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SEISMIC RETROFITTING OF EXISTING UNREINFORCED MASONRY BUILDINGS WITH SEISMIC BAND: FULL SCALE TESTING AND FRAGILITY ANALYSIS

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Abstract

Unreinforced brick masonry (URBM) buildings are very common in many countries. Historically, such structures have shown very poor seismic performance (e.g., during the 2012 Nepal Earthquake). This is owing to the brittle nature of masonry, its poor tensile strength, as well as poor out-of-plane stability and in-plane shear capacity of URBM walls. The seismic performance of such structures may degrade further as a result of weak mortar, which is often utilized due to poor construction practice. Various retrofitting strategies have been proposed in literature to enhance the seismic performance of URBM buildings. This study focuses on performance enhancement of existing URBM buildings by introducing reinforced concrete seismic bands at strategic locations. The proposed easy-to-implement strategy involves: (a) cutting of small grooves on the walls, and (b) filling up these grooves with ready-mix concrete along with nominal reinforcements. To evaluate the efficiency of such bands in terms of performance enhancement, two full-scale single-story identical URBM buildings were constructed in the Structural Engineering Laboratory of IIT Kanpur. One of the building was then retrofitted with the seismic bands as mentioned. The performance of the two buildings were then evaluated by performing slow cyclic tests as per FEMA 461 loading protocol using 1000 kN hydraulic actuators. The results of these tests demonstrated that, while the URBM structure without bands failed in shear at the joints for comparatively lower load, the retrofitted structure continued to carry higher loads up to a significantly larger lateral drift. Utilizing these test results, simplified models of both control and retrofitted specimens are develoed and nonlinear time history analyses are carried out under a suite of earthquake ground motions representing a high seismic zone (Zone IV) of the Indian Code. Seismic fragility curves are then developed for both (URBM and retrofitted) buildings in order to understand seismic performance enhancement due to presence of seismic bands. The results of the study show the potential of the proposed method as an economic, minimally intrusive, easily implementable retrofitting approach, which can be employed for existing seismically deficient URBM buildings.

Keywords: Existing URBM structures; Seismic retrofitting; Seismic band, Full scale testing; Fragility analysis

1. Introduction

Unreinforced Brick Masonry (URBM) forms one of the most prominent types of contruction for rural and heritage structures because of its wide availability, ease of construction, good performance and durability qualities. Since ~3500 B.C.E, brick masonry remained as the most common conventional form of construction for various countries all over the world. In India, 48.06% (as per census of India, 2011) of the entire building stock is contructed using URBM. This type of construction is very popular in urban as well as rural areas.

In general, brick masonry is a heterogeneous material, constituted by burnt brick units bonded together with mortar. Brick and mortar are in the group of quasi-brittle materials, which exhibit poor tensile strength and fail by progressive growth of internal crack. Shrinkage during preparation stages often induces micro-cracks, porous inclusions etc., even prior to any type of loading, which marks the crack initiation. Thus, quality of bonding material and ratio of the sand to the bonding agent usually govern mortar or interface strength. In-plane shear strength of masonry provided by the interface is of utmost importance for seismic resistance. Common failures observed in masonry structures due to seismic loading include out-of-plane collapse due to corner junction failures, shear cracks at corners of wall openings etc. A complete collapse of the structure may take place due to overturning of wall as a result of poor interface at wall-to-wall, roof-to-wall, and floor-to-wall connections. Owing to this, there can be separation of walls, roofs and floors, leading to sudden catastrophic disintegration of masonry structures. Past earthquakes namely, Imperial Valley (1979), Loma Prieta (1989), Latur (1993), Northridge (1994), Kobe (1995), Chamoli (1999), Bhuj (2001), Bam (2003), Kashmir (2005), Sumatra (2007), Sichuan (China) (2008), Durgapur (2008), Haiti (2010), Sikkim (2011), India-Nepal border (2011), Nepal (2012) etc., have revealed the seismic vulnerability of existing masonry structures. Traditional earthquake-resistant design philosophy demands no collapse of structures to save life and property inside or adjoining the building. Thus, earthquake resistant attributes of existing URBM buildings need to be improved. As per modern design codes, existing URBM buildings need to be strengthened using engineering measures for enhanced ductility without compromising the strength.

Many conventional methods are prevalent for improving the seismic attributes of URBM structures. However, sustainability, buildability and economy have been the main focus of the research dealing with good strengthening techniques [1]. A particular strengthening technique may not satisfy all these characteristics. It is now well accepted that confined masonry structures exhibite improved seismic performance. URBM walls using reinforced tie columns at corners and intersections have shown enhanced energy dissipation and in-plane deformation capacity [2]. Confinement of openings on all sides showed a better performance (greater than 40%) than the regular infill walls with only lintel beam over openings. Under lateral loading, evidences of performance enhancement due to proper confinement in comparison with other confinement configurations or schemes are observed from the uniform distribution of cracks over the entire masonry wall [3]. Fig. 1 shows a schematic of the representative details of a typical confined masonry building configuration. Past earthquakes like Chillan earthquake (1939), Bam earthquake (2003), Peru earthquake (2007) etc. have demonstrated the survival of confined masonry building, whereas many URBM buildings suffered total collapse. For confined masonry structures, the parameters including in-plane shear capacity, out-of-plane stability and ductility are found to be higher in comparison to URBM building. Construction of confined masonry is generally simple except for the fact that reinforced concrete (RC) confining elements are needed to be constructed at the same time, which may require additional training for masons. Further, in such structures, the confining RC elements are provided at the time of construction itself, not later. The introduction of RC confining elements in existing URBM buildings remains unpopular. Although a few studies are available on the introduction of semi-conifning RC bands in existing URBM buildings, so far, such schemes have not been intevstigated thoroughly and systematically.

