

STRUCTURAL ENHANCEMENT OF LOW STRENGTH MASONRY USING RETRO-REINFORCEMENT

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Abstract

One of the most effective ways of achieving sustainable construction is to extend the life of an existing structure by repair or strengthening. This needs to be carried out at minimal cost and with minimal disruption to the owners or end users without creating a future maintenance liability. In the case of masonry buildings this can be achieved with the use of retro-reinforcement.

Retro-reinforcement involves the installation of small diameter reinforcing bars into pre-cut grooves or pre-drilled holes in the near-surface zones of the masonry that are likely to be subject to tensile stress. The reinforcement usually consists of stainless steel bars to minimise the risk of corrosion. The principal objectives of adding such reinforcement are to improve flexural crack control, increase flexural and shear strength and to increase robustness and ductility. In the UK the technique has been used extensively to strengthen the masonry cladding of low to medium rise buildings, particularly where cracking has occurred adjacent to a long-span window or similar opening.

This paper summarises recent experimental research into the behaviour of clay brick wall panels containing 2m and 3m span openings. Single leaf walls with different arrangements of reinforcement constructed using very low strength (1:12 cement:sand) mortar were tested under short-term in-plane vertical loading. Similar plain and retro-reinforced wall panels constructed from natural hydraulic lime (NHL2) mortar were also tested. Some of the panels were constructed to simulate damage (cracking and excessive deflection) that occurs in practice. The retro-reinforced walls showed increases in strength of between 59% and 206% when compared with the unreinforced experimental controls. In addition, the load at which first visible cracking occurred and the reserve of strength beyond first cracking were enhanced.

Keywords: Low strength masonry, Reinforcement, Walls

1. Introduction

1.1 Low strength masonry

Stone and brick masonry have been used extensively as construction materials throughout the ancient and modern World (Fitchen 1961, Heyman 1995, Campbell and Price 2003). As a result there are many masonry buildings and other structures such as bridges, towers and viaducts in existence today. Many of these structures were built using some form of lime mortar. Following the development of Portland Cement in the nineteenth century, the use of cementitious mortars for masonry construction increased, particularly for engineering structures. By the latter half of the twentieth century, cementitious mortars were used for most forms of masonry construction apart from specialist renovation or rehabilitation work. In the UK (and elsewhere) some construction professionals have started to specify Natural Hydraulic Lime (NHL) mortar for new build because of the assumed environmental benefits (Key 2009).

In the context of this paper, low strength masonry is taken to be that where the bond between the masonry units and the mortar joints is sufficiently low to have a dominant effect on the mechanical behaviour such as the formation of cracks, the re-distribution of stresses after cracking and the formation of collapse mechanisms. Low bond conditions can exist in masonry constructed of lime or cementitious mortar. In the latter case, low bond conditions can arise when low quality cement has been used or where a lack of quality control on site has resulted in mortars being used with a lower than specified cement content. Irrespective of the type of mortar, a reduction or loss of bond can also occur where there are excessive thermal, moisture or ground movements or where water leaching and other weathering effects have occurred.

Low strength coupled with an inherent brittleness and lack of ductility means that it is sometimes necessary to strengthen masonry structures to improve their robustness, resilience and resistance to static and dynamic load effects in order to meet modern performance requirements. In recent years, the greater focus on achieving more sustainable forms of construction has led an increasing number of construction professionals to seek ways of extending the life of many existing masonry structures. Often this involves some form of strengthening.

