Push over Analysis of Steel Strengthened Masonry

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Abstract

Strengthening of un-reinforced masonry (URM) structures to improve its seismic performance is not only well recognized among the research community but also getting attention of practicing engineers around the globe. The estimation of lateral capacity and performance of masonry becomes complex because of heterogeneous nature of its constituent materials and other associated problems. Heterogeneous and homogenous approaches are generally adopted for analysis of masonry. In heterogeneous approach, bricks, joints and bed mortar are modeled separately whereas; in homogeneous approach the masonry is modeled as one continuum body. Earlier approach requires more efforts but level of accuracy of both the approaches is almost same. In the present work, the homogenous approach is used to model the masonry wall panels. The behaviour of the un-reinforced and steel strengthened masonry wall panels is determined from push over analysis using a commercial finite element package MSC-MARCS 2003. The aim of this analysis was to study the lateral load–displacement behaviour and failure mechanism of strengthened masonry under monotonic lateral loading. Four single leaf panels with aspect ratio of 1.0 were constructed on strong floor of structural laboratory. The square panels having dimension of 1200 mm were constructed using same type of material and workmanship. Of them; one wall panel was unreinforced reference and remaining were strengthened with square steel mesh. Firstly the specimens were tested till failure under monotonic lateral loading and later finite element analysis of the same specimens was carried out using MARCS2003. The analytical lateral force - lateral displacement relationship and region of stress concentrations were analyzed to depict the lateral capacity and failure mechanism of strengthened masonry wall panels. At the end the analytical results were compared with the experimental recorded values and the analytical results were found in good agreement with the experimental values.

Key Words: Masonry strengthening: Steel strips: Push over analysis:

1. Introduction

Significant portion of existing old structures world wide are made of brick masonry units. Earthquakes have shown repeatedly the seismic vulnerability of existing masonry structures. Masonry strengthening specially in shear is of particular interest in seismic prone areas to reduce its vulnerability against lateral loads. In recent past, a variety of masonry strengthening approaches are investigated using FRP and steel [1,2]. A simplified and cost effective masonry strengthening technique was proposed by Farooq [3,4], consisting of galvanized steel strips application as an external reinforcement mesh. This paper presents the push over analysis of the same proposed technique.

The estimation of lateral capacity and lateral displacement of masonry is a complex phenomenon due to heterogeneous nature of its constituent materials and other associated problems. Generally, heterogeneous and homogenous approaches are adopted for masonry analysis. In heterogeneous approach, all the constituent masonry materials are modeled separately, whereas in homogenous approach the masonry is taken as one unit. Qaisar [5] modeled masonry using both the approaches and reported very small variation in results of both the approaches. However, the quantum of efforts is much higher in case of heterogeneous approach. Mustafa Taghdi [6] used simple truss models to predict strength and ductility of low-rise walls retrofitted with diagonal and vertical steel strips. He used a step-by-step analysis procedure for design purposes. The analysis of walls utilizing the procedure provided quite correct descriptions of the sequence of yielding among members capturing accurately the global lateral force–lateral displacement relationships. Different researchers [7,8] have reported that the interface elements used in heterogeneous models represent the interaction between adjoining masonry units eliminating the need for further degrees of freedoms. Dhanasekar [9] and Lourenco [10] proposed macro models for solid masonry, in which the effects of material nonlinearity and progressive failure can be reproduced. They modeled masonry as a continuum with average properties of brick and mortar including appropriate nonlinear behaviour of the brick–mortar interface. Khattab and Drysdale [11] also modeled hollow concrete masonry as a homogeneous body and considered mortar joints as planes of weakness. The appraisal of smeared crack modelling techniques was provided by Lofti and Shing [12]. These techniques enabled the analysis of large panels and through suitable material modeling, the strain softening effects without sudden redistribution of stresses can also be achieved. The compressive and tensile behaviour of masonry units are governed by Von Mises failure surface with a Rankine type tension cut-off. Ghosh et al. [13] used ABAQUS [14] to model solid masonry shear walls in which masonry was treated as two-phase material. Zhuge [15] developed a two-dimensional plane stress element model for the nonlinear analysis of un-reinforced masonry shear walls. This model was developed using a homogeneous material model to predict the detailed load–deflection characteristics and critical limit states of URM walls under in-plane earthquake ground acceleration. In the present study, the homogenous approach is adopted to model the masonry. The behaviour of the URM and strengthened masonry wall panels is obtained from push over analysis using a commercial finite element package MSC-MARCS 2003. The aim of this analysis is to study the lateral load–displacement behaviour of masonry under monotonic lateral loading.

