Abstract

Non-structural elements that are considered primarily sensitive to and subject to damage from inertial loading are classified as acceleration-sensitive elements. The response of acceleration-sensitive non-structural components in buildings is therefore directly affected by the floor acceleration demand that they experience during ground shaking. Reducing seismic damage to these elements is of primary importance not only for economic reasons, but also for maintaining the functionality of the building immediately after the earthquake. The purpose of this paper is to study the floor acceleration demand variation along the height of concrete frame buildings and how the sophistication of the structural modeling can be reduced in such estimation. In fact, development of probabilistic seismic demand models, that usually involve the use of methods such as the incremental dynamic analysis, the cloud analysis, and multiple stripe analysis requires a large number of non-linear analyses of the structure to be run. To reduce the numerical effort, a simplified methodology based on the modal pushover analysis procedure will be proposed and used for the characterization of floor spectra. To this purpose the seismic response of a selected case study, consisting in a six-storey three-bay frame of a reinforced concrete building, dimensioned to be representative of an existing structure designed according to a past seismic code (using the one in force in Italy between 1996 and 2008), will be analyzed. The estimates of peak absolute acceleration and response spectra at each floor level will be compared and contrasted.

Keywords: nonstructural components, probabilistic seismic demand model, correlated engineering demand parameters, interstory drifts, floor response spectra.

1 Introduction

The continued functionality of critical facilities in the face of natural or manmade disasters is essential to community resilience. Performance-based seismic
assessment of these facilities is based on the evaluation of the probability of functionality loss in a period of interest. Functionality can be exposed to earthquake-induced damage to structural as well as non-structural components (NSC). Once the state of damage to all components is known, assessment of residual functionality requires the evaluation of a fault tree (FT) for the function, with basic events consisting of component failures. The FT depends on the topology and properties of the system performing the function. Within a PBEE framework [8, 4], the assumption of conditional independence allows the subdivision of the performance assessment into a sequence of steps: 1) hazard; 2) structural response given seismic intensity; 3) damage of components given structural response; 4) functionality given damage. One such model is needed for loss assessment studies, where Step 4 is simply replaced by a summation of economic value of components’ damage over the structure [11]. In the latter case usually components are classified into so-called performance groups, which lump together all components that are similarly affected by the same response quantity, or engineering demand parameter (EDP). NSCs are usually classified into drift-sensitive (e.g. piping) and acceleration-sensitive (e.g. equipment resting on floors or hanging from ceilings) components. The latter may be rigid or flexible. A complete seismic demand model must include both interstory drifts and floor spectra, or at least drift and some floor acceleration parameters [2].

This paper discusses issues related to the estimation and use of a multivariate seismic demand model for NSC in terms of drift and floor acceleration spectra. A model to generate realizations of floor response spectra based on few parameters derived from the full PSDM is also proposed.

2 Methodology

A detailed estimation of the functionality loss requires a simulation approach and a relatively large number of realizations. In this case, the evaluation of the structural response and state of system functionality in the same inelastic response history analysis (IRHA) it is not practicable because the number of the latter would be too large to be affordable. Thus, the common approach is to exploit a reduced number of IRHAs to obtain a reasonable set of responses to establish a surrogate model to be used in a following Monte Carlo simulation of systems states [8, 11].

The PSDM adopted in this work is a multivariate lognormal model, such as that in [11]. The structural response quantities are interstory drifts and floor acceleration spectral ordinates at selected periods, or floor response spectra (FRS). The use of a lognormal distribution for drifts is well established, and there is support for its use also for acceleration responses [11, 9]. The FRS are calculated from floor accelerations recorded in the IRHA of the structure (alone). Structure-component interaction effects, in fact, are assumed negligible, which is acceptable in the case the mass of the component is significantly smaller than that of the structure (less than 1%, based on the results of the investigations carried out by [12]). The use of elastic FRS for acceleration-sensitive NSCs rests on the assumption that both components and their supports and attachments to the structure remain in the linear elastic range.
The PSDM is used in a PBEE framework to evaluate the mean annual frequency of loss of functionality \( \lambda \), using the total probability theorem expression:

\[
\hat{\lambda} = \int_S \int_d p(F|d) p(d|S) dd\lambda_s(s) = \int_S p(F|S) d\lambda_s(s)
\]

where:

- \( p(F|d) \) is the probability of loss of functionality given a set of NSC demands \( d \) (which requires evaluation of the corresponding NSC capacities from their respective fragility functions and evaluation of the FT)
- \( p(d|S) \) is the joint density of NSC demands given seismic intensity \( S \), output of Step 3 (the multivariate lognormal PSDM)
- \( p(F|S) \) is the outcome of Step 4, and is the integral over the demands \( d \) of the product
- \( p(F|d)p(d|S) \), carried out by Monte Carlo-like simulation

State-of-the-art correct evaluation of the probability density of \( d \) conditional on intensity \( S \) requires the use of a set of recorded ground motion time-series selected to reflect the conditional distribution of ground motions given that \( S = s \). This can be done according e.g. to the selection procedures in [1], [7], and [6]. The multiple-stripe analysis [5] is adopted herein, as it provides the easiest means to account for the variations in ground motion properties with the conditioning intensity measure \( S \) in the total probability theorem expression above. In the following, unless otherwise specified, reference is implicitly made to one of the intensity levels (i.e. a single stripe) \( S = s \).

At each intensity \( S = s \), the adopted PSDM is completely specified through the two \( N \times 1 \) vectors \( \mu ln \) and \( \sigma ln \) of the means and standard deviations of the logarithm of the responses, and through the \( N \times N \) correlation matrix \( R_{ln} \) of the pair-wise correlation coefficients among the logarithms of the responses.

The length \( N \) of the response vector can be expressed as:

\[
N = n_f \times \left(1 + n_T \times n_\xi \right)
\]

where:

- \( n_f \) is the number of floors (and multiplied by one yields the number of interstory drifts of interest, for 2D problems
- \( n_T \) is the number of periods at which the FRS is evaluated
- \( n_\xi \) is the number of component damping ratios of interest

The use of multiple component damping ratios may be needed when, as already mentioned, type and properties of the components are not known at the structural analysis stage.

The number of periods used to discretize the FRS has only computational implications, but this is a relatively minor issue since the FRS are elastic, thus \( n_T \) can be in principle as large as desired. On the other hand, using a limited number of spectral ordinates to describe the full spectrum is also an option, being attractive especially in those cases where both \( n_f \) and \( n_\xi \) are large. Other studies have used a
limited number of spectral ordinates to characterize the FRS [2], introducing peak component acceleration (PCA) demands in three period ranges: peak floor acceleration region (PFAR, for the component vibration period $T_C = 0$ s), short period region ($SPR, 0 < T_C < 0.5T_1$) and fundamental period region ($FPR, 0.5T_1 < T_C < 2.0T_1$). In that study, however, PCA demands in each period range are characterized independently, without consideration of correlation and the final outcome is a set of (marginal) uniform component hazard spectra obtained combining the seismic hazard curve with the PCA distributions. The latter uniform hazard spectra cannot be used for Monte Carlo simulation since they would not provide probabilistically consistent sets of components’ damage states to be used for systemic evaluation. Further, FRS ordinates for components at periods away from the fundamental one or a lower period of the building may have considerably lower acceleration demand.

![Figure 1: Model proposed to generate approximate realizations of FRS.](image)

Instead of using a set of $(n_f \times n_T \times n_\xi)$ spectral ordinates, the FRS can be alternatively simulated with the proposed model shown in Figure 1. According to this model, the acceleration demand $S^z_{a,NSC}$ of NSC located at different floor levels of the building, and characterized by different periods of vibration and damping ratios, can be approximately predicted by means of few parameters by using the following equation:

$$S^z_{a,NSC,approx} (T_{NSC}, \xi_{NSC}, f) = \begin{cases} \max \left( \alpha^e_{a,f}, \eta(\xi_{NSC}) \cdot \alpha^e_{a,f} e^{-\frac{\text{PCA}}{T_{NSC} T_{f} \Psi_{f}}} \right) & 0 < T_{NSC} < T_{n_m} \\ \max \left( \eta(\xi_{NSC}) \cdot \alpha^e_{a,f} e^{-\frac{\text{PCA}}{T_{NSC} T_{f} \Psi_{f}}} \right) & T_{n_m} < T_{NSC} \end{cases}$$

(3)

$$\alpha^e_{a,f} = \text{PCA}_{T_{f}} = \max_{T_{NSC} \in [\text{min}(\text{PCA}), \text{max}(\text{PCA})]} \left( S^e_{a,NSC} (T_{NSC}, \xi_{NSC} = 2\%, f) \right)$$

