MECHANICAL PROPERTIES OF EXTERNALY STRENGTHENED MASONRY

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SUMMARY:
Unreinforced masonry (URM) buildings constitute a significant part of the existing building inventory worldwide. Recent Earthquakes have shown that URM buildings are highly vulnerable to earthquakes. Retrofitting of these masonry buildings is a challenging and important issue in earthquake disaster mitigation. Many of these buildings will fail due to lack of ductility in in-plane mode. The present experimental study is performed on unreinforced brick masonry panels strengthened with Welded Wire Mesh and Micro concrete. This study is aimed to investigate the efficiency of a retrofit scheme to enhance shear capacity of brittle masonry. A series of six unreinforced masonry (URM) panels and twelve strengthened panels have been subjected to diagonal compression tests. Different reinforcement configurations are evaluated. The results show that unreinforced masonry (URM) specimens exhibited sudden brittle failure, whilst strengthened specimens failed in a more ductile fashion and show increase in shear strength.

Keywords: *Unreinforced Brick Masonry; Shear Strength; In-plane behavior; Retrofit*.

1. INTRODUCTION

Poor performance of URM buildings were observed in Bhuj (2001) (Jagadish et al. 2003), Kashmir (2005) (Rossetto and Peiris 2009) and Ankara (2007) (Adanur 2010) . URM walls have adequate strength when subject to in-plane forces, but the door and window openings in the walls are the sources of weakness. Openings result in reduction of the effective cross-sectional area of wall resisting lateral loads. The openings very close to the corners hamper the integral box action by weakening the joints. The piers between door and window openings are the most critical components resisting in-plane action. These are subjected to higher stresses than the portion of the wall above and below the openings. The primary actions in the piers are in-plane bending and shear. If the seismic shear force and bending moment exceeds the capacity of the piers, then it may lead to the failure of piers. Depending on the critical action, piers may fail in any failure modes such as diagonal cracking, shear sliding, toe crushing and rocking (Magenes and Calvi 1997). Masonry has pre-defined planes of weakness along the bed joints. In case of low-rise buildings with low normal stress on bed joints, sliding shear failure may occur as the masonry mobilizes the resistance against sliding failure through friction in the bed joints. In case of significant vertical load, the walls may fail in diagonal shear. The crack is follow the plane of principal stresses that exceed the in-plane tensile strength of masonry. This is the most general type of failure in unreinforced masonry walls characterized by X-shaped cracks occur in short piers due to cyclic shear. The failure mechanism of shear sliding and rocking are displacement based and dissipate more energy during earthquakes, whereas the brittle diagonal shear cracking should be avoided. External application of overlays such as Engineered Cementitious Composites (ECC) (Lin et al. 2010) and Steel Reinforced Grout (SRG) (Borri et al. 2011) were experimentally tested as retrofit solution for diagonal test on masonry Specimens. ECC is a type of strain-hardening cement composite that is directly sprayed onto URM walls. This composite is reinforced with synthetic fibers. The results showed that ECC is effective for in-plane retrofit. SRG is the product obtained by embedding high strength steel cords in cementitious matrix. SRG is tested for adhesion on samples for both parallel and perpendicular to the bond surface (Direct shear test and pull off tests) before implementation for retrofitting of in-plane behavior of masonry specimens. Results showed that there were a significant increase in shear strength of masonry panel but there was a need of in depth research about the size of the mesh and joining between panel faces. Some experimental studies had done by using steel bars as reinforcement for seismic strengthening of masonry panels. Canter core steel insertion(Abrams et al. 2007) and near surface mounted (NSM) (Ismail et al. 2011) are these techniques implemented. In NSM technique different orientation of steel bars are studied. The study concluded that there was an increase in strength up to 189% and vertical or grid reinforced schemes performed better. The helical profile of reinforcing bar establishes a good bond with masonry.

2. EXPERIMENTAL PROGRAM

A testing program was undertaken to investigate the in-plane performance of URM specimens and enhancement in shear capacity of strengthened specimens. Seismic strengthening of specimens was carried out using Welded Wire Mesh (WWM) with micro concrete. Six as-built and twelve strengthened specimens were tested under induced diagonal compression (in-plane shear). Different orientations of WMM reinforcement were used to retrofit the specimens. The failure modes, load versus drift behaviour and shear strength of tested specimens were determined.

