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CALIBRATION OF MATERIAL MODEL FOR ANALYTICAL PUSHOVER-BASED ASSESSMENT OF SELF-CENTRING CONCENTRICALLY BRACED FRAMES

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Abstract

The idea behind the behaviour of the self-centring concentrically braced frame (SC-CBF) system is derived from the flag-shaped hysteretic curve, representing the force-deformation behaviour of self-centring systems, thus tackling the problem of excessive residual drifts and seismic loss. The modelling of the braces is the main focus when considering the SC-CBF system since they are the sole element for energy dissipation. They are modelled with the uniaxial Giuffre-Menegotto-Pinto steel material with isotropic strain hardening. The material model calibration in this study is performed against a combination of numerical simulation results and available experimental data.
The response of the calibrated model is analysed through a comparison between two types of models for SC-CBFs. A frame from an experimental procedure of shake table testing is chosen for numerical and analytical modelling and subjected to half-cycle pushover loading. Finally, it is demonstrated that accordingly calibrated low-fidelity models based on empirical relations yield solid results in terms of dispersion for modern probabilistic risk assessment procedures.

Keywords: Self-Centring, Steel Frames, Concentrically Braced, Bayesian Calibration, Nonlinear Analysis, Risk Assessment.

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1 INTRODUCTION

To improve the seismic resilience of concentrically braced frames (CBFs) and design a structure with negligible residual drifts, several innovative damage control systems have been developed. Although attempts have been made to consider the occurrence of residual displacements in the design of structures, the use of self-centring systems has been deemed a more suitable solution. The innovative system of self-centring concentrically braced frames (SC-CBF) consists of a series of post-tensioned strands running parallel to the beam elements in the system, anchored to the external columns of the frame between their flanges [1].

The primary focus in analyzing the SC-CBF system is the modeling of the braces, as they are the only components responsible for energy dissipation. These braces are represented using the uniaxial Steel02 stress-strain relationship [2], which includes isotropic strain hardening. In this study, the material model is calibrated using a combination of numerical simulation results and existing shake table experimental data. It is established that the Bayesian calibration method yields high quality numerical model since it is sufficiently accurate compared to the reference model and precise since the variance of each parameter is rather low and the dynamic results closely match between the samples.

The response of the calibrated model is analysed through a comparison between two types of models for SC-CBFs. A frame from an experimental procedure of shake table testing [3] is chosen for numerical and analytical modelling and subjected to nonlinear time-history analysis (NLTHA). Regarding the main characteristic points, such as the story drifts, the peak story drifts and the residual drifts, it is observed that they are matching. Since the experimented frame is single story SC-CBF, the response of the structure under lateral loads can be easily represented through a single degree of freedom (SDOF) system. Then, the single-story flag-shaped hysteretic model equations are implemented to obtain the analytical pushover curves of multi-story archetype SC-CBF structures. Nonlinear dynamic analyses of several archetype structures are conducted to determine the fragility curves of the innovative SC-CBF system using a single-degree-of-freedom (SDOF) model. Finally, it is demonstrated that accordingly calibrated low-fidelity models based on empirical relations yield solid results in terms of dispersion for modern probabilistic risk assessment procedures.

2 MODELLING OF SC-CBF

2.1 Energy dissipation system

For the purposes of this research, OpenSees [2] was used as the primary tool for simulating the response of the system and its individual components. The brace model for numerical simulations is based on a previous study by Uriz and Mahin [4], along with additional insights from extensive numerical [5] and experimental results [6].

Figure 1 illustrates a linear beam-column element for modelling structural components. Each structural element is modelled using a multi-fibre beam-column element with distributed plasticity and an appropriate number of integration points. To represent the phenomenon of buckling, at least two elements are required, as one element may undergo yielding under compression without experiencing buckling behaviour. The initial deformation necessary to induce buckling is another important parameter in modelling braces, and it is applied at the centre of their span where plastic hinge formation and section rotation are expected. The distributed plasticity method is employed due to its capability to capture plastic deformations at any point along the length of the elements. In this approach, the cross-section is modelled using fibres distributed in width and thickness. Element integration is performed with an appropriate number of integration points along the element's length using Gaussian integration.

