SEISMIC SAFETY EVALUATION OF MASONRY DWELLINGS THROUGH FRAGILITY FUNCTIONS

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ABSTRACT

The present document explains the evaluation of the seismic safety of an internally reinforced masonry dwelling through the probabilistic approach of Fragility Functions. A very commonly structural plan for a two-story family dwelling was chosen as the target structure. For modeling the non-linear properties of the structure, a comparison was made between the Takeda Slip model and the Takeda Standard model to experimental results of testing of masonry walls. For predicting the skeleton curve of the masonry walls of the structure two approaches were taken. First, a set of empirical equations were proposed for predicting the skeleton curve of masonry walls. The second approach was calculating the elastic stiffness following Tomaževič (1999), Kikuchi and Kuroki (2017) approach for the rest of the stiffness parameters and Matsumura (1988) for the cracking and ultimate strength of the walls were also used. The fragility curves obtained by those methods were compared and finally the most appropriate was chosen to evaluate the seismic safety of the target structure.

Keywords: Equivalent Viscous Damping, Fragility Functions, Reinforced masonry, Takeda Model, Takeda-Slip Model.

1. INTRODUCTION

Since 1965, at least 4 major events have occurred that have caused considerable damage to the capital city or other highly populated urban areas. An evaluation of whether the current design norms for seismic demand are adequate can be done through the use of the fragility functions. The main objective of this study is to evaluate the expected damage for masonry structures with the seismic demand regulated by the current seismic code and evaluate its validity. The secondary objective is to define a more appropriate and efficient procedure for calculating a fragility curve for a given structure.

2. FRAGILITY FUNCTIONS

Flowchart for the procedure for calculating fragility curves can be seen in Figure 1. The method is based on Hwang (1988). Three main steps were followed. The first is the selection of the target building, for which a non-linear behavior should be assumed. The most appropriate hysteretic model should be selected. The second step is the selection of a set of ground motions. The ground motions

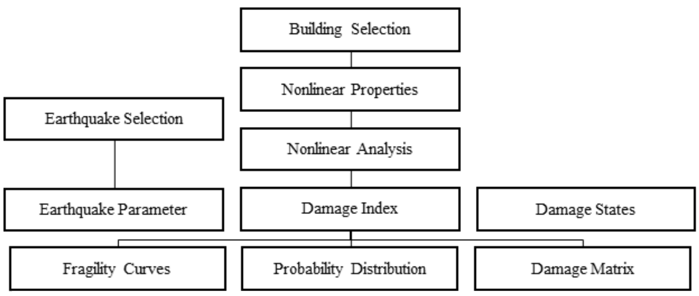

Figure 1. Fragility Functions Analysis Procedure, Based on Blandon (2001).

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should be representative of the geology of the area and its predominant frequency should affect the target structure’s frequency. This can be done by comparing the response spectra. Finally, the third step is classification of the expected damage, for which parameters such as inter-story drift and ductility ratios can be used. Figure 1 shows the described procedure.

3. BUILDING SELECTION

The target structure is shown in Figure 2. The structure corresponds to a typical single family dwelling in El Salvador. The X-direction was selected for analysis; given that it shows a density of walls per floor area lower than in the perpendicular direction. The flexural reinforcement of the walls was D10 every 60 cm. Grout was placed only in the cells with rebars and thus an equivalent thickness was used. According to Burgos, et al. (2015), for masonry walls with a nominal thickness of 15 cm grouted every 60 cm, the equivalent thickness is 10.41 cm.