2.1 Material Properties

The resulting strength of masonry panel depends on the properties of constituting materials like bricks, mortar and the WMM. Two types of masonry specimens were constructed to replicate Indian masonry buildings using new bricks of 225 mm long x 110 mm wide x 75 mm high size. In India for load bearing URM construction two - wythe wall is used while for the internal partition purpose one with a wall is used. For seismic strengthening of specimens welded wire mesh was used. The tensile strength test conducted on welded wire mesh and elastic modulus was determined as per ASTM A370 – 11 (ASTM 2011a). The results are presented in Table 2.1 in terms of average values and coefficient of variation. Mortar compressive strength was determined by performing tests on 50 mm mortar cubes in accordance with ASTM C109/C109M - 11 (ASTM 2011b) and the compressive strength of bricks and masonry were determined in accordance with ASTM C67-11(ASTM 2011c) and ASTM C1314 - 11(ASTM 2011d). From the results it is observed that, the compressive strength of masonry prisms was higher than the compressive strength of mortar cubes and lower than the compressive strength of bricks. This relation was earlier found and reported in the literature(Drysdale et al. 1999).

Table 2.1 Material Properties

Masonry materials		f_b (N/mm^2)	f_c (N/mm^2)	f_m (N/mm^2)
	Mean Value	21.07	2.45	3.72
	% COV	16.63	22.22	19.46
Welded wire mesh		f_t (N/mm^2)	E_s x 10^3 (N/mm^2)	A_s (mm^2)
	Mean Value	850.81	127.23	4.60
	% COV	13.82	12.24	4.38

where: f_b = brick compressive strength; f_c = mortar cube compressive strength; f_m = masonry compressive strength; f_t = yield tensile strength of welded wire mesh; E_s =elastic modulus of welded wire mesh; A_s = net cross sectional area of a single wire of welded wire mesh

2.2 Test Specimens

The experimental program consisted of six set of specimen for in-plane shear testing of unretrofitted and retrofitted samples. Series 1 and series 2 consist of single-wythe and two-wythe thick unreinforced masonry panels. Three specimens under each series were tested as-built (unretrofitted).

Series 3 to series 6 specimens were strengthened prior to testing using different configuration of welded wire mesh reinforcement with a constant thickness of micro concrete as shown in Fig.2.1. For strengthening of panels from series 3 to 6 WMM of 10 gauge (3.25mm) thickness 35 mm square grid spacing is used. Series 3 and 5 masonry panels are strengthened unidirectionally while series 4 and 6 are strengthened bidirectionally as shown Fig.2.1.