Each fibre in the cross-section is represented by material properties based on the Menegotto-Pinto model with isotropic hardening, denoted as Steel02 in the OpenSees material library [2]. According to the recommendations [4], an initial deformation ranging between 0.05% and 0.1% is adopted. At least 8 elements per brace with a minimum of 3 integration points along the element length are applied, along with 10 to 15 fibre layers in height and width of the cross-section to precisely determine plastic deformations in critical sections of the braces. To account for material fatigue under cyclic loading, the basic steel material is characterized by the low-cycle fatigue effect.

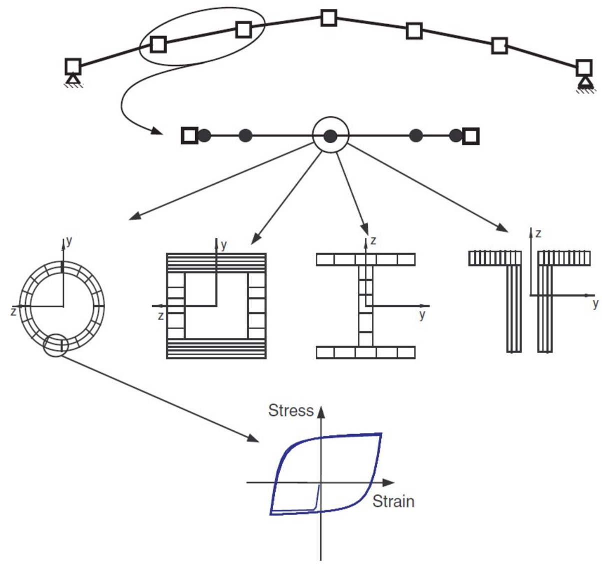

Figure 1: Schematic view of beam-column element for brace modelling [4].

The gusset plates, which, together with the braces, constitute the energy dissipation system and where plastic hinges may also occur during earthquake action, are modelled using rotational hinges [7]. The stiffness parameters of the rotational spring are determined based on experimental results and depend solely on the material and dimensions of the gusset plate.

2.2 Post-tensioning system

The rocking connection is one of the two main components of the post-tensioning system that operates with the help of pre-stressing force. It consists of several phenomena that need to be appropriately modelled and simulated in the analyses. One of these phenomena is the opening of the connection under horizontal dynamic loading. Another aspect to consider when modelling this connection is the effect of the beam height, as well as the forces introduced into the beams and columns due to the opening of the connection and the impacts of the flanges. The model proposed by Christopoulos [8] comprises contact springs and rigid links that appropriately represent the behaviour of the rocking connection. The contact elements are placed on the flanges of the beams and modelled with springs that only work under compression, aiming to simulate the rocking of the flanges in relation to the column flanges. The post-tensioned strands are modelled with truss elements with initial strain corresponding to the pre-stressing force, and they do not take into account geometric nonlinearities. These elements are anchored to the external surface of the columns and keep the frame in the initial position. They are provided with a bilinear material denoted as Steel01 in the OpenSees material library [2]. Figure 2 schematically illustrates the connection at the middle column with the corresponding elements for the connections between the elements of the SC-CBF.

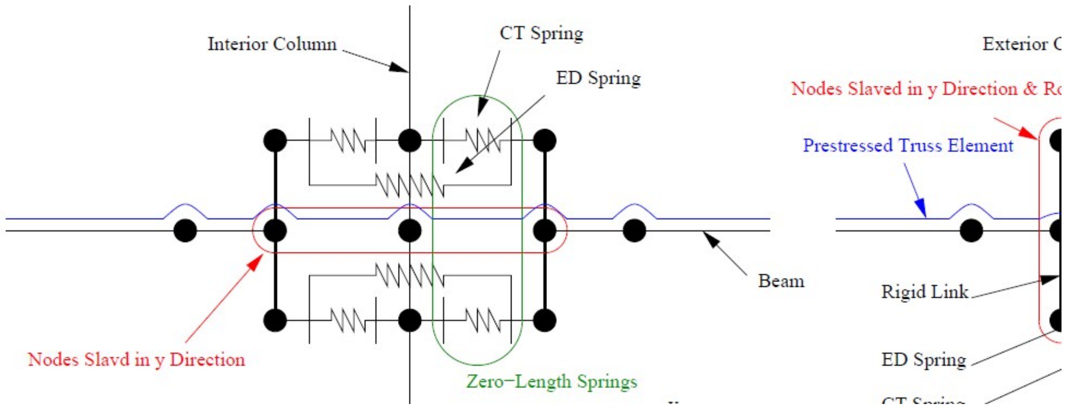

Figure 2: Schematic view of internal and external column connection modelling with constituent elements [8].

