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EXPERIMENTAL STUDIES ON DOUBLE LEAF MASONRY WALLS

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ABSTRACT

The mechanical behaviour of un-reinforced masonry walls subject to quasi-static and dynamic loading is difficult to assess in practice due to a) the inherent unpredictability and variability associated with the performance of the bricks and the mortar both individually and as a composite material b) the difficulty of correctly identifying the boundary conditions and reproducing these under laboratory conditions. Recently, a more 'basic' retrofitting technique that involves building a second masonry panel, parallel to an existing one (i.e. collar-jointed masonry), in order to enhance the behaviour of the existing wall has been trialled in certain real structures. The new leaf is tied to the old panel by means of the collar joint and/or by steel ties.

To assess this practical approach, tests were performed on three different arrangements, namely 1) collar joint without ties, 2) collar joint with ties and 3) ties without a collar joint; all walls were subject to quasi-static loading. The aim was to evaluate the shear capacity of the bond between the two panels, to assess the suitability of such a technique and to establish whether or not there is a need for ties in conjunction with a collar joint. This paper describes the development of a laboratory testing rig which can be used to accurately assess these criteria. Preliminary results illustrate that the test setup to represent the real life situation has been identified and gives consistent results. Also, the use of steel ties with a completely filled collar joint seems to be unnecessary.

KEYWORDS: collar jointed masonry, ties, shear capacity

INTRODUCTION

Even though the use of reinforced concrete and steel frames for structures greater than 3 floors has limited the use of masonry as a structural material, it is still widely used in conjunction with these frames as a cladding material or as an infill material for the space between the structural framing members. However, its mechanical behaviour when used in these latter situations is not fully understood, due to the unpredictability of bricks and mortar both individually and as a composite material. Both under quasi-static and dynamic loading, masonry illustrates a brittle nature, with little or no ductility, and it is prone to a whole spectrum of damages. As a result, its performance is generally characterised as poor [1-3]. This aspect, in combination with the difficulty of correctly identifying boundary conditions when performing tests in the laboratory, makes the development of an accurate predictive model for its mechanical behaviour extremely challenging.

In order to enhance the performance of masonry panels, several methods have been developed over the years. These vary from the use of jacketing and pre-stressing techniques to the use of advanced polymer materials and they have allowed the development of masonry walls that have a more predictable behaviour. Recently though, a more 'basic' retrofitting technique that involves the building of a second masonry panel, parallel to an existing one (i.e. collar-jointed masonry), in order to enhance the behaviour of the existing wall, has been trialled in certain structures. The new leaf is tied to the old panel by means of the collar joint and/or cross cavity bed reinforcement or by steel ties (Figure 1). In practice, the cavity was generally left empty when cross cavity reinforcement or steel ties were used to tie the two leaves together [4].

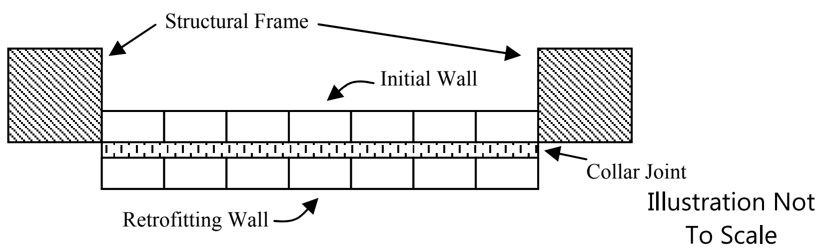


Figure 1: Topology of the new panels in respect to the old walls and the structural frame

From Figure 1 it is apparent that the new panels are not restricted by the structural frame. As a result, the analysis of the composite wall poses a challenge due to the combination of actions. The initial wall is described by an infill panel behaviour whereas the new wall should be treated as an unreinforced masonry panel. The forces imposed on the new panel, and thus its suitability, depend on the bond provided between the two walls.

