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HAZARD-CONSISTENT RESIDUAL DRIFT DEMANDS IN STEEL MOMENT RESISTING FRAMES

F. Zahra[(1)], C. Málaga-Chuquitaype[(2)], J. Macedo[(3)], C. Liu[(4)]

[(1)] *PhD student, Imperial College London, London SW7 2AZ, United Kingdom, f.zahra19@imperial.ac.uk*
[(2)] *Senior Lecturer (Assoc Prof), Imperial College London, London SW7 2AZ, United Kingdom, c.malaga@imperial.ac.uk*
[(3)] *Assistant Professor, Georgia Institute of Technology, Atlanta, GA, jorge.macedo@gatech.edu*
[(4)] *PhD student, Georgia Institute of Technology, Atlanta, GA, cliu662@gatech.edu*

Abstract

Performance-Based Earthquake Engineering (PBEE) requires a seismic-resistant structure to satisfy several performance level objectives where each of them denotes the acceptable risk of a certain damage level and the resulting losses at a certain seismic hazard level. In this context, the residual drift is widely accepted as an indicator of the repairability of a structure and offers a way to evaluate how a structure is likely to perform after ground-motions with different recurrence levels. Notwithstanding all the advantages of PBEE, questions remain regarding the accuracy and reliability of the available procedures to predict structural response at different hazard levels. This process may be cluttered by inconsistencies between the models adopted and the required hazard quantification method. Hazard-consistent ground motions establish a consistent connection between the ground motion hazard and the hazard associated with an engineering demand parameter (EDP). This consistency is achieved by incorporating the spectral variability associated with design ground motions. To this end, the Conditional Scenario Spectra (CSS) is explored in this paper to ensure hazard consistency by developing a set of ground motions with an assigned rate of occurrence that can recreate the results from a probabilistic seismic hazard assessment (PSHA). In this context, this study evaluates the inter-storey peak and residual drift demands of steel moment-resisting frame (MRF) within a hazard-consistent PBEE framework.

Nonlinear response history analyses on 24 steel MRFs with different structural characteristics are performed with a selection of 596 ground motions to cover a wide range of intensities. The rate of occurrence of each ground motion is estimated with the CSS methodology. The structural performance is evaluated in terms of EDP hazard curves to provide a direct link between structural response and its corresponding return period as the base of performance-based structural assessment. The EDP hazard curves are then analysed based on several structural characteristics.

The results show that for immediate occupancy performance level, the inter-storey drifts of all frames are bounded at around 0.04%. Nonetheless, the drifts for life safety and collapse prevention levels vary from 1.00% to 1.48% and from 1.87% to 3.45%, respectively. This pattern is also evident on the residual drifts, where values range from 0.15% to 0.38% for life safety and 0.55% to 1.00% for collapse prevention performance level. The increased variation of the drift at higher performance levels shows that the expected EDP at lower hazard levels is harder to predict and requires more careful consideration. This variation also suggests the need to consider other vectors of IMs, in addition to spectral accelerations, as the IM parameters within the PBEE framework.

Keywords: earthquake engineering, hazard curve, hazard-consistency, conditional scenario spectra, residual drift

1. Introduction

Within a Performance-Based Earthquake Engineering (PBEE) framework, structural design process hinges around the attainment of specified performance objectives for a set of given seismic demand levels. The definition of discrete performance objectives enables an estimation of the expected amount of damage (and its corresponding cost) a structure may experience in the event of an earthquake. Therefore, the PBEE framework provides methods to include the uncertainties innate in the earthquake hazard and the structural response into the calculations. The most widely adopted PBEE framework, known as PEER-PBEE framework, was introduced by Cornel and Krawinkler [1] to offer a design process that includes an intention to achieve a specified performance objective in future earthquakes. The PEER-PBEE evaluates structural performance based on the earthquake it is likely to experience and provides methods to include the uncertainties innate in the earthquake hazard and the structural response quantification [2].

Typically, PBEE requires a seismic-resistant structure to satisfy several performance level objectives: (i) the capacity to withstand small and frequent earthquakes without damage (immediate occupancy performance level), (ii) the capacity to withstand moderate and infrequent earthquakes with limited damage (life safety performance level), and (iii) the capacity to withstand strong and rare earthquakes without collapse (collapse prevention performance level). Such performance objectives are tailored to the function of the structure. Conventional design of structures typically employs 475 years as the return period (10% exceedance in 50 years) for life safety performance level and 2475 years (2% exceedance in 50 years) for collapse prevention performance level [3]. Each performance level objective denotes the acceptable risk of a certain damage level and the resulted losses at a certain seismic hazard level. Losses can be related to structural damage, non-structural damage, or both, which in PBEE are expressed in terms that are easily understood, such as casualties, repair or replacement cost, downtime, and environmental impact. In this context, the structural response is characterised by means of an Engineering Demand Parameter (EDP). The EDP of interest in this study is the structural drift as it plays an important role in structural seismic assessment. The drift is usually evaluated in terms of global drift as an indicator of overall ductility or inter-storey drift as an indicator of damage distribution. Nonetheless, these drifts are transient and can only be monitored by a continuous structural health monitoring system which is not yet commonly installed in most structures. The permanent residual drift, by contrast, is actually measured after earthquake, and it could indicate whether the structure under consideration is unsafe for immediate occupancy or if its ability to respond to aftershocks has been compromised [4]. Moreover, the residual drift is commonly used as an indicator of the repairability of a building and offers a way to evaluate how a structure is likely to perform after ground-motions with different recurrence levels.

