

RISK ASSESSMENT OF REINFORCED CONCRETE BUILDINGS CONSIDERING THE EARTHQUAKE DIRECTIONALITY EFFECTS

Keywords: Seismic risk assessment, non-linear dynamic analysis, uncertainties, earthquake directionality effects

Abstract.*In order to assess the seismic risk of structures, several methods based on pushover analysis have been developed. These methods are very useful in the case of symmetric structures because they can be easily approximated by means of a 2D model. In the case of asymmetric structures, several improvements to the pushover analysis procedure have been proposed in order to consider the effect of asymmetry on the global response. However, in some cases, such methods can be more expensive, from the computational point of view, than the nonlinear dynamic analysis. In this article, we propose to assess the effect of the directionality of the earthquake by using non-linear dynamic analysis and considering uncertainties in the mechanical properties of the materials and in the seismic action. We use as a case study a group of reinforced concrete buildings located in Lorca, Spain, damaged by the earthquake of May 2011. The results found show a good agreement between the observed and the calculated damage.*

1 Introduction

In the seismic risk assessment of buildings are involved two main random variables, the vulnerability of the structure and the seismic action. The uncertainty related to the former depends on the mechanical properties of the materials, the weight supported by the structure, the participation of the non-structural elements, among others. The uncertainty of the latter depends on the fault mechanism and the soil conditions, among many other factors. Even if the seismic action is known, for instance in the case that the earthquake has occurred and an acceleration record is available, there is another source of uncertainty related to the directionality effect. It means that, due to topographical and geological conditions, among others, it is difficult to establish the incidence of the acceleration upon the buildings of the studied area. If data associated with the observed seismic damage are available, one can validate the computational method used for calculating the seismic damage. Several methodologies based on static procedures have been developed for assessing the expected seismic damage of buildings [1, 2, 3, 4, 5]. These methodologies are based on the assumption that the behaviour of the building is governed by the first mode of vibration. For tall or asymmetric buildings, further consideration should be made by taking into account the effects of higher modes in elevation [6,7], in plan [8, 9, 10] or in both, plan and elevation [11, 12,13,14]. However, in some cases, due to the high irregularity of a building, it is very difficult to perform a reliable pushover analysis, which is the base of the procedures based on nonlinear static methods. In these cases, it is necessary to assess seismic damage by performing non-linear dynamic analysis (NLDA) instead of pushover analysis. However, ignoring particular cases, simplified static methodologies were applied in several previous studies in order to calculate the seismic risk of urban areas [15, 16, 17, 18, 19] obtaining successful results. In this article, we propose a methodology to calculate the seismic damage of a group of buildings located in the San Fernando neighborhood of the city of Lorca, Spain, which were affected by the earthquake occurred in May 2011. We analyze the nonlinear dynamic response of the buildings including the uncertainties related to the mechanical properties of the materials and the directionality of the earthquake. On the first hand, the mechanical properties considered as random are the yield strength of the concrete and the yield strength of the steel. The elastic modulus of both variables will be also treated as random variables because they are calculated starting from the corresponding yield strength by means of formulas proposed in the literature. Besides, we conduct a study of the spatial variability of the random samples generated for the mechanical properties. Based on this study, we propose a new approach to consider the spatial variability by taking into account the correlation between the random samples generated for the strength of the concrete. On the other hand, we analyze the influence of the directionality of the seismic action. Other sources of uncertainty for the seismic action have not been considered because the parameters of the mechanism and the recorded acceleration are available. However, we found that buildings with the same structural typology were damaged at different levels. This fact was the starting point of the present work. The results obtained and presented herein show a good agreement between the observed and simulated damage which allows concluding that the proposed approach is a powerful numerical tool to predict the damage of structures due to earthquakes considering directionality effects.

2 Description of the archetype building

The buildings of the San Fernando neighborhood of Lorca damaged by the may 2011 earthquake, have the same typology, 5 stories, reinforced concrete structure, and are configured with columns and waffle slabs. Some of these buildings have been strongly damaged by the earthquake, but, it is relevant, that some of them were not damaged. All the buildings have been built by using the same structural blueprints, but, the difference between their damage levels was

considerable: some of them were damaged beyond the repair limit and others had no damage. A picture of a typical building of this group can be seen in Figure 1, which also shows some damaged columns at the ground floor.

This fact will be explained herein by considering the uncertainties associated with the mechanical properties of the materials and the effect of the directionality of the earthquake. The study is based on a structural model simulated from the original blueprints of the buildings which contain information not only on the geometry but also on the strength of the materials.



Figure 1 Buildings located in the San Fernando neighborhood of the city of Lorca, Spain, affected by the earthquake occurred on May 2011

Figure 2a shows a blueprint of the studied buildings. Figure 2b shows a plan view of the structural model. The axes shown in Figure 2b will be used as a reference for the angle of rotation in the calculations. From Figure 2, it can be observed that the building is weaker in the y direction (according to the axes indicated the figure).

