Article ID:

Twin lintel belt in steel for seismic strengthening of brick masonry buildings

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Abstract: A single-room, single-storey full-scale brick masonry building with precast RC roofing system was tested thrice under displacement controlled lateral cyclic loading, to assess the effectiveness of the basic repair and seismic strengthening techniques. Initially, the virgin building specimen was loaded laterally to failure. In the second stage, the damaged building was repaired by stitching across the cracks, and tested under the same lateral loading. In the third stage, the twice-damaged structure was repaired once more by stitching and strengthened by twin lintel belt in steel and vertical corner reinforcement, and re-tested. The building strengthened by *twin lintel belt in steel* showed about 28% higher strength under lateral loading than the virgin building.

Keywords: brick masonry building; seismic strengthening; twin lintel belt; quasi-static lateral cyclic loading

1 Introduction

The study of earthquake damage to masonry buildings with precast roofing system is important for ensuring protection against future earthquakes. The collapses of masonry walls with precast roofing can be attributed to various reasons, namely (a) out-of-plane failure of walls, which may be due to inadequate flexural strength of unreinforced masonry (Sommers, 1996) or due to lack of integrity of adjoining structural components (Murty *et al.*, 2001), (b) in-plane failure of walls, which may be due to reduced shear capacity of poor quality mortar (Shea, 1993) or due to tension failure along the principal diagonal plane (Tomazevic, 1999), and (c) failure of connection between precast components, which may be due to inadequate connections between the precast roof panels and lateral load-resisting systems (Jain *et al.*, 1994; Murty *et al.*, 2001; Meehan *et al.*, 1973). Thus large slenderness ratios of walls, inadequate tensile and shear strengths of masonry, and inadequate connections between the load-bearing components are the major causes for failure of these structures.

1.1 Code provisions

The general guidelines for earthquake resistant design and construction of masonry buildings in India (IS 4326, 1993) specify the requirements for openings to ensure a desirable seismic response of the masonry wall. The other recommendations in the country include control on the quality and proportions of materials for construction (IS 4326, 1993; IS 13828, 1993). IS 13828 (1993) also specifies strengthening measures for improving the seismic performance of low-strength masonry buildings. Cracks in damaged and undamaged masonry buildings, not wider than 5mm, need to be filled up by pressure injection of epoxy (IS 13935, 1993). For cracks wider than 5mm, IS 13935 (1993) recommends (a) grouting, (b) strengthening with wire mesh, (c) connection between existing walls, (d) prestressing, (e) external binding, and (f) strengthening long wall methods. The dimensional requirements of openings and the recommended strengthening measures specified in international codes of practice (ATC-28, 1991; NBC-109, 1994). IBC (2003) categorize masonry buildings into six seismic design categories A to F, depending on the mode of resistance of lateral loads; the required horizontal and vertical reinforcement are specified along with the required connection details. International standards (ATC-28, 1991; FEMA-273, 1997; EC8 (1-4), 1996) also recommend the methods for repair and strengthening of damaged masonry walls and connections.

1.2 Past studies

Experimental studies under earthquake excitation have been conducted mostly on masonry models than on full-scale masonry structures due to lack of high

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Received date: 2004-08-20; **Accepted date:** 2004-10-04

capacity testing facilities to study prototypes of the large-sized actual structures. Under lateral load tests, both horizontal and vertical reinforcement (Krishna and Chandra, 1965) are effective in increasing the lateral strength and inhibit crack propagation in masonry buildings. Shake table tests (Clough *et al.*, 1979; Manos *et al.*, 1984) on masonry models, with and without openings, showed the permissible level of peak ground acceleration without any damage. Shock-table test on scaled single-storeyed masonry building (Qamaruddin *et al.*, 1978) showed that RC lintel band, corner and jamb steel increased the strength and energy absorption capacity of the buildings. Appropriate design considerations can ensure desirable ductile response (Seible *et al.*, 1994) for masonry building with precast-prestressed hollow-core floor planks. Analytical models for in-plane response of brick masonry in the linear range (Mengi *et al.*, 1984) and in the non-linear range (Mengi and McNiven, 1989 (a) and (b)) simulated the experimental behaviour of similar specimens.

The present study determines the lateral strength of a full-scale single-room brick masonry building with precast RC roofing system under quasi-static lateral cyclic loading. A new method is proposed for the seismic strengthening of existing brick masonry buildings; the effectiveness of this is experimentally investigated. The results are also compared with that of the building with corner reinforcement.