(4)
\[ \beta_{i,f}^s = \arg \min_x \left( \sum_{\min(T_{i,\text{region}}) < T_{NSC} < \max(T_{i,\text{region}})} \left[ \alpha_{i,f}^s e^{-\frac{\pi f_{NSC}}{T_{NSC}}} - S_{a,NSC}^{s} \left( T_{NSC}, \xi_{NSC} = 2\% \right) \right] \right)^2 \] (5)

\[
\min(T_{i,\text{region}}) = \begin{cases} 
0 & i = 0 \\
0 & i = n_m \\
\ldots & \\
\exp(0.5\log(T_i \cdot T_{i})) & i = 2 \\
\exp(0.5\log(T_1 \cdot T_{2})) & i = 1 
\end{cases}
\] (6)

\[
\max(T_{i,\text{region}}) = \begin{cases} 
0 & i = 0 \\
\exp(0.5\log(T_{n_m-1} \cdot T_{n_m})) & i = n_m \\
\ldots & \\
\exp(0.5\log(T_i \cdot T_{z})) & i = 2 \\
\max(T_{NSC}) & i = 1 
\end{cases}
\] (7)

\[ \eta(\xi_{NSC}) = \sqrt{\frac{2}{\xi_{NSC}}} \] (8)

and where: \( S_{a,NSC}^{s} \) is the approximated value of the (pseudo) acceleration of the NSC produced by a seismic intensity level \( S = s \); \( T_{NSC} \) is the period of vibration of the NSC; \( \xi_{NSC} \) is the NSC damping ratio (expressed as a percentage); \( f \) is the floor level where the NSC is located; \( T_i \) is the \( i \)-th period of vibration of the structure; \( T_{i,\text{region}} \) is the period region of the FRS containing the \( i \)-th period of vibration of the structure; \( n_m \) is the number of (considered) significant modes of vibration of the structure; \( \eta \) is a damping correction factor (different from 1 for \( \xi_{NSC} \neq 2\% \)).

In the case this proposed model is used to generate the FRS, the number of parameters which characterizes the PSDM becomes

\[ N = n_f \times (1 + n_p) \] (9)

in which

\[ n_p = 1 + 2 \times n_m \] (10)

where: \( n_p \) is the number of parameters of the proposed model (consisting in \( \alpha_{0,f}^s \), \( \alpha_{i,f}^s \), and \( \beta_{i,f}^s \)).

Feasibility of a further reduction of the number of parameters needed to describe the FRS in the case the law for prediction the variation of \( \alpha_{i,f}^s \), and \( \beta_{i,f}^s \) with the floor level is assumed a-priori is discussed later.
Once the vectors $\mathbf{\mu}_\ln$ and $\mathbf{\sigma}_\ln$ and the correlation matrix $\mathbf{R}_{\ln\ln}$ for each intensity level $S=s$ have been established based on the results of the IRHAs, simulation of any number of realizations of the NSC demand vector proceeds with the well-known formula to generate a multivariate lognormal vector from a vector of standard Normal variables:

$$d = \exp \left( \mathbf{\mu}_\ln + \text{diag}(\mathbf{\sigma}_\ln) \mathbf{L} \mathbf{u} \right)$$  \hspace{1cm} (11)

where:
- $\text{diag}(\mathbf{\sigma}_\ln)$ is the $N \times N$ diagonal matrix with diagonal $\sigma_{\ln}$
- $\mathbf{L}$ is the lower triangular matrix resulting from the Cholesky decomposition of the correlation matrix of the logarithms $\mathbf{LL}^T = \mathbf{R}_{\ln\ln}$
- $\mathbf{u}$ is a $N \times 1$ vector of standard Normal variables

3 Illustrative Example

3.1 Structural model

The methodology is illustrated by means of 6-storey reinforced concrete plane frame dimensioned so as to be representative of an existing structure designed according to a past seismic code (as the one in force in Italy between 1996 and 2008). Information on the span length and the story height of the frame, and details on cross-sections dimensions and reinforcement of the structural members are given in Figure 2. The structure is supposed to be located in a high seismic zone in Italy.

![2D frame](image)

### Materials:

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### Frame dimensions:

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<td>600</td>
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<td></td>
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</tbody>
</table>

Figure 2: Studied reinforced concrete structure.

