DEVICE UNCERTAINTY PROPAGATION IN LOW-DUCTILITY RC FRAMES RETROFITTED WITH BRBs for seismic risk mitigation

F. Freddi1\*, J. Ghosh2, N. Kotoky3 and M. Raghunandan2

1*Department of Civil, Environmental & Geomatic Engineering, University College London, London, UK*

2*Department of Civil Engineering, Indian Institute of Technology Bombay, Mumbai, India*

3*Department of Civil Engineering, National Institute of Technology Meghalaya, Shillong, India*

\*Corresponding Author. Tel.: +447752360204. E-mail address: [*f.freddi@ucl.ac.uk*](file:///C:\Users\fredd\Dropbox\000_Now\00_IITB%20PostDoc%20Needhi\Journal%20Paper\f.freddi@ucl.ac.uk)

# ABSTRACT

Passive control systems, such as buckling-restrained braces (BRBs), have emerged as efficient tools for seismic response control of new and existing structures by imparting strength and stiffness to buildings, while providing additional high and stable energy dissipation capacity. Systems equipped with BRBs have been widely investigated in literature, however, only a deterministic description of the BRBs’ properties is typically considered. These properties are provided by the manufacturer and are successively validated by qualification control tests according to code-based tolerance limits. Therefore, the device properties introduced within the structure could differ from their nominal design estimates, potentially leading to an undesired seismic performance. This study proposes a probabilistic assessment framework to evaluate the influence of BRBs’ uncertainty on the seismic response of a retrofitted RC frame. For the case-study, a benchmark three-story RC moment-resisting frame is considered where BRBs’ uncertainty is defined compatible to the standardized tolerance limits of devices’ quality-control tests. This uncertainty is implemented through a two-level factorial design strategy and Latin-Hypercube Sampling technique. Cloud analysis and probabilistic seismic demand models are used to develop fragility functions for the bare and retrofitted frame for four damage states while also accounting for the uncertainty in the property of BRBs. Risk estimates are successively evaluated for three case-study regions. The results show that, for the considered case-study structure, these uncertainties could lead to an increase of fragility up to 21% and a variation in seismic risk estimates up to 56%.

**Keywords**: Buckling-restrained braces; Seismic devices uncertainty; Reinforced concrete frames; Seismic retrofit; Fragility curves; Seismic risk variability.

# INTRODUCTION

**Earthquakes** are one of the major catastrophic events that render serious damage to the buildings and infrastructure systems. Buildings, primarily, non-ductile reinforced concrete (RC) buildings have experienced severe damage due to seismic events []. For example, a bulk of the damage observed by the buildings in the Bhuj earthquake were attributed to the non-seismic design consideration. The economic loss was about XX billion USD due to Bhuj 2000 earthquake []. While the fatality and the monetary losses directly depend on the exposure considerations of the building, it shall be noted that non-ductile RC buildings are at a higher risk of damage []. Recognizing this fact, research in the study of the vulnerability of non-ductile RC buildings against the seismic event have been carried out. ……... (paper on non-ductile RC building) …….

In addition to the intermittent danger posed by the earthquake on the RC buildings, environmental factors also effect the strength of the building. Several atmospheric factors cause deterioration of the RC structure over a long period of time when environmental agents interact with concrete and steel material []. The RC buildings when located in harsh environmental conditions experience corrosion which further worsens the seismic performance. Corrosion is one of the primary causes for the reduction of the service life of the RC structures by decreasing the capacity of the structure. Thus, when corroded non-ductile RC buildings are prone to higher risk due to future seismic events. A study conducted by xxx that XX% of the total RC building are corroded in USA region which is significant………

Corrosion of rebars in an RC member (especially columns) reduces the effective cross-sectional area of the rebars which invariably affects the strength and performance of the structure under lateral load.Corrosionin longitudinal reinforcement and transverse reinforcement have different deterioration effects on the failure mechanism. Experimental test carried on corroded columns based on percentage mass loss only in **longitudinal reinforcement** shows that there is a significant reduction in the flexural strength of columns, reduction in energy dissipation and drift capacity [Goksu 2014; Meda et al 2014]. As **transverse reinforcement** has a higher probability of being corroded before the longitudinal reinforcement due to the smaller distance to corrosive atmosphere, there is a significant change in the behavior of the column having corrosion in both reinforcements. This gap was studied by Vu and Li (2018) [BingLi] and it was found that the 25% corrosion level in transverse reinforcement (stirrups) can significantly reduce the lateral load capacity and, in addition, change the failure mechanism from flexure to **flexure-shear failure**. It was also found, in the same study, that under high axial load columns with high corrosion faced sudden **axial failure**. Thus, modelling of **shear properties** becomes imperative for studying the effect of corrosion on the shear capacity of the RC members. These modelling techniques for shear are discussed in various research papers (GangWu202, Xu and Zhang 2011, Freddi et al 2021) which accounts for the **flexure-shear interaction of columns** during the application of lateral or cyclic load on the columns.

Until now major research was done in **single-column** tested under axial load with cantilever action or double curvature. Few experimental research works were conducted to study the seismic performance of corroded **moment**- **resisting frames** considering the effect of corrosion. Karapetrou el al (2017) provided a detailed study on low, mid and high rise 2-dimensional frames by considering probabilistic modelling of corrosion for assessing the seismic performance using increment dynamic analysis (IDA). Modelling of 2D non-linear beam-column frames was done in OpenSees [] with fiber sections with a fixed base. The study, done by Couto 2020 [], carried out the sensitivity analysis of corrosion rate on the vulnerability assessment of RC building frames (three types – low, mid and high rise) in Portugal. Non-linearity in fiber section modelling and rebar area reduction was considered for studying the corrosion effect on these ageing frames while doing the vulnerability assessment. Another research by D Sarno Purgliese 2020 [] had conducted a numerical evaluation of RC building frames for a different level of exposure and degradation with smooth rebars. Finite element modelling (FEM) was done using Seismostrut [] by defining material properties based on a new approach discussed in the paper to account for the degradation effects. Non-linear static and dynamic analyses were compared based on top storey drift ratio and base shear. Afsar Disaj et al. 2021 [Kashani] on the other hand plotted the fragility curves considering the combined effects of naturally occurring sequential main shock – aftershock (MS-AS) and progressive deterioration due to corrosion for single bay RC frame. The mechanical properties of longitudinal reinforcement and transverse reinforcement for corrosion were modified based on the equations proposed in Du, Clark and Chan (2205a, 2005b) and (Afsar Dizaj et al 2018.a) [] respectively. This study incorporated the effect of bond-slip phenomena based on the model by Zhoa and Sritharan 2007 []. These studies on RC frames do not comment on the shear failure that occurs in old structures which **are low in ductility due to non-seismic design**.

Research work on single columns expounds on the change in the behavior in failure from flexure to flexure-shear under the influence of corrosion as discussed earlier. There arises a need to study the failure mechanism of existing RC frames due to seismic loading with low ductility which is vulnerable to shear failure. Hence, this study tries to fill this gap by modelling low ductile frames which predominantly fail under shear during a seismic event (NLTHA, methodology for capturing major failure modes).

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## Finite Element Modelling for as-built and deteriorated frame

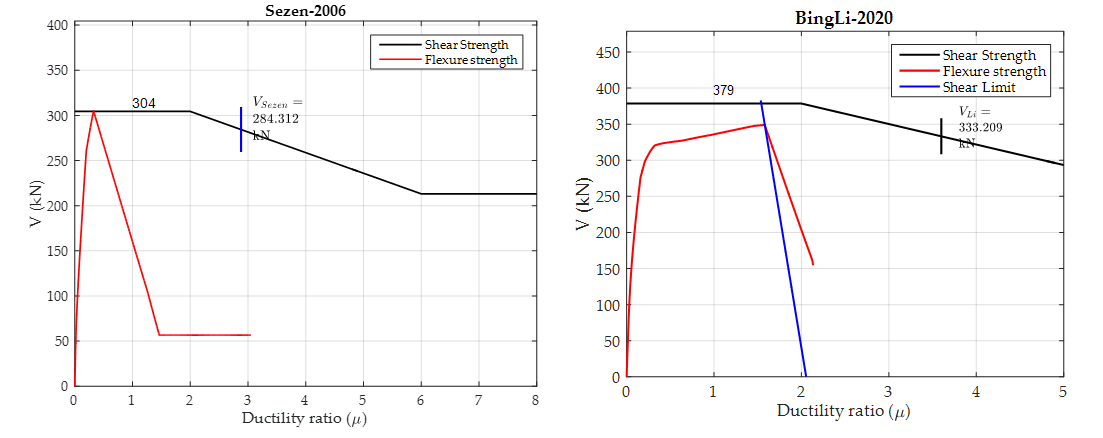
A zero-length element (spring) is used to define the shear limit curve in OpenSees. The shear limit curve (*limitCurve Shear* – in OpenSees) is adopted based on the model given by Elwood (2004) which is derived emperically. This model can be applied for only shear failure columns [Freddi 2021], the frame is joint is combined with ASCE-4[] proposed force-controlled shear limit surface. This is incorporated in OpenSees using *LimitState* material model and *limitcurveShear* which helps capturing the failure when the shear limit or drift limit curve is reached [Freddi 2021] The shear limit in term of shear strength is calculated by the model proposed by Sezen Moehle 2004 []. As shear failure is accompanied by cracking in the RC member, bond-slip phenomena in the longitudinal reinforcement is required to be captured while performing the cyclical test. The longitudinal bar slip causes additional strain in the fibre section which is given by bilinear stress-slip relationship established by Ghannoum and Moehle (2012). *zeroLengthSection* is used to incorporate the bar slip in the lateral load direction by multiplying the slip factor [Ghannoum and Moehle (2012)] to the strain values in the *uniaxialMaterial* in OpenSees.