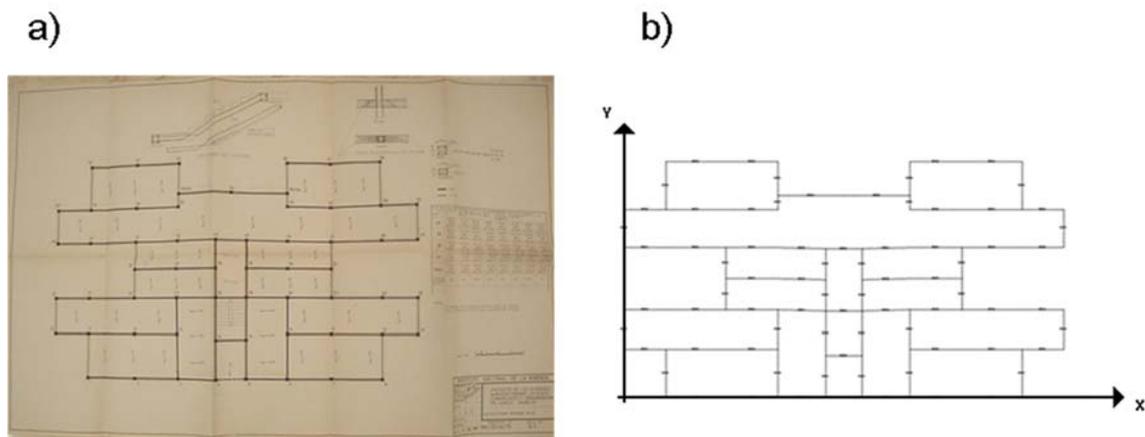


Figure 2 a) Blueprint of the building b) Plan view of the structural model

Figure 3 shows an aerial view of the buildings of the San Fernandoneighborhood. In this picture the buildings have been marked according to the damage caused by the earthquake, estimated by means of the expert opinion. Green color indicates that the building was not damaged; yellow color indicates that the damage is repairable and red color is used to identify the buildings

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which were most damaged. Notice that building marked with red color has been recommended for demolition. One important aspect to be mentioned is that the San Fernando neighborhood is not close from the seismological station where the earthquake was recorded and, therefore, due to topographical and geological conditions, among others, it is difficult to determine the acceleration and its orthogonal components which affect the San Fernando neighborhood. For this reason, the seismic action will be treated as a random variable whose only source of uncertainty is the directionality. For modeling issues, a 3D structural model used in the structural analysis of the buildings is shown in Figure 4. The software to perform the analyses was RUAUMOKO [20]. The modified Takeda model [21] was chosen among the hysteretic models available in the RUAUMOKO program to describe the mechanical behaviour of reinforced concrete.

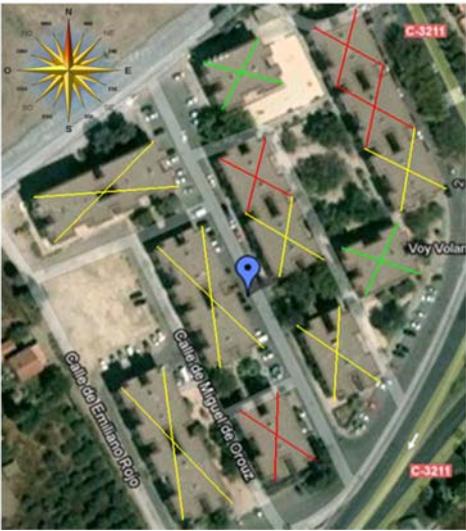


Figure 3 Plan view of the buildings located in San Fernando neighborhood affected by the Lorca earthquake. Green indicates that the building was not damaged; yellow indicates that the damage is repairable; red indicates that there is an imminent risk of collapse

As mentioned above, the buildings are composed of columns and waffle slabs; however, in the numerical model of the building, waffle slabs are approximated by beams with equivalent inertia. Yield surfaces are defined by the interaction diagram of the bending moment and the axial load of columns and the bending moment-curvature in beams. The nonlinear behaviour in shear was not been considered. Loads are applied following the recommendations of Eurocode 2 [22] for reinforced concrete structures. The Rayleigh damping model was used.

