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Seismic Hazard and Nonlinear Dynamic Analyses: Avoiding Collapse in Architectural Heritage

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Abstract Architectural heritage masonry buildings located in seismic areas are highly prone to suffer damage under seismic loading, due to their structural features. In this study, the Salares medieval tower, which is placed in the province of Malaga (Spain) - a zone with high seismic hazard in the Iberian Peninsula- is adopted as a case study. This is a precious heritage building which exhibits severe damages, such as depth cracks and inclination. Owing to those reasons detailed seismic and dynamic analyses are crucial for its preservation. The present work is arranged in three sections that deal with seismic hazard analysis, selection of real acceleration time-histories representative of the site, and dynamic response by means of numerical models. In order to obtain the design earthquake at the specific location, different parameters were analyzed, namely the region seismic activity, the geotechnical and geological conditions at the site and the acceptable risk level. Hazard analysis was performed following deterministic and probabilistic methods. As far as the seismic input is concerned, real accelerogram representative of the 1884 Andalusia earthquake were selected. It was a destructive earthquake that caused catastrophic damages near the analyzed structure. The accelerograms thus obtained were used as input for subsequent non-linear dynamic analyses on three-dimensional finite element models. Different cracking capable constitutive models were considered in order to predict local and global collapse mechanisms. Moreover, the suitability of each model was discussed when crushing, high plastic deformation or response under cyclic loading are concerned. The aforementioned studies yield significant results in order to perform a comprehensive safety assessment under ground motion effects. Thus, regarding the seismic input, the obtained accelerogram provides a relevant application aimed at revising the maximum acceleration and the response spectra of the Codes, as seismic code recommendations are limited in those issues. Furthermore, the dynamic structural response via numerical approach, allows determining damage propagation and collapse probability as a main step to select appropriate repair measures. The followed method could be used to assess and improve the structural strength of similar architectural heritage buildings located in active seismic areas.

Keywords: Hazard analysis, non-linear seismic assessment, ancient masonry structures

Introduction

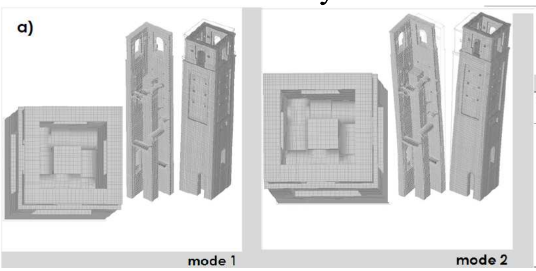
Seismic assessment of ancient masonry constructions is a complex task, due to uncertainties that mainly affect structural behaviour, mechanical material properties and suitability of the analytical or numerical method applied. Moreover, if a rigorous demand prediction for inelastic time-history analysis is needed, the selection of accelerograms that accurately represent the seismic hazard is a crucial issue. The analysis method proposed to overcome the aforementioned issues follows these steps: i) the static response of the structure is analysed in the non-linear range via Finite Element Method, and subsequent eigenvalue analysis is undertaken. Thus, structural weakness and dynamic properties are determined; ii) a hazard analysis is performed in order to define the reference seismic scenario; iii) the most adequate real accelerograms to be used as seismic input are selected; iv) a non-linear seismic assessment of the structural response under different constitutive models is performed. That procedure, which is applied in a single case study, the Salares tower, allows overcoming several problems and uncertainties that are inherent to structural seismic analyses.