The aims of the experimental investigation is to determine the influence of the ties on the collar jointed masonry, particularly in terms of the shear capacity of the collar joint, and to develop a predictive computer model for collar jointed masonry. For these aims to be achieved, initially a test rig has to be developed that represents the boundary conditions of the real, full-scale scenario and of the flow of forces between the inner and outer panels. This paper describes the development of a test rig that represents the conditions met in practice. It also presents the results from the tests performed to date.

EXPERIMENTAL PROGRAM

The test program consists of 3 different types of tests, with multiple tests being performed for each testing arrangement. The interest lies with the shear performance of the bond between the two panels. To accommodate the practical restrictions present in the laboratory it was decided that rather than enclose one panel within a frame and load the frame, monitoring the stresses in each of the two walls, one wall (representing the actual infill wall onsite) would be restrained however the second wall (representing the 'strengthening' wall on site) would be loaded. With this in mind, the main testing rig devised is shown in Figure 2. (The lack of any literature on the particular case makes it extremely difficult to verify the validity of the testing rig. Therefore, at this stage it has only been possible to assess the output in terms of how reasonable it appears. The validity will be verified in the future. This will be achieved initially by the development of a model that successfully describes the current case. The restraint conditions of the model will then be altered to match those of the real case and the output will be assessed. Finally, a real case test is currently being planned to validate the revised model.) The specimens consist of double-leaf walls, where one panel is restrained by the portal and the other is allowed to move freely. The force is applied to the unrestrained panel by a horizontal actuator, while the vertical load cell is used to suppress the vertical uplift of the restrained leaf, imitating the constraint imposed by a deforming frame under the typical separation between frame and panel (see Figure 3) [5-9], and also to quantify the uplifting force that is applied to the frame. To avoid localised failure (corner crushing) at the point of loading and also represent the typical 'stress point' that develops in practice (Figure 3), steel plates were used to spread the load over the top three courses.

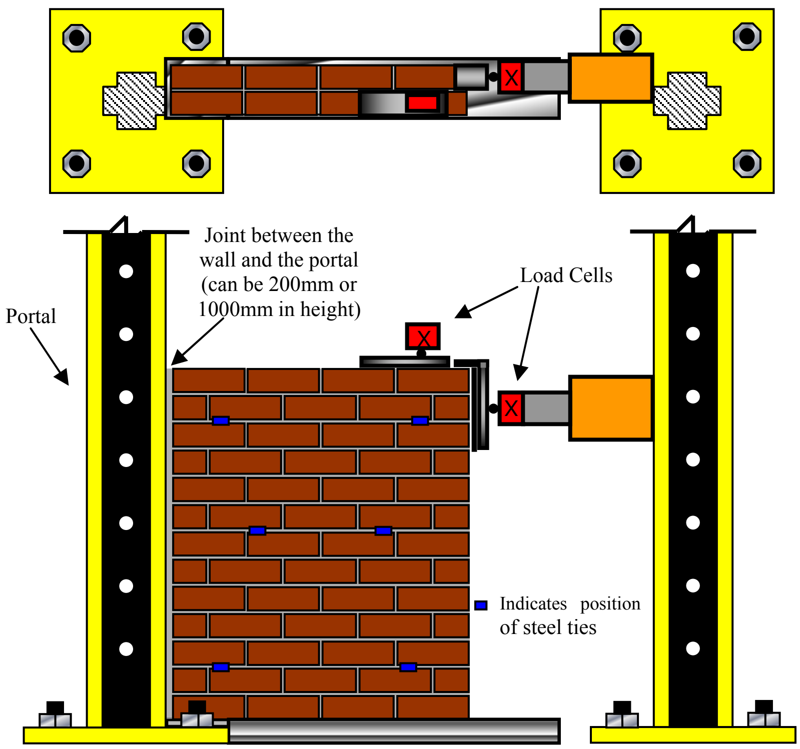


Figure 2: Typical testing rig indicating the positions of the steel ties

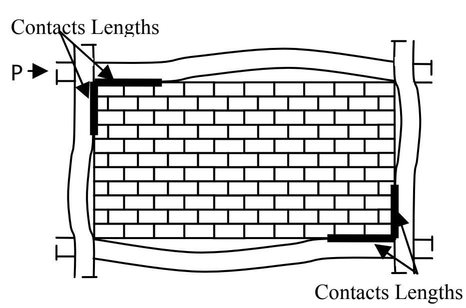


Figure 3: Deformed shape of an infill panel – frame system and typical separation

