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Rehabilitation of heavily earthquake damaged masonry building using steel straps

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Abstract. The purpose of this study is to develop a rehabilitation technique for heavily earthquake damaged masonry buildings. A full scale one storey masonry building with window and door openings was manufactured and tested on the shock table by applying increased amplitude free vibration up to the point where heavy earthquake damage was observed. Damaged test building was rehabilitated with vertical and diagonal steel straps and then tested again. The effectiveness of improvements obtained by the rehabilitation technique was investigated. Steel straps improved the lateral strength and stiffness of masonry walls and limited the lateral displacement of building. Stability of the masonry walls were also improved by the steel straps. Steel straps reduced the natural period of the earthquake damaged masonry building and prevented the failure of the building at the same amplitude of free vibration.

Keywords: masonry building; earthquake damage; rehabilitation; steel strap.

1. Introduction

Like many countries across the world, masonry structures have been widely used in Turkey. The ratio of the number of masonry buildings to the number of total structures, and the ratio of the population living in masonry buildings to the total population at Turkish cities are given in Fig. 1

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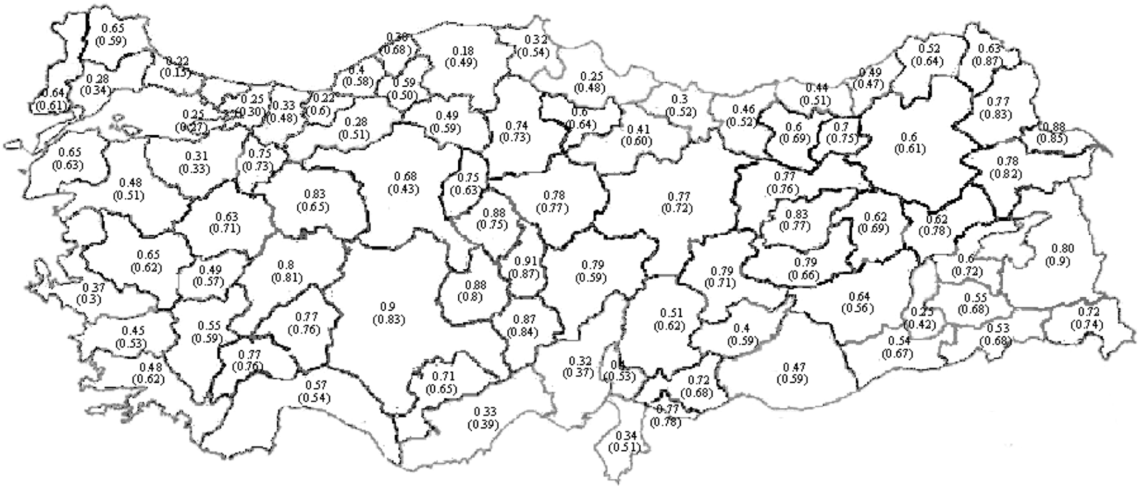
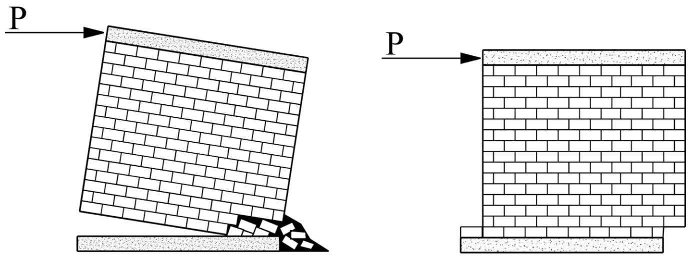


Fig. 1 Percentage of masonry buildings and the number of people living in masonry buildings in Turkey
Note: The number of people living in masonry structures/city population ratio is shown in parenthesis

Building Counting Result (2000), General Population Counting Result (2000). Ratios that are given in Fig. 1 show that currently about the half of the structures in Turkey are masonry structures and the half of the population is living in these structures. In general, these buildings are one or two storey buildings with masonry walls without reinforcement and they only have strength resistance with no ductility. Due to not using suitable masonry blocks in constructions, structures can only carry vertical loads at most, and they are not safe against earthquake loads. In addition to these, window and door openings cause to reduce aspect ratios (l/h, where l=wall length, h=wall height) of the walls, and this also decreases the resistance of the walls against earthquake loads.

