SHANKING TABLE TESTS ON PP-BAND RETROFITTING OF 1/4 SCALE UNREINFORCED STONE MASONRY MODELS

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ABSTRACT: This paper introduces a technically feasible and economically affordable PP-band (polypropylene bands) retrofitting for low earthquake resistant masonry structures in developing countries. Results of the material tests and shaking table tests on building models show that the PP-band retrofitting technique can enhance safety of both existing and new masonry buildings even in worst case scenario of earthquake ground motion like JMA7 seismic intensity. Therefore, proposed method can be one of the optimum solutions for promoting safer building construction in developing countries and contribute earthquake disaster mitigation in the future.

Key Words: stone masonry, polypropylene band, shaking table test, arias intensity

INTRODUCTION

Masonry is the oldest building material. In spite of this, the technological development of masonry in earthquake engineering has lagged behind compared to other structural materials like concrete and steel. Therefore, in earthquake prone regions of the world have resulted in a large number of casualties due to the collapse of this type of structures. This is a serious problem for the societies. Apparently, its solution is straight forward: retrofitting the existing structures. When we propose the retrofitting in developing countries, retrofitting method should respond to the structural demand on the strength and/or deformability as well as to availability of material with low cost, including manufacturing and delivery, practicability of construction method and durability in each region. Considering these issues of developing appropriate seismic retrofitting techniques for masonry buildings to reduce the possible number of casualties due to future earthquakes in developing countries, a technically feasible and economically affordable PP-band (polypropylene bands; PP-band is commonly used for packing.) retrofitting technique has been developed and many different aspects have been studied by Meguro Laboratory, Institute of Industrial Science, The University of Tokyo.

Masonry walls made of regular shape brick units have been widely studied both from experimental and numerical point of view, but scarce experimental information is available for shapeless stone masonry walls that constitute the material still used in the construction of non-engineering structures. Therefore, the present work aims at increasing the insight about the behavior of typical shapeless masonry structure under static and dynamic loading.

Basic experimental study of PP-band retrofitted masonry wallette made of shapeless stone has been done, and some aspects have been studied by Meguro lab (Sakurai 2009). In order to verify the

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suitability of the PP-band retrofitting technique on shapeless stone masonry, further experimental program was designed and executed.

A real scale model test makes possible to obtain data similar to real structures. However, it requires large size testing facilities and large amount research funds, so it is difficult to execute parametric tests by using full scaled models. Recently, structural tests of scaled models become well-known as the overall behavior of the system can be also understood from scaled model. In these experimental program ¼ scale models was used to investigate the static and dynamic behavior of masonry walls.

AXIAL TENSILE TEST OF POLYPROPYLENE BANDS

Preliminary testing of the PP-band was carried out to check its deformational properties and strength. To determine the modulus of elasticity and ultimate strain, three bands were tested under uni-axial tensile test. The test was carried out under displacement control method. The results are shown in **Figure 1**. To calculate the stress in the band, its nominal cross section $15.5 \times 0.6mm^2$ was used.

All the bands exhibited a large deformation capacity, with more than 13% axial strain. The stress-strain curve is fairly bilinear with an initial and residual modulus of elasticity of 3.2GPa and 1.0GPa, respectively. Given its large deformation capacity, it is expected that it will contribute to improve the structure ductility.