### Validation with past experimental studies

For the purpose of validating the OpenSees modelling technique of simulating the shear-critical response, four square RC columns are analyzed under cyclical loading to validate the flexure-shear response (*flexures-shear interaction FSI)*. The columns for validation are selected based on the flexure-shear failure and includes uncorroded and corroded specimens. Table 3.1 gives the basic details and properties of the selected specimens used for the validation exercise.

Figure 3.1a and 3.1b are the pushover and shear limit plots for each specimen; from the plots, it can be concluded all the specimens observe flexure-shear failure. That is, lateral load capacity for each specimen intersects with the shear limit curve calculated as per by Sezen Moehle 2004. The plots of figure 3.2a and 3.2b are hysteric plots for all the column specimens modelled as per the description in the above section. Particularly from Figure 3.2a – specimen S1 -it can be illustrated that the column with low ductility will have a sharp decrease in lateral load capacity after reaching the shear limit. The hysteric plot for S1 is a good match validating our aim of modelling shear element and bar-slip section for the analysis. However, the hysteric plot of C5, CC2 and U2 does not have a good match with the experimental results. Also, the peak lateral load of the test is always higher for these three specimens and the initial stiffness up to the elastic range is clearly visible to be higher than that of the experimental curve. Part of the reason can be attributed to the experimental setup where the ends are not fully rigid and there is a presence of rigid body rotation at the fixed end. Another reason for the overprediction of the peak lateral strength would be the inelastic deformation in the reinforcement bars. However, the deterioration and pinching phenomena is well captured in all of the hysteric plots but the drawback remains in modelling which required manual inputs of these parameters to match with the experimental results. To summarize, the modelling discussed in the above section captures the flexure-shear failure in a satisfactory manner that can be extended to the frame under this study.

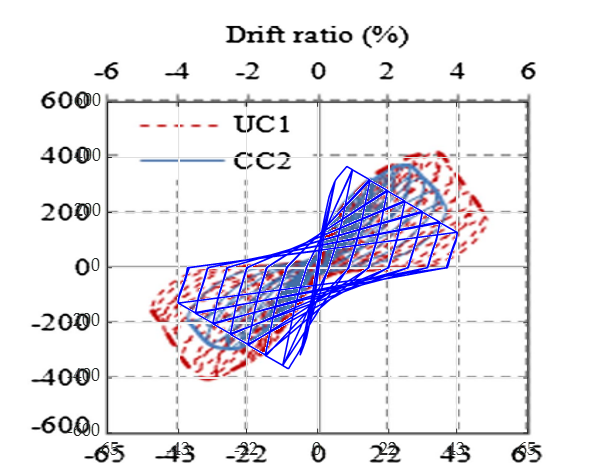
|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Specimen | U2 | C5 | CC2 | S1 |
| Reference | Vu and Li (2018a) [] | Vu and Li (2018a) [] | Vu and Li (2018b) [] | Sezen and Moehle (2006) [] |
| Section, *b x h* (mm x mm) |  |  |  |  |
| Column length, *l* |  |  |  |  |
| Experiment test configuration |  |  |  |  |
| Shear span to depth ratio, (*a/d*) |  |  |  |  |
| Concrete strength (MPa) |  |  |  |  |
| Longitudinal reinforcement |  |  |  |  |
| Yield strength of long bar ( |  |  |  |  |
| Ultimate strength of long bar( |  |  |  |  |
| Transverse reinforcement |  |  |  |  |
| Yield strength of trans reinf. ( |  |  |  |  |
| Ultimate strength of trans reinf. ( |  |  |  |  |
| Nominal Cover, (mm) |  |  |  |  |
| Axial Load, *P* (kN) |  |  |  |  |
| Corrosion level-transverse (%)\* |  |  |  |  |
| Corrosion level longitudinal (%)\* |  |  |  |  |
|  |  |  |  |  |
| Modulus of Elasticity - Concrete |  |  |  |  |
| Modulus of Elasticity – Long Reinf |  |  |  |  |
| Modulus of Elasticity – Trans Reinf |  |  |  |  |
| Ductility ratio |  |  |  |  |
| Yield displacement, (mm) |  |  |  |  |



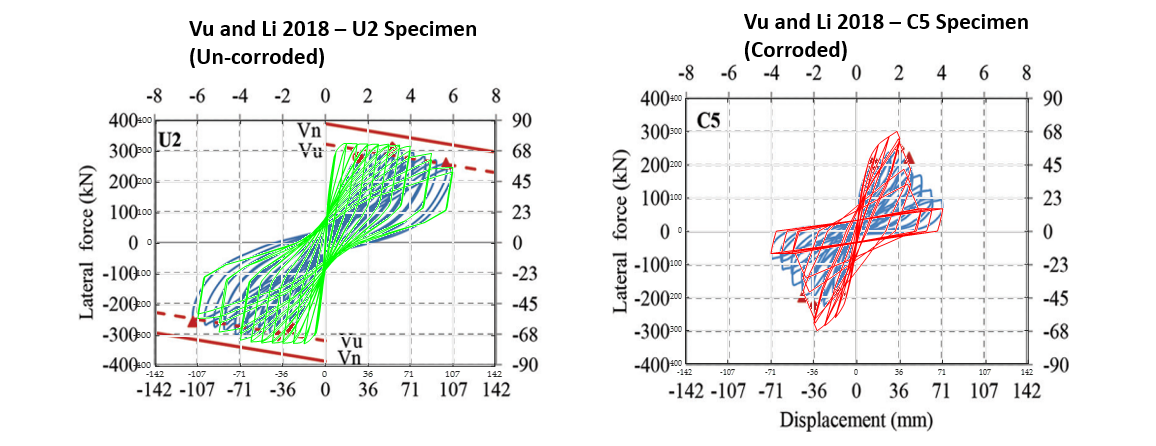
**Figure 3.1 a**



**Figure 3.1 b**



**Figure 3.2 a**



**Figure 3.2 b**

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Among the several viable techniques, the use of dissipative braces has emerged to be an efficient retrofit strategy [4]. These braces are usually constituted by a steel bracing system incorporating dissipative devices, providing a supplemental path for the earthquake-induced lateral loads, enhancing the structure's seismic performance by adding energy dissipation capacity and, in some cases, stiffness to the bare frame. Hence, when introduced within existing RC moment-resisting frames (MRFs), dissipative braces allow the reduction of the seismic response and aids in protecting both the structural and non-structural components of the building. Among others, buckling-restrained braces (BRBs) are one type of yielding device where a sleeve provides buckling resistance to an unbonded core that resists the axial stress [5, 6, 7]. As buckling is prevented, the BRB’s core can develop axial yielding in both tension as well as compression, ensuring an almost symmetric hysteretic behavior. This property allows the development of large and stable hysteretic loops, providing significant energy dissipation capacity, and hence beneficial effects to the structure's seismic performance. The use of BRBs for seismic retrofitting has been widely investigated in the last few years through components as well as large scale experimental tests [8, 9, 10, 11], extensive numerical studies [12, 13, 14, 15, 16, 17], and exploring design methods for their optimal distribution [18, 19, 20, 21]. However, while the effect of some uncertain parameters, such as the ground motion record-to-record variability, is often investigated, only a deterministic description of the dampers’ properties is usually considered.

A comprehensive study of the seismic reliability of a structural system should account for the characterization of the uncertainty in the seismic input, as well as in the geometry and mechanical properties of the structure, including those of dissipative devices constituting the lateral load resisting system, or part of it. The assessment of the propagation of these uncertainties is required to evaluate the structural failure probability, usually defined as the probability of exceeding a specific level of a monitored response parameter [22]. Previous studies have shown that the effect of model parameter uncertainty is usually negligible with respect to the record-to-record variability [23, 24, 25]; however, this may not be the case for structures equipped with dampers since their seismic response heavily depends on the properties of a few numbers of devices [26, 27, 28]. This issue is also highlighted in several design codes, such as the EN 15129, ASCE/SEI 7-16, and ASCE 41-13 [29, 30, 31] that recommends accounting for the possible variation of the device properties with respect to the nominal parameters. However, the current literature provides limited research and knowledge on the sensitivity of the dissipative device properties and influence of uncertainties on the seismic performance of the system [26, 27, 32, 33].

Dampers are produced by the manufacturer in order to meet the design values of some parameters and successively assessed by quality-control tests considering tolerance limits established by seismic and qualification codes [29, 30, 31]. Based on this standard procedure, Dall’Asta *et al.* [26] and Scozzese *et al.* [27] proposed a general framework to evaluate the effects of damper characteristics' variability on seismic risk while considering both linear and non-linear viscous dampers. The methodology proposed in Dall’Asta *et al.* [26] is based on the solution of a reliability-based optimization problem which searches the “worst” combination, *i.e.,* the one that maximizes the seismic risk variation, of the uncertain device's parameters within the range of variation allowed by the tolerance limits. It is worth mentioning that this approach differs from conventional studies wherein a statistical distribution of the structure's geometric and mechanical properties is usually adopted [23, 24, 25]. The outcomes of Dall’Asta *et al.* [26] and Scozzese *et al.* [27] show that the seismic performance may drop as a consequence of the variability within damper properties. Such decline in performance is found to be particularly significant when non-linear viscous dampers are considered. Hence, device-to-device variation is expected to strongly influence the seismic performance of the system equipped with BRBs due to their high non-linear response under seismic loads as preliminarily highlighted in Kotoky *et al.* [28].

