A STEP TOWARDS THE FORMULATION OF A SIMPLE METHOD TO DESIGN PP-BAND MESH RETROFITTING FOR ADOBE/MASONRY HOUSES

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ABSTRACT :

Past earthquakes have shown that the collapse of seismically weak adobe/masonry structures is responsible for most of the fatalities in developing countries. It is, thus, urgent to improve their seismic performance in order to reduce future casualties and to protect the existing housing stock. To encourage seismic retrofitting, inexpensive and easy to implement technical solutions are desirable. Retrofitting by polypropylene band (PP-band) meshes satisfies these requirements. These bands, commonly used for packing, are resistant, inexpensive, durable and worldwide available.

Experiments and advanced numerical simulations have shown that PP-band meshes can dramatically increase the seismic capacity of adobe/masonry houses. Nevertheless, a simple yet accurate design method is still needed to optimize the mesh arrangement and assess its performance. PP-band meshes increase the structure ductility and energy dissipation capacity through controlled cracking. However, large deformations during seismic events are expected and therefore, the design method must take this into account. In this paper, a methodology to design PP-band mesh retrofitted structures is outlined and discussed.

KEYWORDS: adobe house, design method, PP-band mesh, retrofitting, masonry

1. INTRODUCTION

Collapse of weak adobe/masonry houses is responsible for most of the fatalities due to earthquakes in developing countries. Furthermore, the consequent property losses are a threat to the sustainable development of these regions. The only way to change this situation is to increase the seismic resistance of the existing housing stock. Because people living in this type of structures have limited resources, inexpensive and easy to implement solutions are necessary. Retrofitting by polypropylene band (PP-band) meshes satisfies these requirements. These bands commonly used for packing are resistant, inexpensive, durable and worldwide available.

PP-band meshes (see Figure 1) are wrapped on both sides of the walls and attached with wire connectors. After meshes are installed the wall is plastered with either mud, in case of adobe houses, or mortar, in case of brick structures. This cover protects the meshes from the ultraviolet radiation and other external agents, fills any gaps that may be left between the mesh and the wall after its installation, improves the bond between mesh, mortar, and wall, and gives a good appearance to the retrofitted structure.

Experiments and numerical simulations have shown that PP-band meshes improve the seismic performance of otherwise poor earthquake resistant adobe/masonry houses. This is mainly achieved by increasing the structure ductility and energy dissipation capacities. Under moderate ground motions, PP-band meshes provide enough seismic resistance to guaranty limited and controlled cracking of the retrofitted structures. Under extremely strong ground motions, they are expected to prevent or delay the collapse, thus, increasing the survival ratio.

Figure 1 PP-band mesh

Figure 2 PP-band retrofitted house before mortar laying

Although there is plenty of evidence showing the good seismic performance of PP-band retrofitted structures, a simple methodology to design PP-band mesh retrofitting and to assess its seismic response for a particular seismic demand is yet to be developed. In the following sections, such methodology will be outlined and discussed.

2. PROPOSED DESIGN METHODOLOGY

The scope of the design methodology proposed hereinafter is 1-story adobe/masonry houses with flat roofs. Structures with vault/dome roofs are not considered. Figure 3 shows the flowchart of the proposed retrofitting design. The process can be summarized as follows:

1. Determine the original structure strength, V_c, and natural period, T.
2. Calculate the elastic base shear, V, according to the regional seismic code.
3. From the relation between V and V_c, estimate the strength reduction factor, R_d.
4. Choose a certain PP-band mesh density, D, and determine the ductility demand, μ_{dem}, from the μ_{dem} versus R_d graph and also the maximum displacement, $\Delta_{max}=\mu_{dem} \times$ first cracking displacement.
5. Assess Δ_{max}.
   If Δ_{max} is acceptable, proceed with out-of-plane verification.
   If Δ_{max} is unacceptable, reduce the μ_{dem}. Repeat the calculation.
6. Verify that out-of-plane deformations do not cause instability

Determining the properties of existing structures is not easy especially for adobe/masonry houses due to their low quality and great variability. Because these structures are stiff compared to the PP-band stiffness, their initial natural period does not change after retrofitting. Furthermore, experiments have shown that PP-band meshes do not increase the structure strength before cracking. Therefore, it can be assumed that the retrofitted structure will have the same V_c and T as the original, unreinforced, house.