1.2 An overview of masonry strengthening

The structural performance of existing masonry construction can be enhanced either by grouting, pre-stressing or by the use of reinforcement. Reinforcement is usually used to increase the ductility and robustness of masonry construction as well as increasing the load-carrying capacity. The commonly used methods of reinforcing masonry can be categorised as:

- a). *Through-thickness (or partial through-thickness) reinforcement* such as dowels, ties or stitching bars. In such cases, the individual reinforcing bars are installed into pre-drilled holes in the masonry and the space between the bar and the existing masonry substrate is filled with a low-shrink, high bond grout to facilitate composite action (i.e. to create an under-reinforced masonry structure).
- b). *Near-surface reinforcement also known as “retro-reinforcement”*. In this case small diameter reinforcing bars are inserted in pre-cut grooves or pre-drilled holes in the outer (exposed) zones of the masonry and then encapsulated in a similar grout to that referred to in a), above, to create an under-reinforced masonry structure.
- c). *Surface (or external) reinforcement*. As with the two aforementioned forms of strengthening, the principal aim with surface reinforcement is also to create a reinforced masonry structure. In this case reinforcing strips are attached to the exposed, pre-prepared surface of the masonry with some form of high bond strength adhesive such as an epoxy resin. Although surface mounted steel plates or strips have been used to strengthen masonry for well over 100 years, there is a great deal of international research effort to investigate the use of carbon or glass fibre reinforced polymer strip reinforcement for masonry. An alternative form of surface reinforcement consists of a mesh which is embedded in a layer of mortar or concrete which is sprayed onto the surface of the masonry.

Of the above methods, “retro-reinforcement” using austenitic stainless steel reinforcing bars is the method preferred by the author because of the greater reliability of the shear connection between the reinforcement and the existing masonry substrate. This is an essential requirement for composite action and, thus, for the reinforcement to be effective. In the author’s experience of masonry building and bridge strengthening, the condition of the substrate of many existing masonry structures can be highly variable. Typically there can be large variations in the surface moisture content (in particular, zones of dampness) and the exposed surfaces of the masonry can be very friable and unsound because of exfoliation effects, frost damage, salt crystallisation damage and the presence of surface deposits or growths. In addition it is inevitable with masonry that the surface profile will be very irregular rather than being smooth and even. This can lead to the formation of stress concentrations at the changes in surface profile. All of these effects increase the risk of premature de-bonding failure of fibre reinforced polymer (FRP) composite strips or other similar attachments. If variations in the quality of workmanship are also taken into account together with the low fire resistance of FRP composites, the case for retro-reinforcement using steel bars would seem to be increased. In addition, the use of very brittle, stiff materials such as carbon or glass fibre reinforced polymer composites to strengthen unreinforced masonry, another brittle and stiff material, seems to be fundamentally wrong unless the stresses occurring in the new and existing materials under near-collapse conditions can be maintained well below their ultimate strength levels and that premature de-bonding failures can be avoided. As a result of these concerns, the focus of the author’s research and

engineering practice has, in recent years, been on the performance of masonry structures strengthened with retro-fitted stainless steel reinforcement.

1.3 Retro-reinforcement

The principle of using reinforcement in new masonry construction is by no means new (Hamilton 1939, Tobriner 1985, Hamman and Burrige 1939, Edgell 1985). It is equally certain that the concept of adding some form of reinforcement to an existing structure to enhance its performance is also not new. Retro-reinforcement was developed to be a minimum intervention, minimum disruption way of installing small diameter reinforcing bars into existing masonry construction. Typically, it involves the installation of small diameter reinforcing bars into pre-cut grooves or pre-drilled holes in the near-surface zones of the masonry that are likely to be subject to tensile stress. The reinforcement usually consists of stainless steel bars to improve ductility and to minimise the risk of corrosion. The principal objectives of adding such reinforcement are to improve flexural crack control, increase flexural and shear strength and to increase robustness and ductility.

In the UK the technique has been used extensively to strengthen the masonry cladding of low to medium rise buildings, particularly where cracking has occurred adjacent to a long-span window or similar opening. It has been used fairly extensively where there is regularly coursed stone or brick masonry because it is relatively cheap and quick on site to install reinforcing bars into pre-cut grooves in the bed joints (Garrity 1995, de Vekey 2004). Retro-reinforcement has also been used for masonry arch bridges (Derby et al 2008, Garrity 2010) and other masonry structures.

This paper summarises a series of tests carried out on retro-reinforced clay brick masonry wall/beam panels. The effect of existing damage effects, different mortar types and different amounts of reinforcement was investigated.