2. Experimental Program

This experimental program investigates the effectiveness of using steel strips as externally strengthening technique for URM walls. A typical specimen had a length of 1.2 m, height of 1.2 m and width of 0.12 m. Four single wythe masonry specimens were tested at the structural laboratory of the University of Engineering and Technology, Lahore, Pakistan. Of them, one wall was tested as reference specimen up to failure. The remaining three were strengthened using steel strips; out of these three specimens, two specimens were strengthened on a single-face and the one was strengthened on double-face. All the four specimens were subjected to a monotonic lateral load up to failure.

2.1 Test Specimen

The test specimens were intended to represent a typical existing Pakistani masonry structures. Single wythe walls were constructed in a running bond using solid clay brick masonry units. The test specimens had a geometrical aspect ratio of one. The specimens were built using cement-based mortar directly on the laboratory strong floor. The square specimens had a length of 1200 mm and a thickness of 115 mm (Figure 1). Specimens were left to cure at room conditions for approximately 28 days. Specimen US was reference wall. Specimen FSM was strengthened with single sided fine steel mesh. Whereas, specimen SCM was reinforced with single sided steel strip mesh and DCM was strengthened with double sided steel mesh. Table 1 summarizes the test specimens.

Table 1 Masonry wall Designation as per strengthening arrangement

Sr. No.	**Wall Designation**	**Spacing of Steel Strips (mm)**		**Mesh application**
		V	**H**	
1	Single coarse steel mesh wall panel (SCM)	228	228	Single side
2	Fine steel mesh wall panel (FSM)	228	114	-
3	Double coarse steel mesh wall panel (DCM)	228	228	Double side“
4	Reference un-strengthened wall panel (US)	-	-	-

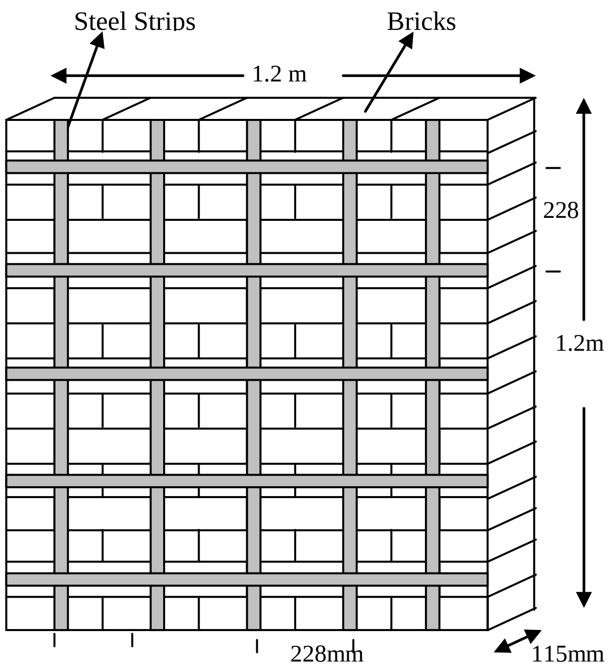


Fig. 1 Line diagram of masonry wall panel

2.2 Brick and Mortar

Typical commercial clay brick units were used for specimens' construction. A brick unit is nominally measuring 235 X 115 X 71 mm. The brick units have an average flatwise compressive strength of 11 MPa, a standard deviation of 1.23 MPa, and initial rate of water absorption 13%. A mortar having one part Portland cement to four parts of sand was used for specimen construction. The average mortar cube compressive strength was 8.4 MPa with a standard deviation of 0.14 MPa. Also, the masonry compressive strength was determined by material testing of masonry prisms according to ASTM. The compressive strength was 4.8 MPa with a standard deviation of 0.4 MPa.

