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Seismic Vulnerability of the Concordia Temple

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Abstract This work focuses on the seismic risk evaluation of the Concordia temple, situated in the Valley of the Temples in Agrigento (Italy). In the paper a general methodology to assess the seismic vulnerability, to be applied also to any kind of structures composed of stone blocks, is proposed. The vulnerability assessment has been conducted by means of equivalent nonlinear static analyses along the principal directions of the structure and the subsequent identification of equivalent single degree of freedom systems. Furthermore, the seismic vulnerability has been expressed both in a deterministic and a probabilistic context by evaluating the severe damage probability.

Keywords: Archaeological heritage, Greek temple, stone block masonry, seismic vulnerability, severe damage probability

Introduction

In the last years a strong need concerning the seismic risk protection of archaeological heritage, respectful of the purpose of the original design, has been established. The work reported in this paper aims at providing a contribution towards modelling methods for the seismic risk evaluation and mitigation of monumental buildings (Lagomarsino 1999, Giordano et al. 2002, Calio et al. 2008) and of particular archaeological structures. In particular, the work focuses on the Concordia temple, situated in the Valley of the Temples in Agrigento (Italy). In the paper a general methodology to assess the seismic vulnerability, to be applied also to any kind of structures composed of stone blocks, is proposed. More precisely, the blocks have been given their real geometry, and have been considered as elastic three-dimensional elements separated by nonlinear interfaces. The interfaces have been modelled, according to a fibre model representation, by means of a discrete number of monolateral contacts allowing for the relative rotation between adjacent blocks.

The case study can be recognised as representative of a wide class of Greek temples showing geometric and constructive similarities. The vulnerability assessment has been conducted by means of equivalent nonlinear static analyses, conducted in the SAP2000 environment, along the principal directions of the structure and the subsequent identification of equivalent single degree of freedom systems. Furthermore the seismic vulnerability has been expressed both in a deterministic and a probabilistic context, the first by means of the evaluation of risk coefficient and the latter by evaluating the severe damage probability.

The Concordia Temple: Historical Description and Architectural Features

The Concordia temple, realised on a basement lying on rocky soil, in view of its conservation state is considered one of the most notable among the sacred constructions of the classical epoch in the Greek world. The Concordia temple was built around the 430 b.C., it shows an architecture rather diffused, for that period, all over the classical world, and it was the last Greek monumental building erected in Italy. The Concordia temple belongs to the Doric style, the most ancient among the Greek architectural styles. It shows a 20x42 m rectangular plan, as depicted in Fig. 1, and a height up to 13,50 m. The temple is surrounded by a colonnade (*peristasi*), erected on a four step basement (*krepidoma*), consisting of 6x13 columns, each one composed of four drums and showing a 6,75 m height, 1,60 m and 1,20 m diameter at the bottom and the top, respectively.