All walls were subjected to a monotonically increasing horizontal load until failure, while being monitored with linear variable displacement transducers. The walls were 13 courses high, 4 bricks wide double-leaf walls with a 20mm cavity. This gives panels with a length to height (l/h) ratio of approximately 1. The loaded panel consists of Engineering Class B units whereas the restrained panel consists of Fletton units, as was the case in practice. The walls were constructed with an ordinary Portland cement : lime : sand (1:½:4½) mortar and they are allowed to cure for 14 days under polythene sheets before testing. Where steel ties were used, there position is shown in Figure 2.

The different specimen arrangements can be classified into three different categories, namely:

a. Collar jointed panels, without steel ties, referenced with the prefix CNT
b. Collar jointed panels with steel tied, referenced with the prefix CT
c. Panels tied only by steel ties, referenced with the prefix NCT

In an attempt to ensure minimal variability of the physical properties of the materials used, all materials were stored inside with the exception of the bricks which were stored outside, under polythene. All bricks were moved inside and stored under room ambient conditions for 84 hours before use.

EXPERIMENTAL RESULTS TO DATE

A number of preliminary tests have been carried out to allow the optimisation of the testing rig and the setup of the monitoring equipment. All the preliminary specimens were built by an experienced builder that came from industry. The first wall to be tested (CNT-I) was collar jointed without ties. The failure mechanism (Figure 4) showed that the initial testing rig had an important design flaw. Due to the eccentricity of the applied load to the restrained panel (Figure 2), the wall failed under a sliding action that clearly cannot take place in practice. So, the testing rig was modified and a 14mm steel angle was placed in the cavity and welded to the steel base (see Figure 5), prohibiting the walls from exhibiting sliding failure at the base in the future, by restraining any lateral movement of the panel resting against the portal.

Figure 4: Sliding failure of CNT-I under the eccentric applied load

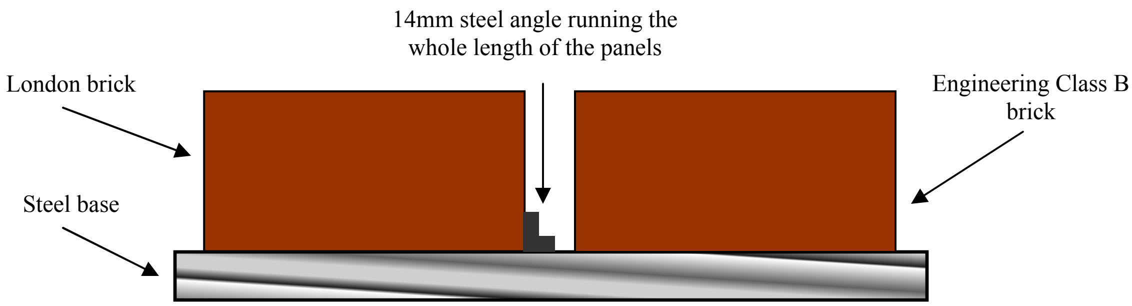


Figure 5: Position of the steel angle that prohibits any lateral movement at the base of the walls

Following this adjustment, three tests were conducted on collar jointed walls without ties (CNT-II, CNT-III, CNT-IV). All three walls failed at a similar ultimate load (34.50 kN for CNT-II, 25.77 kN for CNT-III and 35.56 or CNT-IV) and in a similar way (Figure 6).