2 CBRI channel unit flooring and roofing system

The general cross-section of CBRI-developed precast RC channel unit (Dutta, 2001) is a symmetric C-shape with its outer sides corrugated and grooved at the ends to provide shear key action between adjacent units. The channels are placed in the inverted position with the opening facing downwards. Details on geometry, material and connections while using these 300mm wide channel units are given elsewhere (Dutta, 2001). In the precast flooring and roofing system, the full-span precast units are placed side by side on the brick or stone masonry walls of the room, and are joined by in-situ concrete in the grooves (Fig.1). The bond between in-situ concrete and channel units is expected to impart monolithicity between the units, which helps in transferring the shear force in the transverse and in-plane directions. This precast channel roofing system is used in the present study.

3 Experimental study

3.1 Material properties

Average dimensions of burnt clay brick units used are 230mm×110mm×70mm. The compressive strength is obtained for individual brick units as per the standard test procedure (IS 3495, 1976). Two mixes of mortars are used in the construction, namely (a) 1:6 cement mortar below the lintel level, and (b) 1:8 cement mortar above the lintel level. Locally available sand and 43 Grade Ordinary Portland cement are mixed as per volume to emulate the traditional constructional practices. The compressive strengths of the mixes are estimated from six cylindrical samples of 50mm diameter and 100mm height. The compressive strength, modulus of elasticity, Poisson's ratio, and shear strength of brick masonry prisms are obtained (Dutta, 2001) using both the mixes of mortars. For the nonlinear stress-strain behaviour, the tangent modulus is obtained as the secant slope corresponding to 75% of the ultimate stress. Basic properties of the brick masonry used in the test are listed in Table 1.

3.2 Test setup

A distributed load is applied at eight points on the precast roofing system through a grillage mechanism (Fig.(2a)) to simulate the uniform inertia force generated in the roof slab system during earthquakes. The displacement-controlled loading is applied at a very slow rate to eliminate material strain rate effects. To generate the cyclic lateral loading, this grillage loading system is provided on opposite sides of the roof slab (Dutta, 2001). A 100kN-capacity 50mm-stroke servo-hydraulic actuator (Manufacturer: M/s Instron Inc, USA) is used to apply the displacement-controlled cyclic loading on the building – three cycles each of ±1, ±2, ±3, ±6, ±9, ±12, ±15 and ±18mm. The arrangement of the LVDTs for measuring the responses are shown in Fig. 2(b). In the present study, walls in the direction of application of load are referred to as in-plane or shear walls, and those in the perpendicular direction as the out-of-plane

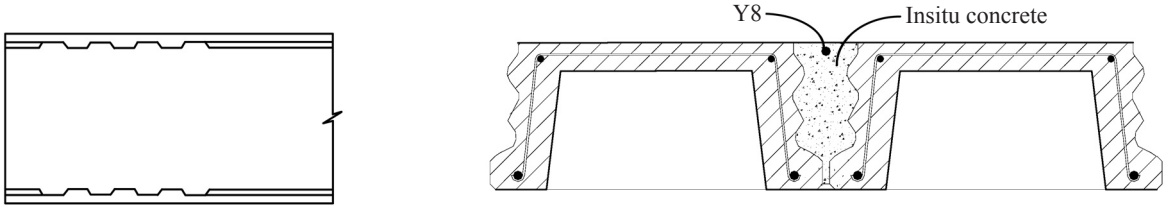

Fig. 1 (a) Plan view of end portion of 300 wide channel sections used for roofing, and (b) Details of joint between two channels units

