GFRP connectors in textile reinforced concrete sandwich elements

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Abstract

In this paper, both experimental and numerical methods are presented to gain an understanding of the structural behaviour related to a TRC sandwich panel with a glass fibre reinforced polymer (GFRP) plate connection system. Double shear tests were conducted on component-scale sandwich panels to characterize the available shear capacity provided by the connectors and panel configuration. Three-dimension (3D) non-linear Finite Element Analysis (NLFEA) was applied to develop a model for the design of TRC sandwich panels while focusing on the connectors. The experimental outcome of the shear tests was applied to validate the corresponding numerical model developed in this work. The need for further modifications to the design of the shear connectors or other parameters such as panel thickness can be established accordingly. This developed FE model can essentially be applied as a design tool to further predict the structural behaviour of the full-scale sandwich elements.

Keywords: sandwich elements, textile reinforced concrete (TRC), glass fibre reinforced polymer (GFRP), shear connectors, experiments, finite element analysis (FEA).

1 Introduction

There is an increasing demand within the building and construction industry for durable, energy-efficient and affordable building components. The FP7 project H-house (Healthier Life with Eco-innovative Components for Housing Constructions) funded by the European Commission aims to develop a number of new building systems suited to a society where environmental awareness and a high degree of living comfort are both required. The concept of the project is to develop new building components for external and internal walls for new buildings and for renovation.

Within this project, the structural behaviour of novel load-bearing sandwich façade elements using thin textile reinforced concrete (TRC) panels, a lightweight foam concrete (FC) insulation layer along with glass fibre reinforced polymer (GFRP) shear connectors has been investigated. The use of GFRP connectors is beneficial in the sense that it has a lower self-weight and thermal conductivity than stainless steel and has been shown to be successfully implemented in thinner façade panels made of TRC [1, 2]. Moreover, due to the reduced thickness of the TRC panels, the design of suitable shear connectors is a challenge which makes up the main focal point of this study. The structural design of this particular solution is rather complex and as such it is investigated both experimentally and numerically.

2 Façade element components

The façade element typology that is studied in this work is a load bearing sandwich element composed of TRC and FC. The intention is that the inner TRC panel is designed to bear the imposed vertical loads along with its self-weight, while a thinner TRC panel is included as a facing layer. The FC core, a cementitious-based material, is included as the insulation layer. Figure 1 provides an overview of the geometry of the TRC-FC façade element.

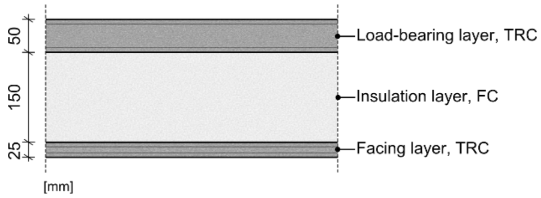

Figure 1. Overview of TRC-FC sandwich element concept

2.1 Materials

2.1.1 Textile reinforced concrete

TRC is an innovative high performance composite material which has been demonstrated to be a promising alternative for façade panel solutions. The TRC applied in this work consisted of a self-compacting fine-grained concrete matrix (d_{max} = 4 mm) reinforced by an epoxy coated bi-axial 2D carbon textile fabric (SOLIDGRID Q90-CEP-21, A = 85 mm^2/m). The primary benefit of replacing conventional steel reinforcement with non-corrosive textile fabric allows for the reduction of the concrete cover thus leading to thinner panels. Furthermore, the dead weight of the TRC panel can also be decreased due to the lower density of the carbon textile fabric (ρ = 1800 kg/m^3).

2.1.2 Foam concrete

FC is a cementitious based material composed mainly of cement, water and foam. It has a range of applications, primarily as filling material, ground insulation and pavement sublayer. In order to be used as a high performance insulation material for building applications, very low density FC was developed (ρ_{dry} = 120 kg/m^3). Given the high volume of foam, the main challenge was to guarantee that the cementitious matrix set quickly to sustain the porous structure without collapse of the foam. For this purpose, a binder system based on calcium-sulfo-aluminate cement was chosen. The successful incorporation of aerogels into FC allowed for a thermal conductivity of 30 mW/m·K.