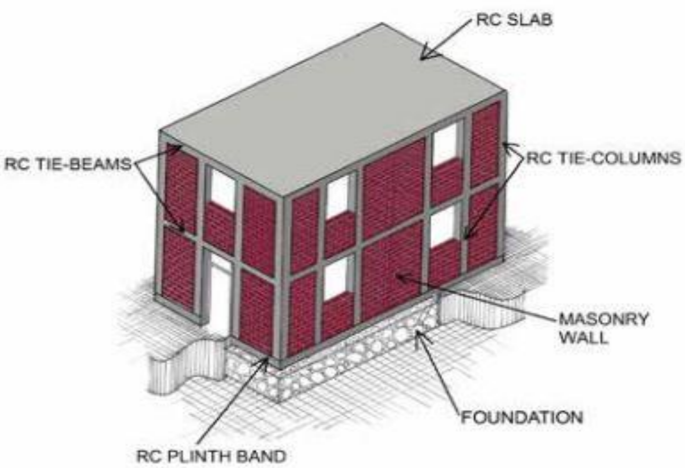


Fig. 1 – Components of confined masonry[4]

1.1 Scope of the Study

This study aims on performance enhancement of existing URBM buildings by introducing semi-confining reinforced concrete seismic bands at strategic locations. Two full-scale single-story identical URBM buildings were constructed in the Structural Engineering Laboratory of IIT Kanpur. One of the building was then retrofitted with the seismic bands. The performance of the two buildings were then evaluated by performing slow cyclic tests as per FEMA 461 loading protocol. Further, utilizing the test results, simplified models of both control and retrofitted specimens are develoed and nonlinear time history analyses are carried out under a suite of earthquake ground motions representing high seismic zones (Zone IV and Zone V) of the Indian Code. Seismic fragility curves are then developed for both (control and retrofitted) buildings in order to understand seismic performance enhancement due to presence of seismic bands.

2. Experimental Programme

2.1 Structural Configuration of masonry models

Two full-scale identical specimens of URBM were constructed over the strong floor of the Pseduo Dynamic Testing Facility of the Structural Engineering Laboratory at IIT Kanpur, India. The masonry buildings were 3.3 m × 2.8 m in plan and 3.0 m in height, built as per prevailing construction practices in India. The walls of the buildings were 210 mm thick, made using locally available solid burnt clay brick units of size 225 mm × 105 mm × 70 mm, and using comparatively weak morter with cement to sand ratio of 1:6. The logic behind using such ratio for the morter is that such ratio is very popular in the construction of residential units in India due to economic constraints. The building was constructed on 150 mm thick RCC foundation of Grade M25 (as per IS 456), resting on the strong floor, as shown in Fig. 2. In-plane walls (in the direction of loading) did not possess any openings, whereas the out-of-plane walls (perpending to the loading) had a window opening on the actuator side and a door opening on the opposite side. After the construction was complete, one of the units was retrofitted using semi-confining horizontal and vertical reinforced concrete (RC) seismic bands as shown in Figs. 3 and 4. Henceforth, these specimens will be referred to as URBM (without any seismic band) and S_URBM (strengthened with seismic bands) specimens.

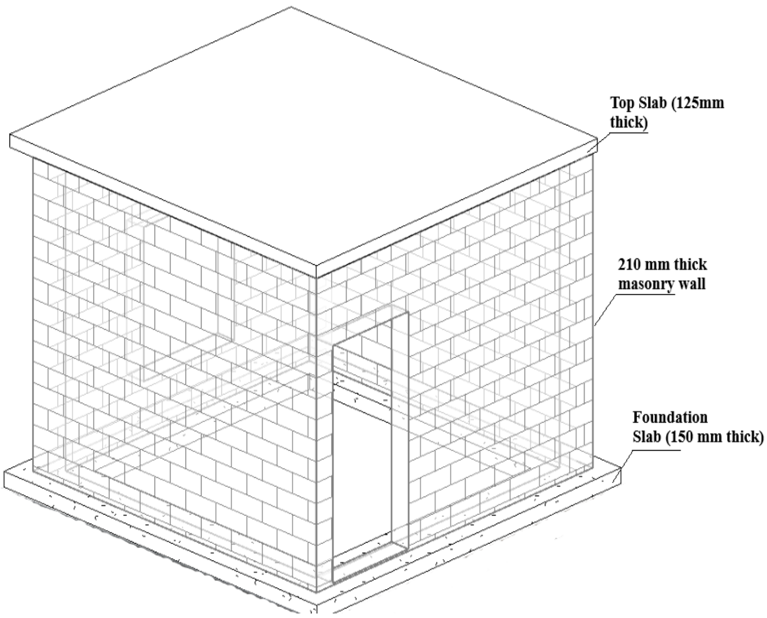
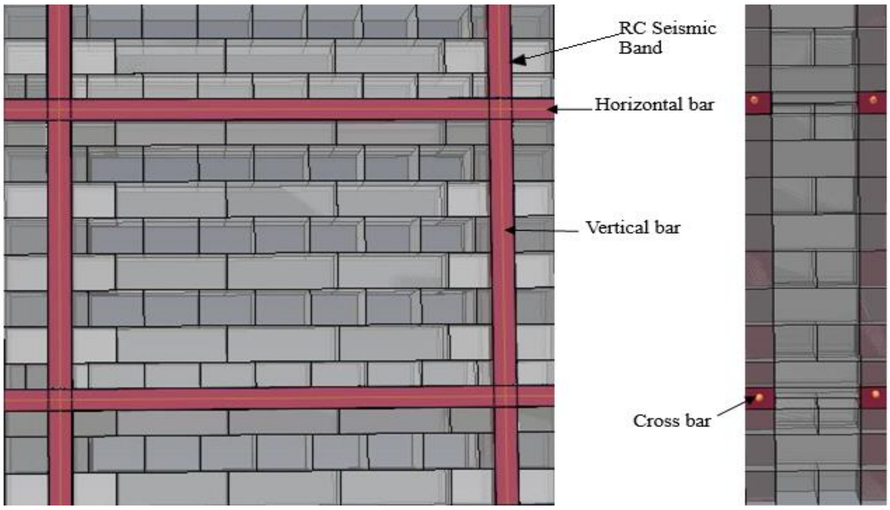