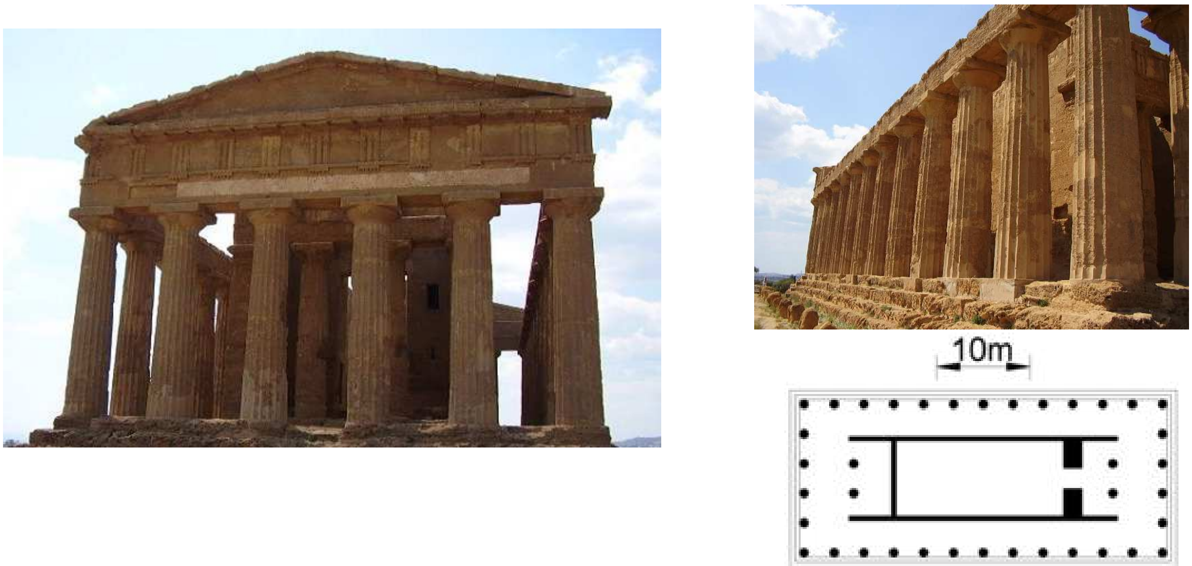

Figure 1: Est façade, North colonnade and plan of the Concordia Temple

At the top of the columns there is a capital consisting of a first circular element (*echino*), narrowing at the bottom where it joints to the top of the column, and of a quadrangular element (*àbaco*) (Fig. 1). Currently, the lintel structure is complete along the temple perimeter, while the decoration appears only on one of the two façades and it is still surmounted by the two pediments which are well preserved. Around the VI century a.C., the temple underwent an invasive modification aiming at utilising the temple as a church. The intervention consisted in the demolition of the back wall of the internal cell and in the realisation of additional walls with arches along the perimeter so that the classical three aisles can be identified. The lateral aisles in the '*peristasi*' and the nave coincident with the cell.

The Structural Model

The most efficacious model for the real structure in the modelling consists in a discrete system where the blocks are given the real dimensions and the contact between blocks is explicitly modelled. A global tridimensional model of the temple is here proposed. In the structural model, the internal cell disconnected by the colonnade, in view also of the presence of the seismic resistant shear walls, has been excluded.

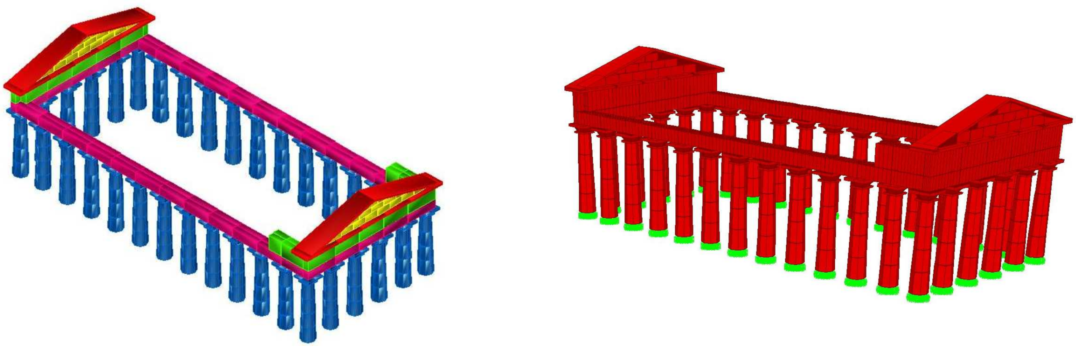

Figure 2: Models in the CAD and SAP environments

The modelling has been developed according to two phases. First, a tridimensional geometric model, accounting for the detailed definition of each block has been realised in a CAD environment (Fig. 2). Then, the model has been imported into the SAP environment providing each element with the relevant loads and the constitutive elastic properties. The model has been developed by utilising tridimensional elastic elements for the blocks. Each block interacts with the adjacent one by means of an interface consisting in a discrete layer of monolateral contacts. Simplifying hypotheses have been introduced in the structural model: the blocks have been considered with a linear elastic behaviour and the nonlinearities have been concentrated at the contact layers between adjacent blocks, precisely by assigning a '*no-tension*' and rigid compression behaviour (coupled with an infinite strenght). Sliding between adjacent blocks has been inhibited, however development of a model accounting for friction sliding will be the object of future work.

