Seismic Performance Assessment of an Unreinforced Masonry Historical Building Using Nonlinear Static Methods

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Abstract. Historical heritages embody the identity and evolution of a people, making their preservation paramount. Buildings from the 17th and 18th centuries constitute a significant portion of this heritage, and evaluating their behavior under potentially damaging actions is crucial for mitigating and preventing irreparable losses. Despite Brazil's location in a tectonically stable region, seismic events with the potential for considerable damage have occurred and could impact these historical structures. Among the methods for assessing the seismic vulnerability of buildings, nonlinear static methods stand out, aiming to quantify the effect of earthquakes on a structure using an incremental lateral force analysis considering nonlinearity, known as pushover analysis. This study aims to evaluate the seismic performance of an 18th-century masonry historical building using a nonlinear static method, notably the oldest one in the urban complex of the central-south region of the state of Ceará. To this end, a finite element numerical model of the main façade was developed to represent the historical building. The seismic performance of the building was then evaluated, and the expected damage to the historical building under seismic loading was quantified.

Keywords: N2 Method, Seismic Vulnerability, Unreinforced Masonry Building, Pushover Analysis

1 Introduction

Whether in Brazil or elsewhere, masonry buildings constitute a large amount of existing constructions in the world. Historical heritage commonly consists of masonry structures. In order to assess seismic performance of these building, many numerical models have been developed to represent its behavior. These strategies vary from block-based models (BBM), continuum models (CBM), macro element models (MM) and geometry-based models (GBM). Due to complexity of masonry structures and the high nonlinear mechanical response, the computational analysis of these structures is a challenging task [1].

The equivalent frame model (EFM) has been widely used to represent masonry structures as a result of simplicity and the computational efficiency for nonlinear analysis and reduced input data. In fact, it has been implemented in most commercial software packages as in 3Muri, MIDAS Gen, CDSWin, Aedes. The EFM is based on the assumption that the nonlinearity response of masonry walls can be replaced by a frame, frequently defined as pier (vertical elements) and spandrels (horizontal elements) [1].

Even though Brazil is located in a stable region, low magnitude earthquakes have taken place and damaged properties. In the city of João Câmara, State of Rio Grande do Norte, Brazil, in 1986 an earthquake of 5.1 magnitude occurred devastating about four thousand houses including a church. Some recent studies have discussed the importance of considering the seismic load in the structures, even in regions with low seismic activity [2], [3].

This paper focuses on the seismic performance assessment of an unreinforced masonry historical building using nonlinear static methods. The building is located in the state of Ceará, Brazil, and is the oldest building in the urban complex. The main façade of the building was modeled using equivalent frame approach with distributed plasticity (DP) through the open-source package OpenSees [4], and the seismic performance of the façade was evaluated through N2 Method.

2 Equivalent-Frame Approach

Structure idealization of piers and spandrels comes from the earthquakes' observation where failures and cracks have been seen in these elements. Meanwhile, the connection between these elements are not usually damaged and these parts of the walls are also known as rigid zones or nodes. The piers may also be defined as vertical resistant elements that carry both vertical and lateral loads. Spandrel elements consist of horizontal element between piers or two openings. According to [1], the spandrels affect the boundary conditions of the piers and the global behavior of the wall, because they have the potential to limit relative rotations or not.

