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Assessment of the seismic behaviour of the original and modified structure of the Veronicas market at Murcia (Spain)

Evaluación del comportamiento sísmico de la estructura original y modificada del Mercado de Verónicas en Murcia (España)

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ABSTRACT

In this manuscript, it is presented, a case-study about the assessment of the seismic behaviour of Veronicas Market. Its structure, initially constituted by steel trusses supported on masonry walls, was subsequently modified by the addition of a mezzanine supported on reinforced concrete piers. Applying the capacity spectrum method, based on an updated non-linear finite element model, the performance levels of both the original and modified structure have been compared to assess the effects originated by the change in the transmission mechanism of the seismic loads.

Keywords: historic masonry structure, non-linear analysis, finite element method, capacity spectrum method, seismic vulnerability.

RESUMEN

En este artículo, se presenta, un caso en estudio sobre la evaluación del comportamiento sísmico del Mercado de Verónicas. Su estructura, originalmente constituida por cerchas metálica apoyadas sobre muros de fábrica fue posteriormente modificada por la incorporación de una entreplanta con pilares de hormigón armado. Aplicando el método del espectro de capacidad, con base en un modelo calibrado no-lineal de elementos finitos, se han comparado los niveles de desempeño de la estructura original y modificada para valorar los efectos originados por el cambio en el mecanismo de transmisión de las cargas sísmicas.

Palabras clave: *estructura histórica de fábrica, análisis no-lineal, método de los elementos finitos, método del espectro de capacidad, vulnerabilidad sísmica.*

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1. INTRODUCTION AND OBJECTIVE

There are numerous historic buildings in the urban areas of European cities (some of them subjected to special consideration and high occupation) which have been built without any seismic design criterion due to their antiqueness. To this respect, it is convenient to adopt preventive strategies in order to reduce, through the corresponding study of seismic vulnerability of the existing structures, the potential devastating effects of an earthquake (1). The seismic vulnerability of either a single structure or a complete urban area is characterized through its intrinsic predisposition to suffer damage during the occurrence of a given earthquake, and it is directly associated with the structural design characteristics (2). The evaluation of such vulnerability is usually evaluated by comparison between the strength capacity of the building and the seismic demand, determining the corresponding seismic performance point (2),(3).

In the particular case of Murcia, due to the potential seismic risk in this region of Southern Europe (4) and, particularly, after the earthquake occurred in Lorca in the last 11th May 2011, it is convenient to assess the seismic vulnerability of the historic buildings located in the urban centres of the cities (5). Moreover, some of these buildings have been adapted to different uses over time, suffering different structural and non-structural interventions (i.e., modifying its seismic vulnerability). This is the case of the *Verónicas* Market, located in the historic centre of Murcia city, near the Segura River (Figure 1); the masonry structure of this relevant building presents a *degree 2* of protection according to the Special Plan of the Historic Buildings of Murcia (6), and it is considered of special importance by the technical standards (4),(5),(6),(7); currently this building continues operating as supply market of the city and represents one of its most important centres of commercial activity.

After the success of *Les Halles* (Paris, 1852), the councils of the big Spanish cities at that time developed similar projects to the market performed by Victor Baltard in Paris (8); such is the case of the *Cebada* Market at Madrid, performed by the architect Calvo Pereira in 1867, or the Valencia Market designed by Ferreres and Morales in 1883 (9). The strong development of the local mining industry (and the consequent population growth) at Murcia promoted the construction of supply markets with similar characteristics to those above mentioned. Within this general trend, the construction of the *Verónicas* Market (9) was developed between 1912 and 1916 under the design proposed by the architect Pedro Cerdán (Figure 2a).

The market was initially organized as a rectangular nave opened to the outside through two large frontal doors; from an architectural point of view, the central space was configured as a widely stance illuminated from above by a covering of iron and glass, which represents a trace in the works of Pedro Cerdán (8). The singularity of this market, where the modernist style of its plant configuration blends with the eclectic style of its facades, lies in its structural type and the successive modifications that the building has experimented over time (particularly, in the period 1975-2009, according to data supplied by the Murcia s City Council). From the point of view of the structural behaviour of the building, it is particularly relevant the great modification performed by the architect Daniel Carbonell in 1975 (10), consisting of a mezzanine floor supported on a reinforced concrete frame structure and connected to the original masonry structure through steel members. Although