2.1.3 GFRP connectors

Flat connectors made of stainless steel paired with reinforcement bars are typically incorporated into sandwich panels as so-called supporting anchors to carry vertical loads resulting from dead load, horizontal loads from wind and warping as well as eccentric loads [3]. In this work, the modification of this conventional flat anchor has been attempted using an alternative material, namely GFRP, for both the anchor and associated reinforcement bars. More specifically, the plate connector, exemplified in Figure 2, is made in an infusion process of E-glass textiles and epoxy resin. Composite bars are pultruded from E-glass direct rovings impregnated with epoxy resin.

Figure 2. Illustration of the GFRP flat connector

The plate connector was initially evaluated as a bolted connection using a FE model developed in ESAComp (v. 4.4.0), whereby the bearing capacity of one bar was estimated to be 4,14 kN. Several laminates were calculated to determine the optimal construction of the plate. Moreover, two alternatives, denoted as Alternative 1 and 2, were investigated in this work. Alternative 2 fundamentally has the same geometry as Alternative 1, yet it encompasses a core material (PVC foam) in the web of the connector. Four reinforcement bars were included on either side of the plate.

2.2 Structural behaviour

Precast concrete sandwich elements typically consist of three segments: an external facing layer (facing or wythe), a thermal insulation, and an internal layer (wythe). The design and production of these three layers can be made to achieve non-composite, partially composite or fully composite behaviours. Non-composite signifies that the facings are independent of each other, while partially composite elements in part transfer shear stresses by means of ties connecting the wythes. As for composite elements, the facings are designed to resist loads as a unit, i.e. full-composite action; accordingly, full shear transfer occurs between the facings [4]. In the developed TRC-FC solution, the structural behaviour highly depends on the strength and stiffness of the incorporated GFRP connectors due to the inherent low stiffness of the FC and minimal thickness of the facings. It is to say that this solution will perform as a partially composite sandwich element. In view of that, the interfacial bond between the FC and TRC panels and the contribution of the core stiffness are considered to be insignificant in the accompanying experimental work and FE-analysis. Furthermore, the level of composite action, i.e. range between non-composite and fully composite, pertaining to the TRC-FC elements will be determined in terms of bending stiffness and strength experimentally using four-point bending tests in future work.

3 Shear test of sandwich element

The purpose of conducting shear tests on the TRC-FC sandwich element was to be able to characterize the shear capacity of the given shear connectors embedded in the sandwich element. It is anticipated that the connector design will be further optimized and the structural behaviour understood using this test method coupled with the FE-modelling presented.

3.1 Test parameters

3.1.1 Specimens

The test specimens were configured as a symmetrical double sided specimen whereby two sandwich elements, as per the thicknesses in Figure 1, were joined at the inner load bearing layer. The thickness of the inner load bearing layer thus amounted to 100 mm. The double shear test is useful as it helps minimize the possible eccentricity of the applied shear load. The specimen geometry and placement of the connectors are illustrated in Figure 3. The FC core was excluded from the test specimens to simulate a worst case scenario such that the FC was assumed to have no contribution to the shear transfer. Moreover, it was found necessary to incorporate four connectors to further stabilize the specimen system and displacement measurements of the middle panel.

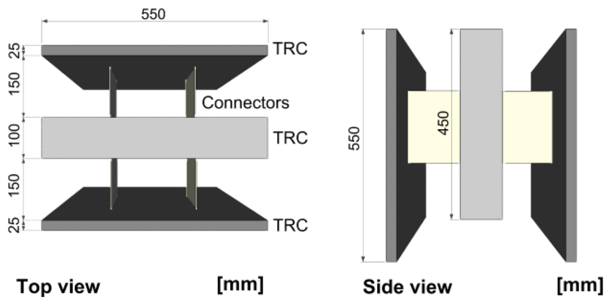

Figure 3. Geometry of test specimen

The elements were denoted according to the connector alternative incorporated in the specimen, i.e. Alternative 1 (A1-X) and Alternative 2 (A2-X). Three specimens of each variation were tested to obtain a representative average behaviour.

