Limiting Structural Damage on Masonry Structures due to Foundation Movement

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ABSTRACT: Over three million low-income houses have been built in the Republic of South Africa over the last two decades, many of which have experienced structural distress due to foundation movement or poor quality construction. The South African low-income housing sector employs a large number of entry-level material suppliers and building contractors, and this results in substantial variability of construction materials, technology and workmanship standards. Underpinning the foundation as a remedial solution to low-income houses is very expensive compared to the actual cost of the product (i.e. the house). Alternatively, it may be cost effective to demolish and rebuild the structure. In this paper, cost effective alternative to underpinning is explored using a proprietary type of reinforcement which is grouted to concrete blockwork. The reinforcement and grout act compositely with existing concrete blockwork, both to repair the cracks and to form deep reinforced masonry beams. This paper covers the initial test programme to investigate and assess the suitability of different reinforcement systems to improve the structural strength and performance of concrete blockwork walling systems. A series of tests were conducted on blockwork wall sections above window and door openings. The material specimens used for the investigation were sampled from a number of construction sites where the quality of the materials varied from site-to-site. The results of the tests showed strength enhancement of the blockwork and an improved ductility.

1 INTRODUCTION

Despite the very stringent housing regulatory environment, a sizeable number of houses built in South Africa experience structural distress resulting in structural failures. This is evidenced by the high number of structural failures as reported by the Regulatory Authority, the National Home Builders Registration Council (NHBRC Annual Report, 2015). An investigation by the NHBRC (2015) into these failures indicate that the root causes for failure are due to:-

- Inadequate structural design caused by improper soil classification resulting in an inadequate foundation solution;
- Construction details that are not in accordance with design specifications;
- Use of unsuitable or poor quality building materials;
- Poor workmanship;
- Poor or non-existence of service infrastructure such as storm-water systems; or
- Complete ignorance and lack of experience of House Constructors.

The General Household Survey of 2013 conducted by Statistics General (2013) indicated that 15.3% of South African households were living in state subsidised houses while a further 13.6% had at least one household member on a demand/waiting list for state-subsidised housing. The survey also included the quality of 2.8 million state housing units delivered by government from 1994 to 2013. The level of quality was measured in terms of whether the walls and roofs of the dwellings were: very good, good, needed minor repairs, weak or very weak. 15.9% of the households surveyed reported their homes had weak or very weak walls; while 15.3% reported the same for their roofing structures.

On the other hand, Table 1 shows a breakdown of claims paid by the NHBRC for the private sector mortgaged houses in all the provinces of South Africa for the period 2014 to 2017 (NHBRC Annual report 2018). From the table, it is apparent that most of the problems are due to settlement of foundations (61%) and failure of superstructure (37%). These problems are, however, interrelated because settlement of foundations will cause structural distress in the walls.

Table 1: Claims Paid Statistics

	2014/15	2015/16	2016/17	Total	%[1]
Fdn	R4,33m	R1,96m	R1,20m	R7,49m	61%
Super Struct.	R2,89m	R0,90m	R0,63m	R4,42m	37%
Roof	R0,15m	R0,04m	R0,03m	R0,22m	2%
Total	R7,37m	R2,90m	R0,39m	R12,1m	

[1]% of total claims paid in the period 2014 to 2017

Structural damages to houses and loss of life has been far more severe on dolomites and heaving clays than any other geological formation in South Africa. Dolomite is a rock mainly composed of a mineral – calcium magnesium carbonate ($CaMg(CO_3)_2$), and is distinguished from other rock types due to its relatively high solubility. Rainwater and percolating groundwater is enriched with carbon dioxide, which results in a weak carbonic acid forming.

The weakly carbonic acid groundwater moves through the joints, fractures and faults in the dolomite and as a result the rock is slowly dissolved over years. Dolomites underlie many of the densely populated areas of South Africa. Approximately 2.5 to 3 million people in South Africa live on dolomites. The result of water infiltration in the dolomites is the formation of sinkholes which subsequently compromises the structural integrity of the houses. A typical sinkhole in a dense township is shown in Figure 1. The risk of formation of sinkholes is not only for integrity of the top-structure, but also for the safety of the occupants, children in the playground and neighbours.

