Client Report :
A design method for use with 6mm diameter Thor Helical tie wires used as retrofitted bedjoint reinforcement

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Prepared for :
Dave Chadwick

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Prepared on behalf of BRE by

Signature	
Name	Alan Ferguson
Position	Head of Structural Masonry

Approved on behalf of BRE by

Signature	
Name	Robert Ramsay
Position	Commercial Director
Date	19th November 2003

BRE
Construction Division
Bucknalls Lane
Garston
Watford
WD25 9XX

Tel : 01923 664200
Fax : 01923 664096

Email : construction@bre.co.uk
Website : www.bre.co.uk

Executive Summary

This paper proposes a design method for use with 6mm diameter Thor Helical helically profiled tie wires, when they are used as a remedial technique to create deep beams for reinstating the structural integrity of buildings suffering from cracking.

The system itself involves retrospectively reinforcing the mortar beds of an existing masonry structure by removing part of the outer portion of a bed and inserting two 6mm diameter helically profiled Thor Helical tie wires encased in a strong grout. Depending on the number and location of the beds reinforced in this way, either deep masonry beams can be created - returning structural integrity to a cracked building - or localised reinforced areas can be produced - reinstating more localised areas of a structure.

The principles on which this design is based assumes that the insertion of the wires into two distinct bed joints at different levels in the wall creates a deep beam: the combination of the wires and the two courses of brickwork surrounding it form chords that represent the upper and lower flanges of the beam; and the unreinforced courses of brickwork between the top and bottom chords can be seen to be equivalent to the web.

Contents

1 Background

This paper proposes a design method - and loading tables - for use with 6mm diameter Thor Helical tie wires, when they are used as a remedial technique for reinstating the structural integrity of buildings suffering from localised cracking.

The method involves retrospectively reinforcing the mortar beds of an existing masonry structure by removing part of the outer portion of a bed and inserting the two 6mm diameter Thor Helical tie wires encased in a strong grout. Depending on the number and location of the beds reinforced in this way, either deep masonry beams can be created - returning structural integrity to a cracked building - or localised reinforced areas can be produced - reinstating more localised parts of a structure.

It should be noted that, in buildings built using cavity construction, rather than just one leaf being assessed for repair, the condition of both the inner and outer masonry leaves should be looked at as providing a solution in just the outer leaf will not necessarily provide a long lasting repair.

2 Design Approach

The BS 5628: Part 2: 2000 [1] method for the design of reinforced masonry beams is intended to be used in new build applications, using new materials; use of the pairs of the 6mm diameter Thor Helical tie wires to form a deep beam is intended for use in existing buildings. The design philosophy being proposed here for the design of the deep masonry beams does not follow the requirements of BS 5628: Part 2.

This method does not include any assessment - or give any advice on the assessment - of the imposed loads on the beams. That aspect is the responsibility of the engineer involved in the particular project. It only covers the design resistance of the beam.

This design method assumes that the insertion of the Thor Helical tie wires into two distinct bed joints at different levels in the wall creates a deep beam: the combination of the wires and the two courses of brickwork surrounding them form chords that represent the upper and lower flanges of the beam; and the unreinforced courses of brickwork between the top and bottom chords can be seen to be equivalent to the web.

3 Design Method

The beams created using this system can be encastred or continuous over supports (in the encastred condition, the ends of the beams are fully restrained against rotation). These beams are formed by adding reinforcing wires, set in high strength grout, into the partially removed mortar beds of existing masonry walls. The masonry provides restraint against in-plane buckling - when longitudinal chords carry compression - and resistance to the horizontal shear forces in the unreinforced beds between the top and bottom chords. It is assumed that the reinforced areas of masonry provide the resistance to the bending moments and vertical forces applied to the beam.

This design method considers the circumstances where a chord consists of two courses of brickwork, between which are inserted a pair of 6mm diameter Thor Helical tie wires, embedded in grout.

As the system is designed so that hogging - as well as sagging - bending moments can be accommodated, the same chords are used in both the top and bottom of the deep beams.