Numerical Simulations

Nonlinear static (*push-over*) analyses have been conducted by means of the presented model for both the transversal and the longitudinal direction by considering a horizontal load equivalent to the seismic action according to the mass distribution. The analyses for both directions have been conducted by accounting for P-δ effects in view of the slenderness of the columns. The results are reported in terms of both deformed configuration correspondent to collapse (Fig. 3) and capacity curve expressed by means of the shear coefficient as function of the target displacement at the top of the pediment (Fig. 4).

Characteristics of Materials The characteristics of the materials to be assigned to the model are those relative to the solid elements. Based on the available information, the following values have been considered for the blocks: mass per unit volume $\gamma = 1.750$ [Kg/m^3], Young modulus E=150.000 [kg/cm^2], Poisson coefficient $\nu = 0{,}15$. The total weight of the structure has been estimated as 12.385,15 kN.

Seismic Action along the Transversal Direction The maximum displacement resulting from the push-over curve in the transversal direction, depicted in Fig.4, is 21,09 cm and corresponds to a bottom shear force 1.889 KN and a shear coefficient C_b=0,15336. The collapse mechanism consists in the rotation of the columns with a consequent translation of the pediment (Fig. 3). However, the mechanism is not fully symmetric since a different behaviour of the two façades can be inferred. The latter difference is due to the better conservation state of a façade with respect to the other, more precisely, characterised by a better connection of the lintel with the corner column. In fact, for the mentioned façade, the blocks of the pediment continue for a short interval along the longitudinal colonnade ensuring a greater stiffness and strength with regard to the transversal actions.

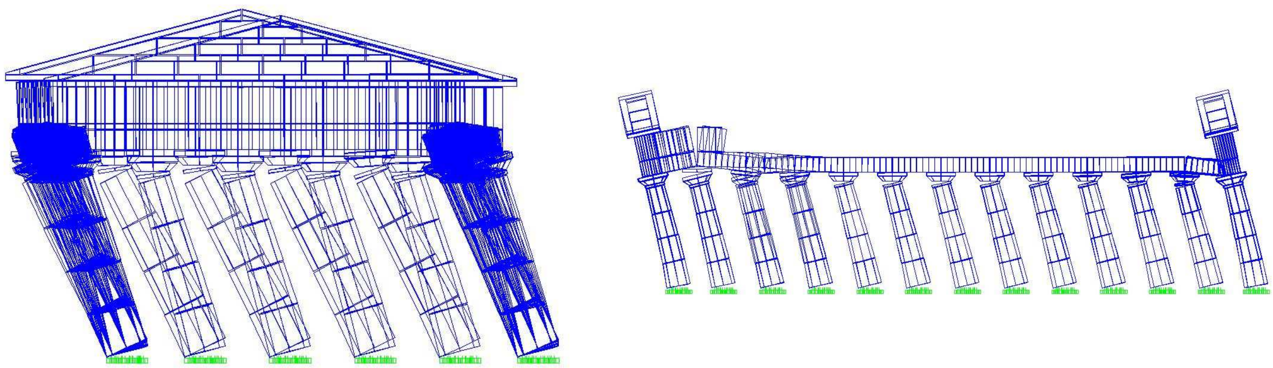

Figure 3: Collapse configurations (transversal and longitudinal directions)

The results show that the collapse mechanism involves the entire colonnade. Hence, the weak element is confirmed to be the column due to its high slenderness and to the mass to be sustained. The element above the columns being non-structural elements.

Seismic Action along the Longitudinal Direction The maximum displacement reached by the push-over curve, depicted in Fig.4, is 10,01 cm, correspondent to a bottom shear force 1.953 KN and to a shear coefficient C_b=0,1577. Analogously to the transversal direction, the collapse mechanism enhances the weakness of the columns and the non-structural role of the pediments (Fig. 3). However, rigid rotations of the pediments, which follow the rotation of the columns, lead in this case to a lower ductility due to the P-δ effects. The collapse mechanism along the longitudinal direction is symmetric.