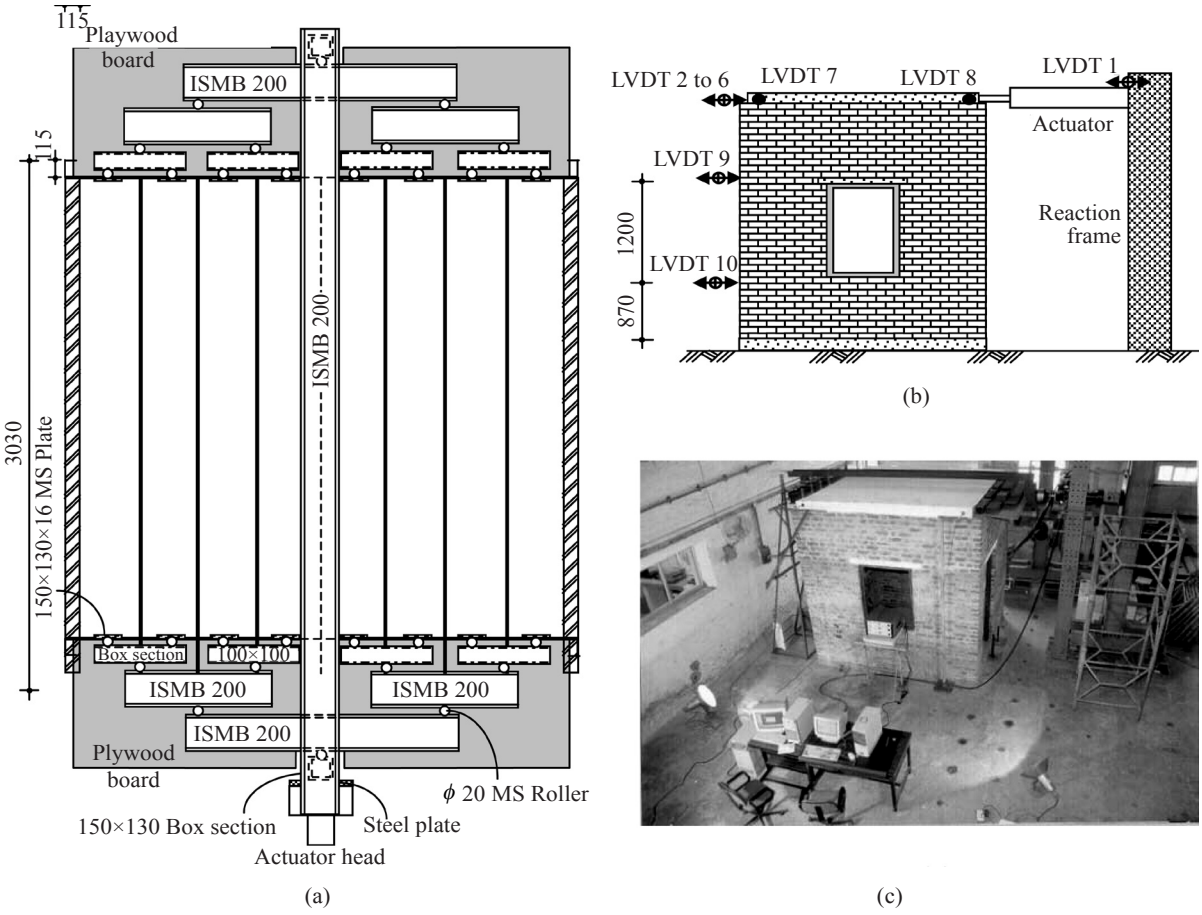

Fig. 2 (a) Schematic of plan assembly of quasi-static displacement loading at the roof level of the building, (b) diagram showing position of LVDTs, and (c) overview of the test set-up for the building specimen with no lintel band

