Strengthening and rehabilitation of masonry using fibre reinforced polymers

Nigel G. Shrive, Mark J. Masia and Shelley L. Lissel
University of Calgary, Department of Civil Engineering, Calgary, Alberta, Canada

ABSTRACT: Fibre reinforced polymers (FRP's) have light weight, excellent durability, and high strength – the latter particularly when the fibres are oriented unidirectionally within the composite. These properties make these materials attractive for use in strengthening and rehabilitation of structures. In Canada, FRP's have been used in various applications in both new construction and rehabilitation. We describe those research projects associated with masonry – post-tensioning with Carbon FRP tendons to provide improved serviceability and reduce crack sizes in damaged structures: wrapping damaged columns with CFRP sheets to restore/increase strength: and lastly using Glass FRP sheets to increase the flexural capacity and energy absorption characteristics of plain and reinforced concrete blockwork. The former two projects were performed at Calgary and the latter at the University of Alberta, Edmonton. All projects show considerable promise for their respective applications. FRP's therefore will provide an additional option for structural engineers when considering these types of applications.

1 INTRODUCTION

Masonry structures may need strengthening for a variety of reasons. Creep within the structure may redistribute loads such that the masonry is carrying more load over time. This may occur from increasing deformations elsewhere in the structure or from redistribution of stress within a structural element itself. If load redistribution to masonry is combined with a reduction in strength from external environmental factors such as weathering, then increasing load with decreasing strength over time can lead to failure. This for example is the probable cause of the collapse of the Civic Tower of Pavia, where the outer masonry picked up load originally carried by the internal rubble filling of the sacco wall (Binda et al. 1992).

With forewarning – usually the appearance of cracks – masonry can be strengthened. One of the earliest methods of strengthening was to place a heated flat iron/steel bar across the damaged area and bolt it to solid material on either side. On cooling, the contraction of the bar would compress the damaged masonry, placing the bar in tension but leaving residual strength to resist any increase in load.

This early post-tensioning has been made more sophisticated over the years. The principle has been retained, although the techniques of achieving the compression have changed. Hanlon (1970) for example ran steel tendons through hose pipes when rehabilitating churches in New Zealand. The VSL method also uses greased ducts but has self-activated anchorages (Ganz 1993b, VSL 1990). In both these methods great care was taken to provide corrosion protection to the steel tendons. Wrapping the steel tendons with water repellant paste and tape has been used (Curtin et al. 1982). The cost of these various techniques for protecting the steel tendons has led to the consideration and use of other options. For example, stainless steel rods have been used (e.g. Curtin and Howard 1991). An advantage of fibre reinforced polymers (FRP's) is their high durability in moist environments. The materials do need to be protected from ultraviolet light which causes enbrittlement of most of the polymer matrices currently in use. The FRP's therefore need to be completely hidden inside a masonry assemblage, or coated with paint. Improved resins are being developed such that even this concern may be alleviated over the next few years. The fibres of interest are Carbon (CFRP) and Glass (GFRP). The materials are produced either in the form of bars, with the fibres parallel to the longitudinal axis of the bar, or sheets. In the latter, there can be a predominance of fibre orientation in one direction if that is desirable for the project at hand. GFRP can also be moulded in the form of a mesh. The latter can be used to replace at least the upper steel mat in a bridge deck (Benmokrane et al. 2000).

In Canada there have been three major thrusts investigating the use of FRP's in masonry over the last few years. These are the use of CFRP tendons to post-tension masonry walls, the strengthening of damaged columns by wrapping with CFRP sheets, and the strengthening of concrete block walls by applying sheets and/or strips on one or both sides of the wall. Each of these projects is described briefly.

2 POST-TENSIONING WITH CFRP TENDONS

Post-tensioning has been applied successfully to a variety of masonry structural forms (Curtin et al. 1982, Sinha 1984, Allen 1986, Ganz 1993a, Ganz 1993b). The advent of advanced composite materials provided an alternative to the corrosion protection measures adopted previously. In particular, Fibre Reinforced Polymers (FRP's) have properties that are attractive for post-tensioning applications. These materials are typically of light weight and high strength. CFRP tendons for example, where the fibres are aligned in the longitudinal direction of the tendon, have strengths in the range of 2000 – 2500 MPa. GFRP is not as strong (1100 – 1300 MPa), whereas Aramid FRP's (AFRP) have a wide range of strengths depending on the manufacturer (Sayed Ahmed and Shrive 1998). The properties of these materials have been published elsewhere (e.g. Benmokrane et al. 1997, Daniel and Ishai 1994).