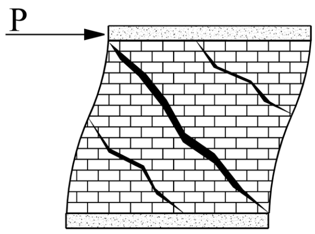
Failure mode and resistance to lateral loads of masonry structures are determined by the parameters such as the axial load level at walls, strength of brick and mortar that was used for connecting bricks, and the aspect ratio of the wall. There are two basic failure modes for in-plane masonry walls under low axial loads when they are subjected to earthquake loads: bad joint slide, and rocking and toe crushing (Fig. 2-a). In plane masonry walls under high axial loads failed with diagonal shear cracks when they are subjected to earthquake loads, because high friction forces created by the axial loads prevent horizontal sliding (Fig. 2-b). If limited relative storey drifts were occurred, masonry structures can retain their stabilities and manage not to fail. Masonry walls without any reinforcements have low energy dissipation capacity and quick loss of strength characteristics. Due to these deficiencies, very quick crack propagation and high damage have been observed under reversed cyclic loads like earthquake loads. Recent earthquakes that occurred in Turkey caused heavy damage at structures' masonry walls without reinforcements or the whole structure collapsed and caused life losses. Masonry structures, which are still in use, have also the same characteristics with the structures that were collapsed under earthquake loads, and they required strengthening.

Many rehabilitation techniques were developed for improving the behavior of the masonry structures under earthquake loading. These techniques include adding R/C layers from inside or outside of masonry walls, filling the door and window openings with R/C or masonry walls, adding new R/C infilled frames, overlaying FRP strips or sheet on existing walls and adding new steel



Rocking & Toe Crushing Bad Joint Slide

(a) Masonry Wall Failure Mechanism at Low Axial Load



Diagonal Tension Crack

(b) Masonry Wall Failure Mechanism at High Axial Load

Fig. 2 Failure mechanisms of masonry building

braced frames Calvi and Bolognini (2001), Alococer, Ruiz, Pineda, and Zepeda (1996), ElGawady, Lestuzzi, and Badoux (2006), Shrive (2006), Turco, Secondin, Morbin, Valluzzi, and Modena (2006), ElGawady, Lestuzzi, and Badoux (2006). Although these upgrading techniques are effective, they require a great deal of preparation work, their construction may disturb the ongoing building functions, and new structural elements may affect the architectural aesthetics of the building. Hence an alternative method of retrofitting is worth considering. The retrofit method proposed in this study consists of adding diagonal and vertical straps of steel on both sides of unreinforced masonry walls. The diagonal steel straps that extend betweens the corners of the masonry walls strengthen it while preventing diagonal tension failure and compression crushing under shear forces. The vertical straps confer a stable ductile flexural behavior to the masonry walls. Finally, stiff steel angels and high strength anchors connecting the straps to the floors prevent sliding of the masonry wall.

Due to the fact that building stocks of many countries are still include many masonry buildings, the number of the studies that investigate the performance of these buildings under earthquake loading are increased Paquette and Bruneau (2006), De Sortisa, Antonacci, and Vestronic (2005), Klingner (2006), Kim and White (2004). But the majority of the studies are investigated the general behavior and failure mechanisms of these buildings under lateral earthquake loading. There is very limited amount of study about the rehabilitation of damaged masonry structures after earthquakes. One of the experimental studies about diagonal steel braced masonry wall was conducted by Saatcioglu *et al.* (2000). In this study, researchers tested low rise masonry walls strengthened with vertical and diagonal steel straps under lateral reversed cyclic loads. Test results showed that strengthening with steel straps is an effective technique, and arrangements of the steel straps increased the in-plane masonry walls' strength, stiffness and energy dissipation capacity significantly. Rehabilitation of light and moderately damaged masonry structures with this technique provided safe use of buildings and social and economical difficulties at these regions were overcame easier than before.

Rehabilitation of earthquake damaged masonry buildings required complicated engineering work. There are significant difficulties at determining the residual lateral strength of earthquake damaged masonry building. Although there are techniques for strengthening of masonry structures, the success ratios of these techniques are not known for rehabilitation of earthquake damaged masonry walls. The purpose of this study is to determine the effectiveness of the rehabilitation technique that involves the application of confining steel straps on earthquake damaged masonry structures. In this experimental study, a full scale one storey masonry building with window and door openings was manufactured. First, the masonry building was tested on shock table and was heavily damaged. Then, the heavily damaged building was rehabilitated by using vertical and diagonal steel straps. Finally, the rehabilitated building was retested. The increase in strength, stiffness and changes in the dynamic parameters of the building were investigated Kuran (2006). Experimental results are reported in this paper.