The beams and columns are the only remaining elements of the system, and their modelling is carried out under the assumption that they remain within the elastic range throughout their service life. Similar to the braces, they are modelled using beam-column elements, but with a single element and 5 integration points along its length. In terms of height and width of the cross-section, 5 fibres are applied, while along the thickness of the flange and the web, 2 fibres are applied. The material used to model these elements is also Steel02.

3 BAYESIAN CALIBRATION OF MATERIAL MODEL

The use of experimental data exclusively for calibration of material model parameters is rather complex and often limited. When considering dynamic testing of structural systems, it is hard to obtain the desired behaviour of an isolated structural element with all the data needed for material calibration. For example, the limitation in overturning moment of a shake table can present difficulties in reaching several cycles of material nonlinearity in the specific element. Also, the testing procedure can be focused on different behaviour parameters of the structural system, such as interstory drifts, and fail to generate sufficient data for the structural element of interest. Finally, the material of the tested elements often fails to meet the designed values and can give higher values of yielding of the material, lesser ductility, varying strains etc. All of these issues regarding experimental data can be overcome by introducing virtual experiments that take advantage of the most advanced computational methods for probabilistic calculations.

The classical calibration of model parameters deals with optimizing the error between a numerical model and experimental data and produces one set of parameters that are considered as the best. Since nothing in earthquake engineering is deterministic, the introduction of model parameter uncertainty, which is a part of the result epistemic uncertainty, allows for addressing the imperfect representation of the real behaviour in a numerical environment. If we know this uncertainty in the modelling parameters for all elements, we can simulate many different model realizations of the same structure. Therefore, the Bayesian approach, Equation 1, is implemented to estimate the updated posterior likelihood of all possible sets of parameters *(A_i)* in order to consider the epistemic uncertainty for the calibration of the numerical model using available empirical data *(E)*.

$$P(A_i \mid E) = \frac{P(E \mid A_i)P(A_i)}{\sum_{i=1}^{n} P(E \mid A_i)P(A_i)} \tag{1}$$

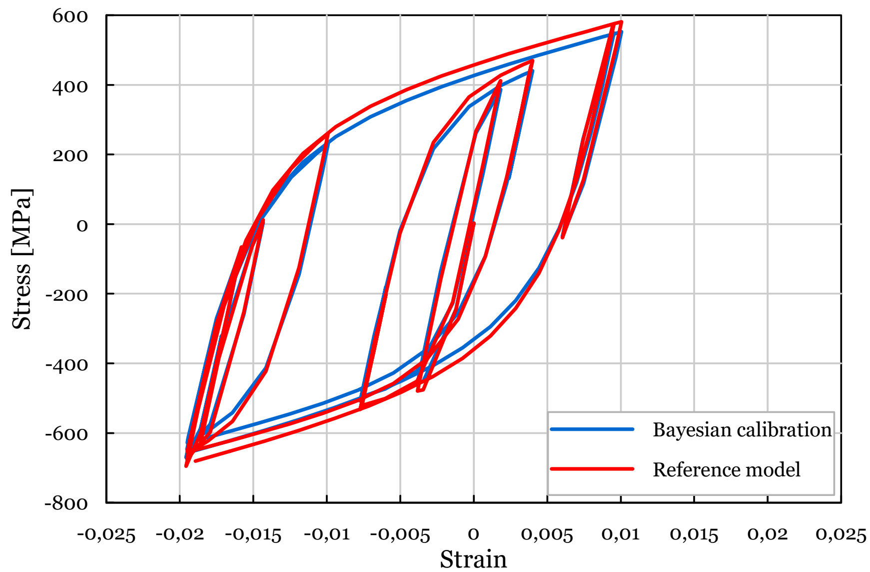

Figure 3: Stress-strain relationship of reference and calibrated model.

The reference model for calibration is the Steel02 stress-strain relationship previously calibrated to experimental results. The values for the main parameters of the reference material model are obtained from coupon testing of the tested braces [9], previous studies' recommendations and engineering judgement. The results of the simulated reference model are then compared to the results of the model realisations for the calibration of the material model. Figure 3 shows the direct comparison between the reference model and the Bayesian calibrated model derived from 500 samples. The sample size is not the same as the number of computational models. The number of models is the total number of times that the computational model is evaluated, and it is based on the TMCMC algorithm gradually changing the target distribution from the prior to posterior in a number of stages. However, it is observed that calibrated model is matching the reference model's response quite well.

