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Non-linear FE modelling of plain and CFRP strengthened masonry panels under out-of-plane loads

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ABSTRACT
In this study, finite element FE analysis is conducted to investigate the non-linear behaviour of masonry panels subjected to out of plane loading and to assess the improvement in structural behaviour of Carbon Fibre Reinforced Polymer (CFRP) strengthened masonry panels. Three dimensional FE analyses are conducted using the commercially available software Abaqus. The masonry panels are modelled using a simplified micro modelling approach. In the FE analysis, the masonry units are modelled using brick elements, the interactions between the masonry units themselves, and masonry and CFRP surfaces are simulated using a cohesive surface based approach, the CFRP material is modelled using shell elements. The viability of the modelling technique in simulating the non-linear structural behaviour for both plain and strengthened cases is verified by modelling two experimental studies of masonry panels available in the literature (one plain and one CFRP strengthened). Finally, numerical parametric studies are carried out to investigate the effects of geometry and CFRP reinforcement ratios on structural performance of masonry panels under out of plane loads.

KEYWORDS: masonry out of plane behaviour, CFRP reinforcement, finite element modelling, cohesive surface based approach, micro model

NOTATION

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k$</td>
<td>initial elastic stiffness of interfaces</td>
</tr>
<tr>
<td>$t$</td>
<td>traction between interfaces</td>
</tr>
<tr>
<td>$\delta^0$</td>
<td>displacement between the interfaces at which damage initiates</td>
</tr>
<tr>
<td>$\delta^1$</td>
<td>maximum displacement between the interfaces at which the total strength is lost</td>
</tr>
<tr>
<td>$\sigma_{max}$</td>
<td>maximum allowable stress between interfaces, at which damage initiates</td>
</tr>
<tr>
<td>$G_f$</td>
<td>fracture energy which governs damage evolution between the interfaces</td>
</tr>
</tbody>
</table>

1. INTRODUCTION

Masonry is one of the most commonly used building materials in the world today, and is particularly prevalent in residential construction. A significant proportion of the world's masonry structures are located in regions prone to natural hazards such as earthquakes, cyclones etc. Unreinforced masonry structures can be highly vulnerable to the horizontal forces associated with these extreme events and hazards. This is due to the relatively low tensile strength of masonry constituents and the inherent brittleness. In such circumstances, the failure and collapse of masonry structures presents a real risk to occupants and is a major cause of casualties. Therefore, a detailed understanding of the structural behaviour of masonry under such horizontal forces i.e. out of plane loads, and how to efficiently strengthen them are vital requirements for resilience. The strengthening of existing masonry can most conveniently be carried out by applying surface strengthening materials. Nowadays, Fibre Reinforced Polymer (FRP) composites are one of the most popular materials employed to upgrade and strengthen masonry structures. The application and use of FRP materials is advantageous due to several attractive features such as ease of application, their comparative light-weight, high tensile strength, durability etc. A masonry wall surface-strengthened with FRP primarily aims to enhance tensile resistance and avoid sudden failure associated with brittle behaviour.

In the recent decades, experimental studies have been conducted to capture and evaluate the structural performance of plain and FRP strengthened masonry panels under out of plane loads, examples of such studies can be found in [1], [2] and [3]. In addition to the experiments, FE analysis can be an efficient tool in the evaluation and prediction of plain and strengthened masonry behaviour under different loading conditions. In general, there are two main FE modelling approaches adopted to simulate masonry, namely, the micro-modelling approach and the macro-modelling approach. In the former approach, both units and mortar can be represented by continuum elements, and the unit-mortar interface is represented by discontinuum elements (referred to as a detailed micro model), Figure 1 (a). The detailed micro model can be simplified, Figure 1 (b), by adding the mortar thickness into the masonry units. The subsequent expanded units are modelled using continuum elements, while the interactions between the expanded units are modelled using discontinuum elements. In the macro model approach, the masonry is considered as a homogenous material, Figure 1 (c), and an equivalent property based on the properties of the masonry constituents is assigned. The decision on choosing a particular modelling approach is usually governed by considering the required accuracy and simplicity [4].