Preliminary Survey of the Tower

General Description The Salares tower is a XIIIth century construction located in Salares, in the province of Málaga, Spain. Although it was built as a minaret, it was turned into the bell-tower of the Santa Ana *Mudéjar* church (Fig. 1e). Andalusian minarets are unique in the Islamic world due to its fidelity to ancient Syrian constructions and owing to its variegated stylistic and constructive influences. Furthermore, those buildings were essential in the evolution of Christian and Islamic architectural styles. Regarding its geometrical and structural features, the tower is of medium size, 3.30 x 3.30 m^2 in lower plan, and it rises 17 m above the current ground level. Morphologically, the minaret is divisible into three structural parts: external walls, central core and barrel vaults. Average thickness of walls is 0.55 m, core cross-section is 0.90 x 0.90 m^2 and barrel vaults cross section is 0.15 m. The inner chamber consists of an anti-clockwise staircase covered by horizontal barrel vaults, which ascend around the square central solid core. The whole structure is built of clay bricks and irregular stones bonded with lime mortar, which were originally covered with painted plaster. Outside walls and core are directly embedded in the soil, acting as foundation. Severe damages are observed, such as: (i) irregular mortar-brick disposition; (ii) no bonded masonry; (iii) poor quality of units and mortar; (iv) moistures; (v) loss of material; (vi) a variegated crack pattern – diffused, thin and passing-through cracks-; (vii) local detachment of the outer wythe, and (viii) local reconstruction with different materials, -stones or poor quality bricks, (Figs. 1a,1b,1c,1d). Furthermore, the tower leans at about 1 degree on the southeast side, and measurements suggest that suffered a rigid body tilting. Prior research on the analyzed structure (Pineda and Sáez 2010) revealed that the causes of the mechanical deterioration and tilt phenomena were previous seismic events and foundation settlement.

Figure 1: most significant damages and general view (a) view of the NE-NW inside corner and the core: spalling and loss of material; (b) wooden roof detail;(c) passing-through crack at southwestern wall; (d) perforated vault;(e) north-eastern and north-western façades

Preliminary Numerical Studies In order to select the ground-motion accelerations which provide a best prediction on the inelastic structural response, nonlinear static and eigenvalue analyses are performed on a detailed 3D Finite Element model, replicating the actual state of conservation. Moreover, as the structural response under seismic loading will depend on the orientation of the structural axis system relative to the ground-motion axes, it is crucial to identify the stronger and weaker structural directions. Damage is directly introduced in the model by vanishing or adjusting the masonry stiffness in the affected areas. ANSYS finite element software is used to develop the model. The 3-D eight-noded solid isoparametric element, SOLID 65, is employed to model the brittle material. The mesh consists of 53,970 hexahedric elements with an average element size of 0.11m. The Drucker-Prager perfectly plastic criterion and the Willam-Warncke failure surface are applied. Those criteria provide neither stiffness degradation of brittle material caused by successive plastic deformation, nor cracks resulting from low cycle fatigue. Nevertheless, both theories yield accurate results on 3-D solid models, in particular when predicted cracking progression is concerned. The stress-strain matrix is adjusted by introducing a plane of weakness in a direction normal to the crack face. A comprehensive description on physical material properties and material calibrating parameters can be found in a previous survey (Pineda and Sáez 2010). The static analysis yields significant data, such as stress distribution, weak elements of potential failure and displacements. Maximum compression level -0.528 MPa- is reached by the basement at the SE façade, whereas maximum tensile stresses –0.19 MPa– , which are higher than tensile strength, appeared in the connections among structural vaults and central core. Those results allow concluding that connections between vaults and walls are confirmed to be one of the most vulnerable parts of the building. Owing to the geometrical features of the tower, the construction presents asymmetric displacements, with a movement towards the southeaster direction. The maximum horizontal displacement, parallel to the SW and NE façades, is 0.002 m. A modal analysis was computed on the aforementioned detailed model, in order to obtain the dynamic properties –natural frequencies, ω_n, and modal shapes, ζ_n - and to serve as a starting point for the transient dynamic surveys, (Figs. 2a, 2b). The first five modal shapes provide the highest mass contribution in transversal directions, and all of them involve global bending. Those shapes are characterized by a high global stiffness and a monolithic behaviour among vaults, central core and walls. The third modal shape displays torsional response, but with a light mass contribution, and when higher shapes are analyzed, weak collaboration among the different structural parts is revealed, and significant out-of-plane deformations are observed. At higher frequencies, the upper parts of the tower are extremely vulnerable.