The glass in some GFRP's is sensitive to alkaline solutions and AFRP's are prone to creep. Despite this latter feature, an AFRP post-tensioned masonry foot bridge was designed and constructed in the U.K. (Shaw and Baldwin 1995). We chose to use CFRP because of the high strength and durability. CFRP tendons have a propensity to rupture under shear or lateral loading. Thus the anchorages used for steel tendons cannot be used on CFRP tendons. The sharp ridges on the wedges of a standard anchorage, that is designed to dig into and grip the steel tendon, cause a carbon fibre tendon to shatter in the anchorage.

The different techniques used to grip FRP tendons were assessed and a new anchorage developed which is larger than the standard steel anchorage. When used with CFRP, the requirements for an anchorage established by the PTI are passed (Sayed Ahmed and Shrive 1998). However, there is some inconsistency in use on the part of others and the anchorage needs to be made more robust in terms of its performance before site use can be recommended (Campbell et al. 2000). This anchorage is made of stainless steel and has a copper sleeve that sits over the tendon where it is to be anchored, to help relieve stress concentrations caused by the wedges.

To avoid the use of metals completely, a concrete anchorage was developed (Campbell et al. 2000). In initial tests, the concrete wedges tended to crack as load was applied. In addition, the anchorage barrel is subject to circumferential tension when a tendon is loaded. Hence, substantial effort was directed at developing an ultra high performance concrete for the wedges and barrel. The material had compressive strength in excess of 200 MPa at 7 days, and high toughness, provided in part by the addition of micro fibres of carbon, 3 mm long. The barrel was also reinforced by wrapping it with carbon fibre sheets cut to the precise length of the circumference. The two ends of the sheet were butted up one against the other rather than overlapped. Interestingly, overlapping was found to cause premature delamination and ripping of the wrap, initiated at the small void that must occur as the carbon fibre sheet is raised to start overlapping the underlying end. Four layers were used, with the butt joints spread around the circumference at 90º intervals to avoid creating a weak zone.

With this anchorage there is no soft sleeve over the tendon: the wedges grip the tendon directly. The disadvantage of the anchorage at the moment is its size – 180 mm long with a diameter of 120 mm. Further developments are required to reduce these dimensions, but the opportunity and potential for a completely metal free post-tensioning system clearly exist.

Whilst developing this post-tensioning system, tests were also performed on diaphragm walls with unbonded post-tensioning tendons (Fig. 1). Flexural, racking and thermal tests have been performed.

2.1 Experimental work

The tests were performed on 3 m high diaphragm walls with the cross-section shown in Fig. 1. The walls were constructed of clay masonry and prestressed to a relatively low compression of 0.65 MPa. Even this low stress level will provide a significant increase in the cracking moment of resistance compared to a plain diaphragm wall – a multiplication in the order of 3 or 4. An even bigger increase is obtained relative to a cavity wall (no cross-webs) – in the order of 75- to 100-fold.

Under high flexural load, the wall will crack along a bed joint (Fig. 2) (Sayed Ahmed et al. 1999). Tests are not taken to failure in these circumstances, but unloaded, and the wall subsequently dismantled. Cracks close up completely on unloading.

In the racking tests, again a considerable increase in capacity is observed. When the load was applied at the top of the wall, the usual mode of failure was a tensile crack at the heel and signs of crushing at the toe. Again cracks closed up on unloading. When load was applied much lower down the wall, to cause “shear cracking”, unloading often left residual spaces in perpend joints. The post-tensioning did not bring the wall fully back laterally, but would close up horizontal cracks in the bed joints.