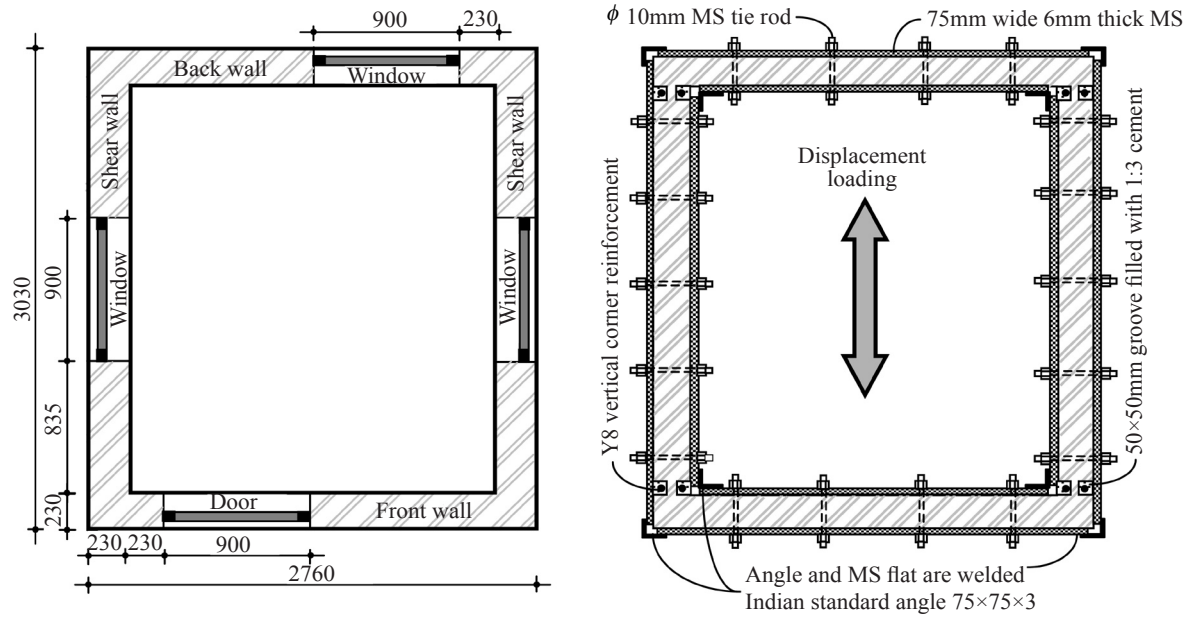

Fig. 3 (a) Wall geometry at window level, and (b) details of corner reinforced and steel twin lintel belt employed to strengthen the building

or cross walls (Fig. 3(a)).

3.3 Repair and strengthening of specimen

The initially tested and damaged building specimen URM is repaired by stitching across the cracks, and this repaired specimen (named URM-R) is re-tested to evaluate the effectiveness of stitching under lateral loading. In the next stage, the damaged specimen URM-R is retrofitted by two methods, namely (a) re-repair by stitching the cracks, and (b) strengthening by providing corner reinforcement and a steel twin lintel belt. The re-repaired and strengthened specimen (named URM-RS) is tested under lateral cyclic loading.

3.3.1 Specimen URM-R

During the first test of the virgin specimen (specimen URM), the two shear walls and front wall sustained considerable cracking. These cracks are stitched. Eight 230mm wide through holes (three each across one shear wall and front wall, and two on the other shear wall) extending upto one brick thick from the cracks on either side, are made across the cracks; details are given elsewhere (Dutta, 2001). The holes are washed with water, and cement slurry is applied to improve the bond between stitch material and the masonry. Two 8mm diameter high yield strength deformed bars are placed across the crack, and the hole is filled with M20 grade in-situ concrete. The concrete is compacted manually in three layers. Then the exposed surface of the concrete is made flush with the face of the wall with cement mortar.

3.3.2 Specimen URM-RS

After performing the cyclic lateral load test on specimen URM-R, the building specimen is re-repaired by stitching and strengthened by providing corner reinforcements and steel twin lintel belt. The same method of stitching the cracks, as discussed in section 3.3.1, is adopted at the new locations of the cracks. The length of the stitch is increased in specimen URM-RS to two brick courses across on either side of the crack as the stitch length of one brick course, provided for specimen URM-R proved to be insufficient.

At either end of each shear wall, two 8mm diameter high yield strength deformed bars are embedded in the masonry, on either side of the wall thickness. Eight 50mm square grooves are made vertically in the masonry at an edge distance of 230mm from the outer corner (Fig.3(b)). These grooves are extended up to half the depth of the foundation beams. The vertical reinforcements embedded in the walls are welded to the reinforcement of the foundation beams. At the top, an angle section is welded to the reinforcement in the channel units that is exposed by chipping the cover concrete; the vertical bars are welded to this angle section. Along the length, the vertical bar is held firmly to the face of the wall with the help of U-nails at intervals of 1m. The groove is filled with 1:3 cement mortar and finished flush to face of the wall.

In specimen URM-RS, a twin lintel band in steel is introduced. At the lintel level, 10mm diameter holes are made through the walls at intervals of 600mm. MS flats (75mm wide and 6mm thick), one on the inner face and another on the outer face of the walls, are bolted to each other at these holes through the walls (Fig. 4). At both the inside and outside corners, the flats coming from the two orthogonal directions are welded with the help of angle sections. The details of connection between the steel twin lintel belt and the building are shown in Fig.3(b).