Codes worldwide provide varying tolerance limits for different types of devices, considering different device properties and the influence of multiple factors, such as*,* imperfections related to the manufacturing process, temperature variation, and aging. Among others, the EN 15129 [29] requires the control of the devices’ variation with respect to the nominal values introducing upper and lower limits of the devices’ properties, defined by a tolerance. For ‘displacement dependent devices’, such asBRBs, the EN 15129 [29] requires performing qualification tests to show that the effective (*i.e.,* secant) stiffness *Keff,b*, and effective damping *eff,b* evaluated in correspondence to the design displacement are in good agreement with the prescribed nominal design values. Tolerances are set to ±15% to account for variation during the manufacturing process. These two control parameters (*Keff,b* and *eff,b*) exhaustively identify the primary characteristics of the device behavior. Therefore, the code-based tolerance limits are also implicitly applied to other related parameters, such as the associated device forces and displacement capacity values. Similar recommendations exist within the ASCE/SEI 7-16 [30] and other seismic codes worldwide. In particular, the ASCE/SEI 7-16 [30] allows for tolerances that could go up to ±20% from nominal design values while also allowing the designer to impose stricter tolerance limits. Contrary to the European code, that requires controlling the effective stiffness *Keff,b* and damping *eff,b*, the ASCE/SEI 7-16 [30] controls the variation in terms of device’s force *Fb* and area of the hysteretic loop *Eloop,b* measured during the tests wherein the latter parameter allows the control dissipation capacity variation of the devices. It is worthwhile to note that both the above-mentioned codes provide a standardized qualification testing procedure and detailed recommendation for the evaluation of the monitored parameters, *e.g.,* in the EN15129 [29] the variations between the properties of the devices and the nominal properties requires evaluation with reference to the 3rd cycle of the qualification test.

Until now, research studies investigating the seismic performance of structures equipped with BRBs [13, 14, 15, 17, 19, 20, 21] only considered a deterministic description of the BRBs properties. Unlike past literature, the present study investigates the influence of the BRBs uncertainty and its propagation on story- and system-level fragility curves, and subsequently on the seismic risk estimates. In particular, this study focuses on assessing the influence of device-to-device variation allowed by the codes when BRBs are used for the seismic retrofitting of existing low-ductility RC MRFs. The present study is performed on a three-story three-bay RC MRF benchmark for which laboratory test data on structural performance under dynamic and cyclic loading for the bare (un-retrofitted) frame exists in literature. For confident seismic response predictions, a state-of-the-art finite element (FE) model that can capture critical failure modes typically observed in low-ductility RC frames is developed and validated against experimental results.

Following model validation with past experimental data for the global-response as well as subassemblage behavior, non-linear time-history analyses (NLTHAs) are performed considering a large set of unscaled natural ground motion records to incorporate record-to-record variability within the seismic response. Comparison of demand data and capacity estimates for the bare as well as retrofitted frame allows the development of seismic fragility curves for different damage states (*DSs*)*:* Slight, Moderate, Extensive, and Complete. The convolution of the seismic fragility curves with the seismic hazards for three case-study regions allows the evaluation of seismic risk reduction as a consequence of the BRBs retrofit implementation within the bare frame. Subsequently, the uncertainties within BRBs parameters are captured using a two-level factorial experimental design method that considers design points as the upper and lower bounds of the recommended tolerance deviations from the nominal values. Multiple cloud analyses for the different factorial design combinations at story- as well as at system-level helps to identify the critical BRB parameter combinations and provides useful insights on the influence of BRBs’ uncertainty on the exceedance probability of different damage states. Seismic risk estimates derived using the upper-and-lower fragility bounds help evaluate the variation in lifetime risk exceedance probabilities as a consequence of device uncertainty.

# METHODOLOGY FOR BRBs UNCERTAINTY PROPAGATION

Figure 1 shows the overall framework used in this study to evaluate the impact of device-to-device uncertainty on story- and system-level fragilities and seismic risk assessment. The framework is structured in five Steps (A through E), which are briefly outlined here and detailed through the rest of the paper.

Step A aims at providing detailed numerical models for the case-study bare and the retrofitted frame structures. The FE model of the bare frame is validated against past experimental results at both components-level cyclic behavior and system-level seismic response to gain confidence in numerical modeling and thereby capture the varying failure mechanisms that may arise during seismic events. Successively, the structure's deterministic response is assessed through non-linear static (pushover) analysis, and the BRBs are designed for a possible retrofit scenario, hence providing their nominal design parameters. Numerical models for BRBs are successively calibrated based on existing experimental data and modeled within the frame. Next, Step B describes and maps the *DS* thresholds in terms of global engineering demand parameters (*EDPs*), *i.e.,* maximum inter-story drift ratio (*IDR*max) for both the bare and retrofitted frame. *DS* thresholds for the *IDR*max can be directly assumed based on the limits suggested in seismic codes [34], or by deriving the *DS* thresholds based on capacity limits at member and/or section-level, *e.g.,* rotation, strength of cross-sections, and material strains (seeFreddi *et al.* [35]), through simplified analysis procedures, such as, pushover analysis, as done in Rossetto *et al.* [36] and Aljawhari *et al.* [37]. The first of these two approaches could be suitable in new ductile structures designed by following modern seismic rules, that ensures well-established relationships between local failures and the global response. Conversely, the second approach is more appropriate to relate local and global *EDPs* in existing structures wherein the relationships between local failure and global *EDPs* may change case by case [14]. The present study follows this second approach and develops structure-specific *DS* thresholds for the *IDR*max, through pushover analyses and based on local-level *EDPs*, *i.e.,* material strains in beams and columns while also accounting for local mechanisms directly in the model, *i.e.,* shear and axial failure of columns. It is worth mentioning that the approach used is affected by the assumption of the simplified analysis procedure, *i.e.,* distribution of forces according to the first mode of vibration in pushover analyses and hence may not be appropriate for structures characterized by significant high mode effects (*e.g.,* tall buildings). A further step forward would be the use of local *EDPs* directly within the probabilistic analysis [14, 38]. However, this usually implies managing a large amount of data, that is often not practical. A detailed description of Steps A and B is reported in Section 3 of the paper.

Step C of the framework evaluates the seismic performance of the case-study structure before and after the retrofit implementation while accounting for the uncertainties related to the seismic record-to-record variability alone. As conventionally practiced, seismic performance is represented using seismic fragility curves that constitute conditional probabilistic statements for meeting or exceeding a specific *DS* given the ground motion intensity [39]. This study utilizes a cloud analysis approach [40] for fragility development wherein a suite of unscaled, yet strong enough, earthquake ground motions are used for NLTHAs such that the structure experiences behavior that spans from the linear to the non-linear domain, thereby covering the different *DSs* definitions. Based on the results of the NLTHAs, probabilistic seismic demand models (PSDMs) are developed for both the bare and retrofitted frames. These PSDMs provide a one-to-one relationships between a selected *EDP*, such as the *IDR*max, and the ground motion intensity measure (*IM*). The spectral acceleration at the fundamental structural period [*Sa*(*T*1)] is selected as the *IM* of choice in this study. Next, the PSDMs are compared with *DS* thresholds (*i.e.,* capacity limits) estimates (see Step B) to develop seismic fragility curves for both the bare and retrofitted frames.

Step D of the framework investigates the influence of device uncertainty on story- and system-level fragilities. The uncertainty in BRBs parameter is captured by a two-level factorial design strategy where parameters are held at “lower” and “upper” design points (*e.g.,* ±15% of nominal estimates in accordance with the EN 15129 [29] and conforming with the recommendations of the ASCE/SEI 7-16 [30]) and are varied independently among the devices at the different stories. Multiple cloud analyses are successively performed to account for all the cases of BRBs uncertainty identified by the two-level factorial method and considering a uniform probabilistic distribution of the upper and lower values. This step helps to draw insights on the influence of uncertainty related to the BRBs device-to-device variability on the probability of exceedance for different *DSs*. Moreover, assuming a Uniform distribution of the uncertain parameters, random combinations are selected using the Latin-Hypercube Sampling (LHS) technique [40] to evaluate the exhaustiveness of the adopted factorial design method in defining the “worst” (*i.e.,* most fragile) combination of uncertain device parameters. Finally, in Step E, seismic fragility curves are convolved with seismic hazard curves of three case-study regions to provide insights on the risk reduction obtained by the retrofit. Risk estimates are successively derived also for the cases accounting for device-to-device variations proving insights on the propagation of the BRBs’ uncertainty in terms of seismic risk. A detailed description of Steps C to E is reported in Section 4 of the paper.

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Figure 1. Proposed methodology for BRBs uncertainty propagation in seismic vulnerability assessment.

# CASE-STUDY STRUCTURE AND FINITE ELEMENT MODELLING

## Case-Study Structure

Although existing low-ductility RC frames may differ in geometry, materials, and distribution of mechanical properties of the structural components, similar failure mechanisms are often observed due to lack of design for seismic actions. Extensive studies on failure modes of RC frames, based on experimental research works and post-earthquake investigations are reported in literature [41, 42]. Among others, typical failure modes in these buildings are related to strong beam/weak column hierarchy, with beams designed to resist the bending actions and columns sized to predominantly carry axial loads, insufficient transverse reinforcement in columns, and lack of seismic detailing, such as*,* inadequate lengthof rebars anchorages, column lap splices in potential plastic-hinge regions, lack of transverse reinforcement in beam-column joints, lack of hooks, among others.

The present study selects a benchmark three-story three-bays RC MRF, representative of non-seismically designed (low-ductility) low-rise RC buildings. This structure is representative of typical constructions in several areas of the mid-west of the USA as well many countries in Europe, and similar earthquake-prone regions in Asia prior to the introduction of modern seismic design codes. While the general approach followed in this study may lead to different estimates of seismic fragilities, risk, and uncertainty for other frame geometries and retrofit configurations, the overall conclusions are expected to remain unchanged. Moreover, the availability of laboratory experimental test results from a 1:3 reduced scale model of the case-study frame [43], as well as frame-subassemblages [44], renders the selected reference structure as an ideal choice for this study. FE model validation against experimental results helps in gaining confidence in the numerical approach as well as predicted building response at both the global- and local-level.