Fig. 2 – Schematic diagrams showing URBM specimens constructed on strong test floor

2.2 Illustration of strengthening technique adopted

The strengthening technique adopted was based on the confined masonry model, which has demonstrated improved seismic behavior during the past earthquakes. Schematic diagram illustrating the adopted technique is shown in Fig. 4. At first, grooves of size 50 mm × 50 mm were cut at the required locations on both sides (outer and inner) of the wall. Then, steel rebars were provided in the seismic bands, one bar per band, with anchor ties between inner and outer bands (links through the wall). The gaps in these bands were then filled with cementitious micro-concrete on both sides of masonry wall specimens. Thus, both vertical and horizontal seismic bands have only one bar embedded in it. The details of materials used are provided in the next sub-section. The horizontal seismic bands were provided throughout at: (i) 0.5m below the roof slab, and (ii) 0.5 above the foundation level, as shown in Fig. 4. The vertical seismic bands were provided at some specific distances along the wall, at four corner sides and around the openings, as shown in Fig. 4. All the vertical seismic bands were connected to the roof slab and base slab by extending the grooves to the roof slab and foundation.



Elevation Vertical Section

Fig. 3 – Schematic diagram showing the main components of semi-confined masonry wall

2.3 Material Properties

Material level experiments were conducted for evaluating the physical properties of masonry (for brick and mortar) and concrete used. The compressive strength and water absorption of brick units were obtained by performing experiments as per codal provisions of IS 3495: 1992 (Part 1 and Part 2) [4, 5]. The compressive strength and water absorption estimated from the experiments were 23.5 MPa and 11.6 %, respectively. An average 28-days compressive strength of mortar was evaluated by using 50 mm cubes as per ASTM C109 / C109M [6]. This average value of the mortar's compressive strength was obtained as 6.9 MPa. Compressive strength of masonry was determined by conducting tests on five brick tall masonry prisms with 10 mm thick cement-sand mortar joint of 1:6 ratio as per IS 1905 (BIS 1987) [7]. The average value of the compressive strength of the masonry was obtained as 4.19 MPa. The diagonal tensile strength of masonry was determined as per ASTM E 519-07 [8], and the average value was obtained as 0.11 MPa. For RC seismic bands, HYSD Bars of 12 mm dia with minimum 0.2 % yield stress of 500 MPa was used. Cementious micro-concrete was used for inducing rapid strength enhancement, possessing compressive strength of 30 MPa and 50 MPa, if tested at 3 days and 50 days, respectively.