The expected R_d will be fairly high due to the relatively low resistance of the adobe/masonry houses. The higher the reduction factor, the larger the ductility demand will be as shown schematically in Figure 3. Intuitively the larger the PP-band mesh density, the less μ_{dem} for the same R_d. The nature of this relation will be discussed in detail in the following sections.

Large deformations and controlled damage are anticipated in PP-band mesh retrofitted structures. Therefore, the most important points to check in the design are maximum displacements at the corners (maximum acceptable displacement associated with in-plane actions) and the wall body (out-of-plane verification). Secondary order effects should be avoided. Excessive out-of-plane wall deformations will reduce their in-plane resistance capacity. Maximum acceptable displacements and out-of-plane verification will be discussed in subsequent sections.

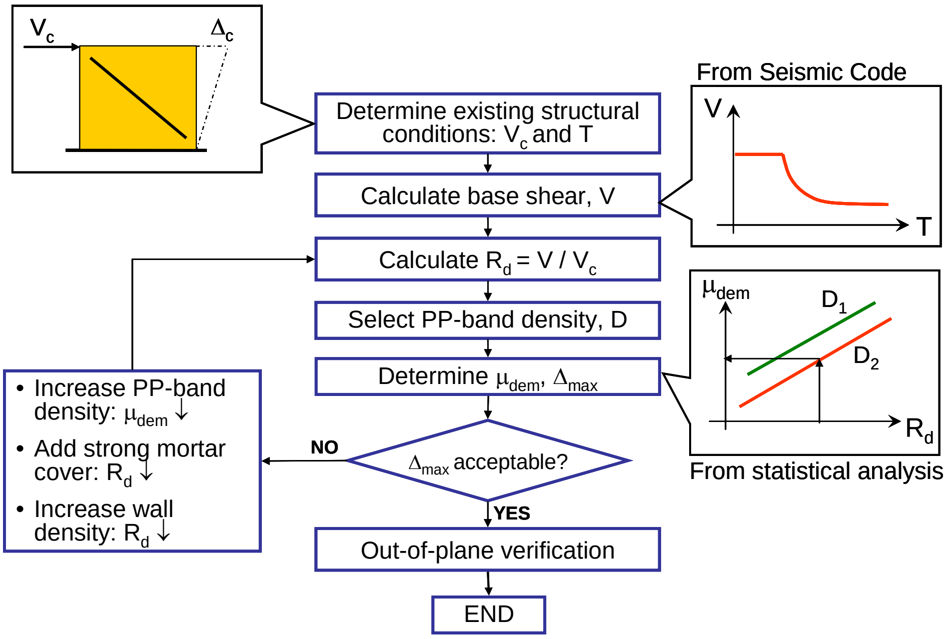


Figure 3 Flowchart of the proposed methodology

If displacements due to in-plane actions are unacceptable, μ_{dem} should be reduced. This can be achieved by increasing the PP-band density. Another solution is to reduce R_d by adding a strong mortar cover or providing additional walls so that the demand on each of them is lower. In the latter case, there will be an increase in mass and as a result V needs to be recalculated. It is also possible to increase the wall density by adding more walls. However, this will change the original floor arrangement and therefore would be more expensive and probably difficult to accept by the house owner.

3. STRENGTH REDUCTION VERSUS DUCTILITY DEMAND

As mentioned in the previous section, the relation between R_d and μ_{dem} for different PP-band mesh densities is needed to estimate the maximum displacement that the structure will experience. In order to develop a simple relation between these two parameters, non-linear time history analyses of several structures subjected to various strong ground motions were carried out.

