EXPERIMENTAL INVESTIGATION OF THE CONFINED MASONRY WALL PANELS

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ABSTRACT
This paper presents an experimental study carried out on six masonry panels with different confinement configurations. Half scale hollow concrete blocks (195mm ×95mm ×95 mm gross), with 15.75 mm wide face shells and 5mm thick mortar joints were used to construct 655 mm square unreinforced hollow masonry test specimens that were confined with or without95 mm thick confining elements of either reinforced grout or reinforced concrete. Diagonal shear tests were carried out similar to the provisions in ASTM E519. A number of strains and deformations were measured with a view to examining the effect of confinement to the unreinforced masonry panel, and the behaviour of the confining elements as well as that of the unreinforced masonry panel. All panels were tested under displacement control. The results indicate that the reinforced concrete elements confine the unreinforced panels significantly better than the reinforced grouted elements.

KEYWORDS: unreinforced masonry, confined masonry, diagonal cracking, bed joint sliding cracking

INTRODUCTION

Under cyclonic wind and/ or seismic action, shear walls attract higher proportion of the lateral load due to high stiffness and hence are regarded vulnerable. As shear walls are major structural components of the building, failure of them can lead to catastrophic consequences. Due to lack of ductility, unreinforced masonry shear walls provide insufficient time for the occupants to evacuate. Therefore there is an interest in confining the unreinforced masonry with a view to improving its ductility. Solid brick masonry walls are shown to be effectively confined by the reinforced concrete tension elements by Tena-Colunga et al (2009) amongst others; the system is known as confined masonry (CM).In parallel, hollow block concrete masonry confined with reinforced grouted cores, known as Wide Spaced Reinforced Masonry (WSRM)is emerging an elegant solution to resist cyclonic wind forces. Comparatively the WSRM system is easier to construct than the CM system as WSRM does not require formwork that is expensive. Therefore, there exists an interest in comparing the structural behaviour, especially the shear resisting mechanism, of these two types (WSRM and CM) of construction system of walls. In this research the effectiveness of grout confinement (GC) and concrete confinement (CC) to an unreinforced hollow concrete masonry panel was examined experimentally. This paper presents the test method and results of the experiment.

Masonry resists shear through different type of failure patterns (sliding, flexural and shear), although the failure mode depends on wall aspect ratio, pre compression level and its plane of weakness [1, 2].

Shear stress can be determined from diagonal tests either using the equation provided in ASTM E 519-02 [3] or RILEM [4]. ASTM assumes that the diagonal compression test produces a uniform shear stress across the whole panel whereas RILEM allows for non-uniform distribution of shear. This paper utilises the ASTM E519-02formula for shear stress τ_i given in Eq. (1).

$$\tau_i = \frac{P_i}{(\sqrt{2})A} \tag{1}$$

In which P_i is the load at i^{th} increment, A is calculated as the bed joint area; for hollow concrete masonry, $A = 2Lt_f$ where L is the length of wallette and t_f is the thickness of the face shell.

RESEARCH SIGNIFICANCE AND METHODOLOGY.

Confining element can effectively increase the load carrying capacity and improve the post-peak rate of load degradation. In particular, the contribution of confining elements such as the grouted core - GC [5-7]and concrete confinement – CC [8, 9] were studied individually by many researchers in different part of the world. However, there were no studies to date that compared the effectiveness of GC and CC confinement to the URM. These reinforced elements confine the URM panel leading to enhancement of its load capacity. To compare the effectiveness of confinement, response of walls under direct diagonal load were examined. Ultimate load, cracking stress, failure pattern and shear stress were used as measures of confinement in this paper. Assuming uniform shear distribution, the shear strain (γ) of the whole of the URM panel was calculated as in Eq. 2.