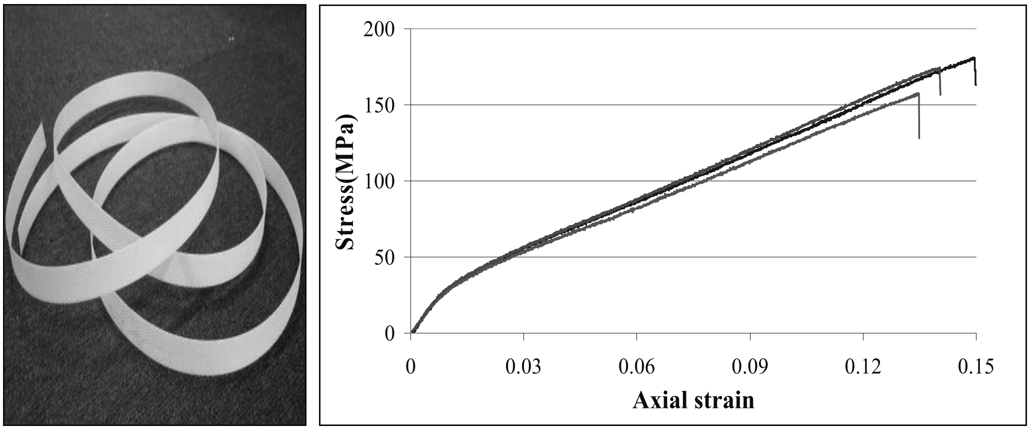

Figure 1. Polypropylene band used for retrofitting (left) and Behavior of PP-band under tension (right)

DIAGONAL SHEAR TEST

To evaluate the beneficial effects of the PP-band mesh retrofitted method, diagonal shear tests were carried out using masonry wallettes with and without retrofitting for shapeless stone. The wallette dimensions were $300 \times 300 \times 150mm^3$ and consisted of 5 stone rows of 3 stones each, and 2 stones in the direction of depth (**Figure 2**). A Cement/Water ratio equal to 0.30 was used. Four wire connectors were used to link the meshes attached from both surfaces of the wallette. Specimens were tested 28 days after construction, under displacement control loading condition. The loading rate was 0.5mm/min up to first 10mm of diagonal deformation, and then it was increased to 2mm/min for remaining deformation.

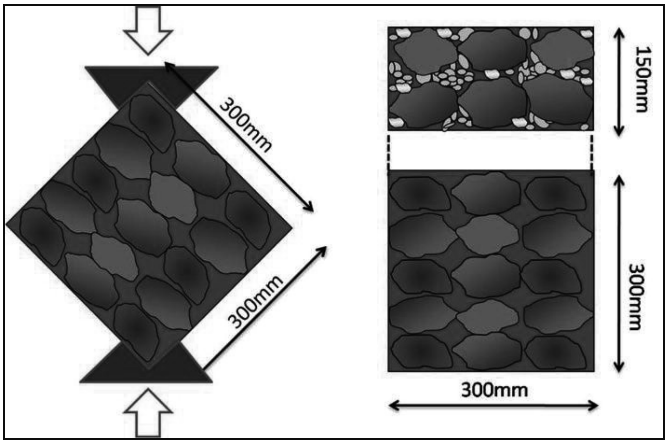

Figure 2. Diagonal shear test specimen dimensions

Figure 3 shows the non-retrofitted and retrofitted specimens at the end of the test, which corresponded to strain, equal to 0.8% and 16%, respectively. In the non-retrofitted case, the specimens split in two pieces just after the first diagonal crack occurred and no residual strength was left. In the retrofitted case, on the other hand, diagonal cracks appear progressively, each new crack followed by a small strength drop. As strain became larger, due to PP-band was penetrated to the specimens, increase of strength was not observed with strain. Although at the end of the test almost all the mortar joints were cracked, the retrofitted wallettes did not lose stability.