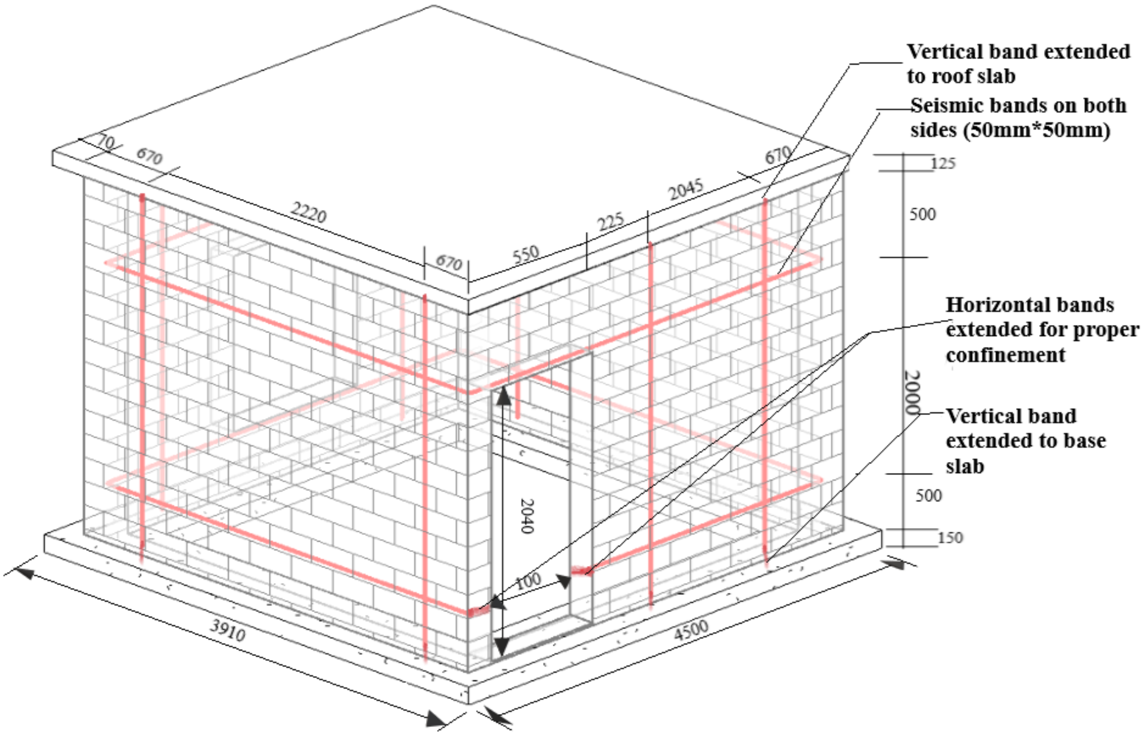


Fig. 4 – Schemaitc diagram of S_URBM specimen with horizontal and vertical seismic bands at strategic locations (All dimensions in mm)

2.4 Experimental Set-Up and Loading Mechanism

Fig. 5 shows the full-scale masonry structures (both URBM and S_ URBM) subjected to displacement controlled reverse cyclic loads that are applied laterally at the roof level. They were built on an RC foundation, which was cast on and firmly fastened to the strong floor using 12 high strength bolts of diameter 70 mm. The loading was applied using 1000 kN /500 mm stroke MTS servo-hydraulic actuators, which were conncted to the roof slabs. The integrated strong floor and wall provided the reaction frame system for application of the lateral loading. Rolled Steel I-sections (beam sections) were placed at the middle of the top slab in order to transfer the load to the entire roof-slab. High strength steel plates were placed at the corners of the masonry walls (both bottom and top) and channel sections were used at the middle to prevent unwanted failure mechanisms. Fig. 6 shows a schematic diagram of the loading system used for the testing.

Fig.5 – Full-scale masonry structures : a) Unreinforced Brick Masonry (URBM), and b) Strengthened/Retrofitted Brick Masonry (S_URBM)

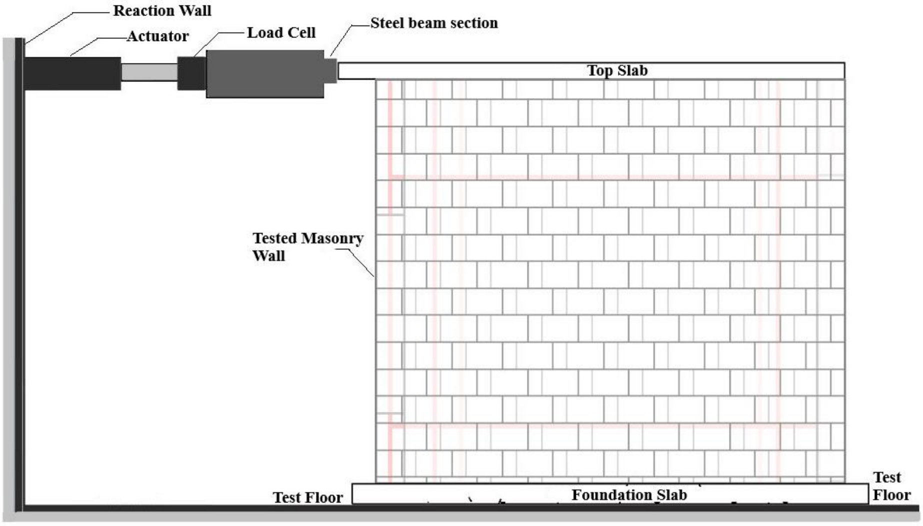


Fig.6 – Schematic diagram showing loading system

2.4 Instrumentation and Data Acquisition

Instrumentation scheme used for the slow-cyclic testing of both structures are shown in Fig. 7. In addition to the load cell and displacement transducer embedded in the MTS actuator, five pairs of Linear Variable Differential Transformers (LVDTs) were installed on the front side (loading side) and one LVDT was used on the backside to measure the displacements of both walls (along the loading direction) at different heights and the roof level as shown in Fig. 7. Apart from wall movements, the instrumentation locations were also selected in order to monitor if there is any slip between the foundation and walls or between the walls and roof slab. Two wire potentiometers were also used to measure the diagonal movement of the walls (see Fig. 7). All the instruments used were of reputed make (https://measurementsensors.honeywell.com/). Data from all sensors were collected using a National Instrument (www.ni.com) data acquision system.