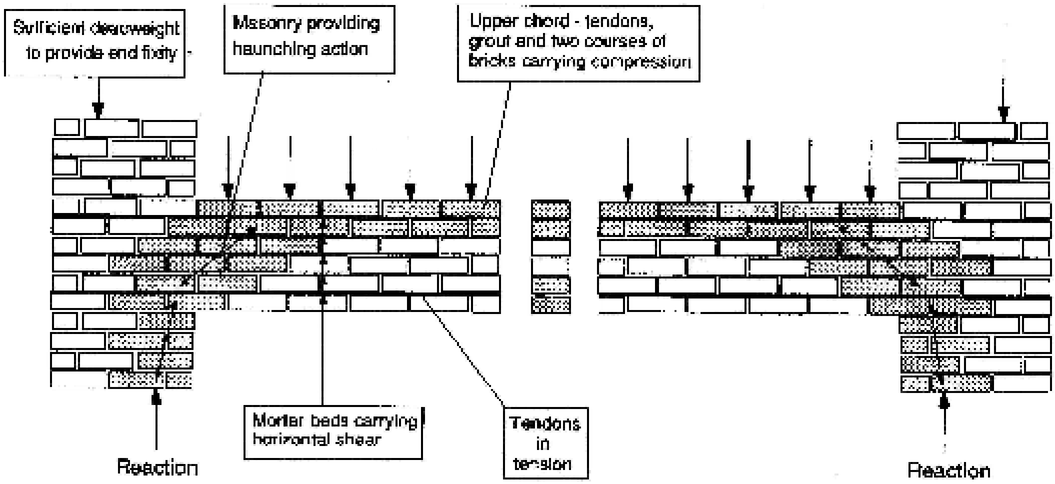

Figure 1: Schematic diagram showing the basis for the design method

Figure 1 shows in graphical terms how the loads applied to a deep beam formed in this way are resisted. This resistance forms the basis for the design method. Essentially, the vertical loads are carried by the chords and the horizontal shear is taken by the unreinforced masonry between the reinforced chords.

It can be seen that a number of key properties are required to determine the resistance of a beam. These are:

- the vertical shear strength of the system – that is, the performance of the system perpendicular to the direction of the reinforcing wires;
- the direct compressive strength of the system – that is, the performance of the system parallel to the direction of the reinforcing wires; and
- a lower bound value for the horizontal shear strength of the masonry between the chords.

Tests have been carried out at the BRE on specimens of this type to determine such properties. In practice, as the system is intended to be used in buildings of various ages, it would be expected that it would be inserted into materials with a variety of strengths. However, in view of the fact that the exact properties of these materials are unlikely to be known in the majority of cases, these tests used weak materials with the deliberate intention of providing lower bound values: a 1: 3: 12 cement: lime: sand (CLS) mortar mix was used throughout, and the units were a yellow multi-stock brick with a compression strength of around 8 N/mm². Once the tests were completed, characteristic values were calculated for the system. The characteristic values obtained from these tests have been used in this design method.

The output of this design method is a series of design tables which give, for a given span, the limiting load that can be applied to the beam.

4 Detailed design considerations

4.1 Partial Safety Factors

The partial safety factors used in this design method have been taken from BS 5628-2:2000. These partial safety factors are there to "make allowance for variation in the quality of the materials and for the possible difference between the strength of masonry constructed under site conditions and that of specimens built in the laboratory for the purpose of establishing its physical properties." (BS 5628-2:2000, Clause 7.5.1)

The serviceability limit state partial safety factors, rather than ultimate limit state partial safety factors, have been used as this design method is only applicable under circumstances where the system is being used to reinstate those portions of a building that have suffered localised cracking failure: it will not be used to act as a means of "reinstating" a house/building that is in imminent danger of collapsing. As has already been stated, under those circumstances the remedial work would need to be tailored to each individual case and the use of the design tables contained in this report would not be appropriate.

The partial safety factors used in this design method are taken from Clause 7.5.3.2 of BS 5628-2 [1], for the serviceability case, as:

Partial safety factor for masonry in compression, $\gamma_{mm} = 1.5$

Partial safety factor for masonry in shear, $\gamma_{mv} = 1.5$

Partial safety factor for steel, $\gamma_{ms} = 1.0$

The design stresses for the different material properties used in this design method have been obtained by dividing the characteristic strength of the material by the relevant partial safety factor.

The γ_{mv} value for the vertical shear resistance of the masonry, has also been taken as 1.5. γ_m values are used to take into account the variability of material properties and workmanship. However, the horizontal shear resistance of the existing masonry will be known as a result of the tests that are required to be carried out on the masonry in-situ so no γ_{mv} value needs to be applied to that.

An additional safety factor of 2.0 has been used for the limiting serviceability condition, contained in section 4.10.

4.2 Effective depth of a deep beam

The effective depth of the beam has been taken as the distance between the centres of the reinforced chords, as a beam of the type suggested in this analysis is capable of resisting both hogging and sagging bending moments. This will tend to be a conservative assumption, as any courses of brickwork above the girder beam will add to both the vertical shear resistance and the bending resistance of the beam.

