



Regression models for predicting the inelastic seismic response of steel braced frames

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ABSTRACT

This paper describes the development of regression equations to predict the peak inelastic drift and acceleration response of steel concentrically braced frame (CBF) building structures. A database of Engineering Demand Parameters (EDPs) is developed by performing a large number of nonlinear time-history analysis (NLTHAs) in OpenSEES for a set of 24 case study CBFs. A consistent and realistic set of mechanical models is achieved by designing each case study frame using Eurocode 8. Multi-variable regression models for peak drift and acceleration response at each storey level are fitted to the NLTHA results data. A nonlinear formulation and robust regression are employed to capture the influences of the Eurocode 8 behaviour factor and brace slenderness and overstrength limits on the EDP profiles over the height of a building. Comparison with existing models shows that the fitted regression equations achieve improved agreement with NLTHA results across a wide range of structural parameters. The regression models can be applied in performance-based design or assessment to predict the peak inelastic response of CBF structures without performing additional nonlinear analyses.

1. Introduction

Concentrically braced frames, or CBFs, are steel frame structures that resist lateral seismic loads primarily through axial forces in diagonal bracing members. The economical design of CBFs in buildings generally employs dissipative behaviour in which significant inelastic deformations are accommodated during extreme earthquake events. Individual braces experience successive tension yielding and compression buckling and display a complex hysteretic behaviour that depends upon member and cross-section slenderness, and brace connection details [1]. Recommendations for accurate and efficient modelling of this behaviour through nonlinear time-history analysis (NLTHA) have been developed [2,3] but the computed seismic response can be sensitive to small variations in model parameters.

To support performance-based design methodologies in earthquake engineering, reliable methods for predicting global and inter-storey drift and absolute floor acceleration, particularly in the post-elastic range, have received increased attention. The prediction of drift demands is important due to their correlation with both structural and non-structural damage, while accurate values of floor acceleration are needed for the design of acceleration-sensitive non-structural

components. For performance assessment and performance-based design employing, for example, the FEMA P-58 methodology [4,5] it is necessary to calculate Engineering Demand Parameters (EDPs), primarily peak inter-storey drift and absolute floor acceleration, at each storey level within a building. Although this can be achieved through sophisticated NLTHA, the application of this method to practical assessment is hampered by the considerable expertise, time, and computational cost required [6]. For example a survey of Portuguese practitioners showed almost half of were unaware of nonlinear analysis methods and only 20% had applied them in practice [7]. The excessive time and computational cost associated with NLTHA is particularly prohibitive for the application of iterative performance-based design procedures [5]. As good structural design should not be dependent on powerful analysis tools, there is a need for reliable methods of estimating nonlinear response demands that do not require NLTHA.

Conventional methods for the estimation of inelastic drift response are based on the equal displacements and equal energies rules [8]. In Eurocode 8 [9], design drift values are determined from elastic lateral force analysis results increased by a displacement amplification factor, q_d . By specifying the same values for this factor and the behaviour (force reduction) factor, q , for all steel structures, the equal displacements rule

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is effectively employed to predict inter-storey drift under all levels of excitation. A number of studies have proposed different methods for predicting global or peak inter-storey drift in steel frame structures (for example [6,10,11]), including studies which developed regression models from the results of large numbers of time history analyses [12–14]. In short, these studies have generally shown that for long period structures the equal displacements rule is generally conservative, but for shorter period structures the rule often under predicts drift. This latter observation is particularly relevant for CBFs, which tend to possess relatively short natural periods due to their inherent lateral stiffness. However, these studies tend to focus on predicting the global peak inter-storey drift response over all floor levels, and do not attempt to predict drift demand in each individual storey, as is required by performance assessment procedures. Methods for predicting storey drifts in each individual storey of a building frame, such as that proposed in this paper, are less common.

Multi-storey braced frames are known to be susceptible to so-called soft-storey effects, where post-elastic drift is concentrated at a single level. Eurocode 8 attempts to limit this effect in CBFs by limiting the difference in brace overstrength over the frame height. The ratio of column-to-brace stiffness can dictate the ability of a braced frame to redistribute plasticity after brace yielding, and has been shown to be an important parameter for the prediction of drift demands in CBFs [12,15]. A number of studies examining the inelastic drift response of different frame types have concluded that drift response is influenced by some measure of ground motion frequency content, such as the mean earthquake period, T_m , [14,16,17].

Values of absolute horizontal floor acceleration over the height of buildings are proposed in both Eurocode 8 and ASCE 7–16 [18] for the design of non-structural components. Eurocode 8 implicitly assumes that the floor acceleration magnification factor, that is the peak floor acceleration (PFA) normalized by the peak ground acceleration (PGA), ranges linearly from a value of 1 at the base, i.e. the PFA is equal to the PGA, to 2.5 at the top of the structure. Similarly, the US provisions base the design of non-structural components on a linear variation over the height of a building, except with a floor acceleration magnification value of 3 at roof level. However, various studies (for example [19,20]) have shown that these values tend to be conservative in many cases.

For linear elastic response conditions, the peak acceleration response, which is known to be sensitive to higher mode effects (for example [21]), can be predicted using modal analysis techniques. For inelastic response, a number of different studies that have used NLTHA to examine floor accelerations in different types of structures have shown that as the degree of inelasticity is increased, floor acceleration magnification is generally, although not always, reduced. Examples of such studies include those by Medina et al. [22] for MRFs in general, Ray-Chaudhuri and Hutchinson [21], Wieser et al. [19] and Flores et al. [20] for steel MRFs and Surana et al. [23] for concrete frames. Wieser et al. [19] attribute this reduction in acceleration magnification with increased inelasticity to the elongation of the period of vibration that occurs with yielding. Taghavi and Miranda [24] concluded that there is a limit beyond which the peak floor acceleration cannot increase regardless of the ground motion intensity, a phenomenon which they term saturation of floor acceleration. As well as the degree of inelasticity, building height has also repeatedly been shown to influence the distribution of floor accelerations over the various stories for both elastic and inelastic response. Taller buildings tend to experience peak floor acceleration values that are reasonably similar to the peak ground acceleration, except at the uppermost storeys where a sharp increase is often observed [20,22,25], a feature termed the ‘whiplash effect’ by Singh et al. [26]. In contrast, low rise buildings tend to see a more gradual increase in floor acceleration magnification over the frame height.

In terms of predicting inelastic acceleration demands, efforts have been made to extend the capabilities of elastic modal analysis techniques by using a force reduction factor to reduce the contributions of the first

[27] (termed the ‘first mode reduced’ method) or first and second modes [21] to account for inelastic response. Both Vukobratović and Fajfar [28]{Vukobratović, 2017, Code-oriented floor acceleration spectra for building structures} and Welch and Sullivan [29] adopt a similar approach for the calculation of floor response spectra, with each using slightly different methods to calculate the force reduction factors.

The FEMA P-58 [4] project, which details procedures for building-specific loss estimation, includes expressions to predict both peak inter-storey drift and floor acceleration magnification at each storey level over the frame height. These can be used in a so-called simplified analysis procedure, which allows performance measures to be estimated whilst avoiding NLTHA. Separate equations to predict *EDPs* are proposed for ‘Moment resisting’, ‘Braced’ and ‘Shear Wall’ structures; these are functions of the fundamental structural period, T_1 , the degree of nonlinearity denoted S , and the ratio of the height of an individual storey level, h_i , to the overall building height H . The proposed expressions for braced frames were developed using the results of NLTHA carried out for 3, 5, 9 and 12 storey archetype frames subjected to 25 near-field and 25 far-field ground motions [30]. However, the use of the term ‘Braced’ in FEMA P-58 is potentially misleading as the archetype structures analysed in the development of the model were eccentrically braced frames designed to NEHRP design provisions [31], as opposed to the concentrically braced frames considered here.

The aim of this paper is to examine the inelastic drift and acceleration response of a series of case study CBFs designed to Eurocode 8, and to develop regression models to predict peak inter-storey drift and peak floor acceleration at each storey level over the height of a building. The models developed consider the impact of ground motion frequency content on these two *EDPs*. While ground motion frequency has been shown to influence drift response in similar studies [12,14,17], its impact on floor acceleration is less clear [21,32]. The predictive models for drift and acceleration developed in this paper are compared to alternatives proposed elsewhere and with values predicted by design codes. The work focuses specifically on CBFs designed to Eurocode 8. Storey drift and floor acceleration response in a CBF are sensitive to the properties of its brace members and their connections, which are determined by the particular rules of the design code employed. The unique design rules imposed on CBFs by Eurocode 8, primarily the exclusion of brace compression resistance from the lateral force resistance model, and limitations on brace slenderness and overstrength, are highly relevant in this regard, and necessitate a case study-based approach. The regression models developed to predict *EDPs* for such structures capture the influence of these design features and enable the practical implementation of performance assessment and performance-based design methodologies.

2. Structural design & modelling

A set of 2, 3, 4, 5, 6 and 7 storey CBF structures, as illustrated in Fig. 1, were designed in accordance with Eurocode 3 (CEN, 2005) and Eurocode 8 and analysed using nonlinear time history analysis. The frames were initially designed for gravity loads according to Eurocode 3, with unfactored permanent and variable loads of 3 kN/m² and 3.3 kN/m² applied at each floor level and 2.5 kN/m² and 0.6 kN/m² at the roof level. Earthquake resistant design was then carried out in accordance with Eurocode 8. Seismic design forces were calculated using the Eurocode 8 Type 1 spectrum with a behaviour factor of $q = 4$. This was done using four different Eurocode 8 design spectral acceleration values, S_d , for each frame height as summarised in Table 1, meaning a total of 24 different concentrically braced frames were designed and analysed.

Bay widths and storey heights were kept constant at 6 m and 3.6 m respectively, except where central columns were used causing braced bay widths to be halved, as shown in Fig. 1. All buildings were square in plan with seismic resistance provided by perimeter braced frames. UK steel profiles were used for the beams and columns, while the bracing members selected were either rectangular or square hollow sections, all

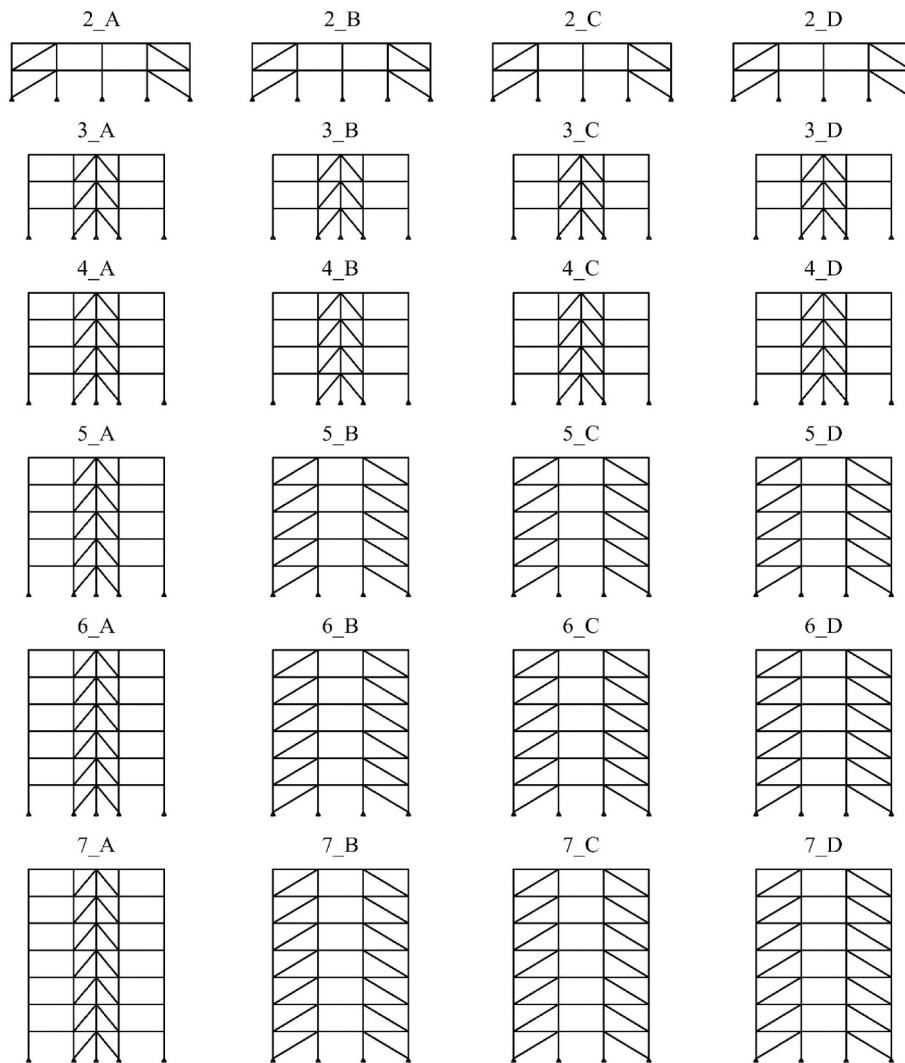


Fig. 1. Elevations of frames analysed: bay widths = 6 m, storey heights = 3.6 m. Refer to Table 1 and Table A1 for structural details of archetype CBFs.

with a design strength of 275 N/mm².