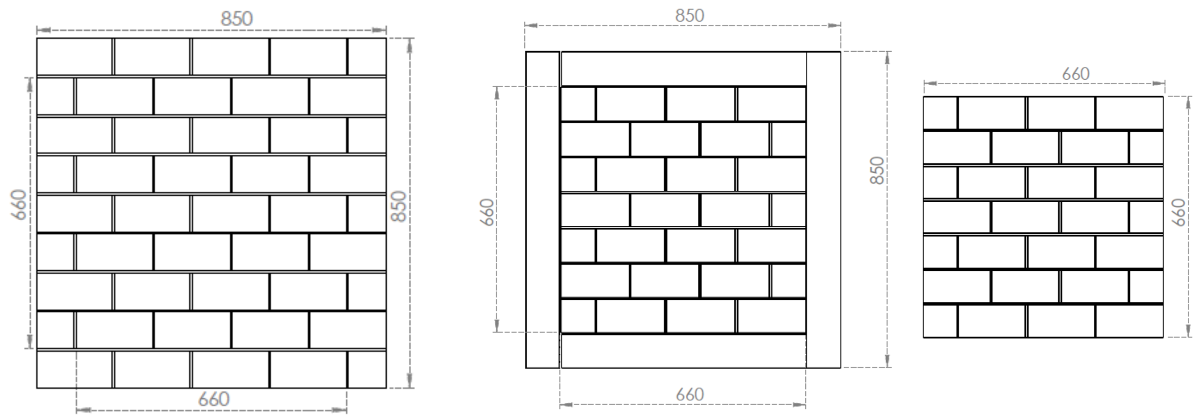
$$\gamma = \frac{(\delta y + \delta x)}{\sqrt{L^2 + H^2}} \tag{2}$$

In which $\delta y \,\&\, \delta x$ are vertical and horizontal incremental displacements respectively and $L \,\&\, H$ are length and height of the wallettes, respectively.

Design of test specimens.

The main objective in this paper is to determine the effectiveness of different confining elements in terms of strength, cracking shear and ductility. To examine these factors, three configurations (no confinement - NC, grout confinement - GC and concrete confinement - CC) were investigated; two walls (A & B) per each configuration were constructed and tested. Fig. 1 shows the configurations of the walls tested.

Confining elements in GC series were core filled grade-30 grout containing 1N12 steel bar; the joints were not designed for moment resistance. Confining elements in CC series were surrounded by reinforced concrete containing 1N12 steel bar placed at centre of each side; the joints were not designed for moment resistance.CC walls were constructed using form work for the pouring of the reinforced concrete confining elements.



(a) GC Wallettes (b) CC Wallettes (c) NC Wallettes

Fig-1: Configurations of the Test Wallettes

EXPERIMENTAL INVESTIGATION.

Specimen constructions

This experimental investigation contains two broad category, material constituent tests and wall panel tests. All these experimental programs were conducted at 1:2 scales using half scale blocks and scale down aggregate size in mortar and concrete confining elements. Particle sizes of sand and aggregate (used in concrete, grout and mortar) were one size finer than those proposed in ASTM C136-06[10].

The hollow blocks used to build prisms and walls contained 53% void, with gross dimensions of 185 mm long, 90.5 mm wide and90 mm high. All masonry works were face shell bedded on the face shell width of 15.75 mm.

The thickness of mortar was 5mm (scaled down by half from 10mm standard size). General purpose cement was used to prepare mortar containing cement: sand ratio of 1:5 (volume basis) which is known as M3 mortar as specified in AS 3700 [11].

All constructions were carried out by an average skilled mason and tested on 14 days from the date of construction. The day after construction, sufficient water was gently sprayed and wrapped with plastic to prevent moisture being escaped for next seven days, later they were opened for air cure inside the laboratory.

Constituent material test specimens

Seventy one specimens were tested to indentify the constituent material properties. All tests had been carried out according to the relevant Australian or the ASTM standards. Standard's guidelines were followed for rate of load application; where absent, a rate of 1 mm/min was adopted. 3mm thick timber sheets were placed at bottom and top of the specimen to refrain any influence of point load and platen confinement due to loading.

Total of fourteen specimen tests were carried out to identify the compressive strength of units (f_{uc}) and modulus of rupture of the units (f_{ut}). Modulus of rupture of the URM was determined by testing eight specimens containing three high stack bonded prisms. These three blocks were bonded using high strength commercially available epoxy and then loaded in four point bending.

Twenty four cylinders of 200 mm high by 100mm diameter were tested to identify compressive strength of concrete grout (f_c).