When each chord contains just two wires, the effective depth of the deep beam - d - is taken to be the distance between the top and bottom reinforcement wires; when each chord has two pairs of wires in adjacent beds, the effective depth of the beam is taken to be the distance between the centre of each chord.

4.3 Tensile capacity

The tensile capacity of the lower chord has been based exclusively on the tensile strength of the steel wires, as it is assumed that the masonry materials do not contribute in tension. The capacity of the section has been determined using the mean cross-sectional area of the steel, and a 5th percentile linear elastic limit stress.

4.4 Compression due to bending

The calculation of the compressive capacity of a deep beam is based, for a single chord, on the characteristic compressive stress obtained from BRE experiments. These were carried out on specimens tested without any pre-compression and are therefore likely to represent a lower bound figure, bearing in mind the amount of masonry surrounding the deep beam in situ.

4.5 Distance between neutral axis and a chord in maximum tension due to bending

The neutral axis is, by definition, the surface of zero stress in a beam. Its location will be determined by the relative moduli of elasticity, E, of the materials in the tensile and compressive zones in the beam. These E values are unlikely to be known: it would be necessary to determine E for a composite material consisting of mortar, bricks, steel and grout, and it would be very difficult to measure them on site. As a result, this design method is based on the assumption that the tensile and compressive strengths of the materials in the beam are proportional to their Young's Moduli: the strengths have therefore been used to determine the position of the neutral axis in this design.

The calculation of the location of the neutral axis has been carried out with the partial safety factors set to 1.0.

4.6 Maximum moments of resistance

The maximum moments of resistance of the beam have been calculated on the basis that, knowing the location of the neutral axis, the contributions due to compression and tension zones are the same. Either one of these could therefore be used in the calculations, and the tensile values were chosen as they are perhaps easier to define with confidence - they depend on the properties of the steel rather than the masonry. The moments of resistance of the beam as a whole are double those due solely to the tensile contribution.

4.7 Maximum bending moment due to the applied load

The limiting bending moment in the deep beam has been determined using simple bending theory. However, the beam exists within a wall, and there are no directly comparable situations for which the maximum bending moments have been pre-determined.

The nearest cases are those for:

1. a beam with fully encastred ends, where the maximum bending moment is $wl^2/12$. This occurs at the supports, with the peak central bending moment being $wl^2/24$.

2. a propped cantilever, where the maximum bending moment is $wl^2/8$ and occurs at the fixed end. However, the only circumstances under which the propped cantilever case might apply to a deep beam would be where one support had failed, and left the end of the beam above it free to move horizontally, but not vertically. This is unlikely to happen in practice – the masonry surrounding the beam would restrain it, and it is much more likely that part of the foundation would fail, resulting in the deep beam being only partially supported.

Neither of these cases fully applies to the deep beams being considered here. However, the fully encastred case is more representative, as the beam is encastred at each end, although the degree of fixity will not be full. As a result, $wl^2/12$ has been used to define the limiting bending moment within the beam.

4.8 Horizontal shear resistance

Testing carried out at BRE suggests that an appropriate limiting horizontal shear stress for unreinforced masonry is around 0.070 N/mm^2. Clearly, it needs to be established that the in situ value is at least as high as this value. In addition, as the presence of a damp proof membrane (d.p.m.) between the chords could not be guaranteed to provide any shear resistance, a d.p.m. should not be present between the chords.

4.9 Vertical shear capacity

It has been assumed that the shear force applied to the beam is taken solely by the upper chord. This is a conservative option.

4.10 Deflection

The limiting span to depth ratio used in this design method is 10. This is a serviceability condition, and it has been derived by taking the figure from Table 10 of BS 5628-2:2000 for the case of a supported beam - 20 - and applying to it a safety factor of 2.

5 On site approach

The approach that it is intended should be taken on site is to ensure that the properties of the masonry materials in the building to be reinstated are unlikely to be lower than those used in the test specimens. Specifically, proof loading should be used to show that the horizontal shear strength of the masonry is in excess of 0.070 N/mm^2 and to establish that the compressive strength of the bricks is greater than 8.5 N/mm^2. In typical masonry, the horizontal loads to be applied to reach a horizontal shear strength of 0.070 N/mm^2 are 3070 N and 4050 N, for 102mm and 215mm thick brickwork, respectively. Note: the value for the 215mm thick brickwork assumes that the vertical joint through the thickness of the wall is filled, and that just one single stretcher bond brick is tested. As a result, this test is likely to prove to be a conservative test.