The seismic design actions to be resisted by the structural members were calculated through elastic analyses using the Eurocode 8 design spectrum method for regular structures. The lateral resistance of the structure was assumed to be provided by the tension diagonals only, as per the design assumptions for CBFs specified by Eurocode 8. Based on the results of this analysis, columns and beams were designed to resist the combination of gravity and seismic actions stipulated by Eurocode 8, with member sizes kept constant at each level. The bracing members were designed to resist axial tension forces only. The Eurocode 8 non-dimensional slenderness limitations and overstrength differential criteria were obeyed in the brace design procedure. The slenderness limitation rules stipulate that:

- $\lambda^- \leq 2$ for non-X braced frames and $1.3 < \lambda^- \leq 2$ for X braced frames (for buildings above two stories).

The overstrength rule states that:

- The maximum brace overstrength over the entire height of the frame, Ω_{max} , cannot differ from the minimum value, Ω_{min} , by more than 25%.

The combination of these two rules, rather than the calculated value of the design axial force, was the controlling factor in the selection of

brace sizes in some cases. This feature of the design process can lead to substantial levels of global overstrength in CBFs designed to Eurocode 8 [1]. Ductility Class High (DCH) requirements were followed, hence all brace members possess Class 1 cross sections, as defined by Eurocode 3 [33], to ensure sufficient ductility capacity for dissipative behaviour. Preliminary details of the designed archetype CBFs and their resulting structural characteristics are given in Table 1. Full details of the section sizes employed for the beams, columns and braces are given in Appendix A.

Planar models of the seismic resisting frames were developed in OpenSEES [34]. Following the recommendations of [2], the brace members were modelled using two force-based nonlinearBeamColumn elements with Gauss-Lobatto integration [35] at three integration points per element. An initial camber displacement of 0.1% of the brace length was employed at the midpoint in order to simulate buckling behaviour. The brace elements were discretised as illustrated in Fig. 2a. All bracing elements were. The beams and columns were also modelled using force-based nonlinearBeamColumn elements with 3 integration points and discretised fiber sections. Nonlinear rotational spring elements, with initial rotational stiffness and yield moment calculated according to the expressions proposed by Hsiao et al. [3], were employed at the brace end point to simulate the end restraint imposed on the braces by the gusset plates. Rigid elements were used to model the remainder of the gusset plates and the sections of the beams and columns in the region of the connection, as illustrated in Fig. 2b. Using the results of shaking table

Table 1

Structural characteristics of archetype CBFs.

Frame ID	Storeys	Design spectral acceleration $S_e(T_1)$ (g)	T_1 (s)	Height (m)
2_A	2	0.11	0.33	7.2
2_B	2	0.15	0.31	7.2
2_C	2	0.19	0.27	7.2
2_D	2	0.22	0.26	7.2
3_A	3	0.11	0.43	10.8
3_B	3	0.15	0.38	10.8
3_C	3	0.18	0.35	10.8
3_D	3	0.22	0.32	10.8
4_A	4	0.11	0.51	14.4
4_B	4	0.14	0.46	14.4
4_C	4	0.18	0.40	14.4
4_D	4	0.23	0.38	14.4
5_A	5	0.11	0.57	18
5_B	5	0.15	0.49	18
5_C	5	0.18	0.45	18
5_D	5	0.22	0.38	18
6_A	6	0.11	0.60	21.6
6_B	6	0.14	0.57	21.6
6_C	6	0.19	0.47	21.6
6_D	6	0.22	0.44	21.6
7_A	7	0.11	0.66	25.2
7_B	7	0.13	0.64	25.2
7_C	7	0.17	0.51	25.2
7_D	7	0.22	0.50	25.2

tests conducted as part of the BRACED program [36,37], the key aspects of this modelling approach have been validated for braced frames designed to Eurocode 8 [38]. The Steel02 material, which represents the Giuffre-Menegotto-Pinto model, was used for all nonlinear components. As illustrated in Fig. 3, a fictitious leaning column element [39] was included in order to account for second order effects due to the gravity loads in the parts of the building not included in the OpenSEES CBF models. Rigid elements are used to connect the seismic frame and leaning column. These links are connected to the columns using very low stiffness spring elements to ensure that the columns are not subjected to additional bending moments.

2.1. Ground motions for NLTHA

To perform time history analysis, 40 ground motion records were selected from the PEER Strong Motion Database [40], as detailed in Table 2. The records were chosen to include a wide range of moment magnitudes and rupture distances. The records selected were limited to non-pulse like records. An important consideration in the selection of the records was ensuring that the scaling factor would not exceed 5, as

shown in Fig. 4. Prioritising this selection criterion led to multiple records from the same events being chosen, which potentially introduced some bias in the simulation results. The study is limited to rock or stiff soil sites, i.e. Eurocode 8 soil class A, B and C, with the selected ground distributed between Class A, B and C sites in similar proportions to the records in large ground motion databases [41,42].

2.1.1. Ground motion frequency content

Although response or Fourier spectra can be used to fully characterize the frequency content of a ground motion record, when comparing response due to different ground motions it is often more convenient to employ a single scalar parameter to represent the influence of the record frequency content. Various methods have been proposed for this purpose; [43] recommend the mean period, T_m , as a good representation of frequency content as it is related to earthquake magnitude, source-site distance and site conditions. T_m was originally defined by Rathje et al. [44] and is determined as the weighted mean of the periods of the Fourier amplitude spectrum over a pre-defined frequency range, where the weights are assigned based on the Fourier amplitudes and calculated using the following relationship:

$$T_m = \frac{\sum_i C_i^2 \times \frac{1}{f_i}}{\sum_i C_i^2} \quad (1)$$

where, C_i is the Fourier amplitude coefficient corresponding to a frequency f_i obtained from a discrete Fast Fourier Transform. In this study, the calculation was performed for frequencies between 0.25 Hz and 20 Hz, i.e. $0.25 \text{ Hz} < f_i < 20 \text{ Hz}$, using a frequency interval of 0.05 Hz, i.e. $\Delta f = 0.05 \text{ Hz}$. The mean periods of the selected ground motion records in Table 2 range from 0.283 s to 0.944 s.

2.2. Analysis procedure

An analysis procedure similar to that used by Kumar et al. [14] and later by Tsitos et al. [45] in studies of moment resisting frames was employed to examine the EDPs of interest. Firstly, an eigenvalue analysis was performed to obtain the frame natural period, T_1 , and mode shape. The frame was then subjected to pushover analysis, with a force distribution based on the fundamental mode shape, to obtain the base shear and inter-storey drifts at the point of first yield, termed $V_{bs,yield}$ and $\theta_{yield,i}$ respectively. An example pushover curve for one of the six storey archetypes, with the points of brace buckling in compression and yielding in tension highlighted, is shown in Fig. 5.

Incremental dynamic analysis was then carried out by scaling the ground motion records (Fig. 4) to ensure that each frame was evaluated at a consistent level of seismic intensity, taking into account the frame's

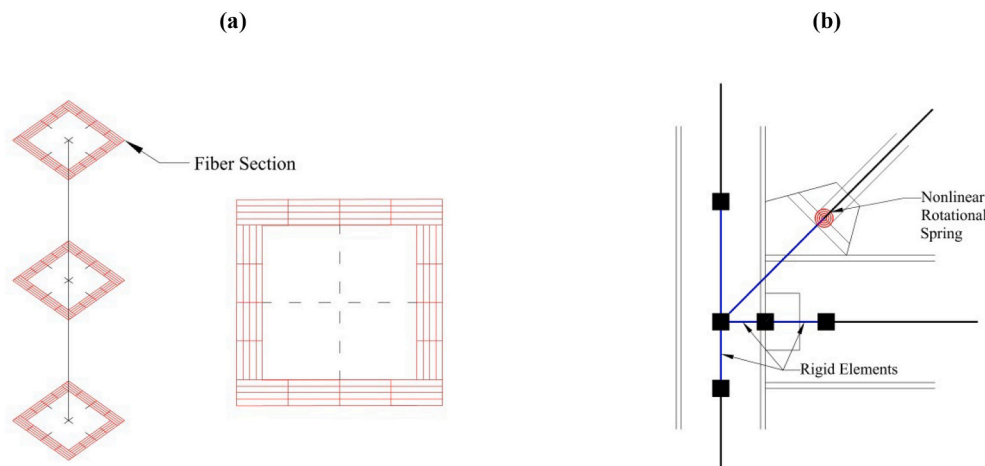


Fig. 2. a) Distributed plasticity Square Hollow Section element with fiber cross section and three integration points; b) gusset plate connection modelling.

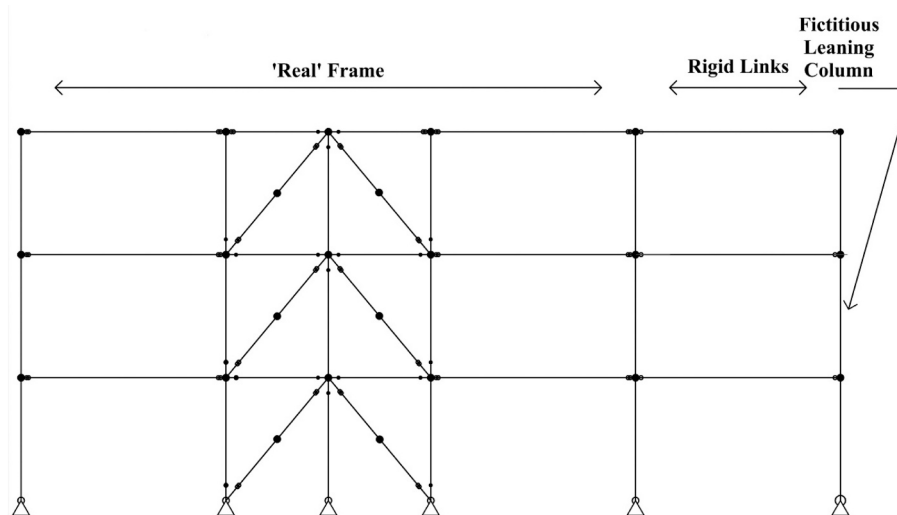


Fig. 3. OpenSEES model of Frame 3_B including leaning column element.

