Low-Cost Retrofitting Solutions For RC Frames Using Masonry Infill Panels

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SUMMARY:
A large number of reinforced concrete (RC) frame structures built in earthquake-prone areas in developing countries are vulnerable to strong ground motions. In this paper a numerical experiment was conducted in which several idealized prototypes representing RC frame structures of school buildings damaged during the Port-au-Prince earthquake (Haiti, 2010) were hypothetically strengthened by adding elements representing masonry infill walls arranged in different configurations and studied under non-linear dynamic analysis. Each configuration had a different ratio R_m of area of walls in the direction of the ground motion (in plan) to the total floor area. The non linear response of the models under three major earthquakes which PGA 0.5g was estimated numerically. The results were summarized in tentative relationships between R_m and interstory drift, Park&Ang damage indexes, and dissipated energy. For $R_m \geq 4\%$ computed interstory drift ratios did not exceed 1.5%.

Keywords: Masonry infill panels, retrofitting, low-cost retrofitting solution, developing country

1. INTRODUCTION

Masonry infill panels are used worldwide as non structural elements in buildings. Although there are many studies on how do they may prevent damage in structures under strong ground motions (*Matjaz Dolsek et all. 2005;, Matjaz Dolsek et all. 2002; Matjaz Dolsek et all. 2004; Özgür Anil et all. 2007; A.M. Reinhorn et all., 1995; G. Michael Calvi et all. 1994; Armin B. Meharabi et all. 1996; Paolo Negro et all., 1996; Alidad Hashemi et all., 2006)* there is a research gap on simple methods and solutions to use the infill walls as a low-cost retrofit solution. Previous studies highlight that the behaviour of masonry infill panels under cyclic loads is less favourable than other more advanced solutions, such as reinforced concrete walls or hysteretic dampers. The hysteretic curve of the infill wall under cyclic lateral loading exhibits severe pinching and its plastic deformation capacity is very limited in comparison, for example, to hysteretic dampers. Failures in the infill can also compromise the frame because the may lead to “captive columns.” However, one of the main advantages of using masonry infill panels to retrofit existing low-rise reinforced concrete (RC) structures is that they can provide a significant increase of lateral strength and stiffness with low cost. This last reason, together with their low technological requirements for installation, makes masonry infill panels an attractive solution for rapid seismic upgrading RC frames in developing countries. Addition of reinforcement, which does not increase the cost and required skill dramatically has a large impact on performance. A clear example of where this idea may be useful is found in Haiti, where in February 2010 an earthquake in Port-au-Prince damaged thousands of RC frame structures that need to be repaired in a short-term period and with very limited economic resources.

In this context, this paper presents an ongoing investigation on the feasibility of using masonry infill panels in low-rise RC frames structures for seismic retrofitting purposes. This study is focused on school buildings that were severely damaged by the strong ground motion in Port-au-Prince (Haiti, 2010). The purpose of this study is to propose quantitative recommendations of the required area of masonry infill walls that may help prevent the collapse of the structure under a moderate earthquake.

2. DEFINITION OF THE PROTOTYPE BUILDINGS WITHOUT INFILL WALLS

In November 2010, a group of researchers visited Port-au-Prince (Haiti) with the purpose of conducting a preliminary evaluation on the state of buildings that did not collapse after the major earthquake that took place in February 2010 (http://nees.org/resources/1797). Detailed information was collected for each building: dimensions and distribution of structural elements, number of stories, plan layout, and use of the building. This study in focused on schools buildings the structure of which consisted of reinforced concrete (RC) frames.

The variety of school buildings investigated was synthesised in several prototypes with different dimension and distribution of structural elements, number of stories and plan layout. The number of stories ranged from 2 to 3. Other variables defining the prototypes are shown in Table 1. Four prototypes identified in Table 1 as H01, H02, H03 and H04 with two stories, and four counterpart prototypes with three stories were studied. In all prototypes the assumed story height was 3m. The size of the columns was assumed to be 30x30cm. The ratio R_{col} of total area of column's cross section to total area of the first floor ranged from 0.41% to 1.08 and is indicated in Table 1. The average dimension of the beams was 30x50cm. The slab was assumed to be constructed with one-way joists.