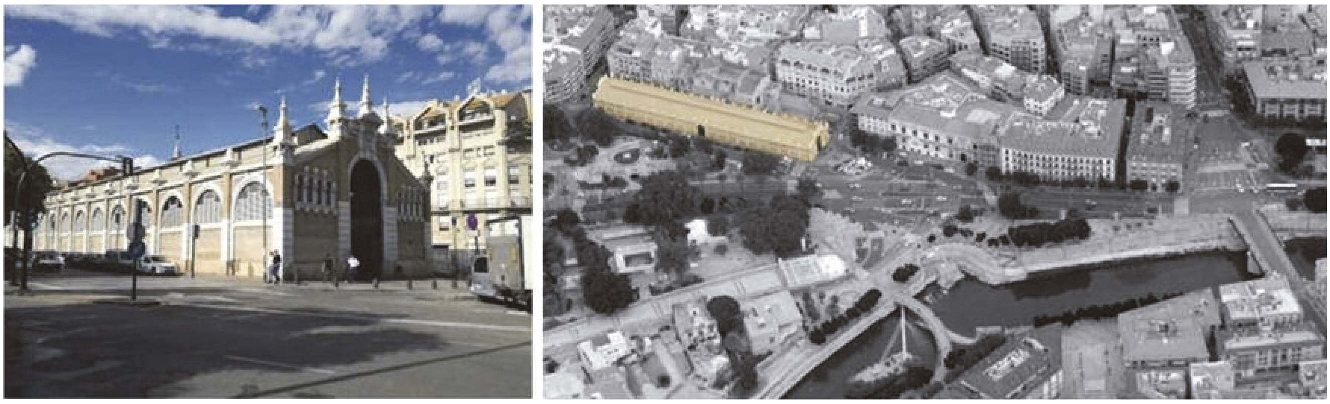
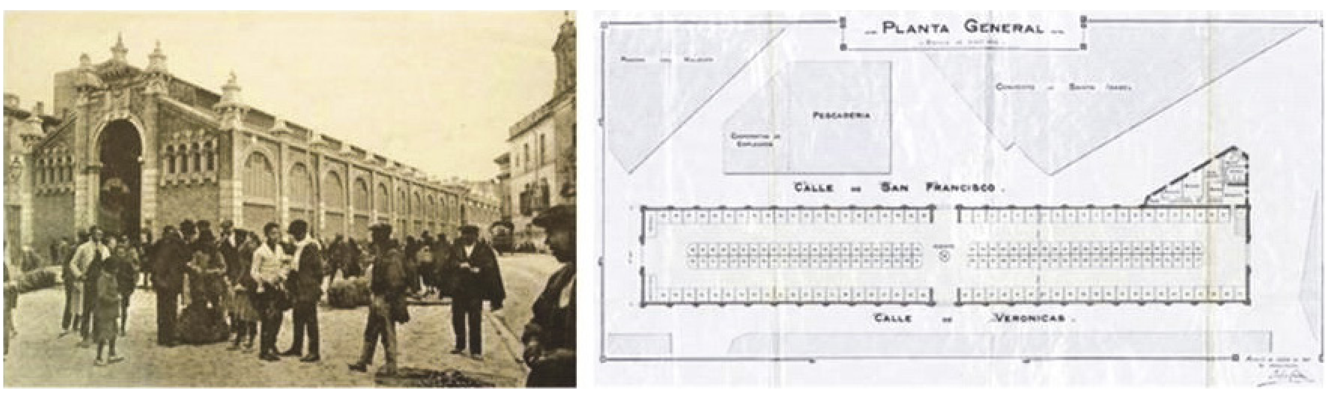


Figure 1. The Veronicas Market: a) current appearance; b) aerial view and location.



a) b)

Figure 2. Original building (source: historical archive of Murcia): a) vintage photograph (1916); b) original plan of the plant configuration.

in 1974 there were already spanish standards about seismic design, from the analysis of the project performed by Daniel Carbonell, as well as an exhaustive in-situ observation of the real structure, it is concluded about the absence of design elements that provide horizontal seismic strength to the market; due to this reason, the seismic vulnerability of the structural assembly might be high. Moreover, this kind of buildings were usually designed in order to resist perfectly vertical (compressive) static loads, but not always load conditions different from the vertical ones (e.g., the seismic actions). The objective of this work is to assess the seismic vulnerability of the *Verónicas* Market using the capacity spectrum method and determining the seismic performance point of the structure before and after the construction of the mezzanine floor. To this aim, this paper has been organized as follows; the Section 2 is dedicated to describe the main characteristics of the structure. Section 3 explains the numerical modelling of the original and modified structure, as well as the subsequent tuning of both numerical models from both experimental in-situ evaluation of the current structure and the reference values of several mechanical parameters extracted from the technical literature.

The numerical modelling has taken into account the non-linear constitutive relationships of the involved materials. The experimental modal properties of the current structure have been obtained from an operational modal analysis based on the measurements obtained in an ambient vibration test; moreover, a complementary set of ultrasound tests have been performed on some masonry members in order to establish a search domain for the tuning of the mechanical properties of the masonry structure. Once both structural models have been calibrated, Section 4 presents their vulnerability assessments in terms of their seismic performance points. Finally, some concluding remarks are drawn to close the paper.

2. DESCRIPTION OF THE STRUCTURE

As the most markets at that time, the building presents a rectangular plant with a length of 110 m and a width of 18.00 m. The original structure is organized into 2 side bearing walls and 22 frames (two of them integrated in the frontal facades, and 20 intermediate frames). The side walls present a thickness of 50 cm and contain two large openings which are the two side entrances to the building. The longitudinal span between frames is 5.15 m, except in the side doors, where the intermediate spans grow up to 5.52 m (Figure 3a).

Each intermediate frame consists of a steel truss supported on two side masonry pilasters with a section of 1.20x0.80 m^2. The frames are stiffened longitudinally through steel beams IPE180 (that rest directly on the pilasters) and steel truss beams with a depth of 40 cm (Figure 3b). The total height of the structure is 7.15 m on pilasters and 12.62 m at the highest point of the roof (Figure 3c).

The whole structure rests in a shallow foundation that combines continuous footings under the bearing walls and single footings under masonry pilasters.

The unidirectional mezzanine floor presents a thickness of 25 cm, supported on dropped beams with a depth of 50 cm (Figure 4a). From the ground floor to the first floor there are 4 lines of reinforced concrete supports with section 30 x 30 cm† (Figure 4b). Likewise, in order to discharge the original pilasters after the construction of the entresol, steel supports formed by two profiles UPN150 and connected to the upper truss of each frame were added from the first floor to the covering of the building (Figures 4c y 4d).