Fig. 4 View of the building after being strengthened by steel lintel belt

4 Results and discussions

4.1 Specimen URM

The original unreinforced masonry building, without any earthquake resistance features, showed a brittle failure mode with diagonal crack in the two shear walls. Cracks started above the lintel and propagated diagonally downwards up to almost the base of the building. Also, the front cross wall sustained a horizontal crack throughout its length that terminated into the diagonal cracks in the shear walls above the lintel level. The experiment was terminated during the 10mm cycle without applying the load in the reverse direction, when one MS roller of the back grillage loading arm slipped off. Thus, the building sustained diagonal cracks only in one direction. The ductility of the building could not be explored.

The load-deflection curve of the building (Fig.5) suggests a linear-brittle behaviour before failure. The ultimate load carrying capacity of the building was 43.7kN at an associated storey drift of 0.03%. Ultimate

lateral load carrying capacity of the building is obtained from other analytical estimates as 42.2kN (Moghaddam *et al*., 1990), and 106kN (IS:1905-1987, 1987). The stiffness of the building from the experiments was 45,813kN/m whereas, the corresponding analytical estimated value of stiffness is 52,478kN/m. The residual strength and displacement recorded are 0.07kN and 0.26mm, respectively, on unloading the structure. The lateral load-deflection hysteresis curve at roof level (Fig.6), suggests a very small energy dissipation capacity of the building. The energy dissipated when the loading was stopped was about 34% of the strain energy input to the building, and equivalent viscous damping obtained from the hysteresis loop at ultimate stage of failure is about 5.5%.

4.2 Specimen URM-R

The specimen URM-R failed in diagonal cracking, with the cracks following the same path as those generated during the loading of specimen URM. The cracks navigated around the periphery of the stiches. The first crack appeared above the lintel level in the first 10mm cycle and propagated up to the top corner of the window openings of the two shear walls. The front wall also sustained considerable cracks along the same crack-paths generated during the testing of URM. In the first 14mm cycle, diagonal cracks travelled below the bottom windowsill level of the two shear walls. In the first 17mm cycle, the cracks propagated further and finally reached the base of the building in the 20mm cycle.

The lateral load-deflection curve (Fig. 5) shows that the building behaved elastically up to a load of 24.4kN at an associated storey drift of 0.2%. The initial stiffness of the building is calculated as 35,247kN/m, while at failure it reduces to 2,522kN/m. The ultimate load carrying capacity was 28kN, which is much smaller than the strength of the original specimen URM. The recorded displacement histories suggest that the roof system stayed as one unit and did not separate into individual channel units. The lateral load-deflection hysteresis curve at the roof level (Fig. 6) shows that the energy dissipated at the ultimate state is about 70% of the strain energy input to the structure, and the equivalent viscous damping at the ultimate stage of failure is about 11%. Ductility of the building is obtained as 4.7. Though the adopted repair measure enhanced the lateral resistance of the damaged building, the lateral strength of the original building could not be regained.

4.3 Specimen URM-RS

The first crack appeared in the first 20mm cycle at the slab-shear wall interface level. Initially, the crack propagation was limited to above the lintel belt level, but later in the third 20mm cycle, it crossed and propagated below it also. Crack propagation below lintel belt was initiated at the stitches made at the lintel level of the two shear walls; the crack was found to propagate along the periphery of the stitches. No crack cut across the corner reinforcements provided. In the 24mm cycle, cracks propagated down towards the base of the building along a new path. Again, the cracks below lintel level went around the periphery of the stitches. The whole slab was

Table 1 Strength properties of brick masonry and its constituents

	Brick	Mortar		Brick masonry			
				Masonry prism		Shear specimen	
		1:6 mix	1:8 mix	1:6 mix	1:8 mix	1:6 mix	1:8 mix
Number of specimens	9	6	6	5	6	6	6
Compressive strength (MPa)	24.2 (4.96)	5.9 (0.26)	2.1 (0.35)	8.2 (0.72)	6.7 (0.62)	-	-
Young's modulus (MPa)	-	-	-	1,043.4 (99.14)	724.9 (182.1)	-	-
Poisson's ratio	-	-	-	0.027 (0.019)	0.024 (0.013)	-	-
Shear strength (MPa)	-	-	-	-	-	0.68 (0.077)	0.56 (0.084)

Note: Numbers in the parentheses give the respective standard deviation.

