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# SEISMIC RETROFIT OF URM WALLS WITH FIBER COMPOSITES

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#### SUMMARY

The experimental results of seven URM walls retrofitted with E-glass composite fabric strips have been briefly discussed. The walls were tested under cyclic out-of-plane bending applied by an air-bag system. Two height to thickness ratios (14 and 28) were investigated. The walls were reinforced with various amounts of composite materials. The specimens were tested without the application of axial loads to eliminate the potential benefits of overburden pressure. The strength and ductility of the retrofitted walls was substantially improved. Field applications have demonstrated the feasibility of this technique.

#### **INTRODUCTION**

Masonry is among the oldest construction material. The mechanical properties of masonry are much more complicated than those of other construction materials. A large number of research projects conducted on masonry in the last twenty years have demonstrated the feasibility of this material for many applications. Most of the recent research has focused on improving the seismic safety of new buildings being constructed with masonry. Unreinforced masonry (URM) buildings constitute a significant part of the existing building inventory worldwide. URM buildings are vulnerable to lateral loads such as those caused by earthquakes. In the event of an earthquake, these lateral forces are transferred to the foundation through load bearing walls. These structural elements may be subjected to in-plane or out-of-plane loads. Failure in URM buildings due to out-of-plane loads is considered a main cause of personal injury and loss of life during earthquakes.

Recognising the shortcomings of URM buildings, there has been a surge of interest in recent years to develop techniques for improving seismic behavior of these structures. A number of techniques have been proposed and a complete overview of these approaches has been documented by Lizundia, et al. (1997). Among the new techniques for seismic retrofit of URM buildings is the bonding of thin sheets of fiber composites to the wall surface (Schwegler and Kelterborn 1996, Ehsani and Saadatmanesh 1997a). These sheets are typically provided in the form of rolls of fabric with different density and fiber orientation. The fabric is impregnated in the field with a resin matrix and is bonded to the wall surface. The light weight of the materials and the ease of installation usually result in significant cost savings compared to conventional retrofit techniques. Following the Northridge and Kobe earthquakes, the use of fiber composites for strengthening of columns, beams, and walls has increased significantly.

Composite materials consist of strong fibers such as carbon, glass and aramid bound together by a matrix. The matrix can be vinylester, polyester, or epoxy resin. Composite materials have been used for more than forty years in aerospace and other industries. The mechanical properties of composite materials depend on the fiber to matrix ratio. In composite materials, the fibers provide strength and stiffness to satisfy design requirements and the matrix provides load transfer among fibers, dimensional stability, and fiber support and protection. Composite materials have many advantages over conventional materials such as steel. Among them are high specific strength, high specific stiffness, corrosion resistance, high fatigue resistance, thermal stability, low cost and ease of installation. By selecting the appropriate fiber, matrix and geometry, composite materials can be tailored to satisfy a specific application. They also present some disadvantages such as low in-plane transverse (i.e. shear) strength, low interlaminar strength, linear elastic behavior up to failure, and potential sensitivity to moisture and UV radiation.

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The fibers have very high tensile strength, typically exceeding that of metals by several folds. For construction applications, usually more than 40% of the total volume of the composite products is the resin matrix whose contribution to the strength or stiffness of the composite is minimal. For the retrofit of masonry walls reported here, a unidirectional glass fabric weighing 18 ounces per square yard (610 g/m<sup>2</sup>) and a two component epoxy matrix were used. The hand lay-up technique used to apply the fabric to the wall resulted in a resin-rich composite. The glass fiber composite was tested in pure tension and its tensile strength was 369 N/mm (2106 lb/in) width of fabric.

#### EXPERIMENTAL STUDY

Seven half-scale unreinforced masonry walls were constructed with solid clay bricks and a low strength mortar. The specimens were divided into two sets; the short walls (710 mm (28 in.) high) and the slender walls (1420 mm (56 in.) high) with a height to thickness ratio of 14 and 28, respectively. All specimens were 1220 mm (48 in.) wide. Each wall was designated with a letter followed by one or two numbers. The letter "S" refers to single wythe and the letter "D" to double wythe. The designation for "S" series specimens is followed by two numbers that refer to the percentage of the composite reinforcement on the south and the north faces of the wall with respect to the balanced condition, respectively. A number indicating the reinforcement ratio for the north face follows the letter "D". The balanced reinforcement ratio was calculated similar to that for reinforced concrete members, assuming that masonry fails in compression at a strain of 0.003 at the same time that the composite fails in tension at a strain of 0.02.