Table 2

Details of ground motion records used in analysis.

Earthquake name	Year	PEER record sequence number	Horizontal component	Moment magnitude	R_{rup} (km)	$V_{s,30}$ (m/s)	Mean period, T_m (s)
Loma Prieta	1989	799	1	6.93	58.65	190.14	0.547
Cape Mendocino	1992	825	1	7.01	6.96	567.78	0.390
Northridge-01	1994	963	1	6.69	20.72	450.28	0.546
Northridge-01	1994	1007	1	6.69	34.2	332.28	0.378
Northridge-01	1994	1080	1	6.69	13.42	557.42	0.520
Chi-Chi_Taiwan	1999	1513	1	7.62	10.97	363.99	0.492
Chi-Chi_Taiwan	1999	1517	1	7.62	11.48	665.2	0.944
Chi-Chi_Taiwan	1999	1524	1	7.62	45.18	446.63	0.512
Parkfield-02_CA	2004	4116	1	6	8.81	246.07	0.435
Niigata_Japan	2004	4209	1	6.63	9.88	372.33	0.618
Niigata_Japan	2004	4219	1	6.63	9.46	480.4	0.380
Niigata_Japan	2004	4223	1	6.63	31.6	493.27	0.486
Chuetsu-oki_Japan	2007	4846	1	6.8	31.43	605.71	0.457
Chuetsu-oki_Japan	2007	4876	1	6.8	12.63	655.45	0.648
Chuetsu-oki_Japan	2007	4886	1	6.8	11.48	338.32	0.477
Chuetsu-oki_Japan	2007	4891	1	6.8	66.44	591.2	0.636
Chuetsu-oki_Japan	2007	4895	1	6.8	10.97	265.5	0.414
Chuetsu-oki_Japan	2007	5194	1	6.8	63.93	384.95	0.551
Iwate_Japan	2008	5482	1	6.9	17.94	458.73	0.283
El Mayor-Cucapah_Mexico	2010	5985	1	7.2	23.42	202.26	0.325
Superstition Hills-02	1987	727	2	6.54	5.61	362.38	0.371
Loma Prieta	1989	741	2	6.93	10.72	476.54	0.343
Loma Prieta	1989	799	2	6.93	58.65	190.14	0.565
Northridge-01	1994	963	2	6.69	20.72	450.28	0.697
Northridge-01	1994	1003	2	6.69	27.01	308.71	0.463
Northridge-01	1994	1080	2	6.69	13.42	557.42	0.463
Northridge-01	1994	1087	2	6.69	15.6	257.21	0.415
Chi-Chi_Taiwan	1999	1524	2	7.62	45.18	446.63	0.383
Chi-Chi_Taiwan-06	1999	3474	2	6.3	10.05	363.99	0.377
Niigata_Japan	2004	4219	2	6.63	9.46	480.4	0.454
Niigata_Japan	2004	4223	2	6.63	31.6	493.27	0.425
Chuetsu-oki_Japan	2007	4846	2	6.8	31.43	605.71	0.485
Chuetsu-oki_Japan	2007	4874	2	6.8	20	561.59	0.602
Chuetsu-oki_Japan	2007	4886	2	6.8	11.48	338.32	0.432
Chuetsu-oki_Japan	2007	4891	2	6.8	66.44	591.2	0.682
Iwate_Japan	2008	5672	2	6.9	63.53	252.68	0.492
Iwate_Japan	2008	5818	2	6.9	12.85	512.26	0.394
El Mayor-Cucapah_Mexico	2010	5837	2	7.2	20.08	229.25	0.544
El Mayor-Cucapah_Mexico	2010	5985	2	7.2	23.42	202.26	0.370
El Mayor-Cucapah_Mexico	2010	5992	2	7.2	16.21	196.25	0.558

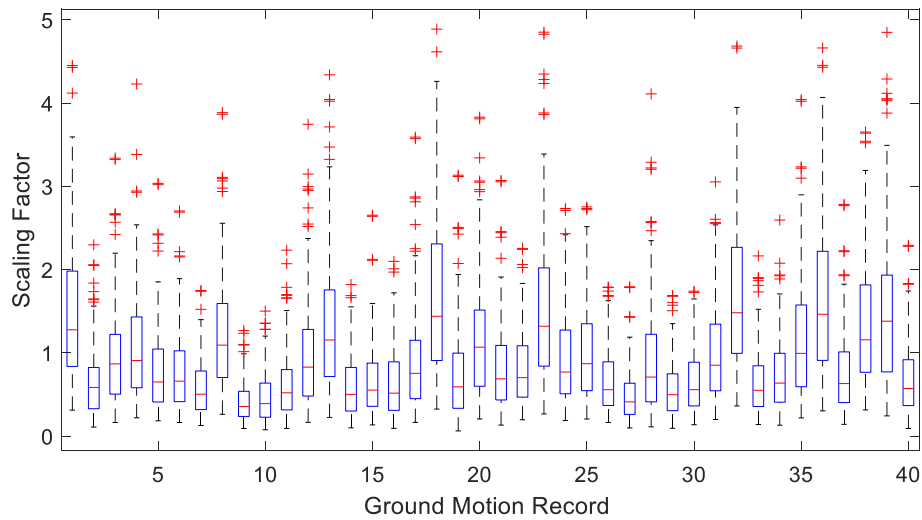


Fig. 4. Scaling factors applied to ground motion records in incremental dynamic analysis.

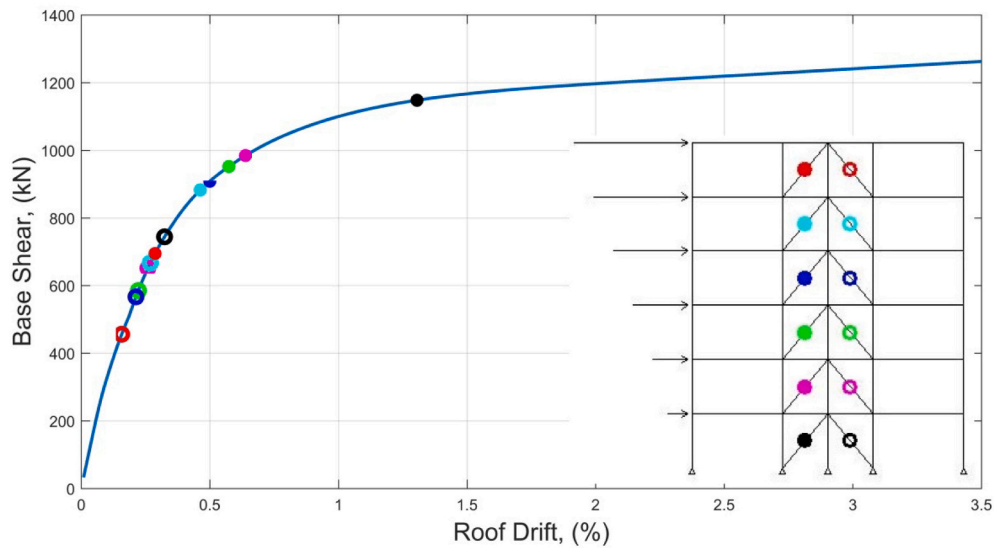


Fig. 5. Hinge location (brace buckling shown by hollow circles, tension yielding shown by solid circles) and pushover curves for the 6_A frame.

yield strength and natural period, T_1 , and different values of the effective behaviour factor, q_{eff} . The scaling factor (SF) applied to each ground acceleration record is given by:

$$SF = \frac{q_{eff} \times V_{bs,yield}}{S_a(T_1) \times \gamma \times m} \quad (2)$$

where q_{eff} is the effective behaviour factor, $V_{bs,yield}$ is the base shear at first yield, obtained from the pushover analysis, $S_a(T_1)$ is the value of the acceleration spectrum of the ground motion record at the fundamental period of the structure being examined, γ is the first mode mass participation ratio (i.e. the ratio of the effective first modal mass to overall mass), and m is the structural mass. It is important to point out that the effective behaviour factor q_{eff} is not the same as the behaviour factor used in design, q . Instead it describes the extent of the nonlinear

response demand experienced by the structure during a particular ground motion.

For each case study building, the ground motion records were scaled to achieve q_{eff} values of 1.5, 2, 3, 4 and 5, representing increasing levels of relative seismic intensity. These scaled records were then used to perform time history analysis using the OpenSEES models previously described. In each time-history analysis, the peak inter-storey drift in each storey and the peak floor acceleration at each floor level was recorded.

3. Results

3.1. Peak inter-Storey drift

Inter-storey drift in each storey level, i , is represented by a drift

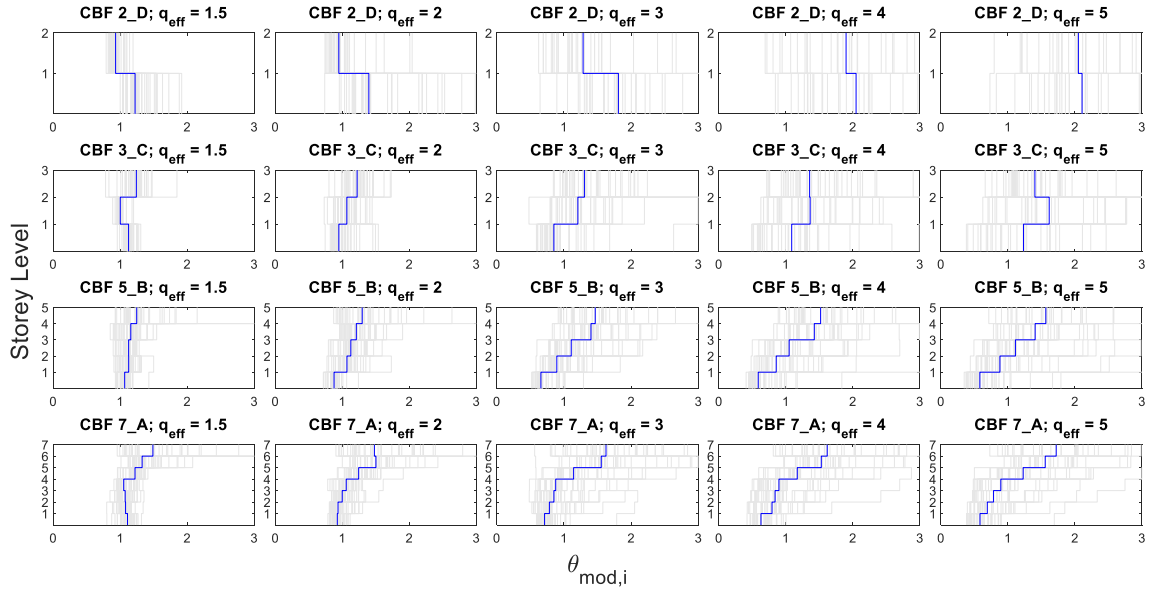


Fig. 6. Example $\theta_{mod,i}$ results obtained from NLTHA for four of the archetype CBFs. Median values shown in blue. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

modification factor [14], $\theta_{mod,i}$ which is defined as:

$$\theta_{mod,i} = \frac{\theta_{max,i}}{q_{eff} \times \theta_{yield,i}} \quad (3)$$

where $\theta_{max,i}$ is the maximum inter storey drift experienced in storey level i , $\theta_{yield,i}$ is the inter-storey drift in level i at the time of first yield anywhere in the structure and q_{eff} controls the relative seismic intensity in each time-history analysis through the scaling factor, SF, in Eq. (2). The use of this drift normalization, which normalises the maximum inelastic drift in each storey by the drift obtained from an equivalent linear static analysis, facilitates comparison with other inter-storey drift models. Some example results for four of the archetype CBFs are presented in

of $\theta_{mod,i}$ were assumed to be lognormally distributed. In earthquake engineering, drift response is commonly assumed to be lognormally distributed (for example [46] or [4]) even if formal tests indicate that this is not true (e.g. [47]). Here the values of $\theta_{mod,i}$ were found to be not strictly lognormally distributed, however, for consistency with other studies, regression analysis was performed using the log-transformed data.

