ENHANCING THE STRUCTURAL PERFORMANCE OF LOW STRENGTH MASONRY WALL/BEAMS WITH RETRO-FITTED REINFORCEMENT

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ABSTRACT

One way of enhancing the structural performance of an existing masonry building is to reinforce the already damaged areas of the building or parts that are likely to be subjected to tensile stress or strain in the future. This needs to be carried out at minimal cost and with minimal disruption to the owners without creating a future maintenance liability. Retro-fitted reinforcement typically consists of small diameter steel bars; stainless steel is usually used 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 3 m span openings. Single leaf (wythe) wall/beam panels 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/beam 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, strengthening, walls

INTRODUCTION

For the purposes of this research, low strength masonry is defined as 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 binder 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.

AN OVERVIEW OF MASONRY STRENGTHENING

The structural performance of existing masonry structures 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). *Retrospectively fitted 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 forms of strengthening mentioned above, 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 strengthening 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 (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 is it 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 appears to be even greater. 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 relatively 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.

RETRO-REINFORCEMENT

The principle of using reinforcement in new masonry construction is by no means new [1, 2, 3, 4]. 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 reinforcing existing masonry construction. Typically, it involves the installation of small diameter steel reinforcing bars into pre-cut grooves or pre-drilled holes in the near-surface zones of the masonry that are likely to be subjected 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, 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 [5, 6], the aim being to create a reinforced masonry beam to support the cavity wall construction across the opening without transferring any significant 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 as well as other structures such as masonry arch bridges [7, 8].

TEST PROGRAMME

The performance of masonry beams newly constructed in the laboratory with bed joint reinforcement has been well-researched. Little research, however, 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. 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/m^3 and a compressive strength of the order of 35MPa. 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 1 : 2.5 (NHL2 : sand) mortar. 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.

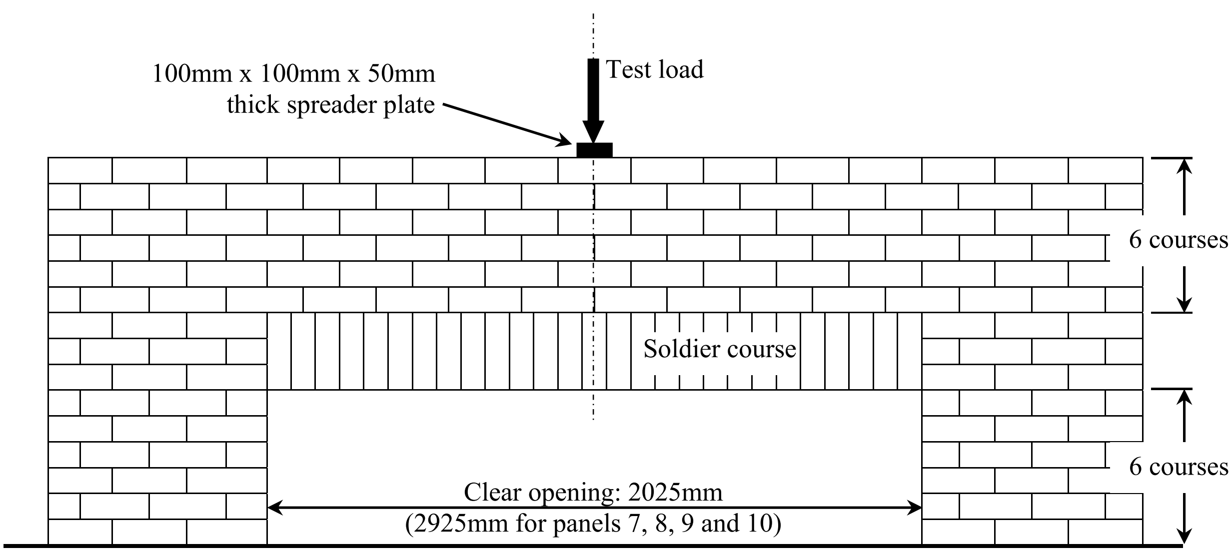

Figure 1: Typical wall/beam panel details showing the test load position

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.