Eighteen prisms were built including some of them were grouted to identify its compressive strength of URM prism (f_{mc}) and compressive strength of grouted prism (f_{gc}).The masonry compression test was conducted on four high stack bonded prism using M3 mortar. The compressive strength(f_{mc} and f_{gc})was calculated using Eq-3 proposed in AS3700[11].

$$f_{mc} = k_a \left(\frac{F_{sp}}{A_d} \right) \quad (3)$$

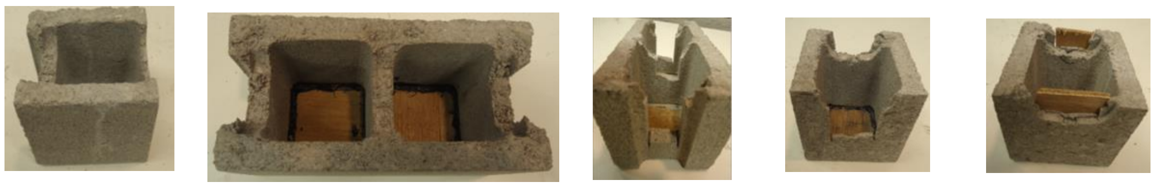
Where F_{sp} is total load at which specimen fails, A_d is design cross sectional area calculated as the multiple of the two face shell lengths and their average thickness and k_a is the aspect ratio factor for the specimen which is 1 for the specimens used.

Wall panels

The URM panels were of 7 courses high containing 3.5 blocks per course. There was no standard half size blocks available so existing blocks were cut at the centre of the block using a rock cutter (Fig-2a). From each full block only one half blocks was prepared and other half was thrashed as its dimension became 8mm lesser due to blade thickness of rock cutter.

For GC series 'U' blocks were required at bottom and top course to retain grout and hence edge half size blocks were required. U blocks were prepared by sealing the bottom cavity of the block using 3 mm thick plywood and bonded by commercially available 'roof-gutter' as shown in Fig-2b. To accommodate reinforcement at the centre of the core full and half 'U'-blocks webs were removed, shown in Fig-2c &2d respectively.

It can be noticed from Fig-2 that virgin blocks' webs are not fully flushed with the face shell through which grout could escape. To prevent this, necessary precautions were made to facilitate enough compaction of the grout. 3 mm thick plywood sections were stuck to fill the space as shown in Fig-2e using commercially available 'timber concrete' liquid nails. For all the vertical cores web gap was blocked before grouting.



(a)½ block (b)'U' block (c) 'U' block with web cut (d)½ 'U' block (e)½ edge sealed 'U' block

Fig-2: Block types

(a)GC Series Wallettes (b) CC Series Wallettes (c) Curing

Fig-3: Construction and curing

The usage of mixture of 'U' full blocks and half blocks are shown in Fig-3a. For CC series, URM panels were first prepared with U blocks at top and bottom courses and web gap filled blocks at edge as shown in Fig-3b. For GC wallettes, initially 660 × 660 mm of URM panels were constructed then next day it was placed at centre of the prepared form work as shown in Fig-3b and rebar was placed at centre of the core then concrete was poured.

GC series were size of 850 × 850 mm containing 4.5 blocks per each core and 9 courses high. These walls' edge cores were filled using grout together with 1N12 rebar.
NC series contained only URM panels of 660mm × 660 mm which were constructed on floor platform vertically. All constructed wall panels then wrapped by plastic for 7days then removed and allowed for air curing until tests as shown in Fig-3c.