While Figure 2 illustrates the story dimensions and beam-column arrangements of the bare frame, also depicted are the placement of BRBs: BRB-1, BRB-2, and BRB-3 for the retrofitted structure. The dissipative braces (BRBs) employed in RC MRFs are typically made by a series arrangement of two components: the BRB device and an elastic steel brace exhibiting adequate over-strength, as shown in Figure 2. The case-study frame has an inter-story height of 3.66 m (12 feet), a total building height of 10.75 m (35.30 feet), and constant bay width of 5.49 m (18 feet). The building is designed for gravity loads only, without any seismic detailing provisions following the pre-seismic design rules of the ACI 318-89 [45]. Furthermore, negligible wind loads for low-rise structures, such as the case-study frame, leads to a complete lack of accounting for lateral loads in the frame design. The building columns are constant square sections of 300 mm × 300 mm, while beams dimensions are 230 mm × 460 mm at each floor. The concrete compressive cube strength is *f*c = 24 MPa, and the reinforcing bars are Grade 40 steel with a yield strength of *f*y = 276 MPa. Further details on the case-study structure and reinforcement configurations within beams, columns, and beam-column joints can be found in Bracci *et al.* [43] and Aycardi *et al.* [44]. According to Steps A and B of the framework presented in Figure 1, the following subsections detail the FE modeling strategy, model validation, BRB design, and *DS* thresholds for both bare and retrofitted frames.

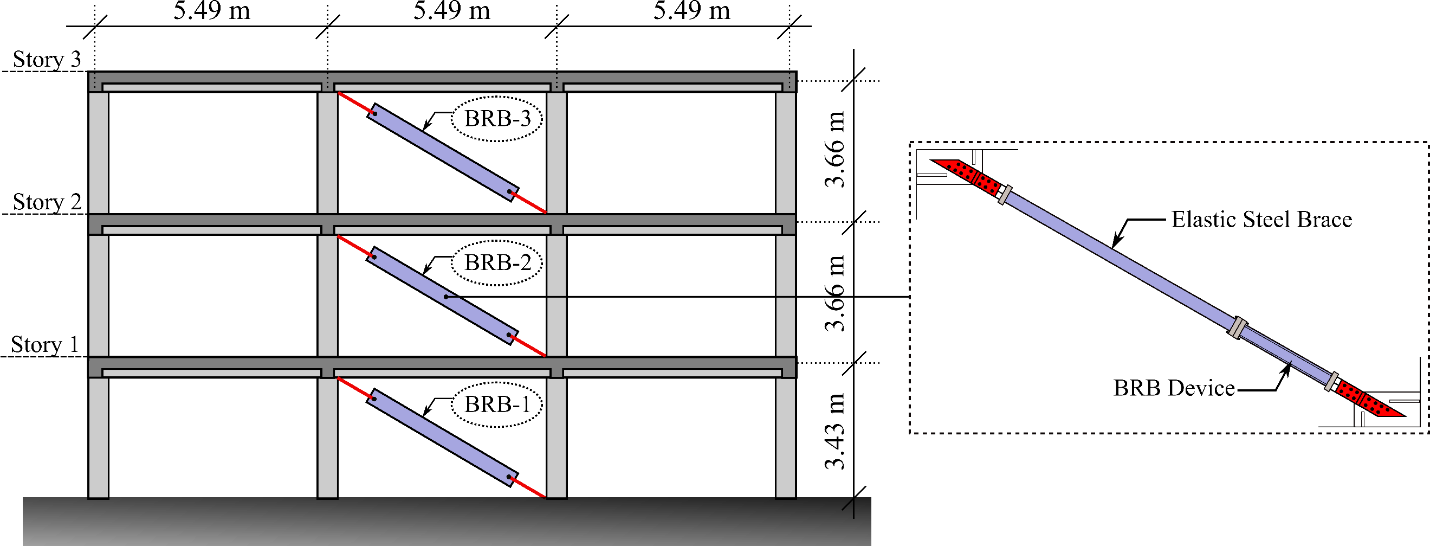


Figure 2. Case-study bare-frame layout (adapted from Bracci *et al.* [43]) also showing placement and arrangement of BRBs for the retrofitted frame.

The FE model strategy for the case-study frame follows the one presented in Freddi *et al.* [14, 35], but offers considerable improvements that focus on: a) better prediction of the local seismic response of structural components, including brittle failure mechanisms that are typical of low-ductility RC MRFs and, b) better representation of the BRBs cyclic behavior by using an advanced material model [7]. It is noteworthy that the high-fidelity models developed in this study enables modelling of local failure mechanisms that is deemed essential while monitoring global response parameters.

## Bare Frame: Modeling and Validation

The FE package OpenSees [46] is used to develop a state-of-the-art two-dimensional model of the case-study frame. Figure 3 shows a schematic representation of the model along with the modeling details of the different components such as beam and column sections, plastic hinge locations, interior and exterior beam-column joints, and shear and axial springs. The non-linear flexural hysteretic response of beams and columns is simulated using the *beamWithHinges* element that consists of a central elastic element and two plastic hinge regions at the elements ends defined by fiber sections [47]. The effective flexural stiffness of the elastic portion of the element is evaluated by the ratio of the moment and the curvature corresponding to the yielding of the first rebar of the section. The plastic hinge lengths for both beams and columns are evaluated based on Panagiotakos and Fardis [48]. In these regions, fiber sections are defined that consider the spread of plasticity within unconfined (cover) concrete, confined (core) concrete, and layers of longitudinal reinforcement. While the core and cover concrete within the fiber sections are modeled using the non-linear degrading *Concrete02* [46] material model, the *Hysteretic* [46] material model is used to model the longitudinal reinforcements. For this material, the parameters controlling pinching, damage, and degraded unloading stiffness are calibrated such that close agreements are attained between the numerical and experimental results for model validation, as elaborated later. The slab is modeled using unconfined concrete material model with an effective width equal to four times the beam's width, as recommended in the ACI 318-89 [45]. The rigid-floor diaphragm is modeled by assigning high axial stiffness to the beams. Gravity loads are distributed on the beams while masses are concentrated at the beam-column intersections.

### Modeling of Shear and Axial Failure within Beams and Columns

While the *beamWithHinges* element used to model beams and columns can adequately capture the flexure behavior, low-ductility frames may also experience non-linear behavior related to shear failure of columns and subsequent loss of gravity load-bearing capacity [49]. Moreover, for the retrofitted frame, BRBs transfer additional axial forces to the columns that could potentially lead to axial failure [14]. Z*eroLength* shear and axial springs are introduced at the top of each column, as shown in Figure 3, by assigning them the *LimitState* uniaxial material[46]. This material model monitors the column response and triggers only when column response reaches the pre-defined shear and axial failure curves, implemented within OpenSees as *limitCurve Shear* and *limitCurve Axial*, respectively.

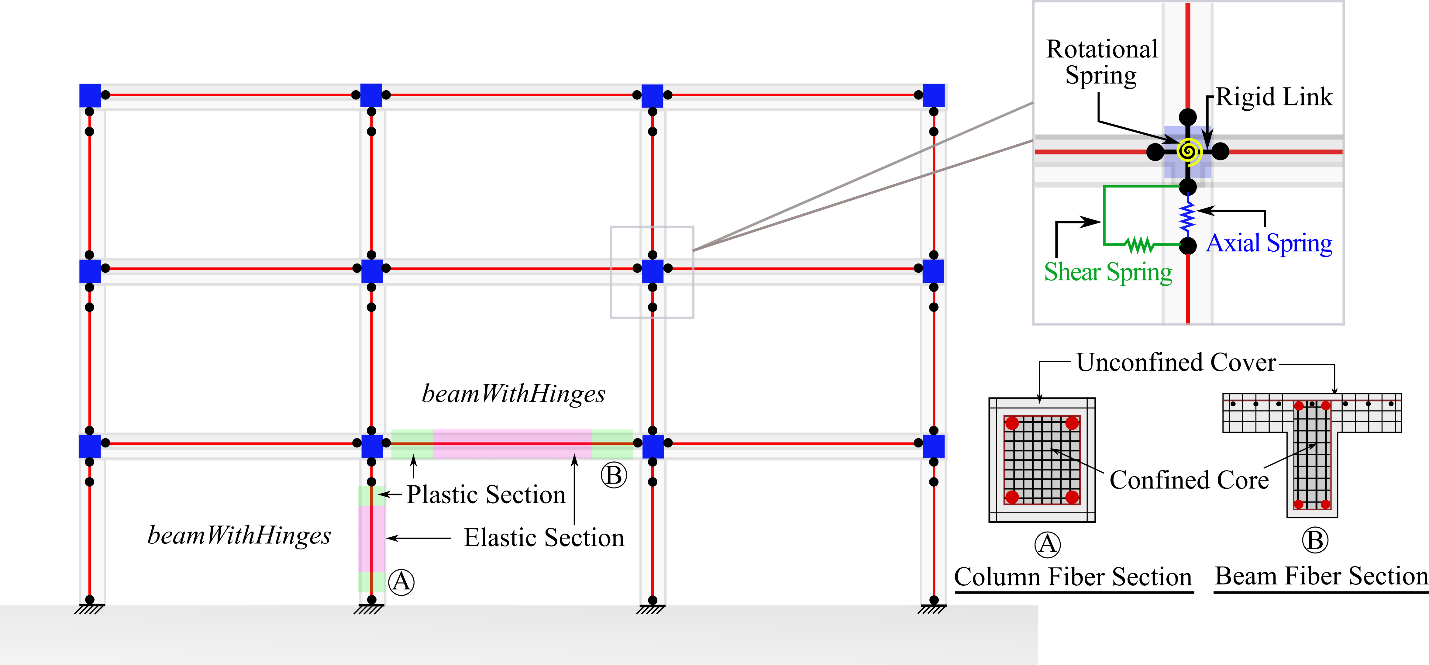


Figure 3. Overview of the numerical modeling strategy for case-study frame.

