Structural Analysis of Historic Construction – D'Ayala & Fodde (eds)
© 2008 Taylor & Francis Group, London, ISBN 978-0-415-46872-5

Vulnerability assessment of churches at Colima by 3D limit analysis models

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ABSTRACT: This work presents the vulnerability assessment of two churches at the state of Colima by means of 3D rigid block models, using a limit analysis approach. The paper contains a description of the buildings and the damage they suffered during the 2003 earthquake. The 3D models were elaborated on the basis of macro-blocks defined according to the damage observed after the earthquake and also, taking into account the typical failure mechanisms in churches reported in the literature. The limit analyses were carried out with a software already tested and validated elsewhere. The results show that the limit analysis approach with rigid block models is a valuable tool for the identification of critical failure mechanisms, their associated failure multipliers and, in a global fashion, for the vulnerability assessment of historical churches. The results also indicate that empirical methods provide a good estimation of the vulnerability level of this buildings type.

1 INTRODUCTION

Seismic vulnerability assessment of ancient masonry buildings is an issue of most importance at present time. The main difficulty is the non-linear behaviour of masonry since very low load levels due to its poor tensile strength. This behaviour, combined with the dynamic character of the seismic action represents a major challenge.

This non-linear behaviour prevents the use of linear-elastic based procedures for other than frequent, low intensity events. Nevertheless, Bani-Hani & Barakat (2006) and Mistler (2006) have recently used modal analyses for the seismic assessment of ancient masonry buildings. The current trends in Earthquake Engineering have the objective to take into account the dynamic non-linear behaviour of structures in a more rational and realistic way and make use of capacity curves obtained by pushover analyses. Penelis (2006) proposed a simplified procedure for pushover analysis of masonry shear walls with openings. A more general procedure consists of performing non-linear finite element analyses of continuous models, generally using a smeared crack approach (Calderini & Lagomarsino 2006, Lourenço et al. 2007). Nevertheless, these approaches present convergence difficulties due to the negative tangent stiffness at the softening branches of the stress-strain relations. Recent developments related to avoid these convergence difficulties assuring positive-definite stiffness matrices in approximated analysis procedures (Rots & Invernizzi 2004 and Oliver et al. 2006). The application of these approaches to pushover analysis of ancient masonry structures is under development (Meza 2007).

Vulnerability is a concept widely used in recent Earthquake Engineering works. Nevertheless, there is not a rigorous and widely accepted definition of it. In general terms, vulnerability measures the amount of damage produced by an earthquake of given intensity over a structure. However, "amount of damage" and "seismic intensity" are concepts without a clear and rigorous numerical definition. The European Macroseismic Scale (EMS 1998) represents a very significant effort to relate damage over a series of construction types with seismic intensities. Here, damage and intensity have different levels assigned in qualitative terms.

The vulnerability curve is a plot relating the seismic intensity with a measure of damage (Shibata et al. 2000). Nevertheless, the construction of the vulnerability curve for a given structure requires a highly sophisticated procedure of non-linear analysis on a numerical model.

A simplified way to assess the vulnerability of a building consists of estimating the strength of a numerical model against horizontal loads. In this paper, the limit analysis procedure proposed by Orduña & Lourenço (2005a, b) is used to assess the vulnerability of two churches at the Colima State in Mexico.

Colima State is located in the highest seismic area in Mexico, the Pacific Coast (CFE 1993). On January 21, 2003, an M7.6 earthquake demonstrated, again,

that historical buildings are one of the most vulnerable structural types. This event damaged a high proportion of the historical buildings in the Colima State, most of them churches. The vulnerability of these buildings both, before and after restoration is unknown. Therefore, a research project with the main objective to assess the vulnerability of the historical buildings in Colima State was carried out. In a first phase of this project, the historical buildings' vulnerabilities were assessed through empirical methods. The results indicate that most buildings have a medium vulnerability, and about 15% are highly vulnerable (Preciado 2007). In a subsequent phase of the project, selected highly vulnerable buildings were assessed by means of more accurate, analytical methods such as FEM and limit analysis.

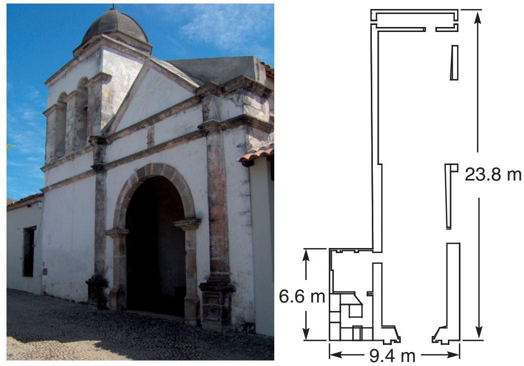
This paper presents the general procedure used to assess the seismic vulnerability of historical buildings by means of limit analysis. The results obtained for two selected churches are presented, and discussed and, finally, the conclusions are stated.

