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EXPERIMENTAL INVESTIGATION OF LOW COST STEEL WIRE MESH RETROFIT FOR STONE MASONRY IN MUD MORTAR

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Abstract

Stone Masonry in Mud Mortar is a widely used typology for construction of buildings for any use including schools in the Hill and Mountain range of the Himalayan region. It is the only local construction material abundant in this region. A major land portion of Nepal lies in the high mountains and rolling hills that accounts for about 83% of the total land area. Due to the economic advantage it offers and the difficulty in transporting external materials through the challenging Himalayan terrain, the stone in mud mortar has been the material of choice in rural schools for decades. In the 2015 Gorkha Earthquake, 60% of stone masonry schools were damaged. With little research done on these structures, countries like Nepal are in dire need for in depth study on the behavior and lateral load capacity of this typology. Most importantly, it is crucial to study the retrofitting measures that can be implemented to make these structures more resilient to earthquakes. Under the Seismic Safety and Resilience of Schools in Nepal (SAFER) project, which is a consortium of several organizations lead by University of Bristol with funding support from Global Challenges Research Fund (GCRF) and the Engineering and Physical Sciences Research Council (EPSRC), a series of full scale wall tests were conducted in Kathmandu on stone masonry walls. Due to the high variability in terms of the stones used and construction practices, a large number was designed to permit statistical processing of the results using locally available materials such as galvanized steel wire mesh for wall retrofit. Monotonic lateral tests were performed to study the effects of retrofit on the load vs displacement behavior. It was observed that after retrofit, the failure pattern changed to more distributed cracks instead of a large localized one. The results also showed that, despite the low cost of the intervention, a significant enhancement was achieved on both lateral capacity (324%) and deformability (131%) of the stone in mud masonry walls. The overall experimental campaign highlighted a practical method for setting up a low cost test in a developing country context, thus paving the way for increasing the number of the available experimental results in regions where such data are scarce.

Keywords: Stone in Mud Mortar; Retrofitting; Locally available materials; Cost Effective techniques

1. Introduction

Nepal is a country dominated by hilly terrain [1], where a majority of the population live in the rural regions [2]. Due to the difficulty in transportation of modern materials in the challenging geography, structures are usually built using locally available materials. As indicated by a 2011 census, a large portion of the building typology in Nepal, more than 40%, fall under low-strength load bearing masonry [3]. Stone being a strong and commonly available building material, stone-in-mud masonry is a ubiquitous material of choice in the country. Naturally, educational communities also adopt this same typology when it comes to building their schools.

Nepal ranks 11th in the world in terms of vulnerability to earthquakes [1] and has suffered a long history of devastating events [4]. The 2015 Gorkha Earthquake, where there were more than 8000 casualties, affected 8,242 public buildings and completely destroyed 25,134 classrooms [5]. A report published by the National Planning Commission estimated the loss to the educational infrastructure and physical assets exceeding $200 million due to the 2015 Gorkha Earthquake [6]. The Structural Integrity and Damage Assessment (SIDA) [7] report prepared for the 14 most affected district states that Load Bearing Masonry schools were the second most affected typology behind steel framed schools. It is also noted that even in the steel framed schools, masonry damage was commonly seen whereas damage to the steel structures were rare. Studies have shown that a majority of masonry building stock has concerning seismic deficiencies [8]–[10], largely because they are built by the owners themselves or using local labor, who do not necessarily have any formal training and hands on knowledge in good construction practice [11]. The buildings are not able to exhibit a proper diaphragm behavior [12]. With sparse wall to wall connection between orthogonal walls and lack of seismic detailing such as tie rods, anchors and ring beams [13], out of plane was the most common mode of failure of masonry walls observed in the 2015 earthquake [5], [7], [14], which is the most serious mode in terms of threat to life-safety [15].

To improve the seismic performance of masonry structures, retrofitting measures have long been devised. One of the common methods include providing retrofitting measures using steel Welded Wire Meshing (WWM) at critical and strategic locations. The cost of this method is approximately one third the cost of reconstruction [16]. But as of now, there has been very little research on the qualitative and quantitative benefits of the retrofitting measures to stone in mud masonry using these techniques. This research was aimed to study the improvement in the lateral load behavior of stone in mud masonry walls upon retrofitting using low cost steel wire mesh retrofit.

