Experimental Studies on GFRP Repaired Hollow-Block Concrete Sections

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Abstract: *This study evaluates two different types of techniques for concrete hollow-block sections reinforced with traditional steel rebars and wire meshes, and compares their structural behaviour to that of an ordinary reinforced concrete beam section. The comparisons are based on the responses both before and after they were repaired with glass fibre reinforced polymers (GFRP). The specimens were subjected to concentrated loading up to initial failure. After failure, the specimens were repaired and loaded once again until ultimate failure. It was shown that the success of the repair by GFRP depended on the mode of failure of the hollow-block concrete beams.*

Keywords: *Masonry, reinforced, concrete beam, hollow-block, GFRP*

1. Introduction

Masonry construction is one of the oldest building systems utilized by the humankind. The basic construction methodology has not changed much for several thousand years; units are laid one on top of another such that they form an interlocking mass in at least the two horizontal dimensions. Trying to achieve interlocking in the third dimension with normal rectangular prismatic units is not practical but a degree of such interlocking is sometimes used in ashlars stonework. Most practical masonry constructions employ a mortar interlayer to allow for small inaccuracies of size between units [1]. The early works of masonry construction are characterized by their massiveness and quality craftsmanship. These two facts limited the application of masonry to modern high-rise construction during the first half of last century. However, the interest of structural engineers in masonry was revived after the introduction of reinforced masonry [2].

Reinforced masonry structures are composite structures which utilize masonry units, mortar, grout and reinforcing steel. The following itemizes the advantages of this construction method:

- Reinforcements provide tensile resistance, resilience and ductility

- Its stiffness minimizes deflections

- Masonry partitions are used as structural elements

- Its composite heterogeneous nature tends to maximize its damping ability of dynamic vibratory effects

- Quality control is simple

- Less form work is needed

- Construction time and cost is reduced

- It does not need high calibre labour

- There is no need for heavy hauling equipment [3].

One of the problems associated with this technique is the load transfer over openings. In the past, it was accomplished by constructing arches. However, this technique is not suitable at present since arches are sensitive to settlement of the building and they require a relatively large height. Reinforced masonry beams have been used for some time to avoid these problems [4]. In the U.S., The National Concrete Masonry Association (NCMA), established in 1918 in Virginia, has made design tables available since 1976 [5,6]. However, information about structural behaviour of reinforced concrete masonry beams using local materials seems scarce [7].

In this study, the effectiveness of two different construction techniques for concrete hollow-block beams is investigated at Civil Engineering Department of College of Technological Studies (Kuwit). Locally available materials are utilized in the construction of these concrete hollow-block beams.

2. Experimental Invistigation

2.1. Experimental Specimens

Eighteen specimens were manufactured and tested; six of these experimental specimens were just reinforced concrete beams and used as reference specimens, six specimens were concrete hollow-block beams that were reinforced with steel rebars, and the remaining six specimens were concrete hollow-block beams that were reinforced with a wire mesh.

Even though the reinforced concrete beams are not really compatible with masonry construction, they are included as reference units. The construction of concrete hollow-block beams reinforced with steel rebars requires special preparation of the blocks to allow the placement of rebars. For the case of reinforcement with the wire mesh, an additional mortar bed is provided to secure the mesh. In order to investigate the effectiveness of the GFRP in restoring the load carrying capacity of the beams, all these eighteen specimens were repaired using Glass Fibre Reinforced Polymers after loading to reach the cracking condition. The repaired beams were once again loaded up to failure [9,10].

Half of the specimens had dimensions of 20×40×120 cm with an effective span of 100 cm while the other half had dimensions of 20×40×240 cm with an effective span of 200 cm. Two different span lengths were considered so that the effect of span-to-depth ratio was included in the study. The designations of the short beams are appended with (S), while the long ones with (L).

