"Façade Seismic Failure Simulation of an Old Cathedral in Colima, Mexico by 3D Limit Analysis and Nonlinear Finite Element Method"

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ABSTRACT

Earthquake protection of historical buildings is fundamental for the economy and development of a country and is a topic of intensive research among the scientific community. Two different material models and approaches such as 3D Limit Analysis and nonlinear Finite Element Method are used and compared for the seismic evaluation of an old masonry Cathedral in Colima, Mexico. It has been strongly damaged by a M7.6 earthquake in 1941, generating strong damage to the main façade with the collapse of the left tower. It was damaged again by a similar event of M7.5 in 2003. Both events mainly damaged the main façade including both bell-towers. In this paper, firstly, the Cathedral is completely modeled and a macro-element representing the observed most damaged part is selected. The seismic assessment results by both, Limit Analysis and nonlinear Finite Element approaches are able to simulate the observed failure mechanisms at the frontal façade and the obtained seismic coefficients are in good agreement. Moreover, the advantages and disadvantages through the seismic analysis process corresponding to the pre-processing, analysis and post-processing by the use of both approaches are detailed.

Keywords: *Strong earthquakes, façades, historical masonry, seismic assessment, failure mechanisms, performance, 3D Limit Analysis, nonlinear Finite Element Method*

1 SEISMIC VULNERABILITY ASSESSMENT OF CULTURAL HERITAGE

Ancient buildings represent a high historical, cultural and heritage value for every society all over the world. Due to the partial or total collapse of this type of buildings observed very often in the history in earthquake (EQ) prone zones, arises a great concern to find reliable and suitable methodologies to keep these invaluable monuments. Their protection is fundamental for the economy and development of certain countries (especially in Europe) and is a topic of intensive research among the scientific community. Assessing the seismic vulnerability of a historical building is a complex task if compared to other existing or new building as explained in the works of Barbieri et al. (2013), Foraboschi (2013), Preciado et al. (2014) and Preciado and Orduña (2014). The main difficulties on the seismic analysis and strengthening of these buildings arise from the heterogeneity of its main construction material, unreinforced masonry (URM). The seismic behavior of this quasi-brittle material is governed by its low tensile strength and, therefore, its nonlinear behavior since very low EQ vibration. These factors, combined with the heterogeneity of materials, anisotropy, lack of good connection, EQ source, frequencies and local site effects, make the seismic vulnerability analysis a complex task. Nowadays, there is an enormous variety of methods to assess the seismic vulnerability of buildings (Carreño et al. 2012). Recent studies in EQ engineering are oriented to the development, validation and application of techniques to assess the seismic vulnerability of existing buildings (Carreño, et al., 2007; Barbat, et al., 2008; Lantada, et al., 2009 and Pujades, 2012). The amount of identified damage in the seismic vulnerability assessment of buildings depends on many factors such as intensity of the seismic action, soil conditions, constructive materials, state of previous damages and structural elements. Another important aspect to consider is whether the structure was designed to resist EQs (nowadays buildings) or only to withstand their own self weight like most of historical constructions.

Seismic vulnerability assessment of buildings is an issue of most importance at present time and is a concept widely used in works related to the protection of buildings. Nevertheless, there is not a rigorous and widely accepted definition of it. In general terms, vulnerability measures the amount of damage caused by an EQ of given intensity over a structure. However, “amount of damage” and “seismic intensity” are concepts without a clear and rigorous numerical definition (Orduña et al., 2008). The selection of a suitable method for the seismic vulnerability assessment mainly depends on the nature and objective of the study, as well as the reliability of the expected results. This means that it is possible to evaluate the seismic vulnerability of a large group of buildings in a quite general manner by following simple approaches (qualitative), or only to evaluate one building in a detailed way by means of refined methods (quantitative). Qualitative approaches allow obtaining a vulnerability qualification of the buildings or group of buildings in terms of seismic vulnerability that could range from low to high, whereas the quantitative ones evaluate the vulnerability in numerical terms (e.g. ultimate force, displacement capacity and failure modes). These approaches are mainly computerized numerical methods and have gained wide acceptance within the structural engineering community are integrated by the Finite Element Method (FEM) and Limit Analysis (Preciado, 2007 and 2011). These quantitative methods have the common characteristic of being more refined than qualitative ones and in some cases require many parameters for modeling the real physical characteristics of the actual structure. Evidently, these facts render quantitative methods more complex and time consuming than qualitative ones. When a professional assesses the seismic vulnerability of an ancient building, he constructs the geometrical model, and then assigns the mechanical properties of materials and boundary conditions together with a suitable constitutive material model. The model is statically or dynamically analyzed in the nonlinear range.

The Cathedral under study (see Fig. 1) is located in the historical center of Colima City, characterized for being at one of the Mexican regions under very high seismic hazard (Fig. 1a) with strong EQs of more than M7.5 and intensities ranging from VII to X, in the Modified Mercalli Intensity scale (MMI). This building is considered as the most important Colonial monument of the state of Colima by its great historical and cultural value.

2 SEISMICITY OF COLIMA, MEXICO

The state of Colima (Colima City is the capital) is located at Western Mexico in the Pacific Ocean Coast and adjoins with the states of Jalisco in the NW direction and with Michoacan in the SW. At national level, the seismic hazard of Mexico is divided in four main zones ranging from A to D, where A represents low hazard and D very high (see Fig. 2a). In the seismological context Colima is distinguished by its important exposure (seismic zone D), being considered one of the Mexican states under most significant seismic hazard (Preciado and Orduña, 2014). Bandy et al. (1995) and Ramirez-Gaytan (2008) describe that the seismic hazard of Colima is determined by three main sources: the active Volcano of Colima that generates constant microseismicity ($M<3.5$); the Jalisco block located between the Rivera and North American plates and the convergence zone between the Cocos, Rivera and North American plates in front of the coastal area (see Fig. 2b). Mexico is located in the Circum-Pacific Ring, characterized by its high inter-plate seismicity. The seismic activity is generated by the convergence of the Cocos and North American plates (6 cm/year in average) and the Rivera and North American plates (4.5 cm/year) (Bandy et al., 1995). In the boundaries between plates have occurred major to great EQs causing strong damage to cities as Manzanillo, Tecoman, Colima, Guadalajara and Mexico. The black arrows depicted in the tectonic map of Figure 2b represent the convergence direction of the Rivera and Cocos plates with reference to the North American plate. Historically, Colima has been subjected to very important EQs of more than M7.5 and intensities ranging from VII to X based on the MMI scale. The most recent strong events that have affected the region occurred on October 9th, 1995 with a M8.0 and on January 21st, 2003 M7.5.