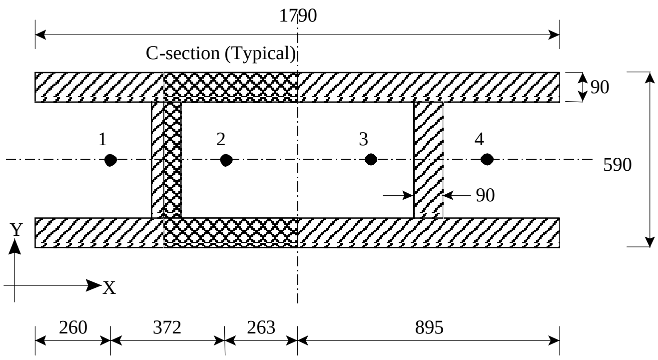

Figure 1: Cross-section dimensions of the diaphragm wall. Measurements in millimeters (Sayed Ahmed et al. 1999)

Thermal tests revealed that tendon forces changed in the opposite direction to those measured with steel tendons (Sayed Ahmed et al. 1999). This is because the coefficient of thermal expansion of CFRP tendons is less than that of masonry (actually close to zero, sometimes positive, sometimes negative). Thus on cooling a wall, the masonry will contract more than the tendon, reducing tendon load. In contrast, the coefficient of thermal expansion of steel is larger than that of masonry, so cooling a steel post-tensioned wall will increase the load in a steel tendon. This is one major difference between post-tensioning with CFRP and steel.

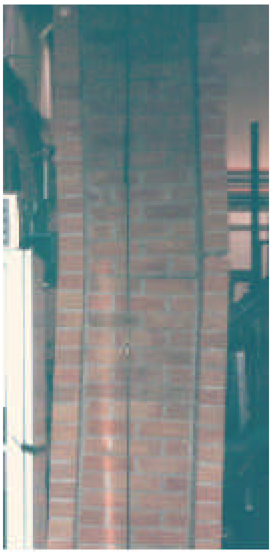

Figure 2: Cracking at "failure" in a flexure test (Sayed Ahmed et al. 1999)

The experiments have been used to develop a set of design recommendations for such diaphragm walls. However, when used to design a retaining wall, it was found that the recommendations placed a severe limitation on the estimated shear strength of the wall. Resolution could have been achieved by increasing the thickness of the webs, but the requirement was excessive. Hence, recent work has been oriented at understanding the shear strength of masonry. The weak points are the web/flange interface, and the permissible in-plane shear in the web.

Results (Lissel et al. 2000) suggest that bed-reinforcement has little effect on shear strength, but can affect post peak behaviour. The problem with reinforcement in general is that when the "shear crack" develops, the reinforcement de-bonds and the strength of the reinforcement is not activated. The usual single, wide crack crossing the bed joint and associated reinforcement is very different to the multiple narrower cracks typically seen in concrete. The most recent tests performed with GFRP ties manufactured to our design (Lissel and Shrive 2001) indicate that with proper anchorage in the mortar, the strength of a tie can be activated. This avenue is being explored further.

3 REHABILITATING MASONRY COLUMNS USING CFRP WRAPS

The use of CFRP wrapping to strengthen existing masonry columns was investigated experimentally. The study was aimed at quantifying the increase in strength that can be achieved and assessing the effect of column size on the strength increase. 18 columns were tested, of three different cross sectional sizes and two different types of masonry unit. Strengthening was achieved by wrapping the square section columns directly with CFRP sheets, or by wrapping the columns after first casting a circular concrete jacket around the column. Significant strength increases were achieved, particularly in the latter case. The study focused on the rehabilitation of damaged columns but the results obtained may also be applicable to strengthening of undamaged columns. These preliminary tests indicate that the use of CFRP wrapping is effective as a technique for rehabilitating damaged masonry columns.

3.1 Experimental program

Each column was initially loaded axially until cracking was observed in the masonry. The columns were then wrapped with CFRP sheets over their full height and retested under axial compression until failure occurred.

Column Construction - The columns were constructed in three different cross sectional sizes (290 mm x 290 mm, 390 x 390, and 490 x 490), using two different types of masonry unit (Figs. 3(a), 3(b)). All columns were the same overall height of 1.2 metres. The cavity formed at the centre of the column in each case was filled with grout. No reinforcement was placed in the cavity. The columns were cured for a minimum of 28 days prior to the first test.

Testing of Unwrapped Columns - Each of the 18 columns was loaded axially in compression until cracking was first observed. The aim was to damage the columns to a stage that would be considered in need of repair but not replacement. The columns were instrumented with displacement transducers, one at each corner, over a gauge length approximately one third of the column height and centred about midheight. The use of four transducers allowed checking that the loading was concentric. The vertical deflection at each transducer, as well as, the axial load were recorded at one second intervals during testing. The four transducers readings were averaged to allow plotting of column load versus axial deflection.