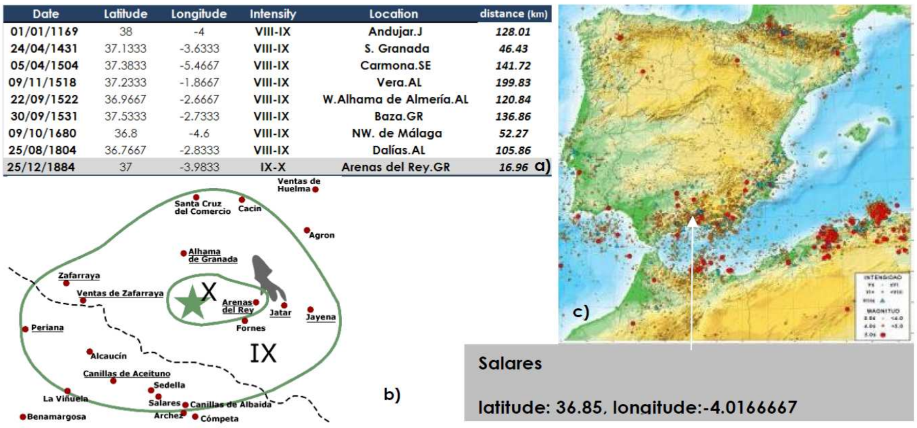
b)

main periods and effective masses "as is" model							
mode No	Period T (s)	x-direction		y-direction		z-direction	
		M eff (%)	Σ M eff (%)	M eff (%)	Σ M eff (%)	M eff (%)	Σ M eff (%)
1	**0.506**	38.213	38.213	21.950	21.950	0.004	0.004
2	**0.498**	21.990	60.202	38.082	60.033	0.000	0.005
4	**0.123**	5.051	66.188	17.179	78.468	0.007	0.012
5	**0.122**	15.178	81.366	4.766	83.234	0.008	0.020
7	**0.076**	0.011	81.382	0.010	83.255	77.035	77.243
8	**0.063**	0.389	81.771	4.341	87.596	0.001	77.244
9	**0.060**	4.602	86.373	0.571	88.168	0.005	77.248
12	**0.052**	0.044	86.490	0.001	88.692	1.535	79.285
20	**0.037**	0.116	88.620	1.218	90.966	0.000	79.513

Figure 2: modal analysis results (a) modal shapes; (b) main periods and effective masses

Hazard Analysis

The reference seismic scenario is defined by performing a Deterministic Seismic Hazard Analysis (DSHA) and a Probabilistic Seismic Hazard Analysis (PSHA) for the area of Salares (Fig. 3c). Following this procedure, the Design Earthquake is completely defined, in terms of magnitude, distance and nature of surface geology. In order to estimate the potential areas from where the most destructive earthquake may emanate, the historical seismicity of the region is studied. Thus, the major historical strong-motions are identified (Fig. 3a) and located in their seismotectonic sources (Martín 1984). The earthquake scenario that dominates the seismic hazard at Salares is a Moment Magnitude 6.3 earthquake, with an Intensity of IX (Fig. 3b) and a Joyner-Boore distance of 15 km. That strong-motion is compatible with the 1884 Arenas del Rey earthquake, also known as the Andalusian Earthquake. It was a destructive earthquake that caused catastrophic damages near the analyzed structure - 690 deaths and 1,426 injuries in the Granada province and 55 deaths and 59 injuries in the province of Málaga-. Regarding geological features, Salares is located in the Alpujarride complex, which is characterized by medium to hide-grade rocks. The upper units include periodotite sills. The site is classified as NEHRP class C, with an average shear wave velocity to a depth of 30 m, v_s30, between 360 m/s and 760 m/s. The most adequate target spectrum that is representative of that seismic event is analyzed (Fig. 4), following a near-field attenuation relation (Ambraseys and Douglas 2003) and a model representative of Europe (Ambraseys et al.1996). The best fit with respect to spectral acceleration content of the 5% damping elastic response spectrum of Eurocode 8 is also considered. The period range of interest of this matching includes all frequencies that are expected to significantly contribute to the non-linear response of the tower, -from 0.1 s to 0.5 s-.