4 SIMPLIFIED PUSHOVER BASED PROBABILISTIC ASSESSMENT

To determine the main parameters of the structural pushover curve in this study, the structure was analysed using the analytical equations developed by O'Reilly and Goggins [10], which include all the parameters of the behaviour of SC-CBF. This model, in correlation with the single-degree-of-freedom nonlinear model, showed good results and alignment of relevant drifts in the nonlinear dynamic analysis of the structural system. It is generally observed that the story drifts are matching with complete following of the peaks in both time histories, Figure 4. The most important observation is the matching of the peak story drift which in this case is well predicted with the SDOF model. Additionally, the residual drifts are adequately matched, since the residual drift in both the experimental and SDOF system response tends to a value of 0. With this observation, the self-centring behaviour of the system is confirmed. The good representation of the self-centring behaviour and accurate prediction of the peak story and residual drifts using simple SDOF model is very useful for performing extensive dynamic analyses where the main performance criteria is the structural drift.

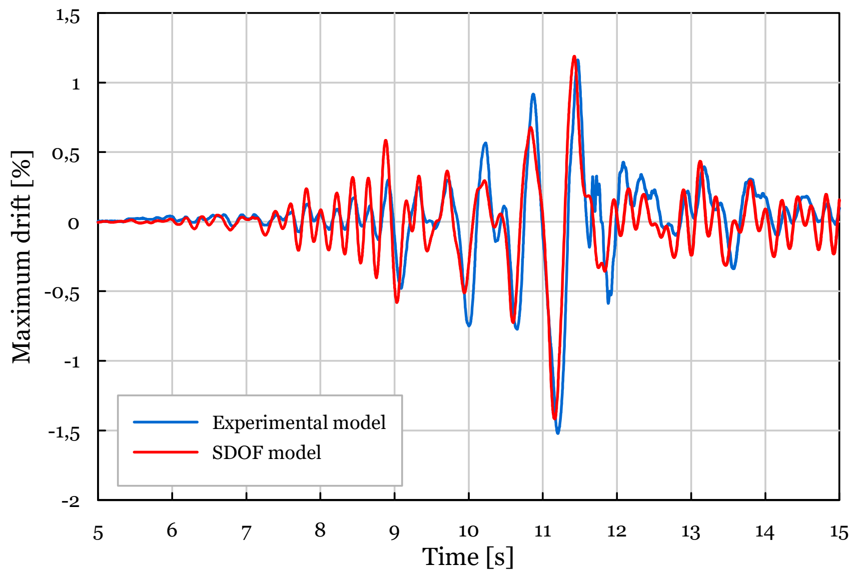

Figure 4: Stress-strain relationship of reference and calibrated model.

Then, 3 different structures with 3, 5 and 8 stories were designed in order to assess the suitability of the simplified pushover-based probabilistic assessment. All structures have typical plan dimensions of 24 x 24 m, span widths of 6 x 6 m, and a floor height of 3.2 m. For design, the elastic spectrum Type 1 and soil type B have been applied [11]. Design ground acceleration of 0.30 g as high seismicity level was considered, with a structural importance factor γ_I = 1. The permanent load, including the self-weight of the elements, is 6.5 kN/m^2, while the variable load is 2 kN/m^2. The total seismic weight W is then calculated according to EN1998 [11] using equation 5.2, where $\psi_{E,i}$ is 0.24, corresponding to stories with corelated occupancy. Since the main elements for hysteretic energy dissipation are concentric hollow section braces, it can be assumed that the modification factor for the total seismic force used for concentrically braced frames, q = 4.0, is justified for use in SC-CBFs as well. Another important provision to consider in the design of SC-CBFs is the distribution of the floor horizontal force between different systems for lateral force resistance. Thus, part of the floor horizontal force is absorbed by the energy dissipation system, while the remaining part is taken by the post-tensioning system [12].

To determine the yielding points from the pushover curve of the structural system of the archetype structures, a relationship based on the fundamental equations for analytical calculation of the system was established. Thus, using the empirical equations [10] and distributing displacements along the height of the structure according to the procedure for simplified analysis of FEMA P-58-1 [13], the calculation of the horizontal force at yield was performed for each archetype frame. The results of the yielding point analysis for each archetype structure are presented in Table 1, while the analytical pushover curves are presented in Figure 5.