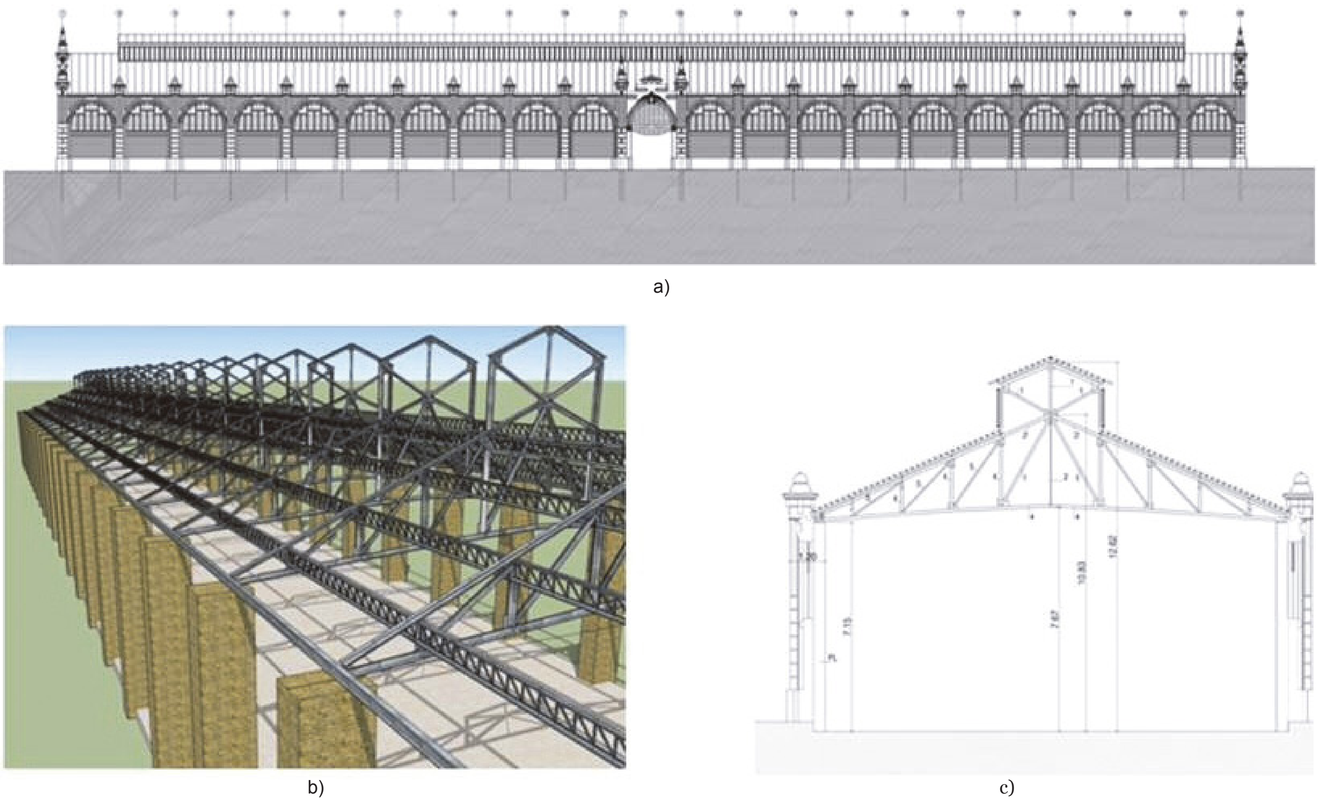
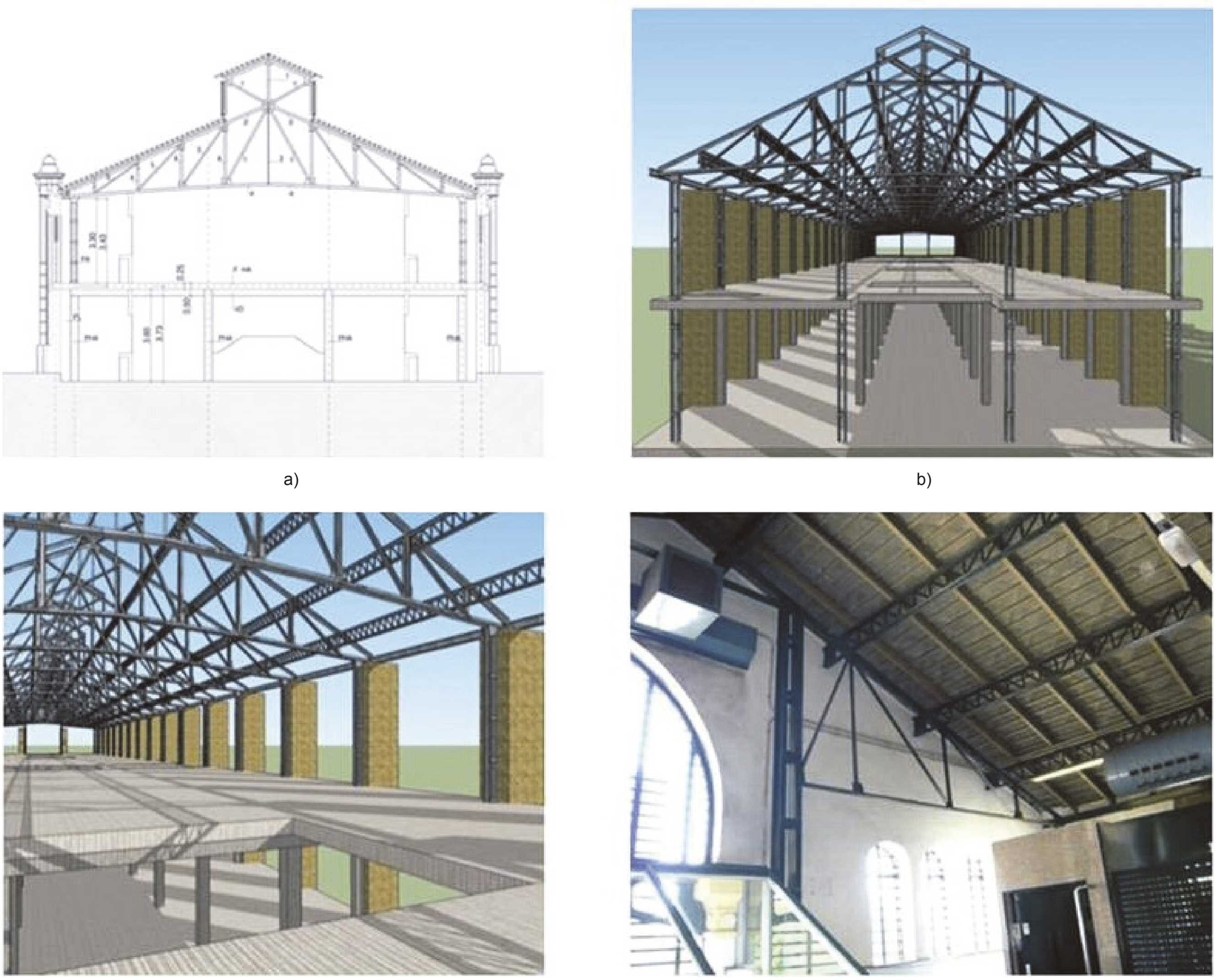