Notwithstanding all of the advantages of the PBEE, FEMA P-58 states several limitations of the current PBEE procedures, which include questions on the accuracy and reliability of the available procedures to predict structural response [5]. This question may arise from the inconsistency of the assumed ground-motion demand with the available hazard quantification method. Hazard-consistent ground motions ensure the selected ground motions response spectra to be consistent with the ground motion hazard curves at the site upon which they are based. This consistency is achieved by incorporating the spectral variability and rate of occurrence of selected design ground motions. This study explores the use of Conditional Scenario Spectra (CSS) to ensure the consistency of the hazard by developing a set of ground motions with an assigned rate of occurrence based on their spectral shape and intensity [6][7].

This study aims to evaluate the inter-storey peak and residual drift demand of steel moment-resisting frames (MRF) within the hazard-consistent PBEE framework. Nonlinear response history analyses of 24 steel MRFs with different structural characteristics are performed. 596 ground motions are employed in these analyses to cover a wide range of earthquake intensities where the rate of occurrence of each ground motion is estimated with the CSS methodology. An extensive collection of nonlinear response history analyses results are used to develop EDP hazard curves which provide a direct link between structural response and its corresponding return period as the base of performance-based structural assessment. The EDP hazard curves are then analysed based on several structural characteristics.

2. PBEE Framework

The PBEE methodology comprises of several steps:

1. Probabilistic Seismic Hazard Analysis (PSHA) to calculate $\lambda(IM)$, mean annual rate of exceedance of certain intensity measure (IM) of ground motion.
2. Probabilistic Seismic Demand Analysis (PSDA) to calculate $G(EDP|IM)$, probability of exceeding certain EDP, given different levels of *IM*.
3. Fragility analysis to calculate $G(DM|EDP)$, probability of exceeding certain damage measure (DM), given different levels of *EDP*.
4. Loss analysis to calculate $G(DV|DM)$, probability of exceeding certain decision variable (DV) limit, given different *DM*. [8]

Subsequently, the mean annual frequency of exceedance of the *DV* can be expressed as,

$$\lambda(DV) = \iiint G[DV \mid DM] \mid dG[DM \mid EDP] \mid dG[EDP \mid IM] \mid d\lambda(IM) \tag{1}$$

The PBEE carried out in this study will be limited to the first and second step of the methodology, i.e., PSHA and PSDA.

2.1 Probabilistic Seismic Hazard Analysis

The first step of PBEE is to perform a PSHA, as introduced by Cornell [9], to estimate the annual rate of exceedance of a certain intensity measure of ground motion. FEMA P-58 suggests three different target spectra derived from PHSA that can be used, namely Uniform Hazard Spectra (UHS), Conditional Mean Spectra (CMS), and Conditional Spectra (CS) [5]. UHS are commonly used design spectra that give the spectral amplitude for each spectral period at a specified return period. The UHS are constructed from many earthquake response spectra that take part in a site-specific hazard curve from PSHA. However, these earthquakes are very unlikely to occur simultaneously; hence, the UHS do not reflect the spectrum of any single realistic earthquake. Baker and Cornell [10] developed the CMS as an alternative to the UHS. The term "conditional" refers to the spectrum being anchored at a selected spectral acceleration at a reference period of interest and the term "mean" refers to the mean value of the number of standard deviations (epsilons) above the median prediction of the ground motion model at different periods given the epsilon at the reference period. One of the limitations of the CMS is that they represent mean values so that they do not capture the peak to trough variability. To address this need, the CS was proposed by Abrahamson and Al Atik [11] where the use of the mean epsilon is replaced with multiple realisations of the epsilons so that the spectral uncertainty about the mean is considered by including their variance [12].

However, FEMA P-58 does not provide any tool to estimate the rate of occurrence of the CMS or the CS. One alternative to estimate the IM's hazard rate of occurrence is by employing CSS as proposed by Arteta et.al. [6][7]. CSS is a set of realistic earthquake spectra with an assigned rate of occurrence based on their spectral shape and intensity. With CSS, the ground-motions spectra are developed so that they are spectrally consistent with the hazard curve at the selected site at all relevant periods. The first step to develop CSS involves ground motion selection from hazard deaggregation at the site. These ground motions are scaled to account for the variability of the peak and trough around a CMS at various hazard levels. The CMS ensure each spectrum to have the correct shape and give the geometric mean response spectrum for all periods of interest [13][14]. Each ground motion is then assigned an initial rate of occurrence according to the hazard level of the UHS at the CMS conditioning period. Finally, the rate of occurrence assigned at each ground motion is numerically optimised such that its calculated hazard matches the target hazard curve. The CSS method was used to assess the drift and base shear demands in a reinforced concrete frame in [6] and to perform fragility analysis of concrete shear walls in [16]. To date, the use of CSS in steel structures is only found in [15]