3.1.2 Setup

The double shear test setup was partly based on similar studies related to shear testing of connectors in TRC sandwich elements, refer to [1, 5]. The shear tests were conducted in this work using a servo-mechanical testing machine (Instron 1195) and the force was recorded by a load cell with a rated capacity of 100 kN. The sandwich specimen was secured into place using a supporting frame made of hollow steel sections which clamped the top and bottom outer TRC panels respectively (see Figure 4). This so-called frame also rotationally restrained the outer panels, while the testing table restrained vertical movement. The load P was evenly distributed to the middle TRC panel via a square hollow steel section acting as a loading beam. The relative displacement was measured on either side of the centre of the middle TRC panel using two linear variable differential transformers (LVDT). The front and back displacement values were compared and it was found suitable to take an average displacement, as further discussed in Section 3.2. The force and deformation were recorded in a data acquisition system with a sampling rate of 10 Hz. The specimens were generally preloaded by a force of 1,5-2,0 kN to allow for the stabilization of the LVDT readings. The tests were controlled by the displacement rate of 0,5 mm/min.

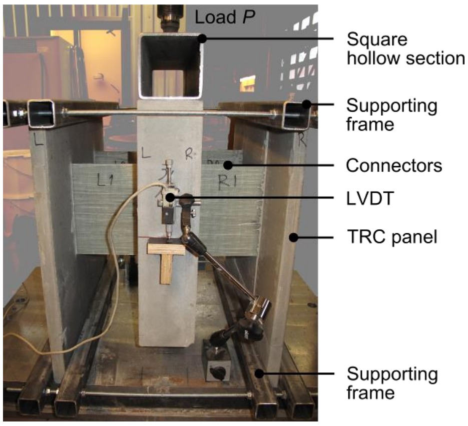

Figure 4. Overview of double shear test setup

3.2 Test results

The shear tests of the sandwich elements were evaluated based on the maximum load, average relative displacement of the middle panel, crack pattern and failure mode. When comparing the load versus average displacement for both Alternatives 1 and 2, it can be generally stated that a similar pre-peak behaviour was observed with the exception of A1-1. The deviation of the initial stiffness for A1-1 is thought to be due to the imperfect placement of the textile reinforcement fabric to the outer surfaces of the middle panel, which in turn minimized the embedment depth of the connector causing premature surface spalling. Moreover, there is a deviation in the maximum load values, P_{max}, which could be largely a result of production flaws of the connectors and panels.

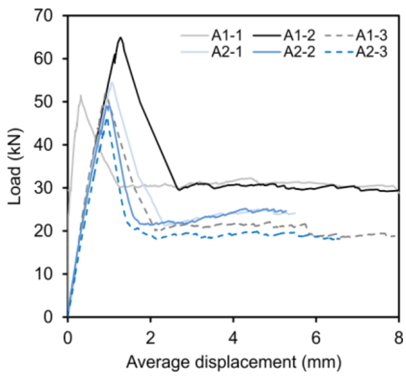

Figure 5. Load versus average displacement for Alternatives 1 and 2

A summary of the main experimental results for all tested specimens are provided in Table 1. The maximum load and corresponding average displacement are presented. As well, the utilized shear capacity per bar in a connector was estimated using a linear shear stress distribution model for a connector as illustrated in Figure 6. It can be seen that the largest deviation is indeed observed for the maximum load values. As well, the average utilized capacity of the bar amounts to approximately 75-85% of the bearing capacity estimated numerically (4,14 kN).

Table 1. Result summary

Specimen	Max load, P_{max} [kN]	Average displacement, u_{avg} at P_{max} [mm]	Utilized capacity per bar, R [kN]
A1-1	51,4	0,3	3,2
A1-2	64,8	1,3	4,1
A1-3	52,0	0,9	3,3
Avg(stdev)	**56,1(7,6)**	**0,8(0,5)**	**3,5(0,5)**
A2-1	54,4	1,1	3,4
A2-2	49,0	1,0	3,1
A2-3	46,4	0,9	2,9
Avg(stdev)	**49,9(4,1)**	**1,0(0,1)**	**3,1(0,3)**