All materials (sand, brick units, mortar and composites) and brickwork assemblages were characterised according to the appropriate ASTM standards. Thus, the compressive capacity of brick units was 45 MPa (6.52 ksi) and the mortar compressive strength was 5190 kPa (754 psi) at 28 days and 7200 kPa (1044 psi) at testing time. The half-scale brick units were assembled together with a quarter-inch wide mortar bed joint constructed of sand passing mesh No. 18. From prisms constructed with five bricks, the compression capacity of brickwork was determined as 20.0 MPa (2.90 ksi) at 28 days and 26.7 MPa (3.87 ksi) at testing time. The modulus of rupture was obtained from brick beam tests as 1482 kPa (215 psi) at 28 days and 1613 kPa (234 psi) at testing time.

The walls were tested in a steel reaction frame. The specimens were simple supported along the top and the bottom edges; the two vertical sides were free. With these boundary conditions, it was intended to reproduce a portion of the wall free of corner and joint interference. A roller condition was provided at the top, i.e. vertical displacements and rotations were allowed. The specimens were subjected to cyclic out-of-plane loading. The lateral pressure was applied through an airbag system which was moved from one face of the wall to the other; more details about the test set-up is presented elsewhere (Velazquez-Dimas, 1998). The load was applied to the walls in two stages: a load-controlled stage, which consisted of two pairs of cycles to observe the uncracked behavior, and a displacement-controlled stage, where the maximum displacement in each pair of cycles remained constant. The stiffness degradation was monitored using two loading cycles for the same displacement level. This procedure was continued until the failure of the wall was reached. In order to exclude the beneficial effect of overburden pressure, no axial load was applied to the specimens. An overall view of the testing frame is given in Fig. 1. The maximum deflection at mid-height of the west edge and the applied pressures were monitored in real time by two voltmeters and the corresponding load vs. deflection curves were plotted on the monitor of the data acquisition system. A number of clinometers and LVDTs and strain gages were also used to monitor the response of the specimens.

The observed behavior was fairly similar for all retrofitted walls. Initially, the specimens were subjected to several cycles of loading to investigate their pre-cracking behavior. Due to the presence of the composite materials, it was difficult to detect the bed joint cracks on the tension face of the walls. To overcome this, two or three thin strips of gypsum were pasted to the face of the wall. These cracks formed at different loads for the various specimens. However, the formation of the first visible bed joint crack, running the entire width of the wall occurred when the tensile strain in the composite strips reached 0.004. The formation of these bed joint cracks resulted in loss of stiffness in the walls for subsequent cycles of loading.

In addition to the bed joint cracks, the initiation of debonding of the composite materials from the brick surface was the next significant stage in the behavior of the specimens. This debonding was usually accompanied by a sound in the composite. This stage contributed greatly to the loss of stiffness for the wall. The gradual development of new delaminations were mapped on the composite strips as shown in Fig. 2. It is noted that in spite of the formation of these delaminations, the walls did carry a large number of load cycles before final failure of the wall was reached.





# Fig. 1. Test setup

Fig. 2. Delamination pattern for Specimen D100

The failure of the walls could be characterised by a number of modes as outlined in Table 1. The most common mode of failure was complete delamination of one or more of the composite strips, although in no case did this delamination occur in a symmetrical manner at both ends of a strip. Examples of specimens failed by this mode are S75/25, S20/40, and D100 that is shown in Fig. 2. Another mode of failure was the rupture of the composite strips in tension. While in coupon tests, the composite strips carried strains upto 0.02, tension failure in walls occurred when the strips had reached strain levels of only 0.01-0.013. This is attributed to the cyclic tension-compression nature of loading that the strips experienced. Consequently, it is recommended not to rely on the full strain capacity of the composites under these conditions.

The load-displacement hysteretic response for a symmetrically-strengthened wall (S100/100) is sown in Fig. 3. This wall was reinforced with three vertical composite strips, resulting in a balanced reinforcement ratio. The behavior of the wall was characterised by excessive cracking along the mortar joints all over the north and the south wall surfaces. The crack patterns on both faces seemed more or less uniformly distributed. Extensive delamination also took place on the composite strips on both faces. By the conclusion of the test, delaminated areas covered approximately 75% of the entire area of the strips. But, delaminated patterns were non-symmetrical for the two faces. On the north face where tensile failure occurred, the east strip delaminated fairly symmetrically above and below the middle brick course. However, the central and west strips delaminated from bottom, midway to the top support. The wall was subjected to 23 cycles of loading, supported a maximum lateral pressure of 11.7 kPa (249 psf) and deflected 58 mm (2.3 in.). The failure load was equivalent to nearly 13 times the weight of the wall and the maximum deflection was almost 4% of the span.