Seismic Vulnerability Assessment

Once the push-over curves have been obtained, the assessment of the seismic vulnerability has been conducted by means of the definition of a single degree of freedom (SDOF) system equivalent to the structural model for each direction.

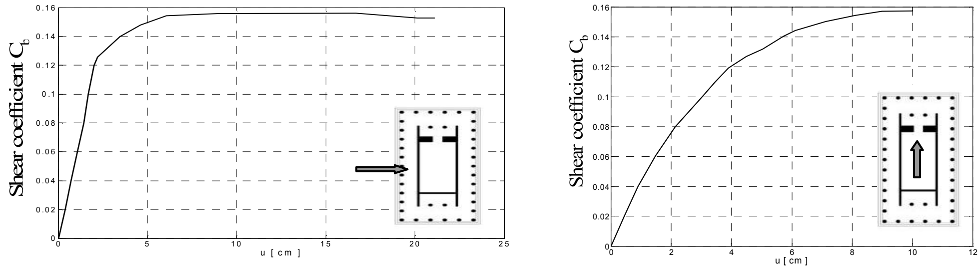

Figure 4: Push-over curves along the transversal and the longitudinal directions

The non linear behaviour of the Concordia temple is characterised by rocking phenomena implying a low energy dissipation. The cyclic behaviour of the implemented constitutive model is characterised by the complete closure of the interfaces and by no hysteresis dissipation. Hence, a non linear elastic constitutive behaviour has been assumed. A linear elastic SDOF system equivalent to the structure has been identified for each analysed direction. In particular, the period has been evaluated by means of the secant stiffness obtained by the maximum target displacement in the push-over curve, while the mass has been evaluated by means of the total mass of the original system. Finally, for the assessment of the seismic vulnerability, the elastic design spectra, provided by the Italian code, have been considered.

The equivalent SDOF systems are characterised by a mass $m^* = 1.238.500\ [N\,s^2/m]$ and by the following data: *a) transversal direction:* equivalent stiffness $K^* = 8.800.000\ [N/m]$, equivalent period $T^* = 2,33\ [s]$; *b) longitudinal direction:* equivalent stiffness $K^* = 19.474.899\ [N/m]$, equivalent period $T^* = 1,58\ [s]$.

The peak round acceleration (PGA) expected at the site (II category) is 0,25 g, according to the Italian code OPCM 3431. However, the expected PGA value has to be increased by making use of the importance factor, which is in this case $\gamma_I = 1,2$ in view of the historical value of the structure.

Assessment of both deterministic and stochastic seismic vulnerability has been conducted. Furthermore, in absence of a soil characterisation, in order to provide the results with more generality, the seismic vulnerability has been evaluated for different soil characteristics and spectral damping ratios. In particular, the categories of soil included in the Italian code OPCM 3431 and the spectral damping values 3%, 5% e 7% have been considered.

Deterministic Assessment of the Seismic Vulnerability The PGA value correspondent to the collapse of the structure can be obtained by equating the collapse shear coefficient $C_{b,u}$ to the expression of the design spectrum correspondent to the period T^* of the SDOF equivalent system. The percentage risk coefficients obtained by dividing the collapse PGA for the PGA expected at the site ($0,3g$ for the case of severe damage) are reported in Tabella.1.

Tabella 1: Risk coefficients for the transversal and the longitudinal directions

Transversal direction		S 1 (cat.A)	1,25 (cat.B,C,E)	1,35 (cat.D)
	3	129,34	82,78	47,90
ξ[%]	5	144,60	92,55	53,56
	7	158,40	101,38	58,67

Longitudinal direction		S 1 (cat.A)	1,25 (cat.B,C,E)	1,35 (cat.D)
	3	75,10	48,06	27,81
ξ[%]	5	83,96	53,74	31,10
	7	91,98	58,87	34,07

The sensitivity of the results to the spectral damping and type of soil can be easily noted. The risk coefficients vary from safe results which guarantee the stability (above 100%) to extreme vulnerability (below 100%, the lowest value is 27,81%). No data concerning with the value to be adopted for the spectral damping is available. However, it must be noted that the damping should be mainly associated to the bouncing and the sliding between the blocks, although neglected in the considered model. The dissipation due to the deformation of the blocks themselves can be considered negligible with respect to the other dissipation sources..