A number of different linear and nonlinear functional forms were considered for regression analysis. Ultimately, based on the standard error of the regression and examination of the model residuals, the functional form given in Eq. (4) was selected:

$$\ln(\theta_{mod,i}) = (q_{eff} - 1) \left(c_0 + c_1 q_{eff} + c_2 \frac{h_i}{H} + c_3 \frac{T_1}{T_m} + c_4 \beta_{k,i} + c_5 \left(\frac{h_i}{H} + c_6 \right)^{\Omega_{max} + c_7} \right) + \varepsilon \quad (4)$$

Fig. 6.

Regression analysis was carried out to fit a model for $\theta_{mod,i}$ at each level to the values recorded in the NLTHAs. For this purpose, the values

where h_i is the height of storey i , H is the overall building height, $\beta_{k,i}$ is the ratio of column-to-brace stiffness at level i (which, as mentioned in the introduction, has previously been shown to influence drift demand

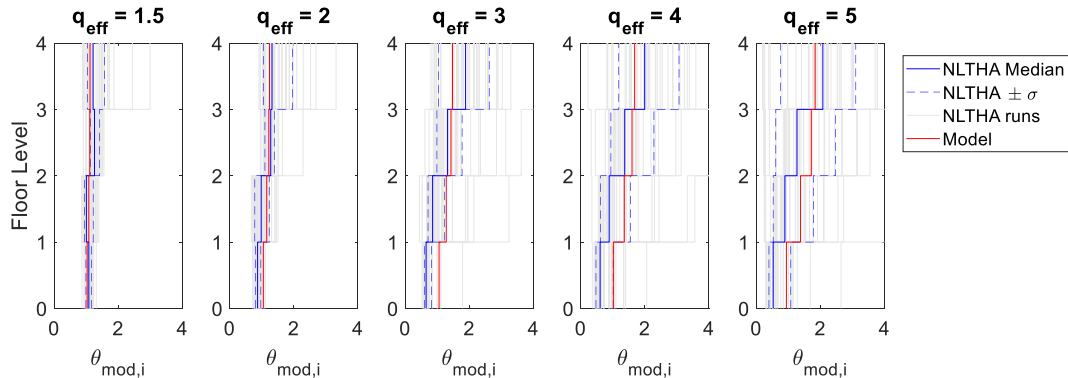


Fig. 7. Comparison of $\theta_{mod,i}$ values obtained from NLTHA (blue) and using the proposed regression model (red) for the 4 storey ‘assessment’ CBF. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

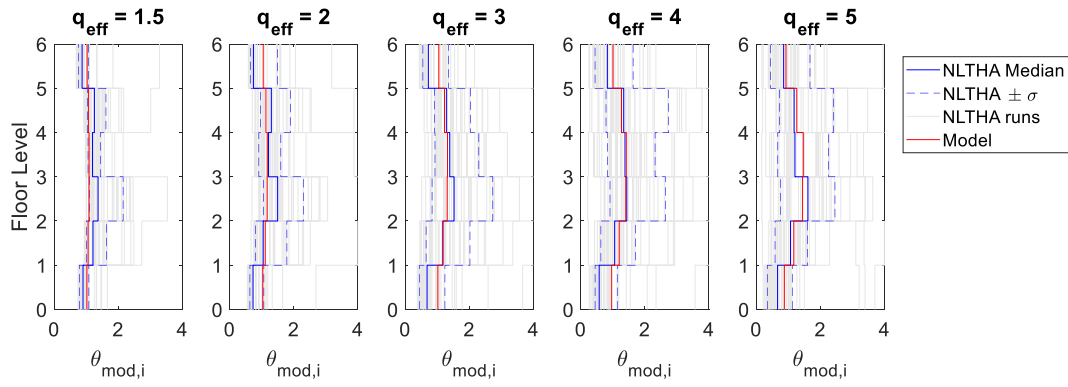


Fig. 8. Comparison of $\theta_{mod,i}$ values obtained from NLTHA (blue) and using the proposed regression model (red) for the 6 storey ‘assessment’ CBF. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

Table 3

Coefficients and 95% confidence intervals for the prediction of $\theta_{mod,i}$ using Eq. (4).

	c_0	c_1	c_2	c_3	c_4	c_5	c_6	c_7	σ
Coefficient	0.127	−0.021	0.999	−0.094	−0.541	−0.470	0.350	0.665	0.374
95% Confidence Intervals	0.105	−0.024	0.922	−0.102	−0.629	−0.540	0.318	0.554	–
	0.149	−0.019	1.075	−0.086	−0.453	−0.400	0.382	0.777	–

in CBFs), Ω_{max} is the maximum brace overstrength over the frame height and ε is an error term, i.e. a random variable describing the model residuals. Column-to-brace stiffness at level i is calculated as the ratio of the bending stiffness of the columns to the lateral stiffness of the braces, which can be shown to be [1]:

$$\beta_{k,i} = \frac{L_{br} \sum I_c}{L_c^3 \cos \alpha \sum A_{br}} \quad (5)$$

where L_{br} is the brace length I_c is the column second moment of area, L_c is the length of the column, α is the angle between the diagonal and the horizontal projection and A_{br} is the brace cross sectional area.

The drift response data is heteroskedastic with respect to h_i/H and q_{eff} , with the spread of the data, that is the record-to-record variability of the response, increasing in the upper storeys and at higher ground motion intensities, as can be observed in Figs. 6, 7 and 8. As this violates the underpinning assumption of least squares regression, robust regression was performed using the MATLAB `nlinfit` function with the Bisquare influence function. The coefficients describing the fitted equation, the associated 95% confidence intervals and the standard error of the regression, σ , are presented in Table 3.

A nonlinear regression model was employed as examination of the calculated $\theta_{mod,i}$ values identified a change in the shape of the inter-storey drift profile over the frame height with increasing values of q_{eff} . For small q_{eff} values, $\theta_{mod,i}$ displays relatively little variation throughout the height of the structure (see, for example, the plots for $q_{eff} = 1.5$ and $q_{eff} = 2$ in Fig. 7). However, as q_{eff} is increased, greater variation in the $\theta_{mod,i}$ value over the frame height is observed. The nonlinear regression model employed is capable of capturing this change in profile with increasing q_{eff} .

The inclusion of the $(q_{eff} - 1)$ pre-multiplier term in Eq. (4) fixes the value of $\theta_{mod,i}$ to unity when q_{eff} is equal to 1, which imposes the

constraint that elastic drift should equal inelastic drift if there is no nonlinear behaviour. The power term, $c_5 \left(\frac{h_i}{H} + c_6 \right)^{\Omega_{max} + c_7}$, in Eq. (4) dictates the shape of the inter-storey drift profile over the frame height and is specific to Eurocode 8 design in which, as mentioned previously, limitations are applied on brace overstrength and non-dimensional slenderness. The slenderness limitations in the code often result in frame designs with relatively large values of overstrength in the upper floor levels, where seismic forces are typically lowest, as brace sections are chosen to satisfy slenderness limits rather than strength demands. In cases where the overstrength in the upper storeys is relatively large (approximately greater than 1.4), CBFs tend to exhibit curved $\theta_{mod,i}$ profiles, with largest values observed in the middle storeys. However, in cases where overstrength at the top level is dictated by brace strength demand and overstrength is close to unity, the $\theta_{mod,i}$ profile tends to increase linearly over the frame height with maximum values observed in the upper storeys. The inclusion of the power term allows the regression model to capture these variations.

In order to assess the performance of the regression models, two further CBFs, a four storey and a six storey structure, were designed in accordance with Eurocode 8, modelled in OpenSees and analysed in a similar manner to those described previously. Details of these frames are given in Tables 4 and 5. Results from these CBFs were not included in the database used to develop the regression equations; their purpose was purely to assess the performance of the proposed equations. Therefore they are referred to as the ‘assessment’ CBFs. Figs. 7 and 8 compare the peak inter-storey drifts predicted by the regression model to those computed using NLTHA for the two ‘assessment’ structures. A number of trends may be observed. Firstly, the predicted median value from the regression model (in red) is usually reasonably close to the median recorded values from NLTHA (in blue). Secondly, here is a notable increase in the level of scatter, i.e. record to record variability, as the behaviour factor is increased. This heteroskedasticity has been accounted for through the use of robust regression, as described above. Finally, the variation in the shape of the $\theta_{mod,i}$ profile between different CBFs, and the ability of the model to capture this, is evident when the two figures are compared.

Table 4

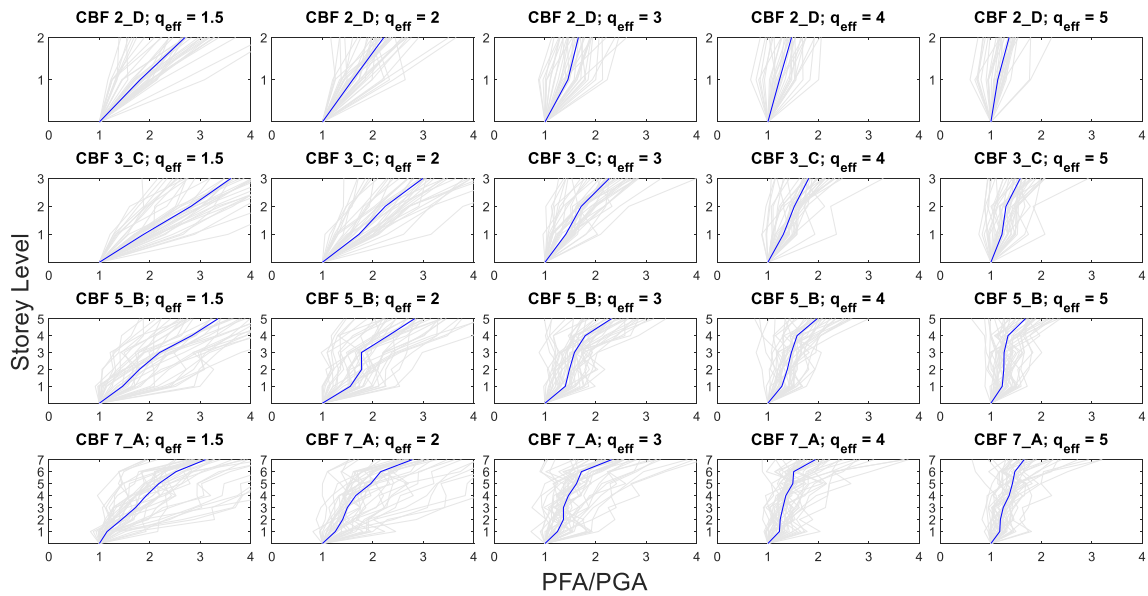
Structural characteristics of assessment CBFs.