The model used to analyze the frame is built in OpenSees [13] with beams and columns modelled with elastic beam elements connected in series to zero-length nonlinear rotational springs located at members’ ends for representing plastic hinges. The material model used for the nonlinear springs is the hysteretic material of OpenSees, with the pinch and the damage parameters set to have a peak oriented behavior and no in-cycle degradation, respectively. A Rayleigh damping proportional to the mass and the initial stiffness matrix (with the rotational springs
being not included) is considered, with coefficients calibrated to provide a 2% damping at the first and third mode periods. Second-order effects due to geometric nonlinearity are accounted for through p-delta transformations.

The first three periods of the frame model are 1.3 s, 0.44 s and 0.25 s, with corresponding participating mass equal to 80%, 11%, and 3%. In Figure 3, the pushover curve obtained with a load vector proportional to the inertia force distribution of the first mode of vibration of the structure is reported.

![Figure 3: Results of a pushover analysis of the frame. The pushover curve is expressed in terms of base shear $V_{base}$, normalized with respect to the total weight $W$, and roof drift ratio RDR.](image)

### 3.2 Ground Motions

In this study, seismic hazard is represented with ground motion records selected using the algorithm of Bradley [1] based on the generalized conditional intensity measure approach [18]. The vector of intensity measures adopted in the selection procedure consists in a set of (pseudo) acceleration spectral ordinates $S_a$ (computed at 14 different periods of vibration $T_i$, using a damping ratio equal to 5%), and the Arias Intensity $AI$ [21]. Among the considered spectral accelerations, that corresponding to a vibration period equal to 1 s is used as the conditioning intensity measure (i.e., $S = S_a (1s)$). Even if alternative intensity measures than the spectral acceleration were found to be good predictors for seismic demand on non-structural components (especially for the case of floor accelerations, as shown e.g. in [23, 24, 25]), $S_a$ is selected in this study because of its computability (i.e., availability of ground motion prediction equations, and correlation equations as well). The prediction equations of Boore and Atkinson [19] and Campbell Bozorgnia [19] are used to predict the $S_a(T_i)$ and $AI$ values, respectively, while the equation of Baker and Jayaram [22] and that of Campbell and Bozorgnia [19] is used for the prediction of the $S_a(T_i) - S_a(T_j)$ correlation and the $S_a(T_i) - AI$ correlation, respectively.

In order to account for site effects, records are selected by assuming rock conditions, and then modified through a site-response analysis. In this case, a linear filter
representing a uniform soil layer overlying a half-space of elastic rock is used to modify the records. Seismic hazard is estimated assuming the frame to be located in Lamezia Terme (Italy). Three seismic intensity levels, corresponding to a return period equal to 100, 500 and 2500 years, are used in the multiple-stripe analysis. Through the analysis of these three stripes, the variation of the PSDM properties with the nonlinearity level of the structural response is evaluated. Studies have shown, in fact, that FRS significantly change both in amplitude and shape when the structure gets into the nonlinear range (e.g., see [26, 27, 17]).

4 Results

In the considered case study, the response vector used to develop the PSDM for the NSCs consists in \( n_f = 6 \) interstory drifts, and, for each floor level, \( n_T = 301 \) ordinates of floor spectral accelerations (corresponding to periods uniformly distributed in the range 0-3 s) calculated for \( n_\xi = 10 \) damping values (ranging from 0.5% to 5%). Thus, the length \( N \) of the response vector is equal to 18066. It is worth noticing that the number of component periods \( n_T \) used to calculate the floor spectral accelerations, which is quite large, was estimated so that to accurately identify the maximum values of the FRS (whose shape is significantly peaked, especially in the linear range of response of the structure). In the following figures, where FRS and PCA plots are shown, unless otherwise specified, reference is implicitly made to the considered case of \( \xi_{NSC} = 2\% \). Figures 4-5 plot the marginal distributions of interstory drifts (normalized by the story height) and floor response spectra (at selected floor levels) obtained for the three considered earthquake intensity levels. It is interesting to observe that, for the case of the interstory drifts, the seismic demand distribution, within the building, slightly changes in shape with the variation of the earthquake intensity. For the case of the floor response spectra, instead, the seismic demand distribution changes.