Stochastic assessment of the seismic vulnerability The evaluation of the seismic vulnerability by means of a proper stochastic analysis accounts for a seismic input modelling by means of a stochastic process. Under the hypothesis of Gaussian and stationary process, the seismic input is characterised by the power spectral density (PSD) function defined inn the frequency domain. The

PSD function is symmetric and defined both in the negative and positive frequency ranges. Its '*unilateral*' counterpart is obtained, in the positive frequency range only, by doubling the power content.

According to the code, rather that a direct stochastic characterisation of the input process, the PSD function can be obtained on the basis of the code design spectrum, hence defined as a PSD function 'compatible' with the design spectrum. More precisely, it is required that the stochastic response of the SDOF oscillators, subjected to synthetic accelerograms generated by a compatible PSD function, do not differ more that 10% from the design spectrum.

The adopted expression for the unilateral compatible PSD function $G_{\ddot{U}_g}(\omega_0)$, under the hypothesis that the response of a SDOF oscillator for varying natural frequency ω_0 and damping ratio ξ_0 is obtained by a stationary input white noise (Cacciola et al. 1999), is the following:

$$G_{\ddot{U}_g}(\omega_0) = \frac{S_{pa}^2(\omega_0,\xi_0) - \eta_U^2(T,0.5)\int_0^{\omega_0} G_{\ddot{U}_g}(\omega)d\omega}{\omega_0\eta_U^2(T,0.5)\left(\pi/(4\xi_0)-1\right)} \tag{1}$$

where $S_{pa}(\omega_0,\xi_0)$ is the pseudo-acceleration design spectrum concerning the severe damage (DS) state as indicated by the code, $\eta_U(T,0.5)$ is the response peak factor in terms of displacement referred to the 50% non-excess probability and dependent on the observation period T.

The objective of the seismic vulnerability assessment, according to a stochastic procedure, requires the definition of probability of non-excess (success) $P_{U_{\max}}(\bar{u};T)$ of a given displacement threshold $\bar{u}$ by the maximum peak displacement variable $U_{\max}(T)$ of the response. The latter probability function, in view of the linear behaviour of the SDOF oscillator equivalent to the structure under study, can be expressed as follows:

$$P_{U_{\max}}(\bar{u};T) \equiv \text{prob}\left[U_{\max}(T) \le \bar{u}\right] = \left[1-\exp\left(-\bar{u}^2\big/2\lambda_{0,U}\right)\right]\exp\left[-\alpha(\bar{u})T\right] \tag{2}$$

where $\alpha(\bar{u})$ is the so called '*risk function*' (Vanmarcke 1975) dependent on the spectral moments $\lambda_{0,U},\lambda_{1,U},\lambda_{2,U}$, of order 0,1,2, of the response process to be evaluated by the response PSD function $G_U(\omega)$ given as:

$$G_U(\omega) = \left|H(\omega)\right|^2 G_{\ddot{U}_g}(\omega) \qquad \text{where} \qquad \left|H(\omega)\right|^2 = \frac{1}{(\omega_0^2-\omega^2)^2 + 4\xi_0^2\omega_0^2\omega^2} \tag{3}$$

