**Evaluation of In-plane Shear Response in Textile-Strengthened Masonry Walls**

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**Abstract.** Frequent earthquake hits and sudden structural collapses worldwide have persistently proven the vulnerability of Unreinforced Masonry (URM) Walls exposed to earthquakes. URM Walls experience in-plane and out-of-plane failures when the structural supports undergo seismic movements. Despite being efficient to withstand high compressive loads, masonry walls exhibit unanticipated collapses due to their brittle nature under the flexural and shear forces. Consequently, the development of strengthening techniques to impart ductility and provide sufficient warning before collapse has been the major focus of masonry research. Traditional techniques are less preferred because of their potential disadvantages. Modern strengthening methods utilizing Fiber Reinforced Polymers (FRP), Welded Wire Meshes (WWM), and textiles, have been proposed by various researchers around the world. This paper will discuss the Diagonal Shear behavior of single wythe URM wallets of dimensions 480 mm x 480 mm x 110 mm strengthened with Geo-grids and Black Carbon Grids. The study revealed that the load-bearing capacity, ductility, and the energy dissipated by the walls have significantly multiplied with textile strengthening.

**Keywords:** Unreinforced Masonry; In-plane behaviour; Diagonal shear; Earthquakes; Textile Strengthening; Geo-grids; Black Carbon Grids; Textile Reinforced Mortar (TRM).

# Introduction

Unreinforced brick masonry walls are the assemblage of brick units that act as load-bearing structures and were preferred due to good compressive strength and long-term durability. These structures are vulnerable to sudden collapse when exposed to earthquakes[1] due to which the confined masonry came into practice. The reinforcement in the confined masonry imparts sufficient deformation to the structure before collapse when subjected to lateral loads. However, adopting confined masonry for existing structures and retrofitting works is highly uneconomical and time-consuming. Hence studies in the masonry structures focus on external strengthening of structures[2]. Various studies focus on improving the load-bearing capacity of the masonry walls using traditional techniques like repointing and rebar jacketing. Due to the disadvantages of these techniques[2], the possibility of textiles from carbon, glass, aramids, and other polymers[3] is being explored. The study of incorporating textiles in the masonry strengthening ensures better bonding with the masonry surfaces along with delay in the collapse of the entire structure. The efficiency of the textile reinforcement is evaluated based on the number of surfaces strengthened, the number of layers used in the strengthening process, the orientation of textiles, etc. Giaretton et al.[3] compared the performance of the in-plane shear behavior of clay and hollow block masonry walls strengthened with glass and polypropylene fibers, comparing both single-sided and double-sided strengthening configurations. Though the failure initiated from the weakest mortar joints was similar to the control specimen, the enhanced shear characteristics of the strengthened specimens were inferred. This study compared the in-plane shear performance of the 480mm x 480mm x 110mm wallets that were externally strengthened with geo-grids that are mainly used in soil reinforcement applications and the Black Carbon Grids (BCG). The tested specimens were analyzed for the modified failure patterns, load-deflection, and ductility characteristics. The obtained test results were compared with that of the control specimens.

# 2 Material Characterization

The masonry walls were constructed with well-soaked local bricks of size 230mm x 110mm x 67mm. Three frogs of the bricks were filled with 1:4 grade mortar and subjected to monotonic compression tests in the Universal Testing Machine according to ASTM C67-20[4]. The average compressive strength of the bricks was found as 12.6 MPa. Similarly, the monotonic compression tests on the 50mm x 50mm x 50mm cubes were carried out as per ASTM C-109/C109M-20a[5] and the average compressive strength of the mortar cubes was identified as 13 MPa after curing for 28 days. The masonry walls were strengthened with uniaxial SGU300 geo-grids with aperture sizes of 60mm x 20mm of tensile strength 300 N/mm2 and Black Carbon Grids (BCG).

# 3 Specimen Preparation

Three single wythe 110 mm thick wall specimens with 480 mm height and length were cast in running bond as per ASTM E519[6]. The configurations of the wall specimens were classified based on the external strengthening adopted in the study and are denoted by the equation “DS-i” where “DS” denotes Diagonal Shear and “i” refers to the adopted external strengthening. For example, DS-C refers to the control specimen cast identical to the site conditions, the DS-GG specimen which is externally strengthened with SGU300 geo-grids on both sides, and the DS-BCG is strengthened with BCG externally. For strengthening the walls, firstly the surfaces of the walls were cleaned and the holes were drilled and inserted with 35mm long black masonry screws acting as anchors.