2.3 Steel Strengthening

The wall panels were strengthened with galvanized mild steel strips having dimension of 45 x 1.3 mm and with yield strength (f_y) of 235 MPa and ultimate strength (f_u) of 319 MPa. The application of the steel strips was a simple and rapid operation. First, holes were drilled at specified spacing in the wall and the steel strips. Second, the steel strips were anchored in form of square mesh on the surface of the wall using HILTI HPS-1 6/15x40 type bolts. The distance between the bolts was 228 mm on the horizontal and vertical direction for specimen SCM & specimen DCM whereas it was 150 mm and 228 for specimen FSM.

3. Experimental Lateral Load - Displacement Relationship

The lateral force displacement relationship of the tested specimens is shown in Figure 15. The effect of steel strengthening on deformation capacity and ultimate lateral strength can be readily seen in the figure. The strengthening improved the lateral strength and lateral displacement. The specimen DCM showed approximately 87% increase in the lateral load carrying capacity compared with specimen US (reference specimen) and ultimate lateral drift of 1.78% was recorded at failure. The lateral strength of specimen US was 90.7 kN at a lateral drift of 1.45%. The sliding failure of specimen US resulted in quite higher lateral drift. The specimen SCM followed closely the behavior of specimen US but due to confinement and reinforcement, the ultimate lateral strength of the specimen SCM was increased by 130%.

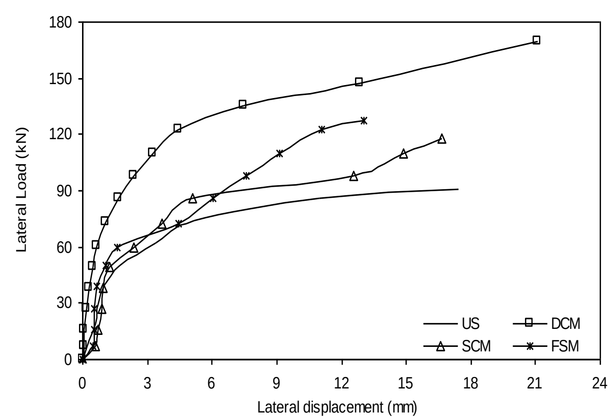


Fig. 2 Lateral load vs lateral displacement relationship

The ultimate lateral strength of specimen FSM was 127.6 kN at a lateral drift of 1.09%. The initial stiffness of strengthened specimens was very much similar to the reference specimen. This may represent an advantage for structures in seismic zones where increasing the initial stiffness will result in increasing the fundamental frequency and seismic demand. The strengthened specimens developed approximately a linear behavior upto yielding except at the end of the test where a significant nonlinear behavior was observed. This nonlinear behavior was due to opening and sliding of shear cracks passing through masonry units. The upgraded specimens have shown better non-linear performance due to application of steel strips.

4. Push-Over Analysis of Masonry Walls using Finite Element Analysis

The estimation of lateral capacity and performance of masonry is a complex phenomenon due to heterogeneous nature of its constituent materials and other associated problems. For the analysis of masonry, mainly heterogeneous and homogenous approaches are adopted. In heterogeneous approach, all the constituent masonry materials are modeled separately to determine the masonry seismic performance, whereas in homogenous approach the masonry is taken as one unit. In the present study, the homogenous approach is adopted to model the masonry. Push over analysis using a commercial finite element package MSC-MARCS 2003 is carried out to determine lateral performance of the masonry wall panels. The aim of this analysis was to study the lateral load–displacement relationship of masonry under monotonic lateral loading.

4.1 Geometry of masonry wall panels

Four masonry wall panels were analysed on MARCS 2003. The geometry of the wall panels was kept same as for the experimental programme described above. The size of the specimen was kept 1.2 x 1.2 m square panel. The bricks were taken as four nodded shell element and the steel strips were taken as two nodded line element. Each brick was divided into four shell elements with middle node at the center of the brick as shown in Figure 3(a). The steel strips were anchored with the masonry wall at the middle node of the bricks, which ensured stress transfer between masonry and steel as was the case in the experimental work. The steel strips were attached to masonry shell elements at middle nodes as shown in Figure 3(b). The connection between steel strips and brick was taken as pin joint. The pull out of bolts was not observed during experimental progamme, therefore it was ignored in the modeling.