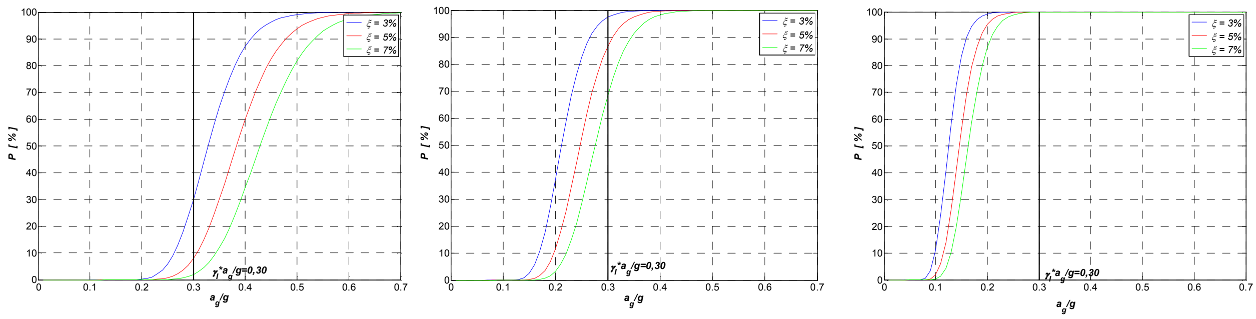

Figure 5: Severe damage probability for the transversal direction and damping ratios 3%, 5% and 7%: a)type A soil; b) type B,C,E soil; c) type D soil

Equation (2) allows the evaluation of the success probability, concerning the considered limit state (severe damage DS), referred to a displacement threshold given by the maximum displacement $\bar{u} = u_{\lim}$ reached by the push-over curve $\bar{u} = u_{\lim}$ ($u_{\lim} = 21{,}09\,cm$ in the transversal direction; $u_{\lim} = 10{,}01\ cm$ in longitudinal direction).

The success probability can be evaluated for different types of soil, damping ratios and for the two considered seismic input directions. In particular, it should be noted that the compatible PSD function on which basis the success probability is evaluated, is dependent on the type of soil and the considered damping ratio; on the contrary, the SDOF oscillator equivalent to the structure is dependent on the considered seismic input directions.

The seismic vulnerability, evaluated in stochastic terms, can also be expressed by means of the failure probability, which takes the meaning of severe damage probability $P_{DS}(u_{\lim};T)$ for the considered limit state. The latter is defined as the complement to the unity of the success probability.

The results concerning the evaluation of the stochastic seismic vulnerability are summarised in Figs.5 and Fig. 6 where the severe damage probability curves, as functions of the ground acceleration, are depicted. In particular, the severe damage probability function is plotted for different values of the damping ratio and types of soil for both the transversal (Fig. 5) and longitudinal (Fig. 6) seismic input direction. By considering the value $0{,}3g$ for the expected PGA, the evaluation of the severe damage probability can be obtained and compared with the values of the risk coefficient, reported in Tabella.1, obtained by the deterministic analysis.

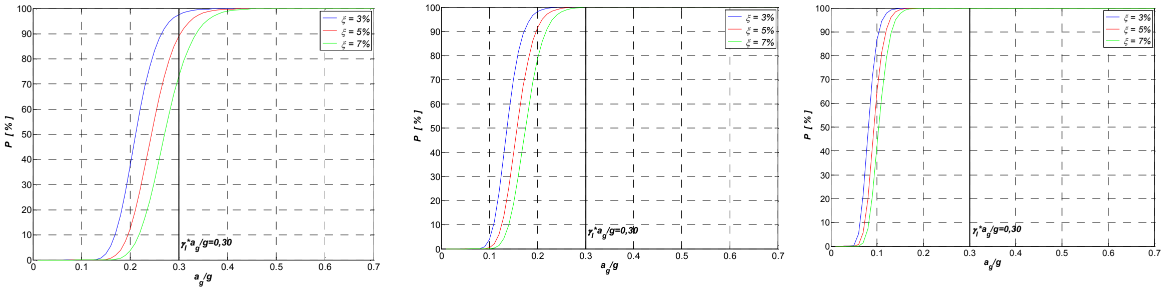

Figure 6: Severe damage probability for the longitudinal direction and damping ratios 3%, 5% and 7%: a)type A soil; b) type B,C,E soil; c) type D soil

For the input along the transversal direction (Fig. 5), the severe damage probability ranges: from 30% to less than 5%, for increasing damping ratio (for type A soil); from 95% to less than 70%, for increasing damping ratio (for type B,C,E soil); equal to 100% (for type D soil).

On the contrary, the longitudinal direction (Fig. 6) is the most vulnerable condition showing levels of the severe damage probability from 95% to 75%, for increasing damping ratio (for type A soil); while 100% is always reached (for type B,C,E and type D soil) regardless of the adopted damping ratio.

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