2 LIMIT ANALYSIS OF 3D RIGID BLOCKS MODELS

A number of authors have demonstrated that limit analysis of models elaborated with rigid blocks is a valuable tool for an approximate structural assessment of ancient masonry structures (Orduña & Lourenço 2005a, b, Giordano et al. 2007). Equation 1 expresses the equilibrium between the forces at the interfaces, $\mathbf{Q}$, and the external loads applied to the blocks: $\mathbf{F_c}$ and $\alpha\mathbf{F_v}$. Where $\mathbf{F_c}$ are the permanent loads, $\mathbf{F_v}$ are the variable loads, α is the load factor, and $\mathbf{B}$ is the equilibrium matrix. Limit analysis provides the load factor producing collapse on the model and the corresponding failure mechanism, through solving a mathematical programming problem that includes Equation 1, the yield conditions, the flow rule, compatibility and complementarity equations (Orduña & Lourenço 2005a, b).

$$\mathbf{F_c} + \alpha\mathbf{F_v} = \mathbf{BQ} \quad (1)$$

In the analyses presented here, the permanent loads, $\mathbf{F_c}$, are the self weight of the blocks and the overload produced by the roof system, represented by additional blocks with the appropriate volumetric weight. The variable loads, $\mathbf{F_v}$, are numerically equal to the permanent ones but applied horizontally, thereby representing the body loads that produce the inertia during an earthquake. In this way, the load factor, α, can be viewed as a seismic coefficient, and represents the amount of the weight applied as horizontal load. The three-dimensional models of the buildings were elaborated on the basis of macro-blocks defined according



(a) (b)

Figure 1. "Virgen del Refugio" Chapel (a) façade; and (b) plan view.

to the damage observed and also, taking into account the typical reported failure mechanisms for churches in literature.

3 RESULTS

Two churches, that were identified as highly vulnerable in the first phase of this project by empirical methodologies, were assessed again by means of the described limit analysis approach, in order to obtain more accurate results. In order to perform a correct assessment of the seismic vulnerability of these buildings, a survey of information related to geometrical and mechanical characteristics of the constructions was essential. This data corresponds to plans, construction materials characteristics, a historical and structural description, previous interventions information and conservation state.

3.1 *Seismic vulnerability assessment of the "Virgen del Refugio" Chapel*

The "Virgen del Refugio" Chapel (Fig. 1), built between 1890 and 1898, consists of two principal parts: the nave and the bell tower. The nave has stone and clay brick walls, while the bell tower material is only stone masonry. Wooden trusses support a roof made of clay slates. The survey after the January 21, 2003 earthquake showed heavy structural damage, such as fairly large pieces of fallen plaster and horizontal and vertical cracks in the façade, walls and bell tower.

In order to study the level of coupling that the roof system introduce to the lateral response of the longitudinal walls, two cases were considered for the blocks representing the roof system (Fig. 2). The first case "Parallel loads" (case 1) considers the vertical reactions of the roof system, uniformly applied directly

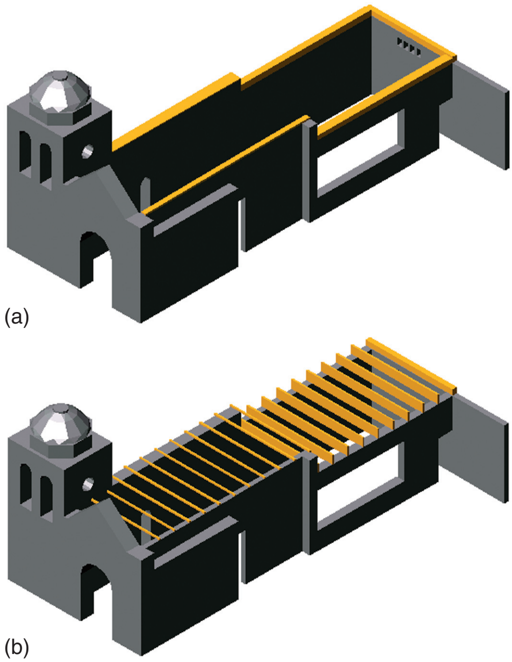


Figure 2. 3D model of the "Virgen del Refugio" Chapel (a) Case 1: parallel loads; (b) Case 2: perpendicular loads.