4.2 Boundary Condition

During experimentation, the specimens were tested as cantilever wall with its base fixed and wall was allowed to move freely at top. Same boundary conditions were set for the model in MARCS 2003 as shown in Figure 4. All the three degrees of freedom at the bottom were restricted. Similar constant axial load was applied at the top of concrete cap beam for all the specimens. The loading increments were kept similar to experimental lateral load application.

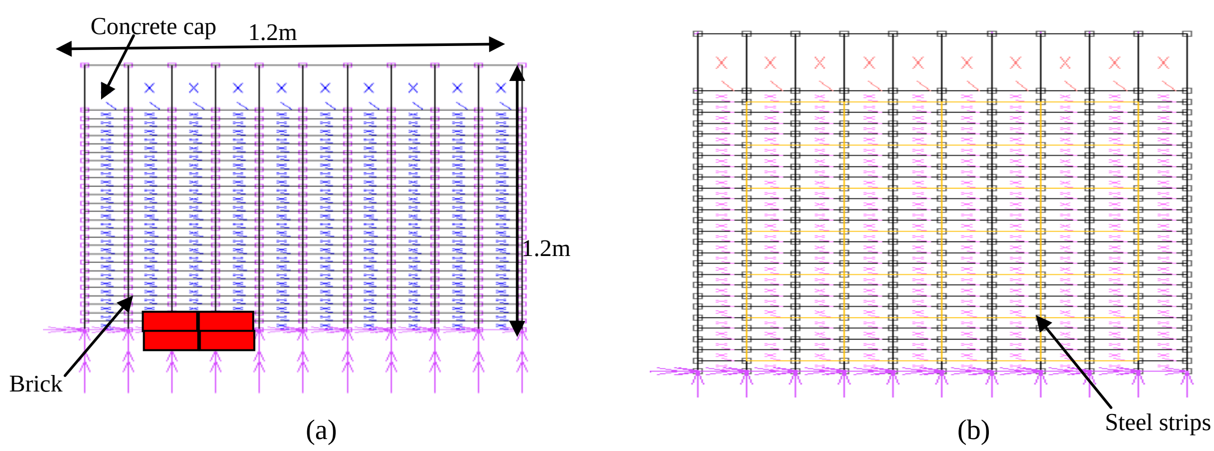


Fig. 3 Geometric model of wall panels (a) un-reinforced masonry (b) strengthened masonry with fine steel mesh

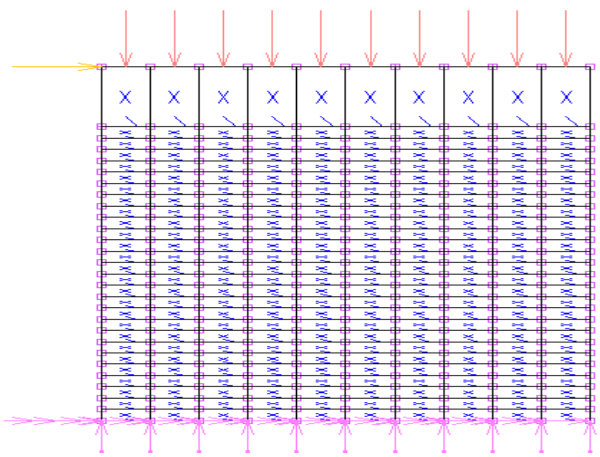


Fig.4 Boundary condition with loading

4.3 Material Property

The material properties of masonry were selected after carrying out the laboratory testing of materials and compared with quoted values by different researchers. The detail of the material properties are given in Table 2. In the model, elasto-plastic isotropic element was selected and Von Mises failure criteria was used.

Table 2: Material Properties

Properties	Masonry	Concrete	Steel
Elastic Modulus (MPa)	5100	30000	200000
Poison's Ratio	0.2	0.3	0.3
Density (kg/m^3)	2200	2400	7890

4.4 An overview of elastic analysis programme

Plane stress elements were used for the analysis of masonry walls. The constitutive relation for linearly elastic and isotropic material for plane stress was used. In order to simulate the base fixity, the bottom nodes were restrained in the horizontal and vertical directions. Firstly, axial pre-compression in terms of pressure of 0.7 MPa (176 kN same as for experimental programme) was applied on the top cap beam and then the experimental lateral load increments were applied at center of the cap beam. The MARCS 2003 was allowed to control the solution automatically adjusting the increments suitably for achieving converged solutions within a minimum period. Four nodded shell element was selected for bricks with isotropic properties and two nodded line element was selected for steel strips.