2. Studies on Out-of-Plane behavior of Low Strength Masonry and Retrofit

Experimental campaigns have been done to characterize the out of plane behavior of masonry walls in the past. Ferrieira et al. [17] has documented the out of plane laboratory tests starting from 1984 to 2013 with static and dynamic tests giving useful understanding of masonry wall behavior over the years. One of the early experiments done in the 1980s, demonstrated the dependence of the out of plane collapse on peak velocities and the input at the top and base of the walls. Tomazevic, Weiss and Velechovsky, in 1991, concluded the cracking of out-of-plane walls and subsequent failure caused in the presence of flexible floors as metioned in Ferreira et al. [17] . The study done by Doherty et al. [18] supported the thesis that out of plane walls were more susceptible to displacement rather than acceleration. At the University of Adelaide, Lam et al. [19] carried out static and dynamic tests on 14 one way regular brick URM wall panels. Using both simple pulses and earthquake ground motions, the study again outlined response spectral displacement as being a much better measure of ultimate performance as opposed to acceleration.

An extensive series of tests with 42 configurations of scaled dry stone masonry walls was conducted at University of Pavia, Italy in 2004 by Restrepo-Velez as mentioned in Ferreira et al. [17]. One of the objectives was to develop analytical expressions to compute collapse multiplier and ultimate static displacement for different mechanisms. Using 8 full scale URM walls, Griffith et al. [20] used a system of airbags to subject the walls to cyclic face loads. A contribution of the vertical pre compression to the post peak strength and displacement capacity was observed. Derakhshan and Ingham [21] tested full scale URM wall in out-of-plane uniform static loading which highlighted the influence of vertical pre compression and its influence on the tri linear model shape. In 2011, D'Ayala and Shi [22] conducted a series of shake table tests and presented the possibility to derive a rational model of cracking and damage behavior of historic masonry, correlated to a few limited geometric and structural parameters.

Al Shawa et al. [23] performed shake table tests on U shaped configuration of tuff masonry. From 34 shake table tests, it showed a substantial stability of monolithic facades after undergoing rocking, when they are bonded well in the transverse direction. Similarly, Costa et al. [24] also observed a considerable energy dissipation and displacement capacity reserve after the mechanism was triggered. Ferreira et al. [25] conducted full scale tests on six sacco stone masonry specimens to characterize the out of plane behavior of traditional unreinforced stone masonry. Two different loading techniques i.e. a system of airbags and an out of plane horizontal line load at the top, were used under three vertical pre-compression states. The test results showed good energy dissipation capability and a significant ultimate displacement capacities relative to the wall thickness within a range of 26% to 45%.

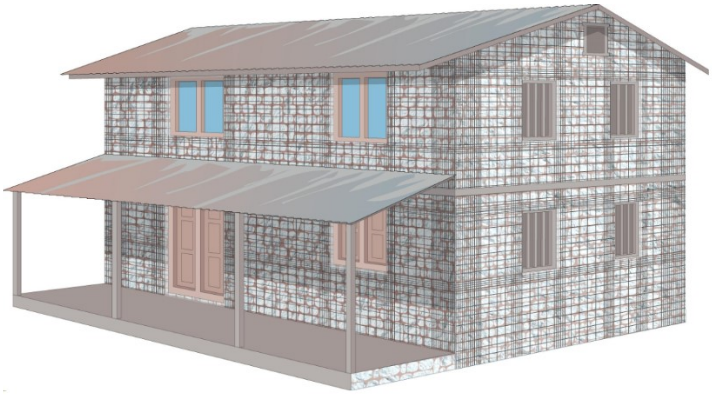
Candeias et al. [26] performed shake table tests on two U shaped mockups made of clay bricks and stone masonry in 2016. The study also incorporated the examination of the effect of torsion on the facades by utilizing dissimilar transverse walls. The tests showed that lower ground motion intensity was found to cause only damage to facades and rocking behavior actually helped to prevent collapse of the stone model. In addition, the stone model somehow showed more ductility. In the same year, Graziotti et al. [27], did an extensive experimental campaign performing out of plane shaking table tests on cavity wall and single leaf components where all the walls suffered one-way vertical bending/rocking failure forming hinges at top, bottom and at mid-height. Degli Abbati and Lagomarsino [28] conducted static and dynamic tests in 2017 on three models and studied the reliability of the rigid block model in describing the rocking response of free standing masonry elements.