The plot of load versus deflection was used to identify a reduction in column stiffness. This coincided with the appearance of first cracking and enabled the loading to be stopped before excessive column damage occurred. The load recorded to cause first cracking provides an approximate measure of the strength of the column prior to CFRP wrapping for later comparison with the wrapped strength.

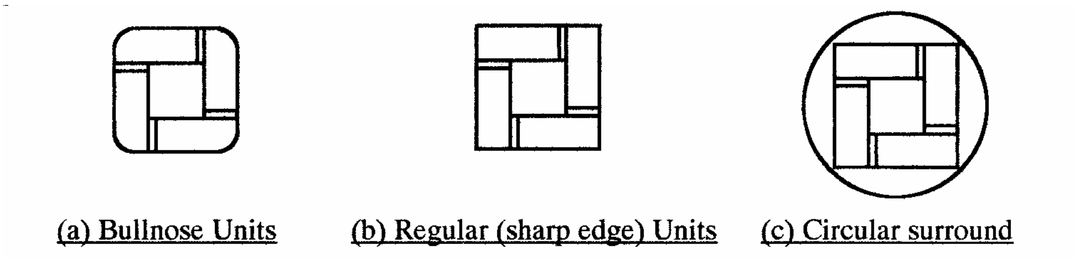

Figure 3: Column details

CFRP Wrapping -The columns were transported to be wrapped with CFRP sheets to industry standards by a coating contractor. Some of the columns required treatment before transportion:

- The columns constructed using bullnose units required no treatment.
- All but two of the columns constructed using sharp edged units at the corners were saw cut along each edge to produce a chamfer of 25mm. The idea behind this was to approximate the effect of the bullnose thereby distributing the confining pressure provided by the wrap over a larger area at the corners of the column and also helping to avoid cutting the CFRP at the sharp column edge. This treatment was applied to all of the intermediate and large columns, and 2 of the small columns constructed from the second type of units (sharp edges).
- The remaining 2 small sized columns and 2 of the intermediate sized columns constructed from the second type of units (sharp edges) were surrounded with cylindrical cardboard column formwork. Concrete was then cast to form a concrete surround to the masonry column (Fig. 3(c)). The cylindrical shape utilises the confining pressure provided by the CFRP wrap around the complete perimeter of the column. By contrast, the square column cross sections experience confining forces in the vicinity of the column corners only.

Following the various preparations all of the columns were sandblasted, coated with epoxy primer and bonding resin, then wrapped with the CFRP sheets over the full column height. The CFRP sheets were in the form of a single layer of unidirectional reinforcement placed with the strong direction circumferentially. The design philosophy relies on the wrap to carry tensile forces around the perimeter of the columns as a result of lateral expansion of the underlying column under axial compressive load. Constraining the lateral expansion of the column confines the masonry and thereby increases its compressive capacity. It should be emphasised that passive confinement of this type requires significant lateral expansion of the masonry / concrete before the CFRP is loaded and confinement becomes active. Experimental results reported by several authors for CFRP wrapped concrete columns indicate that tensile strains in the CFRP sheets are negligible prior to reaching 60 - 70% of the ultimate column strength (Purba and Mufti (1999)).

Testing of CFRP Wrapped Columns - The wrapped columns were each loaded axially until ultimate failure occurred. The same instrumentation was used as for the initial unwrapped column tests. The ultimate load and failure mode were recorded.

3.2 Results and discussion

Unwrapped Columns - The columns were each loaded axially in compression as described above. In each case vertical cracks, extending alternately through the perpend joints and the units, developed in the masonry on all four faces of the square section columns. The loads to cause cracking, summarised by column size, are shown in Table 1.

Plots of Axial Load versus Deflection for the unwrapped columns (Fig. 4a) indicated that the response is essentially linear up until cracking. There was only slight loss of stiffness with loading. The displacement recovery upon unloading was of the order of 80 - 90%.