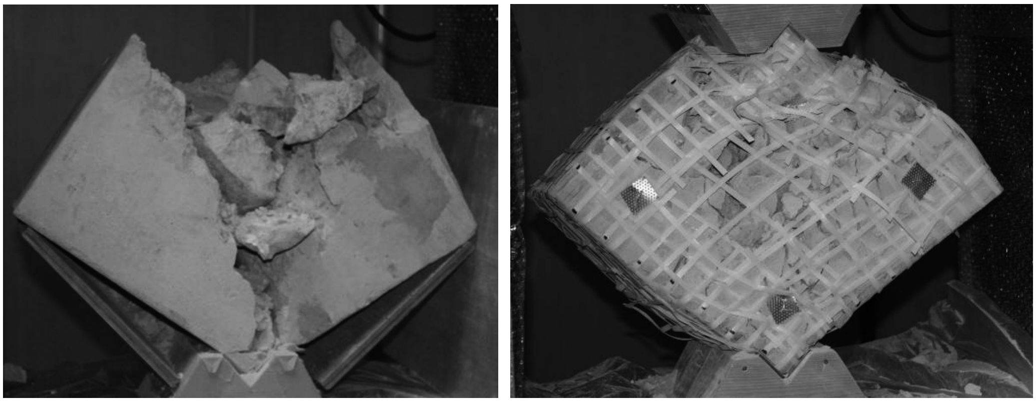

Figure 3. Non-retrofitted (left) and retrofitted (right) specimen failure pattern.

Figure 4 shows the diagonal shear stress variation with strain for the non-retrofitted and retrofitted specimens. In the non-retrofitted case, the average initial strength was 0.08MPa, there was no residual strength after first crack occurred and the specimens split into two pieces at strain equal to 0.8%. In the retrofitted case, although the initial crack was followed by a sharp drop, at least 50% of the peak stress remained. The final strength of the specimen was equal to 0.046MPa and specimen didn’t break even at the strain equal to 16%. Which indicate that; retrofitted specimen was at least 20 times more ductile than non-retrofitted one.

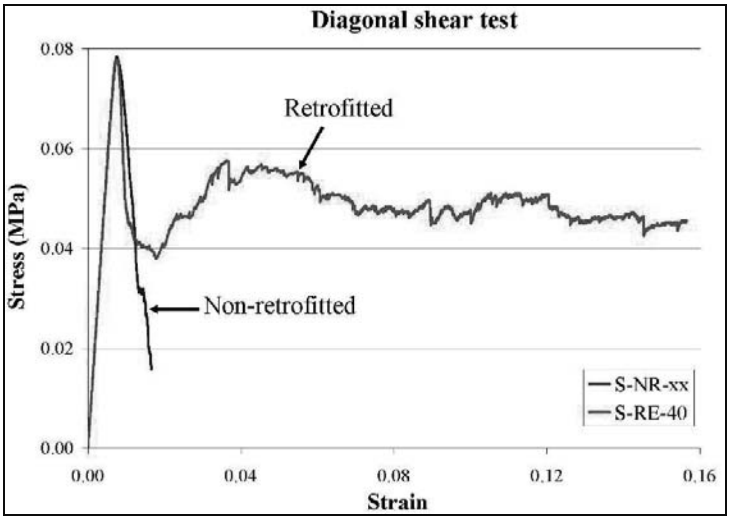

Figure 4. Diagonal shear stress vs. Strain for shapeless stone masonry wallette

SHAKING TABLE TESTS

Considering the shaking table size and allowable loading condition, the model scaling factor adopted was 1:4 as shown in **Figure 5**. Two models were used for shaking table test. The dimensions of both building models were 950mmx950mmx720mm with 100mm thick walls. The sizes of door and window in opposite walls were 290mmx480mm and 370mmx240mm, respectively. Both models were represented one-storey box-like building with timber roof; one model was non-retrofitted and other model was retrofitted with PP-band mesh after construction.

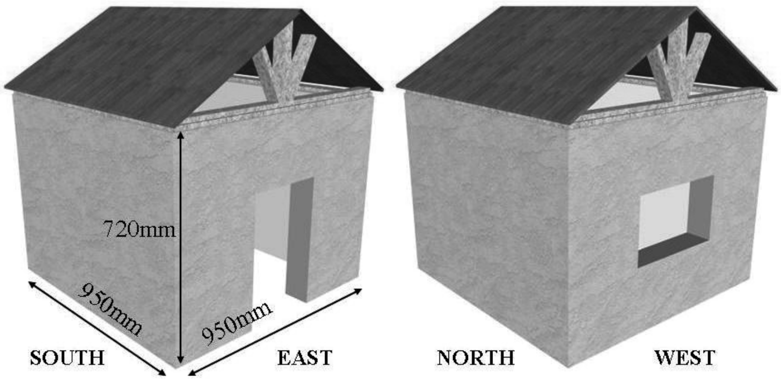

Figure 5. Model dimension (in mm)