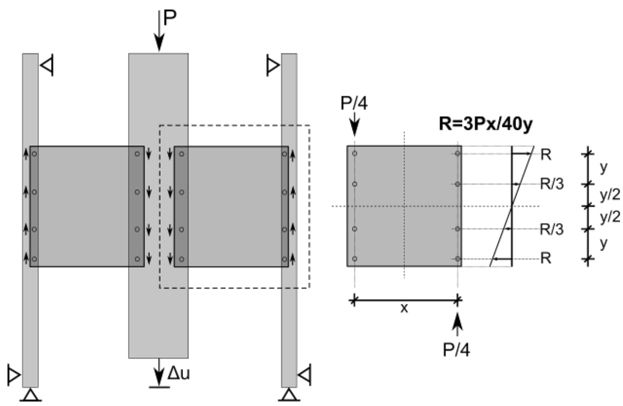

Figure 6. Load distribution and linear shear stress distribution across a connector

It was of further interest to compare the displacement measurements of the two LVDTs over the course of testing. The results related to specimen A2-2 are exemplified in Figure 7 yet a similar trend was noted for all specimens. A minimal discrepancy between the measurements was noted for the pre-peak, while there was no difference at P_{max} and slightly after this point. Thereafter, the non-linear behaviour initiated which caused a larger yet still minimal difference between the displacement values due to the introduction of certain eccentricities in the specimen, i.e. cracking, spalling.

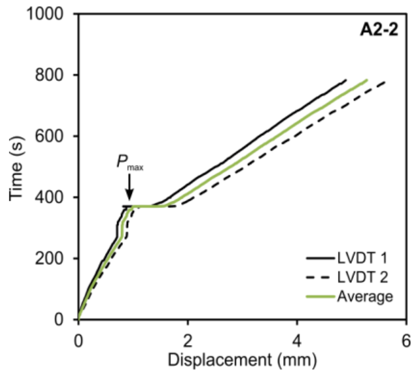

Figure 7. Example of time versus displacement (A2-2)

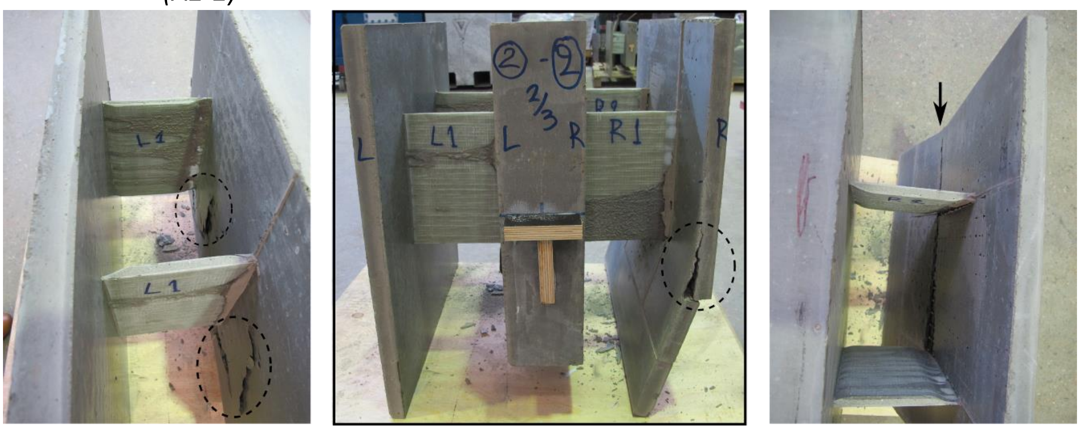

Figure 8. Typical failure mechanisms (A2-2)

The main failure mechanism was marked by concrete cracking for all specimens which can signify that the connectors had a superior bearing capacity to that of the given sandwich element system. To further utilize the capacity of the connectors, the design of the sandwich element can be altered in terms of the panel thickness of and/or reinforcement ratio; alternatively, the connectors could be further optimized. Taking a closer look at specimen A2-2 in *Figure 8*, damage to the specimen initiated in the form of spalling at the bottom edges of the connectors on the inside of the middle panel, which was followed by horizontal cracking at the bottom edge of the connectors on the outer panels. These failure mechanisms were comparable in all tests.

4 Modelling

A multi-level structural analysis of sandwich panels has been proposed in Miccoli *et al.* [6] which is based on the multi-level structural assessment strategy for reinforced concrete bridge deck slabs developed in Plos *et al.* [7]. The first level of structural analysis starts with an initial design using simplified analysis methods. However, for an in-depth evaluation of a design as well as optimization of a design, more advanced structural analyses describing all possible failure modes and resistance models that are more accurate and reliable are needed. Three most advanced methods in the multi-level structural analysis are described below.