Smeared crack finite element modal was used to analyze the masonry wall panels under monotonic lateral loading. The specimens were loaded till failure. The tensile cracking in reinforced masonry can be easily modeled using the smeared crack analogy as it does not require a large number of degree of freedom for modeling crack propagation and is also computationally efficient. In this model, at each integration point the constitutive calculations were performed separately and in each loading increment the cracks were included into the calculations. The tensile cracks were modeled by changing material characteristics to account for induced crack along the orthogonal axes. This affects the stiffness matrix significantly and ensured subsequently redistribution of stresses within the element. This modification of stiffness matrix requires significant iteration within the prescribed load or displacement increment. In addition, Von Mises failure criteria were used for analyzing the masonry wall panel on MARC 2003. According to Von Mises criterion, the yield stage of material occurs when its shear strain energy per unit volume approaches a critical value.

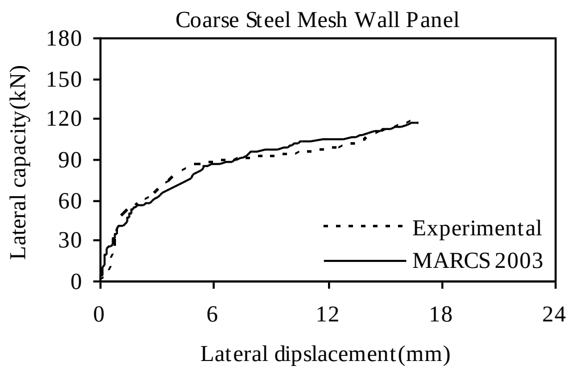
In the output data, components of stresses, strains, displacements and energies etc. for the whole model were extracted during the course of the analysis. For understanding of the stress distribution and potential failure mechanisms of un-reinforced/strengthened wall panels, stresses/strains for elements in the critical regions, reaction force and displacement at the nodes of load application were extracted. Results obtained from post processing of the extracted data are discussed in the following sections.

4.4.1 Load-displacement Behavior

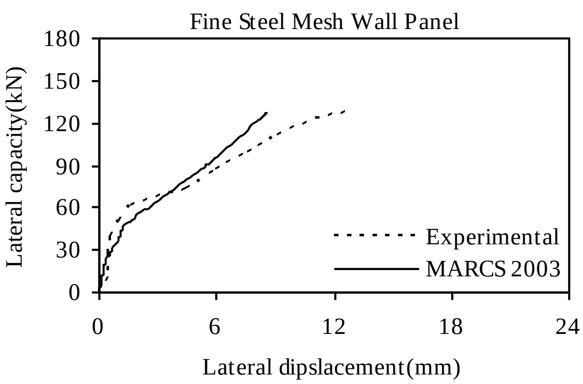
The geometric models for all the specimens with boundary conditions similar to experimental setup were prepared in MARCS 2003. In experimental testing, the stress transfer between masonry and steel strips was through bolts and same was simulated in the model by providing a central node in the brick by dividing each brick element into four shell elements. Push over analysis of all the specimens was carried out on MARCS 2003 to obtain analytical lateral load-displacement relationship of upgraded specimens. The comparison of experimental and analytical results is shown in Figures 5(a-d). The analytical results were found in good agreement with the experimental recorded values except for specimen FSM. For specimen FSM, lesser lateral displacement was achieved as compared to experimental result showing earlier failure of the specimen (Figure 5b). Quite higher lateral displacement was recorded for reference specimen US (Figure 5d) because the specimen experienced sliding failure.