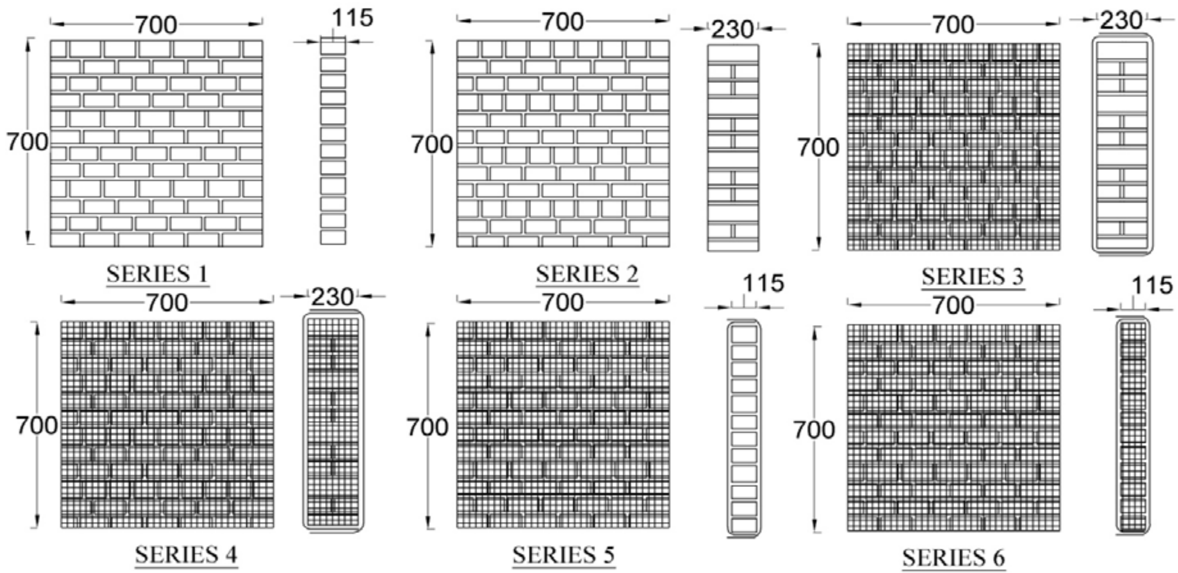


Figure 2.1 Specimen geometry and retrofit details

In bidirectionally strengthened specimens, reinforcement in vertical and horizontal direction were bent on face of thickness to get anchorage. Test specimens were given the notation UDSP or RFDSP, where UDSP refers to Unreinforced Diagonal Shear Panel and RFDSP refers to Retrofitted Ferrocement Diagonal Shear Panel. These wallets were numbered as per sequence of test performed. Specimen dimensions and details of the retrofit application are shown in Table 2.2

Table 2.2 Specimen dimension and retrofit application details

Series	*Specimen*	*Dimensions (mm)* *H*	*L*	*t*	*Retrofit Details*	*WMM Configuration*
1	UDSP -1 UDSP -2 UDSP -3	700	700	115	Unretrofitted one - wyth Thick Masonry Panel	-----
2	UDSP -6 UDSP -7 UDSP -8	700	700	230	Unretrofitted two - wyth Thick Masonry Panel	-----
3	RFDSP-9 RFDSP-10 RFDSP-11	760	760	290	Unidirectional WMM with 30mm Thk Micro concrete on both sides of two - wyth Thick Masonry Panel	G
4	RFDSP-12 RFDSP-13 RFDSP-14	760	760	290	Bidirectional WMM with 30mm Thk Micro concrete on both sides of two - wyth Thick Masonry Panel	G
5	RFDSP-15 RFDSP-16 RFDSP-17	760	760	175	Unidirectional WMM with 30mm Thk Micro concrete on both sides of one - wyth Thick Masonry Panel	G
6	RFDSP-18 RFDSP-19 RFDSP-20	760	760	175	Bidirectional WMM with 30mm Thk Micro concrete on both sides of one - wyth Thick Masonry Panel	G

NOTE: *H*= wallet height; *L* = wallet length; *t*=wallet thickness; *WMM*=welded wire mesh; *G* = grid

2.3 Strengthening Procedure

The strengthening of the unreinforced masonry panel is carried out in Ferro-cement consists of a WMM integrated within layers of rich cement-sand (1:4) plaster or micro-concrete. Such layer of Ferro-cement was provided on both faces of wallet. For integration of Ferro-cement layer with masonry panel, a thick nail (6 mm DIA. Steel rod) was provided across the specimen interconnecting the two layers of WWM. This nails transfer the shear between the WMM layer and the wall through dowel action. The total thickness of Ferro-cement was made 30mm which includes rich cement - sand (1:4) plaster of 10mm as a base coat and 20 mm micro concrete. To achieve accuracy in work a special formwork was prepared and strengthening was carried out stepwise.

2.4 Testing Procedure

The diagonal compression load was applied to the corners of the panels via a hydraulic actuator by INSTRON closed loop UTM of capacity 250T facility. ASTM E519/ E519M - 10(ASTM 2010) standard guidelines were used to investigate the in-plane diagonal shear strength of unretrofitted and retrofitted specimens.The experimental setup for the diagonal compression is shown in Fig. 2.2. The load applied to the panel by a steel shoe placed at the top corner, and transmitted to a similar shoe at the bottom corner. Displacement controlled loading was applied along the diagonal of the test specimens. The rate of loading of INSTRON was kept approximately 0.1mm/min for URM specimen while 0.3mm/min for retrofitted specimen. The displacements of panel diagonals in compression and in tension were measured at the middle of specimen by two linear variable differential transducers (LVDTs) kept on two sides of specimen. These LVDTs were connected to a data acquisition system to record the applied load and deformation of the specimen. Testing of all the specimens was carried out until maximum shear stress degraded to one third of maximum shear strength. For retrofits wallets the reinforcement ratios were calculated in both the direction as shown in Eqn.2.1 which gives insight on the ductile behaviour of strengthened specimens.

