Soil Dynamics and Earthquake Engineering 93 (2017) 162–171

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

A numerical procedure for the pushover analysis of masonry towers

Massimiliano Bocciarelli*, Gaia Barbieri

Architecture, Built Environment and Construction Engineering Department, Politecnico di Milano (Technical University), piazza Leonardo da Vinci 32, 20133, Italy

ARTICLE INFO

Article history:
Received 12 January 2016
Received in revised form
28 July 2016
Accepted 30 July 2016

Keywords:
Masonry tower
Pushover analysis
Plastic hinge length
Computing time
Probabilistic analysis

ABSTRACT

In this paper, a numerical approach for the pushover analysis of masonry towers, having hollow arbitrary sections, is proposed. Masonry is considered a nonlinear softening material in compression and brittle in tension. The tower, modeled in the framework of the Euler-Bernoulli beam theory, is subjected to a predefined load distribution, but the problem is formulated as a displacement controlled analysis in order to follow the post peak descending branch of the structural response. Nonlinear geometric effects and nonlinear constraints (the latter due to surrounding buildings) are also considered. Benchmarking pushover analyses are performed with the commercial code Abaqus in relation to a real case (the Gabbia Tower in Mantua), which proved the accuracy and reliability of the results obtained with the present formulation and the noteworthy reduction of computing time.

1. Introduction

Many historical buildings are constituted by unreinforced masonry and are characterized by a high seismic vulnerability. Indeed, these structures were conceived to withstand the effects of gravity loads but were not provided for adequate lateral resistance and ductility against horizontal loads, such as those induced by an earthquake [1–4].

The analysis of masonry structures is very complex in view of their heterogeneity and uncertainty typical of the constituent materials. Masonry is a non-homogeneous, non-isotropic material, with a mechanical behavior dominated by the nonlinear phase, characterized by negligible strength and brittleness in tension, and dissipative with softening behavior in compression [5].

For these features, the seismic vulnerability of masonry buildings is rarely assessed by linear elastic analysis procedures. Nonlinear dynamic analysis methods represent in principle the most reliable tool. Nevertheless, they are very complex and require a great amount of computational resources and time [6,7] and further research efforts are still needed, before they can be confidently used in standard design [8]. Therefore, nonlinear static (pushover) procedures have been increasingly recognized as effective tools in seismic design and vulnerability assessment: they provide information on both the strength and ductility of the structure, while preserving the simplicity of a static analysis [9,10]. The main outcome consists in the curve relating the displacement of a certain point (named controlling point) to the resultant of a predefined horizontal distributed force applied to the structure. This curve, representing the seismic capacity of the structure, is then compared with the seismic demand, expressed in terms of response spectrum, through specific procedures as the N2 or the capacity spectrum method [11,12].

Due to their increasingly relevant role, several seismic codes and recommendations have recently extended the application of pushover-based methods to existing and monumental masonry buildings [13,14].

Currently, several studies are available in literature dealing with the seismic vulnerability assessment of historical masonry buildings by means of pushover analyses, e.g. [15–19], and in particular of ancient towers [20–22]. It is well recognized that these pushover-based methods may be affected by inaccuracy when applied to structures whose failure mechanisms are influenced by the higher modes of vibration [7,23,24]. For this reason, improved multi-modal pushover analyses were developed, which combine the results obtained using the inertia force distribution related to different modes, see e.g. [25].

A key point, when dealing with pushover analysis of masonry structures, is the determination of the ultimate displacement in the capacity curve. In [13] it is suggested that this is achieved when, in the descending branch, the 85% of the maximum force is reached. This criterion requires the implementation of a softening branch in the masonry constitutive law (perfect plastic models would lead to unrealistic high ductility), and consequently the need to perform force drive pushover analyses with displacement control.

This paper presents a simple and efficient numerical approach for the pushover analysis of masonry towers, having hollow

* Corresponding author.
E-mail address: massimiliano.bocciarelli@polimi.it (M. Bocciarelli).

http://dx.doi.org/10.1016/j.soildyn.2016.07.022
0267-7261/

arbitrary sections. It is assumed that masonry behaves as nonlinear material with dissipative and softening behavior and the structure is modeled in the framework of the Euler-Bernoulli beam theory. A specific topological algorithm is formulated in order to derive, from the three dimensional model of the tower, a bi-dimensional discretization of each section, which is then adopted to construct its moment-curvature curve. Nonlinear geometric effects and the presence of constraints, due to surrounding buildings and governed by nonlinear relationships in terms of displacement versus reaction force, are also considered. The load distribution is assigned as inverted triangular (however, the formulation can deal with any type of load distribution), and the problem is formulated in terms of monotonically increasing quantities, as primary unknowns, such as the section curvatures, in order to follow the post peak softening branch of the structural response. In classical FEM codes this type of analysis can be performed by means of arc-length type procedures, see [26]. To avoid curvature localization, induced by the masonry softening behavior, the concept of plastic hinge is introduced and the determination of its length is dealt with by comparison with classical 3D nonlinear finite element analyses.

Soil is not directly considered in the present model. However, its effect enters in the definition of the seismic demand, through proper coefficients depending on the soil constitution and defined according to seismic codes. The pounding effect, which could be one of the main causes of severe building damages during earthquake, is not considered in the present formulation.

A case study is then proposed, the Gabbia Tower in Mantua, and benchmarking pushover analyses are performed with the commercial code Abaqus [27], whose results are used to validate the numerical procedure. This example also allowed to point out the great reduction of computing time achieved with the proposed approach.