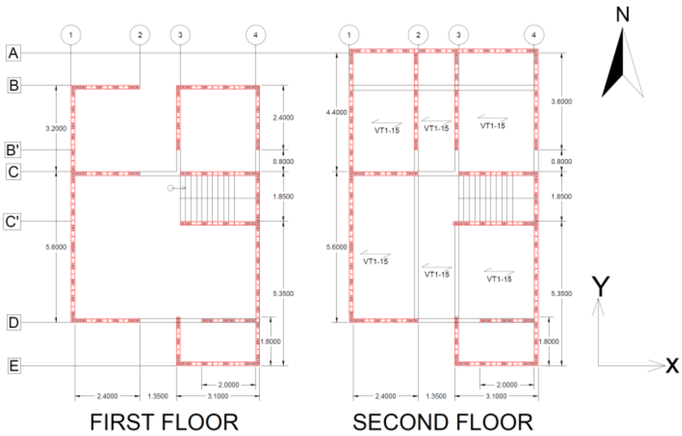
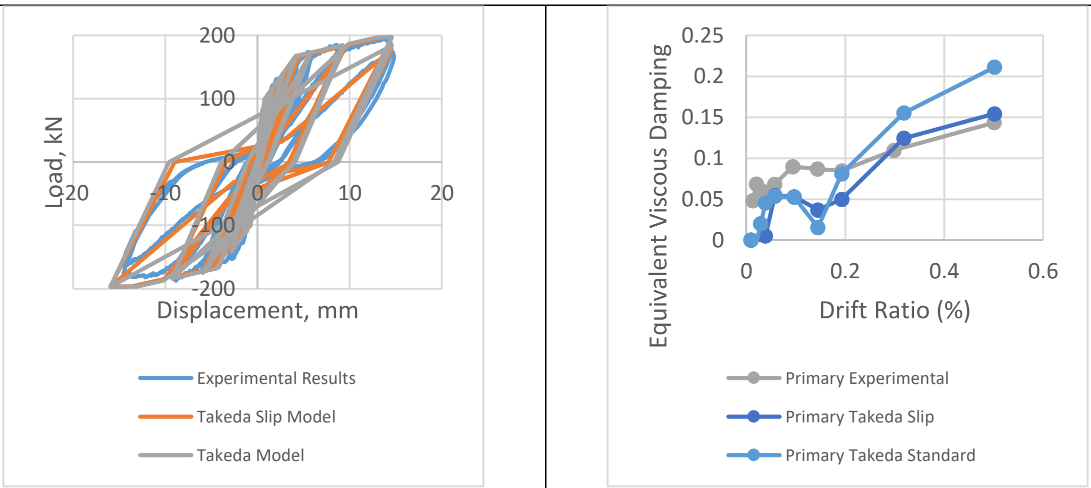


Figure 2. Target structure for the present study.

4. SELECTION OF NON-LINEAR PROPERTIES

4.1. Non-linear behavior modelling of Masonry Walls

A comparison between the Takeda Model and the Takeda Slip Model was made in order to determine the most suitable hysteretic model for masonry reinforced walls. The models were compared with the cyclic loading test conducted by Ramos (2014) and a good agreement was observed for both models.



(a) Comparison of experimental results with selected hysteresis models for Wall A3.

(b) Equivalent viscous damping for primary loading cycles for Wall A3.

Figure 3. (a): Comparison of hysteretic models with test results. (b): Comparison of calculated equivalent viscous damping.

Figure 3 shows a detailed comparison between the experimental results for the A3 wall and the results for the modeling using the Takeda Slip and Takeda Standard models. The equivalent viscous damping was calculated for all the test specimens, comparing the experimental results, the response with the Takeda Standard Model and the response with the Takeda Slip Model. This comparison shows that the Takeda Slip Model fits better with the experimental results than the Takeda Standard Model, even though both models tended to overestimate the equivalent viscous damping. For this reason, the response obtained with the model that considers slip was taken to be more reliable.

4.2. Determination of parameters for the backbone curve for masonry walls

For the determination of the characteristics of the primary curve for masonry walls, Eq. (1) through Eq. (6) were proposed, deduced with the data from the experimental study of the rehabilitation method discussed above. The envelope curve was calculated and then idealized as a trilinear curve. The elastic stiffness, post-cracking stiffness and post-yielding stiffness were calculated for each data set. The cracking, yielding and maximum strength were also calculated. After this, the values were tabulated and empirical correlations between those parameters and two variables were determined. The chosen variables were the inverse of the aspect ratio, l/h, and the ratio of steel reinforcement per gross area of wall. These equations are presented in Table 1, as below.

Table 1. Empirical Equations prepared for this study.