There has been substantial increase in the number of research on retrofit of masonry in the recent years. Dadras et al. [29] reviewed and compared the experimental tests done in the past to study the enhancement on in-plane ductility resulting from retrofitting on unreinforced masonry walls. The results from all the tests were promising with an average ductility factor of 4.68. Placencia and Paredes [30] tested the effectiveness of wire meshes and mortar for confined masonry system in 2017 for two story Ecuadorian houses. The study showed good results in enhancing seismic performance. Papanicolaou et al. [31], [32] in 2008 and 2011 compared the effectiveness of enhancing the out of plane behavior of URM walls by using Textile Reinforced Mortar (TRM) vs Fibre Reinforced Polymer (FRP) and concluded that using TRM was better at improving the performance. An extensive in-situ out of plane test was conducted by Derakshan et al. [33] in 2014 showing the better performance of walls upon seismic enhancements and retrofitting, along with showing importance of in situ testing. There have been comprehensive studies in recent years testing various materials for retrofitting to improve out of plane performance of URM walls like Bellini et al. [34] with Fibre Reinforced Cementitious Matrix (FRCM), Gattesco and Boem [35] with Glass Fibre Reinforced Polymer (GFRP) and De Santis et. al [36] with Textile Reinforced Mortar (TRM). While there is lack of a dedicated study on out of plane behavior of unreinforced stone masonry walls in mud mortar retrofitted with steel Welded Wire Mesh (WWM).

3. Practice of Welded Wire Mesh (WWM) Retrofit in Nepal

The steel Welded Wire Mesh (WWM) is applied as per design on the designated structures. Splints and bandages are typically designed alongside openings and portions of the masonry as required by design. At other locations, Galvanized Iron (GI) wires placed farther apart are provided to prevent local failure of masonry and prevent spalling of masonry units. Through wires or semi through connectors are provided on the walls at certain intervals in a staggered fashion, tying the inner and outer meshes together. Plaster of 20mm and 30mm thickness is applied at the inner and outer face respectively to protect the mesh from corrosion. A reinforced concrete tie member is provided at the base of the walls to ensure fixity. The general process in implementation of retrofitting work using steel wire mesh includes 1) Removal of existing plaster from walls in the proposed area 2) Raking out mortar joints to 15-25 mm depth, cleaning and wetting the surface 3) Excavating the soil for tie beam and laying the reinforcement 4) Applying the WWM on walls and providing anchor rods or through wires to tie inner and outer WWM firmly with the wall 5) Anchoring the WWM to the rebar of the tie beam 6) Concreting the tie beam 7) Applying plaster to the wall 8) Curing

Figure 1 A house with WWM laid over and ready for plaster



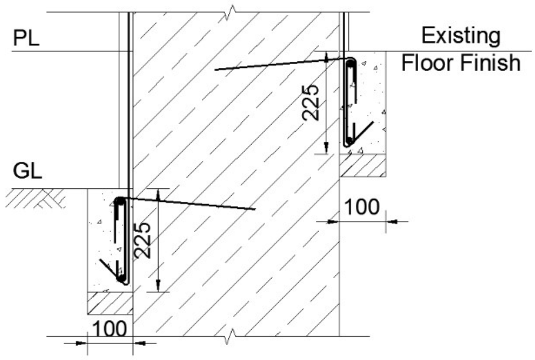


Figure 2 Illustration of a typical WWM layout (left); Typical RC tie member at the base showing through anchor rods (right)

4. Pilot Test

The pilot tests were conducted at the Institute of Engineering Laboratory of Tribhuvan University on April 2019. Two walls were constructed where one was unretrofitted and the other was retrofitted using WWM. The pushover test yielded good qualitative results and demonstration of the effectiveness in enhancing strength and ductility of the walls upon retrofitting was carried out.