2. Experimental investigation

2.1 Test specimen and material properties

The test building was a full scale, single storey masonry building. The building was constructed with the materials that are commonly used in the practical applications. The geometrical dimensions and the details of masonry building are shown in Fig. 3. The dimensions of the building were 3550 mm, 4060 mm at the east-west and the north-south directions, respectively. The height of the building is 2700 mm on a 300×200 mm dimensioned lintel. The building has an 850×1100 mm window and an 850×1820 mm door opening on south and north walls, respectively. The walls at each side of the door and window openings have 1460 mm and 1750 mm widths. The south walls were named as $S1-S2$ and north walls were named as $N1-N2$. The aspect ratio of $S1-N1$ and $S2-N2$ walls were (l/h, where l=wall length, h=wall height) 0.63 and 0.75, respectively. Due to these door and window openings lateral strength and stiffness of the building at the east-west direction was less than those of north-south direction. Upper and lower lintels of the building were constructed as 200×200 mm and 300×200 mm, respectively. Masonry walls and top lintels are constructed on thicker bottom lintels practically. Due to this reason, the bottom lintel of the test building was designed thicker than top lintel. The thicknesses of bottom and top lintels are 300 and 200 mm respectively. Four longitudinal 10 mm diameter reinforcements and 200 mm spaced 8 mm diameter stirrups were used at lintels. The 100 mm thick roof slab of the building was fixed to the upper lintel of the building, and 2 layers of orthogonal 8 mm diameter reinforcements with 300 mm spacing were used at the roof slab. An average concrete cylinder compressive strength of 12 MPa was obtained on the date of testing. Masonry walls of the test building were constructed with perforated clay bricks similar with Turkish masonry structure clay bricks. Size of the clay bricks were $280\times185\times135$ mm. The ratio of the hollow spaces to total cross sectional area of clay bricks were 63%. The picture of the clay bricks is given at Fig. 4. Head and bed joints with an approximate thickness of 20 mm were used at the masonry units. Average compressive strengths of ungrouted masonry and mortar were 4.5 MPa and 4.0 MPa, respectively. The mix designs for the

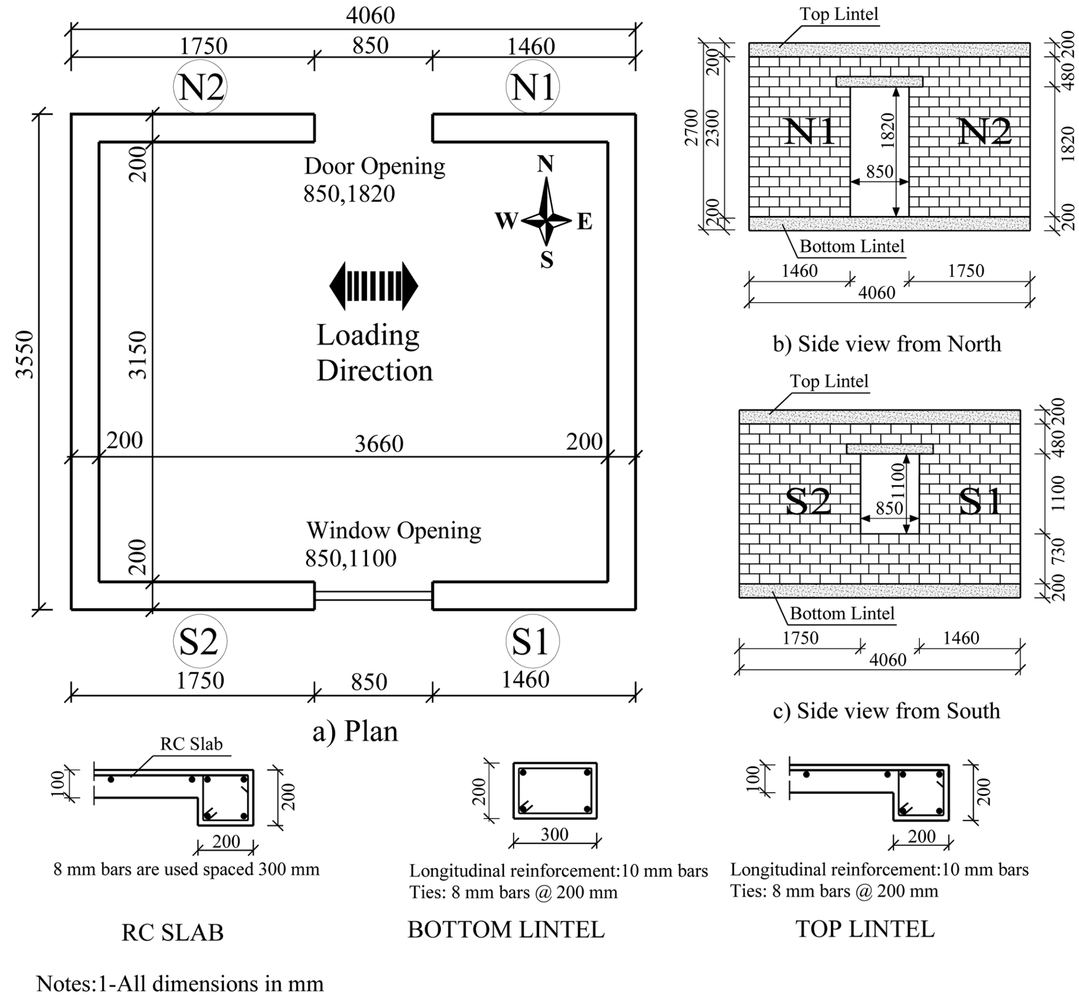
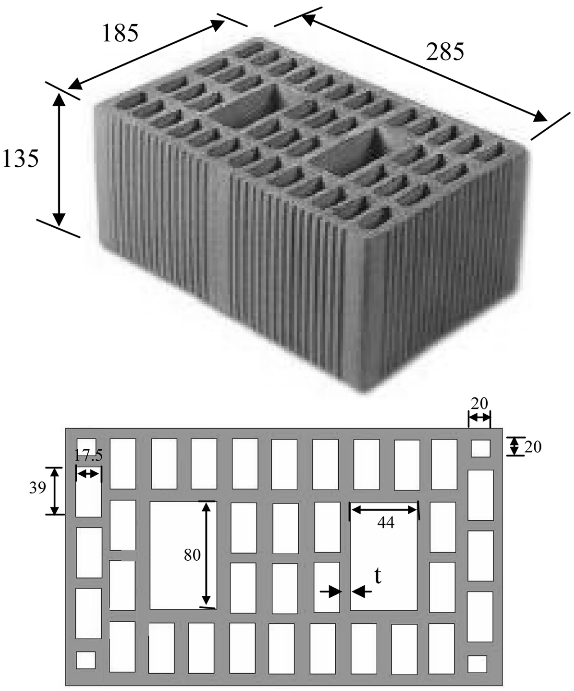


