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ESTIMATION OF THE DYNAMIC RESPONSE OF BUILDINGS THROUGH TRANSFER FUNCTIONS. A PROBABILISTIC APPROACH

Rodolfo J. Tirado-Gutierrez[1], Yeudy F. Vargas-Alzate[2], Ramon Gonzalez-Drigo,[3] Ana M. Zapata- Franco[4]

[1,2,4] Department of Civil and Environmental Engineering. Polytechnic University of Catalonia (UPC), Jordi Girona, 1-3, 08034, Barcelona (Spain)
rodolfo.javier.tirado@upc.edu; yeudy.felipe.vargas@upc.edu; ana.maria.zapata@upc.edu

[3] Department of Structural Engineering. Polytechnic University of Catalonia (UPC), Jordi Girona, 1-3, 08034, Barcelona (Spain)
jose.ramon.gonzalez@upc.edu

Abstract

The significant number of seismic ground motion records to be considered when designing or assessing civil structures is a common restriction. This is because the large computational time involved in the calculation of the nonlinear dynamic response of complex multi-degree-of-freedom system. There are several strategies to overcome this limitation; however, the reliability estimation of the analyzed systems can be compromised. This research is focused on developing a simplified methodology to obtain a probabilistic and reliable estimate of the seismic response of buildings. To do so, a set of transfer functions associated to the dynamic response of a building model are obtained. Then, an optimal transfer function is identified as the one maximizing the prediction of engineering demand parameters (EDPs) when the structure is subjected to a large set of ground motions records. Results show that using a reduced number of records allows developing an enhanced strategy to obtain reliable results in terms of the main statistical moments of the EDPs. This increased capacity to analyze complex systems in an affordable time has important consequences in the identification of optimal designs in terms of material-performance relationship, as well as in the estimation of the expected risk.

Keywords: *Transfer function, dynamic non-linear analysis, spectral analysis, fragility curves, reliability.*

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1 INTRODUCTION

During the last decades, a great variety of investigations has been carried out focused on the estimation of the dynamic response of civil structures in a non-linear regime. This is because in several regions of the planet the seismic risk is still very high. In fact, it cannot be affirmed that there is an area with zero seismic risk. For this reason, the design of earthquake resistant structures has been, and continues to be, one of the main challenges of structural engineering. However, epistemic and random uncertainties appear in all current methodological developments that seek to estimate the seismic effect on the structures.

Depending on the level of sophistication in modelling, the uncertainty varies. It means, the more sophisticated the type of analysis, the lower the level of uncertainty associated with the modelling. In this respect, there are several ways to assess the seismic behavior of a structure, ranging from the Equivalent Horizontal Force method (EHF), through the Response Spectrum Method (RSM), nonlinear static analysis (pushover analysis), to the nonlinear dynamic analysis (NLDA) [1]. The latter is considered the most reliable numerical procedure to estimate the response of a structure subjected to seismic actions. It should be noted that current seismic design practice admits a certain degree of damage, for which the use of the NLDA is essential to properly capture the statistical variability of a structure in a non-linear regime[2].

From the NLDA, it is possible to calculate, over time, the evolution of stresses, deformations and dissipated energy, among other variables, in any element that is part of the resistant structure [3]. This type of analysis allows incorporating characteristics of the nonlinear dynamic response of structural elements such as plasticization, strength degradation and loss of stiffness, among many others. The results obtained through the NLDA are considered as a reference for any simplified analysis and evaluation method used to obtain the seismic response of a structure [4]. However, the computational cost involved has been one of its main drawbacks. Hence, this article focuses on developing a method that allows estimating, in a fraction of time, engineering demand parameters (EDP) considering uncertainties in the seismic action. EDPs of special interest are the story drift, base shear, roof displacement, accelerations, velocities and story displacements, among others. As a case study, a building previously designed through the NLDA is analyzed. The proposed building follows the requirements of the Colombian seismic-resistant design regulations, NSR-10 [5] and the specifications of the American Concrete Institute, ACI-318 [6].

The proposed method will be identified hereinafter as ROM (Reduced Order Methodology) and it is mainly based on the development of transfer functions. Its main basis is the spectral analysis using the Fourier spectrum of signals that represent both seismic action and structural response. [7]. This approach allows complex systems to be analyzed in a short time. It is also expected that the method proposed here will serve to improve the efficiency of seismic risk estimates, thus increasing the protection of urban environments.