3D linear shell analysis – Here, the structural analysis is performed using 3D FE models, primarily based on shell or bending plate theory. The analysis is made assuming linear response to be able to superimpose the effect of different loads, in order to achieve the maximum load effects in terms of cross-sectional forces and moments throughout the structure for all possible load combinations. Since both geometrical simplifications and the assumption of linear material response result in unrealistic stress concentrations, and because the rebar are normally arranged in strips with equal bar diameter and spacing, the redistribution of the linear cross-sectional forces and moments are necessary. Recommendations on redistribution widths for bending moments and shear forces are given in Pacoste *et al.* [8]. The structural analysis can be seen as "linear elastic with limited redistribution" according to Eurocode 2 [9]. The load effect is then compared with corresponding resistance in similar way as in simplified analysis.

3D non-linear shell analysis – In a non-linear analysis, the loads are successively increased until failure of the structure is reached. In practice, due to the excessive amount of work it would require, non-linear analysis cannot be made for all possible load combinations, but only for the most critical loads. At this level, shell (or bending plate) finite elements are used. The reinforcement is included in the FE model but assumed to have perfect bond to the concrete; it is preferably modelled as embedded reinforcement layers in the shell elements, strengthening the concrete in the direction and at the level of the reinforcement bars. In such a model, bending failures will be reflected in the analysis, whereas out-of-plane shear, punching, or anchorage failures are not reflected. Instead they must be checked by local resistance models.

3D non-linear continuum analysis – Compared to the 3D non-linear shell analysis, the reinforcement is modelled using separate finite elements. Furthermore, the bond-slip behaviour of the interface between the reinforcement and the concrete is included. With a fine mesh, individual cracks can be studied and anchorage failure can be reflected in the analysis. With this level of accuracy in the structural analysis, the intention is that no major failure modes should be necessary to check using separate resistance models. In the present work, 3D non-linear continuum analysis has been used to model the structural behaviour of the shear tests, and to further optimize the design of the shear connectors and other parameters such as panel thickness.

4.1 3D non-linear continuum analysis of shear test

Given the symmetry lines, only a quarter of the TRC panel was included in the FE model. Similar boundary conditions as in the experimental set-up were adopted in the FE model. Concrete and connectors were both modelled with 3D solid elements which are based on numerical integration with 1-point integration scheme over the volume. The mean element size was 10 mm. The textile reinforcement was modelled as grid reinforcement, embedded in solid elements corresponding to full interaction.

4.2 Material properties

For the compressive behaviour of the concrete, a modified Thorenfeldt curve was used. The original Thorenfeldt curve describes the stress-strain relationship of a 300 mm long cylindered concrete specimen; see Figure 9. As the strain values in the curve are dependent on the specimen length, the strain needs to be modified to the length of the crushing elements in the model according to [11]. It was then assumed that crushing occurs in one element row above or below the embedded connectors; this was later verified in the analysis and therefore the Thorenfeldt curve was modified to the appropriate size of 10 mm; see Figure 9. For tensile behaviour of the concrete, the tension softening was taken into account using a predefined Hordijk´s curve in DIANA. The material properties adopted are reported in Table 2.

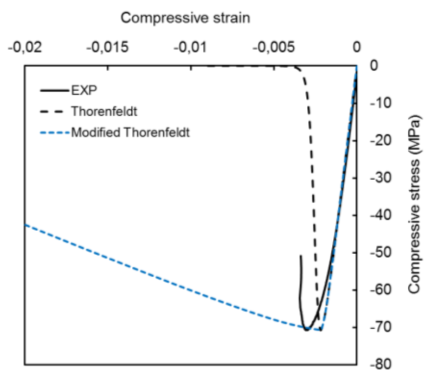

Figure 9. Concrete behaviour in compression

Table 2. Material properties

Concrete	Compressive strength [MPa]	70.8
	Young´s modulus [GPa]	35.7
	Tensile strength [MPa]	3.2
	Fracture energy [N/m]	96
	Poisson’s ratio [-]	0.2
Textile	**Linear elastic material model**	
	Young´s modulus [GPa]	230
	Poisson’s ratio [-]	0.2
Connector	**Non-Linear material model**	
	Tensile strength [MPa]	91.6
	Shear strength [MPa]	69
	Young´s modulus: $E_{(x)}$ and $E_{(y)}$ [GPa]	24.4
	Young´s modulus: $E_{(xy)}$ [GPa]	7.0
	Poisson’s ratio [-]	0.2