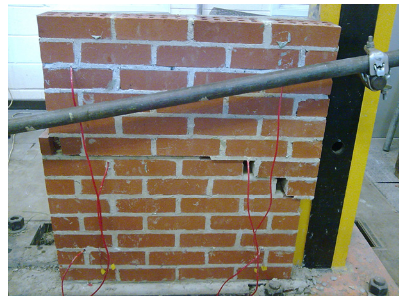
(a) **(b)** **(c)**

Figure 6: Failure mechanisms of walls (a) CNT-II; (b) CNT-III; (c) CNT-IV

Walls CNT-II and CNT-IV failed as a horizontal slip plane developed without separation of the two leaves. In CNT-II, failure occurred around the 7th course from the top whereas in CNT-IV failure took place between the 4th and the 5th course. Specimen CNT-III, apart from failing at a lower load than the other two, also exhibited a slightly different failure mechanism as it exhibited separation of the two leaves and the failure mechanism involved a diagonal crack that started at the 3rd bed joint from the top and finishing at the 7th joint. This is discussed later.

The next specimens that were tested were two collar jointed walls with ties (CT-I and CT-II). The walls failed at 35.19 kN and 37.42 kN respectively following a sliding failure mechanism at the 5th bed joint and at the 6th bed joint, as illustrated in Figure 7. No separation between the two leaves occurred.

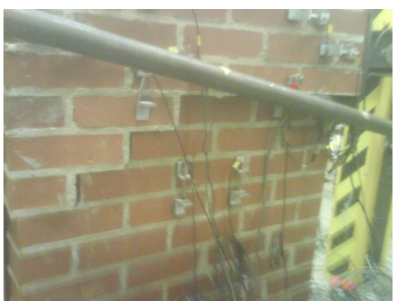
(a)



(b)

Figure 7: Failure of specimens (a) CT-I; (b) CT-II

Finally, two specimens were tested that included only steel ties (NCT-I and NCT-II). In both cases failure involved the development of a diagonal crack (from the 3rd bed joint to the 6th bed joint) following the separation of the two leaves (Figure 8). The failure load of NCT-I was similar to the rest of the results (30.11 kN). However, this was not the case with NCT-II, which failed at a very low load (12.5kN). After close inspection of the test specimen, it was found that the installation of the wall ties had not been performed correctly (they were loose, indicating that they may have been disturbed post installation and before the mortar had set sufficiently). This possibly explains the lower failure load.



(a)

(b)

Figure 8: Failure of specimens (a) NCT-I; (b) NCT-II

The failure mechanisms observed during the preliminary experiments are all similar to the five possible failure mechanisms of an infill panel as reported in [10]. In all cases though, the failure was contained between the first seven courses from the top of the wall; below this level the strains recorded were insignificant (see Figure 9). Based on this, it was decided to monitor only the top part of the walls. The transducers were placed on diametrically opposed positions on the two leaves, so as to investigate the influence of the bond between the two leaves on the stress pattern. The region of failure and the failure mechanisms observed in these tests also meant that verifying the location and development of the commonly proposed corner-to-corner diagonal crack failure mechanism [5] was discounted. Finally, a visual inspection of the 'degree of fullness' of the cavity in the collar joint walls (CNT-II, CNT-III, CNT-IV, CT-I and CT-II) illustrated that the cavity was generally 55-65% full, except wall CNT-III where the cavity was only approximately 40% full. This may explain the lower failure load, as well as the separation between the two leaves observed in this test.