$$p_h = \frac{n t_w l_h}{Ht} \quad ; \qquad p_v = \frac{n t_w l_y}{Lt} \tag{2.1}$$

Where n = number of wires; t_w = wire thickness; l_h / l_y = total length of wire normal to horizontal (h) and vertical (v) axes, respectively; H = specimen height; L = specimen length; and t = specimen thickness.

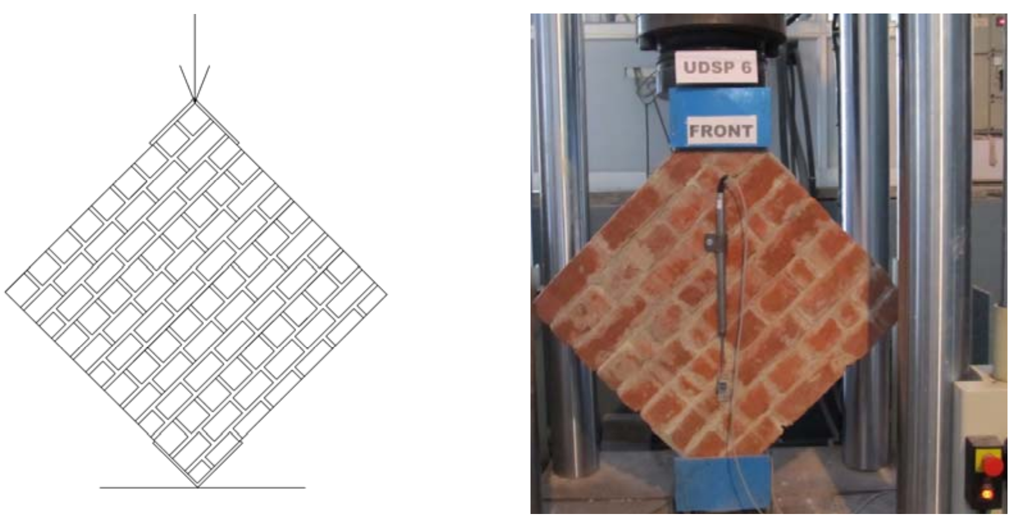


Figure 2.2 Standard test set up for the diagonal compression test

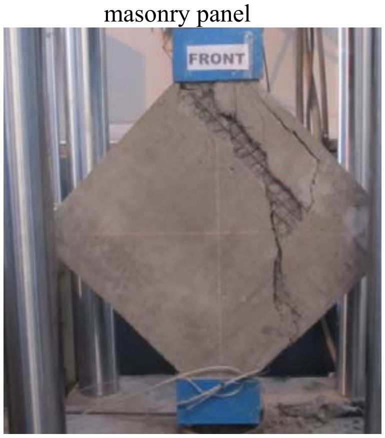
3. BEHAVIOUR OF URM AND RETROFITTED SPECIMENS

The failure of URM specimens was sudden and brittle. These panels failed by the formation of diagonal cracks. The failure was more sudden for one-wythe thick specimen, where the total collapses of specimen followed by the formation of a toothed crack. After development of first crack, the specimen was not able to sustain any further force. After the appearance of cracks, strain measuring devices had been removed to avoid damage to the instrument due to the sudden collapse of specimen. The response of these specimens can be considered as a combined diagonal shear and sliding failure as shown in Fig. 3.1(a) and Fig. 3.1(b)