The choice of the spandrels and piers plays an important role since the effective height impacts the overall capacity of the wall. For regularly distributed openings the identification of masonry piers may be seen as a trivial task, but it becomes complex for irregularly distributed openings. Although there is not a parametric analysis to define the piers and spandrels or even an experimental test to define the effective height, the choice of the piers falls in the criteria conventionally assumed in literature. A common approach is to define spandrels based on the vertical alignment of the openings. The length and height are equal to the distance and width of the adjacent openings. The second step is the definition of the piers which may be taken as the height of the adjacent openings. As defined in [5], the height of piers may also be defined as the average distance of the interstorey height and the height of the adjacent openings. Once the idealization of piers and spandrels are concluded, the rigid nodes are directly revealed. The main failure modes are flexural and shear. While the former refers to crushing and rocking failures and is simulated by the axial load-bending moment (N-M), the latter is related to sliding and diagonal cracking failure and is represented through shear force-deformation/displacement $(V - \gamma)$. The piers and spandrels are implemented as a displacement-based frame element with fiber cross-section to describe the nonlinear in-plane behavior. The spread plasticity model with fiber discretization of cross-sections allow to simulate the axial and flexural interaction. As explained by [6], the integration of the stress on the fibers results results in the sections' axial force and bending moments, while integration of the section force at the integration points provides the element's end forces (Fig. 1). Thus, the two-dimensional finite element model of the façade is generated in open-source OpenSees [4]. In summary, it is a static problem and the nonlinear behavior of the masonry is captured by the constitutive laws assigned to the fibers.

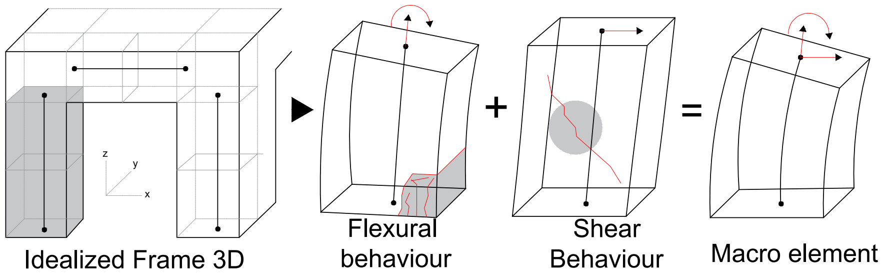


Figure 1. Macro-Element behavior

3 N2 Method

The N2 method briefly compares the capacity of a structure with the demands from the earthquake ground motion. The method was proposed by [7], and it is based on nonlinear static methods. This method aims to estimate the structural response of a building to earthquakes according to static response. A time-domain dynamic analysis is beyond the scope of this work. In the present method, the structural capacity is defined by a capacity curve, given as the structural response from the nonlinear static analysis. The maximum displacement capacity is given as the last point of this capacity curve. First step consists on performing the pushover analysis to obtain the capacity curve of the structure. This analysis conventionally is performed by applying a lateral force on the structure and increasing it until the structure reaches the collapse state. This load pattern may impact in the pushover analysis and some code standards suggests from triangular (TR), inverted triangular (INV), uniform (UN) pattern, concentrated load at top (TOP) or even proportional to mass (MASS)[8]. The main target is to generate the capacity curve which correlates the base shear with the displacement of the structure at a pre-defined control point. The second step of N2 is to transform this capacity curve from a multi degree of freedom (MDOF) to an equivalent single degree of freedom (SDOF) capacity curve. This transformation encompasses changing the MDOF quantities to SDOF quantities. In equation eq. (1), Q^* is the quantities in the equivalent SDOF, Q the corresponding quantities in the MDOF system, Γ is the transformation factor that controls the conversion from MDOF to SDOF and vice-versa.

$$Q^* = \frac{Q}{\Gamma}. \tag{1}$$

The Γ factor is determined by the mass of the equivalent SDOF system (m^*) and the mass of the MDOF system (m_i) and Φ_i is the displacement shape.

$$\Gamma = \frac{m^*}{\sum m_i \Phi_i^2} \tag{2}$$

The third step requires to convert both the bilinear capacity curve and seismic demand (i.e. the response spectrum) to the Acceleration-Displacement (AD) format. This involves finding the reduction factor R_μ, ductility factor μ and displacement demand S_d. In eq. (3), T_c is the characteristic period of ground motion and T is the elastic period of the idealized bilinear system.

$$R_\mu = (\mu - 1)\frac{T}{T_c} + 1 \quad T < T_c \, and \, R_\mu = \mu \quad T \geq T_c \tag{3}$$

Since this displacement demand is in the SDOF format, the fourth step is to convert it back to the MDOF format. The final step is to compare local and global seismic demand with the capacities for the relevant performance level. To clarify the performance point of the structure is the intersection of the inelastic spectrum with the capacity spectrum of the SDOF (Fig. 2) and hence the displacement is transformed back to the MDOF system and the performance at maximum displacement is evaluated on the global and local level [9].