Parameter	Equation	Intensity of Correlation, R^2	Equation Number
First Stiffness, K0	$K_0 = 168.3434\left(\frac{l}{h}\right) - 482.5237\left(\frac{A_s}{A_G}\right)$	0.9837	(1)
Second Stiffness, K1	$K_1 = 23.9699\left(\frac{l}{h}\right) - 55.6689\left(\frac{A_s}{A_G}\right)$	0.9741	(2)
Third Stiffness, K2	$K_2 = 2.9506\left(\frac{l}{h}\right) - 4.2485\left(\frac{A_s}{A_G}\right)$	0.9792	(3)
Cracking Load Strength Estimation	$V_{CR} = 76.0501\left(\frac{l}{h}\right) - 145.2500\left(\frac{A_s}{A_G}\right)$	0.9871	(4)
Yielding Load Strength Estimation	$V_Y = 142.5261\left(\frac{l}{h}\right) - 250.2070\left(\frac{A_s}{A_G}\right)$	0.9952	(5)
Ultimate Load Strength Estimation	$V_U = 168.5398\left(\frac{l}{h}\right) - 254.4770\left(\frac{A_s}{A_G}\right)$	0.9949	(6)

The lateral elastic stiffness of masonry walls, were both flexural and shear deformations are important was calculated according to Tomaževič (1999). The rest of the necessary parameters for calculating the stiffness were Kuroki, M, Kikuchi, K et al. (2017). Ultimate and shear strength of masonry walls were calculated following Matsumura (1988). A comparison of the skeleton curves with the proposed empirical equations and the aforementioned existing equations was later made.

5. EARTHQUAKE SELECTION

A set of ground motions should be selected for conducting the non-linear dynamic analysis of the target structure. 12 acceleration records were selected based on their high amplification in the short period range, meaning that the target structure was easily affected by such ground movement. Then, an Incremental Dynamic Analysis (IDA) was conducted. According to Vamvatsikos & Cornell (2002), IDA is a helpful tool used for evaluating structural performance under dynamic loads or, more specifically, seismic loads. The procedure involves building one or various structural models and applying a set of ground motions, scaled to different levels of intensity. The selected acceleration records were both horizontal components of the January 2001 and February 2001 El Salvador Earthquake, the October 10th, 1986 San Salvador Earthquake, 1995 Kobe earthquake and 1940 El Centro Earthquake.

Also, two artificial records were generated using the phases of both El Salvador 2001 earthquakes, and using a target velocity response spectrum given by the Design Velocity Response of the El Salvador Seismic Design Code.

6. DAMAGE CLASSIFICATION

When comparing the proposed damage limit states with the experimental results of a test conducted in El Salvador, some correspondence can be found. In the study conducted by Ramos (2014), walls are subject to a target drift of 1/200 or 0.5%. Up to this point, the evaluated rehabilitation method is effective. This value is very close to the target drift proposed by FEMA 273 for the S-3 Life Safety Limit State. In the same study, important shear cracking begins at a drift value of 0.1%. Before reaching this value, no significant damage is observed, therefore this adjusted value is proposed for the S-1 Limit State. On the other hand, the ultimate capacity obtained in the TAISHIN project for the concrete block construction system experiments is attained at a drift of at least 1.5, which is the minimum value for the S-5 Collapse Prevention Limit State. The comparison of the values proposed by FEMA 273 and those in this present study can be seen clearly in Table 2.

Table 2. Damage classification for reinforced masonry buildings according to FEMA 273. Target drift is used as classification of damage.

Type	**Collapse Prevention S-5**	**Life Safety S-3**	**Immediate occupancy S-1**
Drift	1.5% transient or permanent	0.6% transient; 0.6% permanent	0.2% transient; 0.2% permanent
Average Experimental Value	1.6% (TAISHIN PROJECT)	0.5% (Rehabilitation Method Study, UCA 2014)	0.1% appearance of important shear cracking in the walls (Rehabilitation Method Study, UCA 2014)
Proposed Value	1.5%	0.55%	0.1%

7. ANALYSIS AND RESULTS

Figure 4a, 4c and 4e shows the overlapped Fragility Functions calculated with the Takeda Standard Model and the Takeda Slip Model. As expected, the functions obtained with the Takeda Slip Model show more brittle behavior than those obtained with the Standard Takeda Model. This was due to the amount of energy dissipated by each model. The loops on the standard model were wider than those in the slip model, which is a typical characteristic of a ductile system. Nevertheless, the responses were very similar. The maximum response of the Standard Model was approximately 15% lower when compared with the Slip Model for the earthquakes selected. The level of PGA at which each limit state started to occur was very similar. Based on all these results it could be concluded that both methods were valid when modelling the non-linear behavior of masonry walls. Nevertheless, the Takeda Slip Model is on the safer side, since the equivalent viscous damping values obtained were closer to the experimental results, and a probability equal to 1 was achieved faster than with the Takeda Standard Model, meaning that the Takeda Standard Model overestimated the capacity of the target structure by a small margin. A comparison of the two sets of fragility functions for the Empirical Equations compared to the Existing Equations is shown in Figure 4b, 5d and 5f. Functions obtained with the standard equations showed a more ductile system, where the same level of damage occurred in a wide range of PGA levels. This corresponded to a more ductile system, which did not seem to reflect the brittle behavior of masonry walls.