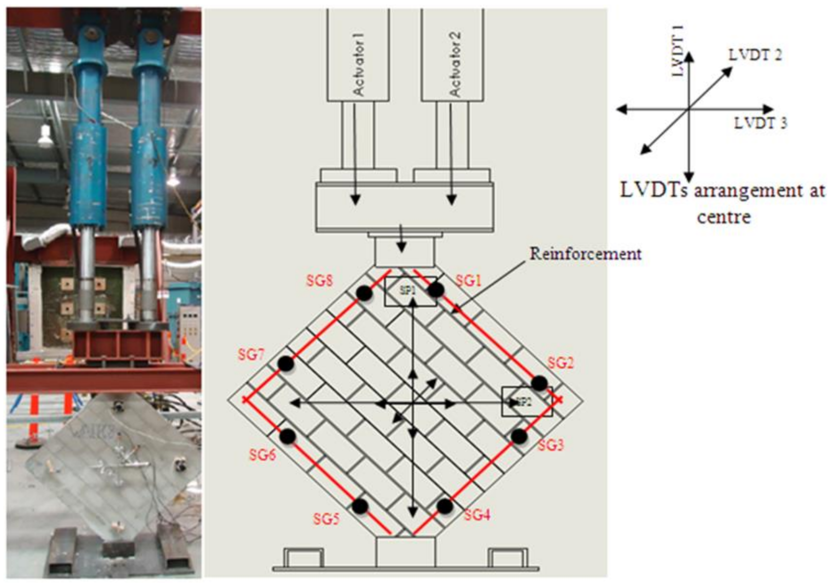
Instrumentations

Different types of instrumentation were adopted for constituent material tests and diagonal tests. Only potentiometers were used for the constituent material tests in addition to the load recorded by actuators. Single actuator capacity of 235kN was used for material tests and dual synchronised identical same actuator with total capacity of 470kN was used for diagonal compression test for wall panels.

For the compression tests on masonry prisms, two LVDTs were located as shown in Fig-4a. Both LVDTs shows little variance, finally the LVDT mounted on the specimen was considered. For the shear triplet tests two LVDTs were mounted on the specimens as shown in Fig-4b and finally average value was considered. For all other single block tests and modulus of rupture tests one single LVDT was set at top of the loading plate as shown in Fig-4a. For compression test on cylinders, only load was measured.

(a) Compression test

(b) Shear triplet test



(c) Diagonal test

Fig-4: Instrumentation

The instrumentation of the masonry wall panels are shown in Fig-4c which consist four string pots, nine LVDTs, sixteen strain gauges (SG) and load cells. Six LVDTs were located at the centre of both specimens to measure the displacement and remaining three were set to measure any possible/ accidental out-of-plane deviation and top plate displacement. Each string pots were attached along the diagonal direction of specimens' each faces to measure diagonal displacement to calculate shear strain. All sixteen strain gauges were attached the purpose to identify strain at steel reinforcing bars to get a sense of the formation of cracks.

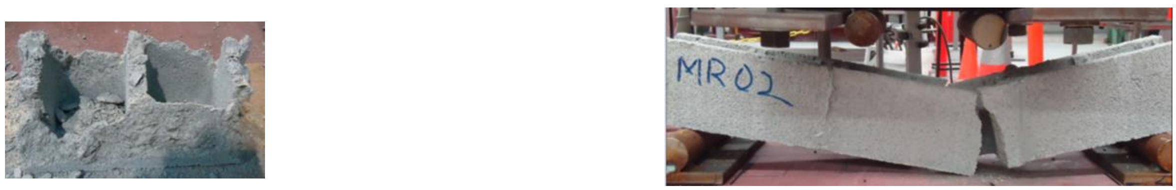
Test method

Displacement control load was applied using the dual synchronised hydraulic actuators of total capacity 470 KN in diagonal direction of the specimen. For wall panels, tests were carried out under displacement control mode at a rate of 1.5 mm/min and tests were terminated upon either fall of load more than 70% of its peak or 2% drift. For the material tests the load application rate was strictly followed according the standards, where absent the tests were carried out under displacement control mode at a rate of 1 mm/min. During the tests the load response of each actuators were monitored. Most of the times it was equal, but sometimes difference up to maximum ratio of 0.45:0.55 was noticed. Where such discrepancies occurred, the moment induced by actuators at diagonal loaded edge was divided by height of the specimen to obtain the equivalent in-plane load. This equivalent in-plane load was arithmetically added or subtracted based on its direction with equation-1 to workout in-plane shear.

Failure mode

Failure modes and strength of constituent material specimens

Typical failure modes of specimens for variety of tests are shown in Fig-5.



(a) Compression failure of blocks

(b) Modulus of rupture

Fig-5: Failure mode of blocks

Under compression, blocks failed at face and web shells. The failures occurred in the modulus of rupture tests were through blocks as shown in Fig-5b. All strength value attributed to the constituent models is reported in Table-1 along with minimum and maximum values recorded. Table-1 reports f_{uc} – unit compressive strength, f_{ut}- unit modulus of rupture, f_c- cylinder compressive strength of grout, f_{mc}- compressive strength of hollow concrete masonry and f_{gc}- compressive strength of grouted concrete masonry. Fig-6 shows the failure patterns of masonry prism for compression tests and shear tests. For compression test on URM prisms, it was visible that the failure occurred at mortar block interface with no or minor cracks appearance in the blocks. However for grouted prisms, the blocks failed where grout appeared without any cracks. The mean strength of URM prisms was 9.2 MPa whereas grouted prisms show 8.8 MPa. It indicates that grouting did not contribute to enhance compression capacity of the prisms.