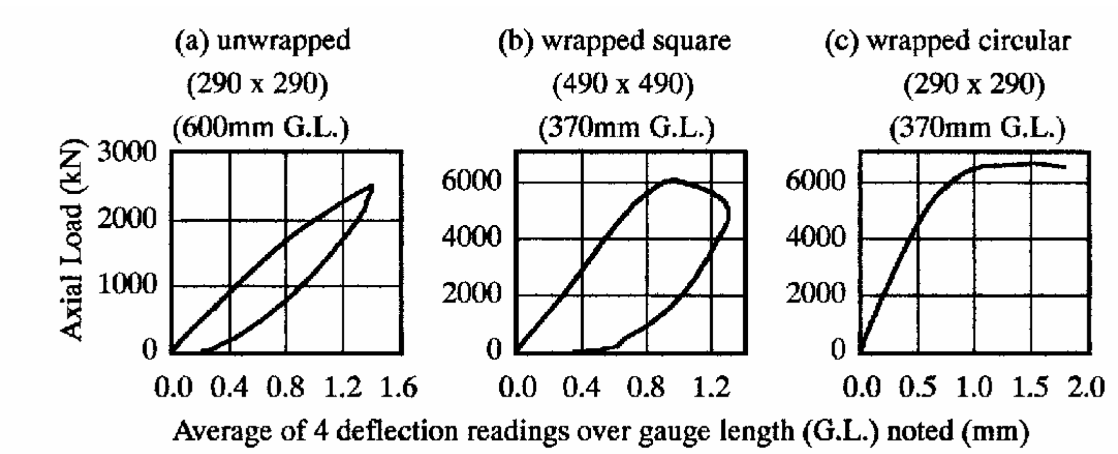

Figure 4: Typical load versus deflection plots (Masia et al. 2001)

CFRP Wrapped Columns - After wrapping with CFRP, thirteen columns of square cross section and four columns of circular cross section were tested. One of the original 18 columns failed completely in the unwrapped test.

The square section columns were each loaded past peak load until a significant decrease in the load carrying capacity was observed. In all cases the CFRP wrap remained essentially intact. In only 2 of the columns, small tears (5 - 10mm in length) in the wrap were observed at the column corners.

During loading, and particularly as the peak load was approached, significant cracking of the masonry and grout could clearly be heard together with the sound of the CFRP wrap delaminating from the flat column sides. In the failed state the square section columns typically displayed horizontal folds in the CFRP wrap coinciding with underlying crushed mortar joints and visible bulging out of the masonry beneath the wrap.

The peak loads recorded for the square section columns, averaged for each column size, are shown in Table 1. Also shown are the load increases (%) from the cracking load prior to CFRP wrapping to the peak load for the wrapped columns. The percentage increase for individual columns ranged between 14 and 73% with a mean increase of 34%. Prior to testing it was expected that the load increase achieved would decrease with increasing column cross section. This hypothesis was based on the idea that the smaller square cross section would have the same area of confinement at the corners as a larger cross section but a shorter length of unconfined material between column corners (and thus a larger proportion of confined material). The results indicate that this is not the case with the greatest load increases observed for the intermediate sized columns and the smallest for the small sized columns. A greater number of tests may be necessary to make further conclusions in this regard.

A typical load versus deflection plot for the CFRP wrapped square section columns is shown in Fig. 4b.

The circular section columns were each loaded until failure occurred. In each case there were very few visible or audible signs of distress prior to a very sudden and explosive failure. Plots of load versus deflection (Fig. 4c) did however provide warning of the imminent failure in each case. After an initially linear response, the column stiffness gradually reduced, followed by extended deformation at close to peak load before failure occurred. In each case the failure load was either equal to the peak load or only marginally less than the peak.

The increase in load capacity from the unwrapped cracking loads to the failure loads for the modified circular and CFRP wrapped columns averaged 200% and 156% for the small and intermediate sized columns respectively. Despite the additional column area, these increases clearly highlight the effectiveness of the CFRP wrap when provided with a circular cross section to confine.

Table 1: Test Results – Summary

Section size (mm x mm)	Average Unwrapped Cracking Load (MN)	Average Wrapped Strength (MN)	Load Increase (%)
Square			
290 x 290	2.1	2.6	24
390 x 390	3.3	4.7	42
490 x 490	4.2	5.7	36
Circular			
290 x 290 (dia 457)	2.1	6.3	200
390 x 390 (dia 559)	3.4	8.7	156

In the cases where the original square section column was CFRP wrapped, the average load increase was in the order of 34%. If a circular concrete jacket is provided prior to wrapping, load increases averaging 178% were observed due to confinement by the CFRP wrap being effective around the full perimeter of the circular cross section.