The *limitCurve Shear* model developed by Elwood [49] is based on an empirically derived force-deformation controlled limit curve that depends on parameters, such as column transverse reinforcement ratio, column axial load, section dimensions, material properties, among others. However, this model is applicable only for columns that yield before experiencing shear failure, that is*,* flexure-shear critical columns, and for drifts greater than 0.01. For a column experiencing pure shear failure, that is, failure before flexure yielding, the Elwood [49] model alone may not be appropriate. Therefore, to capture pure shear failure, this model is combined with the force-controlled shear limit surface proposed by the ASCE-41 [31]. Consequently, through changes within the OpenSees program architecture, the existing *LimitState* material model and *limitCurve Shear* model are modified such that shear failure is triggered when column response reaches either the strength or drift limit curves as shown in Figure 4(a). Once shear failure is detected, the shear springs' properties are updated to represent the expected degrading stiffness of the element. While the degrading slope of the total response *Ktdeg* (Figure 4(a)) is computed based on the shear-friction model proposed by Baradaran Shoraka *et al.* [50], the shear spring response stiffness (*Kdeg*) is estimated from the degrading slope of the total response (*Ktdeg*) and unloading stiffness of flexural response (*Kunload*) [49]. Lastly, the residual shear strength (*Vres*) is assumed equal to 20% of the initial shear strength (*Vn*) to reduce convergence issues during the analyses. For axial failure, a preliminary investigation revealed that the geometric layout, dimensions, and reinforcement ratios within the building columns are sufficient, not only to resist the gravity loads for the bare frame but also additional loads imposed from the BRB device while resisting seismic forces in the retrofitted frame. Consequently, pure axial failure due to loss of load-bearing capacity is not modeled. However, this study explicitly models axial failure that may be triggered once shear failure initiates within the column by using the *limitCurve Axial* developed by Elwood and Moehle [42] as shown in Figure 4(b).

|  |  |
| --- | --- |
| a) | b) |

Figure 4. a) Shear limit curves capable of capturing pure shear as well as flexure-shear failure, and b) axial limit curve capable of triggering axial failure beyond initiation of shear failure.

### Modeling of Beam-Column Joints

Interior and exterior joints in low-ductility RC frames can be subjected to failures due to excessive shear demand, leading to concrete cracking in tension, concrete crushing in compression, or due to loss of bond between the steel reinforcing bars and the surrounding concrete resulting in anchorage failure. Joint shear failures are typically related to insufficient transverse reinforcement in the joint, while bond failures occur due to insufficient embedment length of beam bottom reinforcement within the joint. Figure 5(a) shows the details of the interior and exterior joints of the case-study frame characterized by both these typical shortcomings. Figure 5(b) shows a representation of the FE model of the joints in which the joints region is modeled using a two-node *zeroLength* rotational joint spring and four rigid offsets as done in Jeon *et al.* [51]. In this model, beams and columns are continuous, while the joint model controls their relative rotation. To account for the short embedment lengths of bottom reinforcements of beams within the beam-column joints, a reduced shear strength is considered according to Jeon *et al.* [51], as shown in Figure 5(c). The *Pinching4* [46] material model is used to define the beam-column joint response as shown in Figure 5(d).

### Model Validation: Component-level

The model validation at component-level utilizes the response data of columns and subassemblages of the 1:3 scaled model of the case-study frame from Aycardi *et al.* [44]. Figure 6(a) compares the numerical and experimental results for an external beam-column subassemblage specimen with cyclic lateral drifts up to 3%. The comparison demonstrates the FE model's capability to simulate the cyclic behavior with reasonably satisfactory confidence. Similar comparisons were also conducted for internal and external columns and for an internal beam-column subassemblage. These results are similar to those reported in Freddi *et al.* [14, 35] and, for sake of brevity, are not reported here. While the results from the laboratory tests on the scaled model did not indicate pure shear or flexure-shear failure of columns under cyclic loading or during the application of limited ground motion accelerograms, the model capability to capture these failure modes was tested against other documented test data. Figure 6(b) compares experimental and numerical results for a flexure-shear critical column reported by Sezen and Moehle [52]. The results show that, even for this failure mode, the numerical models satisfactorily simulate the peak shear force and displacement, reasonably capturing strength degradation, stiffness degradation, and energy dissipation.

### Model Validation: System-level

The model validation at system-level was conducted by using the 1:3 scaled model test results from Bracci *et al.* [43]. In this regard, the first three natural periods provided by the 1:3 scaled FE model with uncracked gross stiffness properties obtained as 0.552 s, 0.172 s, and 0.110 s, are found to be in close agreement with those measured by snap-back and white noise tests (0.537 s, 0.176 s, and 0.119 s). A satisfactory agreement is also observed for the first three mode shapes. Additionally, shake table tests results reported in Bracci *et al.* [43] describes the time-history of the 1:3 scaled benchmark frame response under the Kern County 1952, Taft Lincoln School Station, N021E component record scaled for different levels of the seismic intensity with peak ground acceleration of 0.05g, 0.20g, and 0.30g. The simulated numerical test results considering the top story and inter-story displacements show a satisfactory agreement with the experimental results for the three intensity levels. These results, demonstrating the adequacy of the FE model in simulating the global response of the structure, do not show significant improvements with respect to what shown in Freddi *et al.* [14, 35] and, for sake of brevity, are not reported here. It is worth noting that damping sources other than the hysteretic energy dissipation are modeled through the Rayleigh damping matrix where the values of mass-related and stiffness-related damping coefficients are adopted from snap-back test results [43].

|  |
| --- |
| b)  d)  a)  c) |

Figure 5. Modeling of beam-column joints. a) Typical reinforcing details in joints of low-ductility frames; b) rotational springs controlling relative beam-column rotation; c) backbone curve joint shear-strain relationship based on Jeon *et al.* [51]; d) backbone curve moment-rotation relationship of joints.

|  |  |
| --- | --- |
| a) | b) |

Figure 6. Numerical and experimental lateral load-drift cyclic response comparison for a) exterior 1:3 scaled beam-column subassemblage of the case-study frame [44], and b) flexure-shear critical column specimen 1 of [52].

## Retrofitted Frame: BRB Design and Numerical Modelling

While extensive details on the methodology used for BRBs design can be found elsewhere in literature [14, 18, 19], a brief outline of the design process and the parameters involved is presented herein. The primary objectives that dictate the BRBs design process comprises of a) Defining BRBs dimensions such that they produce a controlled increase of the base shear capacity of the system, that is, the base shear of the dissipative system (*Vd,*1) when added to the base shear of the bare frame (*Vf,*1); b) Distributing the stiffness of BRBs among the stories, such that, the first mode shape of the bare frame remains unchanged following the retrofit implementation. This aims at avoiding drastic changes to the moments distribution within the MRF; c) Distributing the BRBs strengths among the stories to ensure simultaneous yielding of the BRB devices. This condition is usually sought in the design in order to maximize the dissipation capacity of the system; d) Calibrating the stiffness and ductility of the BRBs such that the device failure occurs at a design displacement (*du*) defined as per the ductility capacity of the bare frame*.*

The BRB design procedure is based on the displacement distribution of the first vibration mode and uses non-linear static analysis of the bare frame and a Single-Degree-of-Freedom (SDoF) simplification for the definition of some design parameters which are related to the retrofit objectives, such as the design displacement (*du*); the target ductility of the dissipative braces (*d*) and the base shear capacity of the dissipative system (*Vd,*1). The design method provides the strength *Fd,i,* and stiffness *Kd,i* of the BRBs at each story. Following this, the components' properties, such as length, area, and materials of BRB devices and elastic braces, can be easily derived according to a series arrangement.

As shown in Figure 2, the retrofitting is performed by introducing the BRBs in the central bay at each story of the case-study frame. The base shear and the design displacement of the bare frame, defined based on the pushover analysis, are respectively *Vf,*1 = 180.72 kN and *du* = 0.308 m. The value of the design displacement (*du*), is selected close to the maximum lateral displacement capacity of the bare frame corresponding to the Complete *DS* described in the following Section 3.4, fully exploiting the capacity of the RC frame. The base shear (*Vf,*1) is defined after bi-linearizing the capacity curve based on an elastic-perfectly plastic model.

In the present study, the retrofit is designed independently from a specific seismic hazard, and the base shear of the dissipative system (*Vd,*1) is selected equal to the base shear of the bare frame (*Vf,*1), or in other words,strength proportion coefficient *α* =*Vd,*1*/Vf,*1 equals unity [14, 17], hence doubling the base shear resistance of the retrofitted frame. The ductility of the dissipative braces, *i.e.,* dissipative device plus elastic brace (*d*) is assumed equal to 15. The design method provides properties such as strength *Fd,i* and stiffness *Kd,i* of the dissipative braces at each story as summarized in Table 1. Assuming the ductility of the BRB devices (*µBRB* = 20), the yielding resistance of the materials for BRB devices (*fy,BRB* = 250 MPa) and elastic braces (*fy,eb* = 355 MPa), and based on strength *Fd,i*, and stiffness *Kd,i* of the dissipative braces, the properties of the components can be easily derived as reported in Table 1.

Table 1. Design properties of the dissipative braces and components.

|  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Story | Dissipative Braces | | BRB devices | | | | Elastic brace properties | | | |
| *Fd,i*  [kN] | *Kd,i*  [kN/mm] | *ABRB,i*  [mm2] | *LBRB,i*  [mm] | *FBRB,i*  [kN] | *KBRB,i*  [kN/mm] | *Aeb,i*  [mm2] | *Leb,i*  [mm] | *Feb,i*  [kN] | *Keb,i*  [kN/mm] |
| 1 | 213.1 | 38.26 | 852.4 | 3447.6 | 213.1 | 51.92 | 1815.4 | 2622.2 | 613.8 | 145.4 |
| 2 | 183.4 | 25.75 | 733.6 | 4408.1 | 183.4 | 34.95 | 832.1 | 1785.7 | 281.3 | 97.9 |
| 3 | 105.6 | 23.51 | 422.3 | 2780.2 | 105.6 | 31.90 | 1452.0 | 3413.6 | 490.9 | 89.3 |
| \*The steel used for elastic braces and BRB devices have respectively *fy,eb* = 355 MPa and *fy,BRB* = 250 MPa. | | | | | | | | | | |

The dissipative braces are modeled by two elements in series representing respectively the BRB device and the elastic brace. The *‘steelBRB’* material model [7, 53] is used to describes the hysteretic behavior of the BRBs while capturing the kinematic and isotropic hardening, along with the tension-compression asymmetry that typically characterizes these devices, such that the maximum forces resisted by BRBs in compression are typically 10% to 15% higher than forces resisted in tension [7]. In this study, the values of the parameters controlling the kinematic and isotropic hardening as well as the asymmetric behavior in tension and compression have been calibrated based on the results of the experimental qualification tests performed by a manufacturer as shown in Figure 7.