In this study, the simplified micro modelling approach is adopted to simulate both plain masonry and CFRP reinforced masonry panels under out of plane loads. Firstly, the viability of the numerical modelling technique is verified by comparing the numerical results with the experimental results available in the literature. Following this, parametric studies are conducted to assess the effects of geometry and reinforcement ratio on the performance of masonry panels under out of plane loads.
2. DESCRIPTION OF TEST SPECIMENS

As mentioned earlier, two masonry panels (a plain wall and a CFRP strengthened wall) are adopted and simulated to validate the numerical modelling technique. The walls were tested by Accardi et al. [3]. The walls were built from calcareous ashlar units. The dimensions of the walls were 740 x 210 x 2100 mm. The equivalent elastic modulus of the masonry was considered as 2000 N/mm² and average compressive strength of the masonry assemblage was taken as 4 N/mm² as reported in [5] and the Poisson’s ratio was 0.3. The strengthened wall was reinforced with four vertical CFRP strips; each strip was 50 x 2100 mm² and 0.13 mm thick, Figure 2 (a). The CFRP elastic modulus was 230,000 N/mm² and the tensile strength was 3540 N/mm². The CFRP strips were fixed to the wall by epoxy resin. The experiments were carried out in two steps. In the first step, a constant axial vertical load of 80 kN was applied to the top of the walls. In the following step, a monotonic displacement was imposed to the walls at the bottom, Figure 2 (b). The boundary condition at the top of the walls was pinned i.e. rotations were released. At the bottom, the horizontal transition parallel to the loading direction was free, all rotations and movements in the vertical and other horizontal directions were restrained.

3. DESCRIPTION OF NUMERICAL MODEL

The numerical models were developed in the non-linear FE programme ABAQUS. The expanded masonry units were modelled using 8 node hexahedral linear brick elements with reduced integration and hourglass control (C3D8R) [6]. The CFRP material was simulated via 4 node shell elements with hourglass control and 5 integration points through the thickness following Simpson’s rule (S4R) [6]. For the mesh, the element size of the expanded masonry units was chosen as 50 mm and 25 mm for the CFRP, these values were adopted based on a mesh sensitivity analysis. The first CFRP strip was fixed to the wall 50 mm from the wall edge, the CFRP strips were spaced at 195 mm centre to centre. The interactions between all expanded masonry units, and the masonry wall and CFRPs were simulated based on a cohesive surface based approach. This approach follows the traction separation law, the interaction behaviour is assumed as linear based on a user defined initial elastic stiffness (k). The linear behaviour continues until the stresses (t) reach the user defined strength value (σ_u) between two contacting surfaces; this is when the damage initiates. Following this, the damage evolution occurs which can be based on the user defined fracture energy (G_c) between the contacting surfaces, this continues until the loss of total strength between the contacting surfaces Figure 3 [6]. The list of properties used to define the interactions between the contacting surfaces are summarised in Table 1 [7]. In this model, the compressive plastic behaviour and potential compressive failure of the masonry were also considered by defining the stress-strain curve of the masonry system for the expanded masonry units. The stress-strain curve (Figure 4) was developed based on the given ultimate compressive strength in [5] and the details provided in [8]. The analyses were conducted in ABAQUS Standard version 6.13 by following a Newton-Raphson algorithm solution. Two steps were defined; geometric non-linearity was taken into account for both steps. In the first step, the axial vertical load was applied to the top of the wall. Following this, the horizontal displacement was monotonically imposed to the bottom of the wall under displacement control.

4. VALIDATION OF NUMERICAL MODEL

The masonry walls (plain and CFRP strengthened) were modelled following the method described above. Then a direct comparison between the experimental results reported in [3] and numerical results was undertaken. The first comparison was the response of the wall by means of horizontal displacement versus the horizontal force. Secondly, the comparisons between the experimental and numerical failure modes were conducted. A good agreement between the experimental and numerical results was observed in terms of the response and failure modes for both plain and CFRP strengthened walls.

For the plain wall, the maximum horizontal load sustained in the experiment [3] was 2.8 kN and the mode of failure was characterised by the formation of cracks due to opening of four bed joints at the bottom of the wall. A similar response was observed in the numerical model, the maximum horizontal force was 2.76 kN, Figure 5 shows the comparison between horizontal displacements versus horizontal forces obtained from the experiment and the FE model. Figures 6 (a) and (b) also show the failure mode and distribution of compressive stresses, it can be observed that the openings of the bed joints at the bottom of the wall are in good agreement with the experiments. From the compressive stresses distribution, it can be said that no compressive failure i.e. crushing occurred as the stresses are below the compressive strength of the wall, this also agreed with the experiments as no compressive failure was reported.