Figure 4: Percentiles of the marginal distribution of the interstory drift ratio IDR (i.e., interstory drift normalized by story height) obtained for the three considered sets of records (corresponding to different values of the earthquake return period \( T_R \)).
With the increase of the earthquake intensity, the acceleration response in the short period region (SPR) of the spectra increases, while in the fundamental period region (FPR) the spectral acceleration initially increases and then slightly decreases. As a consequence, in some cases (e.g., see at the roof when $T_R = 2500$ years), the peak values of the FRS are recorded in the SPR instead of the FPR. Therefore, in the case of acceleration-sensitive NSCs is very important to consider several seismic intensities to obtain an accurate estimate of the demand in terms of both level and distribution within the building. Correlation among drifts and component accelerations is much lower than the correlation between the drifts at adjacent stories, and becomes almost negligible for high seismic intensity levels. This means that in drift- and acceleration-sensitive NSCs demand tends to become uncorrelated when the structure enters the nonlinear range of response.

Figure 5: Percentiles of the marginal distributions of 2%-damped FRS ordinates calculated at two different floor levels of the structure. $T_{NSC}$ and $T_i$ denotes the component period and the fundamental period of the structure, respectively.

The correlation coefficients among FRS ordinates which correspond to NSCs located at the same floor level, are reported in the plots of Figures 6 and 7. By comparing the plots of Figure 6 and Figure 7, it can be observed that, similarly to what found for the case of the drifts, correlation decreases with the increase of the earthquake intensity. In particular, in the nonlinear range of the structural response, only when $T_{NSC} < T_2$ and $T_{NSC} > T_1$ the correlation remains significant.
Figure 6: Correlation coefficients among the logarithm of FRS ordinates corresponding to NSCs located at the same floor level. $T_R = 100$ years set of records. Red lines define the period values of the first three modes of vibration.

Figure 7: Correlation coefficients among the logarithm of FRS ordinates corresponding to NSCs located at the same floor level. $T_R = 2500$ years set of records. Red lines define the period values of the first three modes of vibration.
The weak correlation observed in the $T_1 - T_2$ region, which decreases when the difference in the components’ period increases and when the components are located at different floor levels, can be explained by the following considerations: when $T_1 < T_{\text{NSC}} < T_2$, the response of the NSCs is not dominated by a single mode of vibration of the structure only, but by both the fundamental and the higher modes of vibration; the correlation among the responses to the earthquake of two different modes of vibration of the structure rapidly reduces when the difference in the periods of vibration of the two modes increases (e.g., see the results of the study of Baker and Jayaram [22] on the correlation of spectral acceleration values); the contribution to the acceleration response of the building of the different modes of vibration varies with the considered floor level. Once the PSDM is built, realizations of the NSC demand vector can be simulated by using Equation (11). As expected, when the spectral ordinates are assumed uncorrelated, the shape of the FRS is much more peaked; as a consequence, depending on the considered value of the component period, the seismic demand of the NSC can be significantly overestimated or underestimated.

5 Proposed model to generate FRS

In alternative to the full set of spectral ordinates, a reduced number of parameters can be used to build the PSDM in the case the FRS are simulated with the proposed model described in Equation (3). In Figure 8, the predictive ability of the damping correction factor $\eta$ is evaluated by comparing the $\eta$ values obtained with Equation (8) with the (mean) PCA values of the FRS derived from the results of the IRHA. In order to compare $\eta$ with the PCA demands, the latter, calculated for different $\xi_{\text{NSC}}$ values, are then normalized with respect to the PCA values obtained for $\xi_{\text{NSC}} = 2\%$. Three period regions corresponding to the period values of the first three modes of vibration of the structure are considered to calculate PCA ($n_m = 3$). By observing the plots of Figure 8, it can be noted that the variation of the PCA with the component damping ratio does not significantly change with the considered period region, earthquake intensity, and floor level where PCA is calculated.

Except for few cases (i.e., for low values of $\xi_{\text{NSC}}$, and the earthquake intensity corresponding to $T_R = 2500$ years), the predictions obtained with the proposed model perfectly match the actual normalized PCA values. In the case the proposed model to generate the FRS is used, the number of parameters needed to develop the PSDM can be further reduced if the following equations are used to estimate the variation of $\alpha_{i,f}$ and $\beta_{i,f}$ with the floor level

$$\alpha_{i,f} = \alpha_{i,\text{roof}} |\phi_i|$$ (12)

$$\beta_{i,f} = c$$ (13)

where: $\phi_i$ is the shape of the $i^{th}$ mode of vibration of the structure (normalized with respect to the displacement at the roof level); $c$ is a constant value assumed for $\beta$ which does not change with the considered floor level and earthquake intensity.
Figure 8: Damping correction factor $\eta$ (defined as PCA divided by $\xi_{NSC}$ for $\xi_{NSC} = 2\%$) for different period regions $T_i$: comparison between mean values obtained with the sets of records, and predicted values using a proposed model.