Figure 3. Original structure: a) Lateral view of the market; adapted from (10); b) 3D modelled detail of longitudinal stiffeners; c) Elevation view; adapted from (10).



a) b) c) d)

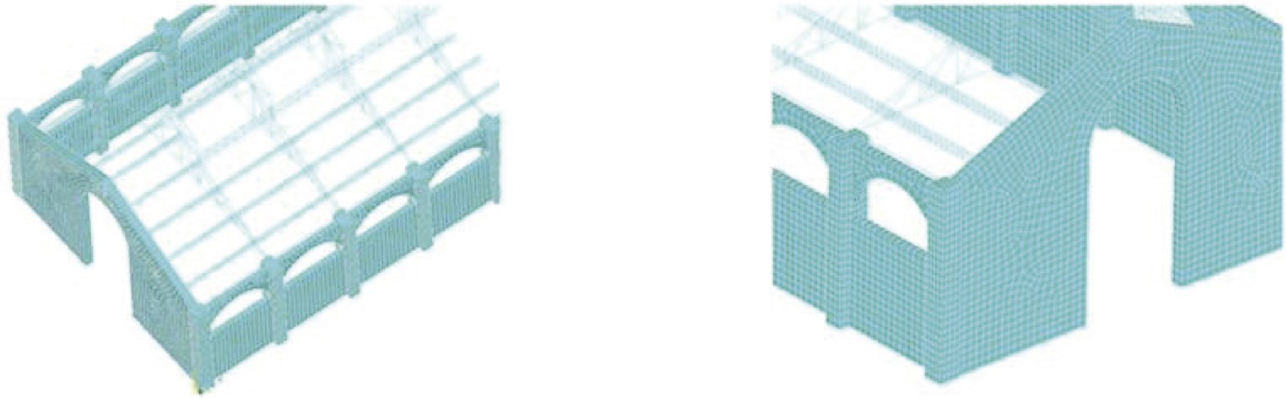
Figure 4. Modified structure: a) Elevation view; adapted from (10); b) and c) 3D modelled details of the entresol and its supporting members; d) Real detail of the steel frames connected to the front walls.

3. NUMERICAL MODELLING

3.1. Finite element model

The behaviour of the market is modelled by using solid finite elements that take into account the stiffness of the masonry members modified by the development of cracking and crushing; in this context, the finite element modelling is supported on the concepts of homogenized material and smeared cracking constitutive law (11),(12), where the masonry is assumed as an isotropic continuum (i.e., the brick units and the mortar joints are assumed to be smeared and an isotropic material represents the masonry assemblages). The application of this type of constitutive models to real 3D ancient structures as the *Verónicas* market is complicated due to the high amount of degrees of freedom of the finite element mesh as well as the experimental tuning of the physical parameters involved in such models (13),(14). In this case, the mechanical behaviour of the structure may be modelled using a macromodelling technique (12), where the bricks, the mortar joints and the interfaces are modelled as single continuous elements, and the mechanical properties of such elements depend on those of the basic components (14).

Two three-dimensional numerical models of the original and modified (or current) structure were developed using the software ANSYS (15). The geometry of both structures has been firstly reconstructed by means of CAD tools, and then all the volumetric members have been imported and meshed in ANSYS by means of Boolean operations. The volumes of all the perimeter masonry members have been glued whereas the wall-to-covering and the pilasters-to-steel beams connections were implemented through coupling of the implied degrees of freedom. The side walls and pilasters, as well as the reinforced concrete supports (in the current structure) were glued between them (in order to perform a more conservative configuration) and they were modelled using eight-node iso-parametric solid elements (SOLID65) having three degrees of freedom at each node. The upper steel trusses and the steel longitudinal stiffeners (i.e., the side beams supported on the pilasters and the steel truss beams) have been modelled using three-dimensional beam elements (BEAM188). Two-dimensional elements with four nodal points (SHELL181) were adopted for the modelling of the reinforced concrete entresol. The average size of the mesh element is 0.3 m for both configurations. Figure 5 contains several snapshots with the proposed finite element discretization for both structural configurations. The



a)

b)

Figure 5. Finite element model: a) original configuration; b) current configuration.

major openings in the front walls and side walls, as well as the stairwells (and their corresponding auxiliary structural members) have been also reproduced; finally, a rigid ground foundation (fixed base model) has been assumed for all the side walls and supports.

To reproduce the plastic behaviour of the masonry and the reinforced concrete members, the Drucker Prager (DP) plasticity criterion (14, 15), originally proposed for pressure-dependent inelastic materials such as geo-materials, was adopted. Zucchini and Louren o (16) validated the DP criterion in order to account for the degradation (by plastic deformation) of the mechanical properties of masonry subjected to compressive stresses. The circular cone yield surface of the plasticity criterion is determined by means of two parameters: the cohesion (c) and the internal angle of friction (φ). These two parameters are related to the uniaxial tensile strength (f_t) and the uniaxial compressive strength (f) according to the following relationships (cf. 17):

$$\varphi = arcsen\left(\frac{f - f_t}{f + f_t}\right) \qquad c = f\,\frac{1 - sen\varphi}{2\cos\varphi} \qquad [1]$$

The above referred yield surface is assumed that does not change with progressive yielding, hence no hardening rules are considered and the material is elastic perfectly plastic (the friction angle and the cohesion are constant and do not depend on the plastic deformation). Finally, the DP plasticity behaviour involves a third parameter, the so-called dilatancy (δ); if the dilatancy is null, then no plastic volumetric strain will be produced. Experimental results for soils and rocks show that the volumetric dilatation obtained by the experiments is less than the one corresponding to δ = φ (14).