Table 1. Type of structural elements in plan

Prototype	In the direction of the ground motion		Perpendicularly to the direction of ground motion		
	Number spans	Span length (m)	Number of spans	Span length (m)	R_{col} %
H01	1	7	8	7	0.41
H02	1	4	5	5	1.08
H03	2	5	4	5	0.68
H04	4	3	4	7	0.54

Because most of the structures damaged by the earthquake of 2010 in Haiti were not designed to withstand strong ground motions, the idealized schools prototypes were designed according to the Spanish code CTE considering only gravity loads. The compressive concrete strength was f_c=25MPa and the yield strength of the steel f_y=400Mpa. The dead load included self weight and a superimposed dead load of 2 kN/m^2 on all floors. The total dead load per unit of area was approximately 8 kN/m^2. The value of the live load assumed was 1 kN/m^2 in the uppermost floor and 2 kN/m^2 in the other floors. Wind and seismic load were not considered to proportion the hypothetical frames.

3. DEFINITION OF PROTOTYPES WITH DIFFERENT INFILL WALL CONFIGURATIONS

Each prototype (bare) frame shown in Table 1 was hypothetically strengthened by adding four different configurations of masonry infill walls (A, B, C and D) as shown in Table 2. The walls were supposed to fill entire frame bays. The infill walls were distributed in plan so that the structure keeps the symmetry in the direction of the ground motion. Each infill wall configuration is associated with a ratio of the area of infill walls in the direction of the ground motion in each story to the floor area of the story, R_m, that is:

$$R_m(\%) = \frac{area\ of\ infill\ walls\ in\ the\ story\ in\ the\ direction\ of\ ground\ motion}{floor\ area\ of\ the\ story} \tag{3.1}$$

Table 2 shows the value of R_m for each prototype and infill wall configuration investigated. When the structure yields and enters in the non-linear range, there is a lengthening of the fundamental elastic period that affects the amount of energy input by the earthquake in the structure. To take into account this lengthening effect, an effective period of vibration T_e has been calculated from the initial elastic fundamental period by using the formulation proposed by Akiyama [10] and it is shown in Table 2.

Table 2. Infill panel configurations

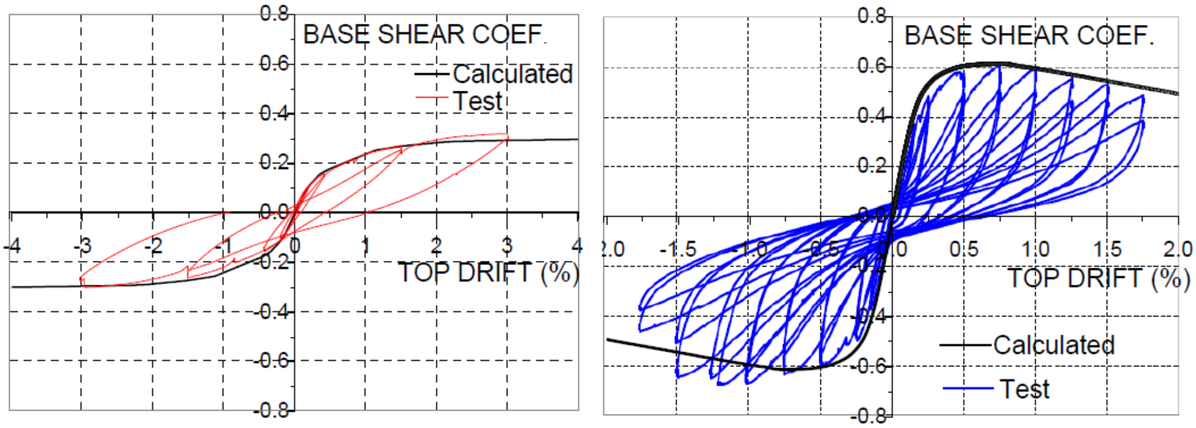
			Infill panel configuration			
Prototype	Number stories	Amount of walls Effective period	A	B	C	D
H01	2	R_m first floor	0	1.5%	1.5%	1.2%
		R_m upper floor	0	1.5%	1.2%	1.2%
		T_e (s)	0.53	0.28	0.29	0.30
	3	R_m first floor	0	1.15%	1.48%	
		R_m middle floor	0	1.15%	1.48%	
		R_m upper floor	0	1.15%	1.48%	
		T_e (s)	0.80	0.46	0.39	
H02	2	R_m first floor	0	1.84%	2.76%	
		R_m upper floor	0	1.84%	2.76%	
		T_e (s)	0.35	0.23	0.19	
	3	R_m first floor	0	2.58%	3.86%	3.86%
		R_m middle floor	0	2.58%	3.86%	3.86%
		R_m upper floor	0	2.58%	3.86%	2.58%
		T_e (s)	0.51	0.42	0.29	0.32
H03	2	R_m first floor	0	1.29%	3.22%	
		R_m upper floor	0	1.29%	3.22%	
		T_e (s)	0.39	0.28	0.19	
	3	R_m first floor	0	1.29%	1.96%	2.58%
		R_m middle floor	0	1.29%	1.93%	2.58%
		R_m upper floor	0	1.29%	1.93%	2.58%
		T_e (s)	0.57	0.42	0.35	0.32
H04	2	R_m first floor	0	1.15%	1.92%	
		R_m upper floor	0	1.15%	1.92%	
		T_e (s)	0.36	0.28	0.24	
	3	R_m first floor	0	1.92%	1.92%	
		R_m middle floor	0	1.15%	1.92%	
		R_m upper floor	0	1.15%	1.92%	
		T_e (s)	0.52	0.28	0.26	