3 HISTORICAL ANALYSIS AND OBSERVED DAMAGE AT THE CATHEDRAL

The Cathedral of Colima, Mexico (Figs. 1 and 3) was built in 1889 and is recognized by the National Institute of Anthropology and History of Mexico (INAH), and the society, as one of the most important Colonial monuments of all the state of Colima due to its great historical and cultural value. The materials used for its construction were fired clay bricks and carved stone with lime mortar for all the vertical elements such as walls and towers and empty fired clay mugs in a mortar matrix for the vaults. The Cathedral is located at the historical center of Colima City. Historically, the building has been strongly damaged by a large EQ in 1941 of M7.6 (MMI X) that generated the collapse of the East tower and strong damage to other parts of the building as illustrated in Figure 3b. In 2003, Colima City was struck again by a similar damaging M7.5 EQ, but was felt with different intensity at the Cathedral´s site (MMI VIII). The rupture mechanism of both strong EQs was generated by the convergence of the Cocos and North American plates. The later EQ generated strong damage to the complete building as shown in the crack patterns of Figure 4. The vaulted cover structure and dome were damaged, as well as the façades, especially the frontal one (North) including both bell-towers. The building was subjected to rehabilitation works and a rough seismic retrofitting measures by the addition of steel mesh and mortar at the dome and cover, as well as reinforced concrete rings at belfries. The rehabilitation and strengthening works were developed by the authorities without a reliable seismic analysis of the

Cathedral. Nowadays, the Cathedral is in very good conservation state as it could be observed in Figure 1. However, the seismic performance of the historical building before and after the intervention is completely unknown. Therefore the need of an accurate assessment of the building by advanced methods of Analysis is evident, in order to have a better knowledge of its seismic behavior before and after the 2003 EQ. The main objective in the long term of the Colima Cathedral project is to propose a better seismic retrofitting measure that follows the current criteria of compatibility of deformations, energy dissipation and reversibility. The main Cathedral´s façade is analyzed in its original condition, before the occurrence of the 2003 EQ, by two methods with different refinement and masonry constitutive material models such as 3D Limit Analysis (3DLA) and nonlinear FEM. The seismic evaluations are compared in terms of both, failure mechanisms and performance simulation. The objective of the present paper is to identify and to discuss the advantages and disadvantages of both analytical approaches in all the stages of the seismic evaluation process (pre-processing, analysis and post-processing).

4 EARTHQUAKE ANALYSIS BY 3D LIMIT ANALYSIS

3DLA with rigid block models is a suitable approximated approach to assess the nonlinear seismic performance (in-plane and out-of-plane) and failure mechanisms of historical masonry structures ranging from small to medium size. 3DLA can be used also with advantage in the case that the information of the building is limited or to rapidly assess a group of small buildings. Limit Analysis, as a simplified tool, does not consider directly the EQ motion and structural damping, nor the main characteristics of the EQ and changes in the modal properties by the nonlinear behavior of masonry. Orduña and Lourenço (2005a, b) proposed a 3DLA with rigid block models procedure as a simplified tool to evaluate the seismic vulnerability of historical masonry structures. This approach considers that the nonlinear behavior of a masonry structure could be represented by rigid blocks interacting between them by means of frictional interfaces with no tensile strength. The interface constitutive model is based on a rigid-perfectly plastic material that does not need parameters of stiffness and softening, only strength parameters are required, being this the best advantage and attractive of the model. On the other hand, it is not possible to evaluate the displacements and deformations of the structure, which is fundamental for energy dissipation assessments in the current performance based design (PBD) philosophy. For the pre-processing stage, the 3D structural model is developed taking into account the monitoring and diagnosis campaigns. The rigid block model for whole buildings or macro-elements, in the sense of Lagomarsino (1998), uses the macro-block modeling approach. In this approach, a single block represents a portion of masonry relatively undamaged, while the interfaces represent potential large cracks produced by the EQ action. Therefore, the rigid blocks model is defined depending on the EQ direction under evaluation (-X, +X, -Y or +Y), since different cracking patterns are triggered at each case. The macro-block modeling makes use of observed damages after real EQs in the present or similar structures and failure mechanisms reported in literature. The interaction between the 3D rigid blocks and foundation is modeled trough frictionant interfaces with no tensile strength. In the solution process the strength parameters are assigned to the structural model. By solving a mathematical programming problem that includes expressions for equilibrium, Eq. 1, yield conditions, flow rule, compatibility and complementary equations (Orduña and Lourenço 2005a), it is possible to obtain, relatively fast, as a result the ultimate lateral load capacity of the model (load factor), failure mechanisms and stresses at the critical sections. Eq. 1 represents the equilibrium between the forces at the interfaces (Q) and the external loads applied to the blocks. Where Fc are the permanent loads, Fv the variable loads, α the load factor and B the equilibrium matrix. In a seismic assessment, Fv contains a lateral load distribution and the limit value of α represents the amount of these loads that produce collapse on the model.

$$Fc + \alpha Fv = BQ \qquad (1)$$

Preciado (2007), Giordano et al. (2007), Orduña et al. (2008) and Orduña and Roeder (2014) have demonstrated that Limit Analysis by 3D rigid block models represents a valuable and practical tool to approximately assess the in-plane and out-of-plane nonlinear behavior of ancient masonry buildings in seismic vulnerability studies. Compared to the refined FEM nonlinear models of an important historical building, the 3DLA model and the few needed material parameters may be used as an advantage for preliminary assessments of historical constructions of small to medium size.

