Engineering Failure Analysis 47 (2015) 129–161

Contents lists available at ScienceDirect

Engineering Failure Analysis

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

Comparative pushover and limit analyses on seven masonry churches damaged by the 2012 Emilia-Romagna (Italy) seismic events: Possibilities of non-linear finite elements compared with pre-assigned failure mechanisms

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ARTICLE INFO

Article history:
Received 9 June 2014
Received in revised form 22 September 2014
Accepted 29 September 2014
Available online 22 October 2014

Keywords:
Failure analysis of masonry churches
Partial failure mechanisms
FE upper bound limit analysis approach
Pushover
Case studies

ABSTRACT

The most suitable way to perform a fast but reliable failure analysis of existing masonry churches in earthquake prone areas is investigated in order to predict the state of damage and eventually failure modes. Different failure analyses are systematically applied on a wide variety of different churches, that suffered damage during the recent 2012 Emilia-Romagna (Italy) earthquake sequence. The weaknesses of some specific parts, as for instance the tympanum, the apse and the lateral long walls of the naves, which are typically responsible for the partial collapse of the structures, are highlighted. The large set of examples considered in this study allows a comparative analysis of pros and cons linked with the practical application of the different procedures. In particular, global FE (finite element) pushover and limit analyses, combined with a plate and shell discretization, are adopted to have an insight into (a) active failure mechanisms and (b) accelerations associated with the formation of partial collapses. Results are compared with what stated by Italian Guidelines on Cultural Heritage for the safety assessment of historical masonry constructions in seismic zones. For masonry churches, it is required to analyze 28 pre-assigned failure mechanisms by means of the application of the upper bound theorem of limit analysis in presence of no-tension materials. It is found that FE limit analysis may provide reliable failure mechanisms – when compared with the other approaches discussed – but requiring a reduced processing time, without the need to adopt questionable a priori choices on the macro-blocks active at collapse.

1. Introduction

The evaluation of the seismic vulnerability of historical masonry monumental buildings [1–4] in general, and of masonry churches in particular [5–14], is a fundamental task in highly civilized countries.

This concept is especially applicable for Italy, which hosts the largest amount of monumental churches in the world and where some earthquakes, occurred in the last few decades (Umbria-Marche 1997–1998, Abruzzo 2009, Emilia-Romagna 2012), severely damaged a number of unique pieces of the architectural heritage.

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http://dx.doi.org/10.1016/j.engfailanal.2014.09.016
1350-6307/

The paper focuses on the analysis of seven masonry churches that suffered damage during the last Emilia-Romagna (2012) seismic event. The devastating 5.9 and 5.8 magnitude shakes, occurred in the north-east region of Italy called Emilia-Romagna respectively on the 20th and 29th of May 2012, caused 28 victims and several collapses of precast concrete and masonry structures [15–17].

It has been recently calculated that, only in the provinces of Modena, Ferrara and Bologna, the churches considered unsafe after the seismic sequence are more than 500.

Generally, all existing masonry structures are rather vulnerable to earthquakes, but churches in particular are not conceived to properly withstand horizontal loads. It is well known, indeed, that churches exhibit partial collapses at very low levels of horizontal acceleration. The reasons justifying such a high vulnerability are in very specific architectonical features, i.e. high and slender perimeter walls scarcely interconnected, long and wide naves carried by slender columns, quite poor masonry quality and presence of flexible wooden roofs [3–14].

The post-earthquake problems still open are (1) to evaluate the most suitable strategy of rehabilitation, limiting times and costs, and (2) to have a quantitative information on the residual resistance for the un-collapsed structures [18–20].

Both previous tasks are rather specific and hard to be tackled, because churches cannot be reduced to any standard static scheme. At present, the most diffused approach adopted in practice, as recommended by Italian Guidelines for the Cultural Heritage [21], is the utilization of the kinematic theorem of limit analysis for no-tension materials, with pre-assigned partial failure mechanisms.

Twenty-eight possible collapse mechanisms are established as the most probable according to their statistical occurrence observed in previous earthquake surveys. Seismic vulnerability considerations are therefore linked to a preliminary identification of the active failure mechanism which corresponds to the lowest value of the collapse multiplier.

Whilst the approach proposed by Italian Guidelines is very straightforward, easily applicable by everyone, even not familiar with limit analysis concepts, it has two rather relevant drawbacks. The first is linked to the risk of an overestimation of the horizontal acceleration at failure, because the upper bound theorem of limit analysis is used. The second is related to the utilization of a no-tension material model. As a matter of fact, such assumption is a safe one, but does not take into account some key features playing an important role in the formation of the failure patterns. Among the others, the most important features are the orthotropy at failure and the actual texture, especially along the thickness of the wall. This latter feature considerably influences the monolithic behavior against out-of-plane loads.

In the present paper, with the aim of estimating the ultimate load bearing capacity of the churches, three different approaches are critically compared and systematically applied to the case studies considered.

The first procedure is a global pushover analysis carried out with the commercial code Strand7 [22], where an isotropic elastic-perfectly plastic material model, obeying a Mohr–Coulomb failure criterion, is adopted for masonry.

While the FE pushover model is global, the reproduction of partial failure mechanisms is obviously still possible [7,8], especially when the in-plane stiffness of the floors and the roof is not accounted for.

The second procedure consists of the estimation of the collapse loads by means of pre-assigned failure mechanisms, as per Italian Guidelines on built heritage requirements. A poor interlocking between perpendicular walls is assumed, in agreement with in-situ surveys, with a monolithic behavior along the thickness. This latter hypothesis is justified by the regular texture of the churches, with walls constituted by multi-head clay bricks, with good through-thickness interconnections secured by transversal blocks (diatoni) regularly present.

The major practical drawback of pre-assigned failure mechanisms is the impossibility to automatically interface with previous FE models of the structure, always required when response spectrum analyses are performed.

The third approach supersedes the aforementioned limitations and relies on FE upper bound limit analyses performed on the entire structure. The approach assumes, for the limit analyses, the same FE discretization previously utilized by standard elastic analyses. The code requires a discretization with rigid triangular elements, with possible dissipation for in- and out-of-plane loads on interfaces between adjoining elements. Mechanical properties of the interfaces may be derived either from homogenization procedures, once the actual geometry and the mechanical properties of the constituent materials are known, or using code of practice prescriptions. The proposed approach works easily with relatively refined discretizations and automatically allows the identification of the active failure mechanism. However, still, some drawbacks remain, as for instance the inability to provide clear crack patterns or damage maps (due to the nature of the model utilized, which is rigid-plastic).

All models are tested on seven masonry churches, exhibiting different geometrical features. Detailed comparisons among the active failure mechanisms provided by all models and the corresponding collapse multipliers are discussed.

It is found that, in the majority of the cases, the three approaches provide similar active failure mechanisms and failure accelerations. However, pushover analyses are generally very demanding, requiring much more time to be performed. The procedure based on the assumption of partial failure mechanisms is certainly the most immediate in the processing phase, because it is the trivial application of the principle of virtual works. However, it requires a detailed preparatory work to transfer data between the idealized scheme of the code and the real cases, which have been managed with the help of a 2D CAD software. In addition, it involves some rough assumptions on the acting loads, totally or partially neglecting some effects induced by arches, vaults, roofs, etc. The collapse loads so predicted are always very conservative and much lower when compared with alternative procedures, clearly demonstrating the intrinsic limitations of the approach.

Finally, the third approach (FE limit analysis) always proved robustness, quickly providing active failure mechanisms in acceptable agreement with both alternative procedures and post-earthquake surveys.