4.5 Observed failure pattern of masonry specimens

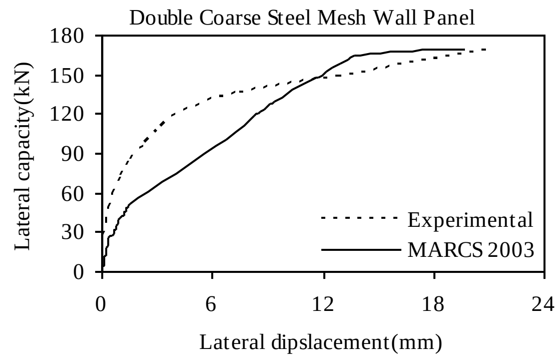
The specimens were tested till failure under incremental loading. The failure pattern observed during test is compared with failure pattern achieved from MARCS 2003. The details are as under:



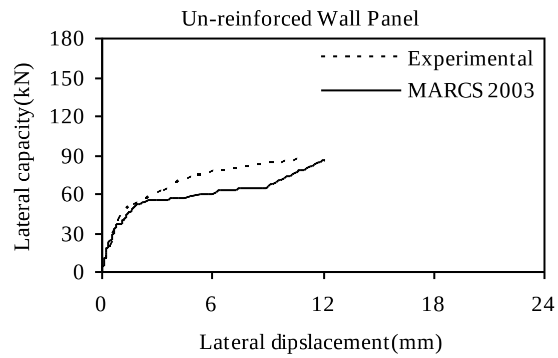
(a) Specimen SCM



(b) Specimen FSM



(c) Specimen DCM



(d) Specimen US

Fig. 5 Load-displacement relationship of masonry specimens

4.5.1 Reference specimen US

The reference specimen US was tested under monotonic lateral load under constant axial stress. Near failure, cracks appeared at right bottom edge of the specimen. In addition to cracking, the specimen US also experienced some sliding along horizontal plane in the bed mortar between 13^{th} and 14^{th} wythes from top of the specimen at later stages of test (Figure 6). Finite element analysis of the same model specimen was carried out. After the analysis, shear cracking stress distribution was observed (Figure 7) to determine the analytical failure pattern and it was found in reasonably good agreement with observed experimental failure mode. Stress concentration can be seen in the left portion of the Figure 7 indicating some cracking and no stress concentration was observed at lower right portion which may be due to sliding.

Fig. 6 Sliding failure of specimen US

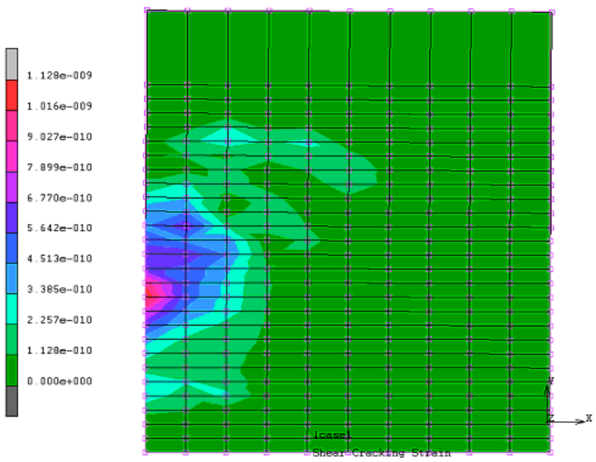


Fig.7 Distribution of shear cracking stress specimen US

4.5.2 Specimen DCM

The specimen DCM was subjected to lateral loading in increments till failure to study its crack pattern at failure. The specimen experienced ductile shear failure. The diagonal cracks appeared which were initiated at load application point and extended towards the right bottom edge. The cracks were initiated generally in the vertical joints and mortar bed extending from top to bottom in the direction of lateral load. The cracks also traveled through bricks at right top point at load application and at the bottom of the specimen DCM causing crushing of bricks as shown in Figure 8. The analytical state of cracking shear stress distribution (Figure 9) indicates that the region of maximum stress concentration is towards the load application side confirming diagonal cracking of the specimen DCM. The experimental failure pattern is being confirmed by analytical model.