For the rehabilitated columns, the masonry/grout core must experience considerable damage and associated lateral expansion before the CFRP wrap takes up any significant load. The CFRP wrap is thus of most benefit in the scenario of column overload.

The test results presented do not enable any decisive conclusions to be made regarding the effect of column size (cross sectional area) on the strength increase achieved.

These preliminary tests indicate that the use of CFRP wrapping is effective as a technique for rehabilitating damaged masonry columns.

4 STRENGTHENING OF CONCRETE BLOCK WALLS

This work was performed at the University of Alberta. In the first series of tests, FRP strips and sheets were applied to one side of hollow, unreinforced concrete block walls that were then tested in flexure. The objectives were to examine the lateral load resistance and behavioural characteristics of this form of construction. The variables assessed were:

- Type of FRP (glass fibre sheets, carbon fibre sheets, carbon fibre strips)
- Amount of fibres per unit area of wall
- Layout of the fibres
- The effects of axial and cyclic lateral loads

The patterns of FRP application are shown in Fig. 5. The results showed tremendous increases in flexural strength and strain capacity over the plain walls (Albert et al. 1998). Under cyclic loading, the envelope curve was very close to the monotonic loading response. The most cost effective gains in strength occurred with GFRP sheets. Hence these were used in a second series of tests where reinforced concrete block walls were given additional strength through the application of GFRP sheets on both sides of the wall. Again, the effect of varying the amount of GFRP used per unit length of wall was assessed, as was the effect of the GFRP with respect to differing amounts of initial steel reinforcement.

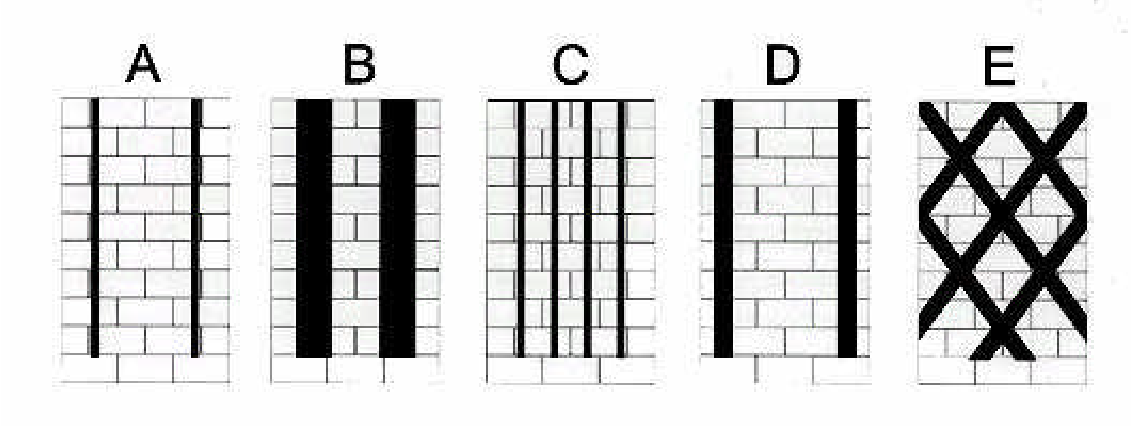

Figure 5: Patterns of FRP Application (Albert et al. 1998)

Reverse cyclic flexural tests were performed on the second series of walls. Four point loading was used, giving two shear zones and a constant moment section over the middle third of each wall. Considerable increases in hysteresis energy were observed together with increases in strength (Kuzik et al. 2001a, Fig. 6). The three different modes of failure observed in the initial series of tests occurred again (fibre rupture due to tension from flexure, flexural shear failure of the blockwork in one of the shear zones and sliding shear failure in a mortar joint). A method of analysis was developed to predict the load-deflection behaviour of the walls accounting for these different modes of failure (Kuzik et al. 2001b). The technique therefore shows tremendous promise for the strengthening of walls in seismic zones.

Although the aesthetic look of the masonry is lost with this technique, as with the column strengthening, the sheets can be covered with a façade to provide the architectural perspective desired and protect the FRP from ultraviolet radiation.