Date	Latitude	Longitude	Intensity	Location	distance (km)
01/01/1169	38	-4	VIII-IX	Andujar.J	128.01
24/04/1431	37.1333	-3.6333	VIII-IX	S. Granada	46.43
05/04/1504	37.3833	-5.4667	VIII-IX	Carmona.SE	141.72
09/11/1518	37.2333	-1.8667	VIII-IX	Vera.AL	199.83
22/09/1522	36.9667	-2.6667	VIII-IX	W.Alhama de Almería.AL	120.84
30/09/1531	37.5333	-2.7333	VIII-IX	Baza.GR	136.86
09/10/1680	36.8	-4.6	VIII-IX	NW. de Málaga	52.27
25/08/1804	36.7667	-2.8333	VIII-IX	Dalías.AL	105.86
25/12/1884	37	-3.9833	IX-X	Arenas del Rey.GR	16.96

Figure 3: (a) major historical seismic events in the region; (b) intensity distribution during the 1884 Andalusia Earthquake, source: Instituto Geofísico de Andalucía; (c) seismicity of the Iberian Peninsula, source: Instituto Geográfico Nacional

As far as attenuation of peak ground acceleration [PGA] is concerned, 0.21g and 0.25g are obtained for the aforementioned relations. It is worth noting that those values are higher than the PGA proposed by the Spanish Code NCSE-02 for this area, -0.21g-. Finally, the European attenuation relation is adopted.

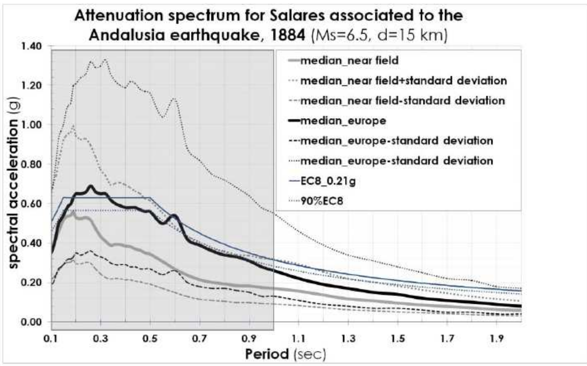


Figure 4: attenuation spectra for Salares associated to the 1884 Andalusia earthquake according to near field relationship proposed by Ambraseys and Douglas (2003) and horizontal response spectrum in Europe proposed by Ambraseys et al (1996)

Selection of Records

An earthquake scenario-based selection is performed; therefore, records that fall in bins around central values of seismic parameters are selected. Those selection parameters are Moment Magnitude, M_w, Joyner-Boore distance, d_{JB}, and site class. Thus, main strong-motion characteristics such as frequency content, spectral amplitudes, spectral shape, and duration – which are related to magnitude, distance, and site class-, are taken into account. If near-source effects are expected, M_w and d_{jb} are crucial parameters. The adopted intervals are [6, 6.6] for the M_w, [0 km, 30 km] for the d_{JB} and [360 m/s, 760 m/s] for the v_s30. Records are selected from a subset of 102 strong-motions of the PEER database, (Fig. 5a). As a 3D input will be applied, only records with two horizontal components and vertical component are considered. Moreover, aftershocks were excluded, and records were corrected through baseline correction and frequency filtering. The dynamic properties of the structure were also considered in the selection. Thus, the period range of interest for matching target spectrum and scaled record spectra is $0.2T_1$–$2T_1$, where T_1 is the fundamental period of the tower, equal to 0.5 s. For each record, the geometric mean of the spectral ordinate of the two horizontal components was used. Pre-selected records were scaled in terms of spectral acceleration for the fundamental period, S_a