In both short (S) and long (L) types of reinforced concrete beams, used as Control Beams (CB), the reinforcement was 2 ϕ 12 steel rebars with a cross sectional area of 2.66 cm^2. Similarly, the concrete hollow-block beams, both (S) and (L), which were reinforced with steel rebars (HBS), had exactly the same amount of reinforcement, 2 ϕ 12 steel rebars. The concrete hollow-block beams that were reinforced with 2 ϕ 6 wire meshes (HBW) had a cross sectional reinforcement area of 0.57 cm^2 for both (S) and (L) types. Obviously, this value is much less than the minimum allowed for reinforced concrete and masonry constructions. These specimens were included in this study to be able to evaluate the failure mode of non-reinforced masonry beams and their reliability. All beams were subjected to one

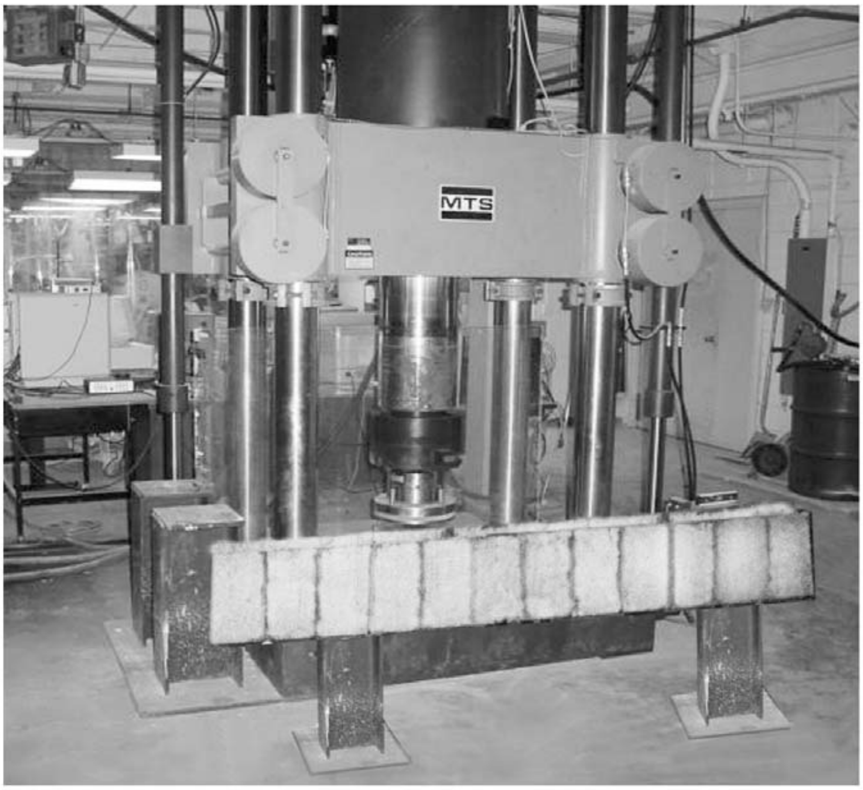

Fig.1 Test Set-up

concentrated load acting at mid-span, as shown in Fig 1.

2.2. Material Properties

The concrete mix used to produce the control beams and grouting the concrete hollow-block beams consisted of cement, fine aggregate and coarse aggregate in the ratio of 1:2.3:3 by weight. The water cement ratio was 0.52 and the amount of cement was 350 kg/m^3. The maximum nominal size of the coarse aggregate was 20 mm. The concrete has been mixed mechanically. The compressive strength determined using 150 mm non-absorbent cube moulds was 17.75 MPa. The compressive strength of the grout determined using 150 × 150 × 200 mm block moulded prisms in accordance with ASTM C-1019 was 14.22 MPa.

The mortar used was produced from Ordinary Portland Cement and fine aggregate in the ratio of 1:3 by volume. The water content is 10% of the total weight of the mortar. The compressive strength of mortar determined using 50 mm cubes in accordance with ASTM C-109 was 14.71 MPa.

The masonry blocks used in the study were conventional 20×20×40 cm concrete hollow-blocks. The average thickness of the face shell was 2.5 cm. The properties of the blocks were determined in accordance with ASTM C-140. The compressive strength of the masonry was determined using two techniques. In the first case, the net strength of one block was measured and found to be 11.66 MPa. In addition, three course grouted prisms (ASTM E-447) were tested and the compressive strength was measured to be

7.79 MPa. The splitting tensile (ASTM C-1006) was found to be 1.13 MPa, while the flexural strength (ASTM C-67) was found to be 2.79 MPa.