In order to provide a tension cut-off for the tensile stresses, in this work the DP criterion is combined with the Willam and Warnke (WW) failure criterion (originally proposed for concrete (18), *apud* (12),(13)), which takes into consideration both cracking and crushing failure modes using too a smeared approach; in this way, the material behaves as an isotropic medium with plastic deformation, cracking and crushing capabilities. In general, five constants are needed in order to define this second criterion; however, in most practical cases (when the hydrostatic stress is limited by $\sqrt{3}$ f), such model may be only formulated as a function of the uniaxial tensile and compressive strengths (12),(15). The presence of a crack at a Gauss point is considered by the modification of the stress strain relationship and the introduction of a weakness plane normal to the crack face. Moreover, two shear transfer coefficients (each one of them corresponding to a crack status: open β_t and closed - β_c -) are also required in the WW criterion; such coefficients represent a shear strength reduction in order to take into consideration slide effects in the crack face. The combined used of both constitutive models was validated by Chiostrini et al. (19) in a macro-element context by modelling several diagonal tests on masonry samples.

3.2. Experimental tests

It is essential to verify the adequacy of the models with the existing structure of the market; this can be carried out using several techniques, among which are the socalled dynamic identification tests (20),(21).

3.2.1. Ambient vibration test and operation modal analysis

An ambient vibration test of the structure was developed under service loadsinduced operational conditions. The experimental data obtained from this test allow obtaining the dynamic parameters of the structure; to this purpose, a preliminary spectral-modal analysis of the current and modified structure was performed in order to design the experimental test. As main results of this preliminary analysis the following conclusions were considered: (i) the vibration modes that must be identified to characterize adequately the response of the structure under seismic action and (ii) the parameters that define properly the experimental tests (sampling frequency, set-ups, duration).

The ratio between the base shear effort of each vibration mode and the total base shear effort has been determined for the first forty vibration modes (number of vibration modes that mobilize more than 95% of the building mass). It has been checked that this ratio is greater than 0.90 for the first transverse and longitudinal vibration modes so only these two vibration modes are needed to approximate the response of the structure under seismic loads. On the other hand, it has been verified both that the efforts originated by the seismic action in longitudinal direction are negligible and that the mechanism to transmit the horizontal loads is not modified by the construction of the mezzanine. For these reason, the experimental test focused on the estimation of the first transverse natural frequency and associated vibration mode.

For this purpose, a gridline is established (coinciding with one of the two longitudinal side walls), compound by 11 points equally distributed, in such a way that each point lay in an intermediate pilaster (Figure 6). Therefore, the structure of the market was discretized into 10 sections (or setups). For each setup, the transverse dynamic parameters of the structure have been determined. The measures were obtained with 4 triaxial accelerometers, sensitivity 10 V/g, type Episensor and made by the company Kinemetrics. The duration of each set-up was 1000 seconds and the sampling frequency was 100.00 Hz (22). From the recorded measurements, the Enhanced Frequency Domain Decomposition (EFDD) is used to extract the modal parameters of the structure. Among the vibration modes identified, as it was above mentioned, only the first one -in particular its corresponding natural frequency- has been used to tune the finite element model of the structure. The first experimental natural frequency of the current structure, in transverse direction, is equal to 10.7 Hz. The previously described practical application was performed using the ARTeMIS Modal software developed by SVS A/S (23).

3.2.2. Ultrasound tests

A set of complementary ultrasound tests was performed in the side masonry walls and in some of the masonry pilasters in order to establish a search domain for the tuning of the mechanical properties of masonry of both finite element models. The method consists in sending repeated ultrasonic pulses in the material by means of a pulse generator and the subsequent sensing of pulses passing through the masonry material. The transit time needed to go through the thickness of the member (i.e., from the sending probe to the sensing one), is registered. Then, ultrasonic pulse velocity (UPV) is calculated from the ultrasonic pulse transit time and the length of the known trajectory. Ultrasonic probes (with frequency equals to 54 kHz) were situated according to the so-called direct transmission (i.e., against each other). Several authors pointed out a reasonable correlation between the compressive strength of masonry prisms and the velocity of the ultrasound wave for the case of direct transmission (24). Nevertheless, UPV is dependent on materials and the type of test technique; due to this reason, a generalized relationship between masonry compressive strength and ultrasonic velocity is not available, thus the UPV test represents only an approximate method for the evaluation of the in-place strength of the masonry. Regarding to the dynamic modulus ($\overline{E}$) and the Poisson s ratio (ν), these parameters can be determined from the velocity of the longitudinal (v_l) and transverse (v_s) ultrasonic waves for a given value of the specific weight (ρ) of the masonry according to the following relationships (25):

$$\nu = \frac{1-2\left(\frac{v_s}{v_l}\right)^2}{2\left(1-\left(\frac{v_s}{v_l}\right)^2\right)} \; ; \; \overline{E} = \rho \cdot v_s^{\,2} \cdot \frac{3v_l^2 - 4v_s^{\,2}}{v_l^2 - v_s^{\,2}} \qquad [2]$$