(T₁,5%), as shown in Fig. 5b. Thus, neither frequency content nor duration of the earthquake are modified. Furthermore, as the Salares tower is a first-mode dominated structure, it is an efficient technique for its safety assessment. After scaling of the records according to spectral matching, a screening aimed at obtaining records with spectral shapes similar to the target spectrum is undertaken. A root-mean-square difference equation, D_{rms}, (Ambraseys et al. 2004) between the spectrum of each real record and the reference spectrum is used. After performing a Coefficient of Variation comparison, (Fig. 5c), three records which exhibit the best goodness-of-fit are selected, Fig. 5d. Finally, the accelerograms are scaled to match a peak ground acceleration of 0.25g (Fig. 6).

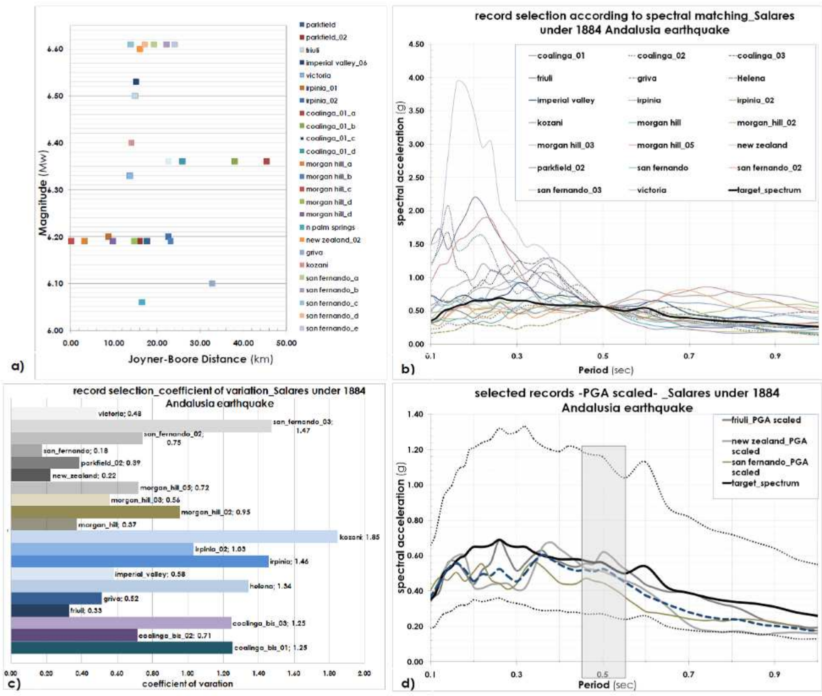
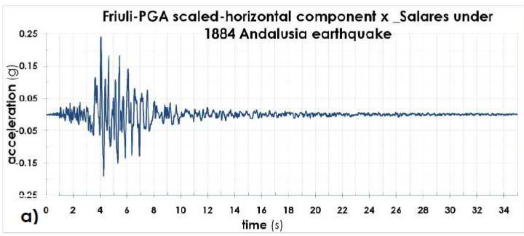


Figure 5: (a) distribution of records with respect to M_w and d_{JB};(b) record selection according to spectral matching;(c)coefficient of variation; (d)comparison among spectra



Earthquake Name	Earthquake magnitude Mw	Hypocenter depth (km)	Epicenter distance (km)	Joyner-Boore distance (km)
san fernando	6.61	13	23.56	19.33
friuli_Italy	6.5	5.1	20.23	14.97
new zealand	6.6	6.4	24.23	16.09
	Vs30 (m/s)	PGA (g)	PGV (cm/sec)	PGD (cm)
san fernando	450.28	0.30	19.83	3.29
friuli_Italy	424.8	0.35	25.06	4.54
new zealand	424.8	0.29	21.07	4.60

b)