The performance-based assessment of any building requires comparing the demand and capacity in terms of displacement. The damage level varies according the specific seismic hazard conditions. Thus, the structural response may be quantified in terms of displacement levels and these are associated with the performance levels or limit states. In [10] the damage limits are: Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). In order to evaluate the performance level through the pushover curve, three approaches have been practiced in literature: 1) Global criteria, 2) Local criteria and 3) Global-Local criteria. For the NC performance level, the global check in pushover analysis assumes the Displacement Capacity of the structure is the point where building's base shear is reduced by 20 %. At global scale, [11] suggests the definition of thresholds for the damage level based on the calibration of extensive application of multiscale approach. The damage levels are defined as: DL1, DL2, DL3 and DL4. The displacement limits are shown in Table 1 where κ_G is the total base shear over the maximum base shear of the pushover curve ($\kappa_G = V_b/V_b max$). The first two thresholds are in the growing branch of the pushover curve and the last two are in the descending branch.

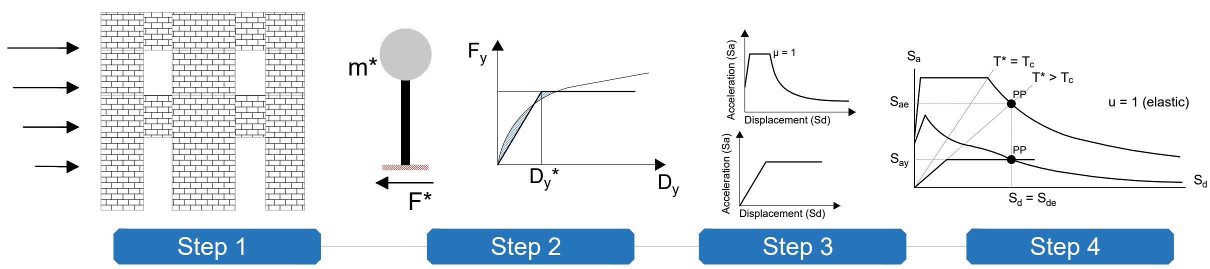


Figure 2. Summary of the N2 Method

Table 1. Criteria for the damage levels of global response of existing masonry buildings

Scale	Thresholds	DL1	DL2	DL3	DL4
Global	κ_G	≥ 0.5	0.95-1	0.8-0.9	0.6-0.7

4 Description of Nossa Senhora do Rosário Church

According to [12] the church Nossa Senhora do Rosário is part of the three hundred heritage buildings of the city of Icó, state of Ceará. The church was built in 1828 and its value is directly related to the religiosity of black people who considered the Virgin of the Rosary as their protector and patroness. This place was considered a symbol of refuge because of the declared discrimination prevalent during that period.

The facade's height is 24 meters and the ground floor plan measures 30.89 meters in length and 17.24 meters in width. The thickness of wall varies from 0.56 meters to 1.0 meters and is constituted of solid bricks. It was modelled with adopted wall thickness of 0.90 meters (Fig. 3, Fig. 4). Since the church is a historical building, the material properties are difficult to acquire through destructive experimental campaign, the data herein used was based on [13] and the geometry was obtained from the original project. The material properties are shown in Table 2.

