperatures (400^{o}C ~ 800^{o}C) was also due to concrete crushing at the top part of the wall accompanied by buckling of the steel sheeting in the same region. However, in these walls, the steel sheet buckling was confined from the top to the middle of the walls (Fig. 4).

3.2 Effect of Elevated Temperatures on Strain Characteristics

The axial load versus axial strain responses throughout the loading history for walls subjected to elevated temperatures of 20^{o}C, 400^{o}C, 600^{o}C and 800^{o}C for steady state duration of 2 hours are compared in Fig. 4. Generally, axial strain (at the top, middle and bottom of the wall) increased with the increase of axial load and temperature. For control wall (not subjected to elevated temperature), the strains at both top and bottom of the wall were less compared with those of walls subjected to elevated temperatures.

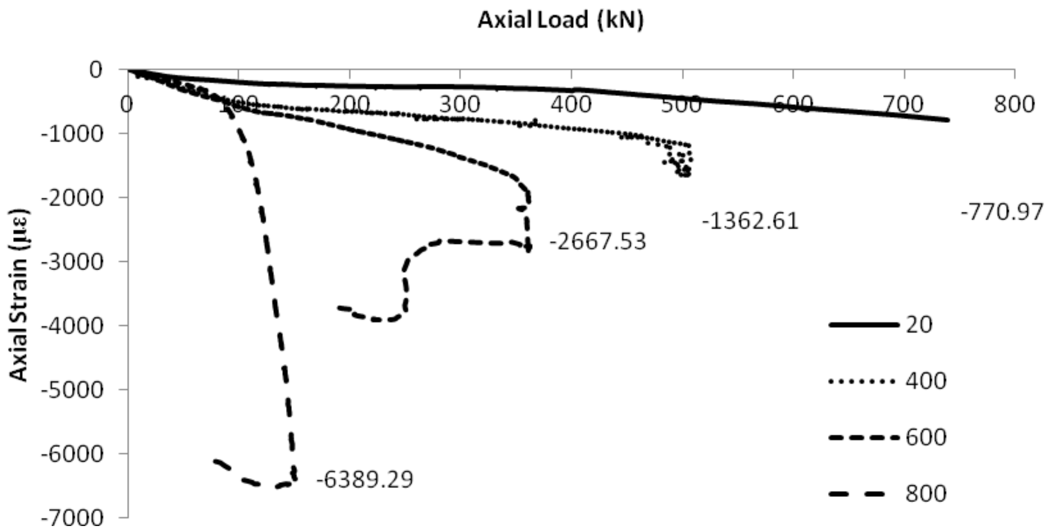

Figure 4: Comparison of strain development in walls exposed to high temperatures (top)

The axial compressive strain (at the top of the control wall - 20^{o}C) continued to increase with the increase of load and reached about 771 micro-strain at the maximum load which was less than the yield strain of steel (2680 micro-strain) indicating no yielding of the steel sheeting. This confirmed the fact that the control wall (without subjected elevated temperature) failed due to concrete crushing and localized steel buckling at the top region as observed in the experiment (Fig. 3). Comparatively closer strains at the top and centre of the wall also indicated that the load was transferred from the top to the centre of the wall – an indication of full mobilization of the composite action between steel and concrete.

The walls subjected to 400^{o}C showed an increased strain compared to control wall (maximum 1363 micro-strain at the top) at failure indicating that steel sheeting did not yield. At 600^{o}C and 800^{o}C, strains at the top (2668 micro-strain, 6389 micro-strain, respectively) of the wall indicated steel yielding at failure (Fig.4). Higher strain development in walls subjected to elevated temperatures can be attributed to the degradation of both steel and ECC properties.

3.3 Effect of Elevated Temperature on Load-Deflection Behaviour

The axial load-deflection responses of the walls subjected to elevated temperatures are compared with that of control wall in Fig. 5. For control wall, load gradually increased up to 744 kN and then dropped down to 570 kN (87% of maximum load capacity) followed by a flat descending branch up to failure. Such drop in load can be associated with the localized concrete crushing and steel buckling at the top of the wall. The axial stiffness of the control wall seemed to remain constant up to the peak load. The walls showed ductile failure with about 9.7 mm axial deformation at failure. Post-peak ductility of the control wall was associated with the high strain hardening capacity of the UHPC in-fill. The load-deflection responses of the walls subjected to high temperatures showed similar behavior compared to control wall except for the following differences:

- The axial stiffness and failure load decreased with the increase of temperature (Table 1).
- Walls also exhibited ductile failure at high temperatures (deformation at peak load increased with the increase of temperature, Table 1 and Fig. 5).

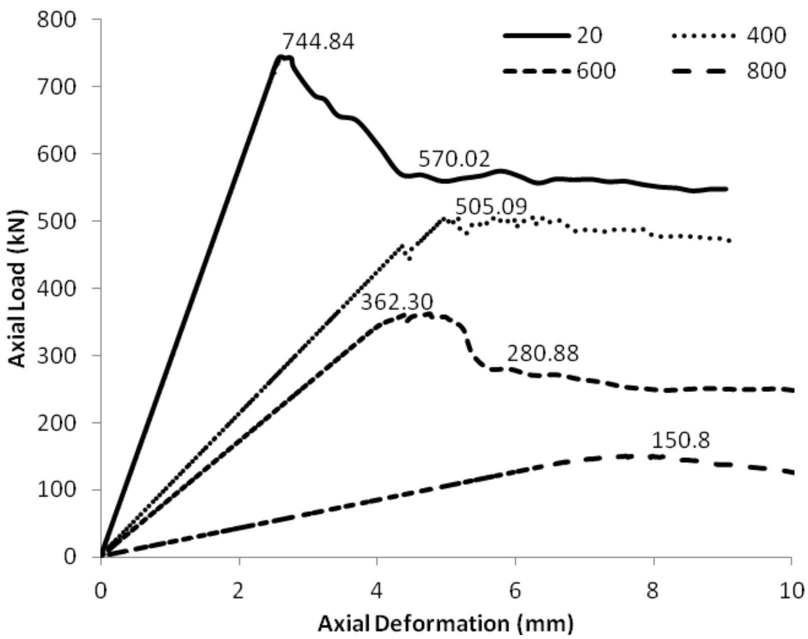

Figure 5: Comparison of axial load-deformation responses for walls exposed to high temperatures