3.1. Material model

Static monotonic tests have shown that the shear force – lateral deformation curve of a PP-band retrofitted walls can be roughly idealized as shown in the left curve of Figure 4. V_c and Δ_c correspond to the shear strength and cracking deformation of the original wall whereas V_r and K_r correspond to the residual strength and stiffness after the wall cracking. The first two parameters are mainly dependent on the masonry itself, V_r depends on both masonry and PP-band mesh and K_r depends mostly on PP-band. Under cyclic loading, the skeleton curve resembles the monotonic one with a gradually decreasing unloading stiffness.

To model the retrofitted adobe/masonry structures, the skeleton curve was further idealized as shown on the right side graph of Figure 4. This simplification is assumed to be conservative as a fraction of the wall strength is not considered. In the graph:

$$\Delta'_c = \frac{V_r - K_r \Delta_o}{K_o - K_r} \tag{3.1}$$

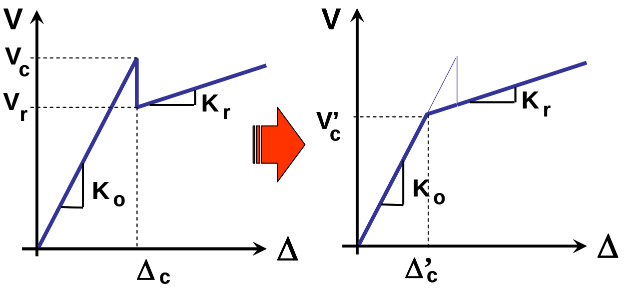


Figure 4 Idealization of shear force versus lateral deformation for a wall retrofitted with PP-band mesh

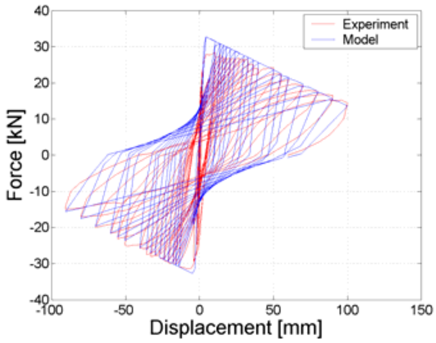


Figure 5 Comparison of experimental results and proposed model

$$V'_c = \frac{K_o}{K_o - K_r}\left(V_r - K_r\Delta_o\right) \tag{3.2}$$

Additionally, the hysteresis was represented with a Modified Clough model with unloading degrading stiffness. Two additional parameters to control the later decay are necessary. In total, the model is completely defined with five parameters. Figure 5 shows the comparison between the force deformation curve obtained with experiments and the proposed model. A fairly good agreement is observed.

3.2. Strong ground motion database

A total of 144 strong ground motion records were considered for the present study as shown in Table 3.1. All of them were recorded at sites with average shear wave velocities higher than 180 m/s in the upper 30 m of the soil profile. In all the cases, the peak ground acceleration (PGA) was larger than 0.1g and they were recorded on free field or the first floor of low-rise buildings. Figures 6 and 7 show the distribution of PGAs and normalized acceleration response spectra for 5% damping. All the records have high frequency contents below 0.4s. The natural period of 1-story adobe/masonry structures fall in this range.

3.3. Analyzed structures

Four structures with mechanical properties representing single story adobe/masonry houses and three different weight roofs as detailed in Table 3.2 were considered for the present study. The parameters were chosen so as to represent one of the two main walls of a 3-m high, 3-m long, 1-story adobe/brick house. In all the cases, V_r/V_o was considered equal to 0.75, a value which experiments have shown is relatively easy to achieve by tightly attaching an adequate volume of PP-band mesh.