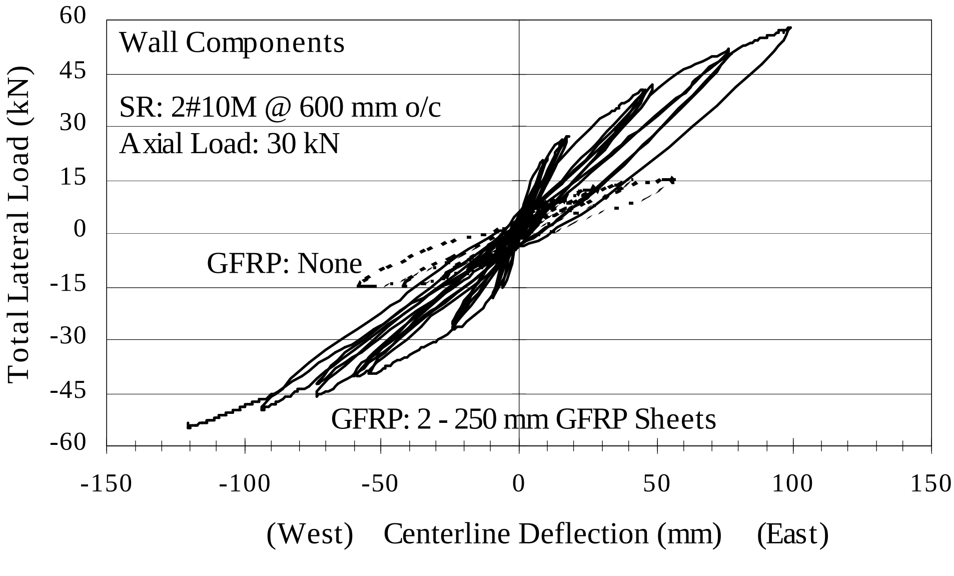

Figure 6: Load versus with and without GFRP (Kuzik et al. 2001a)

5 CONCLUSIONS

The work performed to date shows that there are distinct possibilities for the effective use of FRP's in a variety of applications in structural masonry. If protection against ultraviolet light can be provided, durable long-term solutions can be created for structures requiring light weight strengthening, improved serviceability or specialized structural forms.

Post-tensioning can be used to close or control cracking in damaged structures or to increase the cracking moment of resistance in new construction. The evolution of an all-concrete-and-FRP anchor has the promise and potential of a completely metal free post-tensioning system that would avoid the issue of corrosion completely.

Wrapping of columns with FRP or concrete-and-FRP jackets restores or increases the strength of damaged masonry columns. While this technique is not appropriate for historic structures directly, reconstruction methods might be developed where such strengthening can be faced with the original masonry. In other structures, an exterior façade can be built around the strengthened column if needed. As strengthening here did not follow the expected pattern with respect to column size, further work is needed to elucidate this issue.

There are many plain, partially reinforced and fully reinforced concrete block walls. The simple techniques developed show that considerable improvements in flexural strength and energy absorption characteristics can be obtained through using GFRP sheets applied to blockwork walls. Rehabilitation of partially damaged walls, or straight strengthening of such walls in seismic zones can therefore be achieved very cost effectively.

FRP's will therefore provide structural engineers with another option when considering design of new, or rehabilitation of old masonry. Further work is required to develop full design guidelines, but innovative structures are now being built following specific research programs.

ACKNOWLEDGEMENTS

We gratefully acknowledge the support the ISIS (Intelligent Sensing for Innovative Structures) Network of Centres of Excellence that has supported financially all of the work reported. Substantial help was received from technical staff at both universities in support of the experimental work.

REFERENCES

Albert, M.L., Cheng, J.J.R., Elwi, A.E. 1998. Rehabilitation of unreinforced masonry walls with externally applied fibre reinforced polymers , Structural Engineering Report, No 226, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, T6G 2G7.

Allen, L.N. 1986. Post-tensioned brickwork at rushden fire station, Engineers File Note #1, Brick Development Association, UK, 6pp.

Benmokrane, B., Xu, H., and Mishizaki, I. 1997. Aramid and carbon fibre reinforced plastic prestressed ground anchors and their field applications, Canadian Journal of Civil Engineering, 24(6), p. 968-985.