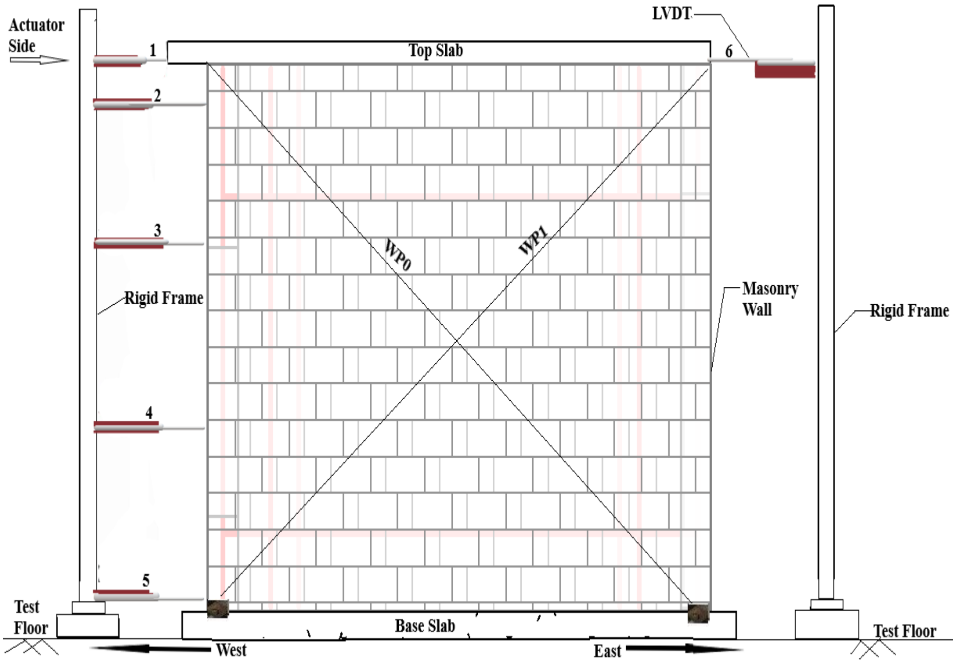
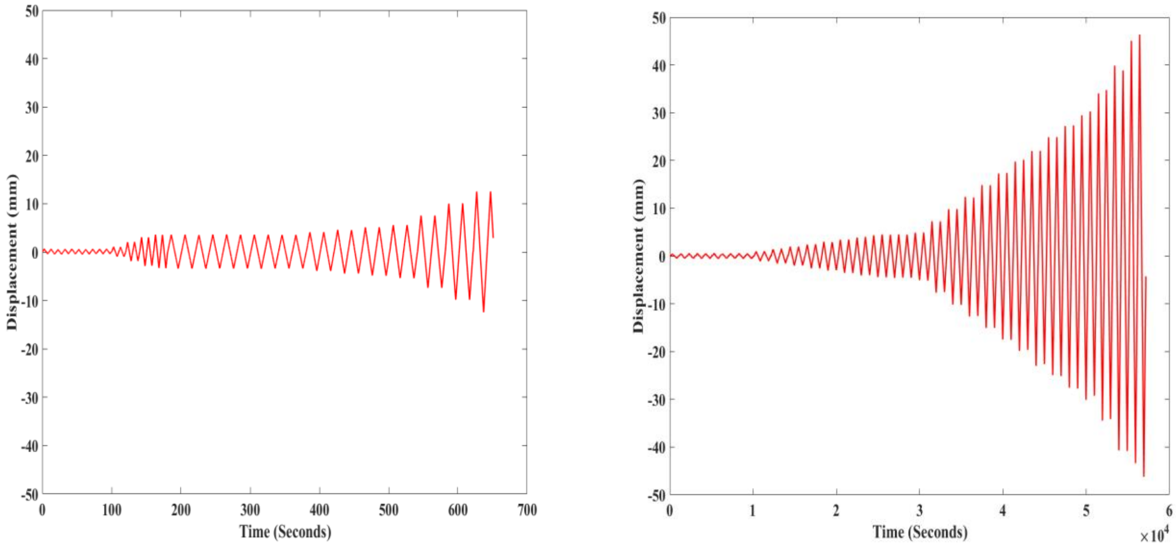


Fig.7 – Positions of LVDTs and wire Potentiometers in URBM and S_URBM structures (Right face of wall from actuator side. Similar configuration was adopted for left face too.)

2.5 Loading Protocol

The test specimens were subjected to slow cyclic displacements with increasing amplitudes, which follows a pre-determined pattern for replicating seismic effects. It is mainly aimed at estimation of damage levels and for relating damage states with concerned demands. Recommended loading history appropriate for hysteretic testing of a single specimen as per FEMA 461 [9] was adopted. Fig. 8 shows the actual loading history used in the current study. The loading history consists of repeated cycles of step-wise increasing displacement amplitudes. The lateral load from the actuator was applied to the roof slab at a very slow rate, ranging from 0.5 mm/min for early cycles to 1 mm/min for later cycles. It may be noted that the URBM specimen showed significant cracking and deformation for which the experiment had to be stopped. For the S_URBM specimen, the loading was continued for cycles with much larger amplitudes.



(a) (b)