Figure 6: (a) scaled horizontal x component Friuli earthquake;(b)non- scaled earthquake properties

Seismic Structural Assessment

Simplified 3D beam model and the aforementioned detailed model are used to predict the nonlinear seismic response. Modelling of seismic action is achieved by introducing three simultaneously acting components of the selected accelerograms at the basement. The most severe component is applied at the most vulnerable structural axis – perpendicular to the SE outside wall-. A Rayleigh model for damping is considered, and the two main modes T_1 and T_2 are assumed to have the same damping ratio equal to 3%. Regarding the simplified model, cantilever beams, comprised of the central solid core, were modelled in order to obtain an upper limit response. Modelling of non-linear material behaviour is achieved by means of a uniaxial nonlinear constant confinement model and a uniaxial trilinear model. The former follows the cyclic rules proposed by Martinez-Rueda and Elnashai. This approach allows coping with the structural response under cyclic loading, taking into account inelastic strain and degradation of strength and stiffness. The latter assumes no resistance to tension and features a residual strength plateau. The specific model calibrating parameters are described in a previous work (Pineda and Sáez 2010). The simplified model provides relevant information when global collapse mechanisms such as the tower over-turning are studied. After analyzing stresses and displacements, it may be concluded that no over-turning is expected. Moreover, those maximum horizontal time-history displacements are compared with that of the first constitutive model. Thus, the maximum displacement reached by the core with the Drucker-Prager criterion is 0.06 m, whereas the simplified models yield a maximum result of 0.075 m, (Fig. 7a). The 3D FE detailed model show that the structure leads to an extremely high cracking pattern in vaults connections, and basement –at NW outside wall- . The maximum compressive stress at the basement – 1.5 MPa- is beyond the compressive strength, (Fig. 7c), and the shear levels at the NE façade -0.7 MPa- are extremely high The connections of the barrel vaults with the inside walls and the central core, doors and SW and SE façades show signs of weakness. From those transient studies, it may be stated that the tower is considered to be in dangerous condition under this seismic hazard scenario.

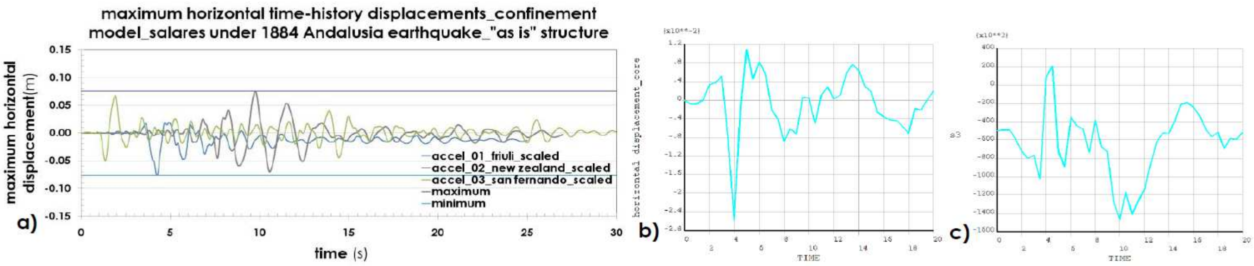


Figure 7: (a) horizontal displacement for the confinement model;(b)horizontal displacement of the core in the detailed model;(c)maximum compressive stresses

Conclusion

The Salares tower is analysed in order to assess its structural behaviour under seismic loading. A number of 3D non-linear Finite Element Analyses with different constitutive models and different complexity levels are performed. As the structural behaviour of this construction exhibits a predominant non-linear phase, the suitability of the record selection procedure is a crucial issue. The proposed method, earthquake scenario -based selection and spectral matching, allows capturing the main seismic and site parameters that significantly affect the inelastic response of complex systems, such as ancient masonry structures. Under the defined seismic events, the building exhibits severe damages and collapse is expected. The proposed method could serve as guidance in order to assess the safety level of similar architectural heritage.

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