Specimens are consisted of 18 rows of 60 stone sets in each layer except openings. It took two days for construction of one specimen. The first 10 rows were constructed in first day and remaining rows were done in following day. The cross-section of the band used was 6mm×0.32mm and the pitch of the mesh was 40mm. The mortar with the mixture ratio of cement, lime and sand=1:7:19 and Cement/Water ratio=0.15% was used for stone masonry to simulate stone masonry buildings in developing countries.

Figure 6. Detail of stone used for construction.

Input motion

Simple easy-to-use sinusoidal motions of frequencies ranging from 2Hz to 35 Hz and amplitudes ranging from 0.05g to 1.4g were applied to the specimens to obtain the dynamic response of both retrofitted and non-retrofitted structures. This simple input motion was applied because of its adequacy for later use in the numerical modeling. **Figure 7** shows the typical shape of the applied sinusoidal wave input motion.

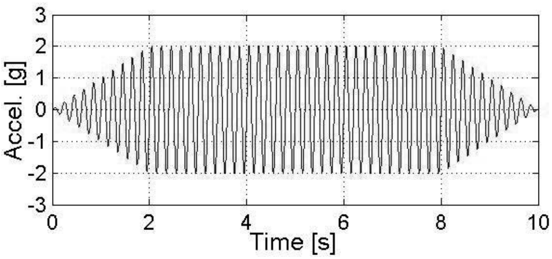

Figure 7. Typical Shape of Input Sinusoidal Motion

Loading was started with a sweep motion of amplitude 0.05g with all frequencies from 2Hz to 35Hz for identifying the dynamic properties of the models. The numbers in **Table 1** indicate the run numbers. General trend of loading was from high frequency to low frequency and from lower amplitude to higher amplitude. Higher frequency motions were skipped towards the end of the runs.

Table 1. Loading Sequence

Amplitude	Frequency							
	2	5	10	15	20	25	30	35
1.4g		50						
1.2g	54	49						
1.0g		48						
0.8g	53	47	43	40	37	34	31	28
0.6g	52	45	42	39	36	33	30	27
0.4g	51	44	41	38	35	32	29	26
0.2g	46	25	24	23	22	21	20	19
0.1g	18	17	16	15	14	13	12	11
0.05g	10	09	08	07	06	05	04	03
Sweep	01,02							

To assess the global and local behavior, specimens were instrumented to measure accelerations and displacements. During the tests, twenty four accelerometers, eighteen on house and six on the roof were installed at the location shown in **Figure 8**. The number of accelerometers was 16, 4 and 4 in the exciting, transverse and vertical direction respectively.

Five lasers, in N-S direction were used to measure displacements. The locations of laser measuring instruments are shown in **Figure 8**. L1, L2, L3 aimed at obtaining the wall deformation at the top level in the direction of shaking. Laser L4 recorded the facade wall deformation at the centre. Laser L5 recorded the deformation at the base. The measured data were recorded continuously throughout the tests. The sampling rate was 1/500 sec in the all runs.