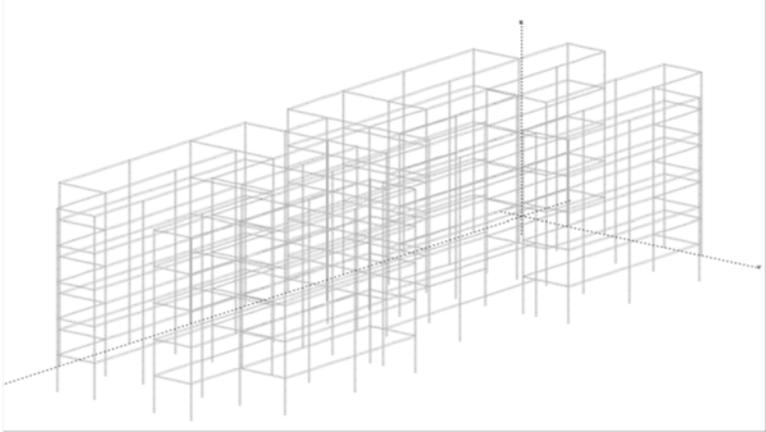


Figure 4 3D structural model of the building

3 Lorca, MAY 2011 earthquake

The magnitude of the Lorca earthquake was 5.1, meaning that it was a moderated earthquake. However, this earthquake caused 9 casualties and relevant economical losses. This fact highlights the high seismic vulnerability of the buildings located in that area, which is similar to that of other regions of Spain. The main issues related to this earthquake can be found in references [23, 24, 25]. The horizontal components of the earthquake are shown in Figure 5.

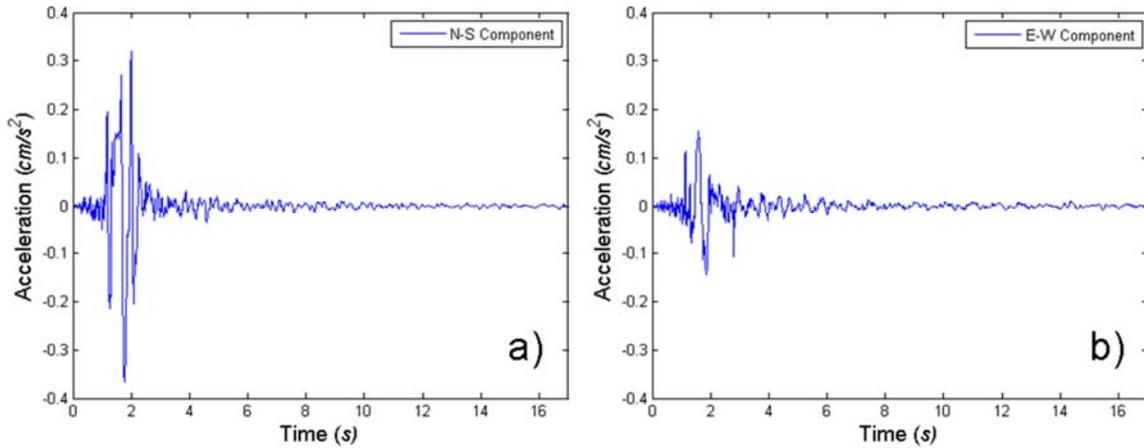


Figure 5 Horizontal components of the May 2011 Lorca earthquake

These accelerograms show high differences between the acceleration records in both direction. This fact indicates that a building belonging to the same structural typology can be more damaged if it exhibits less resistance in one direction and its azimuth angle makes this direction to coincide with the maximum acceleration of the earthquake.

For calculating the variation of the acceleration record due to the rotation of the horizontal components by an angle θ the following transformation is used [26]

$$\begin{pmatrix} \ddot{u}_{x(\theta)}(t) \\ \ddot{u}_{y(\theta)}(t) \end{pmatrix} = \begin{bmatrix} \cos(\theta) & \sin(\theta) \\ -\sin(\theta) & \cos(\theta) \end{bmatrix} \begin{pmatrix} \ddot{u}_x(t) \\ \ddot{u}_y(t) \end{pmatrix} \quad (1)$$

where $\ddot{u}_{x(\theta)}(t)$ and $\ddot{u}_{y(\theta)}(t)$ are the horizontal components of the accelerogram when rotated anti-clockwise with an angle θ , while $\ddot{u}_x(t)$ and $\ddot{u}_y(t)$ are the original components of the record. We have rotated the horizontal components of the accelerograms by an angle θ which varies from 0° to 180° . We considered increments of 1° for this variation and, then, for each angle, we calculate the response spectra of the rotated record. Figure 6 shows the acceleration response spectra obtained after rotating the horizontal components. It can be seen that the variation in terms of spectral acceleration is very high in the range considered. It indicates that the seismic response of a building, when subjected to an earthquake, strongly depends on its azimuth. For this reason, it is mandatory to include the directionality as a random variable in order to get reliable and accurate results. To get an idea of the large variation of the seismic input, the fundamental period of the studied buildings is about 0.8 seconds and, for this value, the spectral accelerations can vary from 0.072 g to 0.3748 g.