2.2 Probabilistic Seismic Demand Analysis

The second step of PBEE is to perform a PSDA to define the probability of exceedance of the EDP as a function of ground motion IMs. The EDP can be obtained by nonlinear response history analysis. The PSDA framework has been used to reach different structural design objectives in the past. Gupta and Krawinkler [17] carried out a performance assessment of 3 MRFs and demonstrated that the storey drift of ductile code conforming structures is mostly within the range of acceptable performance at various hazard levels, except during severe ground motions where P-delta effects can drive the structure to potential collapse. Medina and Krawinkler [18] studied the quantification of several EDPs for nondeteriorating MRFs performance evaluation by nonlinear response history analysis and constructed EDP hazard curves of these frames. Kazantzi et al. [19] concentrated their study on connection fracture and showed that brittle connection fracture causes structures to have large deformation and resulted in low structural reliability. Málaga-Chuquitaype and Bougatsas [20] compared the seismic performance of 4 buildings with one-way and two-way framing systems subjected to bi-directional ground motion using scalar and vector-valued probabilistic methods. Their hazard curves indicate that the influence of the framing systems on structural fragilities depends on the number of storeys and hazard levels. Nonetheless, these past researches on MRF performance assessments have relied on a relatively low number of structural models or case-studies and did not explicitly ensure the hazard-consistency of the analyses, which will be addressed in this study.

PSDA can be done with dynamic pushover analysis, known as Incremental Dynamic Analysis (IDA), as introduced by Vamvatsikos [21] where the ground motion sets are scaled to perform nonlinear response history analysis. The scaling factors used are increased until a certain response limit is exceeded for each ground motion. Nonetheless, the scaling factor used in the IDA may produce unrealistic spectral shapes since the frequency content is considered invariant for events on different intensities where they should be different depending on the hazard level. These unrealistic spectral shapes may result in unrealistic structural responses that are not hazard-consistent [6]. Alternatively, the EDPs can be gathered from a series of response history analyses performed on sets of records with a given rate of occurrence based on CSS. These results can be used to form EDP hazard curves as a part of the PBEE framework. The EDP hazard curve provides a direct link between structural response and its corresponding return period which can provide the base of performance-based structural assessment. The hazard curve can also be used to evaluate whether a risk associated with a demand level is acceptable based on a code or design philosophy.

3. Structural Model

In this study, the MRFs were modelled and analysed using the open-source software OpenSees [22]. Twenty-four plane steel MRFs designed based on Eurocode 3 [23] and Eurocode 8 [24] are considered. Although relatively good predictions of structural response have been obtained with simplified non-degrading models for demand levels well away from structural instability [25], it is now well established that a reliable seismic assessment of structures over a wide range of earthquake intensities requires the consideration to capture the deterioration in strength and stiffness of the structural component [26][27]. The deterioration properties of the MRFs connections in this study are represented by the concentrated plasticity concept with rotational springs. At the plastic regions, the rotational behaviour follows a bilinear hysteretic response built upon the modified Ibarra Krawinkler deterioration model [28]-[31].

The frames have different number of storeys, number of bays, and frame sections as shown in Table 1. On the other hand, the frames have equal storey heights of 3 m and bay width of 5m. The expression under the heading "Section" in Table 1 follow the nomenclature: e.g., 280-360(1-4)+260-330(5-6), meaning that floors 1-4 have HEB280 columns and IPE360 beams and floors 5-6 have HEB260 columns and IPE330 beams.

Table 1 – Steel Moment Resisting Frames Considered

Frame ID	Number of Storeys	Number of Bays	Sections	Stiffness Ratio (ρ)	Plasticity Resistance Ratio (a)
1	3	3	240-330(1-3)	0.47	1.30
2	3	3	260-330(1-3)	0.36	1.60
3	3	3	280-330(1-3)	0.28	1.90
4	3	6	240-330(1-3)	0.54	1.30
5	3	6	260-330(1-3)	0.41	1.60
6	3	6	280-330(1-3)	0.31	1.90
7	6	3	280-360(1-4)+260-330(5-6)	0.38	1.60
8	6	3	300-360(1-4)+280-330(5-6)	0.29	1.97
9	6	3	320-360(1-4)+300-330(5-6)	0.24	2.27
10	6	6	280-360(1-4)+260-330(5-6)	0.43	1.60
11	6	6	300-360(1-4)+280-330(5-6)	0.33	1.97
12	6	6	320-360(1-4)+300-330(5-6)	0.27	2.27
13	9	3	340-360(1)+340-400(2-5)+320-360(6-7)+300-330(8-9)	0.28	2.19
14	9	3	360-360(1)+360-400(2-5)+340-360(6-7)+320-330(8-9)	0.24	2.43
15	9	3	400-360(1)+400-400(2-5)+360-360(6-7)+340-330(8-9)	0.18	2.93
16	9	6	340-360(1)+340-400(2-5)+320-360(6-7)+300-330(8-9)	0.32	2.19
17	9	6	360-360(1)+360-400(2-5)+340-360(6-7)+320-330(8-9)	0.28	2.43
18	9	6	400-360(1)+400-400(2-5)+360-360(6-7)+340-330(8-9)	0.21	2.93
19	12	3	400-360(1)+400-400(2-3)+400-450(4-5)+360-400(6-7)+340-400(8-9)+340-360(10)+340-330(11-12)	0.24	2.60
20	12	3	450-360(1)+450-400(2-3)+450-450(4-5)+400-450(6-7)+360-400(8-9)+360-360(10)+360-330(11-12)	0.26	3.00
21	12	3	500-360(1)+500-400(2-3)+500-450(4-5)+450-450(6-7)+400-400(8-9)+400-360(10-11)+400-330(12)	0.19	3.63
22	12	6	400-360(1)+400-400(2-3)+400-450(4-5)+360-400(6-7)+340-400(8-9)+340-360(10)+340-330(11-12)	0.28	2.60
23	12	6	450-360(1)+450-400(2-3)+450-450(4-5)+400-450(6-7)+360-400(8-9)+360-360(10)+360-330(11-12)	0.3	3.00
24	12	6	500-360(1)+500-400(2-3)+500-450(4-5)+450-450(6-7)+400-400(8-9)+400-360(10-11)+400-330(12)	0.22	3.63