Table 1: Material properties

Test specimens	**Test**	**No. of samples tested**	**Strength parameters (MPa)**			**COV (%)**
			Mean	**Min**	**Max**	
Blocks	$\mathbf{f_{uc}}$	**6**	**18.43**	**16.47**	**19.91**	**6.66**
	$\mathbf{f_{ut}}$	**8**	**2.75**	**2.33**	**3.26**	**10.9**
Grout	$\mathbf{f_c}$	**24**	**30.12**	**26.24**	**32.48**	**6.23**
Hollow Masonry	$\mathbf{f_{mc}}$	**12**	**9.16**	**6.93**	**10.98**	**16**
Grouted Masonry	$\mathbf{f_{gc}}$	**6**	**8.81**	**6.61**	**10.41**	**17.2**

(a) Compression test on URM masonry

(b) Compression test on Grouted masonry

Fig-6: Failure cracks pattern of blocks

Failure modes and strength of masonry panels

The crack initiation can be studied from the signatures of the strain gauges attached to the steel reinforcing bars. The average strain gauge reading against the applied diagonal displacement is shown in Fig-7 for the CC series. Similar graph for GC Wallettes A and B were also derived but not reported here. From Fig-7, it can be noticed that the steel rebars were at compression (negative strain) at the initial loading then suddenly all strain gauges reported positive strains where cracks initiated. This crack initiation was manifested with high resolution camera which took photos at an interval of 5 seconds throughout the test. The initial cracks appeared at 2.9 mm of CC series wall diagonal displacement. At the start, step-crack was initiated at the centre of the wall then propagates towards the loaded edge along with sliding type failure and cracks propagated through blocks finally reinforced grout had been broken. For GC walls, the initial crack formed at 2 mm of diagonal displacement with similar type of crack propagation as CC series.

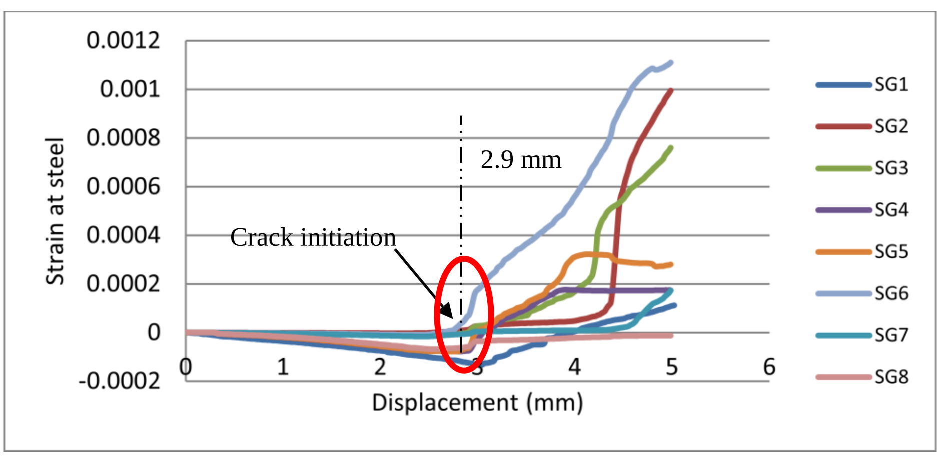
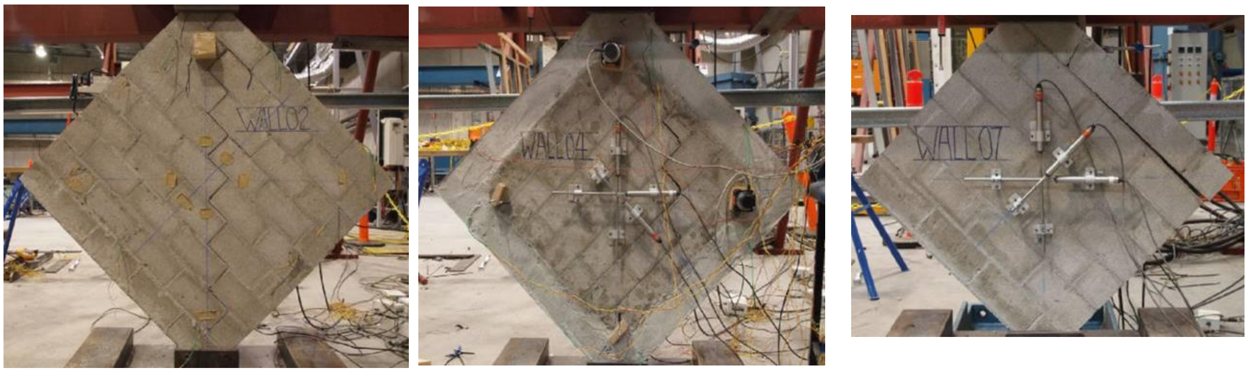