Benmokrane, B., Rahman, H., Mukhopadhyaya, P., Masmoudi, R., Chekired, M., Nicole, J.F., and El-Safty, A. 2000. Use of fibre reinforced polymer reinforcement integrated with fibre optic sensors for concrete bridge deck slab construction, Canadian Journal of Civil Engineering, 27(5), p. 928-940.

Binda, L., Gatti, G., Mangano, G., Poggi, C. and Sacchi Landriani, G. 1992. The collapse of the Civic Tower of Pavia: A survey of the materials and structure, Masonry International, 6(1), p. 11-20.

Campbell, T.I., Shrive, N.G., Soudki, K.A., Al-Mayah, A., Keatley, J.P. and Reda, M.M. 2000. Design and evaluation of a wedge-type anchor for fibre reinforced polymer tendons, Canadian Journal of Civil Engineering, 27(5), p. 985-992.

Curtin, W.G., Bradshaw, R.E., Brown, R.G.D., Parkinson, J., and Tovey, A.K., (Editors) 1982. Reinforced and prestressed masonry, Thomas Telford Ltd., 163 pp.

Curtin, W.G., and Howard, J. 1991. Research on prestressed cantilever diaphragm walls, The Structural Engineer, 69(6), p.105.

Daniel, I.M., and Ishai, O. 1994. Engineering Mechanics of Composite Materials, Oxford University Press.
Ganz, H.R. 1993a. Strengthening of masonry structures with post-tensioning, Proc., Sixth North Am. Mas. Conf., Philadelphia, p. 645-655.
Ganz, H.R. 1993b. Recent experience with post-tensioned masonry in Switzerland, Proc., Sixth North Am. Mas. Conf., Philadelphia, p. 657-667.
Hanlon, J.R.G. 1970. Prestressed Concrete Masonry, Concrete, 4(9), p. 356-358.
Kuzik, M.D., Elwi, A.E., and Cheng, J.J.R. 2001a. Cyclic flexural tests of masonry walls reinforced with GFRP sheets, Proceedings, 9th Canadian Masonry Symposium, Fredericton, New Brunswick. On CD 14 pp.
Kuzik, M.D., Elwi, A.E., and Cheng, J.J.R. 2001b. Numerical model for flexure response of masonry walls reinforced with GFRP sheets, Proceedings, 9th Canadian Masonry Symposium, Fredericton, New Brunswick. On CD 12 pp.
Lissel, S.L., and Shrive, N.G. 2001. Glass fibre reinforced polymer (GFRP) shear connectors for masonry, Proceedings, 9th Canadian Masonry Symposium, Fredericton, New Brunswick. On CD 11 pp.
Lissel, S.L., Shrive, N.G., and Page, A.W. 2000. Shear in plain, bed joint reinforced and post-tensioned masonry, Canadian Journal of Civil Engineering, 27(5), p. 1021-1030.
Masia, M.J., Shrive, N.G., and Tilleman, D. 2001. Rehabilitation of masonry columns using carbon fibre wraps, Proceedings, 9th Canadian Masonry Symposium, Fredericton, New Brunswick. On CD 12 pp.
Purba, B.K., and Mufti, A.A. 1999. Investigation of the behaviour of circular concrete columns reinforced with carbon fibre reinforced polymer (CFRP) jackets", Canadian Journal of Civil Engineering, 26: 590-596.
Sayed Ahmed, E.Y., Lissel, S.L., Tadros, G., and Shrive, N.G. 1999. Carbon fibre reinforced polymer (CFRP) post-tensioned masonry diaphragm walls: prestressing behaviour, and design recommendations, Canadian Journal of Civil Engineering, 26(3), p. 324-344.
Sayed Ahmed, E.Y., and Shrive, N.G. 1998. A new steel anchorage system for post-tensioning applications using carbon fibre reinforced plastic tendons, Canadian Journal of Civil Engineering, 25(1), p. 113-127.
Shaw, G.R., and Baldwin, J. 1995. The construction of end-built prestressed masonry flat arch box girder footbridges, Masonry International, 9(1), p. 1-5.
Sinha, B.P. (Editor) 1984. Reinforced and Prestressed Masonry, University of Edinburgh, Department of Civil Engineering, 475 pp.
VSL 1990. Post-Tensioned Masonry Structures, VSL Report Series #2. VSL International ltd., Berne, Switzerland, 35 pp.