4. NUMERICAL MODELS. NONLINEAR DYNAMIC RESPONSE ANALYSES

4.1. Numerical models

Numerical models were developed for each idealized prototype RC frame with the program IDARC version 6.1 (*R. E. Valles et all. 1996*). All the beams and columns were modelled as perfectly elastic beam elements with two nonlinear springs at the ends. From the dimensions and reinforcement of each RC section, the corresponding moment-curvature relationship was obtained by using the software Response-2000. The beam moment-curvature envelope was idealized with a tri-linear curve, and the hysteretic rule was calibrated to exhibit moderate stiffness degradation, moderate strength degradation and moderate slip or crack-closing behaviour. The parameters that define the stiffness degradation, the strength degradation and the slip or crack-closing behaviour in IDARC 6.1 (*R. E. Valles et all. 1996*)are HC, HBD and HS, and the corresponding values adopted were HC=10, HBD=0.30 and

HS=0.25, respectively. A detailed description of their meaning can be found elsewhere *(Sivaselvan M.V. et all 1999)*. For the columns, a tri-linear moment-curvature envelope was also used, which took into account the interaction between axial forces and bending moments. The hysteretic rules for the columns were calibrated in the same way as the beam elements. Infill panels were idealized as compression-only members. The hysteretic rule followed a modified Bouc-Wen *(J Song et all 2006)* model which takes into account the effect of stiffness degradation, lateral resistance degradation and pinching effect. The parameters that control the hysteretic model were calibrated with the experimental data obtained by some of the authors in laboratory tests conducted at Purdue University in 2008 *(Santiago Pujol et all; 2008)* Fig. 2 shows the comparison between experimental results and the envelope predicted with the numerical model of the RC structure referred in *(Santiago Pujol et all; 2008)*, without and with masonry infill walls. The following hypotheses were adopted: (i) the horizontal diaphragms are infinitely rigid in their own plane; (ii) the influence of infill panels located in a plane perpendicular to the direction of the ground motion are negligible; (iii) the bases of the columns of the first story are fixed; (iv) no torsion effects are considered (i.e. the centre of mass is assumed to coincide with the shear centre in each story).



(a) Structure without walls (b) Structure with walls

Figure 2: Experimental results and numerical simulation of RC structures without (a) and with (b) infill walls

4.2. Earthquake selection

Non-linear dynamic analyses were conducted using three well-known earthquake acceleration records recommended by the Japanese code *(BSL.The building standard law of Japan; 2009)* to evaluate seismic response of buildings: Hachinohe, El Centro and Taft. The peak ground acceleration (PGA) of the original records (without scaling) are summarized in Table 3.