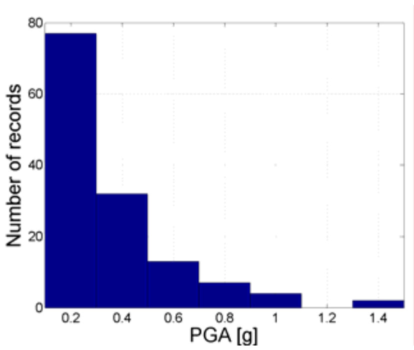


Figure 6 Distribution of the **P**eak **G**round **A**cceleration of the records used in the study

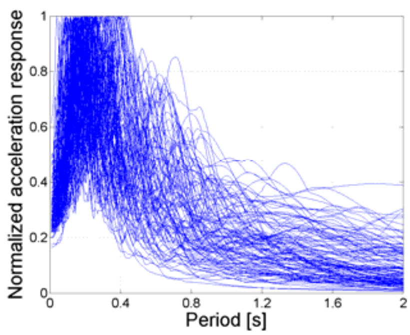


Figure 7 Normalized acceleration response for 5% damping

Table 3.1 Strong ground motion records used for this study

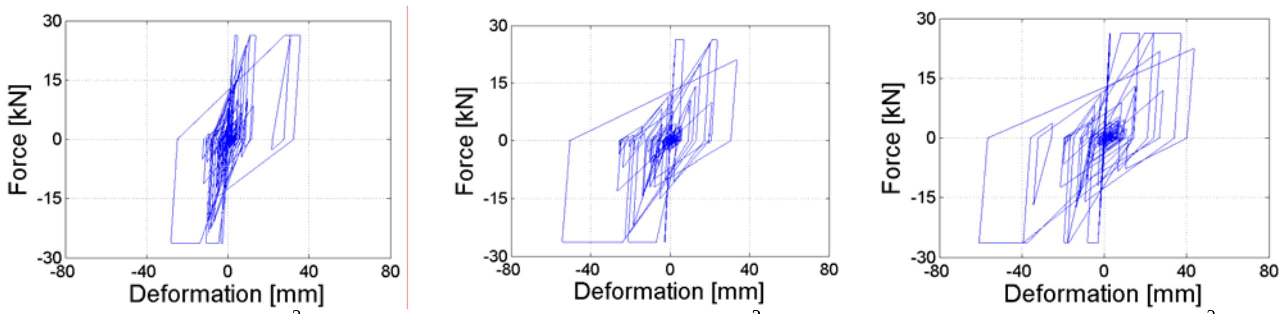
Date	Earthquake	Station
02/07/1987	Baja California (Mw=5.50)	UNAMUCSD 6604
04/25/1992	Cape Mendocino (Mw=7.01)	CDMG 89005, CDMG 89324, CDMG 89509
07/22/1983	Coalinga (Mw=6.36)	CDMG 47T03, USGS 1604, USGS 1606, USGS 1605, USGS 1607, USGS 1608, USGS 1609, USGS 1651
10/15/1979	Imperial Valley (Mw=6.53)	CDMG 5158, USGS 286, USGS 5051, USGS 5053, USGS 5054, USGS 5055, USGS 5056, USGS 5057, USGS 5058, USGS 5059, USGS 5060, USGS 5165, USGS 931, USGS 952, USGS 955
06/28/1992	Landers (Mw=7.28)	CDMG 24577, SCE 24
10/17/1989	Loma Prieta (Mw=6.93)	CDMG 47006, CDMG 47379, CDMG 47380, CDMG 47381, CDMG 57007, CDMG 57064, CDMG 57066, CDMG 57383, CDMG 57425, CDMG 57504, CDMG 58065, CDMG 58130, CDMG 58135, CDMG 58393, UCSC 13, UCSC 14
04/24/1984	Morgan Hill (Mw=6.19)	CDMG 47006, CDMG 47380, CDMG 47381, CDMG 57383, CDMG 57425
01/17/1994	Northridge (Mw=6.69)	CDMG 24087, CDMG 24278, CDMG 24303, CDMG 24399, CDMG 24401, CDMG 24514, CDMG 24592, CDMG 24611, USC 90014, USC 90055, USGS 5080, USGS 655, USGSVA 637
02/09/1971	San Fernando (Mw=6.61)	CDMG 126, CDMG 127, USGS 128, USGS 266
06/28/1991	Sierra Madre (Mw=5.61)	CDMG 24399
10/01/1987	Whittier Narrows (Mw=5.99)	CDMG 14196, CDMG 14368, CDMG 24303, CDMG 24399, CDMG 24401, CDMG 24461, USGS 289, USGS 709
10/04/1987	Whittier Narrows Aftershock (Mw=5.27)	CDMG 24399