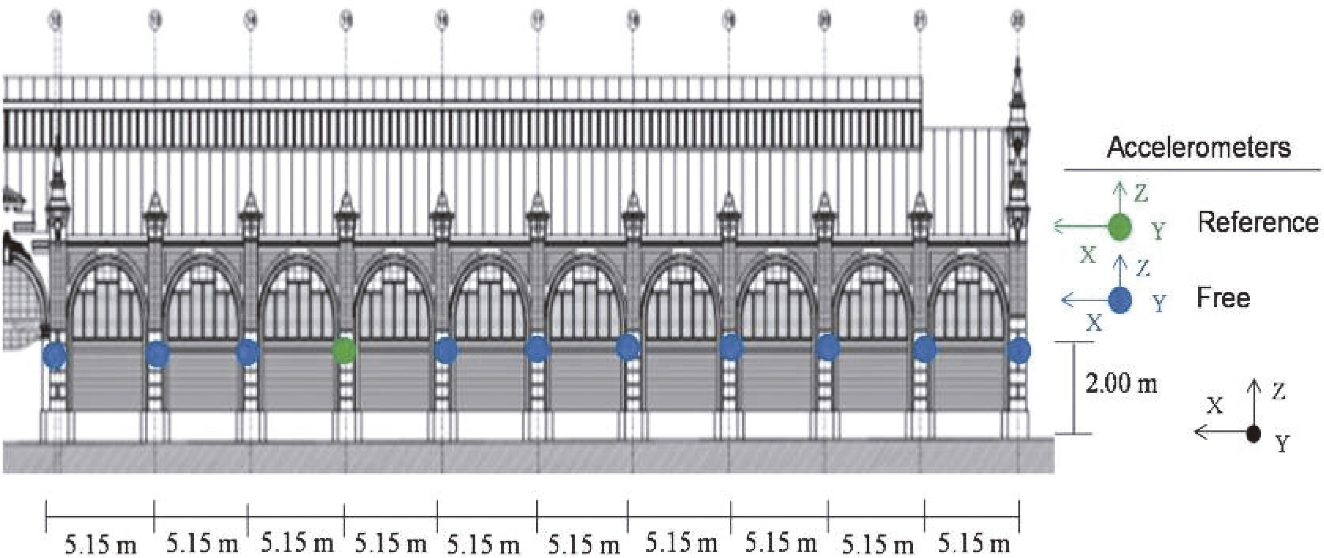


Figure 6. Measurement grid of the ambient vibration test.

3.3. Finite element model tuning

In a macro-element context, the assignment of the mechanical parameters of the DP and WW criteria above described requires a model calibration, being this one of the main objectives in the structural analysis of historic buildings by the F.E. technique (13). In the present case several strategies have been combined to this aim. Firstly, after a sensitivity study of the main physical parameters of the finite element model, it was confirmed, as expected, that the physical parameter with the greatest influence on the dynamic behaviour of the structure was the elastic modulus (E) of the masonry members. The average values of the longitudinal and transverse velocity of the ultrasonic waves registered on the all masonry members were equal to 2920 m/s and 1450 m/s, respectively. From these data, and considering a standard (preliminary) unit weight for the masonry ρ =1500 kg/m^3, the resulting dynamic modulus and Poisson s ratio from Equation [2] are equal to 8429.75 MPa and 0.3, respectively.

The previous value estimated for the Poisson s ratio is in consonance with the value proposed by the EC-6 (26). On the other hand, the value obtained for the dynamic modulus has been used as the midpoint of a search domain for the tuning of the above-mentioned elastic modulus based on the first identified transverse natural frequency of the current structure. Thus, the elastic modulus has been considered as parameter in the finite element model tuning. After this process, the resulting first transverse natural frequencies of the original and the current (modified) structure were equal to 5.64 Hz and 10.76 Hz, respectively, and the final tuned value of the elastic modulus (E) so obtained was equal to 12000 MPa. The unit weight of all the masonry members was adopted from experimental data supplied by local manufacturers, whereas standard values of density were assumed for both steel members and reinforced concrete members. A value of the Poisson s ratio equals to 0.3 was adopted for the steel members, whereas a value of this parameter equals to 0.2 was adopted for the reinforced concrete members. The elastic modulus of the both reinforced concrete members and steel members has been established from the technical information provided by Ref. (10).

Since the Veronicas market does not present any evident serious damage, the remaining values of the material parameters involved in the non-linear constitutive models presented in Section 3.1, which could not be experimentally measured, were extracted from literature references (mainly, laboratory and in-situ testing results of similar masonry materials) in a conservative sense (12, 13, 14). In this line, the authors have considered the work developed by Sinha and Pedreschi (*apud* 17), which proposes a relationship between the elastic modulus and the compressive strength in masonry assemblages under a good state of conservation:

$$f = \left(\frac{E}{1180}\right)^{1/0.83} \quad [3]$$

According to [3], and taking into consideration the value of the previously tuned elastic modulus, a compressive strength equals to 16 MPa is obtained. Moreover, previous works (21, 27) have proposed different ratios between the Young s modulus and the peak compressive strength of a masonry prism tested in laboratory (slightly higher than the one corresponding to the masonry assembly, cf. 17), varying such ratio from 360 to 780. As seen, the final value of the tuned elastic module corresponding to masonry members verifies this range. Additionally, some local brick manufacturers have been consulted about the average compressive strength of a brick similar to the one employed in the masonry structure of Ver nicas market, being such strength around 25 MPa; therefore, the value estimated in this work for the compressive strength of the masonry is reasonably conservative. Finally, for all the reinforced concrete members, the uniaxial characteristic compressive strength of the reinforced concrete was adopted equal to 25 MPa (according to the technical information supplied by the design project of the current structure) and the average tensile strength was assumed equal to 2.6 MPa, according to Ref. (28).

From the previous values of compressive strength, and assuming a tensile strength equal to 0.1 f for the masonry members (29), the preliminary values of the cohesion and the friction angle may be deduced from Equation [1]; Nevertheless, the combination of the two constitutive models introduced in the Section 3.1 requires a careful definition of their yield strengths in order to guarantee the proper intersection of the model domains; technical literature provide criteria according to the experimental evidence in order to perform such choice (14, 19); then, the friction angle and the cohesion have been adapted (i.e., the yield strengths of the DP model has been slightly corrected regarding to the values above defined) in order to verify such rules, and the resulting values have been reported in Table 1. Likewise, in the case of reinforced concrete members, the effect of the volumetric dilatancy has been neglected, whereas in the masonry structure the angle of dilatancy has been assumed equal to the lower limit (i.e., 15°), proposed in (14) for masonry assemblies under a good state of conservation. The assumed values for the parameters related with the crack status (β_t and β_c), like the ones corresponding to the plastic dilatancy of both materials, have been selected in order to facilitate the numerical convergence of the finite element model.

Table 1. Material parameters for constitutive models.