2. Experimental testing of clay brick wall/beam panels

2.1 Introduction

In the UK, and elsewhere, cavity wall construction has been used for low to medium rise buildings, in particular residential properties, for many years. The outer leaf of the cavity wall, typically of stone, brick or block masonry, serves as external cladding and so does not usually support any vertical load other than its own weight. Where there are window and door openings the self weight of the masonry above the opening is usually supported by some form of lintel or similar spreader beam. In many cases the lintels have deteriorated or the original design or construction was inadequate. In some cases poor construction practice has led to the use of low strength mortars or the omission of any lintels. In this latter situation the window frame

supports the masonry. When window frames deteriorate and/or new low stiffness replacement frames are installed, the resulting excessive deflection of the masonry above the opening results in cracking. Even when lintels have been provided, deterioration or rotation of the lintels can also cause cracking. This is a very common problem with many domestic properties in the UK. One solution to this problem has been to install stainless steel bed joint reinforcement in the existing damaged masonry, the aim being to create a reinforced masonry beam to support the cavity wall construction across the opening without transferring any vertical load onto the window frames. Retro-reinforcement has also been used to enhance the structural performance of masonry walls that have been damaged by settlement or subsidence effects, moisture movements or other sources of tensile stress or strain.

The performance of masonry beams newly constructed in the laboratory with bed joint reinforcement has been well-researched. However, little research has been carried out on low strength masonry construction, particularly pre-damaged masonry, which is more representative of the real situation facing building owners and managers. The research summarised in this paper seeks to address this shortcoming.

2.2 The test panels

To provide confidence in the use of retro-reinforcement for low strength masonry a programme of testing was developed to investigate the effectiveness of simple retro-reinforcement strengthening measures on the performance of half brick thick wall panels containing openings. The programme of testing involved seven replicate pairs of wall/beam panels giving a total of fourteen test specimens. All the panels were constructed of the same high porosity 215mm x 102.5mm x 65mm fired clay bricks with a sand faced finish to avoid high brick/mortar bond strengths and to avoid excessive bond between the bricks and any repair grout. The bricks used in the tests, which were hydraulically pressed, had an average water absorption of 14%, a density of approximately 1885kg/m³ and a compressive strength of the order of 35N/mm². The following variables were investigated:

The mortar type. Most of the panels were constructed using a 1 : 12 Portland Cement : sand mortar. This has a much lower cement content than that used in conventional cementitious mortars that comply with modern standards and practice. Four of the panels were constructed using mortar consisting of 1 : 2.5 NHL2 : sand. Mortar with a Natural Hydraulic Lime (NHL)2 binder was selected for the tests because of its low degree of hydraulicity, the intention being to represent a low strength mortar that might be used in heritage construction. Such low strength mortars were selected to test the effectiveness of retro-reinforcement under the most pessimistic conditions that might be encountered in practice.

Size of opening. Most of the panels were constructed with a clear opening of 2.025m. Four of the fourteen test panels were built with an opening of 2.925m. Details of a typical wall/beam panel are shown in Figure 1.