Table 3. PGA of earthquakes used (without scaling)

Earthquake	PGA (cm/s^2)
El Centro	342
Hachinohe	225
Taft	153

The records were scaled as follows. First, the PGA established by the Global Seismic Hazard Assessment Program in rock for a return period of 500 years was determined, that gives for Haiti a PGA range of 1.6-2.4 m/s^2. For this study the safe-side upper-bound value of 2.4m/s^2 was adopted. Next, the PGA was modified to account the soil conditions and the importance of the building by using the formula proposed by the Spanish seismic code NCSE-02 (*Spanish Seismic Code (2002)*) Considering a school building as a construction of special importance and soil type IV, the PGA finally used for scaling the records was 0.5g (here g is the acceleration of gravity). Elastic response spectra for a damping factor of 5% were obtained for the selected earthquakes after scaling, in terms of absolute acceleration, and relative input energy E expressed as an equivalent velocity $V_E = \sqrt{2E/M}$ (*M* is the total mass of the structure). They are shown in Fig. 3.

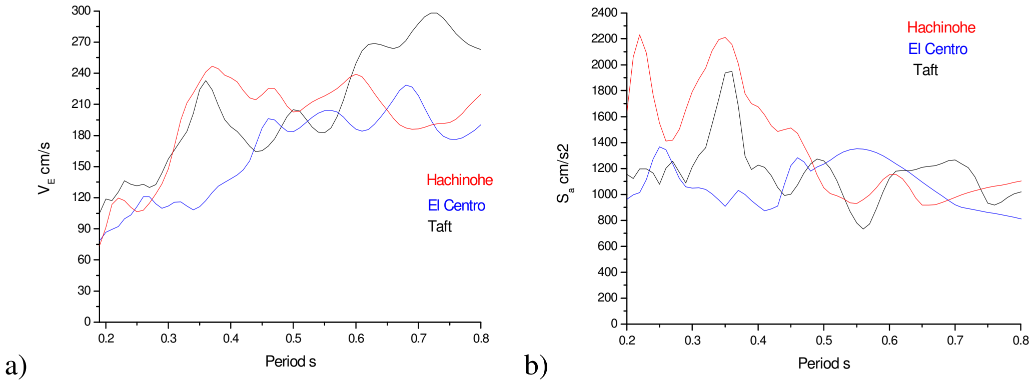


Figure 3. Elastic response spectra

4.3. Response parameters

To evaluate the response of the structure under earthquake loading and the level of damage after a seismic event, four parameters have been chosen: (i) interstory drift of each floor (id); (ii) Park&Ang damage index at the local story level, DI_{story}; (iii) global Park&Ang damage index $DI_{overall}$; and (iv) the hysteretic energy dissipated during the earthquake. The Park&Ang damage index for a structural element is defined by Eqn. 4.1

$$DI_{P\&A} = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y} \int dE_h \tag{4.1}$$

where, δ_m is the maximum deformation experienced, δ_u is the ultimate deformation capacity of the element, β is a constant control parameter usually taken as 0.1; P_y is yielding force of the element and $\int dE_h$ denotes the total (cumulative) hysteretic energy dissipated by the element (cumulative damage). The Park&Ang damage index for a story, DI_{story}, and for the overall structure, $DI_{overall}$, are estimated as shown in Eqn. 4.2 and Eqn. 4.3.

$$DI_{story} = \sum(\lambda_i)_{component}(DI_{component}); \qquad (\lambda_i)_{component} = \left(\frac{E_i}{\sum E_i}\right)_{component} \tag{4.2}$$

$$DI_{overall} = \sum(\lambda_i)_{story}(DI_{story}); \qquad (\lambda_i)_{story} = \left(\frac{E_i}{\sum E_i}\right)_{story} \tag{4.3}$$

Above Park & Ang indexes of damage have been calibrated so that 1 means collapse. The hysteretic energy dissipated by each story is calculated as the sum of the hysteretic energy dissipated by all the columns of the story, plus 50% of the hysteretic energy dissipated by the upper and lower beams. In order to investigate the feasibility of using masonry infill walls to reduce deformations and damage caused by strong ground motions in RC frame structures, the control parameter R_m defined by Eqn.3.1 was used. The infill panels were assumed to collapse when their lateral strength reduced below 50% of the maximum value. It is worth noting that the failure of the infill walls did not determine necessarily the global collapse of the structure. In some cases, the building continued sustaining the ground motion after the collapse of all the infill panels of the story.