2 CASE OF STUDY

2.1 Building description

The building under study is located in the city of Pereira, Colombia. Its approximate height is 73.10 meters (23 stories), measured from the base to the roof. The structural system (combined) is composed by connected walls, columns and reinforced concrete beams. These elements transmit the loads of the building to the supports, guaranteeing balance, stability, as well as the compatibility of deformations with each other. The building has been designed based on the NLDA, considering the seismic hazard of the region according to the NSR-10 (AIS, 2010). Note that NLDA demands that the seismic hazard is represented by means of a group of compatible accelerograms with the design spectrum. That is, the seismic hazard is considered using accelerograms whose response spectra are compatible with the design spectrum.

In order to estimate the dynamic response of the structure, a three-dimensional model has been developed using Frame and Shell type elements. Frame elements are used to model beams and columns; membranes to model floors; and for the structural walls shell elements have been used. For the purposes of the model, and according to the type of foundation and the stiffness and capacity of the soils that serve as support, the building was considered to be embedded in the foundation. Figure 1, Figure 2 and Figure 3 show the 3D model, the elevation of two of the main frames, and the most typical floor plan of the building, respectively. Additionally, Table 1 presents some of the most relevant geometric and material characteristics of the structural system.

Figure 1. 3D-model, front and back view of the building.

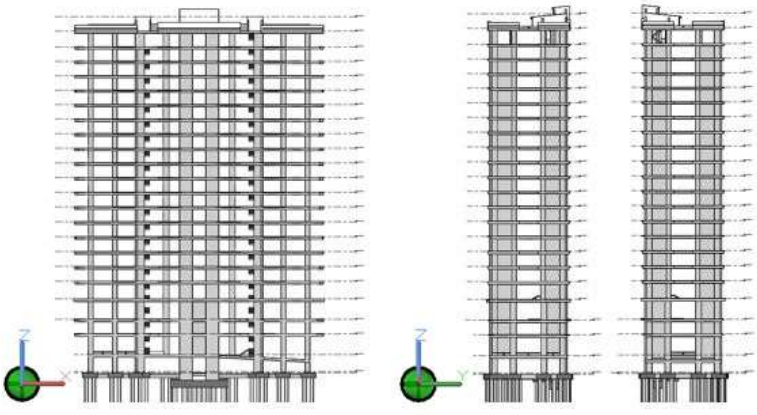

Figure 2. Front and side frames elevations

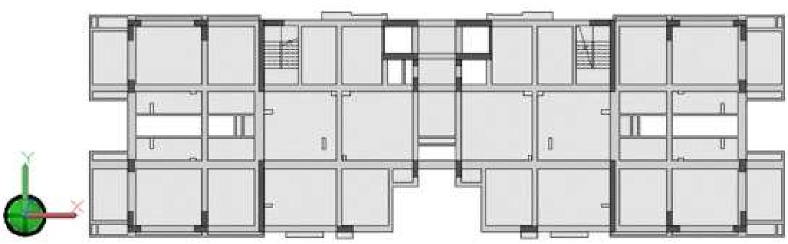

Figure 3. Structural floor plan of the representative building floor

Table 1. Geometric and material characteristics of the structural system

			Beams			Columns			Walls		
Level	**f_c (MPa)**	**H (m)**	**b (cm)**	**h (cm)**	**ρ**	**b (cm)**	**h (cm)**	**ρ**	**b (cm)**	**h (cm)**	**ρ**
0-10	35	3.45	40-60	55	>0.33%	40-50	Variable	1–3 %	40	Variable	1-2 %
11-17	28-35	3.00	40-60	50	>0.33%	40-50	Variable	1-3 %	40	Variable	1-2 %
18-23	28	2.95	40-60	45	>0.33%	40-50	Variable	1-2 %	40	Variable	1-2 %

In this table, f_c represents the compressive concrete strength; *H* is the story height; *b and h are the* width and height of the cross sections; ρ is the steel percentage

2.2 Modelling considerations

The hysteretic properties of the material used, in this case reinforced concrete, are defined from the pivot hysteresis model [8]. This model adequately represents the nonlinear response of reinforced concrete elements, since it includes cyclical axial load effects, asymmetric sections and biaxial bending, among others features. The main advantage of this model is the ability to capture the dominant nonlinear characteristics of the seismic response by means of three simple geometry-based rules. Despite its simplicity, the obtained results fit in an adequate and rational way when compared with the results obtained by applying more advanced models such as fiber models [9]. Regarding the damping model, the Rayleigh proportional damping has been used in this research [10].

The performance of a structure exposed to a seismic event can be described by the maximum state of damage allowed, (represented by the deformation of the hinges). This performance can be defined based on three specific levels (Immediate Occupancy, IO, Life Safety, LS, and Collapse Prevention, CP) proposed in FEMA 356 [11]. Figure 4 depicts these performance levels. Finally, in this research, the yielding surfaces are defined by means of the flexo-compression diagram for columns and moment-rotation diagram for beams, according to the requirements established in ASCE 7-16 [12]. The spectral modal analyses and the nonlinear dynamic analyses have been developed by using the specialized software code for structural analysis, Etabs [13].