Tensile failure occurred on all three north composite strips. Prior to the fracture of the north strips, cracking sounds indicated the imminent failure. While the central and the west strips failed above the middle brick course, the east strip broke at mid-height. Although the balanced reinforcement ratio suggests simultaneous tension and compression failure of the wall, no compression failure of the brick was observed. The composite strips broke

along a jagged line rather than a straight line across the width of the strip. This may be attributed to the uneven wall surface that would create local stress concentration points at the edges of each brick. Due to the failure on the north face, the composite strips of the south face did not fail. A vertical crack passing through the brick was detected underneath the central strip at failure. This indicated that the flexural capacity of the brickwork parallel to the bed joint was reached. Therefore, a minimum amount of horizontal reinforcement needs to be provided to avoid such failure.

Noting the symmetrical behavior of the previous specimen, several walls were retrofitted with different amounts of reinforcement on the south and north faces. This would increase the amount of information obtained from each test. One such specimen is S75/25 whose load-displacement response is shown in Fig. 3. In order to determine the load carrying-capacity of plane brickwork, this wall was first tested as a Control Specimen (without any composite strips). Under these conditions, the wall failed at an applied pressure of 4.13 kPa (0.6 psi) by splitting in two pieces. The wall was later retrofitted and tested again as specimen S75/25. The south face was subjected to 21 cycles of loading while the north face supported 23 cycles. The behavior on both faces was characterised by a progressive bed joint cracking and gradual delamination of the composite strips as the applied pressure increased. Wider cracks were observed on the south face of the wall, where a 2-mm crack width was measured. Cracks spread for almost 75 % of the wall area on both faces. Essentially, linear elastic behavior was observed before a major crack development on both faces. Additional information about this specimen is given elsewhere (Ehsani, et al. 1999).



Fig. 3. Load vs. deflection for Specimens S100/100 and S75/25

Different mode failure occurred on the two faces. Twenty-one cycles were applied to the south face. Due to the severe damage caused by delamination on the three strips, the test was terminated. The top half of the west-strip was almost completely delaminated. The same situation was observed for the bottom half area of the central strip. The east strip was almost fully delaminated from the top to the bottom support. Delamination started at mid-height then spread above and below the middle brick course. No tensile failure on the composite strips was reached due to the fact that the load was stopped at pressure of 12.4 kPa (1.8 psi) at a maximum deflection of 16 mm (0.63 in.). This was done in order to continue the test for the north face.

Very stiff behavior was observed for the north face. The failure occurred by a sudden peeling of the three strips at the top support. On this face, delaminated areas were located mostly above the middle brick course. This failure took place at a maximum pressure of 31.03 kPa (4.5 psi) and for an ultimate deflection of 15.5 mm (0.61 in.). Peeling controlled the mode of failure for both faces, although a tensile failure of the fabric was expected due to the relatively low reinforcement ratios used. However, these peeling failures did not occur until the specimen had resisted significant load and displacement. In addition, the ratio of maximum supported lateral pressure for the north and the south face (PmaxN/PmaxS=2.5) was in agreement with that given by the corresponding reinforcement ratios, i.e. close enough to 3. However, the same pressure ratio was not observed for the first major crack and the first delaminated area on both faces of the specimen.

The envelopes for the peak load at each cycle for all specimens are shown in Fig. 4. The ultimate deflection for the short walls was about 15 mm and that for the slender walls was 80 mm; this demonstrates that the ultimate displacement capacity of the wall is proportional to the square of the height and not the amount of reinforcement.

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Table 1.