(a) URM one - wythe thick masonry panel

(b) URM two -wythe thick masonry panel

(c) Unidirectionally retrofitted two - wythe thick masonry panel



(d) Bidirectionally retrofitted two - wythe thick masonry panel

(e) Unidirectionally retrofitted one- wythe thick masonry panel

(f) Bidirectionally retrofitted one - wythe thick masonry panel

Figure 3.1 Photographs of crack pattern in tested Specimens

The behaviour of retrofitted specimens was governed by the thickness of sample, reinforcement ratio and method of reinforcement. Initial failure was started in the form of diagonal crack which was restrained by the reinforcement, resulting in more ductile failure than that observed in the URM specimens. For unidirectionally reinforced specimens, failure was observed in terms of cracks originating from the edges of the specimen. These cracks changed into major cracks at the later stage as shown in Fig. 3.1(c) and Fig. 3.1(e). At large displacements, there was considerable local crushing of the micro concrete adjacent to the WMM. It was observed that at some locations mesh had ruptured. For bidirectionally reinforced specimens failure in terms of crack was distributed along the compressed diagonal of the specimens as shown in Fig. 3.1(d) and Fig. 3.1(f). These diagonal cracks were changed into major cracks and at the later stage, local crushing of masonry along with micro concrete took place near to loading and supporting shoes. It was observed that this cracked masonry together with restrain reinforcement, develops shear induced dilation mechanisms resulting in increased load and displacement capacity. In Table 3.1 Maximum shear stress (τ_{max}) is listed and also ratio of the shear strength of the retrofitted to corresponding URM specimen (τ/τ_0) is expressed. All the four reinforcement patterns (Series 3 to 6) resulted in an increase in shear strength. The maximum ratio of the shear strength of the retrofitted to corresponding URM specimen (τ/τ_0) was achieved in case of bidirectionally retrofitted one - wythe thick masonry panel which was 7.06. The observed increase in shear strength of the retrofitted to corresponding URM specimen was in the range of 0.57–1.48 MPa in experimental program.

The behaviour of URM and retrofitted specimen is plotted in terms of shear stress (τ) vs % drift (δ).The experimentally measured diagonal force, P was transformed into shear stress, τ as

$$\tau = \frac{0.707P}{0.5t(L+H)} \tag{3.1}$$

where t is specimen thickness, L is specimen length and H is specimen height.
Measured drift values (δ), which is equal to shear strain (γ) is calculated as

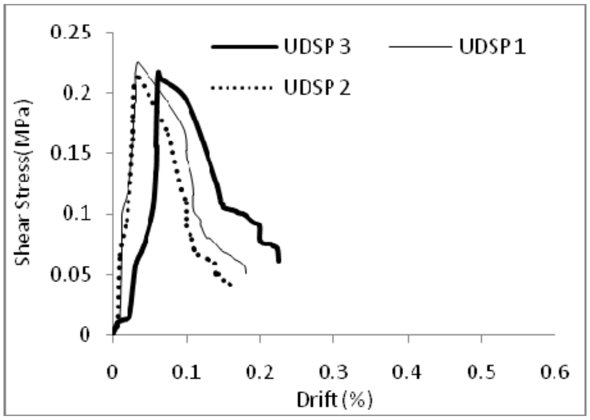
$$\delta = \gamma = \frac{\Delta V + \Delta H}{g} \tag{3.2}$$

where ΔV is diagonal shortening along the axis of applied force, ΔH is diagonal elongation measured perpendicular to the axis of applied force, and g is the gauge length.