Information of stiffness ratio, ρ, and plasticity resistance ratio, α, is also included in Table 1. The stiffness ratio is calculated by:

$$\rho = \frac{\sum (I/l)_b}{\sum (I/l)_c} \quad (2)$$

where I and l are the second moment of inertia and the length of the beam (b) and column (c), respectively. The stiffness ratio controls the frame behaviour in the elastic range [31]. The increase of ρ moves the behaviour of the structure from flexural-dominated to shear-dominated response. The influence of ρ diminishes as the structure moves into the inelastic range because the structure starts approaching a mechanism mode of failure. This mechanism is completed when plastic hinges are developed at the end of the beams and the base of the columns [32]. The plasticity resistance ratio, α, is used to describe the characteristic of the structure in the inelastic range, where it is defined as:

$$\alpha = \frac{M_{RC,1,av}}{M_{RB,av}} \quad (3)$$

with $M_{RC,1,av}$ is the average of the first storey columns plastic moments of resistance and $M_{RB,av}$ is the average of the beams plastic moments of resistance. At a certain level of ductility and cross-section of beams, a higher value of α will delay the mechanism action.

4. Conditional Scenario Spectra

Five hundred ninety-six ground motions were selected from the PEER NGA-West 2 database [33] and Fig. 1 (a) presents their horizontal-component ground motion spectra. The rate of occurrence of each spectrum was calculated using the CSS methodology. One of the main features of the CSS is that the hazard at a site can be recovered over a whole range of periods by using the suite of records used and their corresponding rates. The target and reconstructed CSS spectra for different hazard levels are compared in Fig. 1 (b), where they generally show a good match.