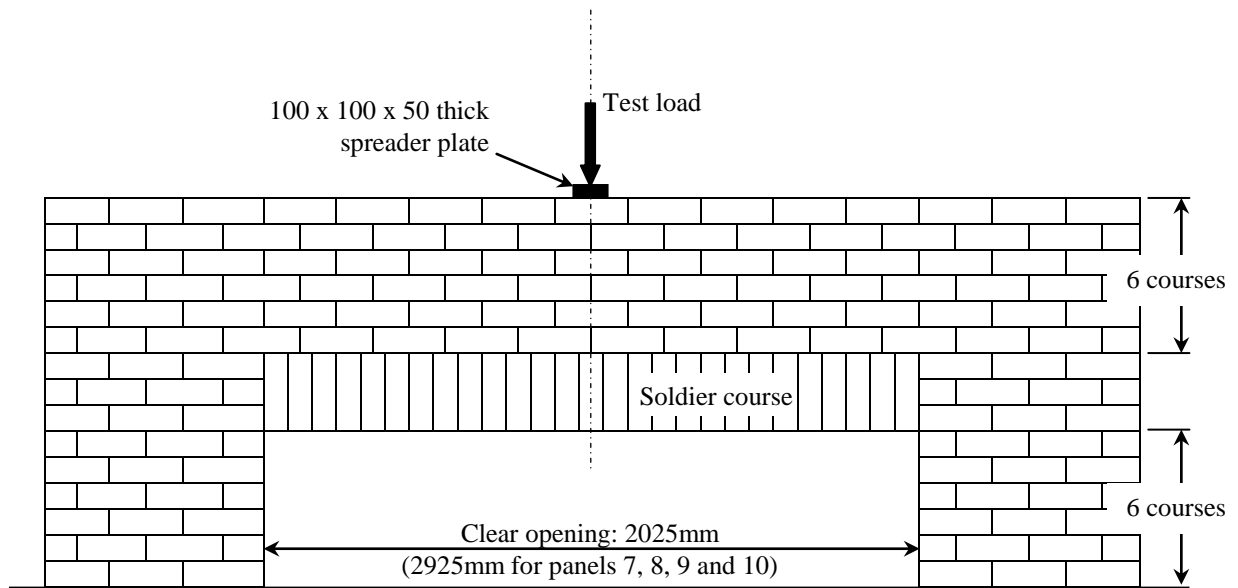


Figure 1. Typical wall/beam panel details showing the test load position

Distribution of bed joint reinforcement. Apart from the unreinforced experimental controls (wall panels 1, 2, 7, 8, 11 and 12 in Table 1), all the panels were reinforced with bed joint reinforcement consisting of a pair of 5mm diameter grade 500 stainless steel reinforcing bars installed in the lowest bed joint immediately above the soldier course. Every third vertical joint of the soldier course was connected to the bed joint reinforcement above using a 3mm diameter stainless steel L-shaped hanger bar. Two of the test panels (numbers 5 and 6) were provided with additional pairs of 600mm long 5mm diameter reinforcing bars in the fourth bed joint above the opening soffit. The reinforcement layout is shown in Figure 2. All the bed joint reinforcing bars were fitted with twisted wire spacers to prevent them from sitting on the surface of the bricks below.