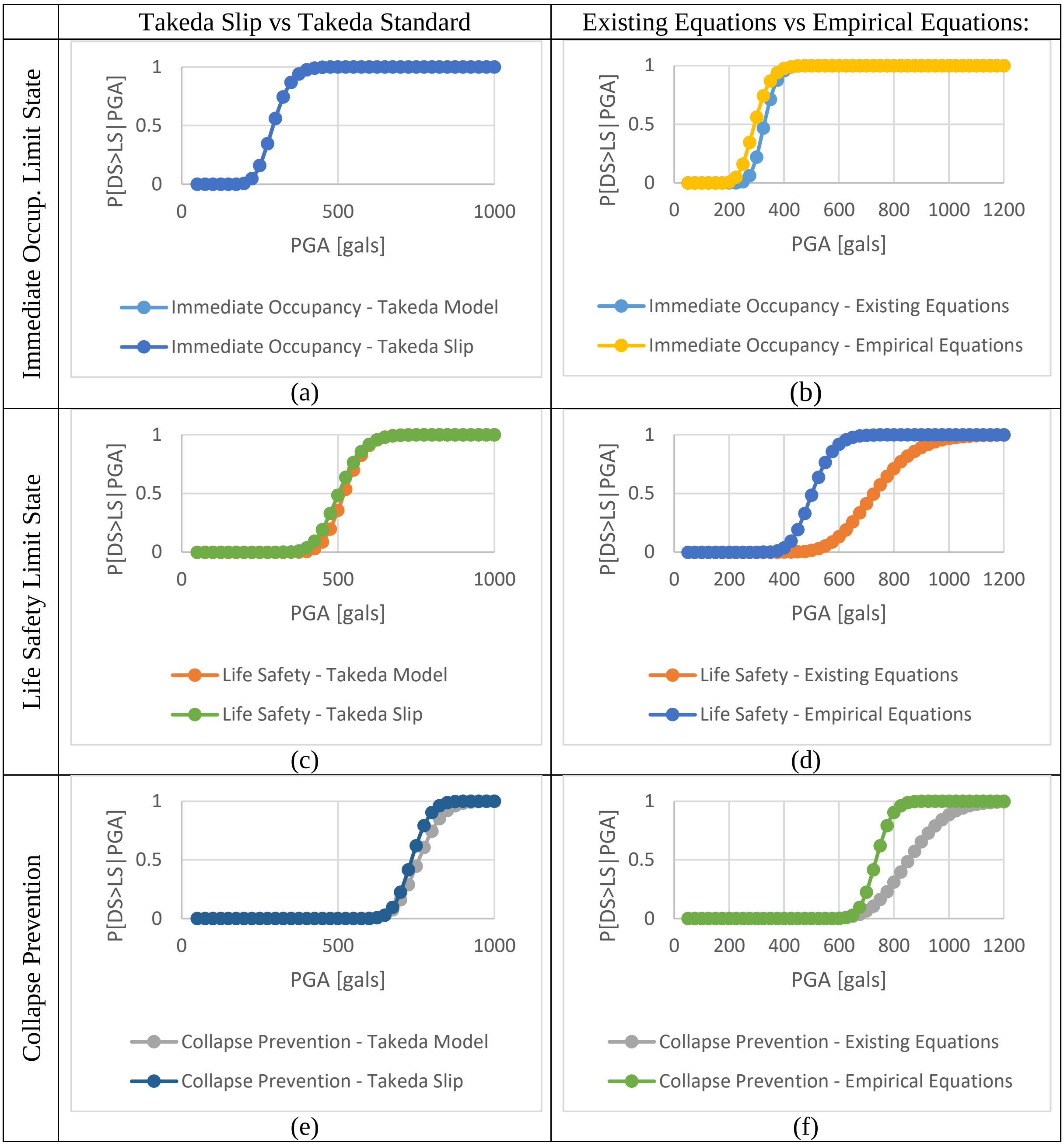


Figure 4. Comparison of Fragility Functions.