Table 2 Comparison of experimental responses of three specimens studied

Specimen	Strength (kN)	Stiffness (kN/m)	Ductility	Energy dissipation capacity (%)	Damping coefficient
URM	43.7* (0.07)	45,813 (-)	-	34*	5.5*
URM-R	28.0 (3.06)	35,247 (2,522)	4.7	70	11
URM-RS	55.8 (3.06)	41,650 (4,810)	3.7	47	7.5

Note: Numbers in the parentheses give the respective residual values at the end of the test.

* Premature termination of experiment due to falling of steel roller.

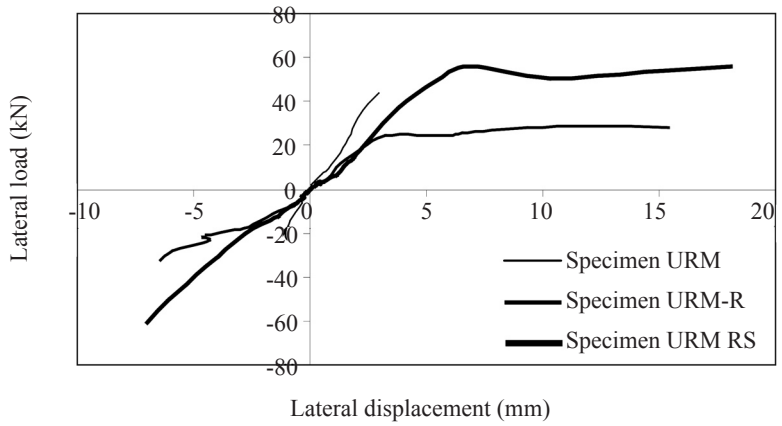

Fig. 5 Lateral load versus lateral deflection graph at the roof level.

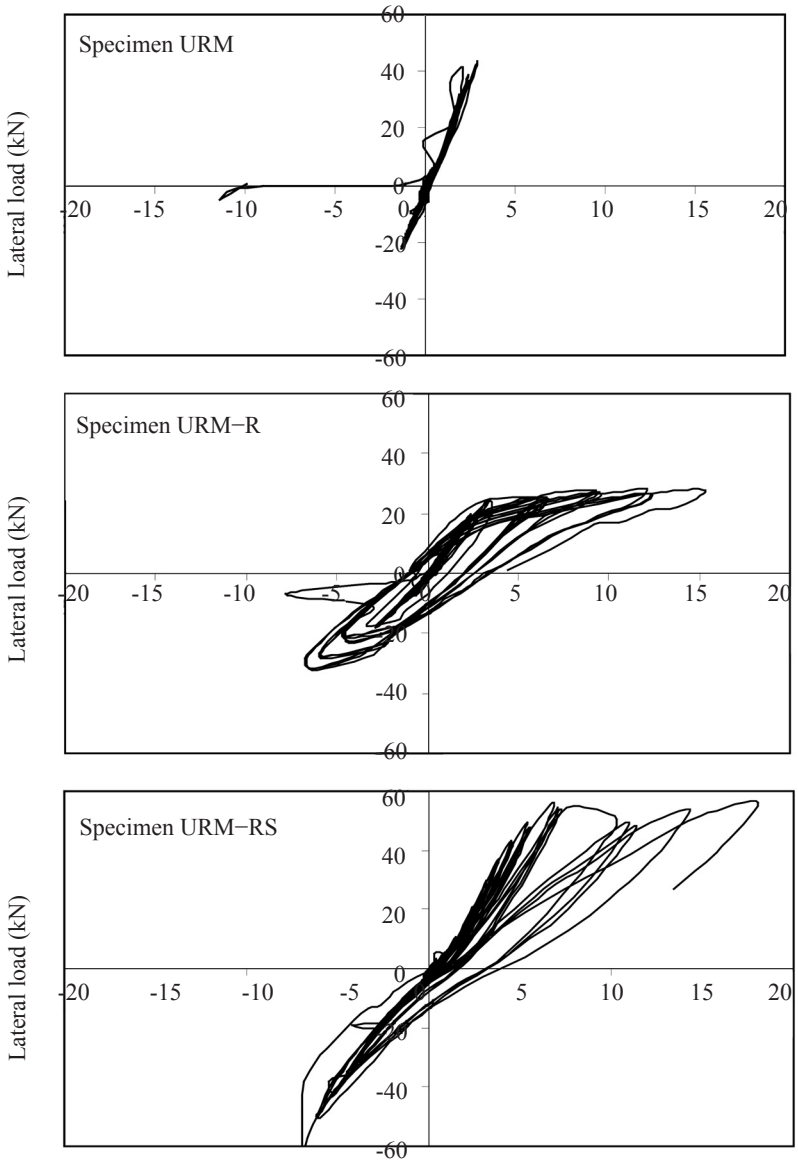

Fig. 6 Lateral load *versus* lateral deflection history at the roof level of the bare, repaired and strengthened specimen

separated from the wall at 55.8kN load and one joint between the channel units opened in the first 28mm cycle.