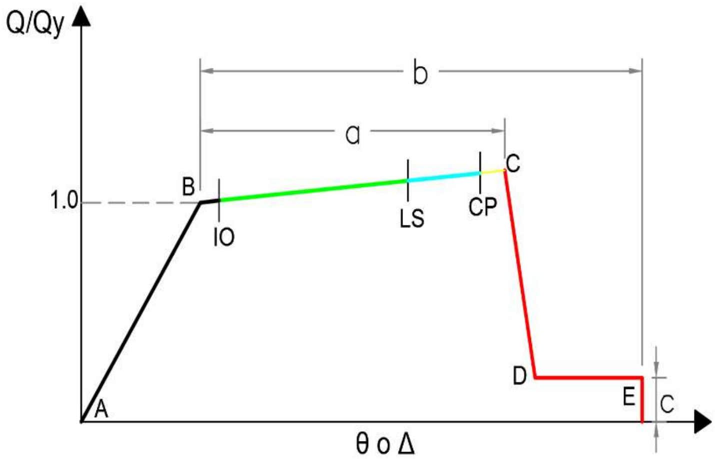

Figure 4. Generalized force-deformation relations for concrete elements or components and levels of structural performance. Source: American Society of Civil Engineers, ASCE 7-16 (2017) and Federal Emergency Management Agency, FEMA 356 (2000)

3 SEISMIC HAZARD

There are several ways to characterize seismic hazard, three of them are used in this work. The first is based on the design elastic spectrum and it is used in the RSM method. This spectrum is obtained from the smoothed curves given by the Colombian seismic-resistant design regulations (AIS, 2010). This spectrum is used to perform a first approximation of the structural configuration. The second way is by using hybrid accelerograms (generally to check performance levels in the design phase). In brief, they are generated from real accelerograms, which undergo a mathematical treatment so that their response spectra fit a target spectrum in a given range of periods. It is worth mentioning that, when employing NLDA in design, the Colombian code NSR-10 prescribes a set of minimum 7 seismic records. Note that the original signals used to develop these records should meet a series of characteristics typical of the region, such as similar acceleration in a range of frequencies, magnitudes, and hypocentral distances to the fault [14]. These features must be as representative as possible of the expected seismic event in the area. Figure 5 shows both the design elastic spectrum for the building location site, as well as the response spectra of ten selected and fitted signals.

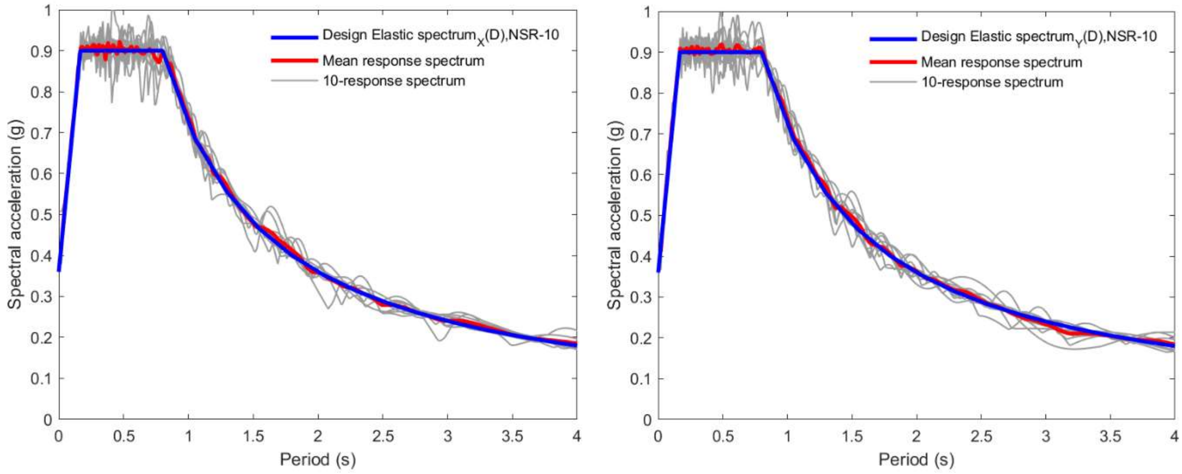

Figure 5. Blue: Design elastic spectrum; red: mean of the response spectra of the 10 fitted ground motion records; grey: response spectrum of the 10 fitted ground motion records

The third way used in this research to characterize seismic hazard consists of selecting real ground motion records (one-hundred in this case) incrementally scaled with respect to an intensity measure (IM) (see Figure 6). Based on these records, fragility functions based on the cloud analysis have been generated (Jalayer et al., 2015). These curves have been used to validate the simplified methodology proposed in this article.