7.1. Evaluation of seismic safety of the masonry dwelling

Fragility Functions calculated with the Takeda Slip Model and empirical equations for the backbone curve were selected. Since the target building is in the capital city, San Salvador, the PGA given by the Seismic Design Code is 0.4g. The maximum recorded acceleration in El Salvador is the October 10, 1986 San Salvador Earthquake, of 524.5 Gal. As can be seen in Figure 5, for the acceleration level corresponding to the Seismic Design Code of El Salvador, the probability of surpassing the Immediate Occupancy Limit State is very close to 1.0. Nevertheless, the expected level of damage should be very light, since the probability of exceeding the Life Safety Limit State is very low, being approximately

0.03. On the other hand, for the maximum recorded acceleration level, the probability of exceeding the Life Safety Limit state is 0.64. Even if this is a very high probability, collapse of the structure should not be expected.

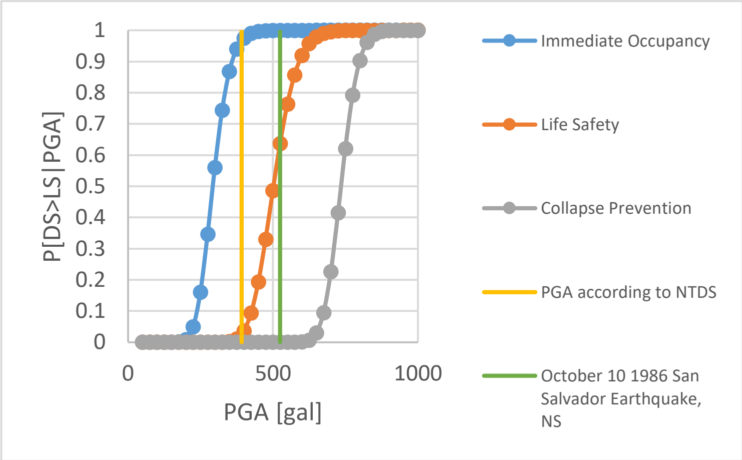


Figure 5. Fragility Function (Takeda Slip and Empirical Equations) overlapped with the two levels of expected acceleration.

8. CONCLUSIONS

Both the Takeda Slip Model and the Takeda Standard Model are appropriate to estimating the non-linear response of masonry wall structures. Nevertheless, the Takeda Slip has a more coherent behavior with experimental results. The slip model also produces better curves in the sense that they are on the safer side of the spectrum because they're more brittle.

The proposed equations seemed to reflect the brittle behavior of masonry wall structures in a more reliable way than the already existing equations. This could be further modified to include other parameters such as axial load level and horizontal reinforcement.

For the maximum recorded acceleration level in El Salvador, the target structure was safe enough not to collapse. Substantial damage should be expected, but the damage should be repairable. It is recommended that further retrofitting and rehabilitation techniques should be studied.

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REFERENCES

Blandon Uribe Carlos Andrés Individual Studies by Participants at the International Institue of Seismology and Earthquake Engineering. - 2001. - pp. 179-191.
FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA) FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings [Report]. - Washington, D.C. : FEMA, 1997.
Jaw Jin-Jen and Hwang Howard H.M. Seismic Fragility Analysis of Shear Wall Structures [Report]. - Memphis : Center for Earthquake Research and Information, 1988.
Kuroki Masayuki [et al.] [Article] // Concrete Engineering Annual Paper. - 2017. - pp. Vol 39, N°2.
Matsumura Akira Shear Strength of Reinforced Masonry Walls [Conference] // Ninth World Conference on Earthquake Engineering . - Tokyo : [s.n.], 1988. - pp. 121-126, Vol. VI.
Ramos Adolfo, Publish Pending [Report]. - Antiguo Cuscatlán : Universidad Centroamericana "José Simeón Cañas", 2014.
Shibata Akenori [Book]. - Sendai : Tohoku University Press, 2010.
Tomaževič Miha - London : Imperial College Press, 1999.
Vamvatsikos Dimitrios and Cornell C. Allin - 2002. - pp. 491-514.