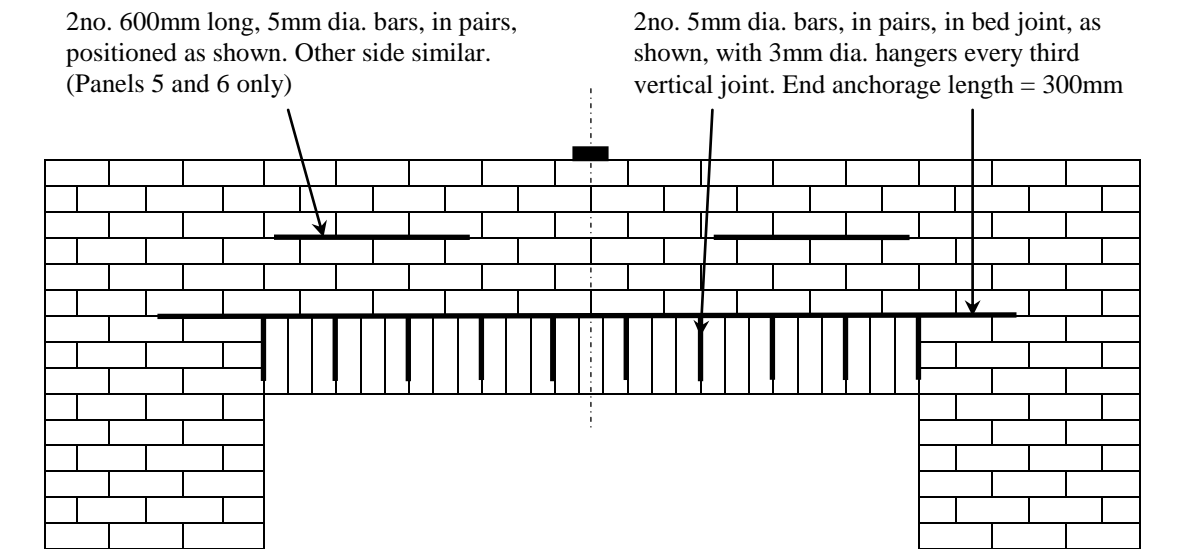


Figure 2. Panel reinforcement layout

Pre-damage. In practice it is fairly common to have to install reinforcement into walls that are already both cracked and deformed. To replicate this situation six of the test panels were constructed with a curved soffit with a 30mm deflection at mid-span. In addition, to simulate the propagation of diagonal cracks from the corners of the opening, the mortar was raked out to a depth of 60mm whilst still relatively fresh and replaced by the same cementitious repair grout that was used with the retro-fitted reinforcement. The curved soffit profile and the “repaired” cracks are shown in Figure 3.

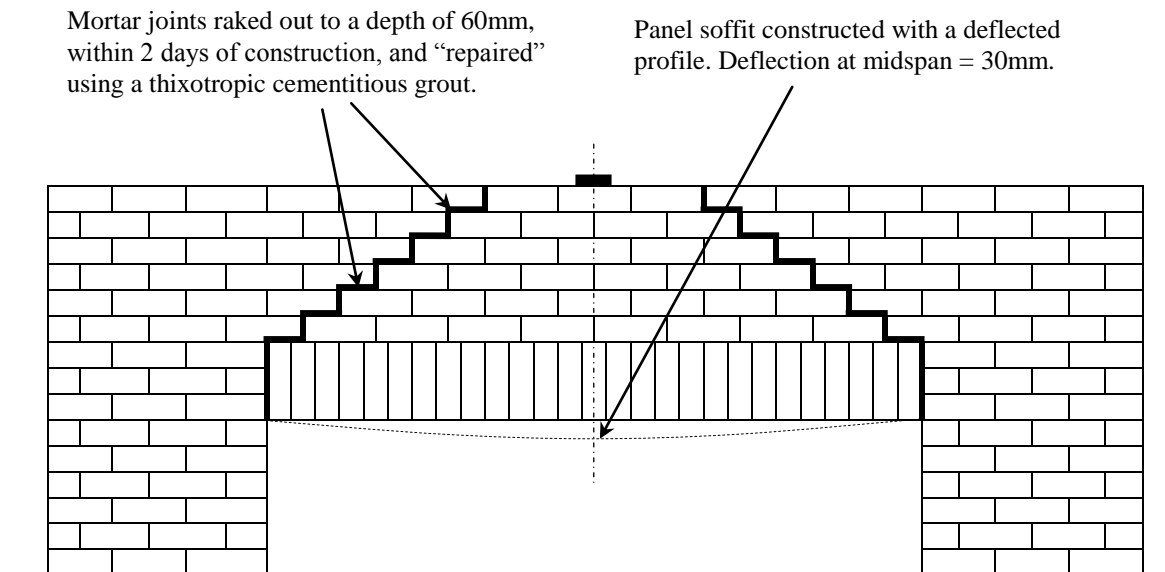


Figure 3. Panel showing simulated pre-damage

The test programme shown in Table 1 was devised to investigate the variables described above.

2.3 Test panel construction

All the panels were constructed against a timber backboard to create an un-pointed finish that would be similar to that achieved on the inner face of a cavity wall. The soldier course of each test panel was built on a propped timber soffit form. This was left in place throughout the installation of any reinforcement and until immediately before each panel was tested. The same bricklayer was used to build each panel in an attempt to minimise any variations in workmanship. All the mortar was weigh batched and all the joints were nominally 10mm thick. On completion of construction, all the panels were left uncovered in the laboratory for a minimum period of 60 days. No special curing measures were employed. For each panel, 4 no. 100mm mortar cube samples were taken. These were left in the laboratory in the same curing conditions as their parent panel then tested in compression on the same day that each panel was tested.