The goal of scaling records for deriving fragility functions is that the structure reaches different performance levels. For the purpose of this study, it should be avoided to scale the same record to different intensity levels since this introduce a false correlation between IMs and EDPs. In summary, the following procedure has been used to select and scale the set of one-hundred ground motion records:

1. Identify the IM to scale. In this case, the average spectral acceleration (AvSa) is used (Vargas-Alzate & Hurtado, 2021).
2. Define the maximum value of the IM in order to define the scaling bands (ten in this case). This value depends on the level of intensity used for the building design.
3. Select one-hundred ground motion records from a database, which have been previously sorted with respect to AvSa.

4. The ground motion record with the highest IM is scaled so that its new IM value belongs to the highest scaling interval. If the IM naturally fulfils the interval condition, no scale factor is considered. This step is repeated with the subsequent records, according to the sorted list, until the desirable number of records belonging to the highest interval (ten) is obtained.
5. The previous step is repeated for all intervals. The scale factor in step 4 is calculated having in mind that the IM values are uniformly distributed within each interval.

Figure 6 shows the geometric mean spectra of the horizontal components of the records selected according to the steps described above.

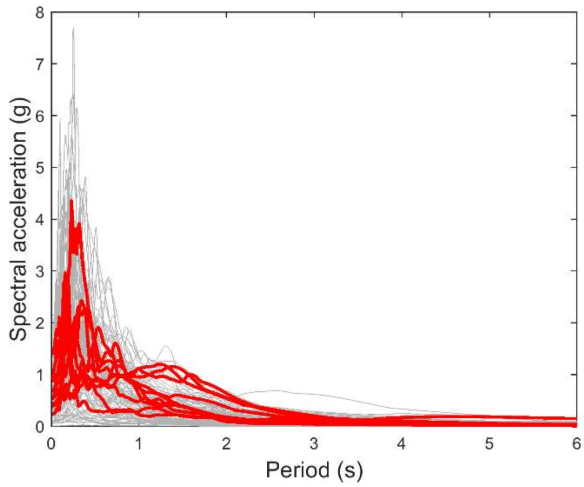

Figure 6. Geometric mean spectra of the 100 selected records (grey) and the selected records, one for each scaled band (red).

Once the building has been designed (Based on the records whose spectra are shown in Figure 5) and the seismic action characterized (Figure 6), the next step is to perform one-hundred time-history analyses, where the evolution of stiffness degradation and damping dissipation of the different elements that make up the structure will be stored and post-processed. Based on these results, the next section is dedicated to explain the reduced order methodology.

4 REDUCED ORDER METHODOLOGY

It is common to find a multitude of physical problems that require a dynamic study. In the field of civil engineering, the dynamic behavior of buildings is one of the most demanded. In this case, the dynamic equilibrium equation allows estimating the temporal evolution of multiple variables of interest to engineering. Note that finding an analytical solution of this equation is only possible for simple cases. For complex systems (MDoFs), the use of advanced numerical methods is required. However, the great variety of dynamic situations to which a structure may be exposed, combined with the need to estimate its behavior in a 'precise' way, highlights the importance of making a good choice about the numerical procedure to be used.

There are several numerical methods to solve the dynamic equilibrium equation (Newmark, Hilber-Hughes-Taylor and Wilson, among others) [16]. However, as commented above, solving this differential equation is computationally expensive if all the variables involved are exhaustively evaluated, especially if the structures fall into the nonlinear range. For instance, the time taken to achieve the one-hundred NLDA results was 4 months. For this reason, a simplified method is proposed herein that allows estimate the dynamic response of the structure in a quick and efficient manner.

In brief, this method characterizes the dynamic response of a structure from a transfer function, *TF*. This function is obtained by considering the Fast Fourier transform (FFT) of the analyzed response (for instance the Maximum Inter-Storey Drift Ratio, MIDR, obtained by means of NLDA) and the Fast Fourier transform of the seismic action used as input [17]. Eq.2 shows the mathematical form that characterizes a *TF*:

$$TF_{i,j} = \frac{f_{i,j}(\omega)}{g_j(\omega)} \tag{2}$$

where $f_{i,j}(\omega)$ represents the Fourier amplitude spectrum of the response variable of the structure; $g_j(\omega)$ is the Fourier amplitude spectrum of the accelerogram; subscripts i and j stand for a specific response of the structure (for instance MIDR of the storey i) and for a specific ground motion record, respectively.