Figure 1: Sinkhole formation on dolomites

Mahachi et.al. (2004) also reported that a number of failures of the super-structures are a result of poor workmanship and not paying particular attention to construction detail. However, as mentioned earlier, super-structure failure may also be caused by failure of the foundation system. Example of super-structure failure caused by foundation settlement is shown in Figure 2. This house was part of a housing complex development in which 20 houses were constructed on a backfilled brick quarry. Densification of the upper 2 m of the soil profile using an impact roller was inadequate resulting in severe structural distress on the super-structure of all the houses. All the houses in this development had to be demolished and the area was condemned for residential development.

Figure 2: Superstructure failure

Other common causes of super-structure failures are observed to be due to structures being erected on 'cut and fill' platforms. Investigations have shown that most of fills are not compacted adequately, and are usually far below the accepted norm for an engineered fill. Over a period of time, a combination of collapse and consolidation of fill due to poor compaction result in localized settlement of the platform causing distress to the foundation and ultimately the superstructure. The remediation works usually involve complete demolishing and rebuilt of the house or expensive underpinning of the foundation.

Availability of good construction materials is a major challenge in a number of areas, particularly in the rural villages of South Africa. It is common practise in these rural areas to find houses constructed with materials (e.g. clay bricks or cement blocks) that are manufactured 'locally' in these areas using local labour, and where no quality management systems exist. The result of this are housing products with varying degrees of quality, and in a number of cases, affecting negatively the structural performance of the houses. In some cases, the scarce availability of raw materials make the house constructors to 'cut-corners' in order to reduce the consumption of materials and hence compromise the quality of the product.

The remediation usually applied to address most of the above problems is the use of underpinning methods using mass concrete or mini piling. These approaches prove to be expensive for a low-income house of $40m^2$ footprint. For a house of this footprint, it may be cost effective to rather demolish, salvage any good materials/products and rebuild the house. There is therefore a need to investigate affordable and effective ways of remediation of low-income houses.

An approach that is being investigated in this paper is the use of a proprietary system, the "HeliBeam". This system was investigated as a possible remediation of walls for low-income houses built using concrete blocks. The system was commissioned for possible use in South Africa by Helifix, U.K. The initial research conducted was to investigate the suitability of the HeliBeam for improving the structural strength of low-income houses; where unacceptable sag of blockwork wall sections occurs over doors or windows. This could be as a result of foundation settlement or poor workmanship, etc. A typical example where a lintel over a door opening was incorrectly installed and also suffered foundation settlements, is shown in Figure 3.

Figure 3: Poor installation of lintel

2 EXPERIMENTAL PROGRAMME

2.1 *The HeliBeam System*

The HeliBeam system (Helifix, 2004) is a proprietary helical reinforcement which is embedded into the bedjoint of existing masonry buildings using a special grout (the "HeliBond"). The effect of the reinforcement and the grout is to act compositely with existing brickwork or blockwork, and has the potential to be used to repair cracks or to form deep reinforced masonry beams. The installation of the HeliBar is illustrated in Figure 4. If these beams can span over areas suffering from ground movement, or where crack stitch is required, they can reduce the need for costly traditional underpinning. This has thus the potential of reducing differential settlement problems in housing.

Fig. 4: HeliBeam system

2.2 *Testing Programme*

The South African subsidy-housing sector employs a large number of entry-level material suppliers and contractors and this results in substantial variability of construction materials, technology and workmanship standards. In view of that, an approach was adopted in which a representative spectrum of material samples were collected from several construction sites and subsequently constructed as HeliBeams and tested.

In general, the quality of the blocks was found to be fair, although they visually varied substantially from one site to another. Blocks which were inferior or cracked were discarded from use in the tests. Large discrepancies were observed between the quality/suitability of sands used for the mortar at different sites. These varied between coarse river sand containing organic elements to fine sand containing large quantities of silt.

The following parameters for the development of test specimens were adopted and implemented in accordance with SANS 10400 (2011) requirements:

- a single skin wall constructed of 140 mm hollow concrete blocks. The sizes of the blocks were 440 x 140 x 215 mm. Representative samples were also tested for the crushing strength which varied from site to site. The strength varied from 2.8 to 3.2 MPa;
- 2 layers of blocks forming the lintel over the opening;
- no reinforcement in between the concrete block layers;
- shell bedding of blocks with hand-mixed mortar of about 5-7 MPa; and
- 1 500 mm clear span of the opening. The choice of 1.5m span was based on the worst probable of an opening occurring in the top structure of a low-income house.