Fig. 3 Details of the test building

Table 1 Mixture design of mortar and plaster

Material	Percentage by Weight (%)
0-7 mm Aggregate	61.0%
Cement	10.5%
Lime	10.5%
Water	18.0%

mortar used in the construction of the brick walls and for the plaster were identical. Mix proportions are given in Table 1. Flexural tension strength of the both mortar and plaster is 6.3 MPa. Strengths of the material are determined by using standard test at FEMA 273 (1997), FEMA 306 (1998) and FEMA 356 (2000) regulations. Both the interior and the exterior surfaces of the walls were plastered with 10 mm thick mortar and then painted with lime. The total weight of the building was 112 kN.



Thickness t=10 mm

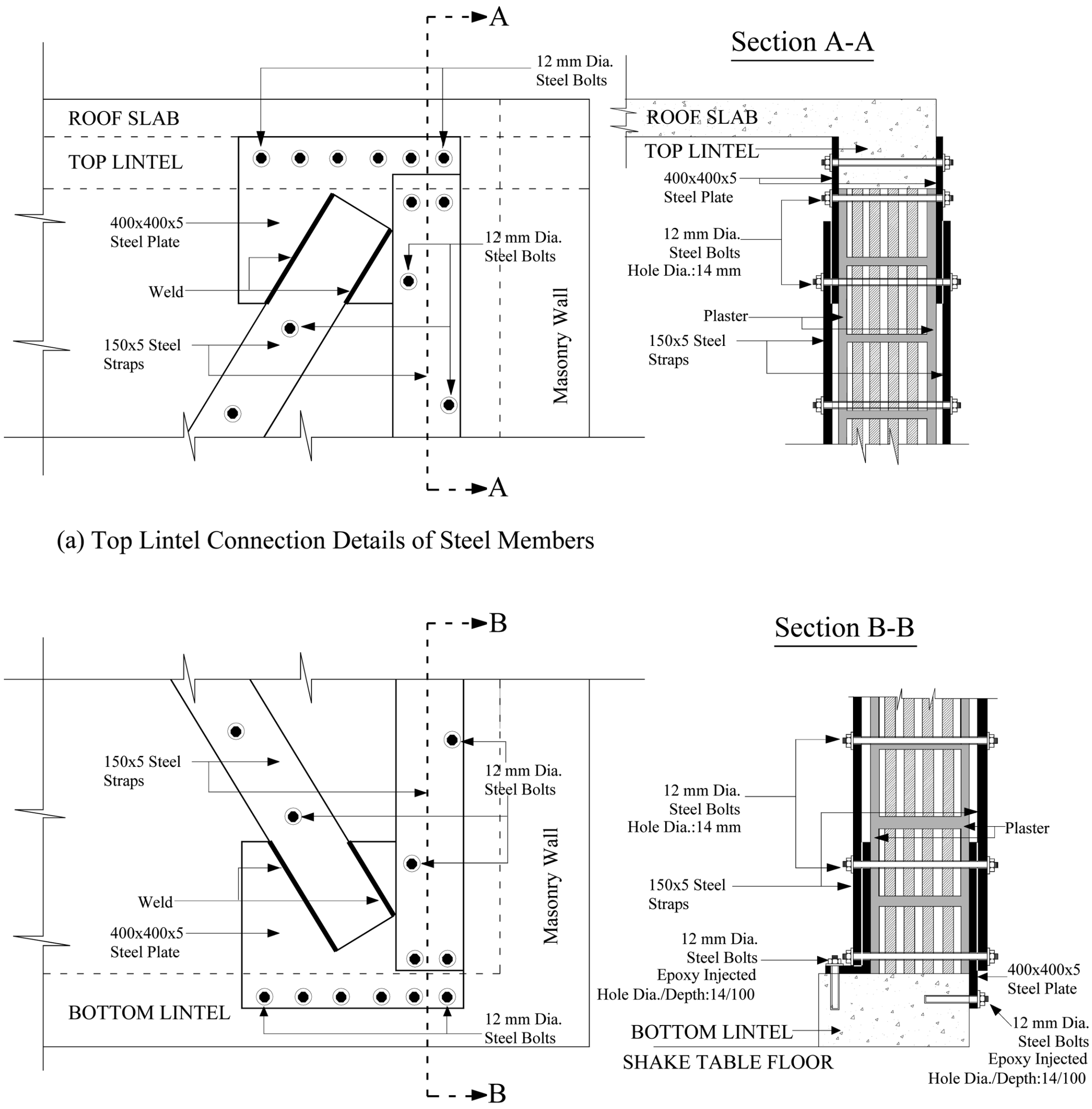
Top view of masonry brick

Fig. 4 Details of masonry clay brick

2.2 Details of the Rehabilitation Technique

In this study, one story masonry test building was constructed and tested under one directional increased amplitude free vibrations. Heavy shear damages were observed after testing. Masonry walls were lost their lateral load carrying capacity at loading direction and they can stay stable due to friction forces. Due to this reason, the steel straps and connection elements were designed such that they can carry equal amount of shear load that the undamaged masonry walls can carry.

First, cracks at the walls of masonry building were filled with high strength mortar, namely Concresive 1406. The measured compressive strength of the high strength mortar was 25 MPa. All steel members were laid without removing any plaster from the masonry wall. The thickness and the width of the diagonal and vertical steel straps were 5 mm and 150 mm, respectively. The steel strap width was chosen to ensure yielding of the cross section in tension prior to the net section fracture at the bolt locations. The measured yield strength of the steel straps was 227 MPa. For connecting the steel straps to building, 14 mm diameter holes were drilled to steel members, walls, and lintels. High strength threaded rods were used for connections. Minor diameter of threaded rods was 12 mm and the yield strength of these rods was 512 MPa. Threaded rods were inserted into the drilled holes and the steel member was fixed to the masonry building from both sides symmetrically by using nuts. Connection was accomplished with $400 \times 400 \times 5$ mm steel plates. Steel plates were fixed with threaded rods that were inserted from the holes drilled to lintels by using nuts from both sides symmetrically. Steel plates that were situated at interior and exterior side of upper lintel were