The main hypothesis is that given the response of a structure to a specific ground motion record, the *TF* obtained through equation 2 can be used to evaluate the response of the system to another record, without having to resort to the use of the NLDA. However, it has been found that the precision of the *TF*s to predict EDPs is variable. Hence, it has been proposed to analyze a subset of the one-hundred records selected in section 3 in order to identify an optimal *TF*. Specifically, ten records are selected, one for each scaled band, with the purpose of performing ten NLDAs that allow obtaining an optimal transfer function. This function has been identified considering the predictive capacity of the *TF*s. Figure 6 shows the geometric mean spectra of the selected records in red line.

Six fundamental steps have been described for the identification of an optimal TF starting from a subset of ten ground motion records:

- Step 1: Perform ten NLDAs using the preselected signals. In this way, the response parameters can be obtained in terms of acceleration, velocity and displacement of the structure (Figure 7, left).
- Step 2: Calculate the EDP of interest (MIDR)
- Step 3: Calculate the *TF*s based on the MIDR. Note that, for each NLDA, it is necessary to calculate a *TF* for the inter-storey associated to each storey ($IDR_{i,j}(t)$) in order to capture the evolution of the MIDR in the entire structure (Figure 7, right).

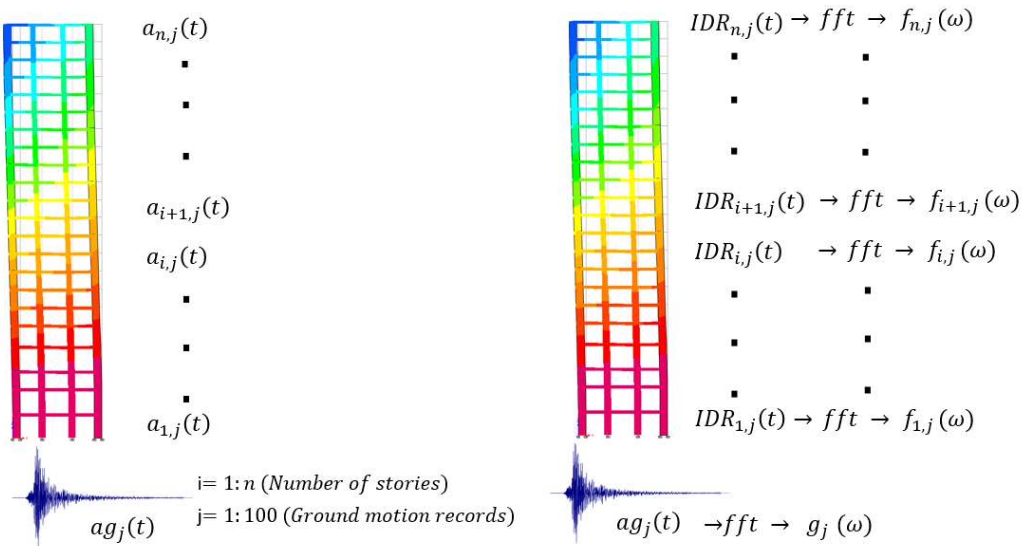

Figure 7. Left: Calculation of the different EDPs for each floor; right: Calculation of the different TFs for each floor

- Step 4: Estimation of the structural response (MIDR) through each one of the ten *TF*s, for the total number of preselected signals.
- Step 5: Selection of the optimal *TF* as the one minimizing the mean square error between the response obtained through the NLDA and ROM.
- Step 6: Finally, using the optimal *TF*, predict the response of the remaining set of ninety ground motion records.

5 ANALYSIS OF RESULTS

In this research work, the results of the MIDR obtained from NLDA are compared with those derived through ROM, for each of the main direction of the analyzed structure (see Figure 8). The ordinates show the MIDR data obtained through ROM whilst the abscissas show those calculated using the NLDA.