Table 3.2 Material properties considered for the study

Structure type	V_c [kN]	K_o [kN/mm]	K_r/K_o	Mass [$\times 10^3$ kg]
Adobe	35	10	0.00, -0.02	8.70, 12.75, 17.25
Brick	100	50	0.00, -0.02	8.70, 12.75, 17.25

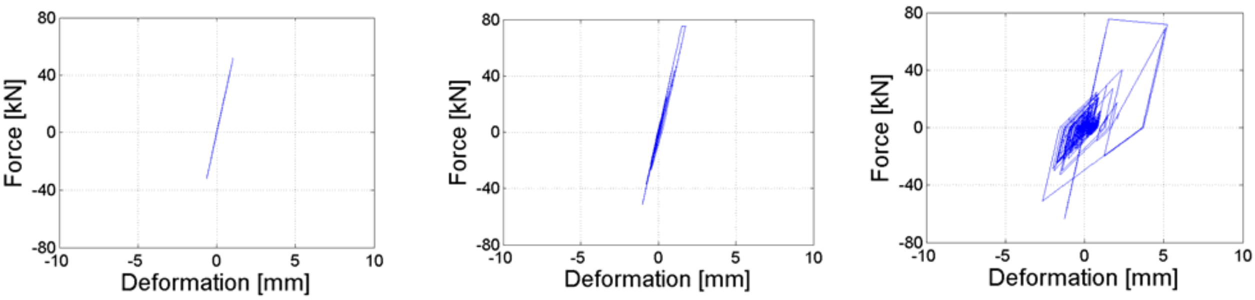
3.4. Results and discussion

Figure 8 and 9 show the force-deformation curves of two groups of structures subjected to the same strong ground motion record. Because the adobe structure has lower strength, the ductility demand is larger. Structures with larger masses experience larger inertial forces and therefore larger ductility demands.



(a) Mass = 8.7 $\times 10^3$ kg (b) Mass = 12.75 $\times 10^3$ kg (c) Mass = 17.25 $\times 10^3$ kg

Figure 8 Results for adobe structure with K_r/K_o = 0.0, input motion: Northridge Eq., Sation USGSVA 637



(a) Mass = 8.7 ×10^3 kg (b) Mass = 12.75 ×10^3 kg (c) Mass = 17.25 ×10^3 kg

Figure 9 Results for brick structure with K_r/K_o = -0.2, input motion: Northridge Eq., Sation USGSVA 637