It is worth noting, in the crack patterns after the 1941 and 2003 EQs of Figures 3 and 4, that the –X direction of the building (main façade to the left, East) was the most vulnerable, presenting strong structural damage with the collapse of the left belfry in the 1941 event. Based on the observed crack pattern, the 3DLA is developed for a seismic action in the –X direction. The crack pattern and our own experience in EQ failure of structures serve as the basis for constructing the rigid blocks model for this specific direction. The interfaces between rigid blocks are modeled as well, based on the direction of the EQ seismic forces, as illustrated in Figure 5. In order to simplify the nonlinear analyses and to avoid non convergence problems related to the size and complexity of the Cathedral only the most damaged part is analyzed. The main façade with both bell-towers is assessed under a seismic action in the –X direction. This specific direction was selected as aforementioned due to the observed strong damages by the 1941 and 2003 seismic events.

The in-plane behavior and failure modes of URM façades under EQ loading mainly depend of the slenderness, vertical loading level and the quality of the masonry components in terms of mechanical and physical properties. When the seismic loading is presented perpendicular to the plane (out-of-plane), the structure shows different behavior and failure modes than those when in-plane loaded, mainly due to instability conditions and connectivity. Historical masonry buildings were constructed considering empirical rules to mainly withstand their self weight, being extremely vulnerable to horizontal inertia forces generated by an EQ. Another important issue that plays an important role in the seismic vulnerability of old buildings is the lack of good connection between elements at the corners or with the roof system due to the low tensile strength of masonry. As a result of the ground shaking, the walls could vibrate out-of their plane or to be pushed by other perpendicular walls, being separated of the rest of the structure and generating a state of instability that could lead to a partial or total collapse. The elevated mass of cupolas and vaulted roofs of historical masonry buildings generate, during an EQ, important inertia forces that could be transmitted out-of-plane to the support walls and façade because the cover does not behave as a rigid diaphragm as nowadays structures. This transmission of forces out-of-plane could lead to the collapse of walls or façade by overturning or the failure of the roof system by instability.

Taking into account the aforementioned, it is assumed that for an EQ in the –X direction the main façade including both bell-towers (see Fig. 6a) is completely disconnected from the nave and generates a macro element independent of the rest of the building (Lagomarsino, 1998). Due to the lack of information about the material parameters, we used in the simulations typical values reported in literature. By means of the reports of INAH (2003) and the historical analysis of section 3, it was observed that the façade is formed by brick masonry with lime mortar and both towers with brick at the lower part and carved stone masonry at the level of belfry. In the analysis was considered a density of 1.6 ton/m^3 for brick masonry and 2 ton/m^3 for carved stone masonry, 0.6 of friction coefficient, and a compressive strength of 2.5 MPa.

Figure 6a illustrates the failure mechanisms obtained by the 3DLA at the Cathedral's façade rigid block model subjected to lateral loads in the –X direction. It is worth noting the propagation of vertical cracks due to horizontal tensile stresses that led to a disconnection of the left (East) tower from the façade, as well as a combination of in-plane shear and out-of-plane bending cracks at the tower´s lower body. 3DLA accurately predicted the observed failure mechanism at the lower body of the tower due to the 2003 EQ. However, 3DLA did not predict any damage at belfry, which was the most important failure mode as observed in 1941 with a total collapse, and did not reproduce the partial damage due to the 2003 EQ (see Figs. 3 and 4). These results are easily explained: Limit Analysis can be seen as a search for the most critical failure mechanism; therefore, it cannot identify partially developed mechanisms. This is also a consequence of that Limit Analysis works only with displacement rates defining the global failure mechanism, and does not consider actual displacements and strains. At ultimate limit state (ULS), the façade rigid blocks model resisted a lateral force of 2050 kN (seismic coefficient of 0.122) as illustrated in the capacity representation of Figure 6b. This seismic coefficient is obtained by the ratio between the resisted horizontal force (base shear) at ULS and the vertical loading, and may be interpreted as the EQ peak ground acceleration (0.122 g) needed for inducing that failure mechanism.

5 NONLINEAR EARTHQUAKE ANALYSIS BY FE METHOD

There is no reliable information available regarding the structural characteristics of the Colima Cathedral in terms of mechanical and dynamic data. During the intervention works developed by INAH (2003), the experimental campaigns were limited to characterize the type of materials of the different structural components by non-destructive sampling. The strengths of materials were not assessed, nor the level of stresses at vertical elements and dynamic characteristics. During the present research work, several technical visits were developed in order to assess by visual inspections the actual conservation state of the building, to perform a photographic survey, and most importantly, to characterize the dynamic properties of the complete Cathedral and bell-towers at the most damaged façade. The natural frequencies were obtained by means of a portable vibration analyzer (triaxial accelerometer) CSI RBM Consultant®, consisting in one sensor and its data acquisition control. The used excitation was ambient vibration (traffic and wind) and registered at the level of vaults and at the bell-towers at a height of 31 m (upper level of belfry). Afterwards, from the acquisition control, the registered data was transferred to a computer and managed with especial software. By means of the vibration spectra, the natural frequency is graphically determined. The complete Cathedral has a fundamental natural frequency in the order of 2.200 Hz in the E-W (transversal) direction and 3.245 Hz in the N-S (longitudinal). The bell-towers have similar natural frequencies between them with no great difference of 1.407 Hz in the E-W (transversal) direction and 1.622 Hz in the N-S (longitudinal) direction (Preciado, 2011).

In order to improve the representativeness of the models and reliability of the results in the seismic vulnerability assessment, they are calibrated with experimental data in the dynamic field.