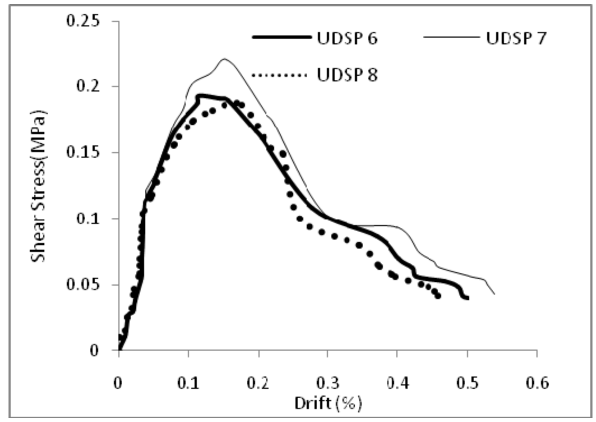
Table 3.1 Test Results

S.N.	*Sample*	q_h 10^{-3}	q_v 10^{-3}	P_{max} (kN)	F_{max} (kN)	τ_{max} (N/mm^2)	τ/τ_0	*Average* τ/τ_0
1	UDSP -1	0.00	0.00	24.65	17.43	0.22	-	-
	UDSP -2	0.00	0.00	25.54	18.06	0.22	-	
	UDSP -3	0.00	0.00	24.19	17.10	0.21	-	
2	UDSP -6	0.00	0.00	44.00	31.12	0.19	-	-
	UDSP -7	0.00	0.00	50.13	35.45	0.22	-	
	UDSP -8	0.00	0.00	42.48	30.04	0.19	-	
3	RFDSP-9	0.23	0.59	211.92	149.85	0.93	4.65	5.17
	RFDSP-10	0.23	0.59	258.44	182.74	1.13	5.65	
	RFDSP-11	0.23	0.59	237.33	167.82	1.04	5.20	
4	RFDSP-12	0.59	0.59	175.36	124.0	0.77	3.85	4.30
	RFDSP-13	0.59	0.59	210.00	148.50	0.92	4.60	
	RFDSP-14	0.59	0.59	202.84	143.43	0.89	4.45	
5	RFDSP-15	0.46	0.77	147.96	104.62	1.30	6.00	6.63
	RFDSP-16	0.46	0.77	169.80	120.06	1.49	6.87	
	RFDSP-17	0.46	0.77	173.61	122.76	1.52	7.01	
6	RFDSP-18	0.77	0.77	187.28	132.42	1.64	7.57	7.06
	RFDSP-19	0.77	0.77	141.96	100.38	1.25	5.77	
	RFDSP-20	0.77	0.77	193.42	136.77	1.70	7.85	

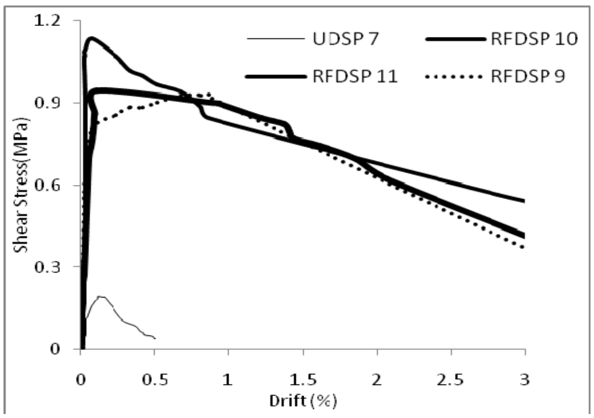
where: *S.N.* = series; q_h = horizontal reinforcement ratio; q_v = vertical reinforcement ratio; P_{max} = maximum applied diagonal force; F_{max} = maximum horizontal shear force; τ_{max} = maximum shear stress; τ/τ_o = ratio of the shear strength of strengthened specimen to that of the unretrofitted specimen and *Average* τ/τ_o =average value of ratio of three specimens



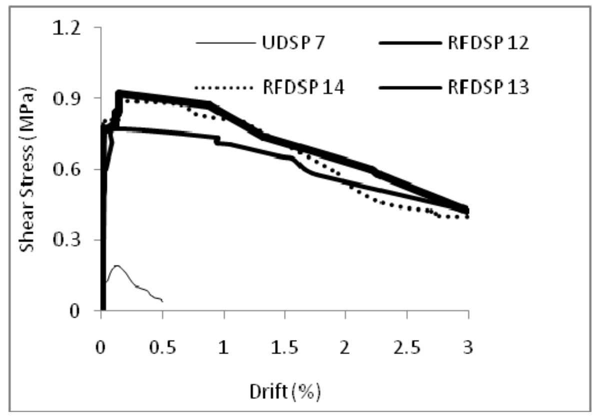
(a) Response of URM one - wythe thick masonry panel