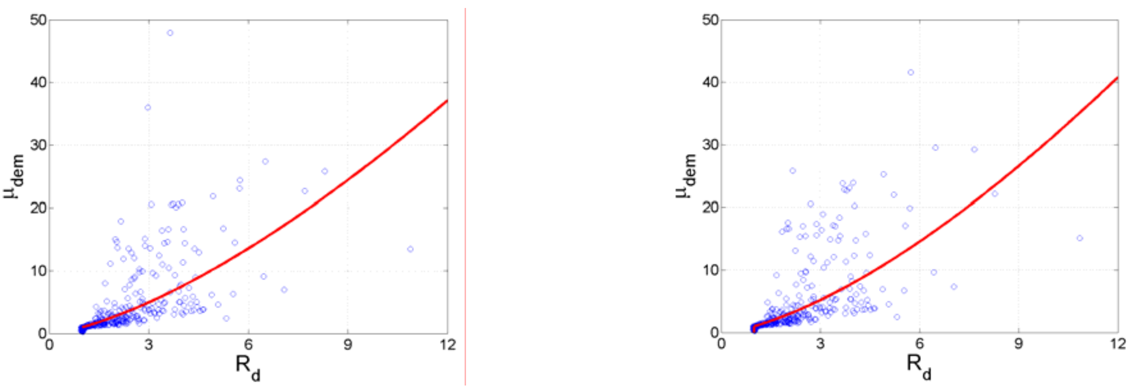
For all the records and structures analyzed, μ_{dem} and R_d were determined and plotted as shown in Figures 10 and 11. Maximum R_d for adobe and masonry structures were 10.9 and 3.4, respectively. The values of μ_{dem} were 41.5 and 23.6 in the same cases. Regression functions were determined for each group as shown in Table 3.3 and are shown in thick lines in Figures 10 and 11. In general, the results are scattered, especially for adobe structures. For instance, for R_d equal to 3, μ_{dem} ranges from 2 to 35, if K_r/K_o=0, and from 2 to 20, if K_r/K_o= -0.2.

Table 3.3 Regression functions obtained for each of the group structures considered in the study

Structure type	K_r/K_o	Regression function	R^2
Adobe	0.00	$\mu_{dem} = 1.0018 \times R_d^{1.4539}$	0.620
	-0.02	$\mu_{dem} = 1.0121 \times R_d^{1.4873}$	0.630
Brick	0.00	$\mu_{dem} = 1.0247 \times R_d^{1.5359}$	0.66
	-0.02	$\mu_{dem} = 0.9905 \times R_d^{1.6512}$	0.71

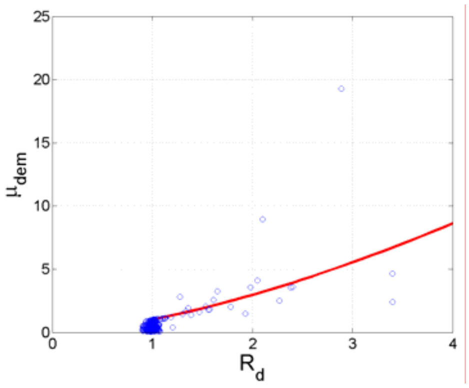
The large scatter does not seem to be caused by the post-peak softening behavior of the structure. Groups with different values of K_r/K_o give similar scattered results. Nor seems it to be caused by the used strong ground motion records, which have similar characteristics as shown in Figure 7. Additional evaluation of the model used is necessary to grasp the causes of the dispersed results.

Even though the results require further evaluation, a few conclusions may be drawn. For instance, it seems that the factor K_r/K_o does not affect considerably the maximum displacements experienced by the structure. Initial and residual strengths (V_o, V_r) are more important as suggested in previous studies. Also, for R_d values lower than 9, μ_{dem} may be considered at most 9 for adobe structures. For masonry, a μ_{dem} of 5, at most, may be expected for R_d up to 4.

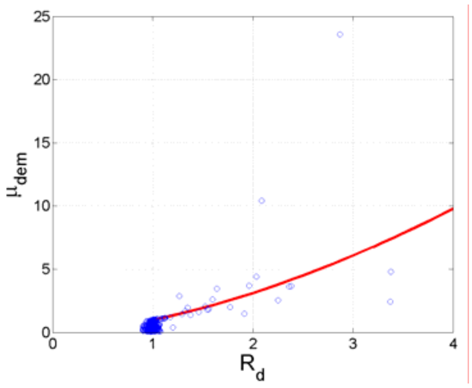


(a) K_r/K_o = 0.0 (b) K_r/K_o = -0.2

Figure 10 Strength reduction versus ductility demand for adobe structures



(a) K_r/K_o = 0.0



(b) K_r/K_o = -0.2

Figure 11 Strength reduction versus ductility demand for masonry structure

A more comprehensive statistical analysis, considering more strong ground motion records and structures with larger R_d is required to reach to a final expression of μ_{dem} as a function of R_d.