The FEM model of the complete Colima Cathedral is illustrated in Figure 7, with a mesh based on quadrilateral elements. The seismic analysis of the façade is developed taking into account the same –X direction as in the 3DLA. By analyzing the obtained results with the 3DLA approach and observed failure mechanisms after the 1941 EQ, it was observed that only the left tower resulted damaged. From these observations, the FEM model is simplified and only the left tower with the interaction of the façade by tensionless springs is analyzed. As in the case of 3DLA, this simplification is developed for practical purposes, and to avoid convergence problems during the nonlinear analyses. Due to symmetry, the left (East) tower was selected for the analysis and no considerable changes are expected in the other two directions (-Y and +Y). The -X model (see Fig. 8a) is simulated with a linear distribution of linear elastic springs with no tension allowed. These springs are usually used to simulate the interaction with other elements of the building. Due to the fact that the static analysis is developed in the –X direction, the only compression springs have no effects and is equal to a model without springs only for this specific direction. It is assumed a disconnection with the façade and nave. This is developed taking into account the natural behavior of URM structures that tends to separate into macro-blocks by the concentration of tensile stresses (cracking) at the connections with other structural elements. The simplified FEM model of the façade (represented by the left tower with springs) has a square plan of 6 x 6 m with a wall thickness of 1.5 m and 31 m height. With the cover (0.10 m thick) the tower has a total height of 37 m and a reinforced concrete slab at belfry (total mass of the structure of 1707.4 Ton). Each of the 3D FEM models is integrated by 859 Shell43 elements and 906 nodes with 5367 degrees of freedom (DOF) and developed by the commercial FEM software ANSYS®. The mechanical properties of the model are defined taking into account the aforementioned for 3DLA. In the generation of the initial FE model there are several assumptions and uncertainties regarding the determination of geometry, material properties, support and boundary conditions. Due to this fact, the initial analytical model may be compared with real physical characteristics of the structure. The model is calibrated or updated through modal analyses by modifying masonry elastic modulus, density and spring stiffness. After following an iterative approach, the numerical and experimental frequencies are in good agreement, as presented in Table 1.

The EQ assessments are developed through nonlinear static analyses by means of the Pushover technique following a displacement load pattern assuming that the tower behaves as a cantilever beam of 1 DOF and implementing the masonry material model developed by Gambarotta and Lagomarsino (1997). The model is capable to simulate the main failure behavior of masonry structures in static and dynamic conditions. This accurate material model has been validated by theoretical background and reported experimental examples in the research work of Preciado (2011). The constitutive model is integrated in the commercial finite element program ANSYS® by subroutines and is based on the macro-modeling approach, which is considered as appropriate for the seismic assessment of large historical constructions. Furthermore, the suitability of the material model in masonry structures has been proved through numerical simulations against experimental results e.g. Calderini and Lagomarsino (2006). The continuum damage model is based on a micromechanical approach where masonry is assumed as a composite medium made up of an assembly of units connected by bed mortar joints. The contribution of head joints is not considered. The constitutive equations are obtained by homogenizing the composite medium and on the hypothesis of plane stress condition. The failure limit states for mortar and unit damage are depicted in Figure 9. The homogenised model is characterized by three yield surfaces determined by tensile failure and sliding of mortar joints considering the Coulomb friction law and compressive failure of units. In summary, if tensile stresses act in mortar bed joints $\sigma_y \geq 0$, three damage mechanisms may become active: failure of units, sliding and failure of mortar bed joints. On the other hand, if mortar joints are under compressive stresses $\sigma_y < 0$, then both damage mechanisms of units and mortar are activated.

The needed masonry material parameters are summarized in Table 2. In order to assess the seismic response of an historical building is recommended to obtain the material parameters through detailed experimental campaigns. This is always a complex task, mainly due to the heterogeneity of masonry, the lack of representative samples and the need of non-destructive tests. In case that it is not possible to obtain all the material parameters, those proposed and calibrated through numerical simulations by Preciado (2011) are recommended.

The failure mode of the simplified FE model of the façade through the left tower and a seismic action in the –X direction is presented in Figure 8b. It is worth noting several flexural cracks at the lower part of the body and a failure of belfry by a combination of flexural cracks out-of-plane and in-plane shear. The failure mechanisms obtained through the numerical simulations are in very good agreement with the observed after real EQs and are characteristic of bell-towers (flexural cracks at body and shear at belfry). The simplified façade presented an ultimate lateral force capacity of 2105 kN and a displacement of 100 mm. The different seismic performances of both methodologies could be observed at the capacity curves illustrated in Figure 6b. It is worth noting that the obtained seismic coefficient by 3DLA of 0.122 is in very good agreement with the obtained by means of the FE method of 0.126.

6 COMPARATIVE BETWEEN 3DLA AND NONLINEAR FEM

The 3DLA approach is a suitable tool to assess the nonlinear seismic performance and failure modes of historical masonry structures ranging from small to medium size. The approach does not present strong convergence problems as the FEM and the calculation time is reduced, giving in a practical manner the ultimate lateral force capacity and failure mechanisms of a structure. On the other hand, in 3DLA with rigid blocks models, it is not possible to calibrate with experimental data, which becomes a great disadvantage in the model calibration/updating stage for having realistic results.

The constitutive material model used in 3DLA is based on a rigid-perfectly plastic material that does not need stiffness and softening parameters, only strength parameters are considered, being this one of the main advantages. On the other hand, it is not possible to evaluate the displacements and deformations of the structure, which are fundamental for energy dissipation assessments in the current PBD philosophy. Due to the fact that the model is developed taking into account the failure mechanisms, the user needs experience on EQ failure and behavior of historical constructions. The model generation is time consuming due to the need of a different model for each specific direction of the seismic action, as well as the interfaces between rigid blocks (Fig. 5). The construction of the rigid blocks model and the impossibility to assess ductility for energy dissipation purposes and model calibration are the main drawbacks of this proposal compared to the FEM approach.

On the other hand, the FE method allows to the user to obtain a detailed seismic analysis of a historical masonry structure in terms of force, displacement and distribution of stresses and plastic strains (cracking). The lateral force capability allows knowing the strength and ductility of the structure to determine the energy dissipation capacity. The modelling process is detailed and cumbersome as in the case of the 3DLA, but is developed only once for the entire model and may be analyzed in any direction because the modelling is not dependent on the seismic action. The modelling technique depends on the objective of the analysis (linear or nonlinear) and the main concerns are the computational time, element size and convergence problems in the nonlinear analyses. In terms of computational time, the FEM takes longer to develop a nonlinear analysis if compared with 3DLA, even in static conditions, and is increased in dynamic nonlinear analysis, taking days or even weeks for developing an analysis. Moreover, the nonlinear analysis by the FEM presents lots of convergence problems, due to the size of the model and mesh distortion.

The material model is very good in accurately predict the failure mechanisms and behavior of URM. Its main drawback is that needs many parameters obtained in laboratory and their calibration by numerical and experimental tests on real scale structural elements. The strong convergence problems are due to the sensitiveness of the constitutive model to the material parameters and needs to be improved for its use in large structures.