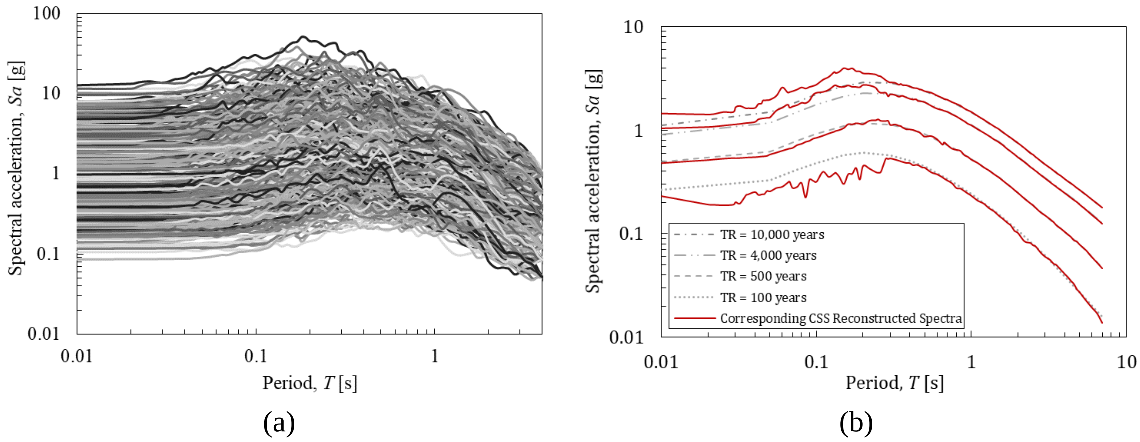


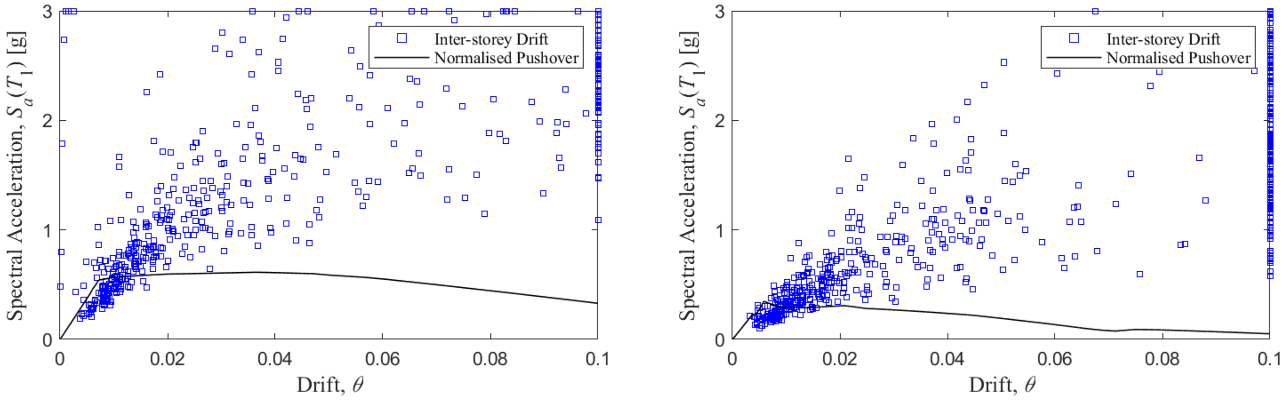
Fig. 1 – (a) 596 scenario spectra (5% damped) and (b) comparison of target and CSS reconstructed spectra.

5. Probabilistic Seismic Demand Analysis of the Moment-Resisting Frames

5.1 Nonlinear Response History Analysis with Conditional Scenario Spectra

A total of 14,304 analyses (i.e., 596 accelerograms × 24 structures) were performed using Opensees [22] on the High-Performance Computing Facility of Imperial College London. Fig. 2 shows scatter plots of the resulted EDP of each ground motion, which is paired with its corresponding spectral acceleration at the fundamental period of the structure, $S_a(T_1)$. The pushover curve of each frame is also included to give an overview of the inelastic behaviour characteristics of the structure. The collapse term in this study is assigned to those runs where the models reach large EDP values that result in unstable frames. These large EDP values are capped at 10% for the sake of presentation. This cap is considered appropriate since FEMA 356 limits the collapse prevention performance level at 5% transient drift [34]. It can be seen from Figure 2 that Frame 24, the taller structure, has more collapse cases than Frame 01.

Generally, spectral acceleration ordinates decrease as the period of structure increases. As we move from Frame 01 to Frame 24, with decreasing spectral acceleration value, they have approximately the same levels of drift, implying the taller structures exhibit greater displacement as expected. It can be seen that around the elastic region, the EDPs generally increase as the spectral acceleration increase. Nonetheless, in the inelastic region, this pattern is changed as the EDPs disperse more as the spectral acceleration increases. This increasing dispersion might be caused by the inelastic response of the MRFs that are influenced by certain ground motion characteristics that are not well captured by the elastic spectral shape used in this study [27]. Therefore, it is suggested to consider other vectors of IMs, in addition to the spectral accelerations, as the IM parameters in the PBEE framework to better predict the structural response [35]-[40].



(a) (b)

Fig. 2 – Spectral acceleration, $S_a(T_1)$, versus EDPs from response history analyses of (a) Frame01 (3S-3B) and (b) Frame24 (12S-6B).

5.2 Hazard Consistent EDP Curve

The results from nonlinear response history analyses are used to develop EDP hazard curves. Fig. 3 (a) shows a scatter plot of the ground-motion rate of occurrence against the residual drift for Frame 12 as an example. It should be noted that the capped large EDP values would not affect the risk estimation since the risk is expressed as a probability of exceedance; moreover, beyond the point of instability, the rates added to the risk are very small so that they would not significantly affect the hazard curve trend. The annual rate of exceedance of the EDP, $\nu(EDP > d)$, as the inverse of return period, *TR*, can be calculated by the following equation,

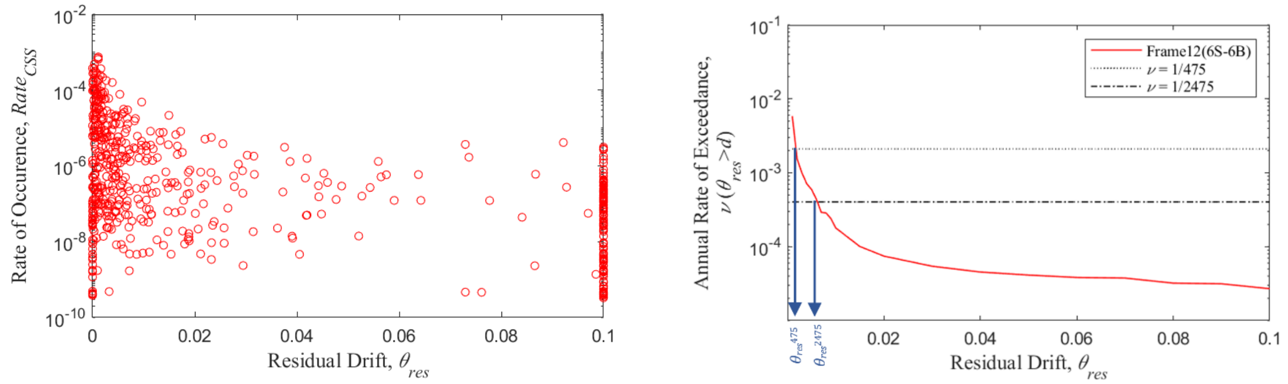
$$\nu\left[EDP > d\right] = \sum_{i=1}^{\#records} Rate_{CSS,i} H\left[EDP - d\right] \tag{4}$$