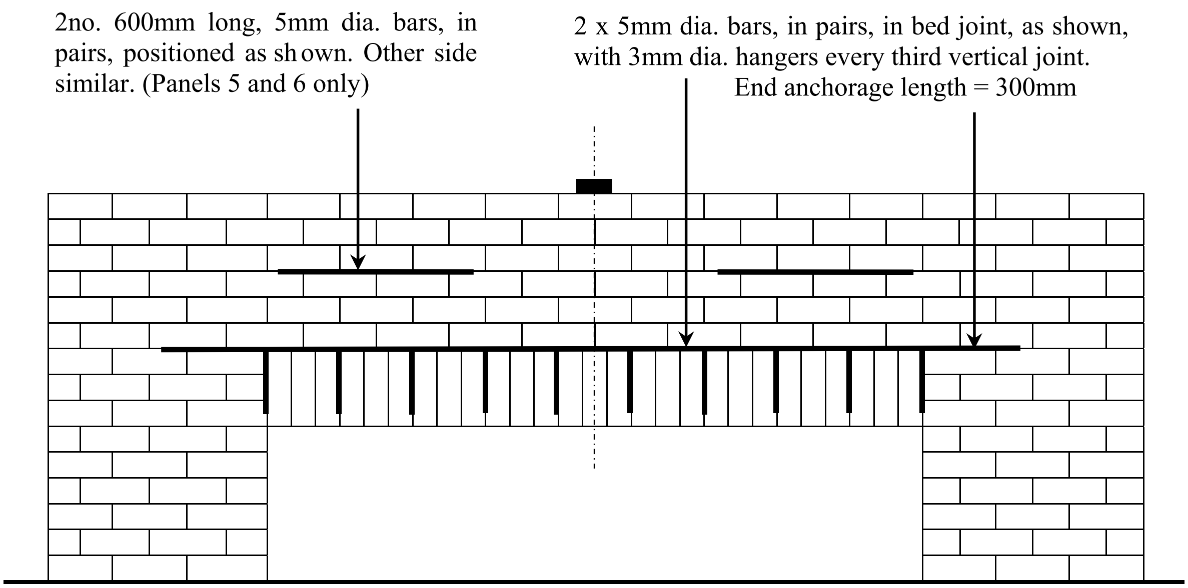

Figure 2: Panel reinforcement layout

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 vertical deflection at mid-span. In addition, to simulate the propagation of diagonal cracks from the corners of the opening, the mortar was raked out of some of the joints 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.