7 CONCLUSIONS

Earthquake protection of historical buildings is fundamental for any country and is a topic of intensive research among the scientific community. Two different material models and approaches such as 3D Limit Analysis with rigid blocks models and nonlinear Finite Element Method were used and compared for the seismic evaluation of an old masonry Cathedral in Colima, Mexico. This building was struck by a M7.6 EQ in 1941 and recently in 2003 by a similar EQ. Both seismic events strongly damaged the main façade, collapsing in the 1941 EQ the left bell-tower. 3DLA is a suitable tool to obtain, in a practical manner, the ultimate lateral force capacity and failure modes of a structure in static conditions. The rigid blocks model is not able to be calibrated with experimental data nor the ultimate lateral displacement is obtained, which is fundamental for energy dissipation assessment. Even when the model generation is time consuming, the obtained results were in very good agreement with the observed EQ damage at the Cathedral and those achieved by the FEM. On the other hand, the FEM approach allows us to obtain a detailed seismic analysis of a historical masonry structure including energy dissipation evaluation through calibrated models. The used material model is very good in accurately predict the failure modes and behavior of URM. Its main drawback is that needs many parameters that are difficult and expensive to obtain and calibrate. The strong convergence problems are due to the sensitiveness of the constitutive model parameters and needs to be improved for its use in large structures. The authors of this paper recommend its use only for the assessment of small and medium size structures.

In brief, 3DLA is a simplified tool that uses few input parameters and provides limited but valuable results. Therefore, this tool is suitable for a quick and cheap structural assessment of small to medium size historical masonry structures. Besides, Nonlinear FEM analysis is a very accurate tool that requires a more comprehensive and costly assessment of masonry mechanical features. Convergence problems and time consuming analyses limit the size of the models that this tool can reliably manage; therefore, it is also limited to the assessment of small to medium size structures or macro-elements. Both, 3DLA and nonlinear FEM analyses are valuable tools with different application niches in the seismic assessment of ancient masonry constructions. The authors recognize that other analysis tools, more accurate than 3DLA and more suitable for practical work than nonlinear FEM, have to be developed in the short term.

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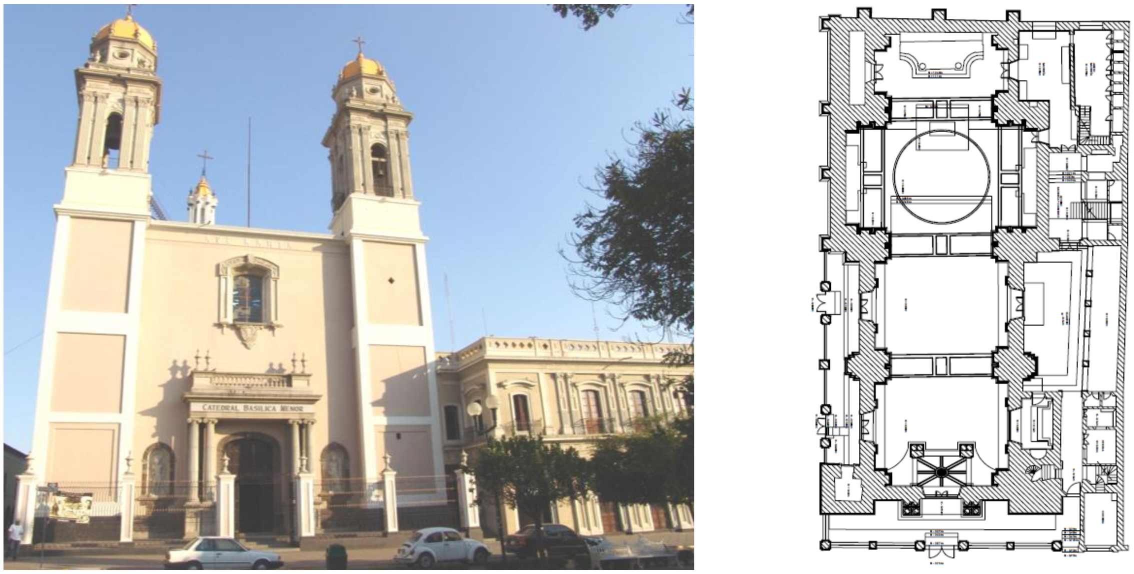
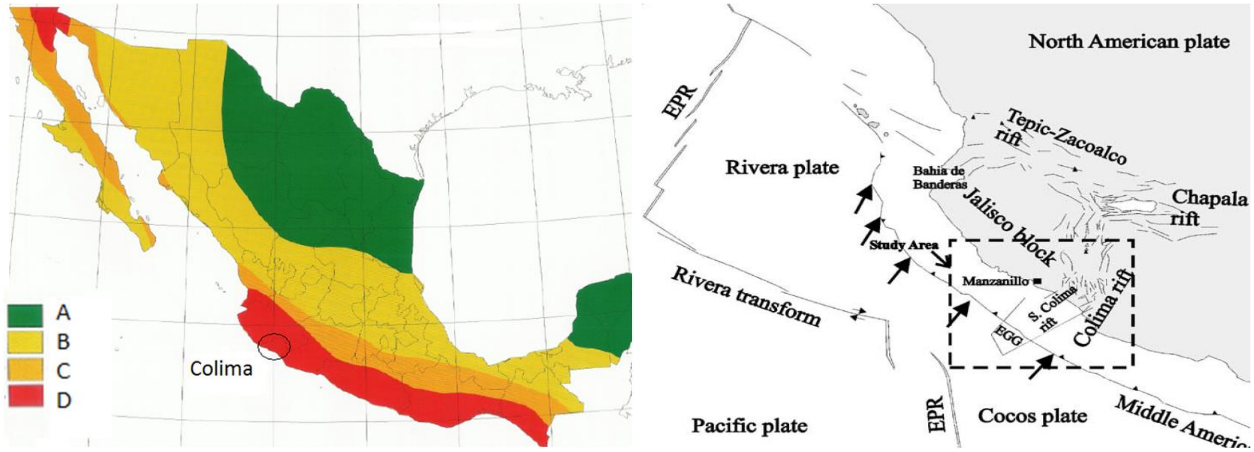


(a) (b)