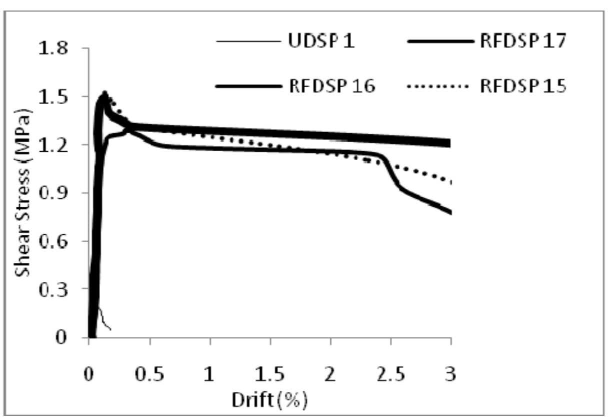
(b) Response of URM two - wythe thick masonry panel



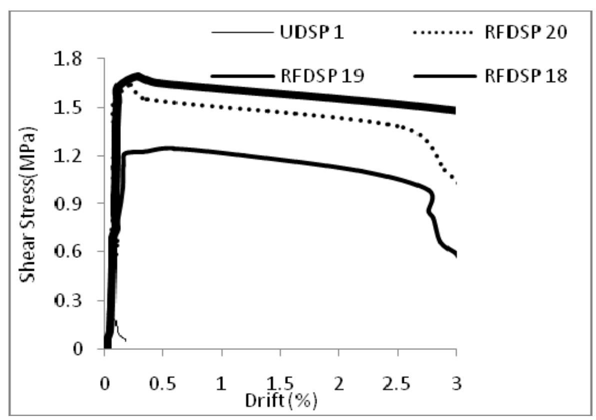
(c) Response of URM and unidirectionally retrofitted two - wythe thick masonry panel



(d) Response of URM and bidirectionally retrofitted two - wythe thick masonry panel



(e) Response of URM and unidirectionally retrofitted one - wythe thick masonry panel



(f) Response of URM and bidirectionally retrofitted one - wythe thick masonry panel

Figure 3.2 Shear stress- drift plots of URM and Retrofitted Specimens

The behaviour of URM and retrofitted specimens were plotted as shown in Fig. 3.2.URM specimens exhibited sudden strength degradation once crack had propagated and subjected to sudden failure. Behaviour of one - wythe thick masonry panel is very sudden and brittle as shown in Fig. 3.2(a). Due to interlocking of bricks behaviour of two - wythe thick masonry was not as sudden and brittle compared to one - wythe thick and drift ratio was almost double that of one - wythe thick masonry specimen as shown in Fig. 3.2(b). Behaviour of all strengthened specimens was linear - elastic as seen in Fig. 3.2(c) to Fig. 3.2(f) up to cracking and then a gradual decrease in the post-peak stress, except for RFDSP 10 which failed by the sudden development of the major crack on one side due to eccentricity in loading. It was noted that response of RFDSP 17 and RFDSP 19 was most ductile and their behaviour were almost straight. The shear strength of these specimens was higher amongst all tested specimens. It was observed that after attaining peak load, crack has been developed exposing the reinforcement. Provided reinforcements restrained the further development of cracks allowing the specimens to undergo large displacement.

4. DISCUSSION AND CONCLUSIONS

A diagonal compression test was conducted to study in-plane shear behaviour of masonry specimens strengthened with welded wire mesh and micro concrete. The effectiveness of the reinforcing schemes to restrain the diagonal cracking failure mode was investigated. Eighteen masonry samples were tested with different patterns of reinforcement applied to one-wythe thick and two-wythe thick specimens. The results were studied in terms of shear strength. The important conclusions made from this study are listed as follows.

1. URM Specimens exhibited sudden strength degradation and subjected to brittle failure whereas the responses of retrofitted specimens were quite ductile.
2. Increase in shear strength was achieved in the range of 0.57–1.48 MPa with the application of the welded wire mesh and micro concrete retrofit technique.
3. Micro concrete holds surrounding welded wire mesh in position resulting in excellent mechanical bond between the reinforcement and masonry.
4. Insignificant debonding was observed between masonry and welded wire mesh.
5. Welded wire mesh with micro concrete is a viable option of retrofit for improving the seismic behavior of masonry buildings.

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