After the minimum period of 60 days had elapsed the bed joint reinforcement shown in Figure 2 was installed. Initially a groove was sawn into the 10mm thick bed joint to a depth of 60mm

using a circular saw fitted with a dust suppressor. A layer of thixotropic cementitious grout was then injected into the back of the groove. (In the case of panels 13 and 14, which were constructed using NHL2 mortar, a lime-based grout was used instead of a cementitious grout). The first 5mm diameter bar, fitted with a twisted wire spacer, was then pushed into the groove; a further layer of grout was injected and the second 5mm diameter bar was installed. A final layer of grout was then injected into the groove and the surface was pointed. In practice, the final layer of grout would be left approximately 15mm shy of the surface of the wall and a layer of pointing mortar, selected to match the existing mortar, would be added to hide the remedial works. The strengthened wall panels were then left for a further 28 days before testing. Again no special curing measures were used for the grout. For each panel, 6 no. 100mm cube grout samples were taken. As with the mortar samples, the grout cubes were left in the laboratory and tested in compression on the same day that each panel was tested.

Table 1. Test panel details

<i>Test Panel Number</i>	<i>Clear opening [m]</i>	<i>Mortar Type</i>	<i>Reinforcing Details</i>	<i>Pre-damaged [YES or NO]</i>
1	2.025	1:12 (PC:sand)	None	YES
2	2.025	1:12 (PC:sand)	None	YES
3	2.025	1:12 (PC:sand)	2 no. 5mm dia. bars in bottom bed joint	YES
4	2.025	1:12 (PC:sand)	2 no. 5mm dia. bars in bottom bed joint + 2 pairs of 600mm long 5mm dia. bars above.	YES
5	2.025	1:12 (PC:sand)	2 no. 5mm dia. bars in bottom bed joint + 2 pairs of 600mm long 5mm dia. bars above.	YES
6	2.025	1:12 (PC:sand)	2 no. 5mm dia. bars in bottom bed joint	YES
7	2.925	1:12 (PC:sand)	None	NO
8	2.925	1:12 (PC:sand)	None	NO
9	2.925	1:12 (PC:sand)	2 no. 5mm dia. bars in bottom bed joint	NO
10	2.925	1:12 (PC:sand)	2 no. 5mm dia. bars in bottom bed joint	NO
11	2.025	1:2.5 (NHL2:sand)	None	NO
12	2.025	1:2.5 (NHL2:sand)	None	NO
13	2.025	1:2.5 (NHL2:sand)	2 no. 5mm dia. bars in bottom bed joint	NO
14	2.025	1:2.5 (NHL2:sand)	2 no. 5mm dia. bars in bottom bed joint	NO

2.4 Test procedure

The pointed face of each panel was covered with whitewash before testing, to aid visual surface crack detection. A vertical central point load was applied to the top of each panel via a solid

steel 100mm x 100mm x 50mm thick spreader plate. The point load was applied using a hydraulic ram controlled by a calibrated hydraulic pump. The reaction was provided by a structural steelwork frame bolted to the reinforced concrete strong floor of the structures laboratory. The load was applied incrementally and, at each increment, the vertical mid-span deflection of the panel was recorded from a dial gauge located on the underside of each panel. During the application of the test load and, in particular, at the end of each increment of loading, the face of each panel was very carefully inspected for the presence of surface cracks. The location and extent of each crack were marked on the surface of the panel using an indelible marker pen (until it became unsafe to do so) and photographs of the surface crack pattern were taken at each load increment. The test continued until the panel either collapsed or was unable to sustain any significant loading.

3. Results

When designing and evaluating the performance of structures, practising engineers are usually concerned with the serviceability limit state of cracking and the ultimate limit state of collapse. With this in mind the behaviour of the test panels is considered at two stages, namely up to and including the formation of the first visible cracks and the post-cracking behaviour up to collapse. Although no attempt was made to measure the surface crack widths during the testing, based on previous surface crack width measurements made by the author using a crack width microscope, surface cracks of between 0.1mm to 0.15mm in size tend to be visible with the naked eye.