Fig-7: Strain gauge reading vs diagonal displacement (CC Wallettes A and B)

However URM walls had shown different failure pattern, the top course slide along the bed joint. This failure may be resisted in the building due to pre compression load arise from above floors. All above failure patterns are shown in Fig-8.



(a) GC Series (b) CC Series (c) NC Series (URM)

Fig 8: Crack patterns of diagonally compressed wallettes

RESPONSE OF THE SPECIMENS AND DISCUSSIONS

Load Vs displacement response.

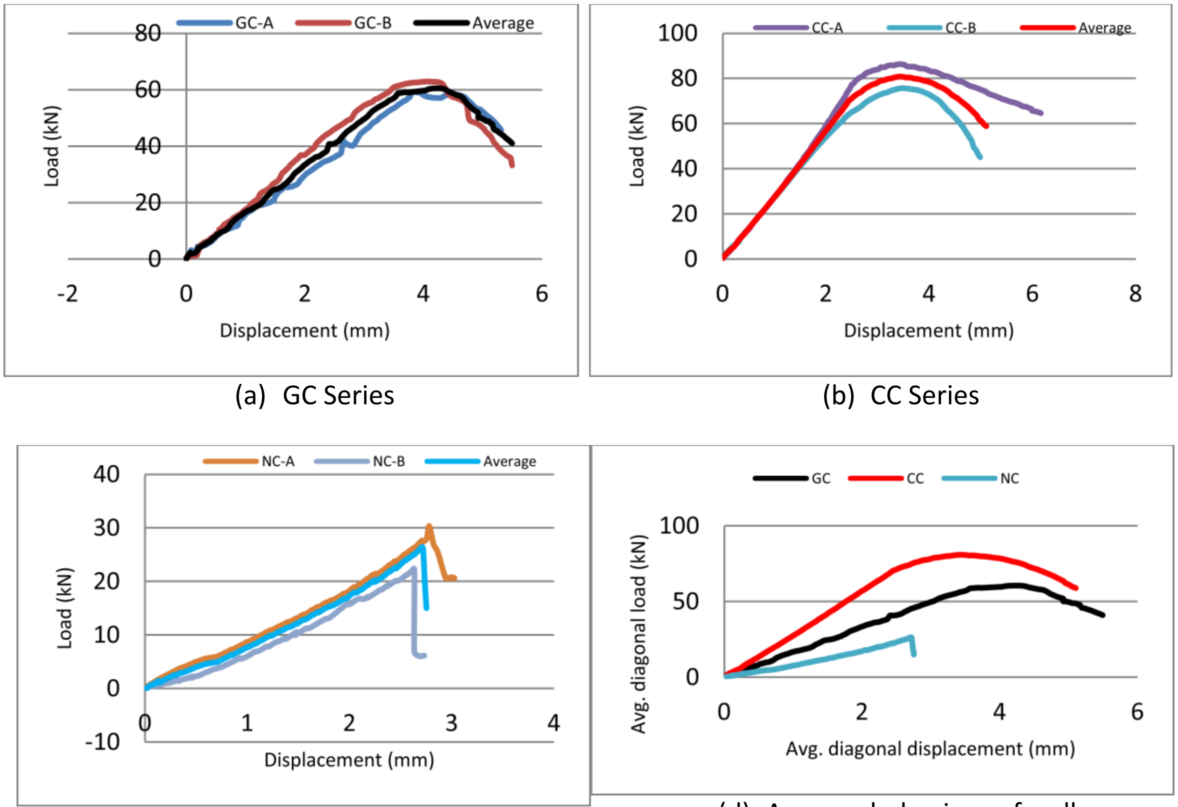
Load vs diagonal displacement responses of all six walls are shown in Fig-9. It can be noticed that the diagonal load capacity of frame structure is significantly higher than core filled wall. All six walls have shown linear elastic behaviour up to 2.5 mm of displacement, and then suddenly URM walls had lost its strength and CC series walls had started yield. However GC series walls show linear behaviour until 3.5mm then it has started to yield. This load and displacements had been converted in to shear stress and shear strain and more analysis had been done below.