4.1 Test Setup

A pump action manual hydraulic jack of 100KN capacity was used for applying the load at 2/3rd height of the wall. Lasers were used to continuously monitor the deflection suffered as load was increased. To prevent overturning of the retrofitted wall, a steel I-section was provided at the bottom of the wall. Both the mesh and GI wires (to prevent local failure) were passed below the I-section and continued to retrofit both faces (refer Figure 5).

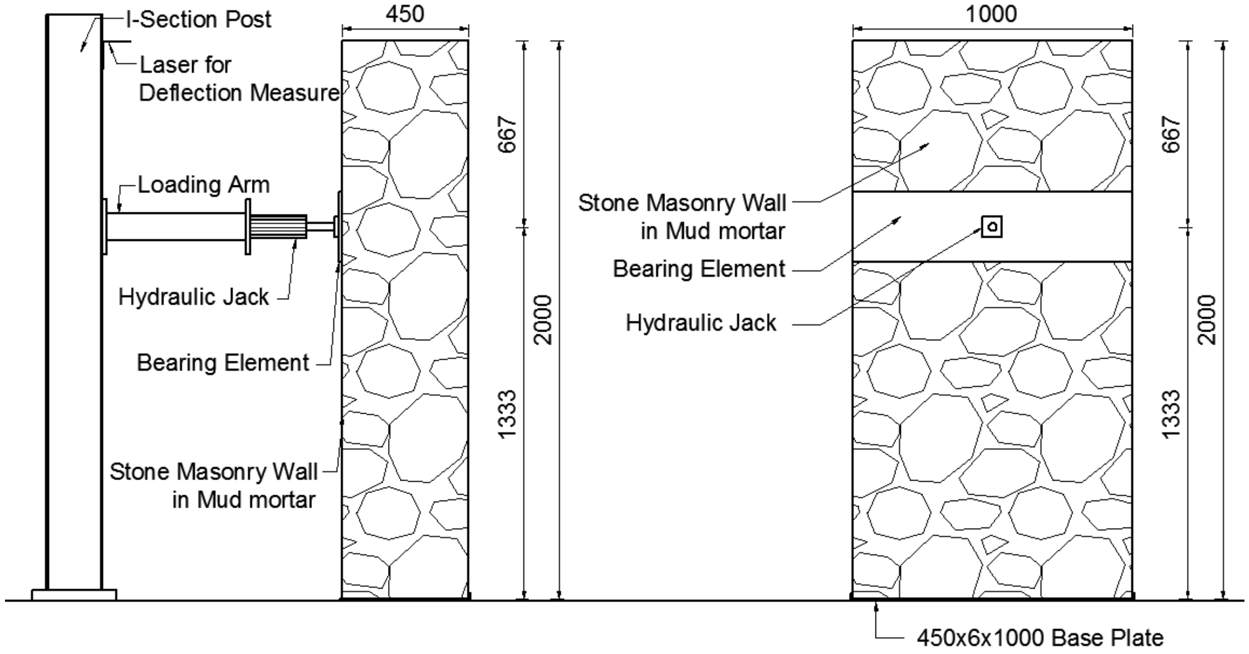


Figure 3 Setup for Pushover Pilot test

4.2 Test Walls

Both the walls were of dimension 1m (length) X 0.45m (width) X 2m (height). The width of the walls were chosen in reference to typical stone masonry schools in Nepal. The stones were brought from the Nuwakot district (North of Kathmandu) where typical stone structures in mud mortar exist, usually built with stones from local quarries. The masons employed for construction were from the mountainous Solukhumbu region, who had experience of building stone in mud houses in their village. The stones were chiseled into random sizes manually. No through-stones were used, so as to replicate walls with no seismic enhancements.