4. MAXIMUM ACCEPTABLE DISPLACEMENT

A maximum acceptable displacement or drift should be defined to guaranty the structure stability. Material tests have shown that PP-band retrofitted walls under in-plane loads can tolerate very large drifts, in the order of 10% or more, without losing their in-plane resistance capacity. However, such large deformations along the plane of certain walls will cause excessive out-of-plane deformations on the walls perpendicular to them. If a structural wall is excessively damaged by out-of-plane actions and consequent displacements, its ability to resist in-plane forces will be reduced.

There are two ways to take into account the interaction of the in-plane and out-of-plane actions on the wall in the design. One is to reduce the in-plane resistance with a penalty factor which should be a function of the maximum out-of-plane displacement. Although presently this point is under study, so far there is no model to determine what factor would be appropriate. Another way is to limit the maximum acceptable displacement to a conservative low value. Although more analyses and calculations are required to determine the most appropriate value, at this point, it is recommended to set it as a half of the wall thickness so that the resultant of vertical loads on the wall (under out-of-plane actions) will always fall within the limits of the wall base. Furthermore, out-of-plane deformations in the wall, not only its sides, need to be controlled as explained in the next section.

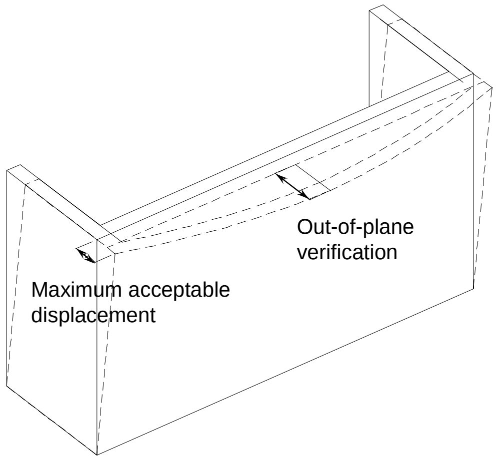


Figure 12 Difference between maximum acceptable displacement and out-of-plane verification

Figure 13 PP-band meshes connecting wall and roof restrain out-of-plane wall displacements

5. OUT-OF-PLANE VERIFICATION

The maximum acceptable displacement discussed in the previous section corresponds to the drift of the walls under in-plane actions or in other words, to the displacement of the walls subjected to out-of-plane actions at their sides. If the unsupported length of the walls under out-of-plane actions is too long, the center of the walls may be subjected to considerable larger displacements perpendicular to their plane. Presently, a model to determine the displacements due to out-of-plane seismic actions for PP-band retrofitted walls is being developed.

Experiments have shown that attaching the PP-band mesh so that it is wrapped around the roof frame as shown in Figure 13 can greatly contribute to control out-of-plane displacements. Whenever possible, it is recommended to install the mesh in this way. Another solution to limit out-of-plane displacements in walls with large length/height ratio is to provide intermediate supports by means of pilasters well attached to the wall with PP-band meshes.

6. CONCLUSIONS

A methodology to design 1-story, flat roof, adobe/masonry houses with PP-band meshes was proposed. Although there are still some issues to be addressed to complete the design procedure, the general process was outlined. It is important to keep in mind that this procedure is just one step towards the development of a simple set of rules of thumb than can be used in the field for determining the most appropriate PP-band mesh arrangement for each particular situation.

Permanent displacements are expected when PP-band mesh retrofitted structures are subjected to strong ground motions. If these displacements are too large, the remaining structure may be either unusable or too expensive to repair. This information is very important to assess the suitability of PP-band mesh retrofitting for different type of structures, taking into account initial investment and eventual reparation costs. A procedure to evaluate permanent displacements of PP-band mesh retrofitted structures will be investigated in the future.

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