4.3 Analysis procedure

The crack model chosen for the concrete was a total strain based model with rotating crack approach. The effective bandwidth length was assumed to match the element sizes. An incremental static analysis was made using an explicitly specified load step size and a Newton–Raphson iterative scheme to solve the non-linear equilibrium equations.

4.4 Results and discussion

Three FE analyses were carried out using different material properties. The results in term of load versus average displacement curves, similar to that in Section 3, are shown in Figure 10. It is to say that the curve corresponding to experimental result represents the average of the three shear tests in Alternative 2; i.e. the average of A2-1, A2-2 and A2-3.

In the first analysis (FE-1), the material properties presented in Table 2 were used. The analysis showed that the specimen failed at the load level of approx. 52,5 kN due to spalling of concrete on the inner panel just below the connectors. Thus, the analysis overestimated the ultimate resistance of the shear specimen by approximately 5%. However, shortly after the maximum capacity had reached, the load dropped to approximately 28 kN and followed by a plateau. At this point, a few bending cracks took place on the outer panel which grew with an increased load. Nonetheless, a relatively small stress level was observed in the connectors and in the textile reinforcement (in both inner and outer panels), and no sign of buckling in the TRC panels was observed. This indicates that the connectors were appropriately designed. Nevertheless, the maximum load resistance of the specimen was associated with the tensile strength of concrete on the inner TRC panel and the load level over the plateau was governed by the bending capacity of the outer TRC panel. To verify the conclusions above, two extra FE analyses were carried out as described below.

In the next analysis (FE-2), the tensile strength and compressive strength of concrete for the inner panel was increased by 30%, which hypothetically would have the same impact as if the thickness of the inner panel had been increased. The results shown in Figure 10 illustrate that the specimen exhibits the same failure mechanism as in analysis FE-1, and that the increased concrete strength has led to an increased load-carrying capacity of the specimen. This observation approves that the maximum load resistance of the specimen is associated with concrete strength on the inner TRC panel.

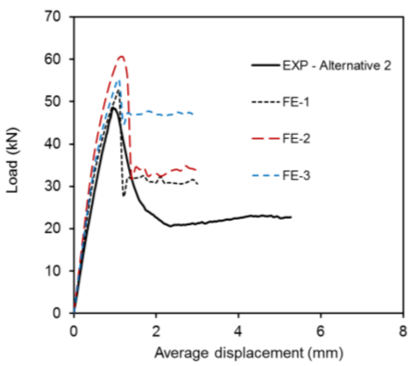

Figure 10. Load versus average displacement for test Alternative 2 and three FE analyses

In the third analysis (FE-3), the inner panel was modelled similar to that in analysis FE-1; whereas, the amount of textile reinforcement was increased by 30%. This analysis has shown relatively similar maximum load resistance as in FE-1; the specimen however maintained much higher load level after the maximum load over the plateau. This observation verifies that the level of load over the plateau is governed by the bending capacity of the outer TRC panel.

5 Conclusions

Overall, the gained understating of the global behaviour of shear specimens through both the tests and FE analyses indicates that:

- The designed plate connection system performed relatively well and enabled a partially composite behaviour.
- The maximum load capacity of the specimen was governed by the tensile strength of concrete on the inner TRC panel, and thus a higher capacity can be reached by improved concrete strength or increased thickness of the inner panel.
- The load level over the plateau was governed by the bending capacity of the outer TRC panel, and thus a higher load level over the plateau can be reached by increased textile reinforcement or increased thickness of the outer panel.
- Further design optimization of the shear specimen with the aim to obtain a well-balanced composite action should be focused on: (a) the use of least material in the plate connectors, and (b) increased bending capacity of the outer panel to avoid a significant drop of the load after the peak load.

6 Acknowledgements

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