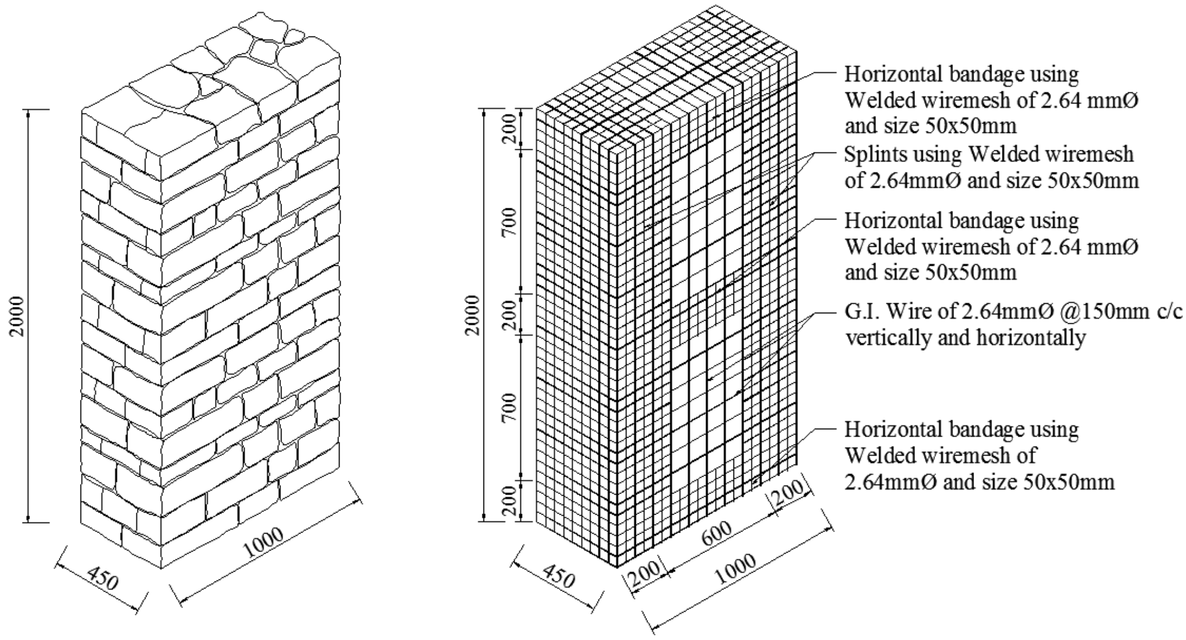


Figure 4 Isometric view of the test wall (left); WWM retrofitting arrangement (right)

The welded wire mesh used for retrofitting comprised of 2.64mm diameter steel wires welded to form a mesh at a spacing of 50mm. The retrofitted wall was provided with vertical splints of width 200mm at left and right extreme ends of the wall. Three horizontal bands of width 200mm were provided at top, middle and bottom of the wall along its height. In the regions, where the splints and bands were not provided, GI wires of the same diameter (2.64mm) were applied at a spacing of 150mm. These wires are used to prevent local failure and spalling of stones. The fixity of the retrofitted wall at the base was ensured by welding steel pipes to an I-section steel beam that was embedded within the wall and flushed to one side of the wall. The welded steel pipes were then connected to a permanent steel structure of the laboratory. This technique to provide fixity was devised because a foundation could not be excavated on the laboratory floor.

4.3 Pilot Test Observations

For the unretrofitted wall, the first observable crack appeared at a height of 0.33m at a top deflection of 49 mm. The crack propagated to form a peak shape across the length of the wall on the tension side (refer Figure 5). The damage suffered was highly localized to the region. Due to the limitation of the arm length of the hydraulic jack, the test had to be halted momentarily at multiple occasions when the arm had to be replaced with successive longer ones. At these points, the wall experienced unloading. The wall failed after suffering a maximum top deflection of 321mm at 4.07 KN.