Frame ID	Storeys	Design spectral acceleration $S_d(T_1)$ (g)	T_1 (s)	Height (m)
4_Assessment	4	0.21	0.38	14.4
6_Assessment	6	0.17	0.70	21.6

Table 5

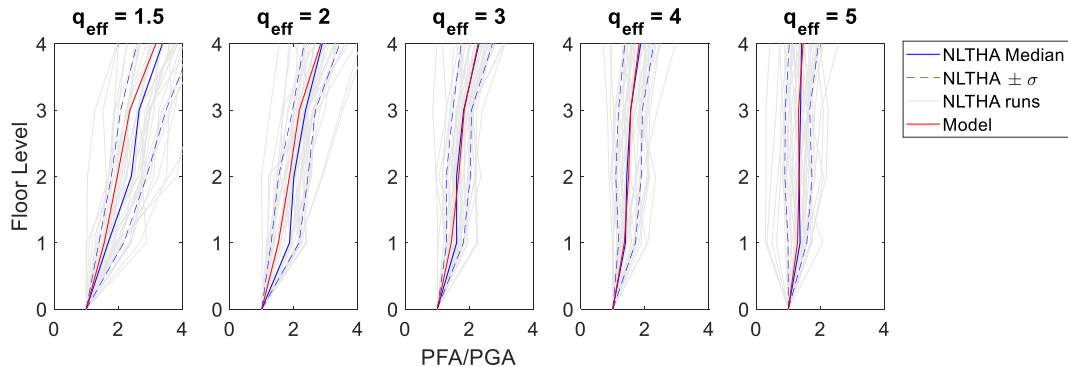
Details of structural designs for assessment CBFs.

Level	Beam	Column	Brace	Brace overstrength Ω_i	Brace non-dimensional slenderness $\bar{\lambda}$	Column to brace stiffness ratio $\beta_{k,i}$
4_Assessment						
4	305x165x40	152x152x44	90x90x4	1.01	1.54	0.012
3	305x165x40	203x203x60	100x100x8	1.02	1.45	0.013
2	305x165x40	203x203x100	120x120x8.8	1.00	1.20	0.018
1	305x165x40	254x254x132	140x140x8.8	1.05	1.01	0.030
6_Assessment						
6	305x165x40	203x203x46	150x100x3	1.63	1.94	0.013
5	305x165x40	203x203x71	140x140x5	1.36	1.47	0.012
4	305x165x40	203x203x100	140x140x8	1.47	1.50	0.011
3	305x165x40	203x203x127	140x140x8.8	1.31	1.51	0.014
2	305x165x40	356x368x153	140x140x10	1.30	1.53	0.039
1	305x165x40	356x368x177	140x140x12.5	1.51	1.56	0.038

**Fig. 9.** Example PFA/PGA results obtained from NLTHA for four of the archetype CBFs. Median values shown in blue. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)**Table 6**

Coefficients and 95% confidence intervals for the prediction of PFA/PGA using Eq. (6).

	c_0	c_1	c_2	c_3	c_4	c_5	σ
Coefficient	2.901	-0.222	-3.022	0.046	-0.386	1.746	0.264
95% Confidence Intervals	2.837	-0.226	-3.201	0.042	-0.406	1.623	—
	2.965	-0.218	-2.842	0.050	-0.366	1.869	—

**Fig. 10.** Comparison of PFA/PGA values obtained from NLTHA (blue) and using the proposed regression model (red) for the 4 storey 'assessment' CBF. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

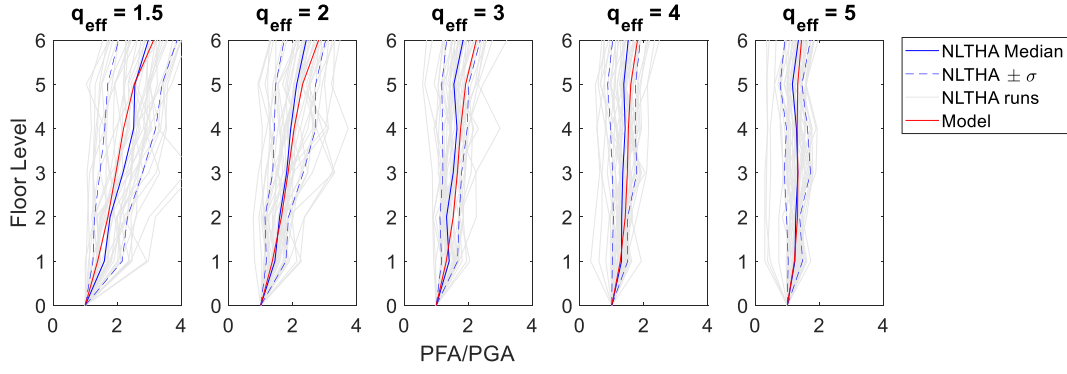


Fig. 11. Comparison of PFA/PGA values obtained from NLTHA (blue) and using the proposed regression model (red) for the 6 storey ‘assessment’ CBF. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

3.2. Peak floor acceleration

Peak floor acceleration was quantified using the floor acceleration magnification factor, that is the ratio of peak floor acceleration to peak ground acceleration PFA_i/PGA . Some example results for four of the archetype CBFs are presented in Fig. 9.

A nonlinear regression model was used to model the variation of this ratio over the frame height. As with inter-storey drift, peak floor acceleration magnification was assumed to be lognormally distributed; an assumption which [24] have shown to be reasonable. The regression equation used to model peak floor acceleration magnification is:

$$\ln\left(\frac{PFA_i}{PGA}\right) = \frac{h_i}{H} \left(c_0 + c_1 q_{eff} + c_2 \frac{h_i}{H} + c_3 NS + c_4 \frac{T_1}{T_m} + c_5 \left(\frac{h_i}{H} \right)^2 \right) + \varepsilon \quad (6)$$

in which NS is the number of storeys and the other variables are as defined in Eq. (4). A nonlinear regression model is used as this allows the equation to capture the fundamental property that the peak floor acceleration at the ground level must equal the peak ground acceleration. Furthermore, the use of a nonlinear model allows the nonlinear variation in acceleration profile over the height of a structure noted in previous studies, including whiplashing effects, to be captured effectively. The coefficients describing the fitted equation, the associated 95% confidence intervals, and the standard error of the regression σ , are presented in Table 6. As with $\theta_{mod,i}$, the data is again heteroskedastic, this time with respect to the relative height. Consequently, as before, robust regression nonlinear was employed.

As with Figs. 7 and 8 for inter-storey drift, Figs. 10 and 11 compare the PFA/PGA ratios predicted by the regression model in Eq. (6) to those computed using NLTHA for the two ‘assessment’ structures examined. The proposed regression model generally gives a good prediction of the median values obtained from NLTHA. The most obvious trend in the data is the change in slope of the PFA/PGA profile with q_{eff} . For lower q_{eff} values a consistent increase in PFA/PGA over the frame height is observed, however as q_{eff} is increased the PFA displays less variation between floor levels, leading to an almost uniform profile at $q_{eff} = 5$. Furthermore, taller frames tend to exhibit more uniform PFA/PGA profiles. These trends have been noted in previous studies on different frame types, and the developed model appears to capture them accurately and consistently for CBFs. Although not particularly evident for the two ‘assessment’ frames, the model is also capable of capturing the

‘whiplash’ effect encountered in taller structures, suggested here by the slight tendency to overestimate roof acceleration in Fig. 11.

3.3. Regression using FEMA P-58 functional forms

To further evaluate the regression models represented by Eqs. (4) and (6), regression analysis was also performed using the functional forms for normalized drift and acceleration proposed in FEMA P-58 (FEMA, 2012), shown in Eqs. (7) and (8). Note that some of the notation used in this study differs from FEMA P-58: q_{eff} , $\theta_{mod,i}$ and PFA/PGA are denoted S , $H_{\Delta i}$ and H_{ai} respectively in FEMA P-58. As previously mentioned, the values of the coefficients given in FEMA P-58 were obtained from the results of analyses of eccentrically braced frames designed to US provisions, so their applicability to CBFs is questionable.

$$\ln(\theta_{mod,i}) = a_0 + a_1 T_1 + a_2 q_{eff} + a_3 \frac{h_i}{H} + a_4 \frac{h_i^2}{H} + a_5 \frac{h_i^3}{H} + \varepsilon \quad (7)$$

$$\ln\left(\frac{PFA}{PGA}\right) = a_0 + a_1 T_1 + a_2 q_{eff} + a_3 \frac{h_i}{H} + a_4 \frac{h_i^2}{H} + a_5 \frac{h_i^3}{H} + \varepsilon \quad (8)$$

Table 7 provides details of the results of regression analysis performed using the functional forms in Eqs. (7) and (8) with the database of structural and response parameters developed in this study. The standard deviation values for Eqs. (7) and (8) in Table 7 display greater variability than those for Eqs. (4) and (5) in Tables 3 and 6; thus Eqs. (7) and (8) offer greater simplicity at the expense of some accuracy, as quantified by the relative standard deviation values.

However, Eqs. (7) and (8) possess a number of intrinsic shortcomings compared to Eqs. (4) and (6). The column-to-brace stiffness ratio and the ground motion frequency content, parameters that have previously been shown to have a significant influence on CBF inelastic drift, are both omitted in the prediction of $\theta_{mod,i}$ using Eq. (7), while Eq. (7) is also incapable of capturing the dependency of the $\theta_{mod,i}$ profile on q_{eff} or overstrength. Furthermore, Eq. (8) fails to capture the fact that at ground level the floor acceleration must be equal to the ground acceleration. FEMA P-58 recommends using linear interpolation between a value of 1 at the base ($h_i/H = 0$) and the value given by Eq. (8) at $h_i/H = 0.2$. On the other hand, compared to Eqs. (4) and (6) the functional forms in Eqs. (7) and (8) are relatively simple with fewer input parameters needing to be calculated and less knowledge of site conditions required.

Table 7

Revised coefficients for Eqs. (7) and (8) using the data developed in this for CBFs designed to EC8.

	a_0	a_1	a_2	a_3	a_4	a_5	σ
Coefficient for $\theta_{mod,i}$	−0.012	−0.640	0.019	1.395	−0.849	–	0.420
Coefficient for PFA/PGA	0.852	−0.217	−0.143	0.195	0.255	–	0.280

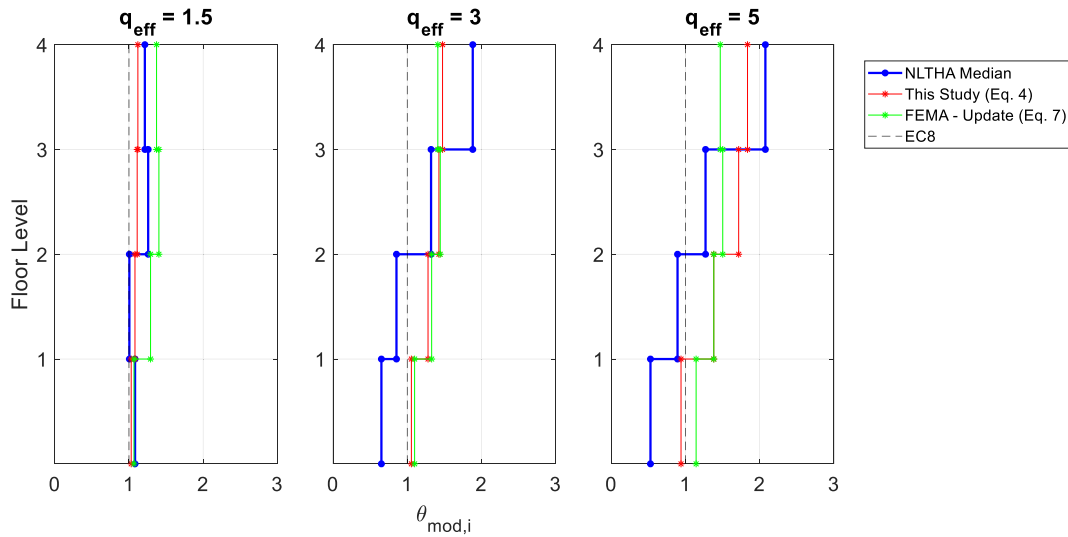


Fig. 12. $\theta_{mod,i}$: Comparison of proposed regression model (Eq. (4)) with NLTHA results, FEMA P-58 predictions and Eurocode 8 design assumptions for the 4 storey assessment CBF.