3.1 Surface crack development

The observed sequence of surface crack formation in the unreinforced and reinforced panels are summarised in Figures 4 and 5, respectively.

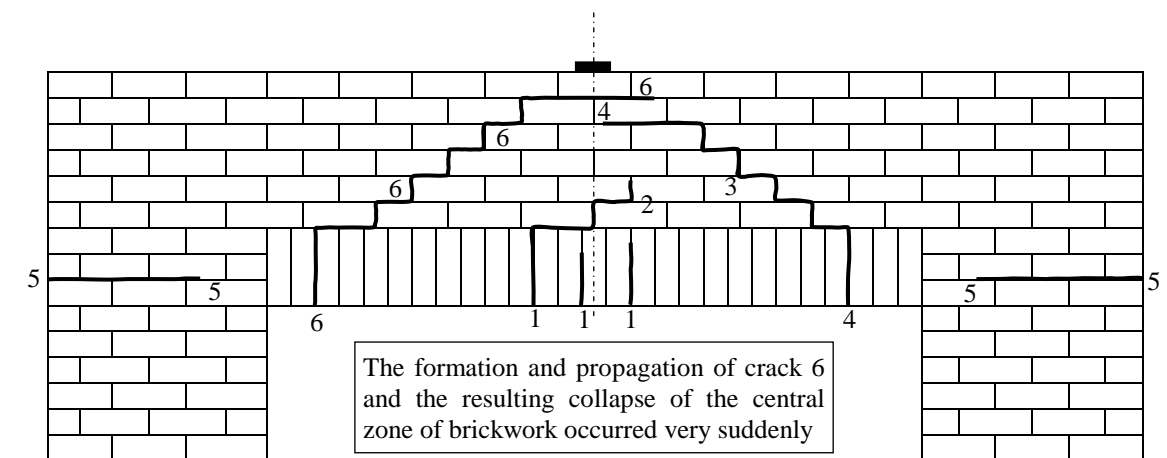


Figure 4. Observed sequence of crack formation (typical unreinforced panel)

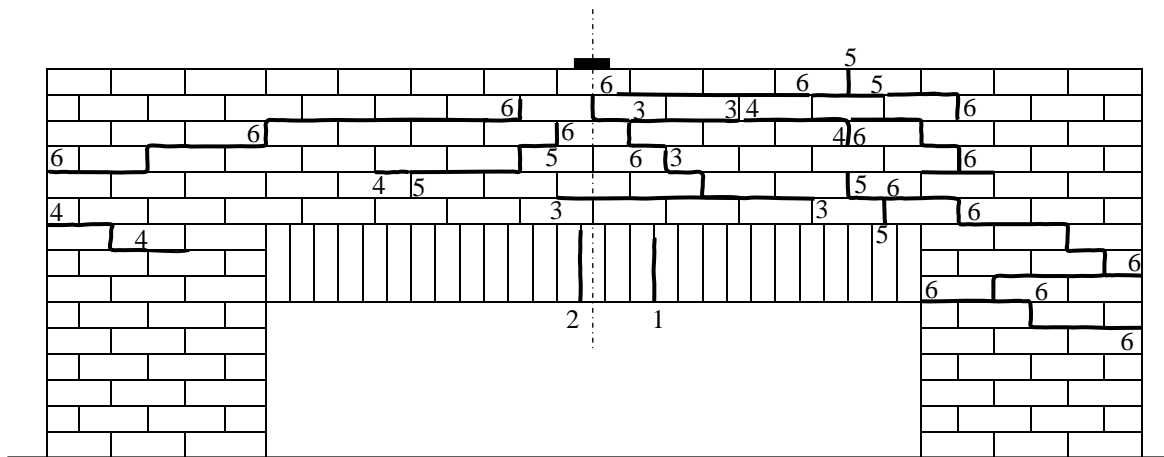


Figure 5. Observed sequence of crack formation (typical reinforced panel)

3.2 Load test results

The load test results are summarised in Tables 2 and 3.