where $Rate_{CSS,i}$ is the assigned rate of occurrence of the ground motions calculated from the CSS and $H[EDP - d]$ is the Heaviside function.

Fig. 3 (b) shows the residual drift hazard curve for Frame 12. This figure includes horizontal lines of the 1/475 and 1/2475 annual rate of exceedance as the indicator of life safety and collapse prevention performance level of conventional structure, respectively. The hazard curve can be used to assess whether a risk associated with a demand level is acceptable based on a code or design philosophy. For example, the corresponding maximum residual drift of Frame 12 for 475 and 2475 years return period are 0.19% and 0.62%, respectively. These residual drifts are well below the performance level objective of conventional structure based on FEMA 356 [34], which are 1% for life safety performance level and 5% for collapse prevention performance level.

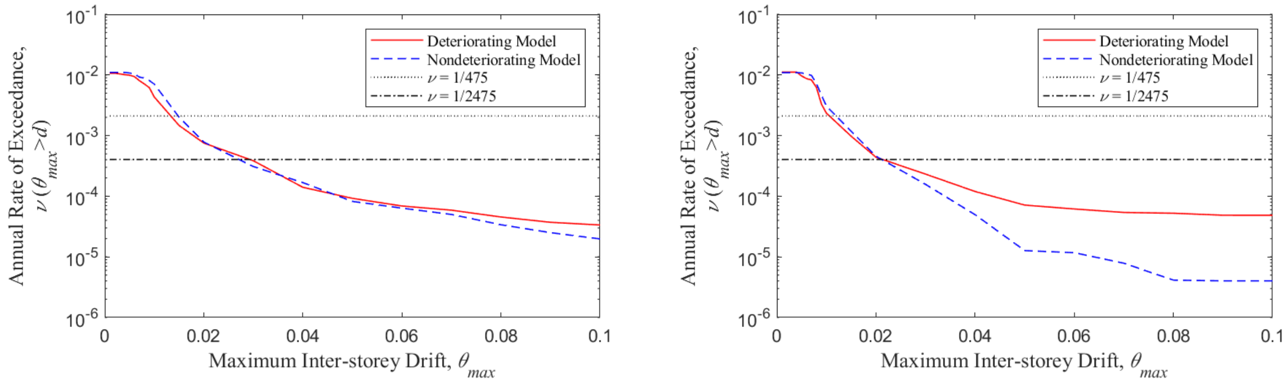
The hazard curves of the frames with and without connection deterioration model are compared in Fig. 4. The MRFs with deteriorating connections are generally found to have higher drift demands, and this pattern becomes more prominent in taller structures, as can be seen in Fig. 4 (b). These results show that the exclusion of the deterioration model leads to underestimation of structural demands and stresses the need to include this model to have a reliable structural seismic analysis, particularly when important P-delta effects are expected like in taller structures.

The inter-storey drift and residual drift hazard curves of several frames are then compared based on the plasticity resistance ratio, the number of bays, and the number of storeys in Fig. 5 to Fig. 7. Frames with a higher plastic resistance ratio are found to have lower demand for the same hazard level due to the delay in plastic hinges formation. The hazard curves also show a dependence with the number of bays where the frames with more bays show higher demand due to the presence of the connection deterioration models on the MRFs model. This result opposes the commonly found result in current studies where larger number of bays is considered beneficial in the seismic response of structures [41][42]. On the other hand, taller structures were found to have higher inter-storey drift demand. This influence of the number of storeys becomes almost negligible in the residual drift demand of the MRFs.



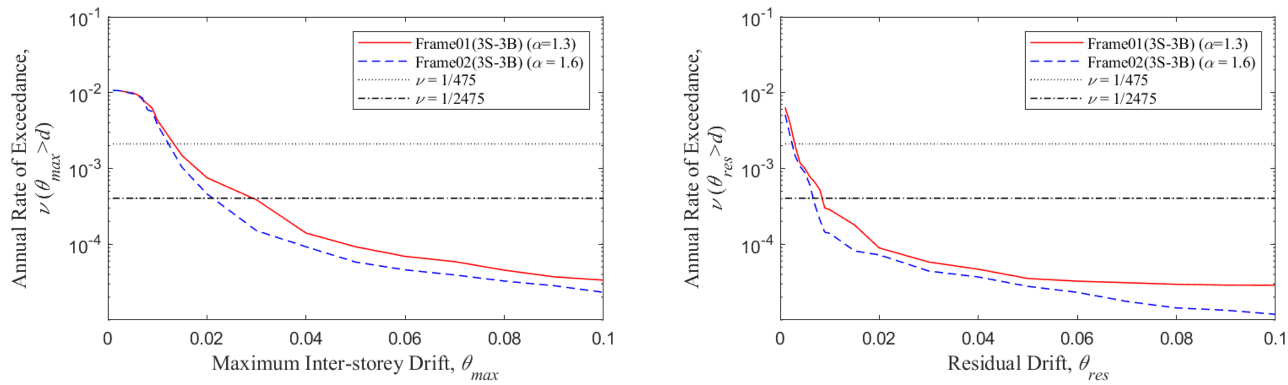
(a) (b)