The modal analyses show that the introduction of the BRBs within the existing bare frame produces a reduction of the structural period from *T*1 = 1.2 s (bare frame) to *T*1 = 0.6 s (retrofitted frame) and confirms that the first modal shape remains unchanged after the retrofit, as expected from the design procedure.

|  |
| --- |
|  |

Figure 7. Comparison of numerical and experimental cyclic response for a BRB device.

## Threshold mapping of Damage States (DSs)

During seismic events, building frames may incur structural damage that can be qualitatively and quantitatively described using discrete *DSs*, such as Slight, Moderate, Extensive, and Complete [34]. These *DS* definitions, when used in conjunction with structural demands imposed on buildings, enable the construction of seismic fragilities functions. Structure-specific *IDR*max thresholds for four *DSs* are calibrated via non-linear static (pushover) analysis with a load distribution according to the first vibration mode (Figure 8), by assessing multiple measurable criteria [36, 37]. Table 2 lists the local-level physical description of building damage associated with each of the *DSs*. Figure 8 depicts the results of the pushover analyses in terms of base shear vs. *IDR*max curves, including markers corresponding to the attainment of the *DSs*. Among all the stories, it is observed that Story 2 experiences largest *IDRs.*

Table 2. Damage states (*DSs*) description and *DS* thresholds mapping.

|  |  |  |  |
| --- | --- | --- | --- |
| Damage states | Description | *DS* Thresholds | |
|  | Building Frame | Bare Frame | Retrofitted Frame |
|  |  | *IDR*max [ % ] | |
| Slight (S) | Yielding of 50% of columns at one story | 0.81 | 0.72\* |
| Moderate (M) | Crushing/Spalling of concrete in 50% of columns at one story | 1.53 | 1.81\* |
| Extensive (E) | Average of Moderate and Complete *DSs* | 2.27 | 2.40\* |
| Complete (C) | Initiation of shear failure in 50% of columns at one story | 2.98 | 2.99\* |
|  | BRB Devices | Max ductility demand [ - ] | |
| Complete (C) | Fracture of one BRB device at any story | - | 25 |
| \*Average value considering pushover analysis along two directions | | | |

As per the design, the introduction of the BRBs, for the same level of *IDR*max, produces a base shear approximately twice that of the bare frame. It is noteworthy that the asymmetric force redistribution due to BRB placement requires the pushover analysis to be conducted along both directions, as shown in Figure 8(a). Each *DS* corresponds to two distinct markers, corresponding to the different directions of pushover force application leading to changes in compression and tension in the frame members due to the additional axial force imposed by the BRBs. Figure 8(b) and Table 2 show a reduction of the Slight *DS* threshold for the retrofitted frame as compared to the bare frame. This is related to an anticipated yielding of the column because of the redistribution of axial loads in the columns of the retrofitted frame due to BRBs. Conversely, the redistribution of axial loads causes an increase of Moderate *DS* threshold value in the retrofitted frame as compared to the bare frame. The Complete *DS* threshold is related to shear failure initiation and remains nearly unaffected by the redistribution of axial loads. The Extensive *DS* threshold is defined as the average of Moderate and Complete *DS*. Lastly, for the BRB device, a maximum ductility demand of 25 is adopted as threshold values, corresponding to the Complete *DS*, based on the recommendations by Fahnestock *et al.* [12]. While these quantitative estimates of the *DS* thresholds are assumed to be a measure in a median sense, a dispersion of 0.3 [34] is assumed to account for the uncertainty associated with *DS* definitions.

|  |  |
| --- | --- |
| BRB Frame (-ve)  Bare Frame  BRB Frame (+ve)  a) | b) |

Figure 8. Non-linear static analysis for the bare and retrofitted frame. a) Representation of the analyses cases and directions; b) Base shear vs. *IDR*max curves and mapping of *IDR*max values with *DS* thresholds.

# SEISMIC FRAGILITY CURVES AND UNCERTAINTY ANALYSIS

Seismic fragility curves for both frame-types and the different damage states (*DSs*) are developed based on cloud analysis approach [40] that utilizes probabilistic seismic demand models (PSDMs) and *DS* thresholds derived in Section 3.4. A suite of 150 unscaled ground motions from the SIMBAD database [54] is utilized for the NLTHAs of the building models and subsequently derive general non-site specific fragility estimates. The SIMBAD database comprises a total of 467 tri-axial accelerograms produced by 130 seismic events globally and provides a statistically significant number of strong-motion records of engineering relevance. The database includes shallow crustal earthquakes with moment magnitudes ranging from *Mw* = 5 to 7.3 and epicenter distance less than 35 km. A subset of 150 records is considered here to provide a statistically significant number of strong-motion records of engineering relevance. These records are selected by first ranking the 467 records in terms of their PGA values (by using the geometric mean of the two horizontal components) and then keeping the component with the largest PGA value, *i.e.,* the maximum component for the 150 records with the highest mean PGA. The minimum and maximum PGA of the selected suite of ground motion are 0.21g and 1.78g, respectively. According to Steps C, D, and E of the framework presented in Figure 1, the following section first compares the deterministic performance of the bare and retrofitted frames under a selected accelerogram, followed by PSDM construction, fragility development, uncertainty propagation of the BRB device, and eventually seismic risk comparison.

## Seismic Response and Fragility Comparison of Bare and Retrofitted Frame with Nominal Design Parameters

An instance of the comparison between the deterministic response of bare and retrofitted frame is depicted in Figure 9. Figure 9(a) shows the time-history response and the inter-story drift ratio reduction for Story 1 (*IDR*1) for the bare and retrofitted frame (with nominal design parameters) when subjected to a ground motion with PGA of 0.53g (*i.e.,* the *y*-component record, NIG020 Station, of the *Mw* 6.6 Mid Niigata Prefecture Earthquake on 23rd October, 2004 at 8.56 UTC). Figure 9(b) and (c) depict the reduction in bending moment and shear force in the interior column A (as indicated in Figure 9) due to the increased lateral strength and stiffness, together with the additional damping capacity provided by the BRBs in the retrofitted frame. While such beneficial effects of reduced inter-story drifts, moments, and shear forces are due to the inclusion of BRBs, interior columns of the retrofitted frame are subjected to additional axial forces, as shown in Figure 9(d). However, a closer inspection reveals that across all ground motions, the increased axial demand on the building columns during dynamic response is still below the axial capacity of the columns and that even in the retrofitted case-study frame, they are not susceptible to pure axial failure. It may be noted that to satisfy equilibrium the bracing also introduces appreciable axial load in the horizontal structural elements. However, in this case, the presence of the rigid-floor diaphragm leads to negligible changes in the response of beams under seismic loads.

|  |  |
| --- | --- |
| a) | b) |
| d)  c) |  |

Figure 9. Comparison of time-history response between bare and retrofitted frames. a) Inter-story drift ratio of Story 1 (*IDR*1); b) Moment in column A of the Story 1; c) Shear force in column A of the Story 1; d) Axial load in column A of the Story 1 for the *y*-component record, NIG020 Station, of the *Mw* 6.6 Mid Niigata Prefecture Earthquake of the 23rd October, 2004 at 8.56 UTC.

Following the deterministic analysis, the randomness in ground motion characteristics is incorporated within the PSDMs. These models are developed after aggregating the *IM*-*EDP* pairs following the 150 NLTHAs of the bare and retrofitted frames. As mentioned earlier, while the maximum inter-story drift ratio (*IDR*max) constitutes the *EDP* of choice, the *IM* is taken as the spectral acceleration at the fundamental time period [*Sa*(*T*1)]. Following the power-law form proposed by Cornell *et al.* [40], when transformed to the logarithmic space, the relationship between the median *EDP* (*EDPmed*) and *IM* can be expressed as:



where, *a* and *b* are the linear regression coefficients. The regression relationship of Eqn. also keeps a track of the zero mean normally distributed model fitting error *βD*|*IM* that assists towards fragility development. While a direct comparison of the PSDMs is infeasible owing to different fundamental structural time periods (1.2 s for the bare frame and 0.6 s for the retrofitted frame), consistently high *R*2 estimates of 0.90 and 0.83 for the bare and retrofitted frame, respectively, underlines the adequacy of the chosen *IM* as *Sa*(*T*1) for *IDR*max predictions. For the BRB devices within the retrofitted frame, the maximum ductility demand, defined as the ratio of peak strain due to ground motion and the device's yield strain, is chosen as the *EDP*. The PSDM *R*2 estimates for the device alone within the retrofitted frame is equal to 0.82. The regression coefficients for the bare frame, retrofitted frame with nominal parameters, and the BRB device are depicted in Figure 10 below.