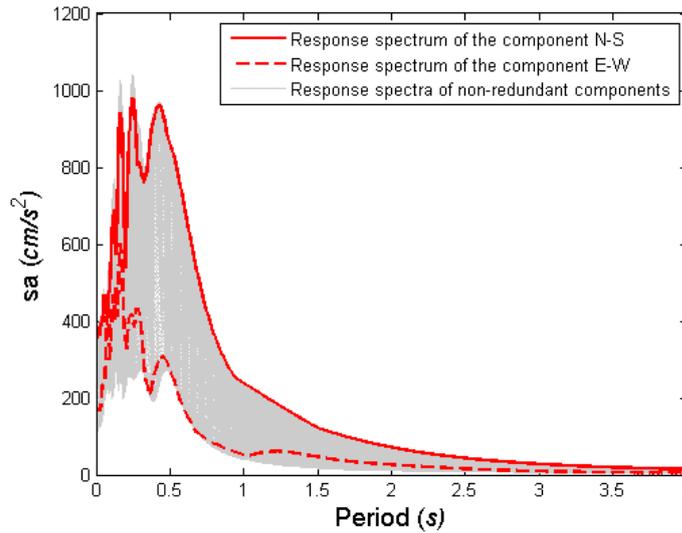


Figure 6 Acceleration response spectra obtained after rotating the acceleration record registered during Lorca earthquake

4 Uncertainties related to the mechanical properties of the materials

Another source of uncertainties of the seismic response is related to the mechanical properties of the materials. This has been studied by Vargas et al. [27] which considered the compressive strength of concrete, f_c , and the tensile strength of steel, f_y , as random variables. It is usually assumed that they follow a Gaussian probability distribution. In this article the characteristics values shown in Table 1 are considered, for f_c and f_y ; these data were taken from the original blueprints.

Table 1 Statistical moments of the mechanical properties of the materials considered herein

Variable	μ_x (kPa)	σ_x (kPa)
f_c	21000	2100
f_y	500000	50000

Other possible uncertainties, like those related to cracking and crushing of concrete, strain hardening and ultimate strength of steel, or the effect of slab participation, axial force variations on column strength, just to name a few, can be also included in the probabilistic structural analysis. However, in this article we consider only the structural uncertainties included in Table 1. Another important aspect to be considered when generating random samples of the mechanical properties of the materials is their spatial variability. This problem is related to the grade of correlation that must exhibit the random samples simulated for the structural model [28]. The study of the spatial variability aims at improving the quality of the random samples generated.

4.1 Hypothesis of total and null correlation

To analyze the spatial variability, at a first stage, we will consider two extreme cases of correlation, namely the *total correlation* and the *null correlation*. In the case of the *total correlation*, it is supposed that all the structural frame elements of the building, i.e. columns and simulated beams, have the same mechanical properties. That is, in order to perform a Monte Carlo simulations based on n

calculated structures, it is enough to generate n random samples of f_c and f_y . In the case of *null correlation*, it is supposed that the correlation of the mechanical properties samples, corresponding to the frame elements, is null. For each Monte Carlo simulation, it will be necessary to simulate as many random independent samples as columns and beams has the building. In order to analyze the influence of the total and null hypothesis of correlation 1000 NLDA are performed for each case. A Two-dimensional frame model is developed using the blueprints of the San Fernando neighborhood buildings. Figure 7a shows the part of the building selected to create the 2D framed structure. The input to perform the NLDA's is the N-S component of the Lorca Earthquake.

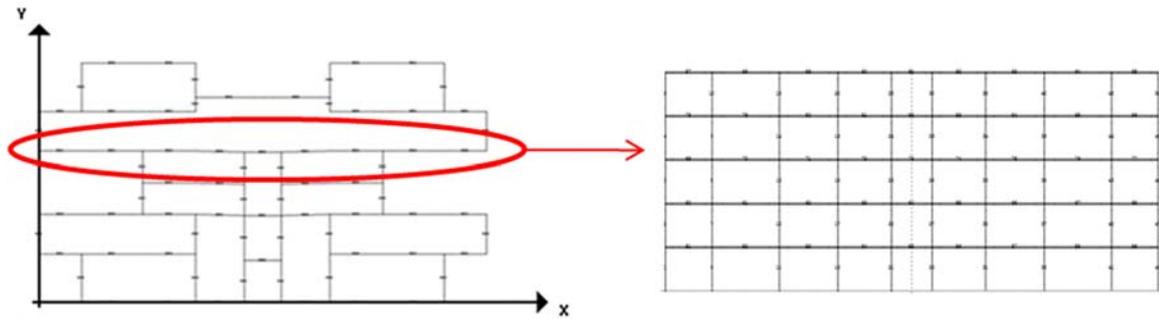


Figure 7 2D model used to analyze the influence of the spatial variability

The 2D model created has 50 columns and 45 beams; however, by the one hand, in order to generate random samples based on the *total correlation*, it is just necessary to generate 1000 samples of f_c and f_y . The correlation between the generated random variables is null but, as the generated sample is the same for all the structural elements, it can be stated that the correlation of the data for each run is total. On the other hand, in order to generate random samples for the null correlation case 95000 independent samples should be simulated and, for each run, one different sample is assigned to each structural element. After performing all the nonlinear dynamic structural analysis for each case, we calculated the damage index of Park and Ang [29] by using the following equation:

$$DI_E = \frac{\mu_m}{\mu_u} + \frac{\beta E_h}{F_y \mu_u \delta_y} \quad (2)$$

where μ_m and μ_u are the maximum and ultimate ductilities, respectively, and the subscript E stands for the element level damage index; β is a non-negative parameter which considers the effect of cyclic loading on the structural damage; E_h is the dissipated hysteretic energy; F_y is the yield load and δ_y is the yield displacement. The global damage index of the structure, DI , is the weighted mean of the damage of the elements, in which the weights are the ratios λ_i of the hysteretic energy dissipated by each element to the total hysteretic energy dissipated by the structure

$$DI = \sum_i \lambda_i DI_E \quad (3)$$

Vargas et al. [30] have used the damage index of Park and Ang to calculate damage curves for buildings by considering uncertainties. They found results consistent with the expected seismic damage for the studied structural typology. Accordingly, in order to analyze the NLDA results for each sample correlation hypothesis, we calculated the maximum displacement at the roof of the building for each run. Figure 8 shows the comparison between the relation of the damage index of Park and Ang to the maximum displacement at the roof, for each sample correlation hypothesis.

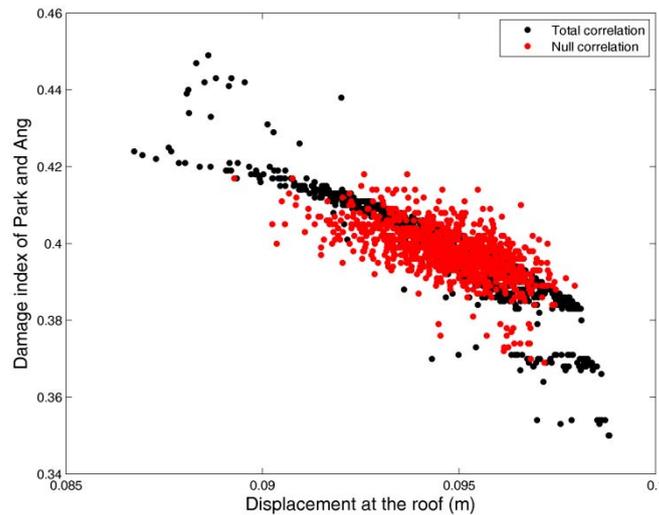


Figure 8 Damage index of Park and Ang as a function of the maximum displacement at the roof. Comparison of the total and null hypothesis for the sample correlation

Figure 8 reveals significant differences between the considered hypotheses. For the case of total correlation, the scattering and the correlation between the damage index and the displacement at the roof is higher than in the case of null correlation. A higher scattering was expected because supposing that all the structural elements have the same strength, despite of the fact that this is not realistic, implies to admit that all the structural elements may exhibit simultaneously extreme values. The latter consideration leads to extremely resistant as well as to extremely weak cases. It is worth noting that most of the seismic design codes prescribe the use of characteristic values by reducing the strength of the materials with a coefficient. This pseudo-probabilistic approach may produce extremely conservative results. Another important aspect affecting the results is the correlation between the damage index and the displacement at the roof, which is higher in the case of total correlation than in the case of null correlation. It is more reasonable a low correlation because of the nonlinearity of the structural response. For all these reasons we conclude that the null correlation hypothesis is more adequate than the total correlation hypothesis. Nonetheless, in the case of concrete, the supposition that the strength of the structural elements is completely independent is not realistic because, generally, in the construction process of buildings, the concrete used for columns of a certain storey proceeds from the same mixture. The same observation is made for the beams of a given storey. For this reason, it is common that the strength of the structural elements belonging to the same storey tends to exhibit a certain grade of correlation. Therefore, we generated the random samples of the strength of the concrete by taking into account these aspects.

Hypothesis of partial correlation

In order to consider the possible correlation that may exhibit the strength of concrete for the elements of one storey, it is necessary to establish a new correlation hypothesis. In this article, we propose the hypothesis that the correlation between the columns or beams of the same storey decrease with the distance. Thus, we propose to construct the correlation matrix for the strength of concrete simulated samples of one storey in the following way:

$$\rho_{i,j} = \begin{cases} i = j & \rho_{i,j} = 1 \\ i = j \pm k & \rho_{i,j} = 1 - \frac{k}{r} \end{cases} \quad (4)$$

where k is a number related to the position of a beam or a column belonging to the same storey of a 2D framed structure. For instance, for the 2D of Figure 7, the column of the first storey numbered as $k=1$ in, will be the first from left to right, the column numbered as $k=2$ of the first storey will be the second one going from left to right and so on. r is a coefficient associated to the rate of correlation between adjacent elements. Therefore, the correlation matrix for the columns of the first storey, which contains 10 columns, for a coefficient $r=10$, will be:

$$\rho_{i,j}(\text{columns}) = \begin{pmatrix} 1 & 0.9 & 0.8 & \dots & 0.1 \\ & 1 & 0.9 & \dots & . \\ & & 1 & \dots & . \\ & & & \dots & 0.9 \\ & & & & 1 \end{pmatrix}$$