In Figure 9, the distribution in elevation of the (mean) PCA obtained from the analysis is compared with that predicted using Equation (12). At the different floor levels, the values of both the actual and the predicted PCA are normalized with respect to the PCA value calculated at the roof (the actual, and the predicted value, respectively).

It is interesting to observe that the distribution in elevation of the normalized PCA does not significantly changes with the increase of the earthquake intensity. This is due to the fact that the studied building is regular: because of this, while the contributions to the response given by the different modes of vibration of the structure change in intensity getting in the nonlinear range, their distribution within the structure remains nearly unchanged.
As a consequence, for each period region, the distribution in elevation of the normalized PCA demands matches the shape of the corresponding mode of vibration of the structure. Actually, it can be noted that in the case of the $T_1$ and the $T_2$ region, the match is almost perfect. In the case of the $T_3$ region, instead, there are some differences. The latter are explained by the fact that in this illustrative example only three period regions have been considered to calculate the PCA demands. However, for some of the used records, the peak of the FRS in the SPR is produced by the fourth mode of vibration of the structure rather than by the third one. By looking at Equation (3), it is clear that $\beta$ is the parameter which quantifies, in the $i^{th}$ period
region, how much the peak value of the FRS in that region exponentially decreases with the increase of distance from the period value of the $i^{th}$ mode of vibration of the structure. The higher is the value of $\beta$, the more significant is the decrease of the PCA demand from the peak. On the basis of the obtained results, it can be noted that $\beta$ in general changes value with the considered period region and earthquake intensity, but remains in many cases almost constant with the floor level.

Based on this observed trend, in order to reduce the number of parameters which define the PSDM, $\beta$ can be approximately assumed constant with the floor level, equal for example to the estimated at the roof. A good fit with the data is obtained in this case study also by assuming $\beta$ equal to 2.

In Figure 10, a realization of FRS generated with the full PSDM (which includes the set of all the considered $n_f \times n_T \times n_\xi$ spectral ordinates), and with the proposed model are shown. It can be observed that even when a reduced number of parameters is used in the proposed model, the match with the realizations generated with the full PSDM is very good, especially for the case in which the equation for the approximate estimation of $\alpha_{i,f}$ only is used.

Figure10: Comparison between the 84$^{th}$ of FRS obtained with the full PSDM and with the proposed model to generate approximate realizations.
6 Conclusions

This paper proposes a probabilistic seismic demand model (PSDM) for the seismic evaluation of nonstructural components in critical facilities. The parameters used in the model to measure seismic demand are interstory drifts, and floor acceleration spectral ordinates at selected component periods and damping ratios. A multivariate lognormal distribution is adopted for the demand vector. A model is also proposed to generate realizations of floor response spectra based on few parameters. When the latter model is used, the number of parameters describing the PSDM is significantly reduced.

A 2D frame of a reinforced concrete building is used to illustrate the implementation of the PSDM. The reported results of the case study are used in the paper to show the importance of accounting for the correlation among the components of the demand vector when the PSDM is developed. It is shown that, in the case of regular structures, correlation decreases with the increase of the earthquake intensity. In particular, interstory drift- and floor acceleration spectral ordinates tend to become uncorrelated when the structure enters the nonlinear range of response. About interstory drifts, only the correlation among those which correspond to adjacent stories remain significant. In the case of floor response spectra, correlation is weak for those ordinates lying in the region which is in between the periods of the fundamental and the higher modes of vibration of the structure. In this period region, correlation decreases when the difference in the components’ period increases and when the components are located at different floor levels.

The variation of the peak component accelerations (PCA) with the component damping ratio is negligible when different period regions, earthquake intensities, and floor levels are considered. In addition, its distribution in elevation does not considerably changes with the increase of the earthquake intensity, and matches almost perfectly with the shape of that mode of vibration of the structure with period in the region where PCA is calculated. The decrease of the spectral ordinates value at periods close to the period value where the PCA is recorded, changes in general with the considered period region and earthquake intensity, but remains in many cases almost constant with the floor level. The results of comparisons between simulations obtained with the full PSDM and the proposed model to generate floor response spectra showed the efficiency of the latter in the prediction of the nonstructural demand of acceleration-sensitive components.

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