The principal materials and structural components were assessed and evaluated prior to commencement of the tests. Five sample lengths of the HeliBars were measured to determine the average cross-sectional area, which was 9.01mm^2. The samples were then subjected to tensile axial loading tests. The results showed that the average ultimate tensile strength was 1100 N/mm^2, with an elastic modulus of 118 kN/mm^2. The stiffness was considerably less than the stiffness of the parent material of 200 kN/mm^2 due to geometric and material properties of the helical shape bar.

The concrete blocks were pre-slotted individually, before the construction process took place. All blocks were slotted in order to accommodate the installation of HeliBars. The grooves were cut to an approximate depth of 23 mm, height 20 mm, and at a distance of about 70 mm from the edge of the blocks. A minimum curing period of 28 days was applied to all beams, which were constructed. Subsequently, the installation of 6mm galvanized carbon steel HeliBars and HeliBond grout took place and again a period of about 20 days was provided for the curing of the HeliBeam installation. Figure 4 shows a HeliBar positioned in the concrete beam.

2.3 *Experimental Set-Up*

For the tests carried out, the structural scheme of a beam with fixed-ends was substituted by a simply supported beam. The relationship between the magnitudes of the bending moments, corresponding to both situations, was then factored in the application of loading. The adoption of this principle enabled a significant reduction in the amount of testing material. The effect of horizontal end restraint, due to the presence of the remaining blocks forming the walls as well as the roof structure, was provided by horizontal tie-screws, installed at both levels of the blocks. Efforts were made to ensure that no pre-stressing in the horizontal direction took place.

The loading configuration simulated a point load which was determined by considering typical loads that would act on an opening of 1.5 m, i.e. Dead Load (G_k) from the roof and uniformly distributed load (Q_k) of 0.5 kN/m^2. Although this load seemed high, it was considered appropriate for low-income subsidy houses *as additional loads are often positioned on top of the roofs* to prevent the uplift of roof sheeting. The ultimate concentrated load, based on SANS 10160 (2011) load combination of $1.2G_k + 1.6Q_k$ was 4.25 kN. The vertical loads were applied via a 50mm wide wooden spacer.

This approach thus led to the adoption of the most conservative situation of one roof-beam placed centrally over the lintel-beam and carrying the loads from a contributory area of 1.5 metres wide. A typical test set-up is shown in Figure 5. The mid-span deflections were measured by means of a dial gauge, and observation of any cracks were noted.

Figure 5: Loading arrangement

The loading was simulated by a set of steel weights, and the tests were divided into three Groups as follows:

Group 1: Reference test group with 3 samples were tested without any HeliBars. The tests were conducted as a baseline for comparison with HeliBeams.

Group 2: Three samples with one HeliBar (placed at the bottom) and three samples with two layers of HeliBars (placed top and bottom). The samples of the blocks were taken from one site.

Group 3: Three samples taken from four different construction sites (i.e. a total of 3 x 4 = 12 samples). All tests were conducted with one HeliBar placed at the bottom. The choices of the sites were dependent on availability of the blocks and to provide a wider spectrum of quality of the blocks.

Based on the above, a total of 21 HeliBeams were tested.

2.4 *Performance Criteria*

A survey of the local literature (relevant design and construction manuals / codes), to identify the standard requirements related to the range of maximum vertical deflections of blockwork, was undertaken. The investigation revealed no specific requirements in this regard, and it was decided to use the recommendation of the South African National Standard, SANS 10400 (2011). To minimise cracking, the standard specifies a limiting deflection value of 1/350 of the span for articulated non-reinforced masonry and 1/300 for ‘lightly reinforced’ masonry. In this investigation a more conservative value of 1/ 350 deflection is used, which translates to about 4.3 mm.

2.5 *Limitations of Experiment*

A number of variables have not been considered in this initial investigation. The following parameters were thus not considered:

- Spans larger than 1.5m. In the event of sink-hole formation, it is likely that the beam may have to span more than 3m;
- The effect of the depth of the beams. Shear deflections may play an important role for deeper beams;
- The size and types of the concrete blocks. The behaviour of solid concrete blocks versus hollow concrete block is different and will influence the behaviour of the HeliBeams; and
- Depth and or position of the HeliBars in the concrete blocks.