5. RESULTS

Each configuration of the prototype frame with infill walls described in section 3 was subjected to the three records described in subsection 4.2 through nonlinear dynamic response analyses. A given configuration of a prototype structure with infill walls was considered "adequate" when: (i) the global Park & Ang damage index $DI_{overall}$ was $DI_{overall} \leq 1$ for at least two of the three ground motions applied;

or (ii) when for two of the three ground motions $DI_{overall}$≤1.2 and for the remaining earthquake $DI_{overall}$<1.0.

Weak column-strong beam failure mechanisms were observed in all building configurations studied. Even in cases where collapse was not reached, cracking took place at column ends. This behaviour was expected given the depth of the beams and the fact that these buildings were designed without paying attention to capacity design criteria. The columns exhibited flexural failure modes; in all cases strength of the columns was larger than the shear demand.

Fig. 4 shows the results of the dynamic response analyses. In the graphs, the infill panel ratio R_m is plotted against relevant parameters of the response: the inter-story drift, $DI_{overall}$, DI_{story} and the hysteretic energy W_h dissipated at beam and column ends (i.e. the energy dissipated by the masonry infill walls is not included). In these graphs, only the prototypes with infill wall configurations which response was considered "adequate" according to above criteria are represented. In each graph, a curve that provides an upper bound of the responses corresponding to a percentile of 85% is proposed. These curves must be considered as tentative, pending of the results of further numerical calculations and experimental results (shaking table tests) to be conducted in the future in this on-going research. From these curves, the required amount of infill panels in terms of R_m can be easily determined so that the structure endures the design earthquake considered (characterized in this study by PGA=0.5*g*), with a desired level of seismic performance characterized in terms of inter-story drift, $DI_{overall}$, DI_{story} or W_h.

Given the brittleness of the buildings in mind and considering the economic constraints, it seems reasonable to target a maximum lateral inter-story drift of 1.5% of the story height for PGA<0.5g. According to Fig. 4, the amount of masonry walls required to limit the lateral drift to 1.5% is approximately R_m=4%. This value R_m=4% yields a damage index at the story level, DI_{story} and for the whole structure, $ID_{overall}$, of 1. This value of R_m=4% should be taken as a minimum; larger values are advisable to reduce DI_{story} and $ID_{overall}$ below 1.

Finally, it is worth noting that after the strong earthquake occurred in Port-au Prince (Haiti, 2010) two guidelines *(MTPTC , "Guide Pratique de Réparations de Petits Bâtiments en Haiti" 2010; MTPTC, "Guide de Bonnes Pratiques pour la Construction de Petits Bâtiments"2010* , have been distributed to the population to provide recommendations on how to retrofit damaged buildings. Fig. 5 shows a table taken from these guidelines that shows the recommended area of the walls in plan in relation to the floor area. For two story buildings located in middle soil type the recommended percentage (4%) coincides with the value R_m=4% discussed above. It must be emphasized that this required amount of infill walls R_m=4% is in one direction. Similar amount of infill walls should be provided in two orthogonal directions of the building. Pending the accumulation of further results, in the light of this study the seismic retrofitting solution consisting on installing masonry infill walls should be limited to buildings up to 3-stories.

6. CONCLUSIONS

This work investigated a potential retrofitting alternative for RC frame structures that have been damaged by severe earthquakes. It consists in adding masonry infill panels (preferably with reinforcement). The main advantages of using masonry infill panels instead of other solutions such as dampers or RC walls are the ease of construction, the low cost, and the minimum technology involved. This solution is especially suitable for developing countries

Several prototypes of idealized RC frame structures with 2-3 stories representing school buildings damaged during the strong earthquake that occurred in Port-au-Prince (Haiti, 2010) were modelled and hypothetically strengthened with different configurations of masonry infill walls. Each configuration was characterized by the ratio of the area of the infill panels in the direction of the ground motion to the floor area, and expressed by a ratio R_m. Non-linear dynamic response analyses were conducted to study their response under three well-known ground motions (El Centro, Hachinohe and Taft) scaled to a PGA of 0.5*g*. According the numerical analyses, the amount of masonry walls required to limit the lateral drift to 1.5% is R_m=4% in one direction. A similar amount of infill walls should be provided in the perpendicular direction. This value is close to that recommended by the Haitian government in recent guidelines for a two story building, Table 4. Pending the vetting against experimental evidence, the seismic retrofitting solution consisting on installing masonry infill walls should be limited to buildings up to 3-stories.