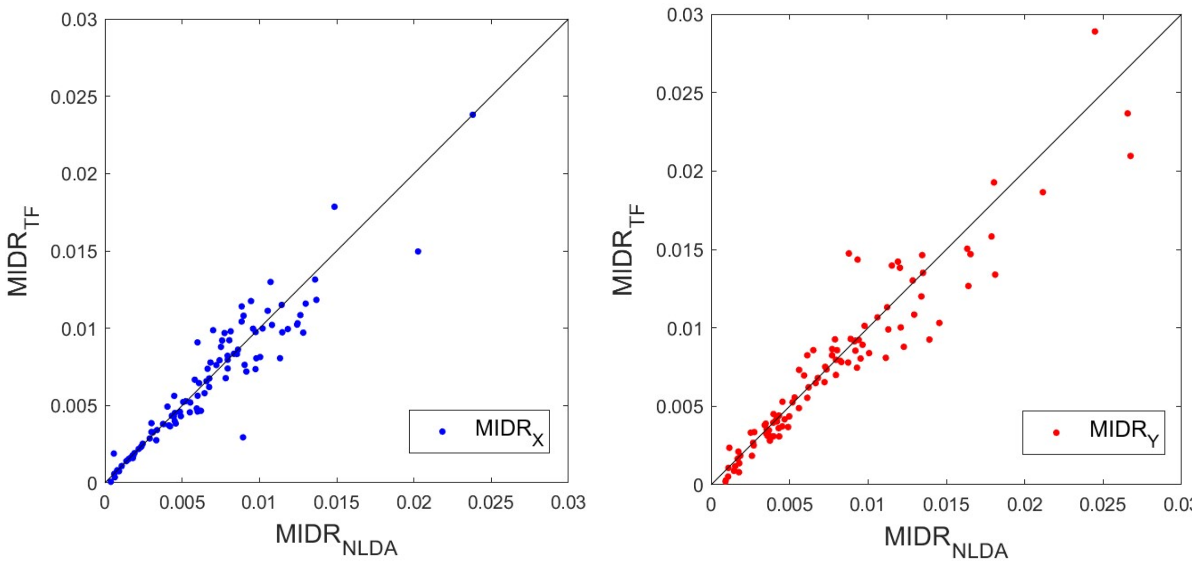

Figure 8. Maximum inter-story drift: MIDR_$_{NLDA}$ Vs MIDR_$_{TF}$

The vulnerability of a building is related to its fragility. This property can be quantified by means of fragility curves. These functions represent the cumulative distribution of the probability of reaching or exceeding a specific damage state, given a determined seismic action [18]. Damage states are usually expressed by parameters associated with strength, stiffness, ductility, or lateral displacements. The description of such damage states depends on the type of structure. For example, the damage state related to 'service' in reinforced concrete structures is described as the beginning of cracking due to flexo-compression and shear, among others, both in beams, columns or walls that are part of the seismic resistant system. For the extensive damage state, the non-structural elements present severe damage whilst the damage to the main structural elements can be significant although the entire structure must be able to continue supporting its own weight [19].

It should be noted that regression analysis plays a central role in the development of fragility functions, since it can be used to identify, describe or predict the response of systems from predictor variables, as well as to derive an optimal mathematical function that models this relationship. It is important to mention that, in cloud statistics, the dispersion and the mean of the bivariate distribution (IM-EDP) play a central role [20]. Thus, using the data shown in Figure 8, fragility curves are derived for NLDA and ROM results. Figure 9 shows the fragility curves related to the service damage state (MIDR=1.00%) using both approaches. QR_x and QR_y

stand for Quadratic Regression. It means, when deriving fragility functions based on the cloud analysis, a quadratic regression model has been used.

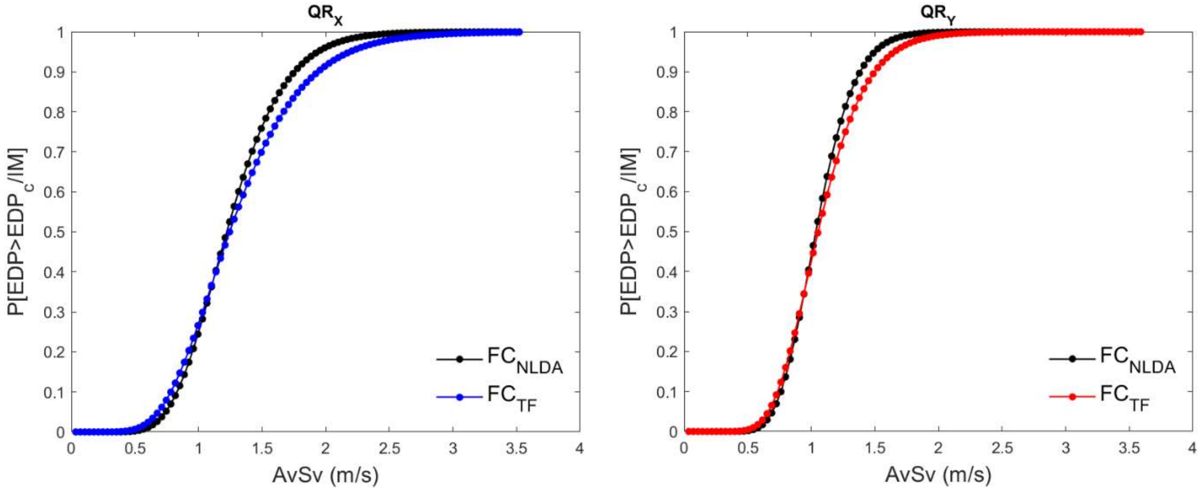

Figure 9. Fragility curves for the same state of damage (service) for the X and Y direction of the building, black curve: NLDA_X_Y, blue curve: ROM_X, red curve: ROM_Y

Figure 10 shows the fragility curves related to the extensive damage state (MIDR=2.50%) using both approaches. These curves (Figure 9 and Figure 10) were calculated using the most efficient IM to predict the MIDR. In this sense, current studies show that the most efficient and stable IM is the average spectral velocity given in a range of periods (AvSv, Vargas-Alzate et al., 2022). This IM has also been the most efficient one when compared in this research.