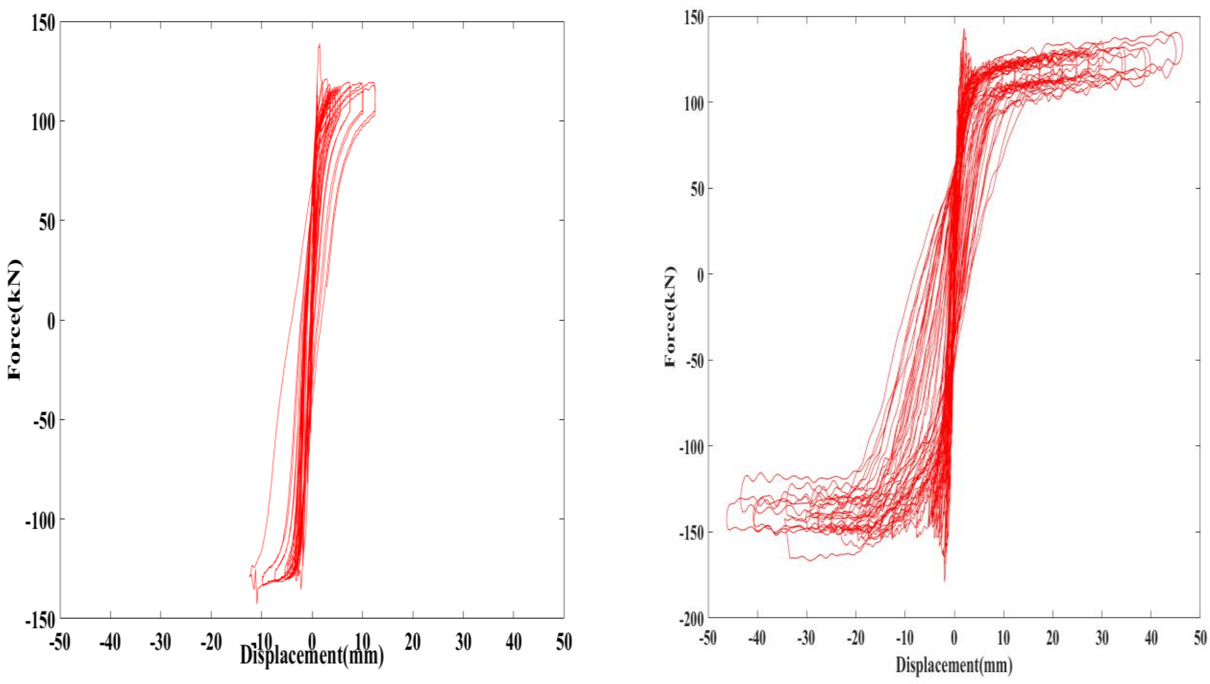
Fig.8 – Recorded loading history for: (a) URBM and (b) S_URBM specimens (as per FEMA 461)

2.6 Experimental Results

Seismic performances of both the URBM and S_URBM specimens were studied with respect to the strength provided by these structures, ductility, crack pattern etc. The performances were observed through the force-deformation curve, stiffness degradation and strength deterioration, as well as from the visual crack paterns. Most importantly, it was felt from the crack patterns and movement of the walls that the URBM specimen was about to fail suddenly by the sliding or complete shear failure of the masonry walls at the roof level, and hence, the experiment was stopped just before a roof displacement amplitude of 15 mm (0.5 % roof drift). This type of behavior was not observed for the S_URBM structure. In fact, when the testing was stopped at a roof displacement of about ~45 mm (1.5 % of roof drift), the structure was still taking the load without significant reduction of strength.

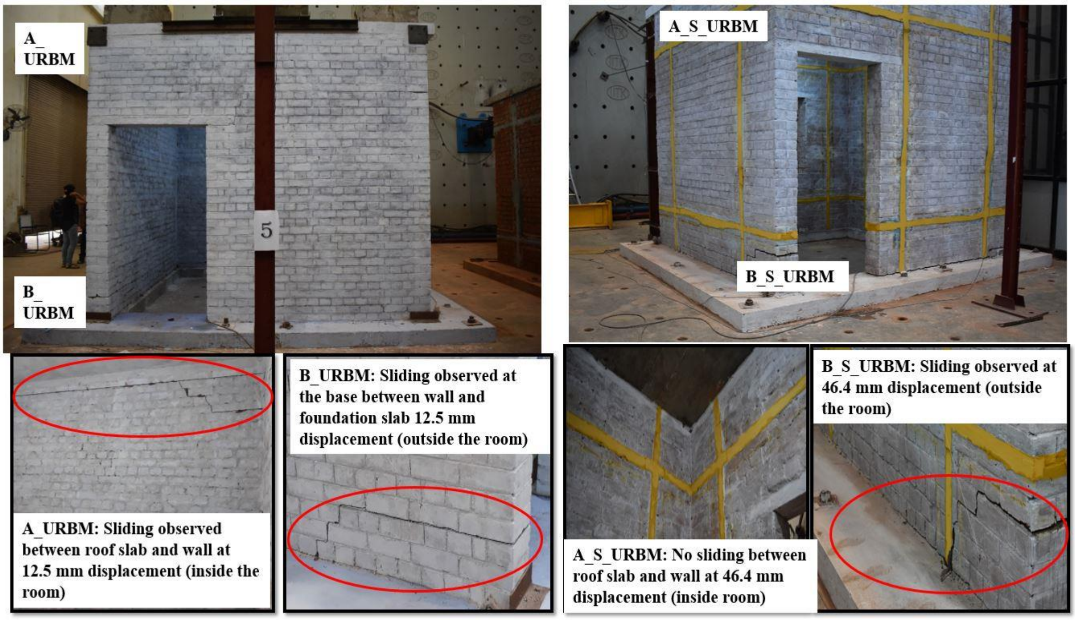
Fig. 9 represents the hysteretic behavior observed for both the structures. One can note from this figure that the strengths attained by URBM and S_ URBM are almost the same, being 138.9 kN for URBM and 142.7 kN for S_URBM. In fact, the initial stiffness of both the structures was almost the same. Hence, it can be concluded that the introduction of the seismic bands does not significantly alter the stiffness or strength of the structure. However, one can note that the S_URBM structure can sustain much larger deformations without posing any risk of collapse. In other words, the ductility capacity (or post-yield deformation) of the S_URBM structure is higher than that of the URBM structure. Based on the observed hysteretic loop, the ductility capacity is calculated as 3.6 for URBM and 13.2 for S_URBM. It is noteworthy that the ductility capacity increases by almost 4 times for the S_URBM structure. This is of considerable importance from the seismic design point of view as it indicates that the S_URBM structure can dissipate much higher amount of energy. Further, one may also note that a slight asymmetry is there in the hysteretic behavior of both structures. This is due to the stick-slip interface behavior of the walls.