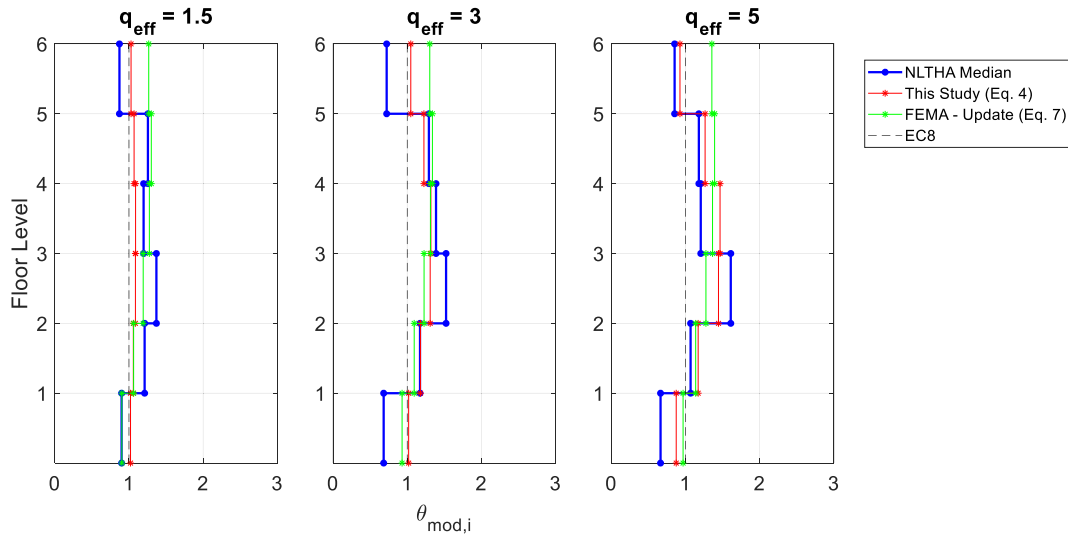


Fig. 13. $\theta_{mod,i}$: Comparison of proposed regression model (Eq. (4)) with NLTHA results, FEMA P-58 predictions and Eurocode 8 design assumptions for the 6 storey assessment CBF.

3.4. Comparison with other models and design codes

This section compares the models for $\theta_{mod,i}$ and PFA/PGA developed above with those given in Eurocode 8. The models developed using the nonlinear functional forms proposed in this paper, i.e. Eqs. (4) and (6) with coefficients given in Tables 3 and 6, and the models fitted to the functions proposed in FEMA P-58, i.e. Eqs. (7) and (8) with coefficients as per Table 7, are considered in this comparison.

Figs. 12 and 13 compare the median $\theta_{mod,i}$ values computed in NLTHA for the two ‘assessment’ CBFs with the results of the nonlinear regression model developed in this study, the updated FEMA P-58 model and the Eurocode 8 rule. As discussed in the introduction, Eurocode 8 implicitly adopts the equal displacements rule. This implies that $\theta_{mod,i} = 1$ is expected throughout the entire structure. It can be seen that the equal displacements rule, and therefore Eurocode 8, is generally a reasonably successful estimator of the average $\theta_{mod,i}$ values observed in the NLTHA but it often underpredicts the highest values of $\theta_{mod,i}$, particularly in the upper storeys. Both the regression model proposed in

this paper and the updated FEMA P-58 model appear to perform better, achieving reasonable agreement with the variation of median NLTHA results throughout both frames. The maximum percentage difference for the cases shown in Figs. 12 and 13 are 78% and 116% for the model proposed in this study and the FEMA P-58 model respectively. Moreover, the majority (roughly 7 in 10) of $\theta_{mod,i}$ values obtained using either equation are within $\pm 30\%$ of the value obtained from NLTHA.

Figs. 14 and 15 compare the models for floor acceleration magnification over the frame height developed in this study, the values given by Eurocode 8 and the median NLTHA results for the 4 storey and 6 storey ‘assessment’ CBFs. Both models developed here are successful in capturing the nonlinear variation in acceleration profile over the frame height, including ‘whiplash’ type behaviour. A change in profile shape as q_{eff} increases is also observed in the NLTHA results. This is captured by both models developed in this study, but not by Eurocode 8, which, as discussed in the introduction, assumes floor acceleration magnification increases linearly from 1 at ground level to 2.5 at roof level irrespective of the degree of nonlinearity. The code values underestimate peak

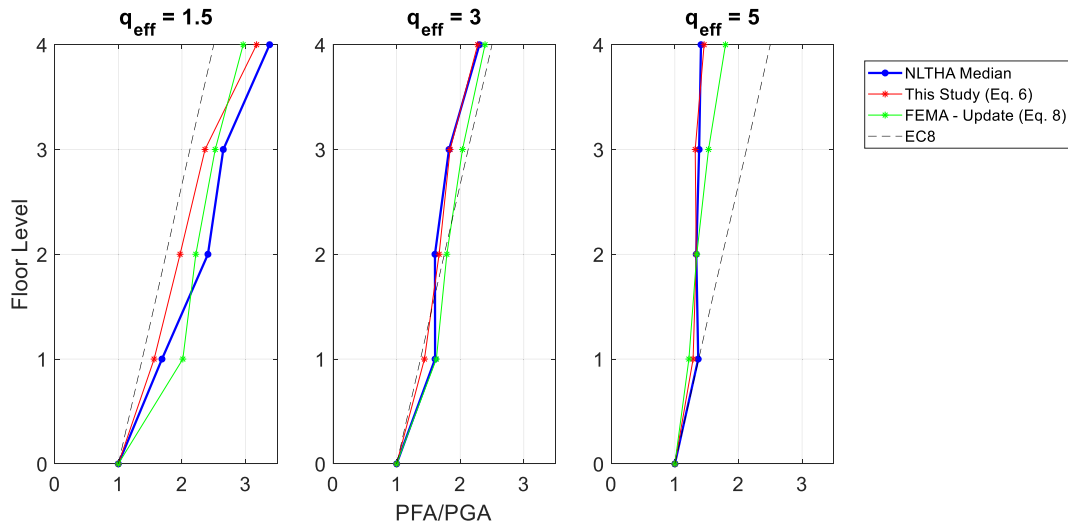


Fig. 14. PFA/PGA: Comparison of proposed regression model (Eq. (6)) with NLTHA results, FEMA P-58 predictions and Eurocode 8 design assumptions for the 4 storey assessment CBF.

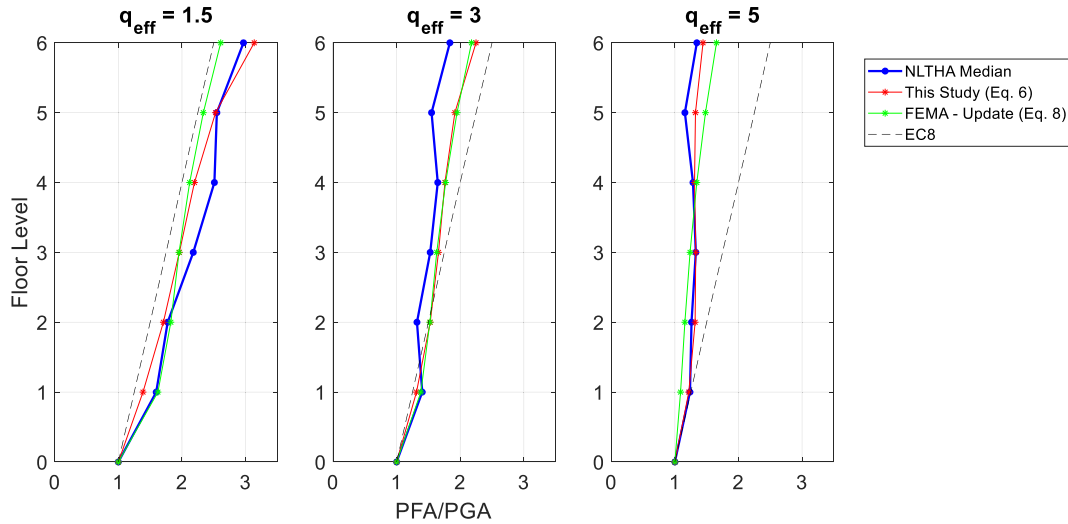


Fig. 15. PFA/PGA: Comparison of proposed regression model (Eq. (6)) with NLTHA results, FEMA P-58 predictions and Eurocode 8 design assumptions for the 6 storey assessment CBF.

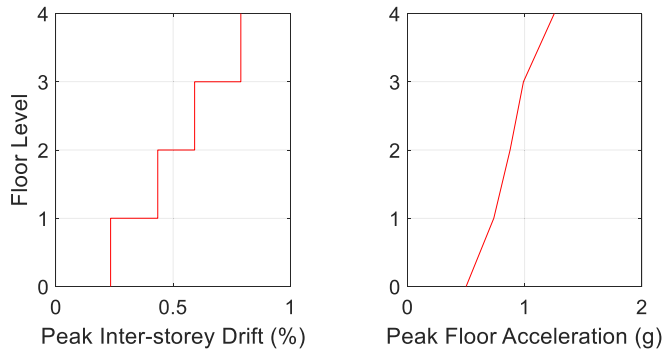


Fig. 16. Peak inter-storey drifts and peak floor accelerations predicted using Eqs. (4) and (6) for an example ground motion intensity of $S_a(T_1) = 1$ g.

acceleration for $q_{eff} = 1.5$ and overestimate it for higher q_{eff} values. This supports the conclusions of a series of other NLTHA-based studies for different frame types [19,20]. As with $\theta_{mod,i}$, both the model proposed in

this study and the updated FEMA P-58 model appear to perform reasonably well. The maximum percentage error for the cases presented in Figs. 15 and 16 is less than 25% for the proposed model and less than 30% for the FEMA model, with the majority of values (roughly 7 in 10) within $\pm 10\%$ of the median NLTHA result. Close inspection suggests that the regression equation proposed here is more capable than the FEMA P-58 expression at capturing the transition between the linearly increasing and saturated response profiles observed with increasing q_{eff} . This is reflected by the fact that the FEMA P-58 model consistently underestimates roof accelerations for lower q_{eff} and overestimates it for higher q_{eff} . In contrast, no such systematic trends appear evident in the results of the model proposed in this study.

3.5. Example application for performance assessment

An illustrative example using the 4 storey ‘assessment’ CBF is provided to demonstrate how the proposed expressions in Eqs. (4) and (6) can be related to ground motion intensity and thereby employed in performance assessment. Of the parameters in Eqs. (4) and (6), building specific values of H , h_b , NS , T_1 , $\beta_{k,i}$ and Ω_{max} can be calculated from the

properties of the CBF, but q and T_m depend on the properties of the ground motion.

Performance assessment is carried out for a defined level of seismic hazard, with seismic hazard typically quantified by a spectral acceleration value at the fundamental period of the structure, $S_a(T_1)$, specific to the site and seismic hazard level of interest. In a complete performance assessment calculation this would be obtained through a detailed seismic hazard analysis of the site, however in this example it is simply assumed that $S_a(T_1)$ is equal to 1 g. A pushover analysis is performed to obtain the base shear at first yield, $V_{bs, yield}$. By manipulating Eq. (2), a value for q_{eff} corresponding to the applied hazard can be obtained as:

$$q_{eff} = \frac{S_a(T_1) \times \gamma \times m}{V_{bs, yield}} \quad (9)$$

This is a measure of the level of nonlinearity induced by a ground motion record that perfectly represents the seismic hazard, and thus has a scaling factor of 1 in Eq. (2). For the 4 storey 'assessment' CBF with $S_a(T_1) = 1$ g, Eq. (9) gives $q_{eff} = 2.74$.