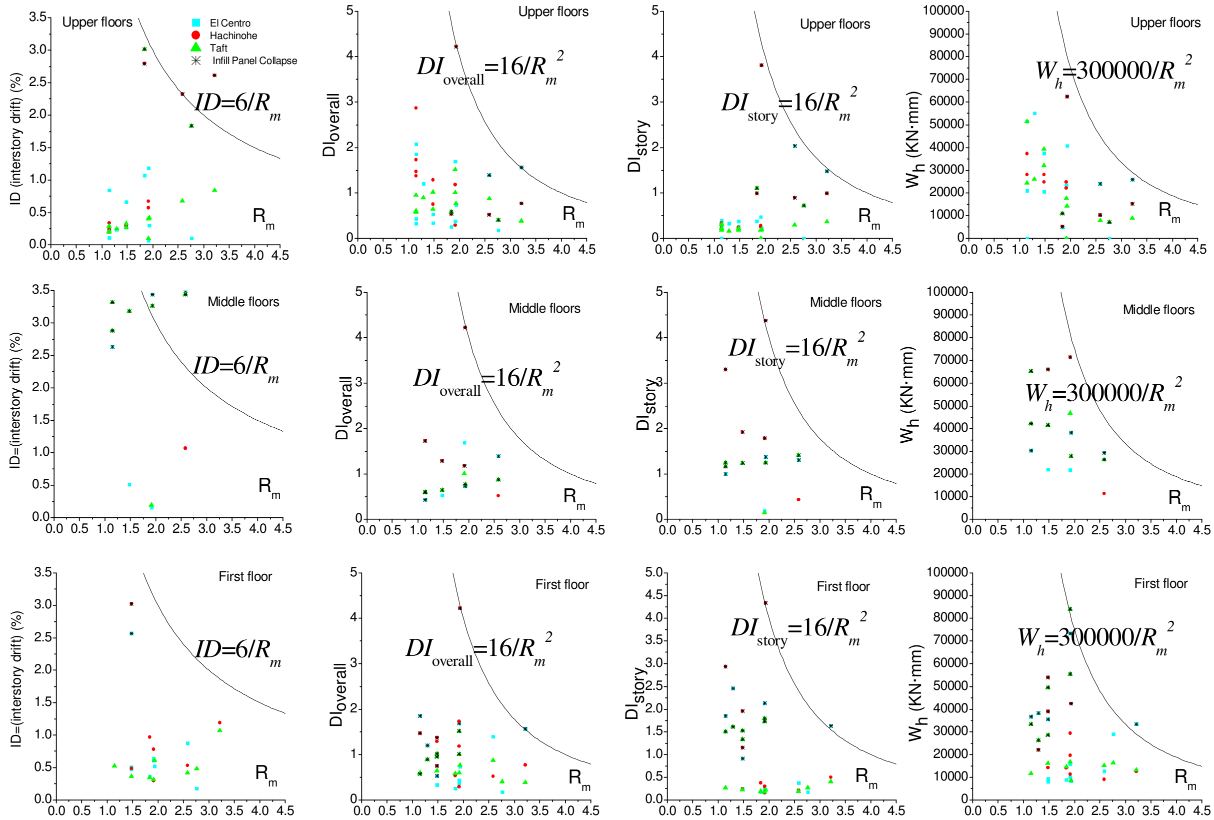


Figure 4. Results of the non-linear response analyses

Table 4. Recent recommendations of Haiti government

Type du Sol	Description	Surface minimale au sol	
		1 niveau	2 niveaux
Dur	Roc Gravier	1.5%	3.0%
Intermédiaire	Sable compacté Argile dure	2.0%	4.0%
Mou ou non compacté	Sable lâche Argile molle	2.5%	5.0%

AKCNOWLEDGEMENT

This study was supported by the Spanish Ministry of Science and Innovation, Projects BIA2008/00050 and BIA2011-26816. It received funding from *Fonds Européen de Dévelopment Regional* (FEDER).

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