To reduce the risk of cracks forming in the masonry, the proof testing should be carried out in areas that are at least 500mm away from the edge of the building and in an area roughly equidistant between the likely position of the chords.

The performance of the system is dependent on the interaction between the WHO60 grout and the wires - the testing that underpins this design method used wires that were placed into the grout in a carefully controlled way and were fully embedded in that grout.

As a result, care must be taken on site to ensure that the wires:

1. are both straight - rather than kinked;
2. do not touch when they are placed in the grout; and
3. are fully encased in the grout.

6 Limitations on the applicability of this design method

This design method is intended to cover the use of the system in a building that either:

1. has localised cracking damage and where the intention is to use the system to reinstate the integrity of that part of the building - with the remainder of the masonry in the building being in a generally good state of repair; or
2. where the cracking is more widespread but not particularly severe in any location.

Note: in the context of this design method, localised cracking should be seen as cracking in a part of a building that is not severe enough to result in that part of the building being in a structurally unstable condition.

This design method is **not** intended to be used in circumstances where the system is used to reinstate a building that is on the point of collapse, or where the masonry in general is in a very poor condition - in this context, masonry not having the required horizontal shear strength would come under the heading of masonry being in a very poor condition.

In these cases – which would be deemed to be failures at the ultimate load – use of the system may still be appropriate, possibly as part of a series of remedial measures. However, additional engineering input would be needed under those circumstances and this design guide must **not** be used.

This design method is based on an analysis that requires the beam to span between supports. For that reason, before the system is used it should be shown that the supports at either end of the beams are stable - or have been stabilised.

7 Further considerations

1. As a prerequisite for the design method to be applicable, it is essential that the reinstatement of the building includes adding suitable material - such as mortar or grout - to ensure that the relevant portions of the building that have cracked are reinstated. This will ensure that compressive loads can be transmitted "immediately" throughout the area of the deep beams, without requiring any appreciable prior movement of that part of the structure to allow this.

2. The performance capacities and stresses quoted in this document were obtained from tests carried out on specimens made using the WHO60 grout, and are only applicable for use with this material. Further testing would need to be carried out to confirm that other materials met the performance requirements.

3. It should be ensured that the Thor Helical tie wires are installed with the wires running 400mm beyond the end of the deep beams to ensure that they are sufficiently well keyed into the surrounding masonry.

4. This design method provides the limiting load that may be applied to the portion of a building repaired using deep beams. However, it includes neither a partial safety factor for load – so this will need to be applied in the analysis – nor any suggestions as to the level of loading to be applied to the beams.

The tables provide limiting loads per metre run of beam covering:

a. the bending forces - carried by top and bottom chords;

b. the horizontal shear forces – the critical area being in the masonry beds between the chords

c. the vertical shear - assumed to be carried by the chord materials only

An overall limit for the span to depth ratio of the beam of 10 has also been applied.

In practice, where the span and the required load are known, the design tables should be used to select the required depth of beam needed.

5. The performance characteristics and stresses quoted in this document were obtained from the results of tests carried out on specimens made with 6mm helical tie wires. As a result, they are only applicable for use with this size and type of material - further testing would need to be carried out to confirm that other materials, that have neither the spring-like character nor the interlocking properties associated with the combination of the helical tie wires and grout or mortar, meet the performance requirements.