Figure 1: The Cathedral of Colima nowadays; (a) General view including the municipal neighbor building and (b) architectonic plan view



(a) (b)

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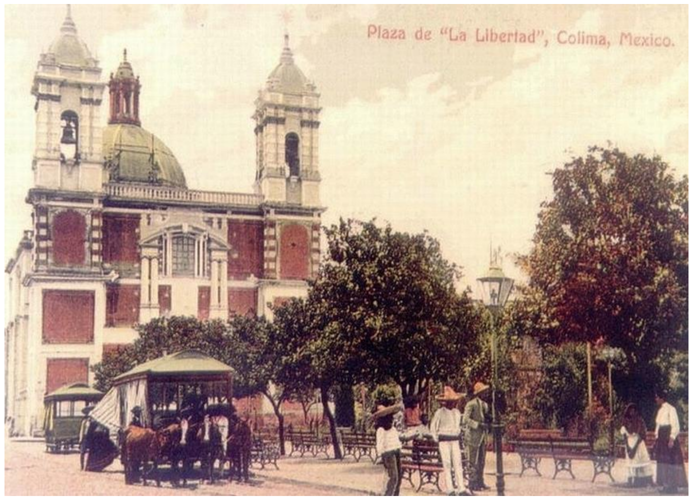
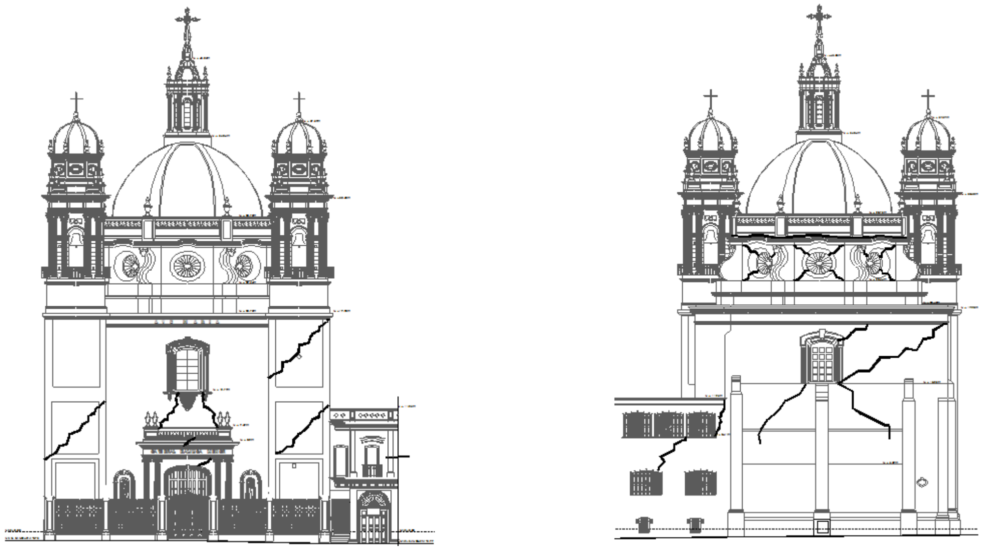


(a)

(b)

Figure 3: Observed damage at the Cathedral of Colima by the 1941 M7.6 EQ; (a) before EQ and (b) after EQ effects on vaults and left bell-tower collapsed



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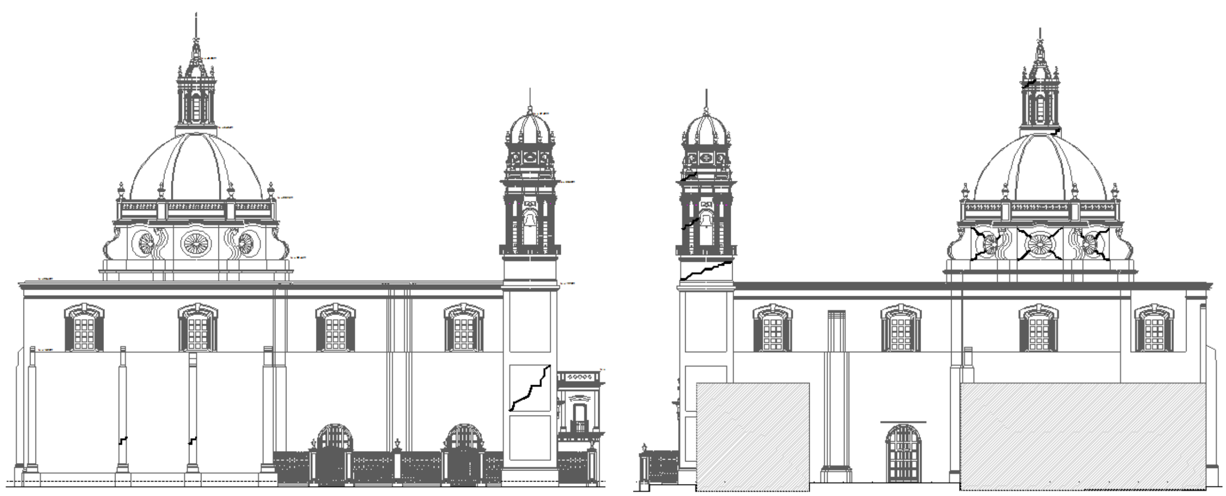

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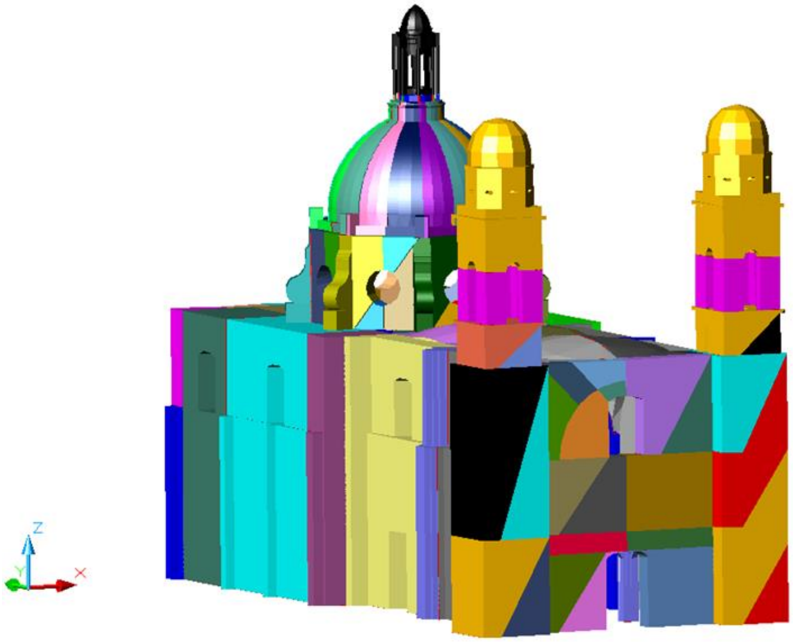