Table 2. Mechanical properties for the constitutive law

$E(MPa)$	$f_m(MPa)$	ϵ_0	$f_{mr}(MPa)$	$\gamma\ (kN/m^3)$	v	ϵ_u
1490.0	6.2	0.0009	0.18	18	0.20	0.010

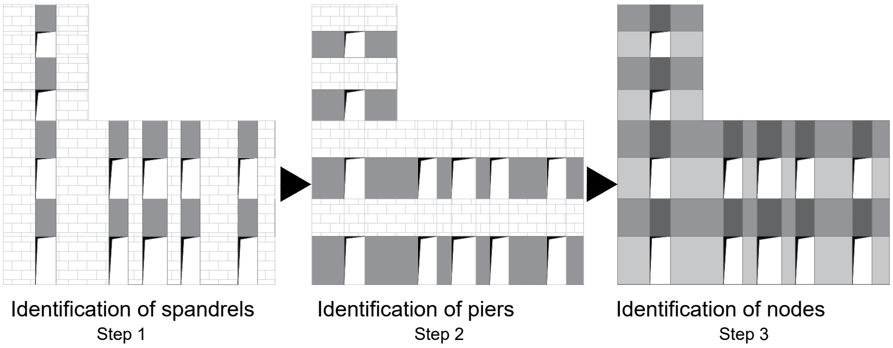


Figure 3. Example of equivalent frame idealization

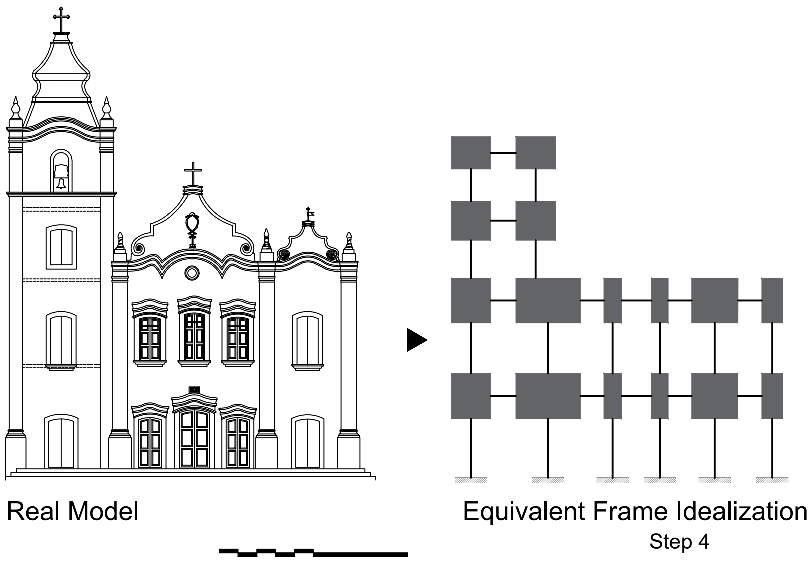


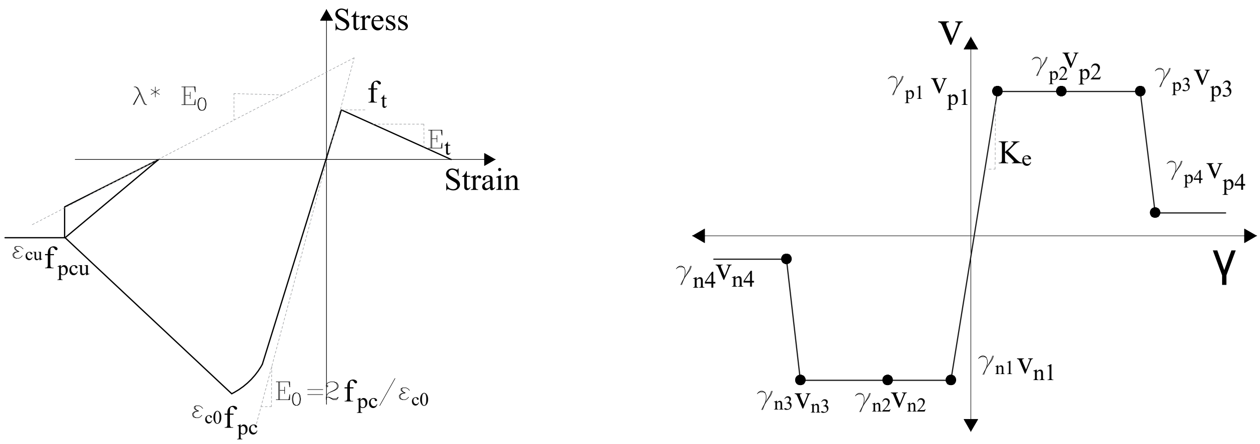
Figure 4. Idealization of Façade of Church