(a) Top Lintel Connection Details of Steel Members

(b) Bottom Lintel Connection Details of Steel Members

Fig. 5 Details of connections

connected with common threaded rods and nuts inserted into drilled holes through upper lintel. Due to thicker bottom lintel extended into interior of the building, connection steel plates were bend like a “L” shape at the bottom end where they were connected with epoxy injected threaded rod to bottom lintel at interior of the building. The details of the connection of steel plates to bottom and top lintel with connection details of steel straps to plates are given at Fig. 5. The view of N-1 wall after rehabilitation was applied is given at Fig. 6.

Diagonal and vertical steel straps were used at both the interior and the exterior sides of the walls that were situated at both sides of the window and door openings. Steel straps were added on both

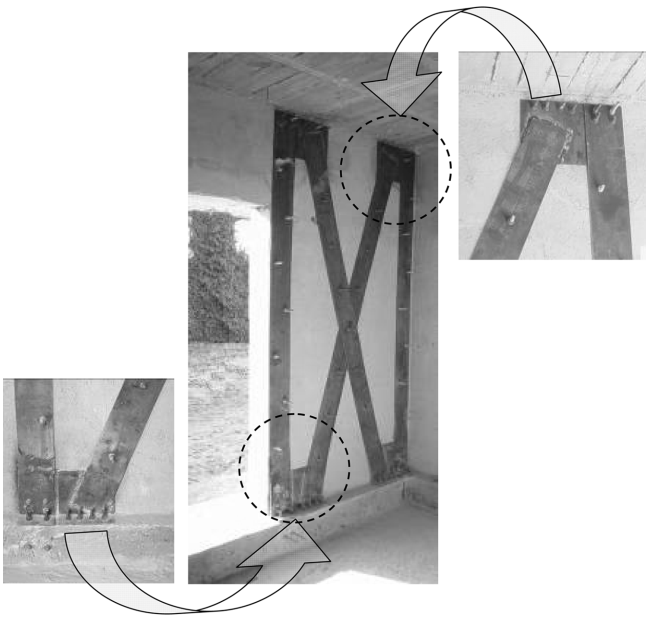


Fig. 6 North side N1 wall of test building after rehabilitation completed

Fig. 7 Test building after rehabilitation

sides of the wall to prevent an eccentric stiffness and strength distribution that may cause twisting of the retrofitted walls. The diagonal steel straps that extend between the corners of the wall strengthened the wall segments and prevented the diagonal tension failure and compression crushing under the shear forces. The vertical straps provide a stable ductile flexural behavior to the walls and prevent tipping. Stiff steel plates and high strength anchors connecting the straps to the top and bottom lintels prevent sliding of the wall. The rehabilitated test building is shown in Fig. 7. Symmetrical straps were connected by tightening nuts at the end of threaded rods from both sides of the walls. Staggered steel threaded rods were placed with 300 mm spacing along the steel straps. The spacing between these bolts was chosen to prevent elastic buckling of the strap. These bolts were also utilized to brace the steel straps and confine the masonry in between them. In addition, use of many bolts were prevented the localization of the load carrying property of the steel plates and were distributed the load all over the masonry wall.