The correlation matrix for the 9 beams of the first storey for a coefficient $r=10$, will be:

$$\rho_{i,j}(\text{beams}) = \begin{pmatrix} 1 & 0.9 & 0.8 & \dots & 0.2 \\ & 1 & 0.9 & \dots & . \\ & & 1 & \dots & . \\ & & & \dots & 0.9 \\ & & & & 1 \end{pmatrix}$$

Once the correlation matrix has been defined, the next step is to generate correlated random samples. As a first step, it is necessary to obtain the covariance matrix which is given by

$$\Sigma_{i,j} = \sigma_{x_i} \sigma_{x_j} \rho_{i,j} \quad (5)$$

where σ_{x_i} represents the standard deviation of the random variable x_i . The next step is to calculate the matrix Φ which contains the eigen-values of the matrix Σ (see eq. 5). Afterward, the covariance rotated matrix Ψ should be calculated by considering the following transformation:

$$\Psi = \Phi^T \Sigma \Phi \quad (6)$$

The diagonal terms of Ψ contains the rotated variances by taking into account the correlation which exist between the random variables. The squared root of the diagonal terms of the matrix Ψ are the rotated standard deviations

$$\sigma_y = \sqrt{\text{diag}(\Psi)} \quad (7)$$

σ_y is a vector which contains the standard deviations of the random variables in an orthogonal space rotated by considering the correlation matrix. Now, let μ_x be a vector which contains the expected values of the random variables x_i . In order to obtain the expected values μ_y in the rotated space, should be applied the following equation:

$$\mu_y = \Phi \mu_x \quad (8)$$

Now, by using the rotated expected values, μ_y , and the rotated standard deviations, σ_y , the next step consists in generating Gaussian independent random samples which can be stored in the matrix $Y_{i,j}$, being $i=1000$ the number of random simulations, and j the number of random variables. For simulating the random correlated samples of columns, for one storey of the 2D model depicted in Figure 7, $j=10$ and, for beams, $j=9$. Finally, the matrix of random correlated samples is calculated as follows:

$$X = Y \Phi \quad (9)$$

Similarly than in the case of total and null correlation, we have generated 1000 of groups of samples by following the hypothesis of *partial correlation* and, again, 1000 NLDA's have been performed for the structure of Figure 7 subjected to the N-S component of the Lorca earthquake. Figure 9 shows the comparison between the damage index of Park and Ang represented in function of the maximum displacement at the roof, for the 3 correlation hypotheses, i.e. *total*, *null* and *partial correlation*. It can be seen that very similar results are found when comparing the null and the partial correlation hypotheses. However, if we calculate the mean and the standard deviation for the analyzed variables, i.e. damage index and displacement at the roof, there are slight differences between both hypothesis (see Table 2).

Table 2 Characteristic values of the random variables analyzed. *tc*, *nc* and *pc* denote *total correlation*, *null correlation* and *partial correlation*

	DI_{tc}	DI_{nc}	DI_{pc}	δ_{tc}	δ_{nc}	δ_{pc}
Mean	0.3975	0.3977	0.3984	0.0948	0.0947	0.0946
Standard deviation	0.0131	0.0067	0.0067	0.0022	0.0013	0.0016
Coefficient of variation	0.0330	0.0167	0.0169	0.0228	0.0140	0.0167

Table 2 shows that the standard deviation of the damage index and of the displacement at the roof is higher for the case of *total correlation*. However, as it was explained above, the latter case is not realistic. The standard deviation calculated by considering the *partial correlation* tends to be slightly higher than the *null correlation*. This was expected because the probability of finding extreme values increases with the level of correlation. Due to the fact that the hy-

pothesis of *partial correlation* is more realistic this will be considered in this article when generating random samples for the concrete strength. Further studies may be conducted in order to analyze how the coefficient r of eq. 4 could be related to the standard deviation of the random dynamic response of buildings.

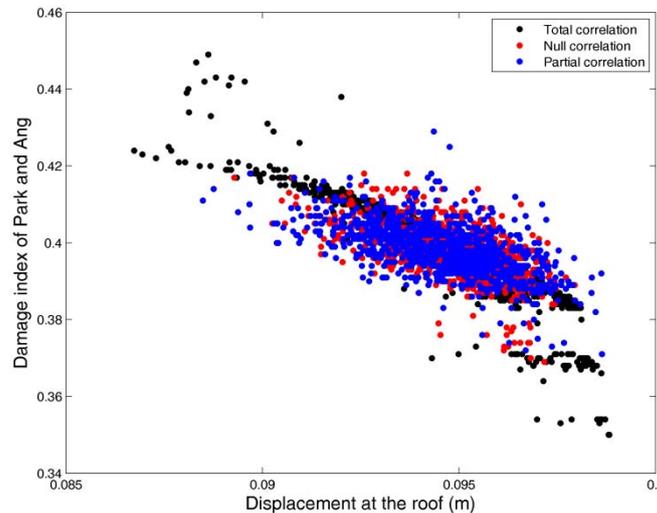


Figure 9 Damage index of Park and Ang as a function of the maximum displacement at the roof. Comparison of the total, null and partial hypothesis for the sample correlation