Table 1: Axial strength and deflection of walls exposed to high temperatures

	Wall specimens			
Temperature	**20 °C**	**400 °C**	**600 °C**	**800 °C**
Axial Load (Net) (kN)	745	505*	362*	151*
Axial deformation at peak load (mm)	2.6	4.5	4.7	7.9
Initial stiffness (k_{et}) (kN/mm)	299	108	86	21
K_{et}/k_{e20}	1.0	0.36	0.30	0.07
N_{et}/Ne_{20}	1.00	0.68	0.49	0.20

*Residual strength after subjecting to elevated temperatures; K_e- experimental initial stiffness of the wall in kN/mm, K_{et}: experimental initial stiffness of the wall at a given temperature 't', K_{e20} :experimental initial stiffness of wall at 20°C, N_{et}: experimental axial load capacity of wall at a given temperature 't', N_{e20}: experimental axial load capacity of wall at 20°C.

3.4 Effect of Elevated Temperatures on Axial Strength and Stiffness of DSCWs

Table 1 also compares the axial strength and axial stiffness of DSCWs at various temperatures compared to control wall 20°C. Figs. 6 and 7 show the variation of axial strength/axial strength reduction and axial stiffness/stiffness reduction with temperature, respectively. Strength and initial stiffness decreased with the increase of temperature. DSCWs lost about 80% of its axial strength and 93% of its initial stiffness at 800°C. This substantial loss of strength and stiffness of the DSCWs was attributed to the poor performance of UHPC at high temperatures. The poor performance at high temperatures (especially between 600°C and 800°C) was substantiated by the explosive failure of UHPC control (cylinder specimens) specimens and substantial loss compressive strength.

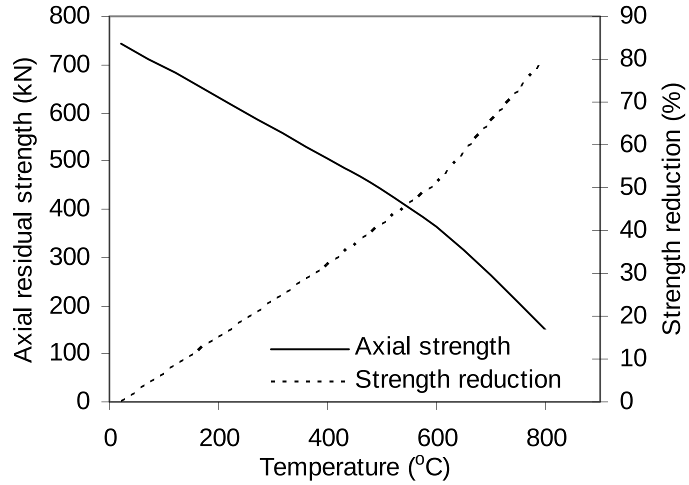

Figure 6: Residual strength of UHPC DSCWs subjected to elevated temperatures

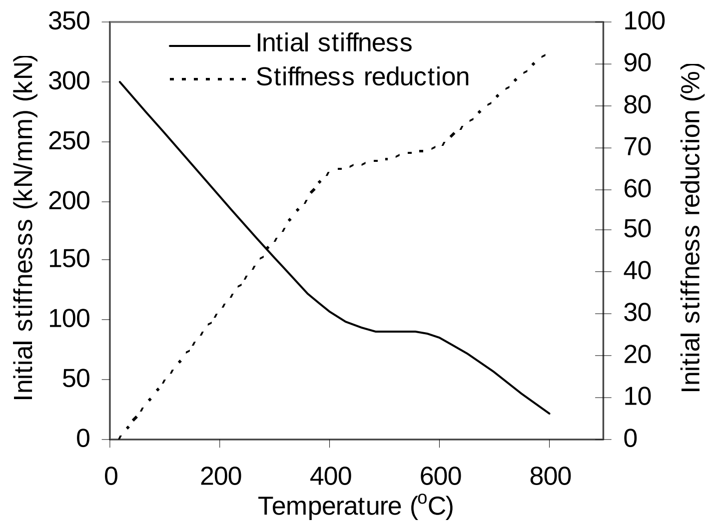

Figure 6: Residual stiffness of UHPC DSCWs subjected to elevated temperatures

5. Conclusions

The behaviour of double skin profiled composite walls composed of ultra high performance concrete (UHPC) is described after being exposed to elevated temperatures of up to 800°C for a duration of two hours. Axial strength and stiffness of walls are found to decrease with the increase of temperature. Residual strength and stiffness of the wall after exposed to 800°C are found to be around 20% and 7% of that of control wall (without subjected to elevated temperature). More research is required and currently in progress to study the wall behaviour at elevated temperatures. The information on fire durability of composite walls will be useful for the design of external fire protection measures.

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