For the retrofitted wall, cracks were more distributed throughout the lower half of the wall. The first observable crack occurred at a top displacement of 67mm. Collapse of the wall finally occurred due to the failure of the bottom GI wires spaced at 150mm spacing along with the vertical wires of splints. The final top deflection suffered before collapse was significantly improved to a value of 769mm at 7.27 KN. These load values were calculated after correcting the error from the analog dial using a digital load cell. Therefore, the absolute value may not be reliable although the figures give a good idea of the relative enhancement in strength.

Figure 5 Damage Pattern on the unretrofitted (left) and retrofitted (right) wall

5. Main Test

Learning from the Pilot test, it was decided to devise a mechanism that can apply a continuous monotonic deflection to the test model. A pulldown setup was designed and constructed at the NSET premises using a chain pulley for the application of load. For load measurement, a much accurate digital load cell was used and lasers were used for deflection measurement.

Since the main test was to be done with semi dressed stones, the stones were brought from the Dolakha district (north-east of Kathmandu), where quarries offering more regularly shaped stones exist. Construction masons were supervised by an experienced head technician having extensive experience in construction of semi dressed stone in mud mortar.

5.1 Test Setup

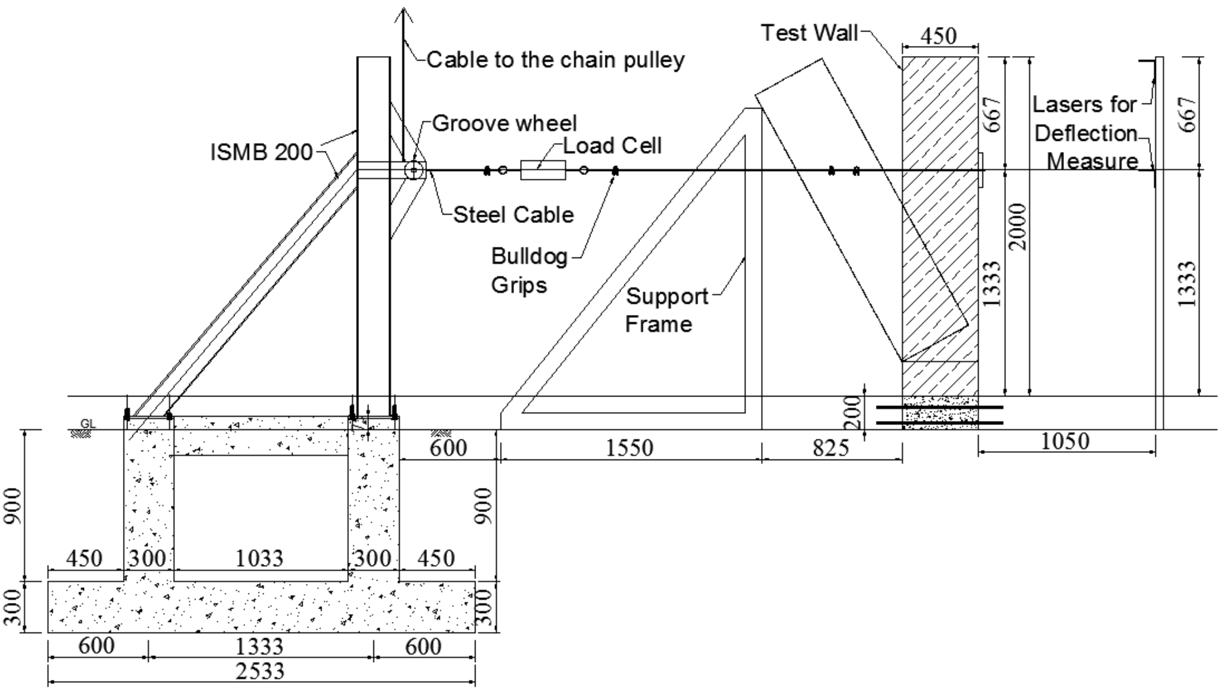


Figure 6 Setup for Pulldown test

Figure 7 Cross section of the RC pedestal (left); Laid rebar before casting (mid); Casted RC Pedestals (right)

A 400mm thick raft-like stone foundation was constructed that prevented any kind of uplifting. Two M20 RC pedestals were designed to serve as a base for each wall. Multiple protruding rebar were left out for the wire mesh to be connected. The transverse U shaped rebar of the pedestal were continued to the base of the foundation and anchored with 100mm thick M20 concrete (refer Figure 7).