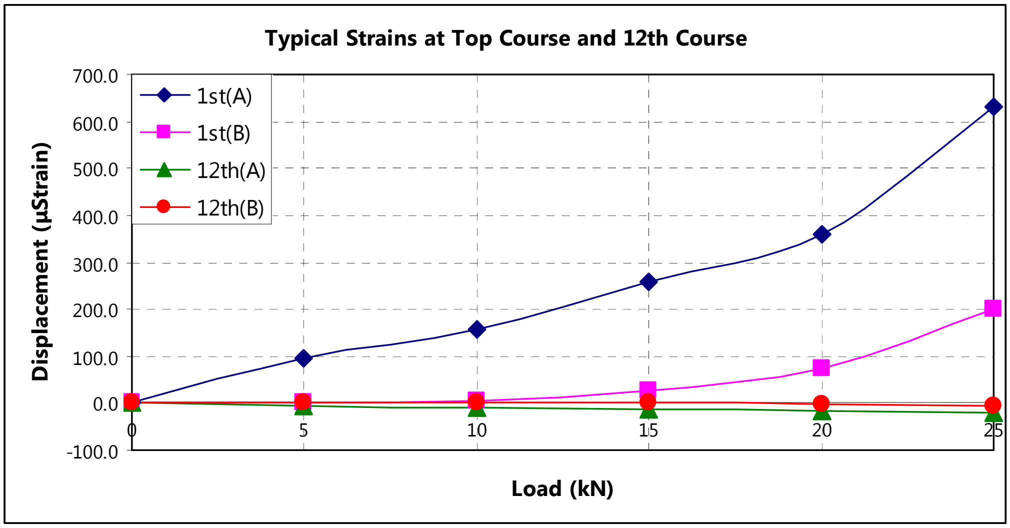
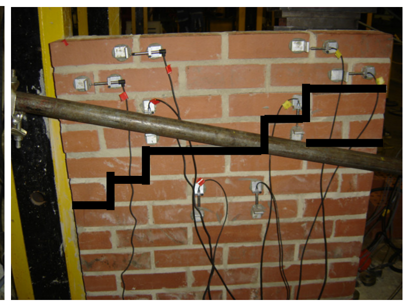


Figure 9: Strain vs Load for CNT-II recorded at the top course (1st course) and at the 12th course; (the letter A indicates a transducer placed on the free, loaded wall and B indicates a transducer placed on the restrained panel)

During these eight preliminary tests, some localised crushing of the top of the vertical mortar joint, at the opposite end of the test specimen to where the load was applied, between the restrained panel and the frame (see Figure 3) was apparent. In practice (see Figure 2) it is possible that this would not occur. It was therefore decided to investigate the influence of this vertical mortar joint and tests were performed with full height mortar joints (as before) and with 200mm high vertical mortar joints which better represent the contact lengths of the typical frame-panel interaction theoretically found at this point (see Figure 2). For these four walls (walls CNT-200a, CNT-200b, CNT-1000a and CNT-1000b, where 200 and 1000 represent the height of the mortar joint), a university employee, who has been trained as a bricklayer, was used instead of the contractor used previously. Figure 10 illustrates the failure mechanism of these 4 walls.

(a) **(b)**

(c)



(d) **(e)**

Figure 10: Failing mechanisms of (a) CNT-200b; (b) CNT-1000a; (c) CNT-1000b; (d) and (e) CNT-200a

All of the four specimens developed a sliding failure mechanism; the plane of failure being at the base (between the bedding mortar and the steel base) on the loaded wall (Engineering bricks) and then extending vertically up the collar joint for one course and then laterally across between the first and the second courses from the bottom of the restrained (Fletton) panel. Failure began just after 20 kN in all specimens, and collapse occurred between 26.5-34.5 kN (CNT-200a at 34.09 kN, CNT-200b at 32.18 kN, CNT-1000a at 26.40 kN and CNT-1000b at 34.37 kN). However, although failure initiated at the base in Wall CNT-200a (Figure 5d), final collapse was due to the separation of the two panels and the development of a diagonal crack on the loaded leaf (Figure 5e). Tests on the mortar confirmed that there was no difference in the properties of all the mortar mixes used for the construction of these four walls. Also, the testing procedure remained the same. However, a visual inspection of the 'fullness' of the collar joint after each test showed that for CNT-200b, CNT-1000a and b walls the collar joint was around 90-95% full. Whereas, a similar inspection of wall CNT-200a showed the collar joint to be only approximately 80% full. From the results of these last 4 tests it appears that if the collar joint is filled to a similar degree, there is, on initial inspection, no influence on the failure load with respect to the height of the vertical mortar joint between the restrained panel and the pylon. However, this conclusion is complicated as it may be affected by the 'fullness' of the collar joint and therefore requires further investigation.

Clearly, the failure mechanisms for these last 4 tests are different to the previous eight tests and by cross-referencing the results of CNT1000 a and b with similar tests in the first set of 8 walls it seems certain that it is the degree to which the collar joints are filled with mortar which is significant. This becomes apparent by the summary of the results of the tests, as presented in Table 1.