The building behaved elastically up to 49.1kN at an associated storey drift of 0.2% before the onset of nonlinear behaviour (Fig. 5). The ultimate load carrying capacity of the building was 55.8kN at a storey drift of 0.6%. Initial stiffness of the building is 41,650kN/m and is reduced to 4,810kN/m at failure. Specimen URM-RS dissipated about 47% of the strain energy input to the structure before failure. The equivalent viscous damping of the building at the ultimate stage of failure is 7.5%. The ductility of the building is 3.4. Figure 6 shows the hysteresis curves of specimen URM-RS. Table 2 shows the comparison of system responses of three specimens obtained experimentally.

5 Comparison of results and discussion

5.1 Specimen URM-R versus Specimen URM

(1) As the specimen URM-R was obtained by stitching the cracks of URM, the cracks in URM-R propagated along the periphery of the stitchings.

(2) As the specimen URM sustained cracks in the forward direction only, the specimen URM-R was also weaker in the forward direction than in the backward direction; this behaviour is evident from the hysteresis curve of specimen URM-R (Fig. 6).

(3) The ultimate load-carrying capacity of the specimen URM-R is much smaller than that of URM. Although the specimen URM-R failed at 28kN, its behaviour during loading in the reverse direction still remained elastic (Fig. 5).

(4) Stiffness of specimen URM-R of the building was 77% of the initial stiffness of the specimen URM.

(5) The ductility, energy dissipation capacity and damping of the specimen URM-R are much more than those of URM, due to the presence of cracks.

In spite of having higher energy dissipation capacity and ductility, the repair measure (stitching) is not effective in resisting lateral load from the strength point of view, as the original strength could not be restored back.

5.2 Specimen URM-RS *versus* Specimens URM-R and URM

(1) In specimen URM-RS, the twin lintel band in steel was effective in restricting the cracks to above the lintel level. But later, due to the presence of stitches at the lintel level, it propagated below the lintel level around the periphery of the stitching, and reached the base of the specimen URM-RS.

(2) In specimen URM-RS, though the length of the stitch was increased than that in specimen URM-R, the cracks were still observed to travel around the periphery of the stitches. Thus, again stitching proves its inefficiency in preventing crack propagation.

(3) The corner reinforcement helped in eliminating the pier action as shown by specimen URM. There was no cracking through the corners where the reinforcements were placed.

(4) The stiffness of the URM-RS was 91% of the original structure, and the strength of the structure was restored by 127% of the original structure.

(5) The energy dissipation capacity and displacement ductility of URM-RS were higher than those of URM and URM-R. The provision of corner reinforcement and twin lintel band in steel proved to be very effective as strengthening measures against lateral load.

(6) The connections between the precast channel units were found to be inadequate in transferring the shear force generated during earthquake excitation.

6 Conclusions

The following are the salient conclusions drawn from the present study:

(1) The twin lintel belt in steel along with vertical corner reinforcement is a very good measure for seismic strengthening of single-storey brick masonry buildings.

(2) The vertical corner reinforcements are very effective in arresting cracks initiation. No crack propagated through the corners when vertical reinforcements are used.

(3) The traditional repair method of stitching is not effective in restoring the strength of the buildings; the cracks propagate along the periphery of the stitches.

(4) Proper anchorage between walls and roof or floor slab is necessary. Bearing area of the precast channel unit flooring and roofing system used in this study, needs to be increased for better load transfer and resistance against lateral loading. A shear key may be introduced in the end channel units to anchor them into the wall masonry.

The current method of usage of the CBRI developed precast channel unit flooring and roofing system is vulnerable to strong horizontal as well as vertical ground motions.

Acknowledgement

This study is conducted as part of a research project under a technical collaboration program in India between the Indian Institute of Technology Kanpur and the Central Building Research Institute (CBRI), Roorkee.

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