Two types of steel reinforcement bars were used. The first type was of diameter 12 mm high grade ribbed steel rebar yielding at 373 MPa and reaching an ultimate capacity of 628 MPa. This type of steel showed a very short yield plateau. The second type of plain steel bars was of diameter 6 mm, wire meshes, having an ultimate capacity of 324 MPa.

The GFRP was commercially available polyester resin using cobalt as a catalyst and peroxide as an initiator. The glass fibres were chopped E-glass fibres. The tensile strength was measured to be 43.15 MPa.

2.3. Fabrication of Specimens

The masonry specimens were built by a qualified mason and were grouted before placing the reinforcement bars. For concrete hollow-block specimens that were reinforced using steel bars or wire meshes, a layer of mortar was placed on top of the grouted masonry beams and then the reinforcements were covered with another layer of mortar. Air curing was applied to all the beam specimens up to the date of testing. After curing, these beams were turned up-side down to have the steel reinforcements in the correct positions. The specimens were tested at an age of 28 days.

After failing the virgin beams, they were removed from the testing frames to be repaired using GFRP [8]. Special care was necessary during this operation to prevent the occurrence of additional damage due to hauling. Therefore, additional upper supports to the specimens were provided before lifting it from the testing rigs. The procedure for applying GFRP was as follows:

- Brush the surface of the beam using a steel brush to remove loose material and dust.

- Add the initiator to the polymer-catalyst mix in the specified ratio.

- Soak the surface of the beam with resin.

- Cover the soaked surface using chopped glass fibre strand and soak it again by resin.

- Press the laminate against the surface to get rid of excess resin and to ensure complete contact with the beam near the corners.

2.4. Test Procedure

The specimens were tested using a 980 kN Universal testing Machine; a dial gauge having a travel of 25 mm was used to record the vertical deflection at the bottom of the mid-span of the beam.

The behaviour of the beams was keenly observed from beginning to failure. The appearance of the first crack, the development and the propagation of cracks due to the increase of load were also recorded. The loading was continued after the initial cracking load and was stopped when the beam was just on the verge of collapse.

3. Experimental Results

3.1. Failure Modes

Various failure modes were recorded for the different beam types. The recorded crack patterns for the different beams tested in this study are given in Figure 2(a, b). The investigation of failure modes can be

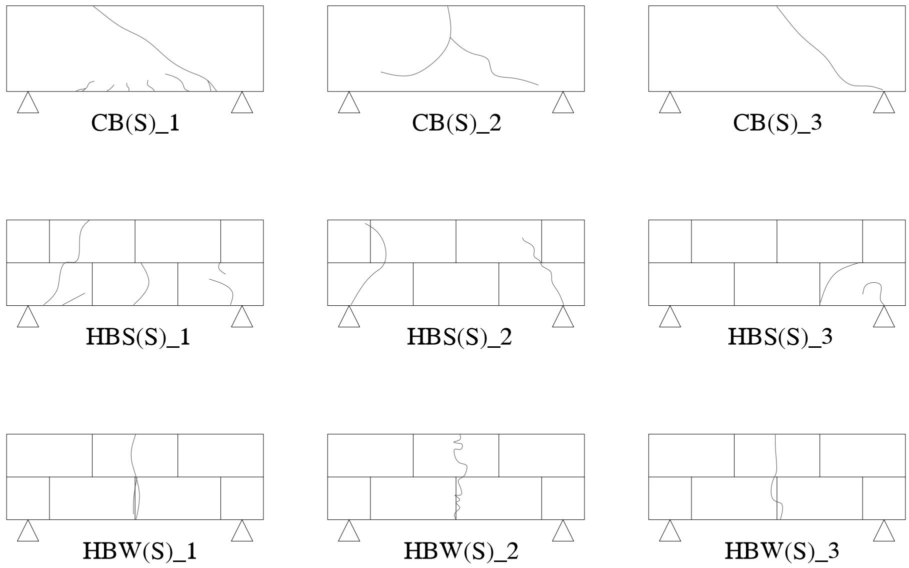

Fig. 2(a) Crack patterns for Short Beams

grouped as follows:

3.1.1. Short beam specimens

Figure 2(a) shows that short beam specimen types BC(S) and HBS(S) mainly developed shear cracks. The exception was HBS(S)_3, in which the cracks were developed near one of the supports. The failure for this case was due to probable adjustment problem in its supports.