Fig. 10 shows the crack patterns at failure for both the URBM and the S_URBM specimens. In terms of crack pattern, the URBM structure demonstrated a brittle manner of crack propagation, without sustaining much load and failed at early displacement stages. Typically, the crack pattern observed is in sliding shear mode due to the shear stress demand being higher than the bond strength between the brick units and mortar. The final failure was observed in the form of separation of roof slab over walls followed by diagonal cracking. Slippage was also observed over the first layer of bricks at the base, since it was clamped with plates at the first layer of bricks. Horizontal and diagonal cracks were very common even at the very early displacement stage (less than 5 mm). For the S_URBM structure, cracks were not visibly observed until a much larger lateral displacement. Also, most of the cracks were concentrated in lower layers of bricks. The cracks were seen propagating at the end of the confining vertical column and finally rocking was observed followed by crushing at the compression toe for higher displacements. Crack propagation was not observed for the S_URBM at corners due to the combined action of the seismic bands. The confining band elements also prevented the crack propagation at the roof level. Hence, the vertical and horizontal confining band elements were found to effectively enhance the seismic performance in case of the S_URBM specimen.



(a) (b)

Fig.9 – Force-roof displacement behaviour obtained from slow cyclic tests on: a) URBM specimen, and b) S_URBM specimen



(a) (b)

Fig.10 – Crack pattern observed at the end of the test for: a) URBM and b) S_URBM specimens

3. Fragility Analysis

Fragility curves have now emerged as an important tool for vulnerability assessment of all kinds of structures. A seismic fragility curve gives the probability that a structure or its component will exceed a certain damage level conditional on the occurrence of a ground motion with specific intensity. The intensity of a ground motion is usually expressed in terms of ground motion intensity measures like PGA, spectral ordinates of response spectrum etc. Fragility curve of a system or component can be derived using the analytical or empirical damage data or both. In this studty, a vulnerability assessment of URBM and S_URBM structures is carried out using fragility curves while considering ground motions of different hazard levels, analytically evaluating the enhancement in seismic performance of URBM building strengthened with seismic bands.

3.1 Numerical modeling

For fragility analysis of masonry structures, a detailed finite element analysis can be computationally demanding and time consuming. Therefore, equivalent simplified SDOF models are developed and used in this study. The spring stiffness and lumped mass of the SDOF systems for both the URBM and S_URBM are chosen such that both provide similar vibrational properties in the elastic zone. The stiffness is estimated from the experimentally obtained force-deformation curves, and the mass is estimated using the experimental model parameters. Further, for simplicity of modeling and considering the experimental results, the material model of the spring is considered to be elastic-perfectly-plastic for both the structures, with the only difference being in terms of the ductility capacities.

3.2 Ground motions

The ground motions used in the fragility analysis include the following seven recorded motions: (i) Burma (Mw 7.2, Station: Mawphlang), (ii) Burma (Mw 7.2, Station: Nongkhlaw), (iii) Meghalaya (Mw 5.5), (iv) North East India (Mw 5.8, Station: Khliehriat), (v) North East India, (Mw 5.8, Station: Mawphlang), (vi)

North East India (Mw 5.8, Station: Nongkhlaw), and (vii) Nepal (Mw 7.8, Station: Kanti Path). Since the URBM structure was designed using the lateral load distribution specified in the IS 1893 (Part 1): 2002 [10] for medium soil profile and location at seismic Zone IV, the selected motions were scaled so that the response spectral ordinates of the scaled ground motion at the fundamental period of the equivalent SDOF building model matches with the code specified design spectral ordinate. It may be noted that the response spectrum of the ground motions were evaluated for 5% damping for consistency with the code specified spectrum. For the response spectrum corresponding to the design basis earthquake (DBE), the code specified spectrum is multiplied by Z/2, i.e., 0.12 as per IS 1893 (Part 1): 2002. In other words, the peak ground acceleration (PGA) of the DBE is 0.12 g. Then, for the generation of the fragility curves, these scaled motions are further scaled with factors starting from 0.2, and increasing at a step of 0.5, to represent ground motions of increasingly varying hazard levels.

3.3 Damage level

The fragility curves are generated considering the following damage levels for URBM and S_URBM structures. For URBM structure, it is assumed that the building is severely damaged if the ductility demand exceeds 1. For the strengthened structure, it is assumed that the structure will get damaged only when the ductility demand exceeds a value of 2. This is a conservative estimate considering the highly promising experimental results, but provides some insight into the seismic performance enhancement provided by the seismic bands. Ductility demand is estimated as the ratio of maximum displacement to the yield displacement.

3.4 Fragility curves:

In this work, the fragility curves are developed as two-parameter lognormal distribution functions, where the parameters (median, c and log-standard deviation, ζ) are estimated through a maximum likelihood method [11, 12]. Fig.11 shows the fragility curves for URBM and S_URBM for the aforementioned damage condition. It can be observed from these fragility curves that a significant reduction in vulnerability can be achieved in case of the retrofitted structure. For median probability of exceedance, for the URBM structure, severe damage occurs around a PGA of 0.22 g, while for the retrofitted structure, this value becomes around 0.45 g.