As a first step, all places at lintels and steel members where the connections were made were determined and marked. Then all marked places were drilled and cleaned. After finishing the connection between lintels and steel plates, steel straps along the walls were placed carefully. For the sake of establishing new successful load carrying system, all of the steel members must be correctly integrated with the existing structure. This can be achieved with good workmanship. In this study, the successes of the rehabilitation of test building were mostly depended on the successful and accurate workmanship.

2.3 Test setup, instrumentation and testing procedure

The test was done by using a shock table. A shock table with 6000×6000 mm dimensions was constructed on a rigid concrete platform and was supported with elastomeric supports. The shock table was freely vibrated after pushing and releasing in one direction. The amplitude of the displacement of the table was between 10 and 65 mm. The test building was damaged in the first test and rehabilitated with steel straps, and then tested again. The building was tested along the east-west direction, which is weak due to window and door openings. 18 free vibrations were applied to the building. Each vibration was damped in 2 seconds. Two accelerometers were attached to the roof slab and the shock table for obtaining relative time acceleration data from each loading. Typical measured time-acceleration graphs are given Fig. 8. These acceleration graphs were used for evaluating the results. Lateral forces that were applied to the test building were calculated by the one degree of freedom system assumption. Lateral displacement of the test building at the direction of loading was calculated by numerical integration of the time-acceleration. In addition, the natural vibration period for each loading was calculated by using the response function of the test building that was obtained from the Fourier transform of time-acceleration Cherry (1968).

3. Experimental results

3.1 Observations during test

The test building was heavily damaged after the first test. The extent of damage is seen from the photographs that are given in Fig. 9. A bad joint at the wall of the building caused a lateral cracks initiation and propagation at 82.1 kN lateral load level. The test building was separated into two parts along this bad joint. Rocking was observed after repeating the loading of masonry structure

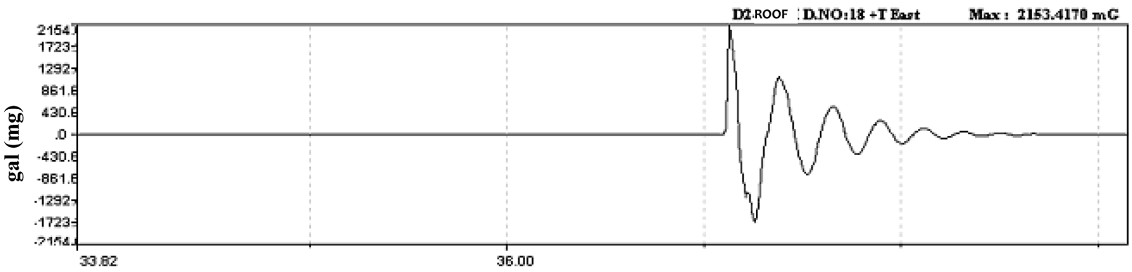
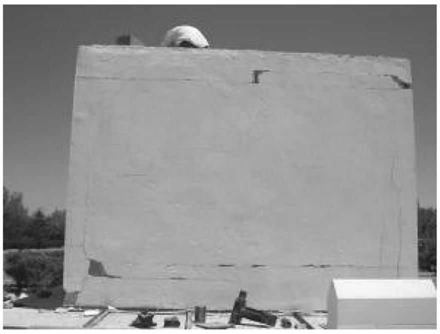


Fig. 8 Measured acceleration graph of building



a) Bad Joint Sliding, Rocking and Toe Crushing Damage of Walls

b) Diagonal Tension Crack Damage of South Side of Walls

c) Bad Joint Sliding and Diagonal Tension Crack Damage of North Side of Walls

Fig. 9 Failure mechanism of test building after test-1

with toe crushing at the outer corners. Diagonal tension cracks were observed starting from the corner of window and door openings at 211.09 kN lateral load level. These diagonal tension cracks were propagated through the vertical and horizontal head joints of the masonry wall, and some of the clay bricks were crushed under the compression load. Diagonal cracks widened and mortar detached in the following loading steps. Test building slid along the foundation lintel due to bad joint sliding at 247.02 kN lateral load level.

Damaged test building was retested after rehabilitation. During the second test, no new crack initiation or damage was observed. Steel straps limited the lateral displacement of the masonry structure and provided stability to the damaged walls. Rehabilitation with steel straps prevented the failure of the building. The anchor bolts across the wall and diagonal straps provided some confinement to the enclosed shear damaged masonry wall which in turn improved the behavior of the masonry wall in the compression zone.