Here, it should be noted that flexural cracks appeared in the flexural span as the load was increased in all the beam specimens. However, further increase of load caused the development of the shear cracks which resulted in the final failure of the beams. Accordingly, it can be stated that ultimate strength of the masonry beams reinforced using steel bars was due to shear and that failure of the reinforced concrete specimens was similar to the masonry beam specimens. For the HBW(S) beam specimens, cracks developed near mid-span and started from bottom to top. These cracks indicate clearly that the lightly reinforced beam specimens failed due to pure flexure.

3.1.2. Long beam specimens

Figure 2(b) shows that failure modes of CB (L) and HBS (L) varied from flexural to shear failure. Here, it has to be noted that ratio of span-to-depth of the beam specimens is five. Flexural cracks in HBS (L) specimens always passed through the masonry joints located at the mid sections of the beams. HBW (L) beam specimen failed in flexure. Here, it has to be noted that the flexural cracks were initiating from masonry joints nearest to the mid-span. In two cases, two cracks developed in the bottom part and they were joined in the upper half. This indicates that the cracks propagated easier through the masonry joints.

After the initial fracture, the cracks for both groups (small and long beams) opened relatively rapidly. In all cases of the beams failing in shear, the complete collapse of the specimens was prevented due to the dowel action of the reinforcing bars, as shown in Fig. 3.

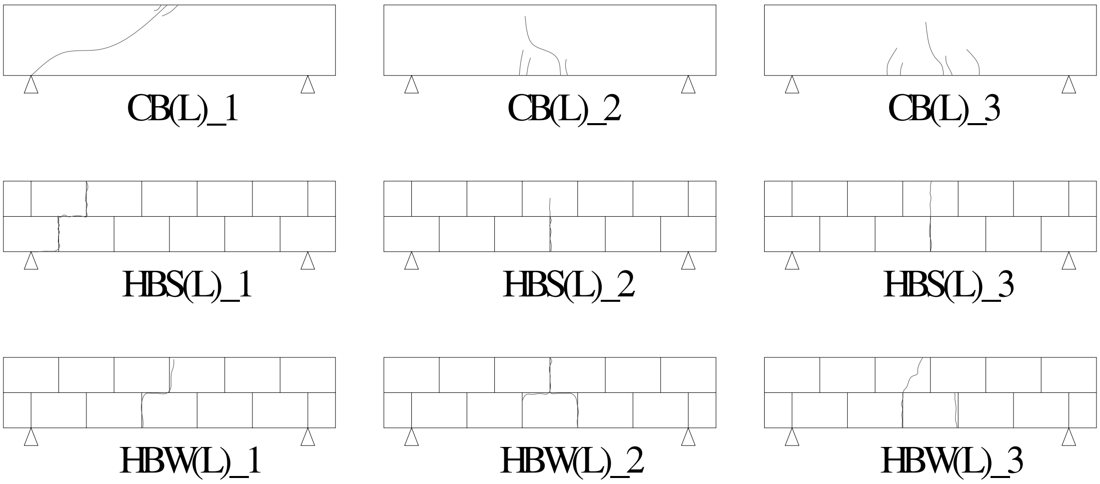

Fig. 2(b) Crack Batterns for Long Beams

Fig. 3 Shear Failure for Beam Tests

Fig. 4 Shear Failure of Beam with GFRP

Table1 The average virgin, repaired and theoretical load carrying capacities of masonry beams

Beam Designation	P_{vir} (kN)	P_{rep} (kN)	P_{th} (kN)	P_{vir}/P_{th} (%)	P_{vir}/P_{CB} (%)	P_{rep}/P_{vir} (%)
CB(S)	119.65	55.51	111.31	107	100	47
HBS(S)	106.70	81.59	111.31	96	90	76
HBW(S)	41.38	41.97	44.33	93	35	101
CB(L)	88.55	87.57	55.70	159	100	99
HBS(L)	77.47	98.75	55.70	139	88	127
HBW(L)	39.13	47.07	22.16	177	44	120

For all the beam specimens repaired by GFRP sheets, it was observed that most of the crack patterns recorded was similar to those of the original beams. These cracks either resulted in fracture of the GFRP layer or resulted in its delamination from the two vertical sides of the beams, as shown in Fig. 4.