Figure 8 Wire mesh tied and welded to the rebar of the pedestal (left); Connections covered with 50mm of 1:6 plaster (right)

A support frame was placed beside the retrofitted wall catering to the need of protecting the load cell and due to safety concerns. Learning from the pilot test, it was anticipated that the retrofitted wall would fail as a unit instead of disintegrating into smaller chunks. Therefore, the need for the support frame was realized. The distance of the support frame was carefully selected so that the retrofitted wall rested on the frame only after suffering complete failure.

Figure 9 Chain pulley arrangement (left); Pulldown setup for retrofitted wall (mid); Lasers placed pointing towards the retrofitted wall, using the unretrofitted wall as support (right)

High strength steel cables of 10mm diameter were used for pulling, forming loops through and around the walls and around the eye bolt ends of the load cell. The chain pulley was setup in such a way that the upward movement of the hook end would be translated to a horizontal pulling movement via a grooved wheel, connected to the reaction frame (also refer Figure 6). The chain pulley was operated by hand by a technician under the supervision of a structural engineer to maintain a uniform rate of displacement. The least count of the load cell was 0.01KN and that of the laser was 1mm. The test walls were painted white for better visibility of the cracks.

6. Results

The maximum load for the retrofitted wall was 11.87KN, which was 2.8KN for the unretrofitted wall. Similarly, the final top deflection improved to 650mm from 281mm, upon retrofitting. The load vs displacement curve obtained after integrating load and displacement data is shown below:

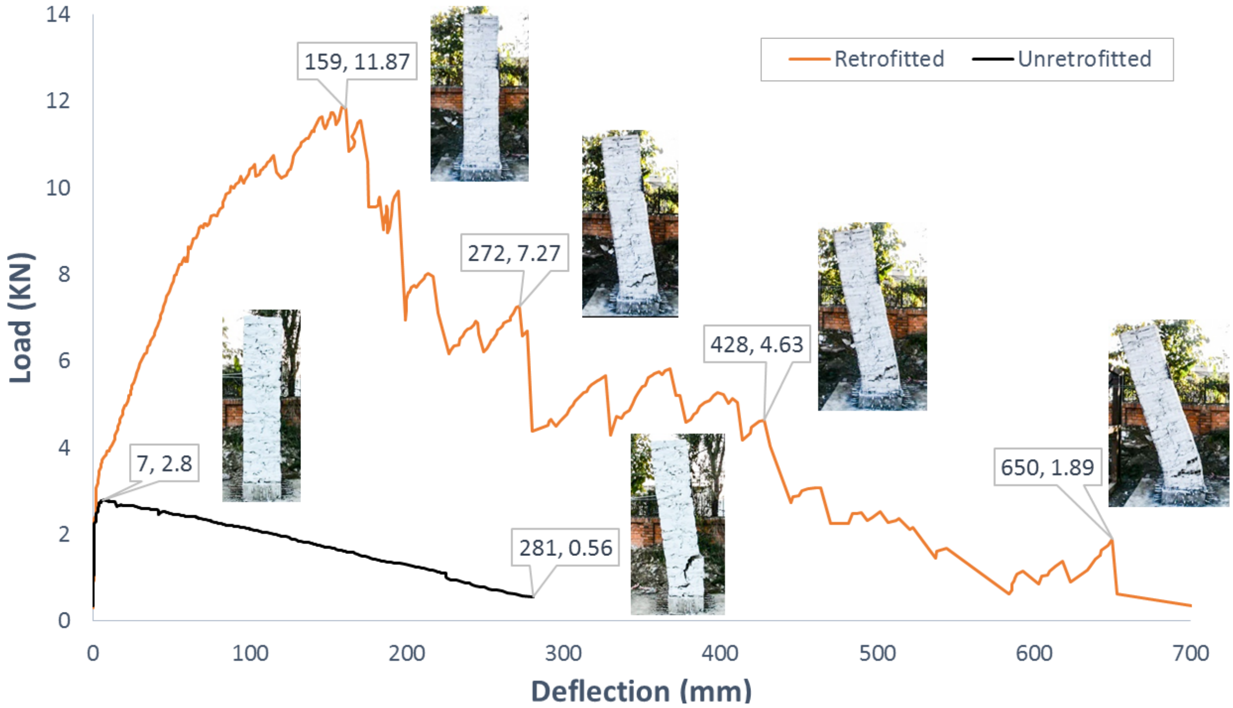


Figure 10 Load vs Displacement curves of the Retrofitted and Unretrofitted walls (pictures correspond to the top deflection and load values to their immediate left)

Figure 11 Final deflected shape of the unretrofitted (left) and retrofitted (right) wall before collapse

Both the walls exhibited a high stiffness on initial loading with both walls withstanding a load of 2.5KN within 1 mm of deflection. For the unretrofitted wall, peak load of 2.8KN was reached at 5 mm of deflection, after which the load displacement curve suffered a smooth downward decline. The cracks were highly localized with major ones appearing on the tension side at a height of 0.5m. Upon increasing the load, the horizontal crack propagated to the front-lower level up to the interface between the lowermost and second lowermost stone unit (refer Figure 11). The average rate of top displacement for the unretrofitted wall was 0.7 mm/sec and the loading increased at an average rate of 0.01KN/sec till the peak load.

For the retrofitted wall, the high stiffness continued till a loading of 3.75KN (6mm deflection), after this the stiffness decreased slightly till a loading of 8.25KN (56mm deflection). The curve is smooth up till this point after which, minor undulations can be seen on curve. The undulations are because of the stone layers slipping at the unit-mortar interface. The curve rises to its peak load of 11.87KN at a deflection of 159mm. At this point, clear visible cracks had formed at a height of 0.25m (1/8th the height of the wall). The curve follows an overall downward trend after this point. Sharp falls and rises follow, where peaks can be seen when different parts of the mesh is activated one after another. Sharp falls can also be seen when the mesh wires yield, causing the welds to break, activating other consecutive regions of the mesh. The most yielding was observed at the connection between the GI wires spaced at 150mm c/c and the lowermost horizontal band, where the welds of topmost horizontal wires of the bands had broken. This happened because the GI wires spaced at 150mm c/c were not fixed to the pedestal rebar and were tied to the horizontal band. In the actual field condition, these wires are anchored to the rebar of the RC tie beam. Ultimate failure of the wall took place at a top deflection of 650mm (1.89KN) caused due to the failure of wires of splints. The average rate of top displacement for the whole loading sequence was 1mm/sec and the loading increased at an average rate of 0.05KN/sec till the peak load.

7. Conclusion

Lateral load tests for unretrofitted and retrofitted stone in mud masonry walls were conducted for random rubble stones (pilot test) and semi dressed stones (main test) in mud mortar. The walls represented typical Nepali construction in both residential and educational buildings. The conclusion that can be drawn has its limitations as very few walls were tested. However, retrofitting of the stone in mud walls using WWM had a significant impact in improving both the lateral load capacity and deformability in both the tests. For a method that costs less than a third of that needed for reconstruction of the same structure, the results look compelling. For the main test, the peak load increased by 324% and the final deflection increased by 131% compared to that for the unretrofitted wall. This result was observed despite the retrofitting technique missing some important details like the farther spaced GI wires not being anchored to the bottom RC element but rather tied to the bottom horizontal wire mesh band. The retrofitted wall was able to deflect by 650mm before failure, which is more than the thickness of the wall itself (450mm), substantially improving the deformability as well. The overall study also highlights a practical method for setting up a low cost test in a developing country context, thus paving the way for broadening the knowledge base for low strength masonry through further experimental investigations in these regions.

Further avenues of research can be testing of other typologies like dry stone masonry walls. The confining effect of plaster that can cater a more uniform activation of the mesh can also explored. Other possible cost effective methods and retrofit techniques using locally available materials can also be studied.

8. Acknowledgement

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