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D100	28	North	1.0	0.59	(15.0)	0.79	(20.0)	205	(9.8)	3.0	(20)	2.77	S	11.0	(279)	5.0	D	0.8	4.6
)/50	8	South	0.5	0.56	(14.0)	1.30	(33.0)	130	(6.2)	1.84	(47)	3.4	7	2.6	(99)	4.6	Ţ	1.3	7.4
S200	2:	North	2.0	0.40	(10.0)	0.86	(21.0)	432	(20.7)	2.75	(02)	5.0	23	10.6	(269)	5.0	D	1.1	5.7
/300	\$	South	3.0	0.56	(14.0)	1.0	(25.4)	475	(22.7)	1.3	(33)	2.3	24	16.0	(406)	2.6	S	0.3	3.4
S300/ 28	28	North	3.0	0.50	(12.5)	1.0	(25.)	350	(16.7)	1.0	(25)	2.0	17.5	16.0	(406)	1.8	NF	0.5	3.4
/100	8	South	1.0	0.50	(12.5)	1.0	(25.4)	245	(11.7)	2.3	(58)	4.0	13	5.3	(135)	4,6	D	0.9	6.0
S100	28	North	1.0	0.50	(12.5)	1.0	(25.4)	245	(11.7)	2.3	(58)	4.0	13	5.3	(135)	4.6	Т	1.0	6.0
/30	4	South	0.3	0.15	(3.8)	0.25	(6.3)	346	(16.6)	0.4	(10)	1.42	17	3.18 *	(81)	2.7	Т	1.0	2.0
S30	1	North	0.3	0.10	(2.5)	0.26	(6.6)	331	(15.5)	0.35	(6)	1.25	16	3.18 *	(81)	3.5	PD	0.7	2.0
/40	4	South	0.4	0.16	(4.2)	0.40	(6.7)	389	(18.6)	0.6	(5.2)	2.1	20	2.12	(52)	3.75	Ð	1.0	3.0
S20	1	North	0.2	0.10	(2.5)	0.26	(9.9)	216	(10.3)	0.5	(12.5)	1.8	11	1.06	(26)	5.0	a	1.2	3.0
/25	4	South	0.25	0.14	(3.5)	0.20	(2.0)	259	(12.4)	0.63	(16)	2.25	13	1.32	(34)	4,3	Ð	1.0	3.1
S75	1	North	0.75	0.15	(3.8)	0.30	(1.6)	648	(31)	0.6	(15.5)	2.5	33	4.0	(102)	4.0	Q	1.2	3.24
Specimen	h/t	Face	p/p,	Deflection at first bed	joint crack in. (mm)	Deflection at first	delamination in. (mm)	Pmax	psf (kPa)	δ <sub>max</sub>	in. (mm)	%Drift	P <sub>max</sub> / Wt	W in (mm)	иши) ли м	Fracture Ductility	Mode of Failure	E <sub>max</sub> (t) %	Rotation at Top (°)

Note: D = Delamination; PD = Partial Delamination; T = Tensile; S = Shear; NF = No Failure. \*This wall was retrofitted with an 18-oz Cross/ply E-Glass fabric For the short walls, this deflection is close to 2% and for the slender walls is about 4% of the wall height. The recommended service deflection for URM elements is 0.007h (Building Code Requirements 1996). For these walls, this limit corresponds to approximately the deflection where the first visible bed joint crack occurred. Therefore, it is recommended to limit the service deflection of retrofitted walls to the same limit of 0.007h; this will ensure sufficient reserve displacement capacity prior to failure of the wall.



Fig. 4. Load vs. deflection envelopes for the specimens

As can be seen from the data in Fig. 4, the presence of composite strips increased the failure load of the walls significantly. The specimens resisted loads, which ranged from 5 to 30 times the weight of the wall. As can be seen, for specimens with h/t = 14, the diplacements have been also reported in terms of the percentage of drift. However, in calculating the drift ratio for the slender specimens (h/t = 28), one must recognize that the height of the double-wythe wall (D100) was twice that of the remaining specimens. These large failure loads are considerably larger than the walls that a typical URM wall may experience during an earthquake. The figure also indicates that the ultimate load is proportional to the amount of reinforcement. The composite overlays transformed the brittle walls into very strong and ductile URM elements. Based on the analysis of the strain gage data and an examination of the behavior of the test specimens, it is recommended that the service loads be limited to that resulting in a longitudinal tensile strain of 0.004 in the composite strips.

#### FIELD APPLICATION

The strengthening technique discussed here is an effective approach to retrofit masonry buildings. The patented system has proven to be economical in many field applications (Ehsani and Saadatmanesh 1996, 1997a,b). The first reported field application was in a single story structure that was damaged during the Northridge earthquake (Ehsani and Saadatmanesh 1996). In this case, the damaged wall was located directly on the property line, making it impossible to strengthen the wall by shotcrete. Both the inside and outside faces of this wall were retrofitted with a single layer of glass fiber composites.

Another application where the authors have been involved with is the maintenance facility of the United Airlines in Oakland International Airport in California. Here, the concern about dust that would be created in a conventional retrofit led the owners to decide in favor of the use of composites. The retrofit was completed in a timely manner with no disruption to the ongoing activities with in the building.

A recent application in northern California is shown in Fig. 5. In this case, the property adjacent to the URM wall was being excavated, causing a concern regarding the stability of the wall. The composite system was used to strengthen the 18.3 m (60 ft.) long by 10 m (30 ft) high wall. The total thickness of the composite was about 3 mm and the project was completed in less than a week.



# Fig. 5. Retrofit of a URM building with glass fiber composites in northern California (Courtesy of External Reinforcement, Inc., Tucson, AZ)

#### CONCLUSIONS

The tests have demonstrated that the described technique utilising fiber composites is very effective in retrofitting URM walls. The ultimate load and deflection of the retrofitted specimens were enhanced by more than an order of magnitude compared to the unretrofitted URM walls. The mode of failure for most of the specimens included horizontal bed joint cracking at midheight of the wall, followed by delamination of the composite strips after a large number of loading cycles. Several field applications have demonstrated the feasibility of this technique.

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