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

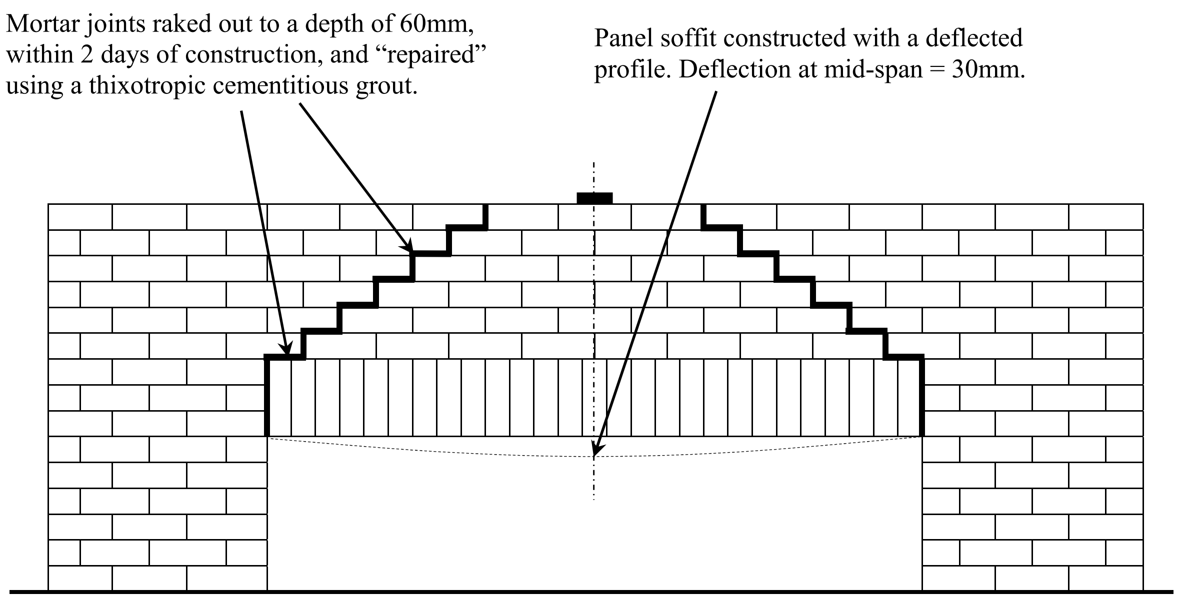

Figure 3: Panel showing simulated pre-damage

Table 1: Test panel details

Test Panel Number	Clear opening [m]	Mortar Type	Reinforcing Details	Pre-damaged [YES or NO]
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

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.

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.

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. The test results up to the occurrence of first visible cracking and up to collapse are summarised in Tables 2 and 3, respectively.

Table 2: Test results - loading to first visible cracking

Test Panel and Description 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)		Mortar compressive strength [MPa]	Grout compressive strength [MPa]	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 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)		Mortar compressive strength [MPa]	Grout compressive strength [MPa]	Failure Load [kN]	Mean Failure Load [kN]	Increase in Failure Load due to steel reinforcement (Increase in load capacity beyond 1st 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		

SURFACE CRACK DEVELOPMENT

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

With the unreinforced panels, as expected, the first signs of visible cracking were seen on the underside (soffit) of the soldier course at or close to mid-span where the magnitude of the flexural tensile stress would have been the greatest. With increasing load, such cracks started to propagate towards the applied load along the bed and perpend (vertical) joints (see cracks 1 and 2 in Figure 4). At or close to the mid-depth of the panel, diagonal cracks were initially noted in the mortar. Again this was expected as the flexural tensile stress combined with the relatively high magnitude mid-depth shear stress would have produced a principal tensile stress exceeding the tensile strength of the mortar resulting in cracking (crack 3). As the vertical deflection of the panel increased under the action of the applied load, the accompanying rotation caused horizontal cracks to form in the panel sides (crack 5). In a real wall panel the restraining effect and vertical compressive self-weight stress of the surrounding brickwork would have prevented such cracking from occurring. As the applied load was increased further, the diagonal cracks propagated both downwards towards the edge of the opening and upwards towards the point of applied load. With very little warning, an additional crack (crack 6) then formed very quickly causing the central area of brickwork to become detached from the rest of the panel resulting in collapse.

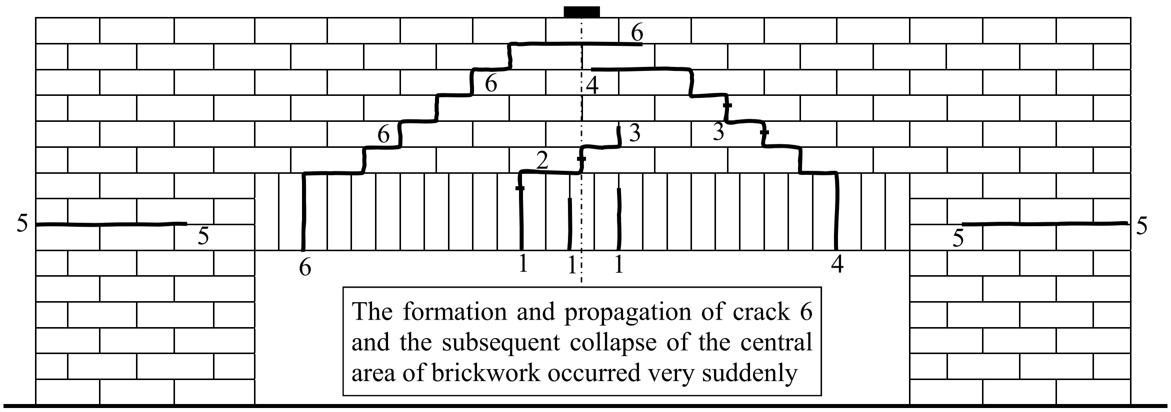

Figure 4: Observed sequence of crack formation (typical unreinforced panel)