Fig. 8 Ductile shear failure of specimen DCM

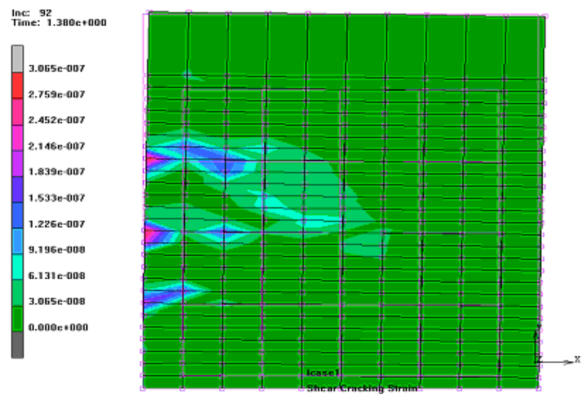


Fig.9 Distribution of shear cracking stress specimen DCM

Fig. 10 Shear failure of specimen SCM

4.5.3 Specimen SCM

The specimen SCM also experienced ductile shear failure and first diagonal crack appeared at the top (Point of load application) and extended towards the right bottom edge as shown in Figure 10. Generally similar pattern of failure was observed as earlier discussed for specimen DCM. After the analysis, various parameters were studied to observe stress distribution and behaviour of the specimen

SCM at failure stage. The region of maximum cracking shear stress was recorded towards the load application side (Figure 11) which validated the observed experimental cracking pattern.

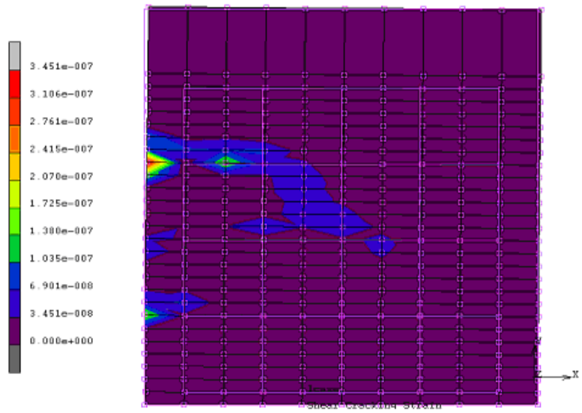


Fig.11 Distribution of shear cracking stress of specimen SCM

4.5.4 Specimen FSM

The specimen FSM was subjected to lateral load in increments till failure. The specimen experienced ductile shear failure with diagonal cracking. The cracks were produced generally in the joints and mortar bed extending from top to bottom. Cracks also traveled through bricks at top near load application point and at the bottom as shown in Figure 12. Various parameters of analytical model were also studied to observe stress distribution within the specimen. Figure 13 shows the region of cracking shear stress distribution within the specimen. The region of maximum stresses was observed towards the load application side similar to experimental failure pattern.

Fig. 12 Shear failure of specimen FSM

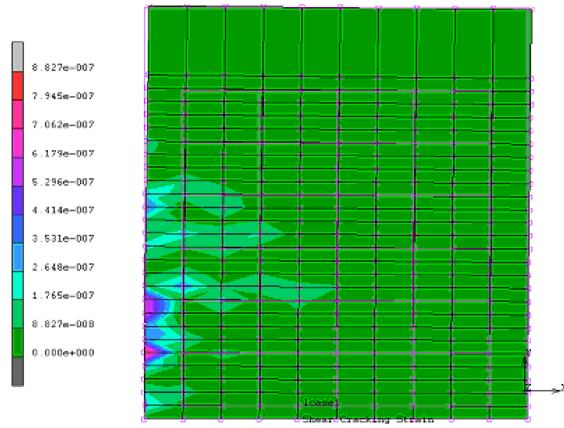


Fig.13 Distribution of shear cracking stress specimen FSM

5. Conclusions

The theory of macro modeling of masonry has been presented in this paper. The modeling was applied to both the un-reinforced and strengthened masonry wall panels. The findings of this research work are as under:-

1. The Finite Element analysis has shown a successful prediction of masonry wall panel's behaviour under monotonic lateral load.
2. The load–displacement curves have been found to be in good agreement, on average, with those obtained from experimental investigation. However, in case of FSM specimen, the experimental recorded lateral displacement was more than the analytical lateral displacement.
3. The FE model has predicted sensible stress distribution in the critical regions of the masonry wall panels. This push over analysis appears to have the potential for predicting the overall behaviour of un-reinforced and strengthened masonry wall panels.

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