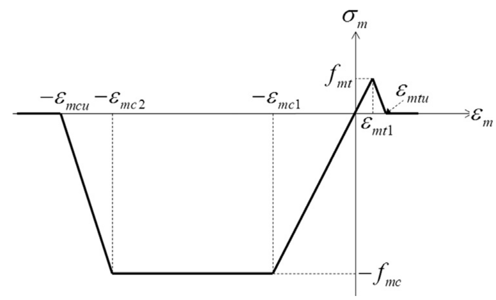

Fig. 1. Stress-strain relationship assumed for masonry.

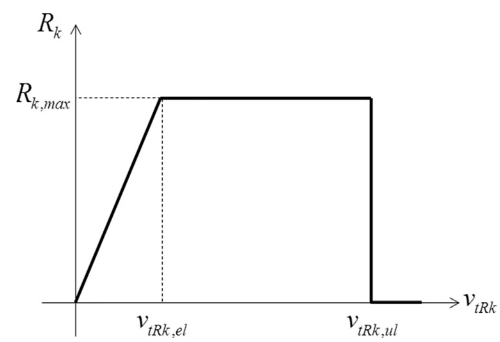

Fig. 2. Force-displacement curve governing the response of the generic k-th constrain acting on the tower.

2. Numerical procedure

2.1. Constitutive equations

Masonry in compression is modeled by an elasto-plastic stress-strain relationship with limited ductility and softening, already adopted in other studies, see [6,8,28]. The behavior under tensile stresses is assumed to be linear elastic up to the tensile strength, followed by a linear softening branch down to zero, see Eq. (1) and Fig. 1.

$$\sigma_m = \begin{cases} 0 & \text{if } \varepsilon_m \geq \varepsilon_{mtu} \\ f_{mt}\cdot(\varepsilon_{mtu}-\varepsilon_m)/(\varepsilon_{mtu}-\varepsilon_{mt1}) & \text{if } \varepsilon_{mt1} \leq \varepsilon_m < \varepsilon_{mtu} \\ E_m\varepsilon_m & \text{if } -\varepsilon_{mc1} \leq \varepsilon_m < \varepsilon_{mt1} \\ -f_{mc} & \text{if } -\varepsilon_{mc2} \leq \varepsilon_m < -\varepsilon_{mc1} \\ -f_{mc}\cdot(\varepsilon_{mcu}+\varepsilon_m)/(\varepsilon_{mcu}-\varepsilon_{mc2}) & \text{if } -\varepsilon_{mcu} \leq \varepsilon_m < -\varepsilon_{mc2} \\ 0 & \text{if } \varepsilon_m < -\varepsilon_{mcu} \end{cases} \quad (1)$$

where: E_m is the Young modulus, $\varepsilon_{mc1} = f_{mc}/E_m$ is the strain corresponding to the compressive strength f_{mc}, $\varepsilon_{mc2} = \mu_1\varepsilon_{mc1}$ is the strain at the end of the plateau, $\varepsilon_{mcu} = \mu_2\varepsilon_{mc2}$ is the ultimate compressive strain at the end of the softening branch, $\varepsilon_{mt1} = f_{mt}/E_m$ and ε_{mtu} represent the strain at the tensile peak stress f_{mt} and the ultimate tensile strain, respectively. In Eq. (1) the sign $-$ is introduced since all material properties are assumed to be positive, while stress σ_m and strain ε_m are positive if tensile and negative if compressive.

Masonry towers are frequently surrounded by other buildings, whose effect is here modeled, in a simplified manner consistent with the assumption of Euler-Bernoulli beam theory, as a series of supports acting along the axis of the tower. This approach may represent a simplification in some cases (e.g. a tower connected to a wall along only one of the edge of the tower) and fully 3D models would be needed to properly deal with these more peculiar situations.

The use of inverse analysis identification techniques, based on measurements of the dynamic behavior of the structure, has been shown to be a promising way to investigate the effectiveness of a constrain, which strictly depends on the degree of connection between the different structural elements, see [29–31].

In the proposed approach, the generic k-th restraint is modeled by an elasto-plastic curve with limited ductility, which expresses the force R_k transmitted by the support to the tower, as a function of the displacement v_{tRk} occurring in correspondence of the restrained section, see Fig. 2 and Eq. (2).

$$R_k = \begin{cases} (R_{k,\max}/v_{tRk,el})\cdot v_{tRk} & \text{if } v_{tRk} \leq v_{tRk,el} \\ R_{k,\max} & \text{if } v_{tRk,el} < v_{tRk} < v_{tRk,ul} \\ 0 & \text{if } v_{tRk,ul} \leq v_{tRk} \end{cases} \quad (2)$$

The above relationship depends on three parameters: the maximum force $R_{k,\max}$ and the elastic and ultimate displacements, $v_{tRk,el}$ and $v_{tRk,ul}$ respectively.

2.2. Curvature versus bending moment curve

According to the above hypotheses and to the assumptions in Fig. 3, the strain distribution along the section can be expressed as:

$$\varepsilon_m(y) = \eta_t - y\chi_t \quad (3)$$

where the axial deformation η_t is defined with respect to the center of gravity G of the section.

Given the axial force N_{Ed} acting on the section (due to the self-weight of the tower) and a curvature $\chi_t = \bar{\chi}_t$, the axial equilibrium is imposed as:

$$N_t(\eta_t) = \int_A \sigma_m(\varepsilon_m(\bar{\chi}_t, \eta_t))dA = N_{Ed} \quad (4)$$