The response of all the reinforced panels to the test load was generally very similar. Initially vertical flexural cracks were seen to form in the soldier course (cracks 1 and 2 in Figure 5), followed by further small diagonal cracks in the mortar (crack 3) which propagated towards the edge of the opening and the applied load. Very small horizontal cracks were also observed in either one or both of the panel sides (crack 4). Up to this stage, the behaviour was similar to that observed in a typical unreinforced panel. However, with a further increase in the applied load, many more horizontal and vertical cracks were seen to propagate through the joints (see cracks 5 and 6). As the panel reached its peak load-carrying capacity, many cracks were seen to propagate under constant load. This was accompanied by very short-term creep behaviour as stresses were re-distributed through the brickwork as more cracking occurred. At failure, there was a marked amount of vertical deflection accompanied by one of the diagonal cracks opening up to several millimetres in width. In spite of this large amount of deformation, complete collapse was prevented by the two reinforcing bars in the lowest bed joint which remained anchored in the brickwork.

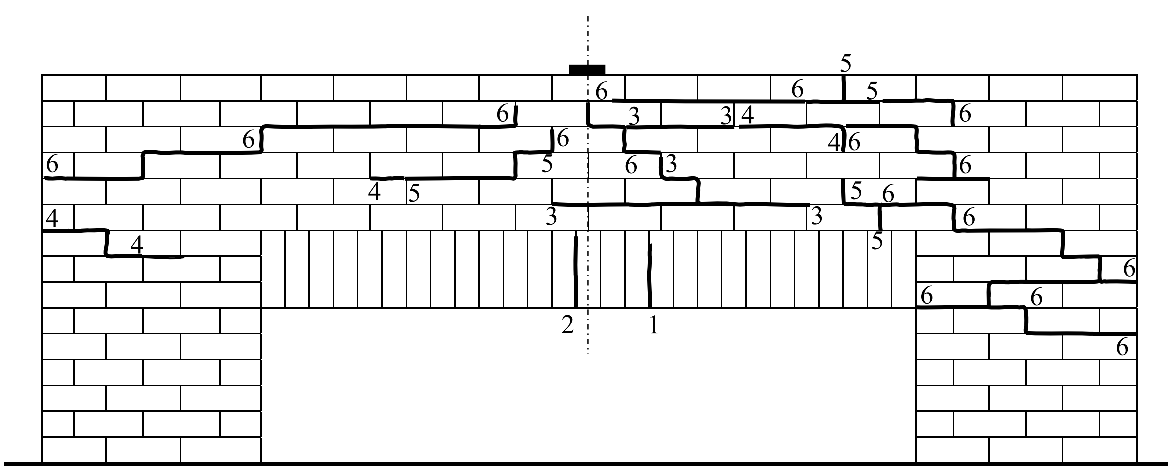

Figure 5. Observed sequence of crack formation (typical reinforced panel)

SUMMARY

The bed joint reinforcement was quick and simple to install. 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 to:

a). Increase the load at which first visible cracking occurred. The increases ranged from 43% (in the panels constructed of lime mortar in which the reinforcement was surrounded by a lime grout) to 357%;

b). Increase the load carrying capacity. The increases ranged from 59% (in the panels constructed of lime mortar in which the reinforcement was surrounded by a lime grout) to 206%;

c). Increase the ductility to avoid sudden brittle failure mechanisms and to enhance the load carrying capacity after first cracking. The increases ranged from 73% (in the panels constructed of lime mortar in which the reinforcement was surrounded by a lime grout) to 379%.

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