The PSDMs and the *DS* thresholds estimates are now utilized to develop seismic fragility curves for the bare and retrofitted frame and for the BRB devices. Assuming that the structural demand and capacity estimates follow lognormal distributions, the seismic fragility curves for a particular *DS* given a ground motion *IM* may be conveniently obtained as:



where, *EDPmed* and *SC* are the median estimates respectively for the seismic demand (Eqn. ) and *DS* threshold (Table 2), *βEDP*|*IM* and *βC* represent the corresponding lognormal standard deviations, and  and  represent the median and dispersion of the lognormally distributed fragility curves. Figure 11(a) and (b) show the seismic fragility curves corresponding to the different *DSs* for the bare and the retrofitted frame together with the fragility for the BRB device. While the difference in the fundamental period hinders a direct comparison of bare and retrofitted frame fragilities, computing the seismic risk aids in highlighting the benefit of the intervention during earthquake events, as reported later in Section 4.3. It is worth noting that the fragility curve of the BRB devices lies close to the one for the retrofitted frame corresponding to the Complete *DS*. This is in line with the BRBs’ design objectives to avoid drastic changes to the distribution of internal action distribution in the frame and achieve nearly simultaneous failure of both the frame and the braces.

|  |  |
| --- | --- |
| ln(*IDR*max) = 1.58 + 0.82 ln[*Sa*(*T*1)]  a) | ln(*EDP*) = 2.37 + 0.90 ln[*Sa*(*T*1)]  ln(*EDP*) = 0.32 + 0.93 ln[*Sa*(*T*1)]  b) |

Figure 10. Probabilistic seismic demand models depicting regression coefficients and goodness of fit *R*2 estimates for a) Bare frame; b) Retrofitted frame with nominal design parameters and the BRB device.

|  |  |
| --- | --- |
| a) | b) |

Figure 11. Seismic fragility curves. a) Bare frame; b) Retrofitted frame and BRB device for different damage states.

## Influence of uncertainty in BRB parameters on the seismic vulnerability of retrofitted frame

The preceding section of the paper focused on investigating the behavior of the retrofitted frame with device parameters held at nominal design values accounting only for the seismic input uncertainty, that is, ground motion record-to-record variability. The present part of the paper expands the previous section by accounting for the uncertainty stemming from device-to-device variation, and investigating the subsequent influence on story- and system-level fragility. As previously discussed, for ‘Displacement-dependent devices’, such asBRBs, the EN 15129 [29] requires quality-control tests to show that the effective (secant) stiffness *Keff,b*, and effective damping *eff,b* are in good agreement with the prescribed nominal design values and within prescribed tolerance limits [29, 30]. Alternatively, the ASCE/SEI 7-16 [30] controls the variation in terms of device’s force *Fb* and area of the hysteretic loop *Eloop,b* measured during the tests. In both approaches, the two control parameters exhaustively identify the main characteristics of the device behavior. Therefore, the code-based tolerance limits are implicitly applied to other related parameters.

In the present study, stiffness, force and dissipation capacity variations are numerically reflected within the model by accounting for variations in the area of the BRB devices (*ABRB*) while keeping the material yield strength (*fy,BRB*) as constant. Variation in the BRB device area (*ABRB*) has been chosen as opposed to the material yield strength (*fy,BRB*) as it induces the highest variation in the device response since alteration of the BRB area affects both stiffness, strength and the hysteretic energy dissipated. Device-to-device variation is assumed in agreement with the tolerance limits allowed by the codes (*i.e.,* ±15% in accordance with the EN 15129 [29] and conforming with the recommendations of the ASCE/SEI 7-16 [30]) and applied independently among the devices at the different stories.

To illustrate the bounds on system- and story-level fragilities, this study utilizes the two-level factorial design of experiments to generate eight distinct combinations of *ABRB* across the different stories, as shown in Table 3. Each combination represents a design run that generates a unique fragility curve following the NLTHA runs, PSDM developments and comparison against the retrofitted frame *DSs* as described in the preceding section. Within the table, the upper (+) and lower (-) estimate of the device area corresponds to +15% and -15% deviation, respectively, from the nominal design parameter.

Table 3. Combinations of BRB parameters generated using two-level factorial design.

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Combination | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| *ABRB3* | + | - | - | + | - | + | + | - |
| *ABRB2* | + | - | - | + | + | - | - | + |
| *ABRB1* | + | - | + | - | - | + | - | + |
| Note: The lower level (-) represents -15%, and the upper level (+) represents +15% variation in the design area of BRB (*ABRB*). BRB-1, BRB-2, and BRB-3 locations are those depicted in Figure 2. | | | | | | | | |

### Uncertainty analysis at system-level

Figure 12(a) depicts the seismic fragility curves within the colored bands corresponding to the Slight, Moderate, Extensive, and Complete *DSs* for all the design combinations included in Table 3. Figure 12(a) also includes the seismic fragility curves corresponding to the nominal design parameters, shown using the bold line for each *DS*. The figure reveals interesting trends. Firstly, the uncertainty bands for each *DSs* underline the significant variation in vulnerability because of the uncertainty stemming from the BRB parameters. For instance, the percentage difference in median estimates between the upper and lower bounds emerge as 18%, 20%, 21%, and 21% for the Slight, Moderate, Extensive, and Complete *DSs*, respectively. This underlines the impact of the BRBs' uncertainty and the need to define appropriate safety coefficients for their design as functions of the tolerance limits adopted in the quality-control tests. Secondly, the lower bound for each *DS* denoting the “best” combination, *i.e.,* the least fragile structure, consistently coincides with combination 1, wherein *ABRB* for each device is held at the upper bound of +15% from the design parameter. This finding is expected since an increase in design parameters uniformly in all the three stories leads to an increase in stiffness and strength, making the overall frame less fragile. Lastly, the upper bound of fragility for each *DS*, representing the “worst” combination, tends to converge towards combination 2, wherein *ABRB* for each device is -15% from the design values. It is also worth mentioning that fragility curves for combinations 2 and 7 are almost identical since the vulnerability of Story 3 is least compared to the other two stories as will be elaborated later. The median (*med*) and dispersion (*ζ*) of the lognormal fragility curves (Eqn. ) for the bare and retrofitted frame are reported in Table 4.

|  |  |
| --- | --- |
| a) | b) |

Figure 12. Seismic fragility of the retrofitted frame for Slight, Moderate, Extensive, and Complete damage states (*DSs*) for a) the mean and 8 combinations of BRB parameters using two-level factored factorial design; b) the 42 + 8 combinations of BRB parameters using Latin-Hypercube Sampling [40].

Table 4. Median (*med*) and dispersion (*ζ*) of the lognormal fragility curves for the bare and retrofitted frame.

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | Slight | | Moderate | | Extensive | | Complete | |
|  | *med* | *ζ* | *med* | *ζ* | *med* | *ζ* | *med* | *ζ* |
| Bare Frame | 0.12 | 0.48 | 0.25 | 0.48 | 0.40 | 0.48 | 0.55 | 0.48 |
| BRB Frame | 0.52 | 0.54 | 1.38 | 0.54 | 1.86 | 0.54 | 2.35 | 0.54 |
| BRB Frame Upper Bound | 0.47 | 0.53 | 1.23 | 0.51 | 1.64 | 0.51 | 2.06 | 0.51 |
| BRB Frame Lower Bound | 0.56 | 0.56 | 1.50 | 0.56 | 2.04 | 0.56 | 2.58 | 0.56 |

While the two-level factorial design strategy adopted for uncertainty depiction capture the extremities of parameter bounds, there may be other intermediate combinations leading to seismic fragilities that may fall outside the bounds depicted in Figure 12(a). To test the exhaustiveness of the proposed approach, an additional 42 Latin Hypercube-based [40] based fragility runs are conducted wherein random estimates of *ABRB*1, *ABRB*2, and *ABRB*3 are sampled assuming a Uniform distribution with -15% and +15% of the design parameters as the lower and upper bounds. It is worthwhile to note that although for a three-dimensional sample space (*ABRB*1, *ABRB*2, and *ABRB*3), a total of 10 × 3 = 30 Latin Hypercube samples as a space filling experimental design would be sufficient [55], 42 points are chosen, such that along with the eight factorial design runs, a total of 50 experimental design runs is attained in this study. The median estimates of seismic fragility for these 42 Latin-Hypercube runs are also found to fall within the lower (“worst” combination) and upper (“best” combination) bounds from the factorial runs, as shown in Figure 12(b) for the moderate and complete *DSs*. These trends remain same for slight and extensive damage states and only two damage states are shown in Figure 12(b) for clarity. The above observations render confidence that the factorial design, adopted for the definition of the combinations, are sufficient to predict the uncertainty band around the seismic fragility curves.

### Uncertainty analysis at story-level

While the previous section highlighted the influence of BRB uncertainty on system-level fragility, this section explores the impact at story-level fragility. Note that developing story-specific fragility curves utilizes story-specific peak inter-story drift ratios and story-specific *DS* thresholds. While the peak inter-story drifts ratios are conveniently obtained from NLTHAs, the story-specific *DS* thresholds estimation requires *DS* mapping between global and local parameters by considering each story independently. Although not presented here, the story-specific *DS* thresholds are found to be similar to those obtained for the whole system, as presented in Section 3.4. Consequently, system-level *DS* thresholds are also used herein to develop story-level fragility curves.

Figure 13 shows the uncertainty bounds for each story-level fragility corresponding only to the Moderate and Complete *DSs* along with a pictorial depiction of the *ABRB* parameter (+ or –) leading to story-specific “best” and “worst” combinations. These observations emerge consistent across all *DSs*. It is interesting to observe that the variation in terms of story-level fragilities due to the BRBs’ uncertainty is significantly higher than the one at system-level. The percentage difference in median estimates between the upper and lower bounds observed for the system-level fragilities was equal to 21% for both Extensive and Complete *DSs*. Conversely, for the most vulnerable story within the retrofitted structure – Story 2, these percentage differences emerge as 42% for the Extensive *DS* and 44% for the Complete *DS*, almost twice as high as the system-level differences.

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| --- | --- |
| Upper Bound (UB)  a)  Complete DS  Moderate DS  Lower Bound (LB)  Nominal | b) |
| c) | |

Figure 13. Seismic fragility of the retrofitted frame for the “best” and the “worst” combinations of BRB parameters for Moderate and Complete damage states (*DSs*) for a) Story 1; b) Story 2; c) Story 3.