There are a number of potentially applicable methods of calculating an appropriate T_m value for application in Eqs. (4) and (6). One option is to select a set of ground motion records appropriate for the site through, for example, the Conditional Spectrum approach [48] and then use the average T_m value of these records. A less computationally demanding technique is to use predictive equations, such as those proposed in [44,49,50], which allow an appropriate T_m value to be calculated as a function of causal magnitude, source-site distance and other site dependant parameters. However, this still requires disaggregation of the seismic hazard to obtain these input parameters. A simple alternative is to use Eq. (10) proposed by Kumar et al. [43] based on the characteristic period, T_c , which can be approximated from the Eurocode 8 design spectrum without the need for hazard disaggregation information, which can be difficult to obtain for some sites.

$$T_m = 1.05T_c \quad (10)$$

These values of q_{eff} and T_m are employed in Eqs. (4) and (6) along with the building specific parameters H , h_b , NS , T_1 , $\beta_{k,i}$ and Ω_{max} to calculate $\theta_{mod,i}$ and PFA/PGA. Eq. (3) is then used to calculate the peak inter-storey drift in each storey, $\theta_{max,i}$, from $\theta_{mod,i}$ and the inter-storey drifts at first yield, $\theta_{yield,i}$, calculated in the pushover analysis. Similarly, by using the peak ground acceleration corresponding to the seismic hazard of interest, which can be obtained as $S_a(T_1 = 0)$ from a seismic hazard assessment and is assumed to be 0.5 g in this example, values for the peak floor accelerations can be calculated.

The resulting $EDPs$ predicted by the proposed models for the 4 storey 'assessment' CBF are shown in Fig. 16. Both inter-storey drift and peak floor acceleration are observed to increase with floor level, with maximum values of approximately 0.8% and 1.3 g, respectively. These results can be directly employed in performance assessment procedures, and the $EDPs$ required for performance assessments at other seismic hazard levels could be obtained by employing appropriate $S_a(T_1)$ values in the above procedure. For CBF design, the combinations of building specific parameters that achieve required levels of seismic performance can be conveniently identified through iterative calculation of $EDPs$ using Eqs. (4) and (6).

A similar process can be followed to predict the drift and acceleration response of other CBFs. As with all regression equations, applicability decreases outside the range of the training data. The equations can be used to predict the nonlinear response of 2 to 7 storey CBFs to non-pulse like ground motions, in cases where seismic design is performed

according Eurocode 8 with design lateral resistance provided by the tension braces only. Accuracy of the equations is likely to be best for the frame and bracing configurations considered in the paper, however the structural archetypes considered in model calibration are generically representative of frames with concentric diagonal bracing, as defined in Eurocode 8.

4. Conclusions

Advanced earthquake engineering methods such as performance-based design and structural optimisation rely upon reliable predictions of key engineering demand parameters ($EDPs$) at multiple levels of seismic intensity. The complex inelastic behaviour of structural forms such as CBFs suggests that these should be obtained through nonlinear time-history analysis (NLTHA). In practice, however, the cost of performing NLTHA analyses is prohibitive for most building design projects, indicating the need for more efficient models that can provide the required seismic demand data with sufficient accuracy.

Multiple NLTHAs were performed in OpenSEES to examine the inelastic seismic response of a set of case study CBFs designed to Eurocode 8. The results of these analyses were used to develop regression models for the drift modification factor, $\theta_{mod,i}$, and floor acceleration magnification factor, PFA/PGA. These allow the $EDPs$ of peak inter-storey drift and peak absolute acceleration to be predicted at each floor level in a braced frame without the need to perform NLTHA. The proposed regression models offer advantages over alternative simplified methods as they provide EDP predictions for each individual storey, they include separately-calibrated regression equations for acceleration as well as drift, and they capture the specific characteristics of Eurocode 8-compliant CBFs, including limits on slenderness and overstrength.

The results of the proposed models were compared to values calculated using procedures in FEMA P-58 and Eurocode 8. Compared to the values predicated in this study, Eurocode 8 generally estimates $\theta_{mod,i}$ reasonably well, but can be unconservative at higher storeys or low T_1/T_m ratios. For peak floor acceleration the code appears quite conservative, particularly for high inelastic ductility demand. The proposed regression equations are simple to employ, and the use of a nonlinear formulation and robust regression successfully captures the heteroskedasticity of the response data with respect to height and behaviour factor; these being associated with the whiplash effect and the highly nonlinear response of CBFs, respectively.

CRedit authorship contribution statement

John Hickey: Conceptualization, Methodology, Software, Formal analysis, Data curation, Writing – original draft. **Brian Broderick:** Conceptualization, Methodology, Writing – review & editing, Supervision.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Appendix A. Full details of archetype structures

Table A1

Details of structural designs for archetype CBFs.

Level	Beam	Column	Brace	Beam	Column	Brace	Beam	Column	Brace	Beam	Column	Brace
2	2_A			2_B			2_C			2_D		
	305x165x40	203x203x46	90x90x3.6	305x165x40	203x203x46	100x100x3.6	305x165x40	203x203x52	120x120x4	305x165x40	203x203x52	120x120x4
	305x165x40	203x203x46	100x100x5	305x165x40	203x203x46	120x120x5	305x165x40	203x203x71	120x120x6.3	305x165x40	203x203x86	140x140x6.3
3	3_A			3_B			3_C			3_D		
	305x165x40	152x152x37	70x70x3	305x165x40	203x203x46	80x80x3	305x165x40	152x152x51	80x80x3.6	305x165x40	203x203x46	80x80x5
	305x165x40	203x203x46	90x90x4	305x165x40	203x203x60	100x100x5	305x165x40	203x203x46	80x80x8	305x165x40	203x203x60	140x140x5
1	305x165x40	203x203x52	90x90x5	305x165x40	203x203x71	100x100x6.3	305x165x40	203x203x86	80x80x10	305x165x40	203x203x86	140x140x6.3
	4_A			4_B			4_C			4_D		
	305x165x40	152x152x37	70x70x3	305x165x40	152x152x44	80x80x3.2	305x165x40	152x152x44	90x90x3.6	305x165x40	152x152x51	100x100x4
3	305x165x40	152x152x51	100x100x4	305x165x40	203x203x46	90x90x6.3	305x165x40	203x203x52	90x90x8	305x165x40	203x203x60	100x100x8.8
	305x165x40	203x203x60	90x90x6.3	305x165x40	203x203x71	140x140x5	305x165x40	203x203x86	100x100x10	305x165x40	203x203x100	140x140x8
	305x165x40	203x203x86	90x90x7.1	305x165x40	203x203x100	90x90x10	305x165x40	254x254x132	140x140x8	305x165x40	254x254x132	200x120x8
5	5_A			5_B			5_C			5_D		
	305x165x40	152x152x37	70x70x3	305x165x40	203x203x46	80x80x3.6	305x165x46	203x203x46	150x100x3	305x165x46	152x152x51	120x120x4
	305x165x40	152x152x51	90x90x5	305x165x40	203x203x71	120x120x5	305x165x46	203x203x86	120x120x6.3	305x165x46	203x203x60	200x120x6
3	305x165x40	254x254x89	90x90x7.1	305x165x40	254x254x107	120x120x7.1	305x165x46	203x203x86	120x120x8	305x165x46	254x254x167	140x140x8.8
	305x165x40	305x305x118	90x90x8.8	305x165x40	305x305x137	120x120x8.8	305x165x46	254x254x107	120x120x10	305x165x46	254x254x167	250x150x8
	305x165x40	356x368x129	100x100x8	305x165x40	356x368x177	140x140x8	305x165x46	305x305x137	140x140x8.8	305x165x46	305x305x198	250x150x8
6	6_A			6_B			6_C			6_D		
	305x165x46	152x152x37	70x70x3	305x165x40	203x203x46	120x120x5	356x171x51	203x203x46	150x100x3	305x165x54	152x152x51	120x120x4
	305x165x40	152x152x51	120x120x4	305x165x40	203x203x113	140x140x6.3	356x171x51	203x203x86	120x120x7.1	305x165x54	203x203x60	140x140x7.1
4	305x165x40	203x203x100	90x90x8	305x165x40	254x254x107	140x140x8	356x171x51	254x254x89	120x120x10	305x165x54	254x254x107	120x120x12.5
	305x165x40	254x254x167	90x90x10	305x165x40	254x254x167	140x140x8.8	356x171x51	305x305x198	120x120x12.5	305x165x54	305x305x198	140x140x12.5
	305x165x40	305x305x198	100x100x10	305x165x40	305x305x198	140x140x10	356x171x51	305x305x198	140x140x12.5	305x165x54	305x305x198	300x100x10
1	305x165x40	305x305x240	90x90x12.5	305x165x40	203x203x46	120x120x5	356x171x51	356x406x235	140x140x12.5	305x165x54	356x406x235	250x150x12
	7_A			7_B			7_C			7_D		
	305x165x40	203x203x46	70x70x3.2	305x165x40	203x203x100	80x80x4	406x178x60	203x203x52	120x120x4	356x171x67	203x203x52	120x120x4
6	305x165x40	203x203x71	100x100x5	305x165x40	203x203x100	90x90x8	406x178x60	203x203x113	140x140x6.3	356x171x67	203x203x113	200x120x6
	305x165x40	203x203x113	100x100x8	305x165x40	254x254x167	140x140x6.3	406x178x60	203x203x127	140x140x10	356x171x67	203x203x113	200x120x10
	305x165x40	356x406x287	100x100x10	305x165x40	305x305x198	140x140x8	406x178x60	254x254x167	140x140x12.5	356x171x67	254x254x167	250x150x10
3	305x165x40	356x406x287	100x100x12.5	305x165x40	305x305x240	140x140x10	406x178x60	305x305x240	250x150x10	356x171x67	305x305x283	250x150x12
	305x165x40	356x406x287	120x120x10	305x165x40	305x305x283	140x140x10	406x178x60	305x305x240	250x150x12	356x171x67	305x305x283	250x150x12
	305x165x40	356x406x340	120x120x12.5	305x165x40	356x406x287	140x140x12.5	406x178x60	356x406x287	250x150x12	356x171x67	356x406x287	250x150x12.5