8 References

1. BS 5628-2:2000. Structural use of reinforced and prestressed masonry. BSI, London. 2000.

Table 1: Span table for use with 102mm thick brickwork

102 mm thick wall

	Span (m)	Depth of beam (m) 0.15	0.30	0.45	0.60	0.75	0.90
Bending	**0.80**	43.6	87.2	130.8	174.4	218.0	261.6
Horizontal shear		**28.4**	56.9	85.3	113.7	142.1	170.6
Vertical shear		32.8	**32.8**	**32.8**	**32.8**	**32.8**	**32.8**
Span/depth ratio		5.3	2.7	1.8	1.3	1.1	0.9
Bending	**1.20**	19.4	38.8	58.1	77.5	96.9	116.3
Horizontal shear		**19.0**	37.9	56.9	75.8	94.8	113.7
Vertical shear		21.8	**21.8**	**21.8**	**21.8**	**21.8**	**21.8**
Span/depth ratio		8.0	4.0	2.7	2.0	1.6	1.3
Bending	**1.60**		21.8	32.7	43.6	54.5	65.4
Horizontal shear			28.4	42.6	56.9	71.1	85.3
Vertical shear			**16.4**	**16.4**	**16.4**	**16.4**	**16.4**
Span/depth ratio		>10	5.3	3.6	2.7	2.1	1.8
Bending	**2.00**		14.0	20.9	27.9	34.9	41.9
Horizontal shear			22.7	34.1	45.5	56.9	68.2
Vertical shear			**13.1**	**13.1**	**13.1**	**13.1**	**13.1**
Span/depth ratio		>10	6.7	4.4	3.3	2.7	2.2
Bending	**2.40**		**9.7**	14.5	19.4	24.2	29.1
Horizontal shear			19.0	28.4	37.9	47.4	56.9
Vertical shear			10.9	**10.9**	**10.9**	**10.9**	**10.9**
Span/depth ratio		>10	8.0	5.3	4.0	3.2	2.7
Bending	**2.80**		**7.1**	10.7	14.2	17.8	21.4
Horizontal shear			16.2	24.4	32.5	40.6	48.7
Vertical shear			9.4	**9.4**	**9.4**	**9.4**	**9.4**
Span/depth ratio		>10	9.3	6.2	4.7	3.7	3.1
Bending	**3.20**			**8.2**	10.9	13.6	16.3
Horizontal shear				21.3	28.4	35.5	42.6
Vertical shear				**8.2**	**8.2**	**8.2**	**8.2**
Span/depth ratio		>10	>10	7.1	5.3	4.3	3.6
Bending	**3.60**			**6.5**	8.6	10.8	12.9
Horizontal shear				19.0	25.3	31.6	37.9
Vertical shear				7.3	**7.3**	**7.3**	**7.3**
Span/depth ratio		>10	>10	8.0	6.0	4.8	4.0

Note: the values shown in the table above are in kN/m run of beam

Table 2: Span table for use with 215mm thick brickwork

215 mm thick wall		Depth of beam (m)					
	Span (m)	0.15	0.30	0.45	0.60	0.75	0.90
Bending	**0.80**	43.6	87.2	130.8	174.4	218.0	261.6
Horizontal shear		60.0	120.0	180.0	240.0	300.0	360.0
Vertical shear		**32.8**	**32.8**	**32.8**	**32.8**	**32.8**	**32.8**
Span/depth ratio		5.3	2.7	1.8	1.3	1.1	0.9
Bending	**1.20**	**19.4**	38.8	58.1	77.5	96.9	116.3
Horizontal shear		40.0	80.0	120.0	160.0	200.0	240.0
Vertical shear		21.8	**21.8**	**21.8**	**21.8**	**21.8**	**21.8**
Span/depth ratio		8.0	4.0	2.7	2.0	1.6	1.3
Bending	**1.60**		21.8	32.7	43.6	54.5	65.4
Horizontal shear			60.0	90.0	120.0	150.0	180.0
Vertical shear			**16.4**	**16.4**	**16.4**	**16.4**	**16.4**
Span/depth ratio		>10	5.3	3.6	2.7	2.1	1.8
Bending	**2.00**		14.0	20.9	27.9	34.9	41.9
Horizontal shear			48.0	72.0	96.0	120.0	144.0
Vertical shear			**13.1**	**13.1**	**13.1**	**13.1**	**13.1**
Span/depth ratio		>10	6.7	4.4	3.3	2.7	2.2
Bending	**2.40**		**9.7**	14.5	19.4	24.2	29.1
Horizontal shear			40.0	60.0	80.0	100.0	120.0
Vertical shear			10.9	**10.9**	**10.9**	**10.9**	**10.9**
Span/depth ratio		>10	8.0	5.3	4.0	3.2	2.7
Bending	**2.80**		**7.1**	10.7	14.2	17.8	21.4
Horizontal shear			34.3	51.4	68.6	85.7	102.9
Vertical shear			9.4	**9.4**	**9.4**	**9.4**	**9.4**
Span/depth ratio		>10	9.3	6.2	4.7	3.7	3.1
Bending	**3.20**			**8.2**	10.9	13.6	16.3
Horizontal shear				45.0	60.0	75.0	90.0
Vertical shear				**8.2**	**8.2**	**8.2**	**8.2**
Span/depth ratio		>10	>10	7.1	5.3	4.3	3.6
Bending	**3.60**			**6.5**	8.6	10.8	12.9
Horizontal shear				40.0	53.3	66.7	80.0
Vertical shear				7.3	**7.3**	**7.3**	**7.3**
Span/depth ratio		>10	>10	8.0	6.0	4.8	4.0

Note: the values shown in the table above are in kN/m run of beam