For the purpose of considering bending failure, five sections were delimited with two at the edge. The nonlinearity of masonry was analyzed through a nonlinear uniaxial stress-strain law throughout the frame. In OpenSees many constitutive laws are available to simulate the nonlinear behavior of materials. In this research the 'Concrete02' uniaxial material law was used even though it has been developed for concrete it represents well the masonry Fig. 5a. The shear force-deformation law (V - γ) was captured by 'pinching4' uniaxial material also available in OpenSees. This material can simulate the pinched load-deformation response and the degradation of the shear stiffness [14] (Fig. 5b). Numerous parameters are required to describe the positive and negative envelope of the shear force-deformation curve. The ultimate shear strength follows the Turnsek and Cacovic failure criteria [15]. The initial deformation γ_1 may be calculated as V_3/K_e since K_e is the initial elastic shear stiffness and suggested by [16] as $K_e = GA_s$ where G is the shear modulus and A_s is the cross sectional of the wall. The ultimate deformation γ_3 according to Italian code [13] is 0.4% and γ_4 is specified as 1% as recommended by [17]. The value of γ_2 is found by the average value of γ_1 and γ_3. The shear strength of the unreinforced masonry walls is given by eq. (4) where is $\sigma_0$the section average compression stress, τ_0 is the diagonal cracking shear strength of masonry panels, l and t are the width and the thickness of the wall section, respectively:

$$V_t = 1.5\tau_{0d} lt\,\beta\sqrt{1 + \frac{\sigma_0}{1.5\tau_{0d}}}, \tag{4}$$

with β coefficient function of the element slenderness.

The residual shear strength V_4 is given as 20% of the ultimate shear strength. The initial shear strengh V_1 and intermediate value V_2 are equal to the ultimate shear strength (V_3).



(a) (b)

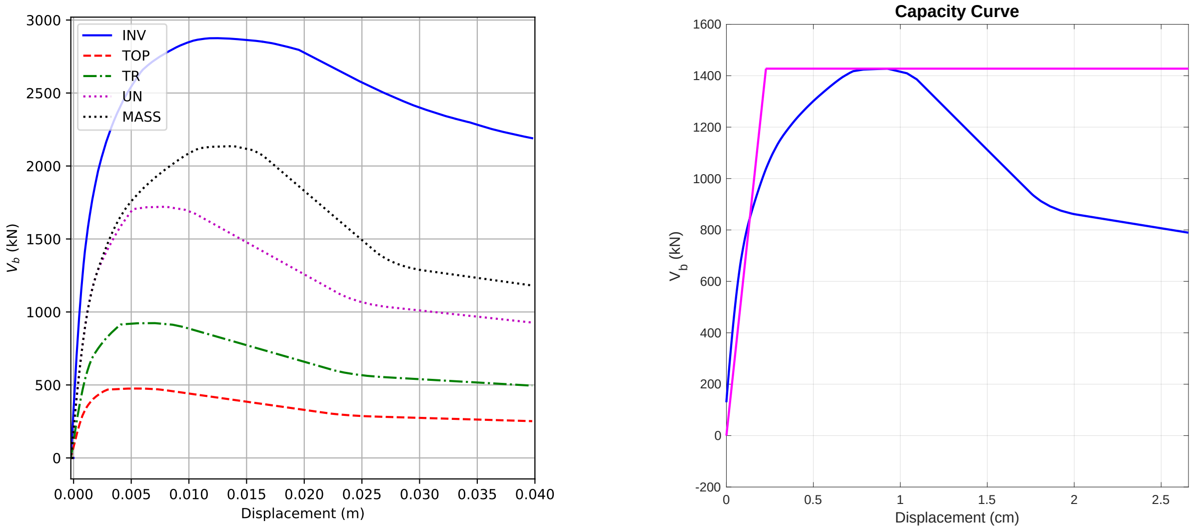
Figure 5. (a) Uniaxial laws of tension and compression assigned to the fibers. (b) Shear phenomenological law of the masonry macro-element.