References

- [1] A.Y. Elghazouli, Assessment of European seismic design procedures for steel framed structures, *Bull. Earthq. Eng.* 8 (2010) 65–89, <https://doi.org/10.1007/s10518-009-9125-6>.
- [2] P. Uriz, F.C. Filippou, S.A. Mahin, Model for cyclic inelastic buckling of steel braces, *J. Struct. Eng.* 134 (2008) 619–628, [https://doi.org/10.1061/\(ASCE\)0733-9445\(2008\)134:4\(619\)](https://doi.org/10.1061/(ASCE)0733-9445(2008)134:4(619)).
- [3] P.C. Hsiao, D.E. Lehman, C.W. Roeder, Improved analytical model for special concentrically braced frames, *J. Constr. Steel Res.* 73 (2012) 80–94, <https://doi.org/10.1016/j.jcsr.2012.01.010>.
- [4] FEMA, FEMA P-58-1: Seismic Performance Assessment of Buildings (Volume 1-Methodology), 2012.
- [5] FEMA, FEMA P-58-6: Guidelines for Performance-Based Seismic Design of Buildings, 2018.
- [6] C. Málaga-Chuquitaype, A.Y. Elghazouli, Inelastic displacement demands in steel structures and their relationship with earthquake frequency content parameters, *Earthq. Eng. Struct. Dyn.* 41 (2012) 831–852, <https://doi.org/10.1002/eqe.1156>.
- [7] M. Araújo, J.M. Castro, Linear analysis for seismic safety assessment of existing steel buildings: Are they trustworthy?, in: 2nd European Conference on Earthquake Engineering and Seismology, 2014.
- [8] A. Velestos, N.M. Newmark, Effect of inelastic behavior on the response of simple systems to earthquake motions, in: *Proceedings of the 2nd World Conference on Earthquake Engineering*, 1960, pp. 895–912.
- [9] CEN, Eurocode 8 EN 1998-1:2004., Design of Structures for Earthquake Resistance, Part 1-1: General Rules, Seismic Actions and Rules for Buildings, 2004, <https://doi.org/10.2514/2.2772>.
- [10] C. Málaga-Chuquitaype, A.Y. Elghazouli, Consideration of seismic demand in the design of braced frames, *Steel Construct.* 4 (2011) 65–72, <https://doi.org/10.1002/stco.201110010>.
- [11] C. Málaga-Chuquitaype, Estimation of peak displacements in steel structures through dimensional analysis and the efficiency of alternative ground-motion time and length scales, *Eng. Struct.* 101 (2015) 264–278, <https://doi.org/10.1016/j.engstruct.2015.07.019>.
- [12] T.L. Karavasilis, N. Bazeos, D.E. Beskos, Estimation of seismic drift and ductility demands in planar regular X-braced steel frames, *Earthq. Eng. Struct. Dyn.* 36 (2007) 2273–2289, <https://doi.org/10.1002/eqe.728>.
- [13] T.L. Karavasilis, N. Bazeos, D.E. Beskos, Drift and ductility estimates in regular steel MRF subjected to ordinary ground motions: a design-oriented approach, *Earthquake Spectra* 24 (2008) 431–451, <https://doi.org/10.1193/1.2923919>.
- [14] M. Kumar, P.J. Stafford, A.Y. Elghazouli, Influence of ground motion characteristics on drift demands in steel moment frames designed to Eurocode 8, *Eng. Struct.* 52 (2013) 502–517, <https://doi.org/10.1016/j.engstruct.2013.03.010>.
- [15] A.Y. Elghazouli, Seismic Design of Steel-Framed Structures to Eurocode 8, in: *The 14th World Conference on Earthquake Engineering*, October 12–17, Beijing, China, 2008.
- [16] T.L. Karavasilis, N. Makris, N. Bazeos, D.E. Beskos, Dimensional response analysis of multistory regular steel MRF subjected to pulselike earthquake ground motions, *J. Struct. Eng.* 136 (2010) 921–932, [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000193](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000193).
- [17] J. Hickey, B. Broderick, Influence of mean period of ground motion on inelastic drift demands in CBFs designed to Eurocode 8, *Eng. Struct.* 182 (2019) 172–184, <https://doi.org/10.1016/j.engstruct.2018.12.055>.
- [18] ASCE, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, 2017, <https://doi.org/10.1061/9780784414248>.
- [19] J. Wieser, G. Pekcan, A.E. Zaghi, A. Itani, M. Maragakis, Floor accelerations in yielding special moment resisting frame structures, *Earthquake Spectra* 29 (2013) 987–1002, <https://doi.org/10.1193/1.4000167>.
- [20] F.X. Flores, D. Lopez-Garcia, F.A. Charney, Assessment of floor accelerations in special steel moment frames, *J. Constr. Steel Res.* 106 (2015) 154–165, <https://doi.org/10.1016/j.jcsr.2014.12.006>.
- [21] S. Ray-Chaudhuri, T.C. Hutchinson, Effect of nonlinearity of frame buildings on peak horizontal floor acceleration, *J. Earthq. Eng.* 15 (2011) 124–142, <https://doi.org/10.1080/13632461003668046>.
- [22] R.A. Medina, R. Sankaranarayanan, K.M. Kingston, Floor response spectra for light components mounted on regular moment-resisting frame structures, *Eng. Struct.* 28 (2006) 1927–1940, <https://doi.org/10.1016/j.engstruct.2006.03.022>.
- [23] M. Surana, Y. Singh, D.H. Lang, Effect of response reduction factor on peak floor acceleration demand in mid-rise RC buildings, *J. Inst. Eng. (India)* 98 (2017) 53–65, <https://doi.org/10.1007/s40030-017-0202-6>.
- [24] S. Taghavi, E. Miranda, Probabilistic study of peak floor acceleration demands in linear structures, in: *15th World Conference on Earthquake Engineering*, 2012, pp. 1565–1572.
- [25] E. Reinoso, E. Miranda, Estimation of floor acceleration demands in high-rise buildings during earthquakes, *Struct. Des. Tall Spec. Build.* 14 (2005) 107–130, <https://doi.org/10.1002/tal.272>.
- [26] M.P. Singh, L.M. Moreschi, L.E. Suárez, E.E. Matheu, Seismic design forces. I: rigid nonstructural components, *J. Struct. Eng.* 132 (2006) 1524–1532, [https://doi.org/10.1061/\(ASCE\)0733-9445\(2006\)132:10\(1524\)](https://doi.org/10.1061/(ASCE)0733-9445(2006)132:10(1524)).
- [27] M.E. Rodriguez, J.I. Restrepo, A.J. Carr, Earthquake-induced floor horizontal accelerations in buildings, *Earthq. Eng. Struct. Dyn.* 31 (2002) 693–718, <https://doi.org/10.1002/eqe.149>.
- [28] V. Vukobratović, P. Fajfar, Code-oriented floor acceleration spectra for building structures, *Bull. Earthq. Eng.* 15 (2017) 3013–3026, <https://doi.org/10.1007/s10518-016-0076-4>.
- [29] D.P. Welch, T.J. Sullivan, Illustrating a new possibility for the estimation of floor spectra in nonlinear multi-degree of freedom systems, in: *Proceedings of the 16th World Conference on Earthquake Engineering (16WCEE)*, Santiago, Chile, 2017.
- [30] Y.-N. Huang, A. Whittaker, Simplified Analysis Procedures for Next-Generation Performance-Based Seismic Design, 2012.
- [31] FEMA, FEMA 440 - 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures and Accompanying Commentary and Maps, 2003.
- [32] C. Málaga-Chuquitaype, M.E. Psaltakis, G. Kampas, J. Wu, Dimensionless fragility analysis of seismic acceleration demands through low-order building models, *Bull. Earthq. Eng.* 17 (2019) 3815–3845, <https://doi.org/10.1007/s10518-019-00615-2>.
- [33] CEN, Eurocode 3 EN 1993-1:2005, Eurocode 3: Design of Steel Structures - Part 1-1: General Rules and Rules for Buildings, 2005.
- [34] F. McKenna, OpenSees: a framework for earthquake engineering simulation, *Comput. Sci. Eng.* 13 (2011) 58–66, <https://doi.org/10.1109/MCSE.2011.66>.
- [35] A. Neuenhofer, F.C. Filippou, Evaluation of nonlinear frame finite-element models, *J. Struct. Eng.* 123 (1997) 958–966, [https://doi.org/10.1061/\(ASCE\)0733-9445\(1997\)123:7\(958\)](https://doi.org/10.1061/(ASCE)0733-9445(1997)123:7(958)).
- [36] S. Salawdeh, J. English, J. Goggins, A.Y. Elghazouli, A. Hunt, B.M. Broderick, Shake table assessment of gusset plate connection behaviour in concentrically braced frames, *J. Constr. Steel Res.* (2017), <https://doi.org/10.1016/j.jcsr.2017.07.022>.
- [37] J. Goggins, B.M. Broderick, A.Y. Elghazouli, S. Salawdeh, A. Hunt, P. Mongabure, J. English, Shake table testing of concentrically braced steel structures with realistic connection details subjected to earthquakes, *Structures*. (2018), <https://doi.org/10.1016/j.istruc.2017.12.003>.
- [38] T. Ryan, B.M. Broderick, A. Hunt, J. Goggins, S. Salawdeh, Recommendations for numerical modelling of concentrically braced steel frames with gusset plate connections subjected to earthquake ground motion, *J. Struct. Integr. Maint.* (2017), <https://doi.org/10.1080/24705314.2017.1354154>.
- [39] L.F. Geschwindner, A practical look at frame analysis, stability and leaning columns, *Eng. J.* 39 (2002) 167–181.
- [40] B. Chiou, R. Darragh, N. Gregor, W. Silva, NGA project strong-motion database, *Earthquake Spectra* 24 (2008) 23–44, <https://doi.org/10.1193/1.2894831>.
- [41] E. Yenier, M.A. Sandikkaya, S. Akkar, Report on the fundamental features of the extended strong motion databank prepared for the SHARE project, Deliverable 4 (2010) 44.
- [42] S. Akkar, M.A. Sandikkaya, M. Şenyurt, A.A. Sisi, B. Ay, P. Traversa, J. Douglas, F. Cotton, L. Luzzi, B. Hernandez, S. Godey, Reference database for seismic ground-motion in Europe (RESORCE), *Bull. Earthq. Eng.* 12 (2014) 311–339, <https://doi.org/10.1007/s10518-013-9506-8>.
- [43] M. Kumar, J.M. Castro, P.J. Stafford, A.Y. Elghazouli, Influence of the mean period of ground motion on the inelastic dynamic response of single and multi degree of freedom systems, *Earthq. Eng. Struct. Dyn.* 40 (2011) 237–256, <https://doi.org/10.1002/eqe.1013>.
- [44] E.M. Rathje, N.A. Abrahamson, J.D. Bray, Simplified frequency content estimates of earthquake ground motions, *J. Geotech. Geoenviron.* 124 (1998) 150–158, [https://doi.org/10.1061/\(asce\)1090-0241\(1998\)124:2\(150\)](https://doi.org/10.1061/(asce)1090-0241(1998)124:2(150)).
- [45] A. Tsitos, M.A. Bravo-Haro, A.Y. Elghazouli, Influence of deterioration modelling on the seismic response of steel moment frames designed to Eurocode 8, *Earthq. Eng. Struct. Dyn.* (2018), <https://doi.org/10.1002/eqe.2954>.
- [46] C.A. Cornell, F. Jalayer, R.O. Hamburger, D.A. Foutch, Probabilistic basis for 2000 SAC federal emergency management agency steel moment frame guidelines, *J. Struct. Eng.* 128 (2002) 526–533, [https://doi.org/10.1061/\(ASCE\)0733-9445\(2002\)128:4\(526\)](https://doi.org/10.1061/(ASCE)0733-9445(2002)128:4(526)).
- [47] N. Buratti, P.J. Stafford, J.J. Bommer, Earthquake accelerogram selection and scaling procedures for estimating the distribution of drift response, *J. Struct. Eng.* 137 (2011) 345–357, [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000217](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000217).
- [48] J.W. Baker, Conditional mean spectrum: tool for ground-motion selection, *J. Struct. Eng.* 137 (2011) 322–331, [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000215](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000215).
- [49] E.M. Rathje, F. Faraj, S. Russell, J.D. Bray, Empirical relationships for frequency content parameters of earthquake ground motions, *Earthquake Spectra* 20 (2004) 119–144, <https://doi.org/10.1193/1.1643356>.
- [50] W. Du, An empirical model for the mean period (tm) of ground motions using the NGA-West2 database, *Bull. Earthq. Eng.* 15 (2017) 2673–2693, <https://doi.org/10.1007/s10518-017-0088-8>.