Table 2. Test results: loading to first visible cracking

Test Panel and Description <i>PD = Pre-damaged U = Unreinforced R1 = one layer of steel R2 = two layers of steel C = OPC mortar (and grout) L = lime mortar (and grout)</i>		Mortar compressive strength [N/mm ²]	Grout compressive strength [N/mm ²]	Load @ first visible cracking [kN]	Mid-span vertical deflection [mm]	Mean Load @ first visible cracking [kN] (Increase due to steel reinforcement)
1	2.025m, PD, U, C	0.9	49.8	1.6	0.15	1.6 (control)
2	2.025m, PD, U, C	0.8	49.8	1.6	0.10	
3	2.025m, PD, R1, C	0.9	55.0	6.6	0.62	5.6 (250%)
4	2.025m, PD, R1, C	0.7	55.0	4.6	0.39	
5	2.025m, PD, R2, C	0.6	54.7	1.6	0.09	3.1 (94%)
6	2.025m, PD, R2, C	0.6	54.7	4.6	0.48	
7	2.925m, U, C	0.6	---	0.1	0.13	0.35 (control)
8	2.925m, U, C	0.7	---	0.6	0.27	
9	2.925m, R1, C	0.9	56.7	1.6	0.45	1.6 (357%)
10	2.925m, R1, C	0.6	56.7	1.6	0.41	
11	2.025m, U, L	1.8	---	4.6	0.28	4.1 (control)
12	2.025m, U, L	1.7	---	3.6	0.33	
13	2.025m, R1, L	1.3	3.9	4.6	0.32	5.85 (43%)
14	2.025m, R1, L	1.3	5.8	7.1	0.31	

Table 3. Test results: loading to collapse

Test Panel and Description <i>PD = Pre-damaged U = Unreinforced R1 = one layer of steel R2 = two layers of steel C = OPC mortar (and grout) L = lime mortar (and grout)</i>		Mortar compressive strength [N/mm ²]	Grout compressive strength [N/mm ²]	Failure Load [kN]	Mean Failure Load [kN]	Increase in Failure Load due to steel reinforcement (Increase in load capacity beyond 1 st cracking)
1	2.025m, PD, U, C	0.9	49.8	5.1	4.85	0 (control)
2	2.025m, PD, U, C	0.8	49.8	4.6		
3	2.025m, PD, R1, C	0.9	55.0	13.6	13.6	180% (143%)
4	2.025m, PD, R1, C	0.7	55.0	13.6		
5	2.025m, PD, R2, C	0.6	54.7	15.1	14.85	206% (379%)
6	2.025m, PD, R2, C	0.6	54.7	14.6		
7	2.925m, U, C	0.6	---	1.6	2.1	0 (control)
8	2.925m, U, C	0.7	---	2.6		
9	2.925m, R1, C	0.9	56.7	5.6	4.85	131% (203%)
10	2.925m, R1, C	0.6	56.7	4.1		
11	2.025m, U, L	1.8	---	6.6	6.35	0 (control)
12	2.025m, U, L	1.7	---	6.1		
13	2.025m, R1, L	1.3	3.9	7.6	10.1	59% (73%)
14	2.025m, R1, L	1.3	5.8	12.6		

4. Summary

The bed joint reinforcement was quick and simple to install. Allowing for the inherent variability of masonry, in particular low strength masonry, all the test panels behaved in a fairly consistent and predictable manner. None of the reinforcement or grout showed any signs of premature de-bonding failure at any stage of the testing. The 300mm end anchorage of the reinforcement in the lowest bed joint was found to be very effective at preventing collapse.

The addition of bed joint reinforcement was found, in all cases to: increase the load at which first visible cracking occurred; increase the load carrying capacity; increase ductility to avoid sudden brittle failure mechanisms and to enhance the load carrying capacity after first cracking. These improvements in performance were most evident in the brickwork panels constructed and strengthened using cementitious mortars and grouts.

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