5 Results and Discussions

The response spectrum used as seismic demand was obtained from the Brazilian seismic code [18] for a return period (T_R) of 475 years (Ultimate Limit State requirement) and a soil type E with SPT $N \leq 15$ in the first 30 meters. The Table 3 summarizes the results of the pushover analysis for several load patterns and their respective performance points. It is worth to mention that for all patterns, except the top one, the bilinear capacity curve overcome both the elastic and inelastic spectrum. The Fig. 6a shows the pushover curves for different load patterns and the bilinear approximation of the capacity curve (Fig. 6b). A significant difference is observed among inverted triangular load pattern where capacity attained 2876.02 kN meanwhile the application of a lateral load at the top of the tower of church conducted to a maximum shear base 475 kN. This abrupt reduction of capacity is due to the fact that the load pattern is not distributed along the height of the church leading the pier at the top of the church to reach ultimate flexural capacity. The initial branch of pushover curve related to the elastic behavior is similar for uniform and mass proportional load pattern. The inverted triangular load pattern reached elevated values of capacity since the load is concentrated at the base of the church. On the other hand, the triangular load pattern showed an equivalent behavior to the load pattern at the top of the church. Since the performance point (*PP*) is the maximum displacement demand d^*_{max} expected under a given earthquake, the displacement capacity of the façade of church in each load pattern overcomes the *PP* and as aforementioned this displacement capacity is on damage level NC (near collapse). The thresholds of damage levels for the church's facade in the pushover curve are displayed in Table 4.

Table 3. Load Patterns and Performance Points for Nossa Senhora do Rosário Church

Load Pattern	$V_b max$ (kN)	Performance Point (cm)	μ	d^*_{NC} (cm)
Uniform	1721.101	-0.1506	-0.7983	1.72
Mass' Proportion	2136.376	-0.2563	-0.7531	2.17
Top Node	475.787	0.2825	2.7372	1.53
Triangular	924.171	0.100	0.669	1.64
Inverted Triangular	2876.02	-0.4427	-1.674	3.42



(a) (b)

Figure 6. Pushover curves for different load patterns (a) and the capacity curve and its bilinear approximation (b).

Table 4. Thresholds for the damage levels for proportional mass load pattern

Global	DL1	DL2	DL3	DL4
δ (cm)	0.1363	1.38379	2.17729	3.06128

6 Conclusions

Nonlinear static analysis represents a feasible tool for performance based assessment of existing buildings in order to evaluate if fulfills pre-defined performance levels. This work aimed to assess the seismic performance of a historical church through Performance-Based procedure. The in-plane capacity curves for the building case studies were obtained using nonlinear static (pushover) analysis, and the performance points of the structures were determined through nonlinear methods.

As observed in this manuscript some aspects must not be neglected as the choice of piers and spandrels, as well as the seismic load pattern and even the selection of node control. These also have a significant impact on the results mainly in irregular buildings. As noted the inverted triangular load pattern has introduced elevated capacity due to concentrated load at base while for uniform and mass proportion load pattern showed similar results in the growing branch. Taking in account the displacement demand obtained form the intersection of capacity curve and inelastic spectrum using the N2 method, the displacement capacity of the structure is higher than the expected demand. The in-plane behavior of structure as studied in this work indicates that for the design earthquake the church is expected to perform without significant damage. In future works, the authors intend to validate the results through nonlinear dynamic analysis since the method is more accurate once it considers the dynamic behavior of the structure and does not require the transformation of the capacity curve to SDOF format and seismic load may be modeled as a time history. Also, it is suggested to perform not only in global scale but also in macro element scale evaluating the interstorey drift θ_{wi} by any wall and level. It is worth to emphasize that the contribution of horizontal displacement and rotation of nodes may be considered in the analysis. In element scale, the attainment of the of local thresholds from constitutive laws whether checking the reaching of yield point or the ultimate point is strongly recommended.

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