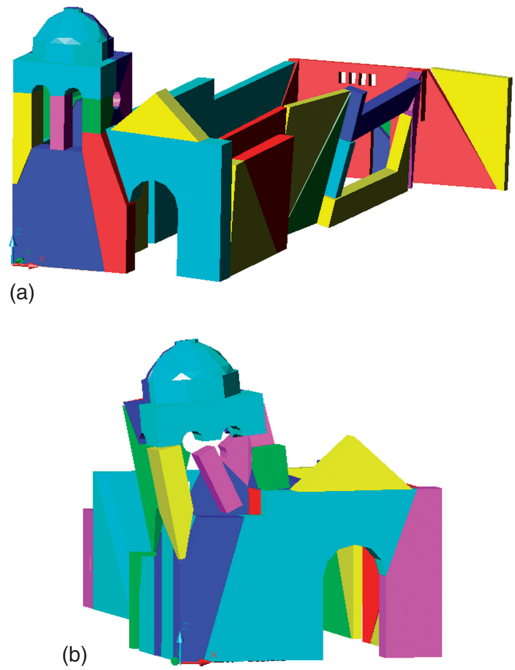


Figure 3. Case 1 failure mechanisms for seismic action on (a) +X direction; and (b) −X direction.

over the walls without coupling (Fig. 2a). While in the second case, "Perpendicular loads" (case 2) the model includes rigid blocks representing both, the weight and the coupling action of the wooden trusses, as shown in Figure 2b.

The models for the two cases were analysed for seismic actions in the +X, −X, +Y and −Y directions. Figure 3 presents the model failure mechanisms for horizontal loads on the −X and +X directions, for Case 1. The main failure mechanism of the model for loads on the +X direction, shown in Figure 3a, is the overturning of the lateral wall with a collapse load factor of 0.37. The results for the model subject to loads in −X direction (Fig. 3b) show a failure mechanism that represents the imminent collapse of the bell tower with a collapse load factor of 0.38.

Figure 4 shows the failure mechanisms for seismic actions on the −Y and +Y directions. The failure mechanism of the model for loads in +Y direction, illustrated in Figure 4a, is the overturning of the back façade with a collapse load factor of 0.19. The failure mechanism for the model subjected to earthquake in −Y direction (Fig. 4b) shows the overturning of the main façade under a load factor of 0.18.

Figure 5 shows the failure mechanisms for the Case 2 models with horizontal loads in +X and −X directions. The main failure mechanism of the model for +X loading (Fig. 5a) is the overturning of the

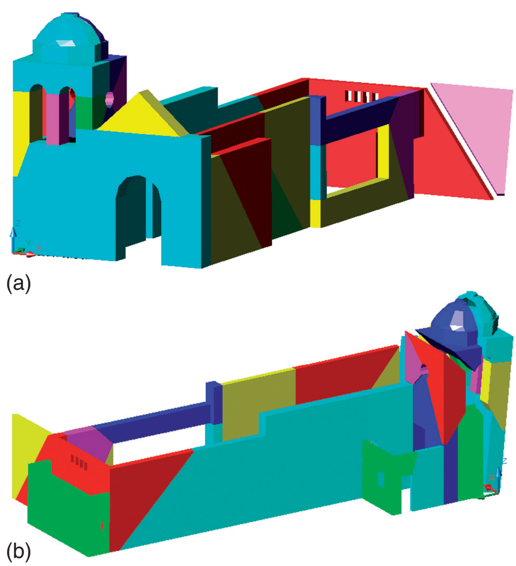


Figure 4. Case 1 failure mechanisms for seismic actions on (a) +Y direction; and (b) −Y direction.

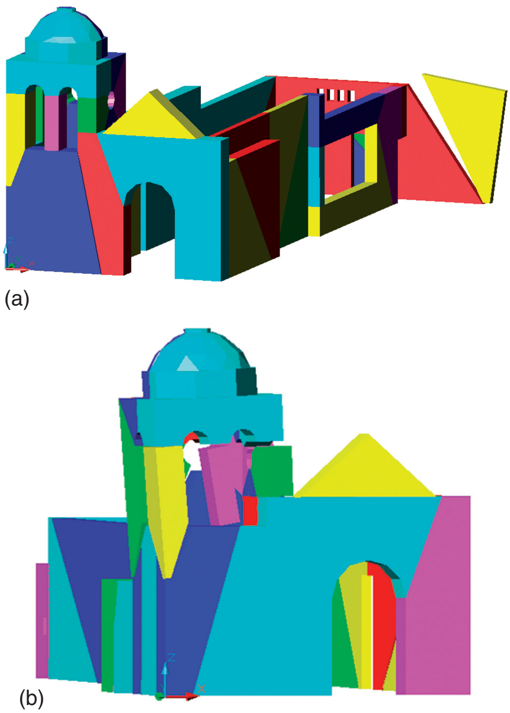


Figure 5. Case 2 failure mechanisms for seismic actions on (a) +X direction; and (b) −X direction.