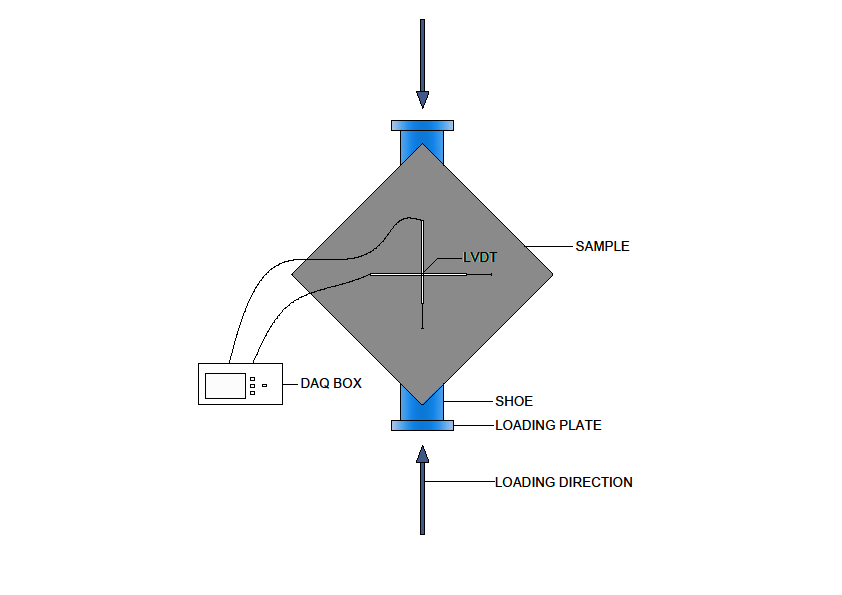
|  |  |
| --- | --- |
|  |  |
| a) | b) |
|  |  |
| c) | d) |
|  |  |
| e) | f) |

**Fig.1.** a) Unreinforced brick masonry wall b) Anchorage position marking c) Wetting specimens with cement slurry d) SGU300 strengthening with the first coat of 1:4 grade mortar e) and f) Final coat of 1:4mortar followed by plastering of the wall specimens

The walls were then wetted with a thin layer of cement slurry, the first coat of 1:4 grade mortar layer, and the textiles were tightened with the anchors. The specimens were coated with another layer of mortar, ensuring the previous layer had not been set and plastered for a smooth finish.

# 4 Experimental Setup

Every specimen was subjected to Diagonal Compression Testing as per ASTM E519/519 M-15[6]. The entire testing was carried out in a 75T loading frame. The specimens were placed on the steel shoes on the top and bottom. A 30T load cell was connected to the hydraulic actuator and the compression was applied on a steel shoe on the top. Two Linear Variable Differential Transducers (LVDTs) were placed to measure deflections: one above the actuator and the other one in the horizontal direction of the specimens. The test setup utilized in the study is illustrated as a schematic diagram in Figure 2. The compression was applied based on varying strokes and every instrument was connected to the Data Acquisition System for recording the data from testing.