Fig. 3 – (a) Scatter plot of residual drift rate of occurrence based on CSS of Frame 12 (6S – 6B) and (b) its hazard curve.



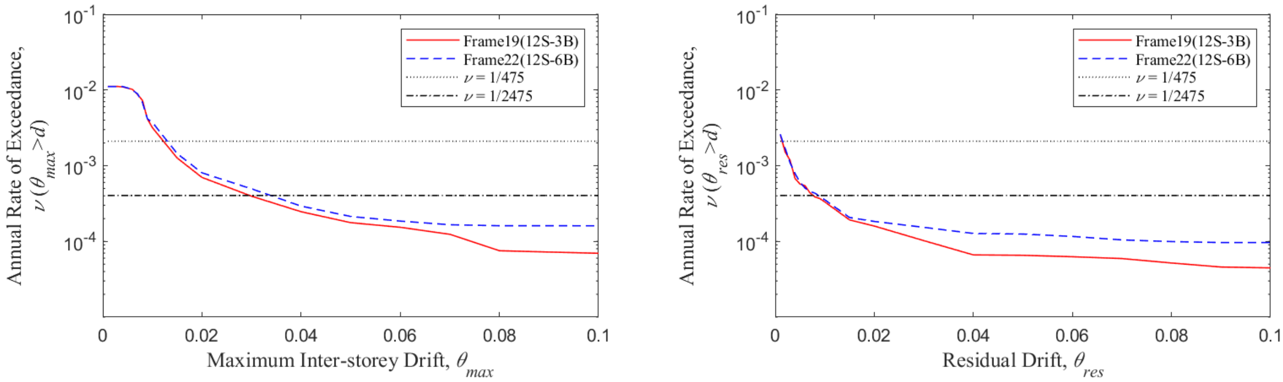
(a) (b)

Fig. 4 – Influence of connection deterioration model on inter-storey drift hazard of (a) Frame01 (3S-3B) and (b) Frame24 (12S-6B).



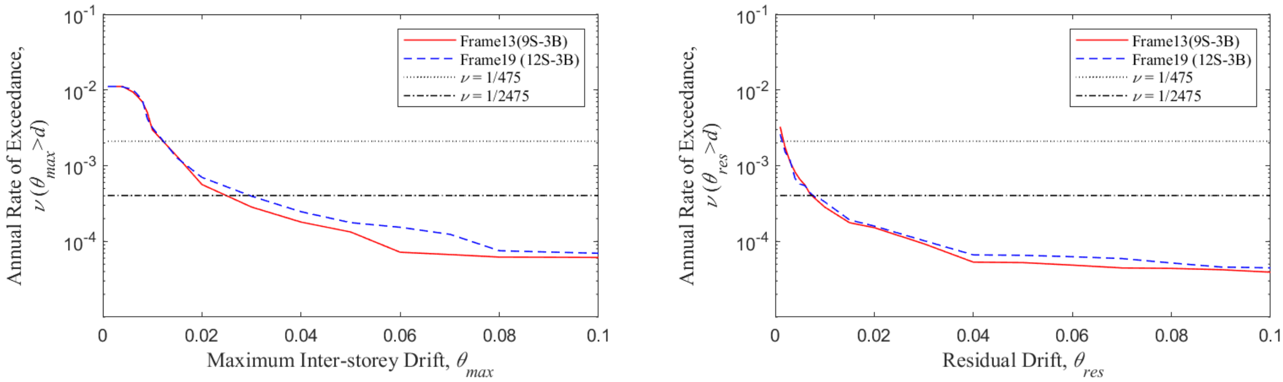
(a) (b)

Fig. 5 – Influence of plasticity resistance ratio, α, on (a) inter-storey drift hazard curve and (b) residual drift hazard curve.



(a) (b)

Fig. 6 – Influence of number of bays on (a) inter-storey drift hazard curve and (b) residual drift hazard curve.



(a) (b)

Fig. 7 – Influence of number of storeys on (a) inter-storey drift hazard curve and (b) residual drift hazard curve.

The associated EDPs of the frames for life safety (*TR* = 475) and collapse prevention (*TR* = 2475) limit states are summarised in Table 2. It is shown that all frames are within the required performance level objective of a conventional structure as stated in FEMA 356 [34], which are 2.5% transient drift and 1% permanent drift for life safety performance level and 5% transient or permanent drift for collapse prevention performance level.