No. of stories	$\theta_{max,y}$ [%]	$F_{b,y}/W$	$F_{b,y}/F_{b,des}$
3	0,3154	0,28	1,44
5	0,3154	0,28	1,44
8	0,4071	0,30	1,56

Table 1: Data for analytical pushover curve of designed structures.

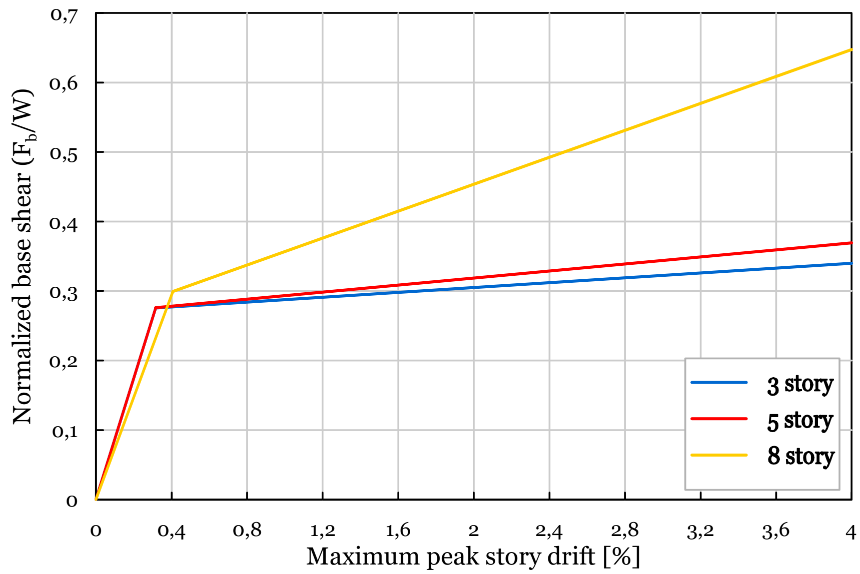

Figure 5: Analytical pushover curves for 3, 5 and 8-story SC-CBF structures designed for high seismicity regions.

Nonlinear dynamic analyses of the archetype structures to determine the behaviour factor were conducted using a single-degree-of-freedom (SDOF) model. Single-degree-of-freedom systems serve as a powerful tool for extensive analyses in earthquake engineering. The SDOF model is the simplest model for dynamic analysis of structures, requiring minimal computational capacity while providing accurate results depending on the modelling approach. In recent times, numerous procedures with extensive analytical requirements are executed using SDOF models, and there are several analytical procedures based on SDOF models [14-16]. Considering that the main control parameters for determining performance criteria in the applied analysis involve story drifts, the SDOF model serves as a suitable replacement for complex nonlinear models.

During the transformation from a system with multiple degrees of freedom to a single-degree-of-freedom (SDOF) system, it is necessary to modify certain parameters from the structural behaviour curve. Specifically, the application of the constant Γ is required to scale the yielding strength of the structural system and the maximum roof displacement. This constant is typically computed based on the dynamic characteristics of the structure. However, for the sake of simplification, in cases where a detailed model of the structure is unavailable, it is assumed to take a value in the range of 1.2 to 1.3 in the study.

No. of stories	Performance objective	$AvgS_{a,50\%}$ [g]	β
3	LS	0.451	0.344
	GC	0.647	0.236
5	LS	0.493	0.305
	GC	0.626	0.214
8	LS	0.543	0.234
	GC	0.705	0.199

Table 2: Fragility curve parameters for the two different performance objectives of the designed structures.

The performance of each archetype structure is evaluated based on two performance objectives: global collapse (GC) and life safety (LS). These two performance objectives represent the minimum requirements of the INNOSEIS approach for evaluation of the behaviour factor, and they are compared to the mean annual frequency of exceedance of 2% in 50 years and 10% in 50 years, respectively [17]. The peak story drift is taken as the main engineering demand parameter for determining the performance objectives. The values for the two performance objectives are defined as 2.5% drift for LS evaluation, while for GC, it is 4%. The results of the performance analyses are provided as fragility curves for the two different performance levels and presented in Table 2 and Figure 6.