In all stories and *DSs*, it is observed that the lower bound of story-level fragilities correspond to the cases with a decrease of BRB’s area (*i.e.,* -15%) at that story and an increase (*i.e.,* +15%) in the other two stories. The opposite trend is observed for the upper bound of story-level fragilities. For instance, Figure 13(a) shows the story-level fragilities at Story 1. For this story, combination 3 from Table 3 corresponding to an increase of *ABRB1* and a decrease of *ABRB2* and *ABRB3* dictates the lower bound for each *DS* denoting the “best” combination or the least fragile structure. Conversely, combination 4 corresponding to a decrease of *ABRB1* and an increase of *ABRB2* and *ABRB3*, defines the upper bound for each *DS*, denoting the “worst” combination, or the most fragile structure. Similar conclusions can be drawn for the other two stories. Figure 13(b) shows how combinations 5 and 6 define respectively the lower and upper bound for Story 2, while Figure 13(c) shows how combinations 7 and 8 define respectively the lower and upper bound for Story 3. Interestingly, the detrimental effects on other stories are beneficial for the story under consideration. On the contrary a -15% *ABRB* for that story and +15% *ABRB* for the others leads to high inter-story drift concentration and consequent formation of soft story-mechanism.

## Comparative assessment of seismic risk and influence of uncertainty

This section first exemplifies the benefit of implementing BRB devices in the case-study low-ductility RC frame from the perspective of seismic risk and then investigates the impact of BRBs uncertainty on risk calculation of the retrofitted frame. For illustration, the present study selects three regions across the continental United States, namely Los Angeles (California), Caruthersville (Missouri), and Charleston (South Carolina), that represent locations with unique seismic hazard characteristics. While the fragility estimates derived earlier are non-site specific, ideally, the spectral shape effects for site specific risk analysis should be accounted for. The results of the present sections are provided for demonstrative purposes only. The data for the hazard curves corresponding to the specific time periods for the bare and retrofitted frame are obtained from USGS [56]. The soil site class across all case-study locations is assumed to be the boundary of *B*/*C* with the average shear wave velocity for the top 30m of soil (*Vs*30) equal to 760 m/s. Utilizing the seismic fragility curves for the bare or retrofitted frame and regional hazard information for the case-study locations, the lifetime (assumed as *T* = 50 years [57]) probability of *DS* exceedance, *PTf*, can be computed as:



where, the annual probability of *DS* exceedance, *PAf*, can be computed from the convolution of seismic fragility and hazard [*H*(*IM* = *im*)] as:



Figure 14 shows the *T* = 50 years probability of exceedance for both the bare and retrofitted frames considering BRBs with nominal design parameters across all *DSs* and case-study regions under consideration. The inset within this figure depicts the seismic hazard curves for these regions. It is worth noting the relative ‘flatness’ of the Caruthersville and Charleston hazard curves compared to the Los Angeles hazard curve represents the low probability - high consequence events in Central and Eastern US compared to the West coast. Moreover, it is worth reminding that the retrofit design is performed by imposing a strength proportion coefficient *α* =*Vd,*1*/Vf,*1 equal to unity [14, 17], merely doubling the base shear resistance of the retrofitted frame independently from a seismic hazard. Therefore, for the Collapse *DS*, the *PTf* is close to the code-based target risk for Collapse (*i.e.,* 1% in 50 years according to [30]) only for the case of Caruthersville, Missouri, while for the other two locations, the risks estimates show an overdesign of the retrofitting.

More interestingly, the comparison of the risk estimates for the bare and retrofitted frames in Figure 14, including the tabulated percentages, provides insights in terms of seismic risk reduction across all *DSs* and case-study regions. While the bare frame experiences the highest lifetime seismic risks when located in Los Angeles, California, the BRBs effectiveness also emerge as most prominent in this region with risk reductions of 66%, 85%, 83%, and 83% respectively for the Slight, Moderate, Extensive, and Complete *DSs*. The lowest risk reduction percentage (38%) corresponds to the Slight *DS* when the frame is assumed located in Caruthersville, Missouri. The retrofit effectiveness measured in terms of seismic risk reduction for the structure located in Charleston, South Carolina, lies intermediate to the other case-study locations.

While Figure 14 provides the opportunity to compare the relative performance of the BRB retrofitted frame with the bare frame, Figure 15 illustrates the percentage difference in seismic risk bounds. These percentage differences are computed as (*PTf*,*UB* - *PTf*,*LB*) / *PTf*,*LB* × 100%, wherein *PTf*,*UB* and *PTf*,*LB* correspond to the highest and lowest estimates of the 50-year seismic risks obtained following the convolution of the upper and lower bounds of the *DS* specific fragilities of the retrofitted frame with the regional hazard curves (Eqn.s and ). These results underline the substantial impact of uncertainty in BRB device parameters on seismic risk estimates variation. Across all locations, the divergence between the upper and lower bound estimates of the risk are observed to increase with the *DS*, with the highest increment corresponding to the complete *DS* indicative of structural collapse. Lastly, the relative ‘flatness’ of the hazard curves for Caruthersville and Charleston leads to a lesser percentage difference between the bounds compared to that for Los Angeles.

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|  |
| Percentage reductions in lifetime seismic risk through the implementation of BRBs as a retrofit measure   |  |  |  |  | | --- | --- | --- | --- | | Location | Los Angeles, CA | Caruthersville, MO | Charleston, SC | | Slight (S) | 66% | 38% | 53% | | Moderate (M) | 85% | 63% | 78% | | Extensive (E) | 83% | 65% | 77% | | Complete (C) | 83% | 68% | 77% | |

Figure 14. Seismic risk comparisons for the bare and retrofitted frame for different *DSs* and the three selected sites.

|  |
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|  |
| Seismic risk comparisons for the lower and upper bound of the retrofitted frame for different DSs   |  |  |  |  |  |  |  | | --- | --- | --- | --- | --- | --- | --- | | Location | Los Angeles, CA | | Caruthersville, MO | | Charleston, SC | | |  | Lower Bound | Upper Bound | Lower Bound | Upper Bound | Lower Bound | Upper Bound | | Slight (S) | 0.0978 | 0.1232 | 0.0640 | 0.0726 | 0.0215 | 0.0261 | | Moderate (M) | 0.0133 | 0.0188 | 0.0221 | 0.0281 | 0.0043 | 0.0059 | | Extensive (E) | 0.0063 | 0.0094 | 0.0139 | 0.0187 | 0.0023 | 0.0033 | | Complete (C) | 0.0033 | 0.0052 | 0.0091 | 0.0129 | 0.0013 | 0.0020 | |

Figure 15. Percentage difference between upper and lower bounds for seismic risk across all damage states (*DSs*) and case-study regions.

# CONCLUSIONS

Buckling-restrained braces (BRBs) have emerged as efficient tools for improving the seismic performance of existing structures. Systems equipped with BRBs have been widely investigated in literature, however, only a deterministic description of the BRBs’ properties is usually considered. These properties are provided by the manufacturer and successively validated by quality-control tests that ensure the variation to be limited within code-based tolerances. Therefore, the properties of devices introduced within the structure could differ from their nominal design values, potentially leading to an undesired seismic performance. This paper investigates the sensitivity of BRB device parameters and the influence of their uncertainty on the seismic response and fragility of building structures. For case-study purposes, a benchmark three-story three-bay RC moment-resisting frame is considered and modeled in OpenSees. The numerical model of the RC frame and the BRB devices are validated against experimental results to gain confidence in numerical model capabilities. Structure-specific damage state (*DS*) thresholds for the maximum inter-story drift ratio are derived through pushover analyses, based on components-level engineering demand parameters, for both the bare and retrofitted frame, and are subsequently used for fragility curves derivation. BRBs’ uncertainty is defined compatibly with standardized tolerance limits from the quality-control tests and implemented through a two-level factorial experimental design method and Latin-Hypercube Sampling (LHS). Multiple cloud analyses for the different factorial design and LHS combinations are performed to identify the critical BRB parameters combinations that provides useful insights on the influence of BRBs’ uncertainty on the exceedance probability of different damage states. Probabilistic seismic demand models are successively developed and used to derive fragility functions for the bare and retrofitted frame for four damage stateswhile also accounting for the uncertainty in the BRB device. Fragility curves are successively convolved with the hazards for three case-study regions providing insights on the benefits provided by the introduction of the BRBs as well as the possible variation in seismic risk estimates as a consequence of the propagation of the device uncertainty.

The results obtained for the selected case-study structure show that: a) the fragility curves at system-level are affected by the uncertainty stemming from the BRB parameters with percentage differences in median estimates between the upper and lower bounds as 18%, 20%, 21%, and 21% for the Slight, Moderate, Extensive, and Complete damage states, respectively; b) the variation in terms of story-level fragilities due to the device uncertainty is significantly higher than the one at system-level and, for the most vulnerable story within the retrofitted structure (Story 2), these percentage differences emerge as 42% and 44% respectively for the for the Extensive and Complete damage states, which is almost twice as high as the system-level differences; c) the two-level factorial experimental design method is sufficient to predict the uncertainty band around the seismic fragility curves; d) while the comparison of the risk estimates for the bare and retrofitted frames show the effectiveness of the BRBs, the percentage differences in seismic risk bounds show substantial impact of uncertainty in BRB device parameters on seismic risk estimates variation. Across all locations, the divergence between the upper and lower bound estimates of the risk are observed to increase with the *DS*, with the highest increment corresponding to the complete *DS* with values up to 56%; e) the present study illustrates a methodology evaluating the sensitivity of BRB devices and the influence on the seismic performance of a RC frame. The obtained results underline the impact of the BRBs' uncertainty and the need for appropriate safety coefficients for their design of dissipative devices as functions of the tolerance limits adopted in the quality-control tests. The results presented in this paper are limited to the investigated case study structure under the specified retrofit option. Future studies should investigate the influence of BRB device variability on different case study structures, considering different retrofit levels (*i.e.,* different proportions of the base shear between the MRF and the BRBs) and the influence of different tolerance limits.

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