Fig. 5 to Fig. 7 show that for immediate occupancy performance level (ν = 1/95), the inter-storey drifts of all frames are bounded at around 0.04%. Nonetheless, as can be seen in Table 2, the drifts for life safety and collapse prevention levels vary from 1.00% to 1.48% and from 1.87% to 3.45%, respectively. This pattern is also evident for the residual drifts where the drifts range from 0.15% to 0.38% for life safety performance level and from 0.55% to 1.00% for collapse prevention performance level. The increasing variation of the drifts at higher performance level shows that the expected EDP at this level is harder to predict and requires careful consideration. This variation stresses again the need to consider other IMs, in addition to spectral accelerations, to have a better prediction of the structural response

6. Conclusions

This study has evaluated the inter-storey and residual drift demands of steel MRF within a hazard-consistent PBEE framework. Nonlinear response history analyses of 24 steel MRFs with different structural characteristics were performed with 596 ground motions. The CSS framework was used to estimate the rate of occurrence of selected ground motions while ensuring their consistency with a PSHA-based hazard quantification. The nonlinear analyses provided the EDPs, which were used to develop EDP hazard curves that provided the annual rate of exceedance for different EDP thresholds.

The EDP hazard curves developed show that the presence of the deterioration model at the structural connections increases the demand of the structure, which appears to be more notable in taller structures. The plasticity resistance ratio, number of bays, and number of storeys are also found to influence the structural demand of the MRF at different stages of their response. All hazard curves show an increase on the variability of structural demand at lower hazard levels. The increased variability indicates that EDP at lower hazard levels, i.e., at higher performance level, is harder to estimate and requires careful consideration. It also suggested that other IMs, in addition to spectral accelerations, should be considered to have a better prediction of the structural response of moment resisting frames.

Table 2 – Associated EDPs for Life-Safety Performance Level (475 Years Return Period) and Collapse Prevention Performance Level (2475 Years Return Period) of Conventional Structures

Frame ID	Number of Storeys	Number of Bays	Stiffness Ratio (ρ)	Plasticity Resistance Ratio (α)	Structural Period (T_1)	Max. Inter-Storey Drift (θ_{max})		Residual Drift (θ_{res})		Max. roof Displacement (Δ)	
						θ_{max}^{475}	θ_{max}^{2475}	θ_{res}^{475}	θ_{res}^{2475}	Δ^{475}	Δ^{2475}
1	3	3	0.47	1.30	0.66	1.39%	2.95%	0.29%	0.85%	0.09	0.19
2	3	3	0.36	1.60	0.62	1.30%	2.18%	0.22%	0.67%	0.08	0.17
3	3	3	0.28	1.90	0.59	1.28%	1.87%	0.25%	0.55%	0.08	0.13
4	3	6	0.54	1.30	0.68	1.48%	3.40%	0.38%	1.00%	0.09	0.20
5	3	6	0.41	1.60	0.64	1.38%	2.68%	0.27%	0.73%	0.09	0.18
6	3	6	0.31	1.90	0.60	1.34%	2.20%	0.34%	0.60%	0.08	0.15
7	6	3	0.38	1.60	1.10	1.46%	2.71%	0.21%	0.73%	0.17	0.29
8	6	3	0.29	1.97	1.05	1.43%	2.67%	0.17%	0.57%	0.16	0.28
9	6	3	0.24	2.27	1.01	1.41%	2.53%	0.15%	0.56%	0.15	0.26
10	6	6	0.43	1.60	1.13	1.48%	2.79%	0.32%	0.88%	0.18	0.31
11	6	6	0.33	1.97	1.08	1.45%	2.59%	0.22%	0.67%	0.17	0.29
12	6	6	0.27	2.27	1.04	1.35%	2.47%	0.19%	0.62%	0.15	0.28
13	9	3	0.28	2.19	1.38	1.26%	2.57%	0.18%	0.72%	0.20	0.40
14	9	3	0.24	2.43	1.35	1.31%	2.47%	0.21%	0.67%	0.20	0.38
15	9	3	0.18	2.93	1.29	1.32%	2.12%	0.18%	0.59%	0.20	0.35
16	9	6	0.32	2.19	1.38	1.33%	2.70%	0.18%	0.81%	0.20	0.40
17	9	6	0.28	2.43	1.35	1.24%	2.55%	0.18%	0.67%	0.20	0.37
18	9	6	0.21	2.93	1.30	1.27%	2.30%	0.18%	0.63%	0.20	0.36
19	12	3	0.24	2.60	1.67	1.28%	2.97%	0.15%	0.77%	0.25	0.51
20	12	3	0.26	3.00	1.56	1.25%	2.54%	0.15%	0.65%	0.24	0.45
21	12	3	0.19	3.63	1.49	1.00%	1.92%	0.16%	0.58%	0.23	0.41
22	12	6	0.28	2.60	1.65	1.36%	3.45%	0.15%	0.87%	0.25	0.56
23	12	6	0.30	3.00	1.55	1.29%	2.69%	0.17%	0.67%	0.24	0.44
24	12	6	0.22	3.63	1.48	1.02%	2.17%	0.16%	0.59%	0.22	0.41

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