3.2 Evaluation of Test Results

Results of the experiments are summarized in Table 2. Although the maximum lateral accelerations for the two tests were the same, lateral forces that acted on the building at the second test were larger than the first test due to the extra mass of steel straps that were used for rehabilitation. Graphics of the lateral load coefficient (Lateral load/mass of the structure) is given in Fig. 10 for both experiments before and after the rehabilitation. Although the brittle masonry walls were

Table 2 Test results

Measurements		TEST-1	TEST-2
Maximum Acceleration at Roof (m/sec^2)		2.2 g	2.2 g
Maximum Lateral Load (kN)		247.0	261.6
Lateral Displacement at Maximum Load (mm)		45.4	7.6
Maximum Drift Ratio (%)		1.68	0.28
Natural Period of Structure (sec)	Initial	0.068	0.067
	End of Test	0.11	0.082
Lateral Stiffness of Structure (kN/mm)	Initial	97.7	107.6
	End of Test	40.5	71.2

Note: TEST 1: Damaging test of building
TEST 2: Test after rehabilitation
Ground acceleration g=9.81 m/sec^2

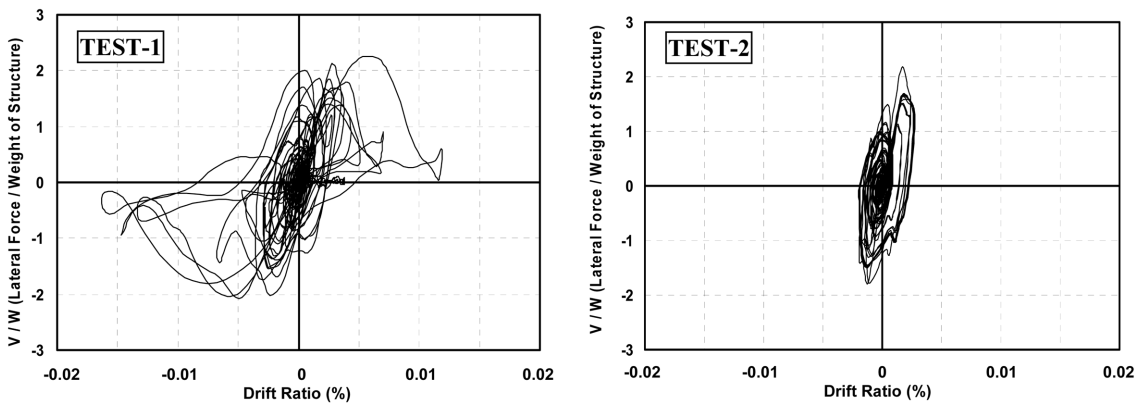


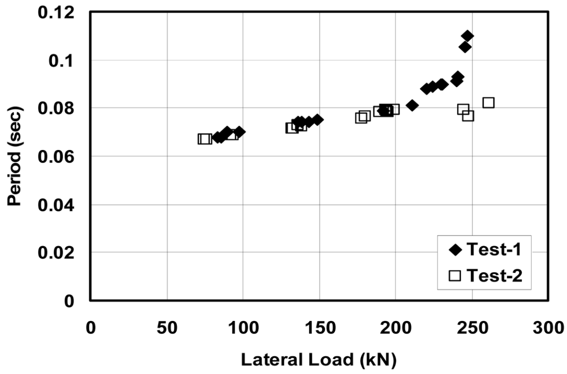
Fig. 10 Load coefficient-drift ratio graphs at maximum load

damaged, steel straps successfully limited storey drift ratio. Maximum storey drift ratio was 0.28% after the application of steel straps for rehabilitation. Steel straps provided stability to the damaged masonry walls, and reduced lateral displacement of the building.

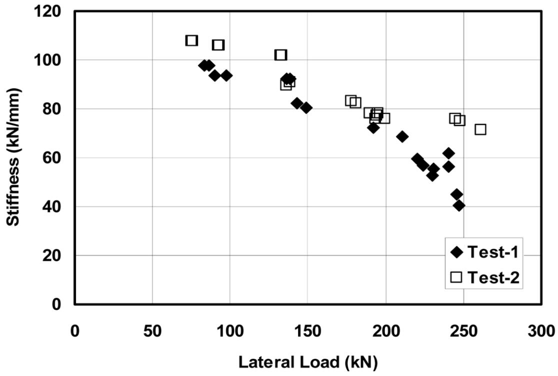
Changes in the natural vibration period, lateral stiffness and storey drift ratio are shown in Fig. 11. The applied rehabilitation technique reduced building's period, and significantly increased the lateral stiffness. The lateral stiffness of the building was reduced by 60% compared to the initial stiffness after the first test. The lateral stiffness of the damaged building was restored after rehabilitation with steel straps. The lateral stiffness of the building was nearly the same before starting the second test. After finishing the test of the rehabilitated building, the lateral stiffness of the building was dropped by 34%.