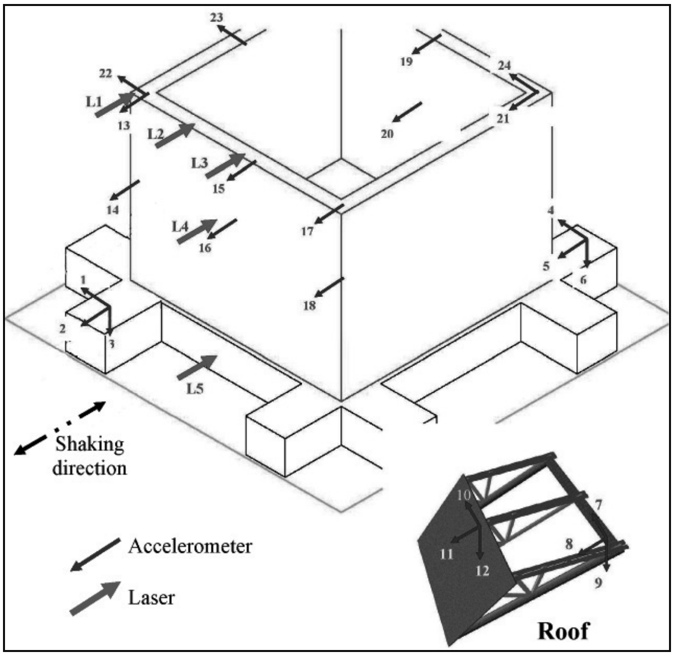

Figure 8. Location of accelerometers and lasers

Crack propagation

For non-retrofitted specimen up to Run 27, no major crack was observed. Major cracks were observed closer to openings from Run 28. At the run 28, crack was observed at the top corner of the door and window opening, and it propagates up to top layer of the wall. After that, cracks widened with each successive run.

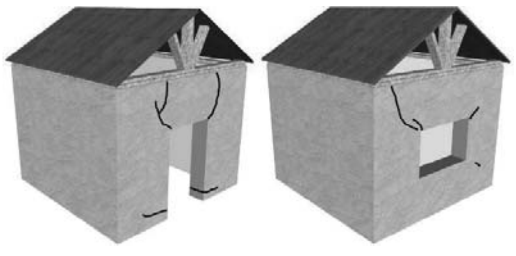

Figure 9. Crack patterns observed on non-retrofitted model after Run 28

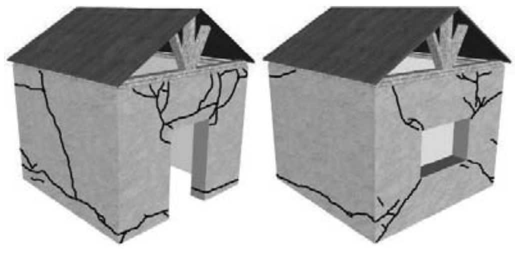

Figure 10. Crack patterns observed on for non-retrofitted model after Run 46

At run 45, there were large amount cracks observed in walls in the direction of shaking. Exciting cracks widened and connection between adjacent walls was become weak. In case of walls perpendicular to shaking direction, top part of the east wall (part, above the door opening) was separated from the specimen. At run 47, all top part of the wall with opening was totally separated from the specimen, and some parts were fallen from a specimen. Now the all walls were separated and act independent and this finally led to the structure collapse.