For the CFRP strengthened wall, the maximum horizontal load recorded from the experiment was 3.86 kN, while it was 3.98 kN from the FE model. Figure 5 shows a good agreement between the experimental and numerical responses in terms of horizontal displacement verses horizontal load. It was reported in the experiments that the initial stiffness of the CFRP strengthened wall was not affected by the CFRP reinforcement. As can be noted in Figure 5, the initial stiffness for both plain and reinforced numerical models is the same. After the formation of cracks due to opening of bed joints, then the reinforcement contributed by taking tensile stresses and consequently improved the ultimate horizontal load carrying capacity of the wall. The same scenario was also observed from the FE model, the opening of bed joints first occurred at the bottom of the wall; this corresponded to a horizontal displacement of 8.5 mm at the bottom Figure 7 (a). After that, the CFRP reinforcement began to take more stresses. Figures 7 (b), (c) and (d) respectively show the failure mode of the CFRP strengthened wall, compressive stress distribution in the masonry wall and tensile stresses in the CFRP strips at the end of analysis i.e. horizontal displacement equal to 35.13 mm. It can be noted that the compressive stress values are below the compressive strength of the wall, thus crushing under compression has not taken place, and again this was the case with the experimental results.

5. PARAMETRIC STUDIES

A series of parametric studies were conducted to study the geometric effects on response of plain masonry walls under out of plane loading by investigating different slenderness ratios. A second study focused on the effects of CFRP reinforcement ratios on enhancing the structural behaviour of strengthened masonry walls under out of plane loading.

5.1 Effect of wall slenderness

In the original experiments, the slenderness ratio (height/thickness) of the plain wall was 10. This wall was
considered as the control wall. For the parametric study, two walls with different slenderness ratios were tested, namely, 12.5 which was more than the control wall's slenderness ratio by 1.25 and the slenderness ratio of the second wall was 7.5 which was 0.75 of the control wall’s ratio. In both cases, the walls were left unreinforced, the material properties and the loading conditions were the same as the control wall. As would be expected, it was observed that the wall with the higher slenderness ratio had a lower resistance to horizontal force compared to the control wall; the maximum horizontal force sustained by the relatively taller wall (with 12.5 slenderness ratio) was 1.91 kN. This value was 68% of the maximum horizontal resistance force of the control wall. Figure 8 shows the comparisons between the responses of the wall with different slenderness ratios. For the relatively shorter wall (slenderness ratio = 7.5), the maximum horizontal force resistance was 4.98 kN, more than the resistance of the control wall by 1.78 times. In terms of the failure characteristics, the control wall’s failure mode was previously shown in Figure 6 (a), the failure mode of the wall with 12.5 slenderness ratio was similar to the control wall Figure 9 (a). In the relatively shorter wall, the failure modes can be described as the opening of the lower three bed joints Figure 9 (c) and crushing of the wall toe under compression. It can be observed in Figure 9 (d) that the compressive stresses reached the ultimate compressive strength value (4 N/mm²) of the wall. While the compressive stresses were lower than the ultimate compressive strength capacity of the walls with higher slenderness ratios Figure 6 (b) and Figure 9 (b).

5.2 Effect of CFRP reinforcement ratio
The effects of reinforcement ratio on the out of plane behaviour of the masonry walls were considered by testing three different reinforcement arrangements in addition to the experimentally tested reinforcement wall. In the experiment, the wall was reinforced with 4 CFRP strips; the ratio of reinforcement area over the masonry wall surface area was 27%. In this study, the experimentally tested wall was considered as a reference wall. The reinforcement ratios of the three other walls analysed were 6.75% (1 CFRP strip, positioned along the central vertical axis), 54% (8 CFRP strips, position of first strip was fixed right at the edge of the masonry wall, spacing between the strips was 100 mm centre to centre) and 100% (CGRP strips continuous across the surface), CFRP thickness was kept constant throughout.

