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Failure analysis of seven masonry churches severely damaged during the 2012 Emilia-Romagna (Italy) earthquake: Non-linear dynamic analyses vs conventional static approaches

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ABSTRACT

This study presents a detailed failure analysis of seven masonry churches, which were severely damaged during the 2012 Emilia-Romagna (Italy) seismic sequence, by means of 3D FE non-linear dynamic simulations. The main aims of the numerical investigations are: (1) to carry out a sufficiently wide sensitivity study on different specific case studies to have an insight into the role played by the geometry – which is always unique for churches – and by the irregularities; (2) to validate or address the limits of applicability of the most widespread conventional static approaches recommended by Italian Code, i.e. kinematic limit and pushover analyses. Non-linear dynamic analyses are carried out assuming that masonry behaves as a non-linear material exhibiting softening and damage, frictional behavior and different strength in tension and compression. The set of case studies investigated in this work shows how conventional static approaches are still capable of roughly identifying the most critical macro-elements that usually activate a failure mechanism, but that the results (e.g. collapse acceleration, behavior factor) are affected by a level of approximation that may considerably depend on in-plan irregularity and hypotheses done on the interlocking between contiguous walls. From the comparative analyses carried out, it is authors' opinion that it is always beneficial to perform different types of analysis in order to have a comprehensive insight into the portions of the structure that can suffer a partial collapse with high probability. Once the active mechanism is identified, non-linear dynamic analyses, with more sophisticated material models and with refined FE discretizations of the critical regions, may be useful to deepen the knowledge of the behavior of such complex structures under seismic actions.

1. Introduction

It is estimated there are more than 64,000 churches in Italy. The majority is classified into the group of "monumental historical structures", is masonry construction and is located in high-seismicity regions. It has been recently calculated that, after the devastating 5.9 and 5.8 magnitude earthquakes occurred in Emilia-Romagna respectively on 20th and 29th of May 2012, the churches considered unsafe – i.e. with an induced level of damage that prevents their utilization – have been more than 500 only in the provinces of Modena, Ferrara and Bologna. After more than two years from the seismic sequence, a

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great part of the damaged churches is still awaiting for seismic rehabilitation and suitable strengthening to improve their performance under horizontal loads.

As well known, churches are not conceived to properly withstand earthquakes [1–11] and post-seismic surveys have demonstrated that the activation of partial collapses [12] is critical even at very low levels of horizontal acceleration. 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 [13–17] promote the activation of mechanisms involving few macro-blocks into local failures [18–27].

Unfortunately, churches cannot be reduced to any standard static scheme [18–27]. Italian Guidelines for the Cultural Heritage [28] help practitioners in the safety assessment. To evaluate the acceleration at collapse, they suggest a quite rough and conventional approach based on pre-assigned partial failure mechanisms and the utilization of the kinematic theorem of limit analysis within the assumption of a no-tension material model for masonry. An abacus of twenty-eight possible collapse mechanisms, based on experience of failures observed during past earthquakes, is provided.

Despite the simplicity of such an approach, however, some drawbacks are quite evident, as for instance the risk to overestimate the horizontal acceleration at failure due to the identifications of wrong failure mechanisms and the rough simplifications introduced in the actual geometry of the case under study.

To overcome such limitations, a FE upper bound limit analysis procedure has been recently applied successfully by one of the authors of this paper on a variety of different examples, including many churches in seismic regions [18–20].

An alternative to limit analysis, which however still remains scarcely available in commercial codes (even if some exceptions, basically conceived for either geotechnical problems [29] or specialized to masonry arches [30], are present in the literature), is represented by pushover analysis. Italian Guidelines on the Built Heritage [28] implicitly state that the equivalent frame approximation cannot be used for existing buildings with peculiar geometries, as for instance castles, churches and towers, but at the same time suggest to perform a conventional static analysis also with simplified materials where softening is not included. As well known, indeed, non-linear static analyses, conducted when the global behavior beyond the maximum load carrying capacity is required, cannot be easily obtained using refined FE discretizations with many 2D and 3D elements, as needed for a realistic analysis of masonry churches. In addition, complex material models are not generally implemented in the majority of the commercial codes available for standard design. Several commercial software packages usually put at disposal elastic-perfectly plastic models, with isotropic behavior, friction (either Mohr–Coulomb or Drucker–Prager strength domains) and associated flow rule.

In this context, it appears rather clear that, despite the level of complexity of the FE analysis which still remains almost prohibitive for a great part of the involved practitioners, the limits of applicability of a non-linear static approach should be deeply investigated and considered with care.

One of the main purposes of the paper is to quantitatively analyze if the utilization of simplified material models for masonry and the application of horizontal loads according to static approaches are anyway acceptable and, if so, what is the level of accuracy and approximation of the results obtained.

To validate the effectiveness of such simplified procedures, as suggested by Italian Code, in the present paper time-consuming non-linear dynamic analyses are performed, using detailed FE discretizations of the same seven churches analyzed in [11]. An isotropic non-linear material exhibiting softening and damage, frictional behavior and different strength in tension and compression is used for masonry.

The present numerical investigations may therefore be regarded as the natural sequel of previously presented [11] static and simplified approaches, where the novelty is represented by the application of non-linear dynamic analyses to quite complex geometries. Attention is mainly focused on the role played by the geometric variability (especially in plan) of the different case studies analyzed, with particular regard to geometric irregularities and interlocking between perpendicular walls.

Furthermore, it is emphasized that 14 non-linear dynamic analyses (carried out applying the same accelerogram, separately, along the longitudinal and transversal directions for each church) are performed on rather demanding FE models, each one requiring up to 6 days to be processed on workstations equipped with large RAM. While such computational effort is obviously impossible to manage in common design, the study should be regarded as a reference in the field, because it puts in evidence – by means of the sensitivity analysis conducted discussing the role played by the different geometries – how effective can be a simplified approach based on the a-priori assumption of a given failure mechanism or a standard elastic response spectrum procedure.

Kinematic FE limit analyses predictions (which provide similar results to the ones obtained by FE 3D pushover approaches, as demonstrated in [11]) are systematically compared with non-linear dynamic analyses performed on the aforementioned seven masonry churches, exhibiting different geometrical features. Detailed comparisons among the active failure mechanisms provided by both limit analysis and FE non-linear dynamic approach are reported. It is found that, in the majority of the cases, the active failure mechanism is identified correctly, but that the results (e.g. collapse acceleration, behavior factor) are affected by a level of approximation that may considerably depend on in-plan irregularity and hypotheses done on the interlocking between contiguous walls. From a quantitative point of view, a non-linear dynamic analysis is able to provide – among other information – residual displacements of the control nodes. By using these data, practitioners should decide, on the basis of their own experience or consolidated models, if such values are compatible or not with the equilibrium condition of the macro-block where the failure mechanism is acting. If not, it means the accelerogram applied causes a partial failure of the structure. It is also interesting to investigate, as done in this paper, such a result is in agreement with what provided by a standard eigen-frequency analysis associated with the response spectrum related to the applied