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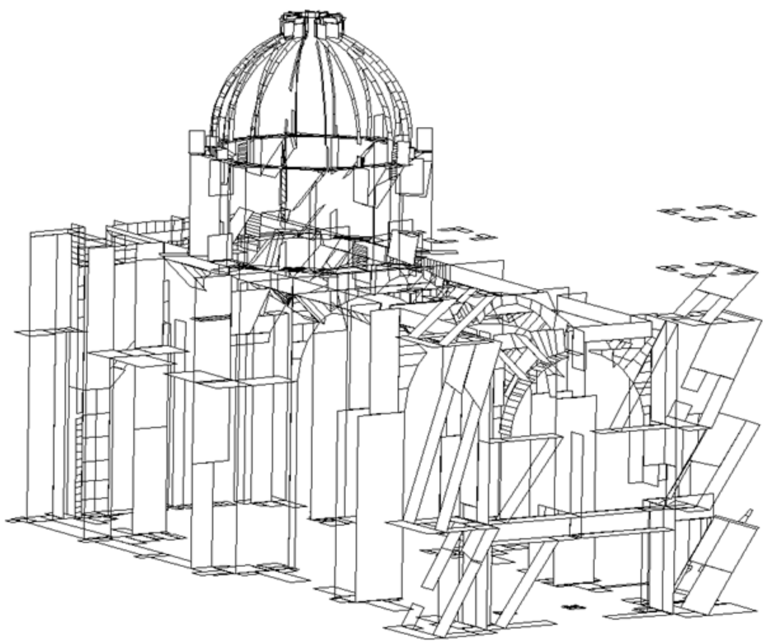
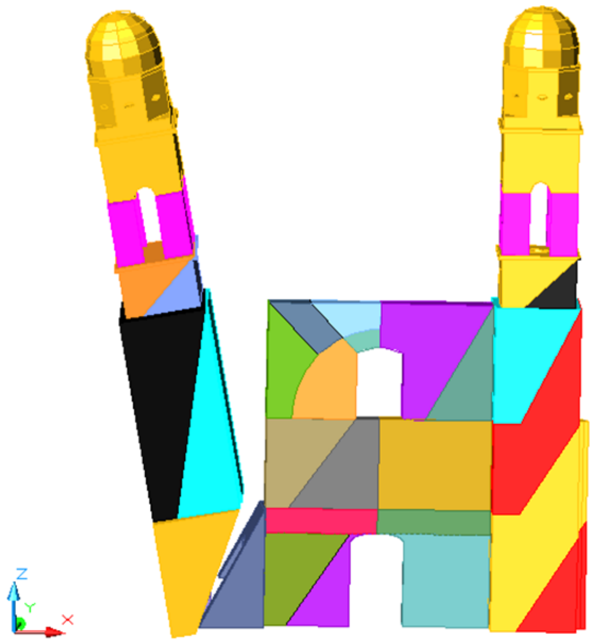
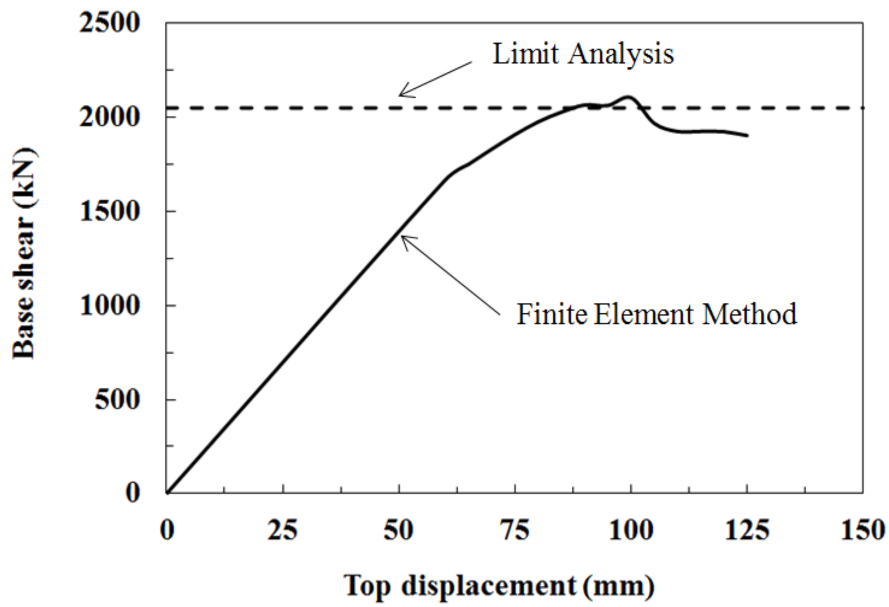


(b)

Figure 5: 3DLA model of the Colima Cathedral for a seismic action in -X; (a) rigid blocks based on observed damages by the 2003 EQ and (b) interfaces between blocks



(a)



(b)