Further research will be required to investigate the effects of the above parameters.

3 TEST RESULTS AND DISCUSSIONS

Group 1 Observations:

Upon the release of the supporting shutters on the beams, all unreinforced beams deflected instantly under their own self-weight, with a visible sag of between 4 and 5.5 mm. This was accompanied by several horizontal and vertical cracks developing in the mortar, with crack widths between 1 mm and 2 mm, as is evidenced in Figure 6.

The results of the loading tests are presented in Figure 7, where the load is plotted against mid-span deflection. The loading of the beams introduced an additional sag of between 1.5 mm and 2 mm. The HeliBeams could not carry a load greater than 2 kN as this was accompanied by large deflections and further opening of the cracks. The resultant, total mid-span deflections (under a load of 2 kN) was between 5.5 mm and 7 mm, which are above the performance limit of 4.3 mm. It was therefore concluded that unreinforced beams clearly fail to fulfil their functional requirements. The variability of the results, as is evidenced in Figure 7, is also indicative of the variability in quality of the concrete blocks.

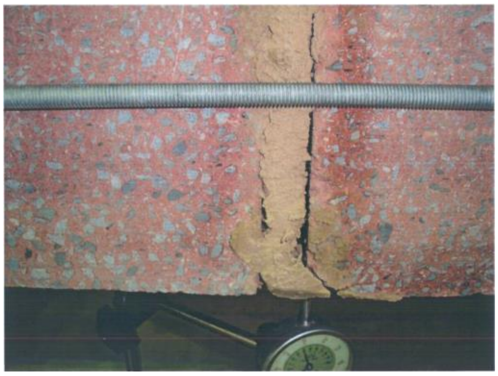

Figure 6: Cracking of unreinforced block beam

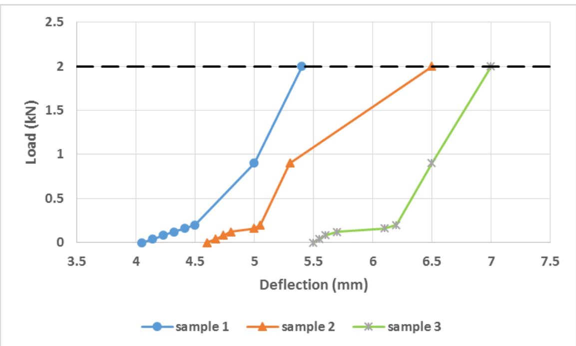

Figure 7: Load-deflection for Unreinforced Beam

Group 2 Observations:

The results for this group of tests are presented in Figure 8 for beams reinforced with bottom layer of HeliBars and Figure 9 for beams reinforced with both bottom and top layers of HeliBars. It can be seen that both graphs indicate better performance of beams fitted with HeliBars. A slightly better performance corresponded to beams fitted with the double layer of HeliBars (Figure 9). In Figure 9, the initial sag, upon the removal of the supports to shutters, was only about 0.5 mm, with a final deflection of about 0.9 mm under a load of 4 kN. In comparison, the initial deflection of the beams with a single HeliBar was of the order of 0.4-1.2 mm and a total deflection of between 1 to 2 mm. The scatter of the results is much less reduced for two layer reinforcement compared to single layer reinforcement, indicating the enhancement effects of the HeliBars. The apparent similarity between both situations, shown in Figures 8 and 9, indicate a situation in which the resistance offered by the bottom HeliBar may be sufficient to cope with the imposed level of the loading and does not require the engagement of the compression top-bars.

These results, further illustrates the excellent composite action of the HeliBars and HeliBond.