5 Probabilistic nonlinear dynamic approach

In order to consider the effect of the directionality of the earthquake upon the buildings studied, the system of axes shown in Figure 2b is used. Thus, for an angle θ equal to zero, the N-S component acts in the y axis and the E-W acts in the x axis. Once defined this aspect, a series of nonlinear dynamic analysis were performed by considering not only the variation of the angle of the seismic action respecting the building, but also the uncertainties associated to the mechanical properties of the materials. 3600 nonlinear dynamic analyses have been performed, in which the angle of rotation of the earthquake was treated as a random variable with a uniform-probability distribution in the interval $(0^\circ, 180^\circ)$. The mechanical properties of the materials have been considered as random using the *partial correlation* hypothesis. Once the NLDA's have been performed, the damage indices of Park and Ang have been calculated for each simulation case. Figure 10 shows the variation of the damage index as a function of the incidence angle of the seismic action and also shows the uncertainty associated to the mechanical properties of materials. As it was expected, the seismic damage depends strongly on the angle of rotation and on the uncertainties associated with the materials properties.

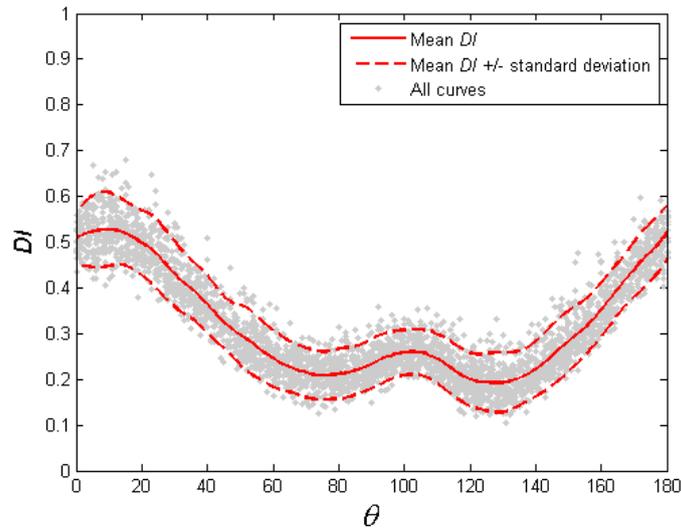


Figure 10 Variation of the damage index considering the angle of rotation of the earthquake and the uncertainties associated with the mechanical properties of materials

6 Discussion of the results

To evaluate the distribution of damage, it is convenient to group the results as in Figure 11, where histogram which exhibit the distribution of the damage calculated is shown.

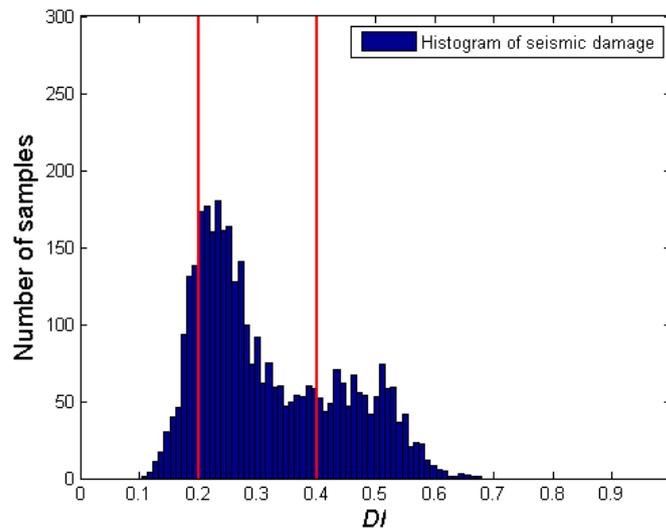


Figure 11 Histogram of the expected damage of Park and Ang

This histogram shows that the obtained distribution function has a bimodal shape. This is due to the fact that the expected damage, when the seismic action is maximum in the y direction, is greater than in the case when it is maximum in the x direction. This may explain why some buildings suffer more damage in one direction than in another. To make a comparison between the observed damage and the simulated one, it is necessary to establish some damage thresholds. Park et al. [31] assigned a qualitative description of the damage depending on the calculated value of the damage index. For example, they state that, if the damage index of a building is greater than 0.4, the repair cost exceeds the construction cost of the building; for lower damage, repair is economically viable. If the calculated damage index is less than 0.2, Park et al [31]

consider that damage is slight, corresponding to '*sporadic occurrence of cracking*'. These thresholds are important in the analysis of the results because, as it is shown in Figure 3, some buildings were not damaged and others were as strongly damaged as to be recommended for demolition.