Criterion	Parameter	Masonry members	Reinforced concrete members
DP	Cohesion (MPa)	2.75	3.9
	Friction angle (°)	50	50
	Dilatancy (°)	15	0
WW	f (MPa)	16	25
	f_t (MPa)	1.6	2.6
	β_t	0.15	0.15
	β_c	0.75	0.75

4. COMPARATIVE SEISMIC ASSESSMENT

4.1. Push-over analysis

The study of the seismic behaviour of the *Verónicas* market has been performed using a push-over analysis (12, 30). The loads applied on the building do not change with the progressive degradation of the building during loading, and it does not account for the progressive changes in natural frequencies due to yielding and cracking on the structure. However, a high computational effort is required if this stiffness degradation is taken into consideration since it requires the solution of a modal problem at each load step. In this work, as alternative to expensive computational dynamic analyses, timeinvariant load distributions were considered with the aim to analyse the limit behaviour of the masonry building under seismic loads (11),(12).

The 3D numerical model previously tuned has been used to assess the modal behaviour of the *Verónicas* market under the original and current configuration. In the modal analysis performed in ANSYS (15) any type of non-linearity has not been considered. The tuned first transverse vibration mode of both configurations involves the displacement in the transverse direction of the building (Figure 7). According to this analysis method, and considering the shape of the considered vibration mode, the effects of the seismic loads have been evaluated by the application of a triangular load pattern of transverse forces proportional to the product of the masses by the displacements of the corresponding first transverse vibration mode (12),(31). The capacity diagrams were built considering the average displacement on the top level of the side wall and its base shear. Figure 8 represents the capacity curves with respect to the cases of loading in transverse direction for both configurations (original and current). The primitive (or real) pushover curve (Figure 8a) has been transformed to the ADRS (Acceleration-DisplacementResponse Spectra) format; subsequently, the bi-linear capacity spectrum for both configurations have been obtained (Figure 8b) according to ATC-40 (30); as seen, the ultimate real displacement is equal to about 1.8 mm and 4.8 mm for the current and original configuration, respectively.

Finally, Table 2 reports for the two mentioned configurations: (i) the total mass of the structure, (ii) the participation factor (PF), (iii) the base shear effort ratio (ratio between the base shear effort associated with each vibration mode and the total base shear effort under seismic action) and (iv) the yield (D_y), ultimate (D_u) and performance (D) displacement of the structure (according to the results of the capacity curves).

Table 2. Keypoints of the capacity curves for the two configurations.

	Original structure	Current structure
Total mass of the structure	1280 ton	2620 ton
PF	748.31 m^{-1}	794.94 m^{-1}
Base shear effort ratio	96.00%	90.78%
Yielding displacement (D_y)	0.0034 m	0.0010 m
Ultimate displacement (D_u)	0.0048 m	0.0018 m
Performance displacement (D)	0.0046 m	0.0011 m

4.2. Seismic assessment

The seismic demand has been obtained according to the Spanish standard NCSE-02 (7). For the case under study, according to the geotechnical report, the ground under the structure has been classified as type III based on the soil classification provided by the Spanish standards (7); the peak ground acceleration for the reference return period (a_g) at Murcia, considering an importance coefficient for special constructions

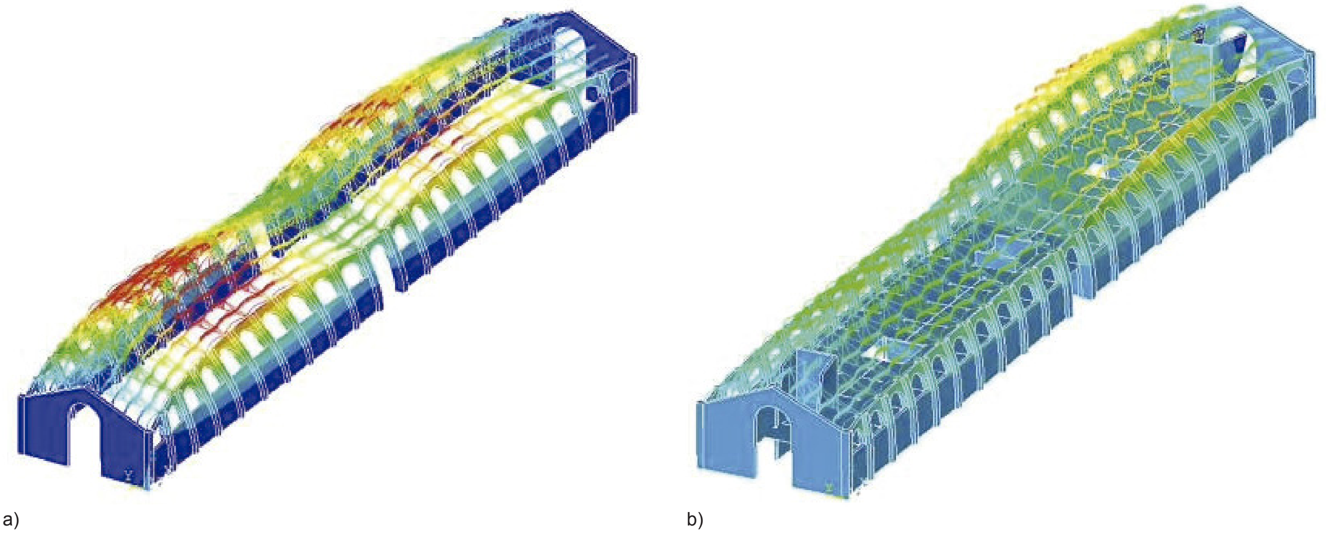


Figure 7. First tuned vibration modes for the original (a) and current (b) configurations.