For both damage states, fragility functions show how the approximation made by ROM (blue and red curves) exhibits a good fit to the curve obtained by the target method (NLDA).

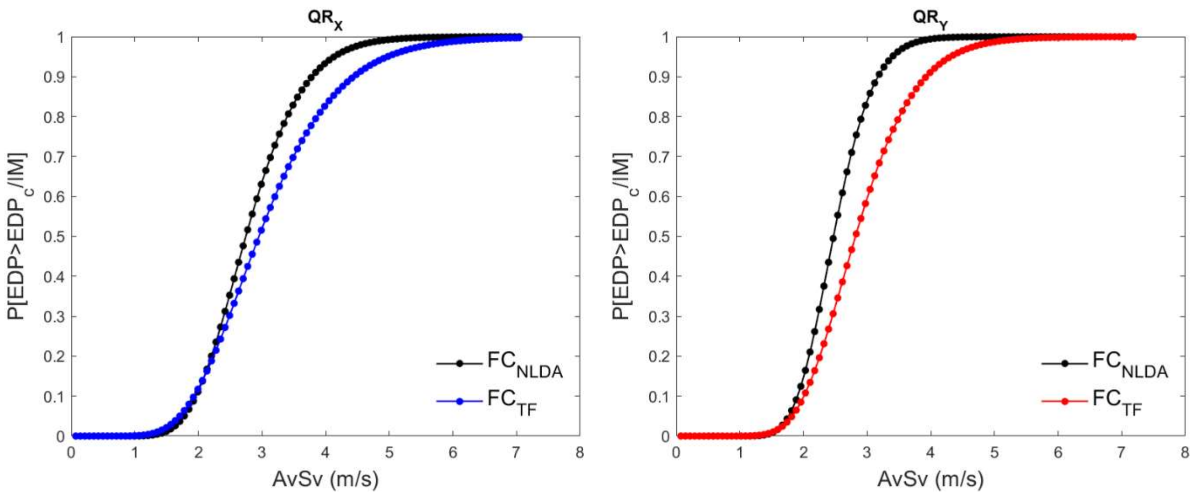

Figure 10. Fragility curves for the same state of damage (extensive) for the X and Y direction of the building, black curve: NLDA_X_Y, blue curve: ROM_X, red curve: ROM_Y

6 CONCLUSIONS

This work has been motivated by the need to calculate the probabilistic nonlinear structural response of complex systems in an affordable time. The developed method exhibits similar statistical distribution as those obtained with NLDA, which allows evaluating the seismic vulnerability of complex systems in a reliable and efficient manner. This improved ability to analyze this type of system may derive in a significant contribution for methodologies related to seismic risk estimates, increasing the protection of our urban environments.

It is noteworthy that, in a vulnerability and seismic risk assessment, regardless of the methodology to be used, it is essential to follow an approach that takes into account the non-linear behavior of the structure, as well as the uncertainty associated with the seismic action. Note that the most important source of uncertainty in the dynamic response of a structure comes from this variable. This is due to the complex interaction of the mechanisms involved in the release of energy and the propagation of waves. For this reason, it is important to have a significant number of seismic records that allow characterizing the seismic hazard adequately.

In this respect, nowadays exists adequate methodologies, tools and technology to design and evaluate a structure potentially exposed to the most restrictive and demanding performance requirements. Hence, current computational capacity, combined with advanced statistical approaches, allow the development of probabilistic frameworks to estimate the nonlinear response of civil structures exposed to seismic actions. However, reducing the computational cost without unduly sacrificing precision would help to generalize the use of more exhaustive theories. To do so, it is necessary to develop tools that allow the analysis of complex systems in an affordable time, such as the one proposed herein. However, in order to properly calibrate the proposed method, a higher number of calculations must be made, where structures with different natural periods and configurations, both in plan and in elevation, are included.

It is important to mention that the maximum inter story drift is one of the most complex EDPs to evaluate. This is due to its relative character, which can be affected by the phases in the Fourier transformation. However, a good approximation of the fragility curves has been observed, even for the extensive damage state.

Finally, it can be concluded that the proposed simplified method presents a high potential for future research, since it will allow calculating seismic risk scenarios under robust statistics. It should be noted that this method can be improved in terms of efficiency if, for example, transfer functions are developed through signals that excite the structure in a wider range of periods, for instance, using white noise signals.