back façade with a collapse load factor of 0.40. The results for the model subject to earthquake in −X direction (Fig. 5b) presents collapse of the bell tower under a load factor of 0.38. For this Case 2, the failure mechanisms for seismic actions on the −Y and +Y directions exactly the same than in the Case 1 (Fig. 4).

Analysing the results for both cases the limit analysis for Case 2 appeared to show more accurate results, because they represent the same failure mechanisms actually exhibited by the church after the 2003 earthquake.

3.2 *Seismic vulnerability assessment of the Cathedral "Basilica Menor de Guadalupe"*

The cathedral (Fig. 6) was built in 1889 and is considered the most important historical building in Colima State. This building is larger and more architectonically sophisticated than the chapel *"Virgen del Refugio"*.

The vertical structure of the construction includes a stone masonry foundation, thick masonry walls made of solid clay bricks, and two bell towers made of stone masonry at bottom and clay brick masonry in the upper part. The roof system consists of masonry vaults, and a clay brick masonry cupola, both supported on arches of the same material. The masonry of the vaults is an interesting case because the pieces are small pots of fired clay. May be the original idea was to achieve a light weight material, compared to solid pieces masonry. The survey after the 2003 earthquake showed extensive cracks in most walls, fairly large pieces of fallen plaster and towers fractured. In the roof system were large and extensive cracks in most vaults and large pieces of fallen plaster.

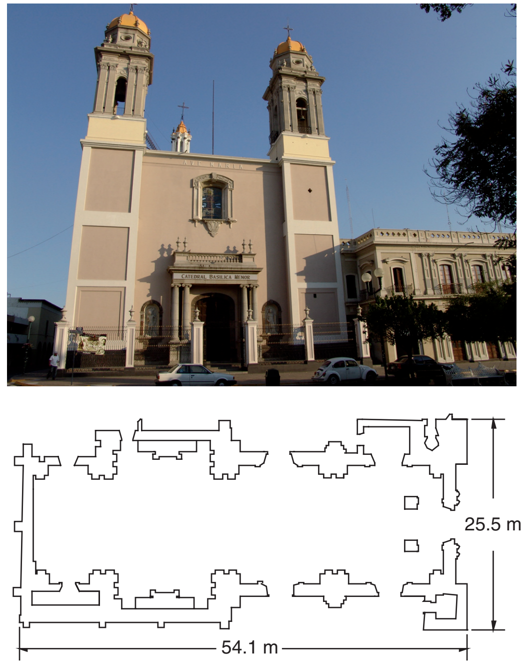


Figure 6. Cathedral "Basilica Menor de Guadalupe" (a) façade; (b) plan view.

A three-dimensional model of the whole building (Fig. 7) was developed taking into account the same considerations that in the first church. Due to the size and sophistication of the Cathedral, the macroblocks were considered in order to analyse only a seismic action on the −X direction by the limit analysis approach. This criterion was assumed after the observed damage post 2003, the historical analysis, and the first evaluation of the building by empirical methods. Besides, this information permitted to identify the part of the building most vulnerable to a seismic action. This was the main façade and the bell towers (Fig. 8).

The results for the model subject to earthquake in −X direction (Fig. 8) represents the imminent collapse of the north bell tower with a collapse load factor of 0.12. The results obtained in the limit analysis of

Figure 7. Full model of the cathedral.

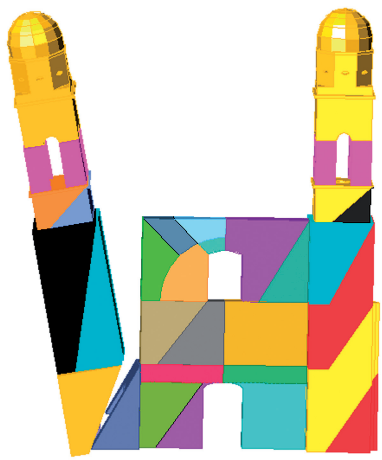


Figure 8. Main façade and bell towers failure mechanism for a seismic action on the −X direction, load factor: 0.12.

this part of the building are consistent with the damage observed. The historical analysis showed that the same bell tower collapsed in a 1941 earthquake with similar magnitude than the 2003 one.