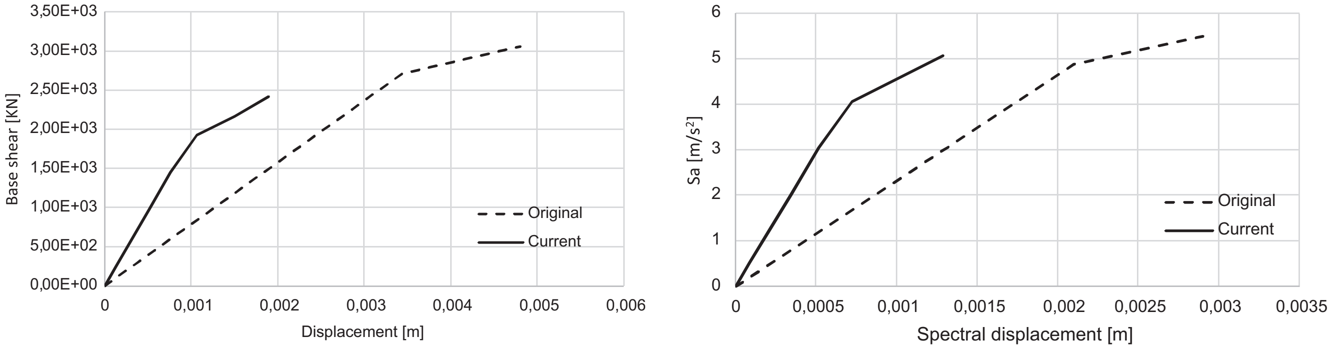


Figure 8. Capacity curve (a) and bilinear capacity spectrum (b) for the original configuration and the current configuration.

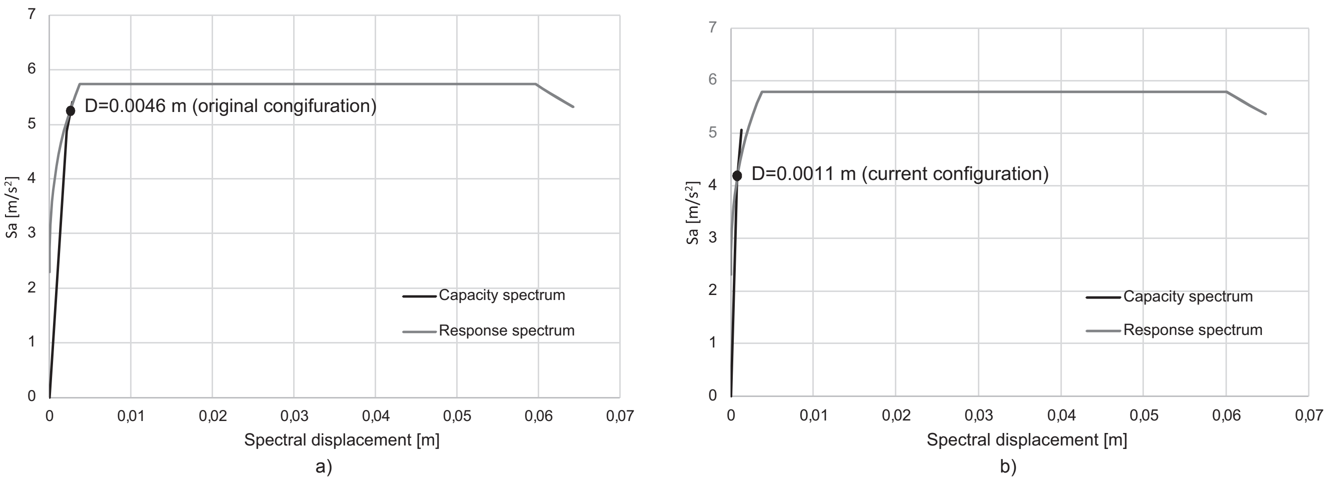


Figure 9. Seismic performance points for the original (a) and the current (b) configuration.

(ρ =1.3), is equal to 2.28 m/s². Due to the presence in the building of several structural members with different damping characteristics, the damping ratio (φ) has been estimated using the formulation proposed by the Spanish standard for bridge design under seismic actions (32):

$$\xi = \frac{\sum_i \xi_i E_i}{\sum_i E_i} \qquad [4]$$

where E_i is the elastic energy of the i-th member associated with the first transverse vibration mode and ξ_i is its corresponding damping ratio. This weighted damping ratio has been calculated for both configurations (the original and the current structure); the value of the elastic energy has been calculated from the finite element model of each configuration and the values of the damping ratio for each material (masonry, steel and reinforced concrete) has been extracted from (7). The resulting values are equals to 4.9% and 4.8% for the original and the current structure, respectively. Subsequently, these damping ratios have been modified in order to include the hysteretic damping during the performance analysis. As result of this analysis, Figure 9 illustrates the bi-linear capacity spectrum and its corresponding response spectrums in format ADRS, as well as its intersection, the performance displacement for each structural configuration, (a) original and (b) current. As it is shown, the performance displacement (D) is higher in the original structure than in the current one, due to the stiffener effect of the reinforced concrete structure (formed by the mezzanine on the reinforced concrete piers). On the other hand, the performance displacement of the original configuration exceeds the yielding spectral displacement of the structure and it is near to the ultimate (or failure) displacement of the structure (D=0.0046 m,, D_y=0.0034 m D_u=0.0048 m), i.e., corresponding to an extensive damage state (cf. 2) whereas the performance displacement of the current structure does not exceed the yielding displacement and it corresponds fully to a mild damage state (D=0.0011 m; D_y=0.0010 m; D_u=0.0018 m), what implies a significative improvement in the seismic performance of the ancient structure.

5. CONCLUDING REMARKS

The Verónicas market is a masonry ancient building of Murcia City that suffered a great structural transformation in 1975 through the addition of an entresol supported on a reinforced concrete structure. This work has developed a case-study of the seismic vulnerability of both the original and the current structure of the market, taking into account the non-linear behaviour of the materials involved at each configuration. The corresponding finite element model has been calibrated through in-situ dynamic tests of the masonry members; such values have been contrasted with the ones measured in laboratory masonry structures tested by other authors and a good level of agreement has been found. The vulnerability comparative assessment so performed indicates an improvement of the seismic performance of the ancient building due to the presence of the reinforced concrete structure. This fact results particularly relevant since the addition of reinforced concrete members to existing masonry structures might aggravate their seismic vulnerability, due mainly to discontinuity effects in the local stress transmission between both materials; nevertheless, in this case the combination of both type of elements implies a decreasing of the seismic damage from an extensive level to a little or mild one, resulting in a higher capacity for the dissipation of the seismic energy through the ductility of the modified structure.

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