The walls were analysed under the same constraint and loading conditions and using the same material properties and geometry as the strengthened wall used in the validation of the numerical models. It was observed from the numerical analyses that the reinforcement arrangements did not improve the initial stiffness of the walls in each case, the contribution of the reinforcement in carrying tensile stresses started after opening of bed joints at the bottom of the walls; it is worth mentioning the same mechanism was reported from the experiments [3]. Figure 10 shows the comparisons between the horizontal force-displacement responses of all the reinforced walls, it can be noted that the elastic response of all walls are similar, this confirms the absence of reinforcement effects on improving the elastic behaviour. After the bed joint openings, the stresses were transferred from the masonry walls to the CFRP strengthening. The walls with higher reinforcement ratios possessed higher lateral force resistance. The wall reinforced with 1 CFRP strip (this reinforcement was 25% of the reference wall’s reinforcement) sustained a maximum lateral force of 2.9 kN, this force was 73% of the reference wall’s horizontal force capacity. In the wall reinforced with 8 CFRP strips (double the reinforcement of the reference wall), the maximum horizontal force sustained was 5.3 kN, which was 1.36 times greater than the capacity of the reference wall. The lateral resistance of the wall with 100% reinforcement ratio was 7.75 kN, which was almost twice the capacity of the reference wall. It can be noted that the improvement in lateral resistance does not linearly increase with increasing the CFRP reinforcement ratios. In addition, the failure mechanism of all the walls was similar i.e. opening of bed joints at the bottom of the walls Figure 11 (a), (b) and (c). It was noted that tensile stresses in the CFRP materials tend to be lower as the reinforcement ratio increases, Figure 12 (a), (b) and (c). In all cases, the maximum tensile stresses in the CFRP were less than 1/3 of the tensile capacity. The higher lateral forces were sustained by the walls with higher reinforcement ratios: this was due to redistribution of stresses afforded by the CFRP whereby the compressive stresses were transferred to the unreinforced face of the walls. Correspondingly, the walls with higher reinforcement ratios had higher compressive stresses on the unreinforced face Figure 13 (a), (b) and (c).

With a view to efficient and economic engineering in practice, a glass fibre reinforced polymer (GFRP) reinforcement arrangement was also tested in this study. GFRP is relatively less expensive than CFRP, so it was worth examining the enhancement effects of GFRP in comparison to CFRP, especially given the comparatively low stresses in the CFRP arrangements. For this purpose, a 4 strip GFRP reinforcement was considered with the same arrangement as the experimentally tested wall with 4 CFRP strips, the modulus of elasticity and tensile strength of GFRP composites were considered as 75000 N/mm² and 1550 N/mm², respectively, based on GFRP properties available in [9]. The thickness of the GFRP strips and interaction properties between the GFRP and the wall were similar to the CFRP cases i.e. 0.13 mm thickness and the same epoxy resin were assumed. The FE results showed that the ultimate horizontal load carrying capacity of the GFRP strengthened wall was 3.18 kN, which was 82% of the ultimate horizontal load sustained by the 4 CFRP strip-reinforced wall and it was more than by 1.1 times the wall strengthened with 1 CFRP strip. Figure 14 shows a comparison between the horizontal load displacement responses of the walls strengthened with 4 CFRP strips, 1 CFRP strip and 4 GFRP strips. The failure mode was similar to the other CFRP strengthened cases, which was opening of bed joints at the bottom of the wall Figure 15 (a). The tensile stresses in the GFRP strips were again less than 1/3 of the ultimate tensile capacity Figure 15 (b), the masonry compressive stresses were slightly more than the compressive stresses in the walls strengthened with 1 and 4 CFRP strips Figure 15 (c).

6. CONCLUSIONS
In this study, FE numerical analyses using a simplified micro model were conducted to capture the non-linear behaviour of plain and strengthened masonry walls subjected to out of plane loads. In the three dimensional FE models, failures due to opening of masonry joints and crushing of masonry under compression were taken into account. The FE models were firstly validated against experimental results available in the literature. The FE models were shown to be capable of capturing the ultimate horizontal load capacities, deflections and failure mechanisms for both plain and CFRP reinforced masonry walls. On this basis, parametric studies were undertaken to study the effects of slenderness ratio and the CFRP reinforcement ratio on the non-linear out of plane response of the masonry walls. To sum up, the plain walls with
relatively lower slenderness ratio possessed larger ultimate horizontal load carrying capacity. The failure modes of walls with relatively lower slenderness ratios were a combination of opening of lower bed joints and toe crushing of masonry under compression. In the reinforced walls, the horizontal load carrying capacities increased with increasing the ratio of CFRP reinforcements. The effects of the reinforcement became apparent after the opening of bed joints, thus the elastic response of the reinforced walls was the same regardless of reinforcement ratios. In addition, the higher reinforcement ratios enabled transfer of greater compressive stresses to the unreinforced face and at the toe of the walls. Therefore, crushing of toes should be checked for when significant reinforcement ratios are being applied in practice, otherwise it could lead to catastrophic sudden failure of masonry under compression.