The natural period of the test building was measured as 0.068 seconds. After the first damaging test, the natural period was increased by 1.55 times. Rehabilitation with steel straps reduced the natural period. The natural period of the rehabilitated building at the beginning of the second test was measured as 0.067 seconds, and was increased by 1.2 times after the test was finished. Rehabilitation prevented the increase in period of the damaged building.

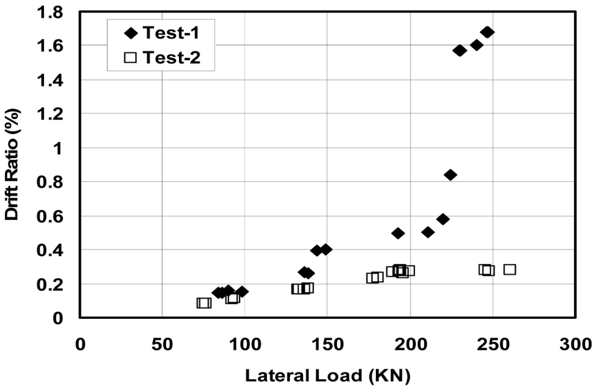
The lateral load carrying capacity of the north and south walls of the test building was calculated by equations given in FEMA 356 (2000) regulations for unreinforced masonry walls' strength



a) Lateral Load-Period Graphs



b) Lateral Load-Stiffness Graphs



c) Lateral Load-Drift Ratio Graphs

Fig. 11 Test result comparisons

(Rocking and Toe Crushing, Diagonal Tension Crack, Bad Joint Sliding). The measured maximum lateral strength and the analytically calculated values for the wall parts are given in Table 3. The obtained test values are larger than the analytical values. Experimental bad joint sliding and diagonal tension crack loads were 6% and 10% larger than the analytical results, respectively.

Table 3 Comparison of experimental and analytical results

Walls	Failure Modes of FEMA 356 [7]								
	Vr & Vtc (kN) Rocking & toe crushing capacity			Vdt (kN) Diagonal tension crack capacity			Vbjs (kN) Bad joint slide capacity		
	Experimental	Analytical		Experimental	Analytical		Experimental	Analytical	
		Walls	Structure		Walls	Structure		Walls	Structure
N1	82.1	14.54	65.68	211.09	39.80	191.94	247.02	53.14	233.68
N2		18.30			56.17			63.70	
S1		14.54			39.80			53.14	
S2		18.30			56.17			63.70	

*N1 and N2 are north walls of building; S1 and S2 are south walls of building

Experimental rocking and toe crushing load values were larger than analytical results by 25%.

4. Conclusions

In this study, a full scale one storey masonry building was tested on shock table. After heavily damaging the masonry building in the first test, the second test was performed on the building rehabilitated with vertical and diagonal steel straps. Effectiveness of the rehabilitation technique, and the improvements obtained by the rehabilitation on the damaged structure were investigated. The following results were obtained from the tests;

- Wide shear cracks were initiated and propagated at the first damaging test. Bad joint caused sliding, and rocking of the masonry building was also observed. The load capacities calculated with equations that were suggested by FEMA356 (2000) regulation were found to be consistent with experimental results.
- Rehabilitation of masonry structures by using steel straps is a successful technique. Steel straps prevented separation of cracks, new crack formation, and propagation. In addition, they increased the lateral load capacity and lateral stiffness of the walls. Steel straps decreased the lateral displacement of the masonry building, and prevented the out of plane rocking of the walls. The natural vibration period of the building was also reduced with this technique.
- After the damaging test of the masonry building, lateral stiffness of the wall were reduced 141%, the natural vibration period of the building was elongated 12 times, and story drift ratio was increased 62%. When the damaged masonry walls were rehabilitated with vertical and diagonal steel straps, damaged walls were gained their lost lateral stiffness again, natural period of the building was reduced, and story drift ratio was dropped. As a result building seismic performance was significantly improved.
- Steel straps and joints carried lateral loads successfully and no damage was observed at joints. Use of many bolts were prevented the localization of the load carrying property of the steel plates and were distributed the load all over the masonry wall. As a result local crushing of clay bricks was prevented and damage was kept under control. The details and connections used to ensure continuity between the steel strap system and the base and top lintel enhanced the sliding friction resistance.
- Steel straps were significantly reduced story drift ratio of the earthquake damaged building. The maximum story drift ratio was measured as 1.68% during damaging part of the test. Maximum storey drift ratio was restored up to 0.28% after the application of steel straps.
- All of the steel members must be correctly integrated with the existing structure for achieving successful load carrying system. This can be reached with good workmanship. In this study, the successes of the rehabilitation of test building were mostly depended on the successful and accurate workmanship.

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Conversion factors

1 mm = 0.039 in

1 mm^2 = 0.00152 in^2

1 kN = 0.2248 kips

1 MPa =145 psi