Figure 11. Non-retrofitted model after Run 48

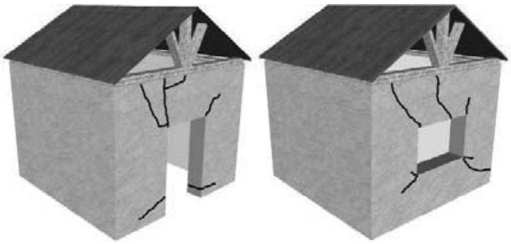

Figure 12. Crack patterns observed on retrofitted model after Run 31

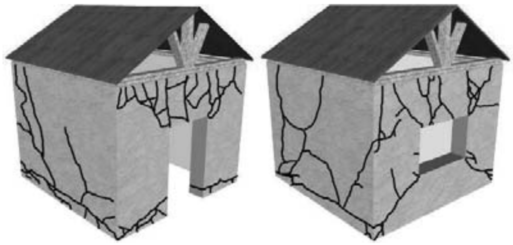

Figure 13. Crack patterns observed on for retrofitted model after Run 48

Figure 14. Retrofitted model after Run 48

Figure 15. Retrofitted model after Run 51

For retrofitted specimen up to run 30, no major crack was observed in this model. Major cracks were observed closer to openings from Run 31. After those new cracks appear in each run and cracks widened with each successive run, thus, extensive cracking was observed. Although the PP-band mesh kept the structure integral during the shaking. In later stages, there was significant permanent deformation of the structure. At the final stage of the test, run 52, with 37.3mm base displacement, 4 times more than the input displacement applied in run 48 and 1.5 times more velocity, virtually the building had substantial permanent deformations. However, building did not loose the overall integrity as well as stability and collapse was prevented in such a high intensity of shaking.

Analysis on test results based on JMA Scale

The performances of the models were assessed based on the damage level of the buildings at different levels of shaking. Performances were evaluated in reference to five levels of performances: light structural damage, moderate structural damage, heavy structural damage, partially collapse, and collapse.

Table 2. Damage categories

Category	Damage extension
D0: No damage	No damage to structure
D1: Light structural damage	Hair line cracks in very few walls. The structural resistance capacity did not decrease noticeably.
D2: Moderate structural damage	Small cracks were observed on masonry walls. The structure resistance capacity decreased partially.
D3: Heavy structural damage	Large and deep cracks were observed on masonry walls. Some bricks are fallen down. Failure in connection between two walls.
D4: Partially collapse	Serious failure and Partial structural failure were observed on walls and roofs, respectively. The building was in dangerous condition
D5: Collapse	Structure is totally or partially collapsed.

The Japan Meteorological Agency seismic intensity scale (JMA) is a measure used in Japan to indicate the strength of earthquake ground motions.

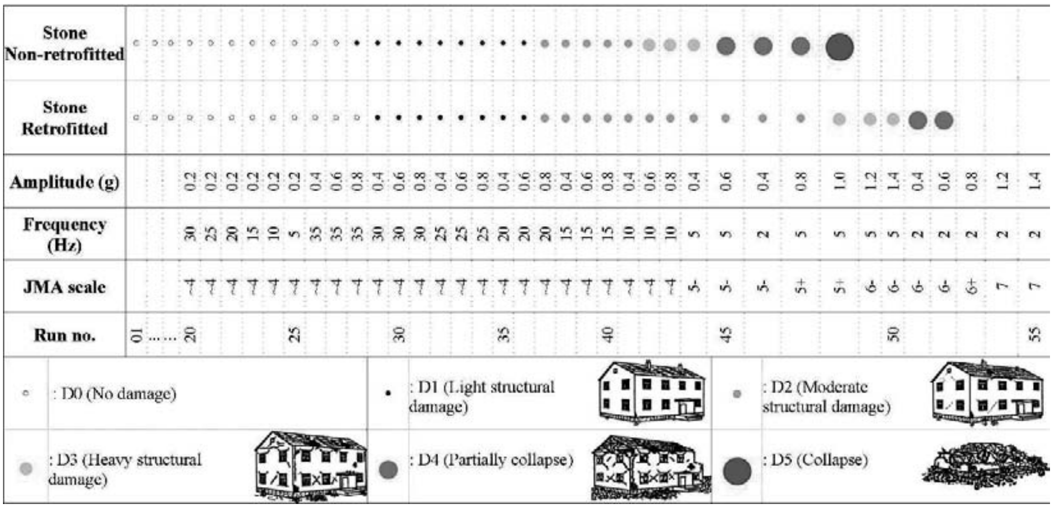

Figure 16. Performance evaluation based on input motion intensity by JMA scale

Figure 16 shows the performances of model houses with different JMA intensities. Collapse of the non-retrofitted building was observed at the 48th run at intensity JMA 5+. The retrofitted building performed heavy structural damage level at the 48th run at which the non-retrofitted building was collapsed. Moreover, heavy structural damage level of performance was maintained until the 50th run. It should be noted again that this model survived 4 more shakings in which many runs were with higher intensities than JMA 5+ at which the non-retrofitted building was totally collapsed before reaching to the final stage at the 52nd run.