Table 1: Summary of the Test Results

Specimen	Cavity Fullness	Ties	Failure Load (kN)	Failure Mechanism
CNT-II	~55%	No	34.50	Horizontal slip plane on the 7th bed joint; no separation
CNT-III	~40%	No	25.77	Diagonal crack from the 3rd to the 7th bed joint (loaded panel) and separation
CNT-IV	~65%	No	35.56	Horizontal slip plane on the 4th bed joint; no separation
CT-I	~60%	Yes	35.19	Horizontal slip plane on the 5th bed joint; no separation
CT-II	~65%	Yes	37.42	Horizontal slip plane on the 6th bed joint; no separation
NCT-I	0%	Yes	30.11	Diagonal crack from the 3rd to the 6th bed joint (loaded panel) and separation
NCT-II	0%	Yes	12.50	Diagonal crack from the 3rd to the 6th bed joint (loaded panel) and separation
CNT-200a	~80%	No	34.09	Horizontal slip plane on the 13th bed joint (loaded panel) and on the 12th bed joint (restrained panel) and separation
CNT-200b	~90%	No	32.18	Diagonal crack from the 2nd to the 6th bed joint (loaded panel) and separation
CNT-1000a	~90%	No	26.40	Horizontal slip plane on the 13th bed joint (loaded panel) and on the 12th bed joint (restrained panel) and separation
CNT-1000b	~95%	No	34.37	Horizontal slip plane on the 13th bed joint (loaded panel) and on the 12th bed joint (restrained panel) and separation

In an attempt to explain the difference in the failure mechanisms between the preliminary 8 tests and the last 4 tests, the following summary is proposed. Walls with a cavity approximately 60% full will fail with a horizontal slip or due to a diagonal crack, without the separation of the two leaves and the failure will take place in the upper half of the wall. When the fullness of the cavity was considerably lower (~40%; CNT-III) the failure will involve the separation of the two leaves, though it was still confined to the upper part of the panel. When the percentage of mortar in the collar joint is in the region of 90-95%, the panels will fail with a horizontal slip located at the very bottom of the wall. Finally, when the collar joint is about 80% full, the horizontal slip at the bottom will start to develop first, but ultimately, failure will be due to a diagonal crack in the upper part of the wall. Hence, there appears to be a threshold value of approximately 80% which dictates whether the test fails due to a diagonal crack towards the top of the wall or a horizontal slip at the bottom of the wall. In practice, it would not be unreasonable to expect a variation in the fullness of the collar joint and for a joint to be 80% full may be considered optimistic. This observation on the influence of the 'fullness' of the collar joint will be very useful when calibrating the computer model.

With regards to the use of ties, it seems that their use with a collar joint is not justified, providing the collar joint is over 80% complete. When comparing the performance of the CNT walls those of the CT walls it appears that all the walls failed at similar loads. The same seems to be true when comparing the CT walls to those of the NCT walls, although the authors are aware of the limited number of experiments that are available on which to base such a comment. The next step is to look at the extent of load sharing across the two panels by examining the strain data recorded on the surface of the walls. This will be presented in the future.

CONCLUSIONS

1. The testing rig developed appears to give consistent results.
2. There seems to be a correlation between the degree of fullness of the collar joint and the type and location of the failure. When the collar joint is 90-95% full, then failure occurs at the bottom of the panels in the form of a horizontal slip. Lower percentages (around 60%) move the location of the failure into the upper half of the wall. Very low degrees of fullness (around 40%) result in failure by separation of the two panels. Walls with 80% (a threshold value) seem to develop a failure mechanism that combines a horizontal slip at the bottom with a failure at the top half. The authors are aware that the panels being investigated are small and that the results may be more influenced by the size effect. This is to be investigated.
3. The results illustrate that when the collar joint is properly filled then there is no need for ties. However, due to the variability in workmanship on site, steel ties may still be needed. This has to be investigated.

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