To workout cracking shear load, the diagonal displacement at which the cracks were initiated was identified from recorded video and photos and analysis on attached strain gauges on rebar as shown in Fig-7. The diagonal load corresponding for that diagonal displacement can be found from Fig-9.

Table 2 contains the initial cracking load and the ultimate load obtained from these tests, where P_{cr}- cracking load and P_{ult}- ultimate diagonal load capacity of the wall.

Table 2 Test results

Series	Wall No	P_{cr}(kN)	P_{ult}(kN)
1	GC-A	30	59
	GC-B	37	63
2	CC-A	84	86
	CC-B	72.5	75.5
3	NC-A	26.5	30
	NC-B	18	22



(a) GC Series

(b) CC Series

(c) NC Series

(d) Average behaviour of walls

Fig 9: Diagonal load Vs diagonal displacement

From the average of the two wallettes in each of the series, it can be seen that the GC elements enhance the diagonal load of the URM by 131% whilst the CC elements increase the same by 211%. It may therefore be inferred that the CC elements are 0.76 times more effective than the GC elements. However construction of GC is much easier and eliminates the need for formwork. Either confining system can therefore be economical depending on the shear demand and construction methods available.

Shear stress-strain response

Shear vs strain response of all six wall shows linear behaviour until it reaches ultimate load then shows different behaviour in its post peak region. During post peak region wall NC series walls show sudden degradation due to sliding of top course along mortar joint. However reinforced grout walls have shown more gentle degradation in the post peak region.

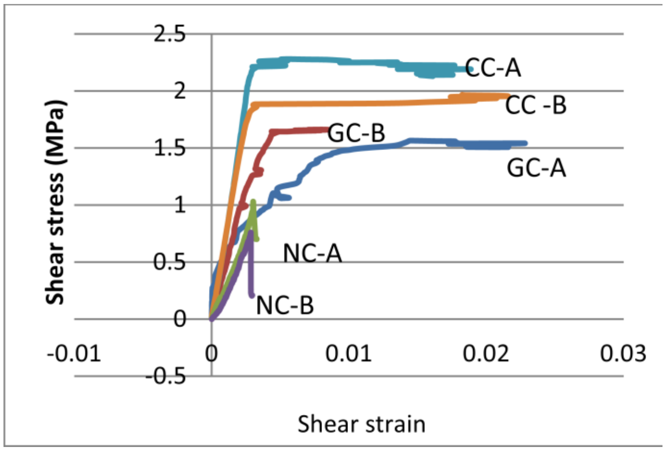


Fig 10: Stress strain response

It can be seen that the average shear strength of the URM (NC series) panels is significantly enhanced by the confining elements.

CONCLUSIONS

The in-plane shear behaviour of URM walls strengthened by the reinforced grout by means of core filled and concrete confined systems was examined. The study evaluated the effect of each method of construction by means ultimate shear strength enhancement and cracking shear strength. From the results presented, the following conclusions were made:

1. The average shear strength of concrete confined wallettes was significantly enhanced compared with grouted confinement. It seems concrete confinement is 0.76 times more effective than the grouted confinement.
2. The initial cracking of the concrete confined wallettes occurred at 2.9 mm diagonal load whilst the same for the grout confined wallettes occurred at 2.0mm diagonal load. The concrete confinement therefore appears more effective than the grout confinement. However, to form the concrete confinement, formwork will be required, which would add to the cost.
3. There was evidence that cracks initiated at centre of the panel then propagated towards the edge.

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