From these results, it can be concluded that a structure retrofitted with PP-band meshes would be able to resist against strong aftershocks. Moreover, it proves that even though houses retrofitted with PP-band were cracked due to strong earthquake, it could be repaired and be expected to withstand subsequent strong shakes.

Analysis on test results based on Arias Intensity Scale

The Arias intensity was initially defined by Arias (Arias A., 1970) as

$$I_a = \frac{\pi}{2g}\int_0^t a^2(t)dt \tag{1}$$

and was called scalar intensity. It is directly quantifiable through the acceleration record a(t), integrating it over the total duration of the shaking. The arias intensity is claimed to be measure of the total seismic energy absorbed by the ground.

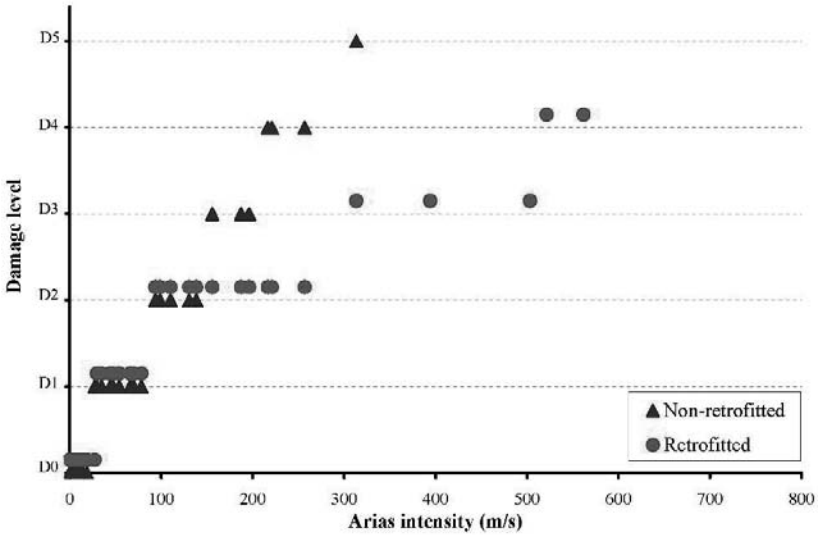

Figure 17. Performance evaluation based on arias intensity scale.

Figure 17 shows the performance levels of each specimen against the dynamic motion based on Arias intensity scale. From the results, retrofitted model damage level performance was at least three times better than that of the non-retrofitted model.

CONCLUSION

This paper discusses the results of diagonal shear tests and shaking table tests that were carried out using non-retrofitted and retrofitted models by PP-band meshes.

- The effect of the PP-band meshes was not observed before the appearance of initial cracking. However, after cracking, they effectively helped to increase the ductility of walls parallel to the shake direction, i.e. subjected to in-plane loading, to prevent the toppling of the walls perpendicular to the shake, i.e. subjected to out-of-plane loading, and to keep the integrity of the structure by limiting corner damage. With this mechanism, PP-band mesh could avoid the typical failure modes observed in masonry structures.

- A scaled dwelling model with PP-band mesh retrofitting was able to withstand larger and more repeatable shaking than that without PP band retrofitting.
- Considering the easiness of installation and inexpensiveness of PP-band mesh, it can be considered as one of the best solutions to overcome the quality deficit of existing building stock in developing countries and therefore, reduce the number of human fatalities in future seismic events.

From the experimental results, it was found that PP-band retrofitting technique proposed can enhance safety of both existing and new masonry buildings even in the worst case scenario of earthquake ground motion like JMA 7 intensity. Therefore, proposed method can be one of the optimum solutions for promoting safer building construction in developing countries and can contribute earthquake disaster in the future.

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