It was also shown that GFRP composites successfully enhanced the load carrying capacity of masonry under out of plane loads. Although less than the enhancement obtained from the CFRP with the same reinforcement ratio, the effect was comparable. In this case, the use of GFRP was an effective alternative to CFRP composites. Nevertheless, the choice of the masonry strengthening material is based on the degree of enhancement desired.

ACKNOWLEDGEMENTS
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REFERENCES
Table 1
The summary of mechanical properties used to define interactions between contacting surfaces (extracted from [7])

<table>
<thead>
<tr>
<th>Type of interaction</th>
<th>$k$ (N/mm$^3$)</th>
<th>$\sigma_{\text{max}}$ (N/mm$^2$)</th>
<th>$G_f$ (N/mm)</th>
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</thead>
<tbody>
<tr>
<td>Interaction between expanded masonry units</td>
<td>34.78</td>
<td>0.055</td>
<td>0.0025</td>
</tr>
<tr>
<td>Interaction between masonry and CFRPs</td>
<td>8.33</td>
<td>0.5</td>
<td>0.1028</td>
</tr>
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</table>

Figure 1  Finite element modelling techniques: (a) detailed micro modelling, (b) simplified micro modelling and (c) macro modelling (based on [4])

Figure 2  The details of the experimented walls (based on [3]): (a) the dimensions of the tested wall, (b) the supports and loading conditions

Figure 3  Traction separation response between the contacting surfaces based on cohesive surface based behaviour (extracted from [6])
Figure 4  Compressive stress strain curve for the masonry walls

Figure 5  Comparisons between force-displacement curves obtained from experimental and numerical results for both plain and CFRP strengthened walls

Figure 6  Failure mechanism obtained from the numerical analysis of plain wall: (a) opening of 4 bed joints at the bottom and displacement distribution (mm), (b) distribution of compressive stresses at ultimate load (N/mm²) (Scaling factor = 30)
Figure 7 Numerical results from strengthened wall (4 CFRP strips): (a) first bed joints openings at 8.52 mm displacement, (b) failure mode at full displacement i.e. 35.13 mm, (c) distribution of compressive stresses at ultimate load (N/mm$^2$) and (d) distribution of tensile stresses in CFRP strips at ultimate load (N/mm$^2$) (Scaling factor = 30)

Figure 8 Numerical horizontal force-displacement responses of plain walls with different slenderness ratios
Figure 9 Numerical results from testing plain walls with different slenderness ratios: (a) bed joint openings at slenderness ratio = 12.5, (b) distribution of compressive stresses at ultimate load (N/mm²) (slenderness ratio = 12.5), (c) opening of three bed joints and (d) distribution of compressive stresses and toe crushing in the plain wall at 7.5 slenderness ratio at ultimate load (N/mm²) (Scaling factor = 30)

Figure 10 Numerical horizontal force-displacement responses of strengthened walls with different reinforcement ratios
Figure 11 Failure modes (opening of 4 bed joints from the bottom) of strengthened walls with different reinforcement ratios: (a) 1 CFRP strip, (b) 8 CFRP strips, and (c) CFRP sheet (Scaling factor = 30)

Figure 12 Distribution of tensile stresses in CFRP materials at ultimate load (N/mm²): (a) 1 CFRP strip, (b) 8 CFRP strips, and (c) CFRP sheet (Scaling factor = 30)

Figure 13 Distribution of compressive stresses in the walls strengthened with different reinforcement ratios at ultimate load (N/mm²): (a) 1 CFRP strip, (b) 8 CFRP strips, and (c) CFRP sheet (Scaling factor = 30)
Figure 14  Comparison between numerical horizontal force-displacement responses of the masonry walls strengthened with CFRP and GFRP composites

Figure 15  Numerical results from strengthened wall (4 GFRP strips): (a) failure mode at full displacement i.e. 35.12 mm, (b) distribution of tensile stresses in the GFRP strips at ultimate load (N/mm²) and (c) distribution of compressive stresses in the masonry wall at ultimate load (N/mm²) (Scaling factor = 30)