3.2. Failure Loads

The average failure load (Pvir) values of all the beam specimens are given in column 2 of Table 1. Results for the same specimens after they were repaired (Prep) by GFRP are presented in column 3 of Table 1.

4. EVALUATION OF EXPERIMENTAL RESULTS

4.1. Theoretical Estimates

The flexural load carrying capacities of the different beam specimens were estimated using the ultimate design theory. Here, it should be noted that the yielding strength of the lightly reinforced beams was found to be lower than their cracking strengths. Thus, the latter was considered to be the ultimate strength of these beams. The theoretically estimated load carrying capacities (P_{th}) for the beam specimens are presented in column 4 of Table 1. The estimated shear load carrying capacity of the sections is about 144.16 kN. The ratio of the experimental virgin load carrying capacity to the theoretical one ($P_{vir}/\ P_{th}$) is presented in column 5 of Table 1.

It can be seen from the Table 1 that the experimental and theoretical results are in relatively good agreement for the short beams; on the other hand, the experimental load carrying capacities of the long beams are higher than the theoretical values by 39% to 52%. One of the reasons for this is that the beams were supported on hinged-hinged supports. This could have enhanced the arching action thus increasing the load carrying capacity.

4.2. Effect of Construction Type

The load carrying capacities of the different types of masonry hollow-block beams were normalized through division by the load carrying capacity of the reinforced concrete beams of the same span. The results of these relative load carrying capacities ($P_{vir}/\ P_{CB}$) are presented in column 6 of Table 1. It can be seen that the percentage value of the load carrying capacities of masonry hollow-block beams reinforced by steel rebars is approximately 90% of the corresponding ones of control beams. Accordingly, it is practically possible to consider them having the same load carrying capacity.

The percentage values of the load carrying capacities of the lightly reinforced masonry hollow-block beams varied from 35% to 44% of the corresponding ones of control beams.

4.3. Effect of Span-to-Depth Ratio

By reducing the span-to-depth ratio of the studied beams, it is seen that the load carrying capacity increases and that the failure mode changes from flexure to shear. For lightly reinforced masonry hollow-block beams, the shear failure was not achieved. For properly reinforced concrete beams, the shear failure occurred for relatively high span-to-depth ratios due to the absence of shear reinforcement and the non-continuity of the grout for HBS-group.

4.4. Effect of Reinforcement Ratio

As expected, the reduction of the reinforcement ratio is associated with reduction in load carrying capacity. However, it is worth noting that the use of very low amounts of reinforcements which are less than the minimum allowed is still useful. Its effect is the increase of the reliability of the beam behaviour.

4.5. Effect of Repairing

The ratios of the load carrying capacities of the beams after repair to their load carrying capacities before repair (P_{rep}/ P_{vir}) are presented in column 7 of Table 1. It is seen that the ratio of restored load carrying capacities for the CB(S) and HBS(S) were 47% and 76%, respectively. The load carrying capacities of all the remaining beams were restored. CB(S) and HBS(S) failed at lower loads and higher deformations without having increased toughness. These beams failed in shear. This implies that the repair of shear damaged beams was not efficient.

5. Conclusions

The following conclusions could be drawn from the results of this study:

- The construction of reinforced masonry hollow-block beams is feasible since they possess load carrying capacity approximately equal to that of reinforced concrete beams.

- The use of very light reinforcement like wire meshes for reinforcing masonry hollow-block beams is viable as long as the beam is anchored in the walls to allow it to benefit from arching effect. The main effect of the reinforcement is to increase the reliability and repeatability of the failure behaviour.

- The success of the repair of damaged masonry hollow-block beams using GFRP depends on the failure mode of the beam. When wire mesh is considered, the repair using GFRP can restore the load carrying capacity of the beam. The efficiency of GFRP is higher for beams failing in flexure.

- It is safe to estimate the load carrying capacity of masonry hollow-block beams based on the rules applicable for reinforced concrete beams.

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