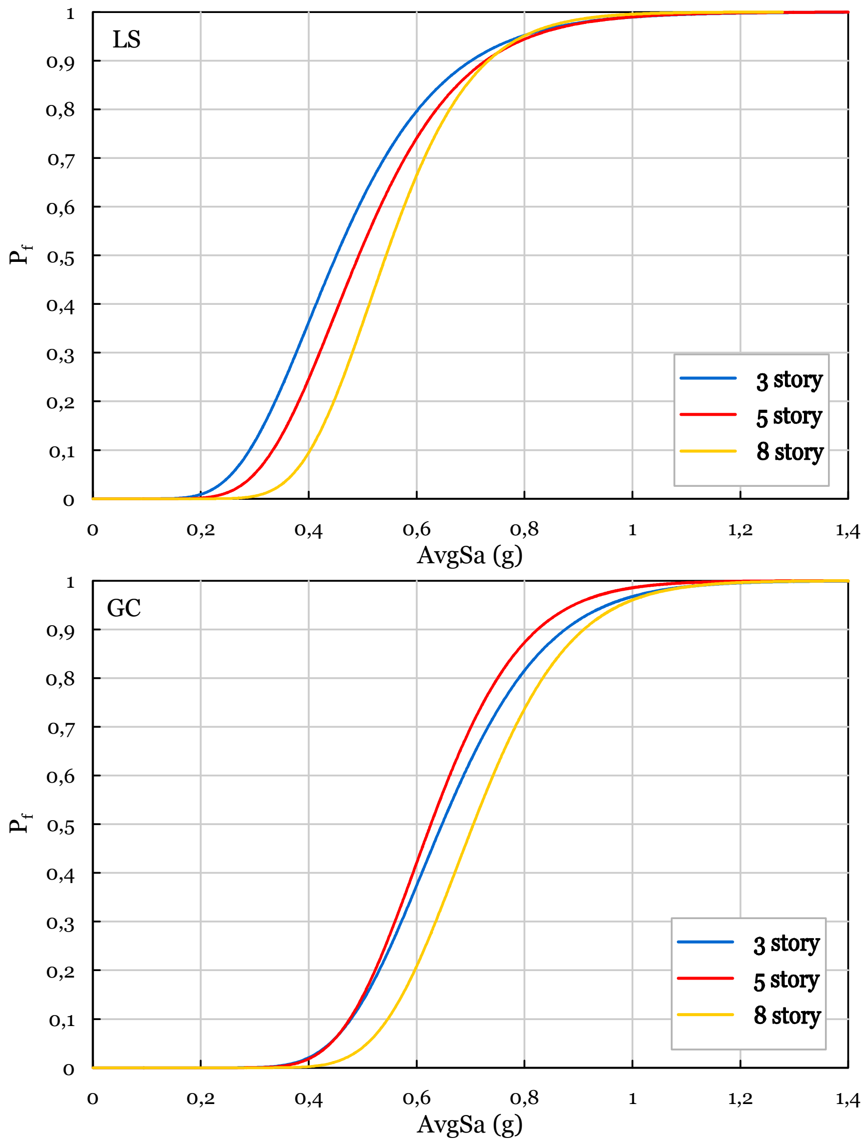

Figure 6: Analytical pushover curves for 3, 5 and 8-story SC-CBF structures designed for high seismicity regions.

5 CONCLUSIONS

This study demonstrates that Bayesian calibration offers a robust framework for refining the Giuffré-Menegotto-Pinto (Steel02) material model using a combination of numerical simulations and experimental data, leading to improved fidelity in the representation of SC-CBF behaviour. Even simplified, empirically-based models (e.g., SDOF pushover models) can adequately represent the behavior of SC-CBF systems when properly calibrated, making them suitable for probabilistic seismic risk assessments. Good agreement is observed between the calibrated numerical models and analytical predictions at yielding, target drift, and unloading points, validating the modeling methodology for various frame configurations. The pushover analysis procedure, aligned with FEMA P-58 and INNOSEIS guidelines, accurately estimates key performance metrics (yield force, drift ratios, overstrength factors) and can be extended to multi-story archetype structures. Therefore, low-fidelity models calibrated against detailed dynamic analyses, provide a computationally efficient yet accurate means of generating fragility curves and evaluating performance objectives. Finally, the mean annual frequency of exceedance of the mean value of investigated intensity measyre can be utilized to correlate with the relevant hazard curves and determine the risk for the corresponding level of spectral acceleration.

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