On the one hand, if we analyze the histogram of Figure 11 according to the damage thresholds described previously (see the red lines), we can establish the percentages of number of buildings that suffered slight, moderate and extensive damage; it is assumed that extensive damage is not repairable and, therefore, the building should be demolished. This analysis can be done by counting the number of samples contained in each interval. This count shows that 511 samples presents a $DI < 0.2$, 2109 samples are in the range of $0.2 > DI < 0.4$ and for 980 samples $DI > 0.4$. Whereas the number of samples is 3600, the probabilities of occurrence of each damage state are 14.19%, 58.86% and 26.95% for slight, moderate and extensive damage, respectively. On the other hand, the damage assigned by experts after the 2011 Lorca earthquake (see Figure 3) indicate that 13.33% of the buildings had damage slight, 60% had moderate damage and 26.67% had extensive damage; in this last case, demolition was recommended. It can be seen a good agreement between the calculated and observed damage which highlights the importance of considering the uncertainties in the risk and vulnerability analysis of structures.

From the available data, it is difficult to establish which is the rotational angle to simulate the orthogonal components of the acceleration acting upon the San Fernando neighborhood. However, it can be appreciated in Figure 3 that the axes of some buildings are rotated approximately 90° respective to the other ones. Thus, for instance, if damage probability distributions are analyzed for two orthogonal angles, e.g. 40° and 130° , we obtain the probability density functions shown in Figure 12. The principal moments of these probability density functions are shown in Table 2. It might be supposed that 40° coincides with the more damaged direction and 130° coincides with the less damaged direction. Similar distributions to that shown in Figure 12 are obtained for almost all the rotational angles, except in the interval between 50° and 70° . Thus, it can be discarded that the acceleration components acting on the San Fernando neighborhood, will be compatible with those generated by a rotational angle within the later interval. However, this is merely a hypothesis; the theoretical basis on which it is established are beyond of the scope of this paper.

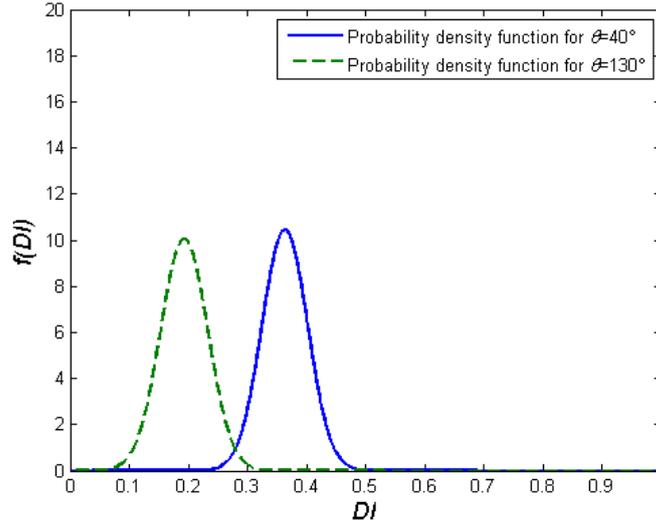


Figure 12 Probability density functions of damage for $\theta = 40^\circ$ and $\theta = 130^\circ$

Table 3 Statistical moments of damage for $\theta = 40^\circ$ and $\theta = 130^\circ$

θ°	μ_{ID}	σ_{ID}
40°	0.3634	0.0382
130°	0.1930	0.0397

7 CONCLUSIONS

One of the main objectives of the present study is to calculate the influence of the directionality of the ground accelerations of an earthquake on buildings located close to each other and belonging to the same structural typology. With this aim, we used the nonlinear dynamic analysis, also taking into account two kinds of uncertainties: i) related to the position of the buildings in terms of their azimuth and ii) related to the mechanical properties of the materials. We have used as a testbed a group of asymmetric buildings which were damaged at different levels by the earthquake occurred in May 2011 in Lorca, Spain. As the seismic action was known, it was not necessary to consider the uncertainties due to the earthquake-to-earthquake variation. Moreover, a study to analyze the spatial variability of the materials was conducted by generating random samples of the mechanical properties. We considered three hypotheses to analyze the spatial variability: *null correlation*, *total correlation* and *partial correlation*. We have concluded that the best hypothesis is that of partial correlation because it takes into account that the frame elements depending on their separation exhibit a different correlation. This means that the correlation between elements decreases with distance. For the seismic action, we have considered the rotational accelerations of the record and, by using the Latin Hypercube Method, we have simulated 3600 cases to estimate the occurrence probabilities of different damage grades. One of the major conclusions of this article is that the observed damage is very similar to the calculated one. Another important aspect is related to the relatively high computational cost to calculate the results. However, as mentioned above, the results are in good agreement with the observed damage. It is important to note that simplified methods for obtaining a damage index similar to the Park and Ang one are being developed [32] and they will allow obtaining similar results to the presented herein but with a reduced computational effort. Due to the

treatment of the uncertainties related to the seismic action and to the mechanical properties of the materials, the results obtained in this article can be extended to assess the behavior of others buildings of similar typology affected by the earthquake of Lorca. It is important to note that, if the angle with which the acceleration record should be rotated in order to simulate its effect in a given area is known, the results obtained can be extended to predict the distribution of the seismic damage with less uncertainty.

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