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7 REFERENCES

[1] P. Fajfar, “Analysis in seismic provisions for buildings: Past, present and future,” in *Geotechnical, Geological and Earthquake Engineering*, vol. 46, 2018. doi: 10.1007/978-3-319-75741-4_1.

[2] Y. F. Vargas-Alzate, N. Lantada, R. González-Drigo, and L. G. Pujades, “Seismic risk assessment using stochastic nonlinear models,” *Sustain.*, vol. 12, no. 4, 2020, doi: 10.3390/su12041308.

[3] J. Moehle, Y. Bozorgnia, and T. Y. Yang, “The Tall Buildings Initiative,” *Struct. Eng. Assoc. Calif.*, 2007.

[4] Y. Vargas, "Análisis estructural estático y dinámico probabilista de edificios de hormigón armado. Aspectos metodológicos y aplicaciones a la evalua- ción del daño.," p. Spanish, 2013.

[5] A. colombiana de ingeniería sismica- AIS, "Reglamento Colombiano de Construcción Sismo Resitente NSR-10," *Scielo*, p. Spanish, 2010.

[6] ACI 318, *Building Code Requeriments for Reinforced Concrete, American Concrete Institute, Detroit, Michiga*. 2019.

[7] J. W. Cooley, P. A. W. Lewis, and P. D. Welch, "Application of the Fast Fourier Transform to Computation of Fourier Integrals, Fourier Series, and Convolution Integrals," *IEEE Trans. Audio Electroacoust.*, vol. 15, no. 2, 1967, doi: 10.1109/TAU.1967.1161904.

[8] R. K. Dowell, F. Seible, and E. L. Wilson, "Pivot Hysteresis Model -Dowell, Seible & Wilson ACI 1998," *ACI structural journal*, vol. 95. pp. 607–617, 1988.

[9] Y. Ling, "Parameter optimization for Pivot hysteresis model for reinforced concrete columns with different failure modes," no. April, pp. 2167–2187, 2022, doi: 10.1002/eqe.3658.

[10] A. Carr, *Ruaumoko-Inelastic Dynamic Analysis Program; Department of Civil Engineering, University of Canterbury: Christchurch, New Zealand*. New Zealand, 2000. doi: 10.13140/RG.2.1.4755.8567.

[11] FEMA, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," *Rehabil. Requir.*, no. 1, pp. 1–518, 2000.

[12] ASCE 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Reston, VA: American Society of Civil Engineers, 2017. doi: 10.1061/9780784414248.

[13] CSI, "Computers & Structural, INC," *Struct. Earthq. Eng. Software, www.csiamerica.com*, 2017.

[14] C. B. Haselton, A. S. Whittaker, A. Hortascu, J. W. Baker, J. Bray, and D. N. Grant, "Selecting and scaling ground motions for performing response-history analyses," in *15th World Conference on Earthquake Engineering*, 2012.

[15] Y. Vargas-Alzate and J. Hurtado, "Efficiency of Intensity Measures Considering Near- and Far-Fault Ground Motion Records," *Geosciences*, vol. 11, no. 6, p. 234, May 2021, doi: 10.3390/geosciences11060234.

[16] T. C. Fung, "Numerical dissipation in time-step integration algorithms for structural dynamic analysis," *Prog. Struct. Eng. Mater.*, vol. 5, no. 3, 2003, doi: 10.1002/pse.149.

[17] J. Bobadilla, "La transformada de Fourier. Una visión pedagógica," *Estud. fonética Exp.*, vol. 10, p. Spanish, 1999.

[18] R. L. Bonett Díaz, "Vulnerabilidad y riesgo sísmico de edificios. Aplicación a entornos urbanos en zonas de amenaza alta y moderada.," dissertation, Universitat Politècnica de Catalunya, 2003.

[19] J. Vielma, "Damage Trhesholds for Limit States of Reinforced Concrete Framed Buildings Designed According To Aci- 318/Ibc-2006," *Rev. Int. Desastr. Nat. Accid. e Infraestruct. Civ.*, vol. 8, no. 2, p. Spanish, 2008.

[20] F. Jalayer, R. De Risi, and G. Manfredi, "Bayesian Cloud Analysis: Efficient structural fragility assessment using linear regression," *Bull. Earthq. Eng.*, vol. 13, no. 4, pp. 1183–1203, 2015, doi: 10.1007/s10518-014-9692-z.

[21] Y. F. Vargas-Alzate, J. E. Hurtado, and L. G. Pujades, *New insights into the relationship between seismic intensity measures and nonlinear structural response*, vol. 20, no. 5. Springer Netherlands, 2022. doi: 10.1007/s10518-021-01283-x.