4 CONCLUSIONS

The seismic vulnerabilities of two buildings of different sizes and sophistication were assessed by means of three-dimensional rigid block models, using a limit analysis approach. These analyses allowed knowing the failure mechanisms of every building and the corresponding load factor. As already mentioned, this factor can be interpreted as a seismic coefficient. Meli (1998) suggests, as a reference, that for ancient masonry buildings located in high risk seismic areas, this coefficient should be between 0.1 and 0.3. CFE (1993) affirms that for modern buildings located in high risk seismic areas, such as Colima, the seismic coefficient depends of the type of soil, fundamental period of vibration of the structure, ductility and damping. For structures located in Colima, the seismic coefficient should be between 0.50 and 0.86. Taking into account these references, and the results obtained by limit analysis, the "Virgen del Refugio" chapel has a medium seismic vulnerability, while the Cathedral has a high seismic vulnerability. The results showed that the limit analysis approach with rigid block models is a valuable tool for the identification of critical failure mechanisms, their associated failure multipliers and, in a global fashion, for the seismic vulnerability assessment of historical churches. The results also indicate that the empirical methods, observed damage and historical analysis provide a good estimation of the vulnerability level of this buildings type.

REFERENCES

Bani-Hani, K. & Barakat, S. 2006. Analytical evaluation of repair and strengthening measures of Qasr al-Bint historical monument-Petra, Jordan. *Engineering Structures* 28: 1355–1366

Calderini, C. &Lagomarsino, S. 2006. A micromechanical inelastic model for historical masonry. *Journal of Earthquake Engineering*, 10(4): 453–479.

CFE 1993. *Seismic Design Manual* (in Spanish). Mexico City: Federal Commission of Electricity.

EMS 1998. *European Macroseismic Scale 1998*. G. Grünthal (ed.) Conseil de L'Europe, Cahiers du centre Européen de Géodynamique et de Séismologie.

Gilbert, M. Casapulla, C. & Ahmed, H. M. 2006. Limit analysis of masonry block structures with non-associative frictional joints using linear programming. *Computers & Structures* 84: 873–887.

Giordano, A. De Luca, A. Mele, E. & Romano A. 2007. A simple formula for predicting the horizontal capacity of masonry portal frames. *Engineering Structures* 29: 2109–2123.

Lourenço, P. B. Krakowiak, K. J. Fernandes, F. M. & Ramos, L. F. 2007. Failure analysis of Manastery of Jeronimos, Lisbon: How to learn from sophisticated numerical models. *Engineering Failure Analysis* 14: 280–300.

Meli, R. 1998. *Structural Engineering of Historical Buildings* (in Spanish). México City: ICA Foundation.

Meza, J. M. 2007. *Method for the seismic performance assessment of historical masonry buildings* (in Spanish). Mexico City: National Autonomous University of Mexico, master thesis.

Mistler, M. Butenweg, K. & Meskouris, K. 2006. Modelling methods of historic masonry buildings under seismic excitation. *Journal of Seismology* 10:497–510.

Oliver, J. Huespe, A. E. Blanco S. & Linero, D. L. 2006. Stability and robustness issues in numerical modeling of material failure with the strong discontinuity approach. *Computer Methods in Applied Mechanics and Engineering* 195:7093–7114.

Orduña, A. & Lourenço, P. B. 2005a. Three-dimensional limit analysis of rigid blocks assemblages. Part I: Torsion failure on frictional interfaces and limit analysis formulation. *International Journal of Solids and Structures* 42(18–19):5140–5160.

Orduña, A. & Lourenço, P. B. 2005b. Three-dimensional limit analysis of rigid blocks assemblages. Part II: Load-path following solution procedure and validation. *International Journal of Solids and Structures*. 42(18–19):5161–5180.

Penelis, G. G. 2006. An efficient approach for pushover analysis of unreinforced masonry (URM) structures. *Journal of Earthquake Engineering* 10(3): 359–379.

Preciado, A. 2007. *Seismic vulnerability assessment of historical buildings at Colima State* (in Spanish). Coquimatlan: University of Colima, master thesis.

Rots, J. G. & Invernizzi, S. 2004. Regularized sequentially linear saw-tooth softening model. *International Journal for Numerical and Analytical Methods in Geomechanics* 28:821–856.

Shibata, A. Motosaka, M. & Hagiwara, Y. 2000. Prediction of damage rate of building groups in urban areas by strong earthquakes. *12 World Conference on Earthquake Engineering*, Auckland, New Zealand.