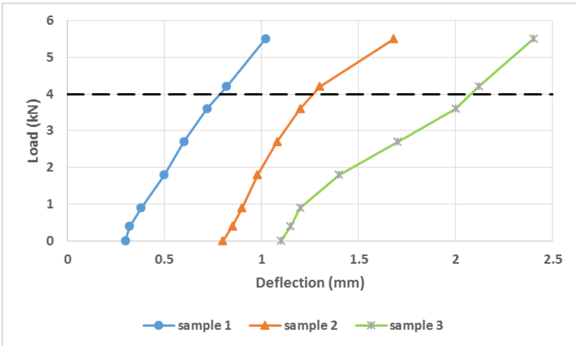

Figure 8: Load-Deflection (Bottom layer HeliBar)

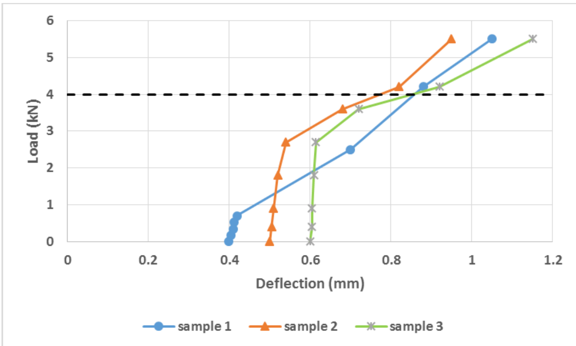

Figure 9: Load-Deflection (Top and Bottom layer HeliBars)

Group 3 Observations:

The aim of Group 3 tests was to assess the overall performance and consistency of the HeliBars as affected by the variability of the quality of materials in the South African mass-housing construction industry.

Four sets of three beams each were tested from different sites. Like the Group 2 samples, these tests were carried out with a load of up to 5.5 kN which exceed the required ultimate design load of 4.3 kN by 30%. The results of the tests showed a behavior similar to that shown in Figure 8 and a summary of the test results is presented in Table 2.

Table 2: Summary of Results

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Site	Sample No	Initial Defln (mm)	Total Defln (mm)[1]	Defln under load (mm)[2]
A	A1	0.7	1.3	0.6
	A2	0.9	1.5	0.6
	A3	0.9	1.8	0.9
	Average	**0.8**	**1.5**	**0.7**
B	B1	0.3	0.8	0.5
	B2	0.8	1.3	0.5
	B3	1.0	1.8	0.8
	Average	**0.7**	**1.3**	**0.6**
C	C1	1.1	1.9	0.8
	C2	1.1	2.0	0.9
	C3	1.2	2.2	1.0
	Average	**1.1**	**2.0**	**0.9**
D	D1	0.7	0.9	0.2
	D2	1.0	1.2	0.2
	D3	1.2	1.6	0.4
	Average	**1.0**	**1.2**	**0.3**

[1] Deflection at a load of 4 kN

[2] Total deflection – Initial deflection

The results as presented in Table 2 show some variability of the deflections. The average initial deflection after the shutters were removed varied between 0.8 mm and 1.0 mm. This initial settlement is considered insignificant and it is not necessarily wholly attributable to the mid-span deflection of the beams but rather it could be a contribution of settlement on the supports, compression of the sand grains and a deflection of the supporting beams.

Furthermore the deflections which were measured under the load were less than 1.0 mm and the maximum total deflection was less than 2.0 mm at the ultimate load of 4.0 kN, which is within the permissible value of 4.3 mm. No structural cracks were observed in any of the specimen that were fitted with HeliBars.

4 CONCLUSIONS

A laboratory testing programme was carried out in order to investigate and assess the structural performance of the HeliBeam system applied to unreinforced block-work wall sections above door and window openings. It was based on a series of loading tests and measurements of deflections using specimens of beams constructed of material obtained from six different construction sites. The loads, which were applied, exceeded the magnitude of the ultimate loads due to self-weight and imposed loads as specified in SANS 10160 (2011).

Initially a set of benchmark tests were undertaken using three unreinforced beams. The results proved to be unacceptable due to crack formation, as well as the magnitudes of deflections which exceeded the allowable limits. Subsequently a set of comparative tests was carried out to evaluate the situations of a single HeliBar (positioned in the bottom layer of the blocks) versus two layers of HeliBars (installed in top and bottom layer of the blocks). Both situations indicated a very good performance, although the latter solution proved to provide a slightly better structural performance.

Also, results obtained from four sets, of three beams each, constructed out of material obtained from different construction sites and fitted with one HeliBar proved again to offer a more than satisfactory structural performance.

It can therefore be concluded that the HeliBeam system has the potential of improving the strength of blockwork and may offer an effective variation on traditional underpinning practices. However, further research work is still required to investigate the effects of shear, position of HeliBars and span of beams.

5 ACKNOWLEDGEMENT

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