**Fig.2.** Schematic representation of the Diagonal shear test setup

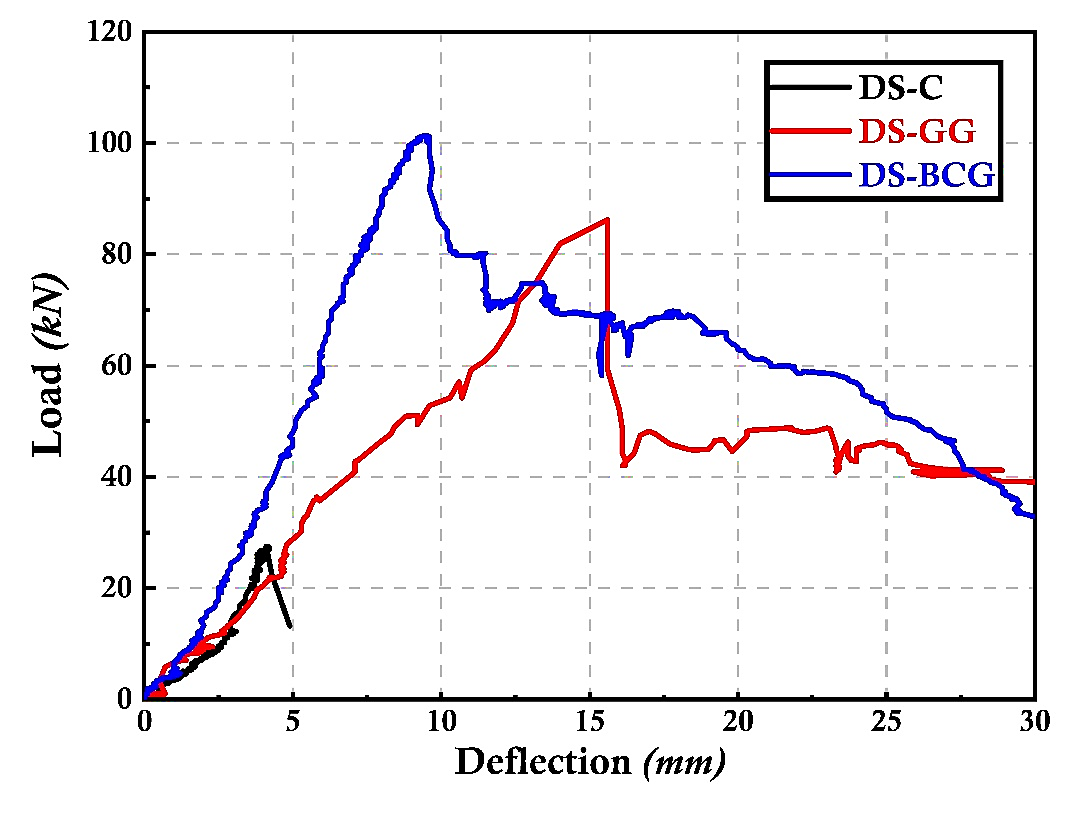
# 5 Results and Discussion

**5.1 Failure Pattern and Load Deflection Behaviour**

The failure patterns of the three specimens are illustrated in Figure 3. The control specimen DS-C experienced a brittle failure in the initial deflections whereas every strengthened specimen exhibited ductile failure at the higher deflections. The cracks initiated at the weak mortar joints in every specimen while increasing the applied compression. The control specimen experienced a shear step failure along the mortar joints after attaining the peak load of 27.5kN, followed by the splitting of the wall. The strengthened specimens exhibited a similar crack pattern originating from the weakest mortar joint at the bottom as that of the control specimen. When the load was increased, the cracks extended to the top of the specimen, and the geogrid and BCG external strengthening yielded controlling the shear in the specimens before failure. The specimen DS-BCG exhibited the highest stiffness and the peak load of 101.6kN after which it failed by a significant toe crushing. However, the specimen DS-GG that failed by multiple crack formations along the main vertical diagonal showed a better post-peak performance compared to DS-BCG. The poor post-peak performance of the specimen DS-BCG is attributed to the toe crushing, which led to the gradual strength degradation in the higher deflections. Table 1 includes the maximum load, deflection at maximum load, and the failure modes of the specimens in the study.

|  |  |  |
| --- | --- | --- |
|  |  | |
| a) | b) | |
|  | |
| c) | |

**Fig. 3.** Crack Patterns of DS-C, DS-GG, DS-BCG



**Figure 4.** Load-deflection behaviour

**Table 1.** Observations from the Diagonal Shear Test of the specimens

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| **Specimen ID** | **Failure Mode** | **Cracking Load (kN)** | **Peak load (kN)** | **Deflection at Peak Load (mm)** | **Ductility ratio** |
| DS-C | Brittle Shear Step Failure | 27.5 | 27.5 | 4.1 | 1.13 |
| DS-GG | Ductile failure due to multiple cracking | 68.96 | 86.2 | 15.6 | 1.25 |
| DS-BCG | Ductile toe crushing | 81.57 | 101.2 | 9.3 | 1.56 |

**5.2 Ductility**

Ductility is measured based on the response of the specimens to the applied loading after attaining the peak strength and is an essential parameter to quantify the behaviour of the structure during earthquakes. The ratio of ultimate displacement to the yield displacement at the cracking load called ductility ratio, measures the ductility of the specimen. The first crack load occurred when the load reached 80 percent of peak load and hence ductility is calculated for that. As the control specimen failed by sudden shear, the specimen did not have a post-peak behaviour, which is identified by the sudden drop in the load-deflection curve in Figure 4. The ductility ratio of the specimen DS-C is 1.13 and strengthened specimens DS-GG and DS-BCG showed a marginal increase in ductility ratios with 1.25 and 1.53 respectively, contributing to 11 to 36 percent enhancement.

**Conclusion**

Three specimens – one control without any strengthening, one externally strengthened with BCG and the last one externally strengthened with SGU300 Geo-grids were subjected to the Diagonal Shear Test. The major observations from the test are as follows:

1. The external strengthening offered to the specimens controlled the sudden failure of the specimens unlike the one observed in the control specimen.
2. The load-bearing capacity increased between 2.2 to 2.7 times in the strengthened specimens.
3. The specimen strengthened with BCG attained the maximum load and the higher stiffness among every specimen utilized in the study.
4. Despite the significant enhancement in load-bearing capacity of the strengthened specimens, the ductility increased marginally between 11 to 36 percent of the ductility ratio of the control specimen.

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