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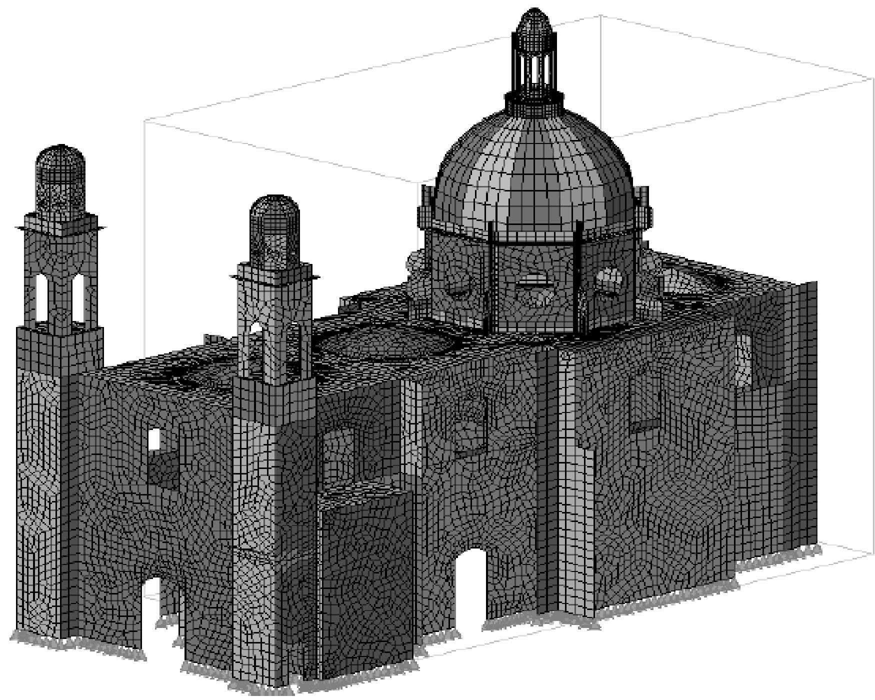

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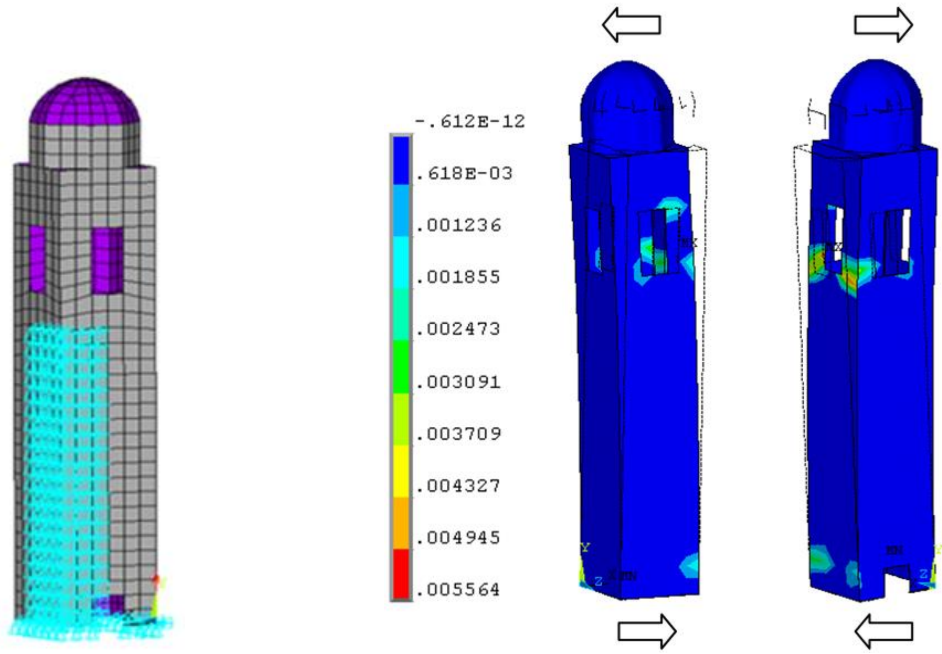

(a) (b)

Figure 8: Results of the nonlinear EQ analysis (-X) by FEM; (a) FEM simplified model of the Cathedral´s façade and (b) failure mechanisms (front and back) at ULS

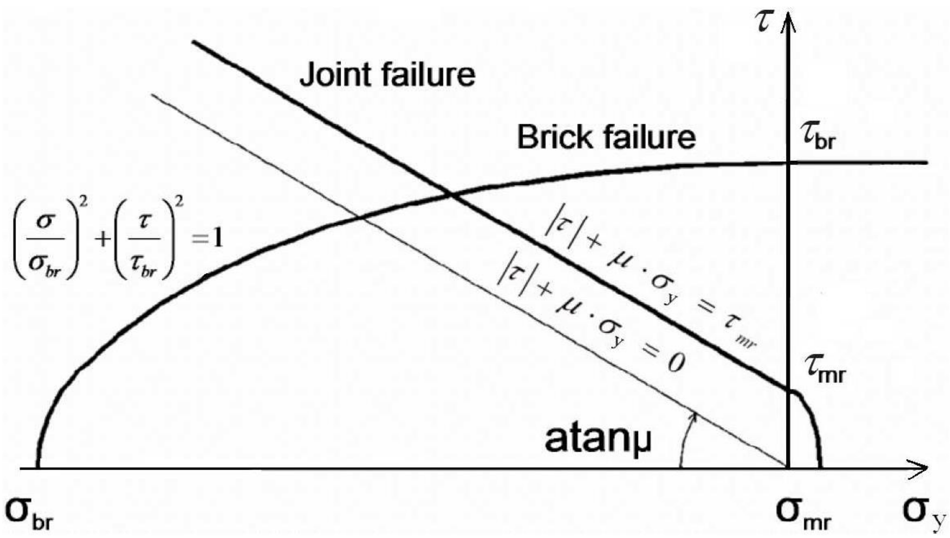

Figure 9: Mortar joint and brick failure domains (Gambarotta and Lagomarsino, 1997)

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Table 1: Numerical vs. experimental frequencies

Mode type	**Experimental Frequency (Hz)**	**FE Frequency (Hz)**	**Error (%)**
1st flexural E-W	1.4067	1.4193	0.89
1st flexural N-S	1.6222	1.6174	0.30

Table 2: Summary of masonry inelastic parameters for the material model

Parameter	Value	Unit
σ_m: tensile strength for mortar	0.25	MPa
τ_m: shear strength for mortar	0.35	MPa
c_m: shear inelastic compliance for mortar	1	-
β_m: softening coefficient for mortar	0.7	-
μ : friction coefficient for mortar	0.6	-
σ_M : compressive strength of masonry	2.5	MPa
τ_b : shear strength of units	1.5	MPa
c_M : inelastic compliance of masonry in compression	1	-
β_M : softening coefficient of masonry	0.4	-