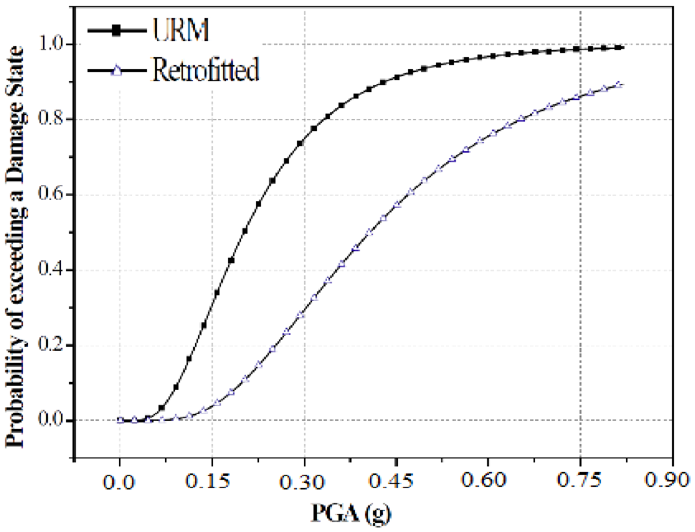


Fig.11 – Fragility Curves for URBM (URM) and S_URBM (Retrofitted) structures

4. Conclusions

This study demonstrates the effectiveness of introducing semi-confining seismic bands in existing unreinforced brick masonry structures (URBM) for improving seismic performance. For this purpose, two full-scale single-story identical URBM buildings were constructed in the Structural Engineering Laboratory of IIT Kanpur. One of these structures was retrofitted by reinforced concrete seismic bands at strategic locations by cutting of small grooves on the walls, and filling up of these grooves with ready-mix concrete along with nominal reinforcements. The performance of the two buildings were then evaluated by applying slow cyclic load at the roof level as per FEMA 461 loading protocol. LVDTs and wire potentiometers were used to measure the deformation of the specimens at various levels. Experimentally obtained force-deformation cuves of both the URBM and S_URBM structures were carefully studied. It was found that the seismic band does not significantly alter the initial stiffness or the strength of the buildings. The results of these tests however demonstrated that, while the URBM structure without bands has a tendency to fail in shear at the joints for comparatively lower load, the retrofitted structure continued to carry higher loads up to a significantly larger lateral drift. The hysteritic behaviour indicated a much enhanced energy dissipation capacity and ductility of the strengthened specimen. This was also obvious from the crack patterns observed at the end of the tests. Utilizing these test results, simplified SDOF dynamic models of both the URBM and S_URBM specimens are developed. Nonlinear time history analyses are then carried out under a suite of earthquake ground motions representing a high seismic zone as per the Indian Code. Seismic fragility curves are then developed for both buildings in order to understand the seismic performance enhancement due to the presence of the seismic bands. For median probability of exceedance, it is found that a significant performance enhancement can be achieved by introducing seismic bands in existing masonry structures.

Although, further analysis are required for both specimens in order to derive more understanding and further conclusions, the results of the study show the potential of the proposed method as an economic, minimally intrusive, easily implementable retrofitting approach, which can be employed for existing seismically deficient URBM buildings.

5. Acknowledgements

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6. References

[1] Bhattacharya S, Nayak S and Dutta SC (2014): A critical review of retrofitting methods for unreinforced masonry structures. *International Journal of Disaster Risk Reduction* **7** (2014): 51-67

[2] ElGawady M, Lestuzzi P, Badoux M (2004): A review of conventional seismic retrofitting techniques for URM, *13th IB2 MaC*.Holland: Amsterdam; 2004.

[3] Singhal V, and Rai DC, (2018): Behavior of Confined Masonry Walls with Openings under In-Plane and Out-of-Plane Loads, *Earthquake Spectra* 34 (2), 817-841

[4] Jain SK, Brzev S, Bhargava LK, Basu D, Ghosh I, Rai DC and Ghaisas KV (2015).“Confined Masonry for Residential Construction, IIT Gandhinagar.”

[5] Bureau of Indian Standards (BIS). (1992b): Indian standard methods of test of burnt clay building bricks—Part 1: Determination of compressive strength, IS 3495, 3rd Rev., New Delhi, India.

[6] Bureau of Indian Standards (BIS). (1992c): Indian standard methods of test of burnt clay building bricks—Part 2: Determination of water absorption, IS 3495, 3rd Rev., New Delhi, India.

[7] ASTM C109/C109M-11b, Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. Or [50-mm] Cube Specimens).

[8] IS 1905 : 1987 (Reaffirmed 1998) :Code of Practice for Structural use of Unreinforced Masonry (Third Revision).

[9] ASTM E 519:07 – Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages.

[10] IS 1893 (Part 1)- 2002: Criteria for Earthquake Resistant Design of Structures. Part-1, Bureau of Indian Standards, New Delhi.

[11] FEMA 461 (2007), Federal Emergency Management Agency, "Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components", Washington D.C., USA.

[12] Shinozuka, M., Feng, M., Uzava, T., (2000). Statistical Analysis of Fragility Curves, *Technical Report MCEER 2000* State University Of NewYork at Buffalo, NY.

[13] Shinozuka, M., Feng, M. Q., Kim, H., Uzawa, T., and Ueda, T., 2003. Statistical Analysis of Fragility Curves, *Report No. MCEER-03–0002*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

